**GOVERNMENT OF
NEWFOUNDLAND AND LABRADOR**

FLOOD RISK MAPPING PROJECT
GOULDS AND PETTY HARBOUR AREA

Submitted to:

**Water Resources Management Division
Department of Environment and Conservation
Government of Newfoundland and Labrador**

**Submitted by:
AMEC Environment & Infrastructure,
a Division of AMEC Americas Limited
133 Crosbie Road, PO Box 13216
St. John's, Newfoundland & Labrador
Canada, A1B 4A5**

**March 2013
TA1112735**

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March 25, 2013

AMEC Project # TA1112735

To: Department of Environment and Conservation
Government of Newfoundland and Labrador
4th Floor, Confederation Building, West Block
PO Box 8700, St. John's, NL, Canada A1B 4J6

Attn: Mr. Amir Ali Khan, Ph.D, P.Eng, Manager
Hydrologic Modelling Section
Water Resources Management Division

Dear Sir:

Re: Flood Risk Mapping Project for Goulds and Petty Harbour
Final Project Report

AMEC Environment & Infrastructure, a Division of AMEC Americas Limited, is pleased to provide the final report for the above noted project.

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

Yours truly,

AMEC Environment & Infrastructure,
a Division of AMEC Americas Limited

J. Chris Innes, B.A.
Information Management Group Lead
Ottawa, Ontario

Peter Nimmrichter, M.Eng. P.Eng.
Senior Water Resources Engineer
Mississauga, Ontario

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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 Environment & Infrastructure, a Division of AMEC Americas Limited (AMEC).
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.
4. Third party information reviewed and used to develop the opinions and conclusions contained in this report is assumed to be complete and correct. This information was used in good faith and AMEC does not accept any responsibility for deficiencies, misinterpretation or incompleteness of the information contained in documents prepared by third parties.
5. The services performed and outlined in this report were based, in part, upon visual observations of the site and attendant structures. Our opinion cannot be extended to portions of the site which were unavailable for direct observation, reasonably beyond our control.
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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APPROVALS

March 25, 2013

Prepared by: Peter Nimmrichter, M.Eng., P.Eng. — Date

March 25, 2013

Reviewed by: Ron Scheckenberger, M.Eng., P.Eng., Senior Review — Date

March 25, 2013

Approved by: Chris Innes, B.A., Project Manager — Date

Rev.	Description	Prepared By	Reviewed By	Approved By	Date
001	Draft Report v001	PN	KD	CI	December 14, 2012
002	Draft Report v002	PN	KD	CI	February 28, 2013
003	Final report	PN	RS	CI	March 25, 2013

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PREFACE

AMEC Environment & Infrastructure, a division of AMEC Americas Limited was retained by the Province of Newfoundland and Labrador, Department of Environment and Conservation, Water Resources Management Division in October 2011 to develop flood risk mapping for the areas of Corner Brook and Goulds and Petty Harbour focusing on three watercourses, namely;

- Petrie's Brook in the Corner Brook Area
- Corner Brook Stream in the Corner Brook Area
- Petty Harbour River in the Goulds and Petty Harbour Area

Two reports have been generated for this project, each detailing the development of the flood risk mapping specific to the focus areas of the Province.

This report summarizes the development of flood risk mapping along Petty Harbour River in the Goulds and Petty Harbour Area.

EXECUTIVE SUMMARY

Infrastructure, whether built, human or natural, is critically important to people and economies. The purpose of infrastructure is to protect the life, health, property and social welfare of all of its beneficiaries from the weather elements, to host economic activities and to sustain aesthetic and cultural values. When infrastructure fails under extreme weather conditions and can no longer provide services to communities, the result is often a disaster. As the climate changes, it is likely that risks for infrastructure failure will increase as weather patterns shift and extreme weather conditions become more variable and regionally more intense. Since infrastructure underpins so many economic activities of societies, these impacts will be significant and will require adaptation measures. Adaptation planning enables government and industry to understand the impacts, risks and opportunities posed by a changing climate and provide a basis for preparation of strategic roadmaps towards long-term resiliency.

As global climate changes, and increases in human population, development and green energy demand continue in the coming decades, understanding and sustainable management of water resources will be critical. One potential result of the interplay of these global changes is an increase in flooding. To assist with planning in and around potential flood zones and to minimize damages associated with flooding, information on the projected spatial extent and expected frequency of floods is critical. The factors that affect flooding must also be evaluated periodically, particularly when those factors are subject to on-going change. Changes in climate and development can have significant impacts on flood risk and both have been changing at an increasing rate. The nature of these changes and their associated impacts on flood risk need to be evaluated on a periodic basis.

AMEC Environment & Infrastructure, a division of AMEC Americas Limited (AMEC) was retained by the Water Resources Management Division (WRMD) of the Province of Newfoundland and Labrador in October 2011 to develop flood risk maps for areas along the Petty Harbour River and selected tributaries. The flood risk mapping project was completed using acceptable industry techniques and currently available data. The technical guidelines developed under the Canada-Newfoundland Flood Damage Reduction Program (Hydrologic and Hydraulic Procedures for Flood Plain Delineation, Environment Canada, 1976) provided the basis for the guiding principles and approaches for all components of the study. This basis was then supplemented with additional guiding principles, by WRMD, which are reflective of current technological and data methods. These guiding principles included the following:

- Use established engineering methods, tools and software,
- Use Geographic Information Systems (GIS) tools and software,
- Incorporate land cover analysis based on optical satellite imagery,
- Incorporate LiDAR digital elevation data and orthophotography,
- Use the most up-to-date climate data, and
- Use climate change projections up to year 2100 to model potential flood risk.

This report summarizes the development of flood plain mapping defining the 1:20 year and 1:100 year annual exceedance probability (AEP) flood risk for existing land use and climate conditions (2012) and three future time frames, namely, 2020, 2050 and 2080 for study reaches along Petty Harbour River (and tributaries) in the Goulds and Petty Harbour area.

A review of the known flood events in the Goulds and Petty Harbour area identified thirty-eight (38) events which have occurred in all seasons of the year and all months of the year except June. Five (5) flood events had ice jamming identified as the primary cause. The City of St. John's commented that Hurricane Igor (September 2010) also caused extensive flooding in the area.

The Goulds and Petty Harbour study area has been previously assessed for flood risk in 1996 by the BAE-Newplan Group, on behalf of the Government of Newfoundland and Labrador.

The field program for this study included collection of high-resolution LiDAR DTM of the Petty Harbour River watershed which was completed in November and December of 2011. Further, a field survey for the Petty Harbour River was conducted which included forty-one (41) hydraulic structures (i.e. bridges, culverts, weirs, etc.).

The Petty Harbour River watershed is influenced by three (3) dams, namely Petty Harbour Dam (which is also referred to as the Forebay Dam), Bay Bulls Big Pond Dam and Cochrane Pond Dam.

The 1:20 year and 1:100 year AEP streamflows were estimated for the Petty Harbour River watershed using both statistical and deterministic methodologies. Comparative assessment of the flow estimates over the range of methodologies concluded that the deterministic model results provided a good and supportable estimate of streamflow for these watersheds. The methods used in the current study led to comparable flood flow estimates which provide confidence in the results.

It is understood that any hydrologic model is sensitive to a variety of input parameters including rainfall and Soil Conservation Service Curve Numbers (SCS-CN). These parameters were developed based upon the best available soils information from Agriculture Canada and land cover data as provided by WRMD; the latter reflecting conditions in late 2011. Further, limited statistical streamflow data is available for the watershed. As such, it is recommended that the deterministic analysis results, based on the hydrologic modeling software HEC-HMS from the US Army Corp of Engineers, be carried forward for use in the hydraulic model for base case conditions.

A hydraulic model based on the USACE program HEC-RAS was developed for reaches of the Petty Harbour River covering a linear distance of approximately 31.6 km (with 729 cross-sections). The model was developed based on field surveyed bathymetric data and LiDAR survey conducted in November and December of 2011. It should be noted that the open water flood assessment is based on summertime 1:20 year AEP and 1:100 year AEP floods.

The hydraulic model developed for this study was also used to evaluate the potential flood conditions (i.e. resultant water levels) associated with ice jamming events. The evaluation for Cochrane Pond Brook and Raymond Brook confirmed that along limited reaches of the watercourses, computed water levels associated with ice jams have the potential to generate water levels exceeding 1:100 AEP open water event levels.

Since all hydraulic model input parameters were selected based on reliable background information, it is expected that the uncertainty associated with model output is minimal. As such, it is recommended that the hydraulic model be used as the basis by which to simulate the base case (i.e. existing land use and hydraulic conditions) and climate change flood scenarios.

An evaluation of the potential impacts of climate change on flood risk was completed. Estimates of flood plains for the periods 2020, 2050 and 2080 were computed and delineated. Two sources of rainfall estimates for these future periods were determined. Dr. Joel Finnis, an Associate Professor in the Department of Geography at Memorial University provided one set of estimates (12 hour and 24 hour durations) for St. John's. AMEC, as a component of the current project, developed projected IDF relationships for the Environment Canada St. John's Airport station. It was concluded from this assessment that climate change has the potential to increase flood risk in the Goulds and Petty Harbour area.

It should be noted that there is a great deal of uncertainty with all climate models, statistical downscaling and projection of rainfall to point locations. The quantification of rainfall and, subsequently, flood plain estimates should not be interpreted as an accurate portrayal of possible future events. These estimates provide a good indication of upward and downward trends and general sense of the magnitude of the potential change but should not be considered absolute.

Key recommendations stemming from the assessments completed for this study are outlined, as follows:

1. It is recommended that the municipalities located within the study area adopt the flood lines developed by the current study for its municipal plan and development regulations.
2. It is recommended that the municipalities located within the study area and their partners make use of the up-to-date LiDAR topographic data and orthophotography which was collected for this study for relevant municipal initiatives.
3. The St. John's Airport rainfall station relative to the Petty Harbour River Watershed lies some distance away from the approximate centroid of the watershed. As such, it is recommended that a rainfall station local to the Goulds and Petty Harbour Area, that would support assessment of IDF relationships, be installed to support watershed analysis and give insight into local meteorological conditions specific to the area.
4. It is recommended that the municipalities located within the study area engage in a program to measure water levels at designated watercourse crossing structures during flood events. This will provide a database of information which could be used to support both hydrologic and hydraulic modelling in the future.

5. It is also recommended that a program focused on unregulated streamflow data collection be developed for Petty Harbour River and its associated tributaries. Additional recording stations at strategic locations (e.g., outflow from each of the unregulated tributary areas) would provide a foundation of data that would enhance the hydrologic model calibration/validation process.

6. It is recommended that that HEC-GeoHMS, HEC-HMS, HEC-GeoRAS and HEC-RAS be used in future watershed and flood studies as their use both simplifies the development of deterministic models, as well as provides for the generation of a significant warehouse of information that can be used for other ancillary purposes beyond hydrologic assessments.

7. It is recommended that special consideration be given to higher water levels (than those based on the 1:100 year AEP flow) associated with ice jam conditions. For instance, the community can opt to designate the “ice jam” flood inundated area as a special policy area which will allow the community to enact specific policies/guidelines regarding development while recognizing the local expectation (base on historical occurrence) of ice jamming.

8. It is recommended that the municipalities located within the study area consider stream and/or structure rehabilitation in the areas where water levels exceed the river banks during the 1:100 year AEP flood and spill over land. This will confine extreme flood flows to the river channel and avoid the risk of overland flooding.

9. It is recommended that meteorological conditions in the Goulds and Petty Harbour area be monitored towards determination of increasing trends in rainfall and generally extreme weather.

10. It is recommended that climate change be integrated into municipal planning in those areas where increasing flood risk is relevant such as infrastructure and emergency planning.

11. It is recommended that this study should be revisited in approximately ten (10) years, after which time additional detail may be available from rainfall and streamflow gauges in the basin.

12. It is recommended that flood studies be initiated in early spring or sooner. Starting these projects in early spring will provide the time necessary to better plan field programs that can be conducted over the summer months. This allows surveying to be conducted during low flow conditions and allows for easier and safer access during summer months. Another benefit is that it potentially allows for the collection of more model calibration data. Flow metering (when required) and water surface profiles can be conducted in the spring when river levels are typically high, and also in the late summer when river levels are low. This would help to provide a good range of model calibration and validation data.

13. It is recommended that LiDAR topographic survey and orthophoto databases continue to be used for future flood risk mapping studies as they provide an accurate means of collecting high quality topography information over large areas.

14. It is recommended, although fundamental principles remain the same, that the "1976 Hydrologic and Hydraulic Procedures for Flood Plain Delineation" be updated to reflect current technological and engineering practices in regards to flood plain delineation and development of flood plain mapping.

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1.0 INTRODUCTION

As global climate changes and increases in human population, development and green energy demand continue in the coming decades, sustainable management of water resources will be critical. One potential result of the interplay of these global influences is an increase in flooding. Floods have the potential to cause significant personal injury, damages to property and loss of life. To assist with planning in and around potential flood zones and to minimize damages associated with flooding, information on the projected spatial extent and expected frequency of floods is critical. The factors that affect flooding must also be evaluated periodically, particularly when those factors are subject to on-going change. Changes in climate and development can have significant impacts on flood risk and both have been changing at an increasing rate. The nature of these changes and their associated impacts on flood risk need to be evaluated on a periodic basis.

Over the past several decades, the Goulds and Petty Harbour area has experienced problems with flooding. Development initiatives, in combination with anticipated climate change impacts, have the potential to significantly affect flood risk in this area. These developments, as well as anticipated climate change impacts, highlight the need for a comprehensive flood risk study and associated new flood risk mapping.

Under the Canadian constitution, flood plain management is subject to the jurisdiction of the provinces, as they are primarily responsible for water resources and land use matters. The objective of the Federal government, by way of its program, 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 damages 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 has been carried out under cost shared Federal-Provincial agreements.

The Federal minimum criterion for defining a flood risk area is the 100 year flood (i.e. a flood that has one chance in one hundred of being equalled or exceeded in any given year). However, the Federal government adopts provincial criteria if they are more stringent. For example, in British Columbia the 200-year flood is used, in Saskatchewan the 500-year flood is used, and in parts of Ontario a "Regional Storm" (based on Hurricane Hazel or the Timmins Storm) or highest observed flood is used.

Newfoundland and Labrador joined the Flood Damage Reduction Program (FDRP) in 1981 signing General and Mapping Agreements and two years later a Studies Agreement. Since signing these agreements, the Province has delineated over thirty (30) areas and flood risk information maps have been produced for the benefit of Federal, Provincial and Municipal governments, private companies and the general public. These maps illustrate the area flooded under the 1:20 year and 1:100 year annual exceedence probability (AEP) floods. The 20-year flood was used to designate the floodway and the 100-year flood to designate the flood fringe. The FDRP ended in 1999 with the final study in Newfoundland and Labrador being completed in 1996.

The Department of Environment and Conservation's Water Resources Management Division (WRMD) first incorporated climate change projections into flood risk mapping in 2008/2009, when the flood risk maps for Stephenville and Cold Brook were updated. The Stephenville/Cold Brook study was the first in Canada to delineate climate change-based Regulatory flood risk mapping but only included the worst case climate change scenario.

AMEC Environment & Infrastructure, a Division of AMEC Americas Limited (AMEC) was retained by the Water Resources Management Division (WRMD) in October 2011 to develop flood risk maps for the areas of Petty Harbour River and its associated tributaries. The flood risk mapping project was completed using acceptable industry standard techniques and data currently available. The technical guidelines developed under the Canada-Newfoundland Flood Damage Reduction Program (Hydrologic and Hydraulic Procedures for Flood Plain Delineation, Environment Canada, 1976) provided the basis for the guiding principles for all components of the study. This basis was then supplemented with additional guiding principles and approaches, by WRMD, which are reflective of current technological and data methods. These guiding principles included the following:

- Use established engineering methods, tools and software,
- Use Geographic Information Systems (GIS) tools and software,
- Incorporate land cover analysis based on optical satellite imagery,
- Incorporate Light Detection and Ranging (LiDAR) digital elevation data and orthophotography,
- Use the most up-to-date climate data, and
- Use climate change projections up to year 2100 to model potential flood risk.

This report summarizes the development of flood plain mapping defining the 1:20 year and 1:100 year AEP flood risk for existing conditions (2012) and three future time frames, namely, 2020, 2050 and 2080 for study reaches along Petty Harbour River.

1.1 Study Areas

The Goulds and Petty Harbour study area, the focus of this report, includes Petty Harbour River and tributaries, including Raymond Brook, Cochrane Pond Brook, Dirty Bridge River, Doyles River, Forth Pond Brook, Forest Pond Brook and a number of unnamed watercourses. The majority of these water features lie within the Goulds area. The total watershed area is 139.2 square kilometres. Petty Harbour River discharges into the Atlantic Ocean at Petty Harbour. The most recent flood risk study completed for Goulds and Petty Harbour area was completed in 1996.

Figure 1-1 provides regional perspective and Figure 1-2 illustrates a local perspective of the study watershed.

1.2 Work Scope

The primary study tasks can be summarized as follows:

1. Conduct a thorough review of existing information for the purpose of understanding the nature of flooding for the individual watercourses and the circumstances contributing to past flood events. This aspect of the study is detailed in Section 2 of this report.
2. Co-ordinate a field program to collect data required to support preparation of the LiDAR / GIS mapping database, to establish historical flood levels and to calibrate/verify the selected mathematical models. This aspect of the study is detailed in Section 3 of this report.
3. Acquire LiDAR data and orthophotography. This aspect of the study is detailed in Section 3 of this report.
4. Carry out a land use / land cover classification using remote sensing technology. This aspect of the study is detailed in Section 4 of this report.
5. Provide climate change projections for input into hydrological models. This aspect of the study is detailed in Section 7 of this report.
6. Conduct a hydrologic investigation of the study watershed areas to determine the flows associated with the 1:20 year and 1:100 year AEP floods by comparing streamflow record analysis with flows obtained by modeling the physiographic features of the watersheds using specified precipitation/snowmelt input. This aspect of the study is detailed in Section 4 of this report. Sensitivity analysis associated with the hydrologic model is detailed in Section 6 of this report.
7. Using flows obtained from the hydrological analyses, perform a hydraulic analysis to determine water surface profiles associated with the 1:20 year and 1:100 year AEP floods. This aspect of the study is detailed in Section 5 of this report. Sensitivity analysis associated with the hydraulic model is detailed in Section 6 of this report.
8. Develop flood plain maps illustrating the flood inundation zones for the 1:20 year and 1:100 year AEP floods. This aspect of the study is detailed in Section 8 of this report.

Section 9 of this report provides conclusions and recommendations that stem from this study.

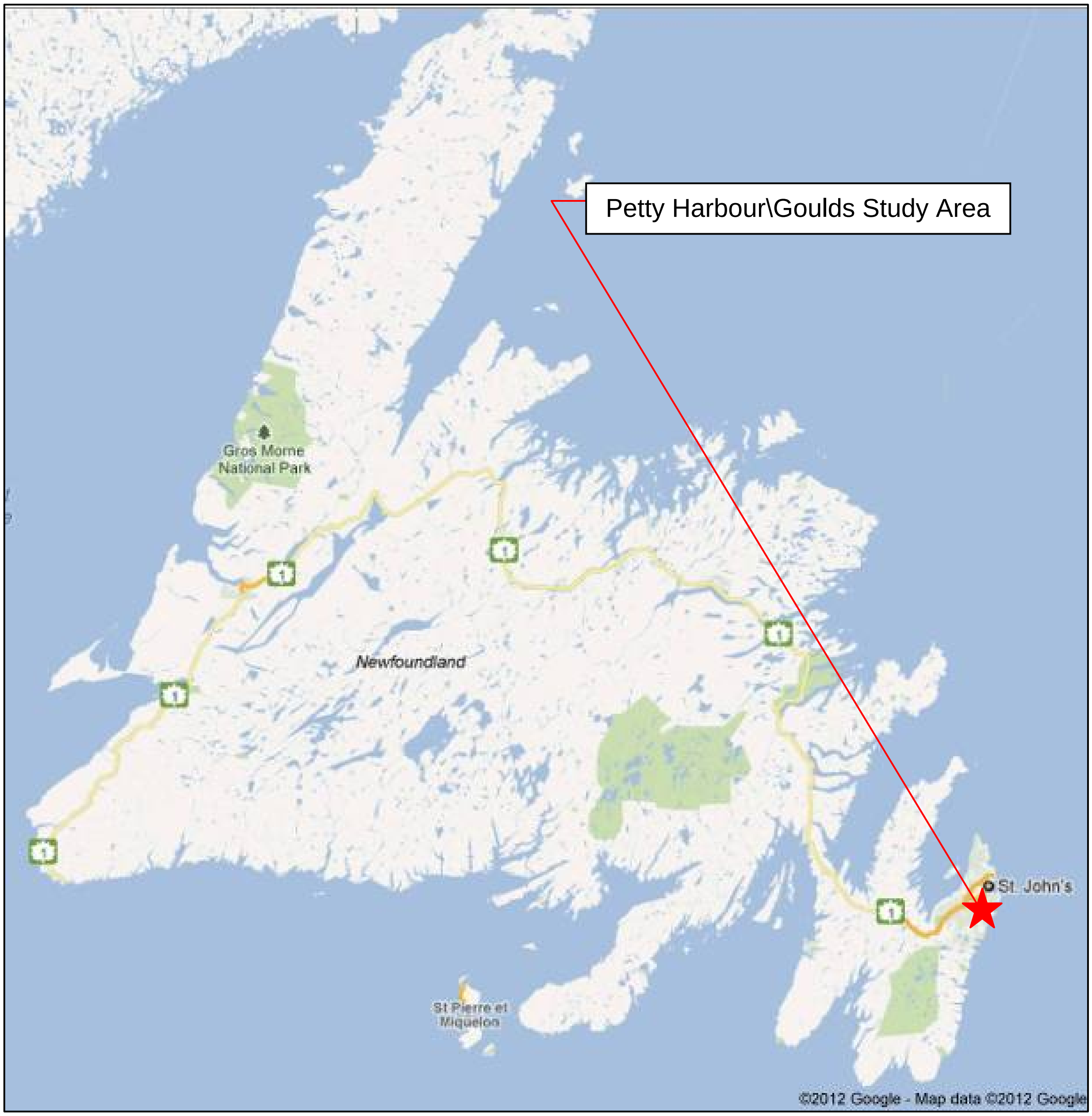

Figure 1-1: Study Area - Regional Context

Figure 1-2: Goulds and Petty Harbour Study Area - Local Context

2.0 BACKGROUND INFORMATION

A thorough review of existing information was completed to obtain an understanding of the historical flooding problem in the study areas and the factors responsible for past floods. A summary of the information sources that were reviewed is outlined in the following sections.

2.1 Historical Flooding / High Flows

As noted in the *"Flood Risk Mapping Study: Goulds, Petty Harbour and Ferryland*" (BAE-Newplan Group, 1996):

> "... flooding in the Goulds area has been frequent, and to some degree, is an annual occurrence. Although flooding is regular, its effects appear limited to periodic roadway overtopping, some basement flooding, and regular flooding of barns and parking lot at Avalon Raceway. There has also been loss of life at Goulds when a young child drowned (in April 1964) after she fell into a swollen stream."

Figure 2-1: Goulds Area Flooding – October 17, 2009[1]

A review of historical flooding and high flows was completed for this study founded upon data summarized in the Flood Events Inventory 1950-2011 (AMEC, 2012), recorded streamflows at Water Survey of Canada station below Petty Harbour Dam (#02ZM001) and events documented in the previous hydrotechnical study (BAE-Newplan Group, 1996). Additional information was also requested from the City of St. John's Engineering Department.

Information from each of these data sources is described below.

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[1] Source: http://www.thetelegram.com/Arts---Life/Environment/2009-10-17/article-1454294/Goulds,-Mount-Pearl-face-flooding/1

2.1.1 Flood Events Documented in the Flood Events Inventory

The Flood Events Inventory for the period of 1950-2011 (AMEC, 2012) was reviewed for definition of historical flooding for the Goulds and Petty Harbour area.

The Flood Events Inventory documents only six (6) flood events in the Goulds area which all occurred in the months of January, February and March. Of these flood events, three (3) identified ice jamming as the primary cause, while the remainder were rainfall on snowmelt caused flood events.

Appendix A includes a table summarizing the flood events and associated damages within the study areas as documented in the Flood Events Inventory. Of the documented flood events, only the flood of February 26, 1988 is coincident with the maximum peak flow for that year (at the Water Survey of Canada streamflow gauge below Petty Harbour Dam - #02ZM001). The flood events of 1987, 1989, 1990 and 1991 documented in the inventory occurred on days which were different than the day with the recorded peak flow for that year.

Further, the image of flooding provided in Figure 2-1 (noted from October 17, 2009) is not a documented flood event in the flood events inventory (which is an omission given the online source of this flood event information).

2.1.2 Flood Events Documented in the 1996 Hydrotechnical Study

The previous hydrotechnical reporting identified twenty-one (21) “potential” flooding events including a number which are not presently defined in the Flood Events Inventory. These “potential” events are based on historical reports of flooding in the neighbouring Waterford River and areas adjacent to St. John’s over the period 1934 to 1982. A further twelve (12) confirmed flood events are also documented in the Goulds area, as are another four (4) in Petty Harbour for 1964 and 1982 to 1994. All of the occurrences of flooding documented in the Flood Events Inventory are also documented in the 1996 hydrotechnical report.

Only the Goulds area has experienced ice jam flooding problems. Five cases of ice jam flooding are presented in the Flood Risk Mapping Study for Goulds, Petty Harbour and Ferryland (BAE-Newplan Group, 1996). A summary of the dates and descriptions of these ice jam flood occurrences is contained in Table 2-1. EC also provided photographs of several ice jam flooding events in the Goulds area which confirmed the ice jam events in February 1988 and January 1989. These records indicate that the most frequent ice jam initiation sites are the outlets of Cochrane Pond Brook and Raymond Brook at Third Ponds.

For simplicity, the historical flooding section of the 1996 report has been re-produced as a component of Appendix A of this report.

Table 2-1: Summary of Historical Ice Jam Flooding in Goulds (source: BAE-Newplan Group, 1996)

Dates	Description
Winter 1985/1986	Ice jams formed at outlet of the rivers into Third Pond following a rainstorm caused flooding of a house near Raymond Brook and caused water to flow across the Avalon Raceway.
February 26-27, 1987	Ice jam on Ryan's River (Cochrane Pond Brook) behind Avalon Raceway caused some basement flooding and ice pieces to shove against houses; the ice jam was removed using explosives.
February 26-March 2, 1988	Ice jam on Ryan's River (Cochrane Pond Brook) behind Avalon Raceway caused some basement flooding in nearby homes; the ice jam was removed using explosives.
January 9, 1989	Ice build-up on Raymond Brook caused flooding of river banks for a week and the flooding of a resident's septic tank; the ice jam was removed by the Emergency Measures Division ice demolition team.
Spring 1994	Roadway flooding and a bridge was overtopped (or nearly overtopped) as a result of snow beneath it or debris and rafting ice. Water was 2.5 ft. deep at entrance to main barn at the raceway. Raceway staff indicated that the predominant source of flooding was the southern Raymond Brook and not the more northern Ryan's Brook.

2.1.3 City of St. John's - Additional Flood Event Information

The compiled inventory of documented flood events was submitted to the City of St, John's Engineering Department for review and augmentation. It was requested, by the City, that Hurricane Igor (September 22, 2010) be added to the list as this storm was noted as having caused extensive flooding in all the rivers in the Goulds.

During Internet searches conducted for this project a video compilation of flooding associated with Hurricane Igor was found to contain a still photograph of spill occurring at Petty Harbour Dam. The image below was captured from the online video[2]. The person standing on the penstock in the lower centre of the image provides an indication of scale.

An estimation of the flow (about 60 m^3/s) over the dam, as depicted in the photograph, was determined using the stage-discharge rating curve established for the dam (see Section 2.3.2) and estimating the depth of flow over the spillway (0.9m) from the photograph.

[2] Source: http://www.youtube.com/watch?v=4fAOyOoOT7I … the image in Figure 2-1 occurs at approximately the 50 second mark in the video.

Figure 2-2: Petty Harbour Dam during Hurricane Igor

(source: http://www.youtube.com/watch?v=4fAOyOoOT7I)

2.1.4 Review of High Flows at Water Survey of Canada station 02ZM001

A summary of the peak flows recorded at the Water Survey of Canada station below Petty Harbour Dam (#02ZM001) is provided in Appendix I for the period 1963 to 2010. The information provided by Water Survey of Canada only includes maximum daily average flows and not maximum instantaneous flows. The recorded maximum daily average flows are in the range 3.34 m^3/s (January 1, 2002) to 115 m^3/s (October 24, 1983). Peak daily average flows have been recorded in all months of the year except June and July.

Of particular interest is the event of October 24, 1983 when a peak flow of 115 m^3/s was recorded below Petty Harbour Dam. The peak flow that occurred on this date was substantially greater than the next highest peak flow on record (April 12, 1986 - peak flow = 74.8 m^3/s) by 40.2 m^3/s. A review of the rainfall for the month of October 1983 (see Figure 2-3) indicates that rainfall prior to the recorded maximum flow totalled about 115 mm (over the period October 14 to 24). The streamflow gauge below Bay Bulls Big Pond began operation in 1988 so no comparative data was available.

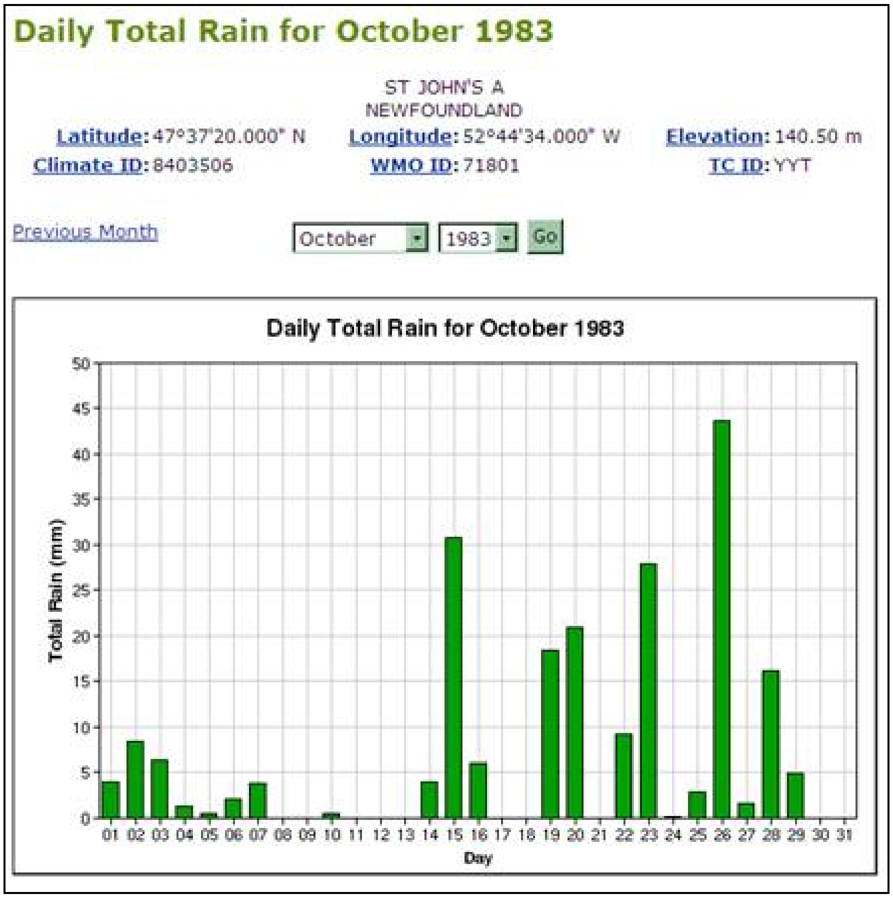

Figure 2-3: St. John's Airport Daily Rainfall - October 1983
(source: National Climate Data and Information Archive)[3]

As noted in Section 2.1.3, the recorded maximum daily flow at Petty Harbour Dam (gauge #02ZM001) was 47.6 m^3/s on September 22, 2010; during Hurricane Igor. The City of St. John's noted extensive flooding in all the rivers in the Goulds during the 2010 hurricane event. The October 24, 1983 event had a recorded streamflow more than double that experienced during Hurricane Igor, yet no flood event is documented in the Flood Events Inventory on this date in the Goulds and Petty Harbour area. It should also be noted that October 1983 is not documented as a flood event date in the 1996 study report. Also, no specific information regarding flooding in the Goulds and Petty Harbour area was found via Internet searches for the October 1983 date. Although the lack of evidence of flooding in the Goulds and Petty Harbour area for October 1983 is circumstantial, it suggests the recorded streamflow of 115 m^3/s may be in error.

2.1.5 Summary of Known Flood Events

Table 2-2 summaries the occurrence of flooding in the study area as documented in the Flood Events Inventory (AMEC, 2012), the 1996 hydrotechnical study, information provided by the City of St. John's and flood events identified via Internet searches completed for this project.

[3] Source: http://www.climat.meteo.gc.ca/climateData/canada_e.html

Table 2-2: Summary of Documented Floods in the Study Area

Month	Goulds	Petty Harbour[1]	Potential[2]	City of St. John's[3]	Internet Searches[4]	Total
January	1	1	4			6
February	3		2			5
March	4					4
April	2	1	1			4
May	1					1
June						0
July			2			2
August			1			1
September			1	1		2
October		1	4		1	6
November			2			2
December	1		4			5
Total	*12*	*3*	*21*			*38*

NOTES:
1. One documented flood was identified as having occurred in 1940 however no month was documented.
2. "Potential" events are based on a review of historical reports of flooding in the neighbouring Waterford River and areas adjacent to St. John's over the period 1934 to 1982.
3. The City of St. John's identified the rainfall associated with Hurricane Igor as resulting in extensive flooding in the Goulds and Petty Harbour area.
4. Internet searches completed for this project identified flooding on October 17, 2009.

2.2 Previous Studies

The most recent previous flood risk report, namely: *"Flood Risk Mapping Study: Goulds, Petty Harbour and Ferryland*", completed by BAE-Newplan Group in March 1996 was reviewed to support the current study. The following conclusions and findings stem from the review of the previous study:

- The hydrologic simulation program QUALHYMO was used to simulate hydrographs and peaks flows. The watershed area determined for this evaluation was 137 km^2.
- The rainfall distribution used for modelling was based on the 1988 "*Hydrotechnical Study of the Waterford River Area"*. A 12 hour storm distribution produced maximum flows in the Waterford River Basin. The same design rainfall event was used for the Goulds' assessment. Rainfall totals of 80mm and 97mm, respectively, for the 20 year and 100 year storms were determined from Intensity-Duration-Frequency (IDF) data for St. John's Airport.
- Model calibration was founded on recorded streamflows at the Water Survey of Canada station on Raymond Brook and at the outlet of Petty Harbour Dam. It was noted that regulation of flows upstream of the two streamflow gauge sites makes comparison of recorded flows with simulated flows very difficult. Further, the City of St. John's extracts large volumes of water from Bay Bulls Big Pond as a component of its water supply. It was also noted that water supply extractions were not accounted for in the modeling.

Computed flood flows from other studies were also presented in the 1996 report as outlined in Table 2-3.

Table 2-3: Goulds Area Flood Flows from Other Studies

Location	Flood Flows (m^3/s)			
	1:20 year		1:100 year	
	A	B	A	B
Cochrane Pond Brook	64	79	82	109
Doyle's River	78	62	99	90
Raymond Brook	64	86	81	113
Third Pond	194	173	248	235

Notes:
A - Delcan (1991)
B - Regional Analysis (1990)

- Petty Harbour flood flows were determined by statistical analysis of maximum daily peak flows from Water Survey of Canada records for the years 1963 to 1993 as outlined in Table 2-4.

The peak flows determined for various locations in the study area are outlined in

- Table 2-5. It should be noted that the flows for the 1996 were based on frequency analysis of QUALHYMO simulated annual peak flows (1961-1992).

Table 2-4: Maximum Daily Flows based on Frequency Analysis downstream of Petty Harbour Dam

Return Period	WSC Data (1963-1993)	1987 Reported[1] Flood Estimate (1954-1983)
1:20 year	76	68
1:100 year	101	Not reported

NOTES:
1. Newfoundland Design Associates, 1987

Table 2-5: 1996 Flood Study Computed Peak Flows

River / Location	Computed Peak Flows (m^3/s)	
	1:20 year	1:100 year
Cochrane Pond Brook	72	91
Doyle's River	68	84
Raymond Brook	61	83
Dirty Bridge River	9.6	12.6
Fourth Pond Brook	16.7	13.8
Third Pond	171	233
Petty Harbour	76	101

- A statistical analysis was also performed on the annual peak outflows from Third Pond to derive the 1:20 year and 1:100 year return period outflow rates. These were 147 m^3/s and 168 m^3/s, respectively. These flow rates were determined to yield water levels of 69.6m and 69.78m, respectively.
- It was determined that hydrologic model-based simulated flows above Petty Harbour Dam and flood frequency analysis based flows below Petty Harbour Dam, would be used for the hydraulic assessment.
- Hydraulic analyses completed for the study were based on the HEC-2 program. The study area was split into two models. The first starting at Petty Harbour and extending to the downstream side of Petty Harbour Dam. The second had Third Pond as the downstream boundary extending along Doyle's River, Cochrane Brook and Raymond River.
- Analysis of tides and storm surges at Ferryland determined the 1:20 year and 1:100 year high tide and surge levels as 1.93m and 2.17m, respectively. These water levels were used to delineate flood risk areas at the mouth of the Petty Harbour River. The storm surge and high tides were combined to estimate a worst case scenario resulting in computed water levels of 1.97m and 2.23m, respectively, for the 1:20 year and 1:100 year return periods. Wave run-up was also considered to add between 0.5m and 4.9m to the storm surge and high tide water levels.
- The starting water surface elevation for Petty Harbour was taken as the mean high tide (average of high tide level) of 0.62m as recorded at St. John's Harbour.
- In the Goulds area, notable river flooding events have almost always occurred when river ice has been a contributing factor. A few flood events were identified when ice effects were limited and flow observations made. These events, documented in Table 2-6, were used for calibration of the hydraulic model.

Two other previous studies were completed, in 1987 and 1991, for the Goulds and Petty Harbour area. The results of these studies were reviewed and integrated into the 1996 BAE-Newplan Group study and, as such, were not reviewed independently.

Table 2-6: Hydraulic Model Calibration Data

Location	Estimated Flood Elevation (m)	Estimated Discharge (m^3/s)
Cochrane Brook @ section 40	70.5 70.0	19.0 9.0
Doyle's River @ section 500	76.6	72.0

2.3 Additional Background Information

The information described below was made available to AMEC from WRMD, or with the assistance of WRMD, from a third party.

2.3.1 Information from WRMD

At the onset of the study, WRMD provided AMEC with the following information:

- SPOT satellite images covering the study areas.

  SPOT satellite imagery was provided to AMEC by WRMD with assistance from Iunctus Geomatics Corp. The SPOT images were delivered as previously ortho-rectified datasets and with a combination of clipped and/or full scenes that included 2.5-meter panchromatic, 2.5-meter fused natural color (3-band), and 10-meter resolution multispectral (4-band). Four image acquisition dates were included; one in 2009, one in 2010, and two in 2011 (refer to Appendix C for additional details).

- Topographic Mapping

  - 1:50,000 National Topographic Series Mapping (digital).

  - Community scale (1:2,500) digital topographical mapping supplied by the Surveys and Mapping Division, Department of Government Services and Lands, dated to 1984.

    This topographic map data was provided to AMEC as a series of one thousand nine hundred and seventy-one (1,971) ESRI SHP files as a combination of structured (Corner Brook area) and un-structured (Goulds and Petty Harbour area) datasets. Structured datasets are vector based and have been organized into layers and are GIS useable. Unstructured data represents digital conversion (scans) of hardcopy maps that have been vectorized but not organized into layers. The usefulness of the unstructured maps is limited to use as a backdrop image in a GIS application.

  - City of St. John's 1:500 scale digital topographic mapping.

    WRMD assisted AMEC in obtaining the City of St. John's map for flood risk map development.

  - In anticipation of the production of flood plain maps, a deliverable of this project (a street names layer) was created specific to the study reaches designated for floodplain map development. Street names were sourced from Google Earth™.

- Rainfall estimates

  <u>Historic Rainfall</u>

  Existing conditions rainfall data for St. John's was originally based on the Environment Canada rainfall station at St. John's Airport (weather station # 8403506). The currently available IDF relationship for this station (dated April 13, 2010) was updated with additional available recorded data at that station (i.e. to 2012). This effort was subsequently superseded by an IDF relationship prepared by CBCL Limited (CBCL) as a component of their flood risk mapping assignment for WRMD for the Town of Logy Bay – Middle Cove – Outer Cove.

CBCL updated the City of St. John's IDF relationship using data from Environment Canada and new data from City of St. John's rain gauges. CBCL included rainfall from hurricanes Gabrielle, Igor and Chantal in its analysis which had the effect of increasing the 100 year 24 hour precipitation IDF value to 136.8 mm (compared with 110.6 mm from the current Environment Canada published IDF relationship dated April 13, 2010.

Future Rainfall

Rainfall estimates for the future periods 2020, 2050 and 2080 were provided to AMEC by WRMD. Two sources of projected rainfall data were provided, namely;

- *Climate Change Scenarios for Atlantic Canada Utilizing a Statistical Downscaling Model Based on Two Global Climate Models*, Gary Lines, Michael Pancura, Chris Lander and Lee Titus, Meteorological Service of Canada, Atlantic Region, Science Report Series 2009-01, July 2008.

- Dr. Joel Finnis, an associate professor of Synoptic Climatology in the Department of Geography at Memorial University in St. John's, Newfoundland. The projected rainfall estimates for St. John's and Stephenville for 2050 were provided at the request of WRMD. The St. John's projected data were based on the revised St. John's IDF relationship described above.

A third approach was used by AMEC to develop projected IDF relationships for the required future periods which uses a statistical model that derives the sensitivity of extreme precipitation to climate conditions from the historical climate information for a site. This approach, which is referred to as the delta approach, is used to reduce some of the inevitable bias inherent in projections of future climate. A detailed description of the methodology and results is provided in Section 7 and Appendix D of this report.

2.3.2 Dams

Three (3) dams are located in the Petty Harbour River Watershed (see Figure 2-3) for which data was required to facilitate either hydrologic and/or hydraulic modeling, namely:

- Petty Harbour Dam (also known as the Forebay Dam) (Newfoundland Power Inc.)
- Bay Bulls Big Pond Dam (Newfoundland Power Inc./City of St. John's)
- Cochrane Pond Dam and Outlet (Newfoundland Power Inc.)

The Petty Harbour hydro-electric development power plant was placed into service in 1900 and has three generating units (G1, G2 and G3) with a combined capacity of 5.3 MW under a net head of 57.9m. Storage is provided by structures at Bay Bulls Big Pond, Cochrane Pond and the Petty Harbour Forebay which is also known as First Pond.

Information regarding dams in the study area was sourced from Newfoundland Power Inc. (Newfoundland Power) and other available reporting as could be abstracted from the available literature and topographic mapping. The information obtained from the Newfoundland Power included basic information regarding elevations of the structures and stage-storage-discharge relationships (ref. Appendix M). Internet searches identified the two reports from Newfoundland Power (2009 and 2012) which provided some photographic information specific to Cochrane Pond Dam and the general description of the Petty Harbour hydro-electric development. Photos of the dams are presented on the following pages.

Petty Harbour (Forebay) Dam

The Petty Harbour (forebay) dam is a concrete structure located at the end of First Pond and at the entrance to the gorge down to Petty Harbour. The total length of the dam is 76.2m which includes a 40.5m long well shaped ogee spillway with flip buckets. The maximum height of the structure is about 9m. The intake structure, located in the south side of the dam, is fitted with a steel gate and controls release of water from the forebay into the penstock. A dewatering sluice is located in the north side of the dam but it is permanently plugged with concrete. The dam was originally constructed in 1900, but has been upgraded and repaired since. Concrete repairs are documented in 1979 and 1981. A stability analysis of the dam resulted in the installation of bedrock anchors in 1992. Key elevations associated with the Petty Harbour Dam are provided in Table 2-7.

Table 2-7: Petty Harbour River Dam - Key Elevations

Characteristic	Elevation (m)
Dam Crest	65.02
Spillway Crest	63.55
Penstock Invert	57.46
Dam Invert	55.32

If significant rainfall or runoff is predicted in the watershed, Newfoundland Power will take measures to lower First Pond to 296ft (61.42m) prior to the rainfall event although this may not always be possible.

Target water elevations for First Pond provided by Newfoundland Power confirmed that no specific target water levels have been defined on a daily or seasonal basis for First Pond. The overall target operating range for First Pond was noted as between 61.42m and 63.55m (or the top of the overflow spillway). A summer minimum operating water level of 62.64m was also identified.

Three turbines (G1, G2 and G3) provide generation capability at the power station. Units G2 and G3 operate automatically based on water levels in First Pond. Unit G1 is not automated and is only brought on line manually to avoid spill when the reservoir is above 301.5ft. Flow rates associated with each of the turbine units is outlined in Table 2-8.

Table 2-8: Unit Loading Flows

Unit	Unit Loading Flows (m^3/s)	
	Efficient Load	Peak Load
#1	2.09	2.93
#2	2.09	2.93
#3	3.99	5.12

NOTES:

1. Unit G1 water usage data was not been provided by Newfoundland Power, However it was indicated that water usage would essentially be the same as G2 as they are similar units.

The storage potential in First Pond is limited and Newfoundland Power operates the system on a "run of the river" basis. Turbine operation at the power station turbines is based on water levels in the First Pond (ref. Table 2-9). Turbine operation directly relates to penstock flow and from June 15 to September 15 "Low Inflow" rules are followed. The system operates during the remainder of the year following "Normal Inflow" rules.

It was noted by Newfoundland Power that it cannot be assumed that operation of Unit G1 would be possible during extreme weather. In a typical storm, Newfoundland Power would bring Unit G1 online (to take advantage of higher available flows) however the plant is operated by a limited number of staff that are pooled between multiple plants. An emergency at another plant, dam, etc. could take the operator away from Petty Harbour. If the unit went offline (i.e. it was "tripped") while the operator was away it could not be put back online until the operator could physically return to the plant, the potential for which could be limited during an extreme event.

It was also noted by Newfoundland Power that during the height of extreme weather/storms it is not uncommon for the generators to trip (i.e., turn off) due to electrical system instability or communication trouble. Most often Newfoundland Power is able to get the units back on in short order, however again, this is not a definite. At Petty Harbour, Newfoundland Power only has one (1) transmission connection back to the island grid. Should Newfoundland Power lose this link they would be unable to run the system at full capacity. If the distribution system was still intact in Petty Harbour, Newfoundland Power would feed the town however this would not use the full plant capacity. If the transmission link and local distribution system are damaged such that they cannot be energized, output from the plant would be zero (0). In this case the water usage at the plant would also be zero (0), as Newfoundland Power does not run water through the plant with no load.

Newfoundland Power did review operations at Petty Harbour during Hurricane Igor in 2010 however the plant was offline for scheduled upgrades.

Table 2-9: General Water Management Elevations / Turbine Operation

Water Level (ft)	G1	G2		G3	
	High Inflow	Normal Inflow	Low Inflow	Normal Inflow	Low Inflow
Below 298	shutdown	shutdown	shutdown	shutdown	shutdown
298-299	shutdown	Efficient load Waiting to auto start	shutdown	shutdown	shutdown
299-300	shutdown	Efficient load Waiting to auto start	shutdown	shutdown	shutdown
300-301	shutdown	Efficient load	Peak load Waiting to auto start	Efficient load Waiting to auto start	shutdown
301-301.5	shutdown	Peak Load	Peak load Waiting to auto start	Peak Load	Peak Load
301.5-302	Peak Load	Peak Load	Peak load Waiting to auto start	Peak Load	Peak Load
Above 302	Peak Load	Peak Load	Peak Load	Peak Load	Peak Load

Bay Bulls Big Pond Dam

Bay Bulls Big Pond is the largest reservoir in the system and water stored here is also used as the municipal water supply for the Regional Water System, serving the City of St. John's, the City of Mount Pearl, the Town of Conception Bay South, and the Town of Paradise. Spill and controlled release from Bay Bulls Big Pond are discharged into Raymond Brook, which in turn flows into First Pond.

Bay Bulls Big Pond Dam is an earthfill dam with a rock fill overflow spillway. The dam was re-constructed in 1999. The dam crest has a length of about 120m and a crest elevation of 127.84m. The spillway has length of about 40.0m and an overflow elevation of 125.73m. A low-flow concrete outlet/sluice gate (1.8m x 1.8m with sill elevation 118.27m) provides downstream flow augmentation capability. The maximum gate opening is 0.91m (or about half of the conduit height). Key elevations associated with the Bay Bulls Big Pond Dam are provided in Table 2-10.

Table 2-10: Bay Bulls Big Pond Dam - Key Elevations

Characteristic	Elevation (m)
Dam Crest	127.84
Spillway Crest	125.73
Gate/Sluiceway Invert	118.27
Dam Invert	117.81

At present, the gate is to be left open a minimum of about 1.5 inches to maintain flow for fisheries. However, Newfoundland Power is currently working with the Department of Fishery and Oceans and the City of St. John's to review the minimum flows required for fisheries purposes from Bay Bulls Big Pond gate. At all times, Newfoundland Power operates the gate at Bay Bulls Big Pond considering this requirement. Once the head pond water level falls below the dam safety water level limit (spill minus 1.41m) Newfoundland Power augments downstream flows for fisheries considerations. Water levels are then primarily affected by the City's water supply extraction rate.

It was noted by Newfoundland Power that minimum flow determination may result in higher than normal water levels in Bay Bulls Big Pond and that care must be taken by dam operators and supervisors that dam safety and flood reduction is first and foremost.

In case of a predicted heavy inflow, Newfoundland Power will make efforts to drain the forebay as much as possible prior to the start of the rainstorm. Further, the gate at the dam should be closed to the minimum setting.

If the Bay Bulls Big Pond gate is to be opened, the opening process is to be gradual to prevent downstream flooding. For a large change, the gradual opening process would be over about 3 days with the operator verifying downstream conditions visually after the water had reached the area of concern. Approximate travel time for flow between Bay Bulls Big Pond and the First Pond is about 10 hours.

The year-round normal operating water elevation at Bay Bulls Big Pond is 1.41m below the spill crest or 124.32m. This is a requirement for dam safety purposes and can be assumed to be the worst case pre-flood condition. Typically Newfoundland Power does not extract more than fisheries flow from Bay Bulls Big Pond unless the water level is above this dam safety limit.

The overall target operating range for Bay Bulls Big Pond is between 118.41m and 125.73m (or the top of the overflow spillway).

As noted previously, Bay Bulls Big Pond is the municipal water supply for the Regional Water System. Actual demand from this reservoir was 78,200 m^3/day and 73,100 m^3/day in 2004 and 2005 respectively. This daily water taking averages to about 0.9 m^3/s. The projected 2026 and 2056 high daily demands are 81,900 m^3 (about 0.9 m^3/s) and 101,400 m^3 (about 1.2 m^3/s) respectively. (Newfoundland Design Associates, 2007)

<u>Cochrane Pond Dam and Outlet</u>

There are two structures which combine to regulate water levels on Cochrane Pond. Newfoundland Power notes the dam/spillway (re-constructed in 1996) is located on the west side of the pond and the wooden outlet structure (re-constructed in 1998) is located on the east side. Both structures act as free overflow spillways with no manual (i.e. gates, stop logs, etc.) operation/control capability. As such, Newfoundland Power does not have any control at Cochrane Pond and therefore Cochrane Pond water levels naturally regulate dependent on inflow.

Spill from Cochrane Pond via the outlet is discharged into Cochrane Pond Brook, which also flows into First Pond at Petty Harbour. Key elevations associated with the Cochrane Pond Dam are provided in Table 2-11.

Table 2-11: Cochrane Pond Structures - Key Elevations

Characteristic	Elevation (m)
Dam	
Spillway Crest	139.79
Dam Crest	141.00
Dam Invert	138.00
Outlet	
Sill Crest	139.20
Spillway Crest	140.00
Dam Crest	141.00
Dam Invert	137.60

Spill from Cochrane Pond via the dam is discharged to Paddy's Pond, part of the adjacent Topsail hydro-electric development, which lies to the west of the Trans-Canada Highway ultimately discharging to the ocean on the west side of the Avalon Peninsula.

A target normal operating water level for Cochrane Pond was not provided by Newfoundland Power. As such, for modelling purposes a normal operating water level of 139.20m or the top of the outlet sill was assumed.

Stage-Storage-Discharge Relationships

The stage-storage-discharge relationships for the three (3) dams are provided in Appendix M along with scans of the data provided by Newfoundland Power.

The stage-storage-discharge relationships provided from Newfoundland Power were reviewed as a precursor to their use for hydrologic modelling. It was noted that the reservoir surface area information was not consistent with the surface area information measured from the topographic mapping. It is presumed that the inconsistency relates to a conversion error in the Newfoundland Power documented data for the dams (ref. Appendix M). For example, a documented surface area of 0.15 km^2 was measured as 1.5 km^2. This is corrected in the stage-storage-surface area relationships provided in Appendix M.

For Petty Harbour Dam, the ogee spillway rating curve was independently determined using discharge co-efficient as documented in "Design of Small Dams" (USBR, 1987).

Stage-discharge relationships for Cochrane Pond Dam and Outlet and Bay Bulls Big Pond Dam were determined assuming a broad-crested weir configuration using a discharge co-efficient of 1.7 (SI units).

The stage-discharge relationship for the sluice gate at Bay Bull Big Pond, reflecting a downstream low-flow augmentation configuration (i.e. open 1.5 inches) was provided by Newfoundland Power.

In general, the stage-discharge relationships provided by Newfoundland Power and those generated independently for this project were in agreement.

Figure 2-4: Dam Locations in Petty Harbour River Watershed

Figure 2-5: Petty Harbour (Forebay) Dam
(source: AMEC windshield survey, November 2011)

Figure 2-6: Cochrane Pond Outlet
(source: Newfoundland Power, 2012)

Figure 2-7: Bay Bulls Big Pond Dam
(source: AMEC windshield survey, November 2011)

Figure 2-8: Cochrane Pond Dam
(source: WRMD)

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3.0 FIELD PROGRAM

The field data collection program, focused on collection of the following data, was completed in November and December 2011.

- Historical flood levels (subject to identification in the field using high water marks)
- Survey of hydraulic structures including upstream and downstream natural watercourses sections
- Survey of natural watercourse sections associated with streamflow monitoring locations
- Photographic survey of hydraulic and other relevant watercourse features
- High resolution Light Detection and Ranging (LiDAR) topographic data along with ortho-imagery of the floodplain.

The field program was planned through a desktop exercise using available mapping which identified watercourse crossings and cross-sections for below waterline survey. The overall field program, including a windshield survey, and in-stream survey of hydraulic structures and channel sections, was completed by geomatics staff from AMEC's St. John's office.

3.1 Windshield Survey

A windshield survey was completed in teams of two staff in consideration of safety issues associated with the remoteness and dangerous access conditions that some locations presented. In sheltered locations, a danger due to slips and falls was possible due to frost and ice forming overnight. This issue did not impact the project significantly; however safety for the field staff was a primary concern in the successful completion of this component of the project work.

The main objectives of the windshield survey were:

- Assessment of existing watercourse and floodplain conditions
- Identification of deviations from available mapping
- Identification of potential flood damage zones
- Initial data collection and photography of watercourse crossings
- Estimation of channel and overbank roughness coefficients
- Creation of a photo database of the subject areas designated for flood plain preparation – these photos were attached to relevant sections in the HEC-RAS model.

3.2 Cross Sections and Structure Survey

3.2.1 Cross Sections

Sections not associated with watercourse crossings were proposed to be defined using LiDAR data only. With small watercourses, such as those that are the focus of this study, the below waterline capacity is limited and is not expected to contribute significantly to conveyance for the 1:20 year and 1:100 year AEP floods. However, WRMD requested below waterline survey at a number of locations within the study areas to determine supportability of this proposed approach. The information gathered through this aspect of the field collection program was integrated into the hydraulic models of the three watercourses by adding a single cross-section X,Y point located at the centerline of the section with a depth interpolated between the nearest surveyed cross-sections when compared with the LiDAR abstracted section elevation at that point.

Figure 3-1 depicts the extent of field survey programs for the Goulds and Petty Harbour study area. Details related to these field efforts are described below. Sample photos illustrating the nature of the subject watercourses are provided in the following pages.

Fifteen (15) open water sections (see Figure 3-1) within the Goulds and Petty Harbour study area were surveyed in the field for below waterline data. The results indicated that low flow water depths along the study reaches (specific to hydraulic modeling) are in the range of about 0.1m to 0.3m (where surveyed). Plots of these sections are provided in Appendix L. The following additional comments are relevant to low flow water depths (at the surveyed locations) surveyed along specific study reaches.

- Depths along the Raymond Brook study reach are in the range 0.1m to 0.2m
- Depths along the Cochrane Pond Brook study reach are in the range 0.1m to 0.3m
- Depths along the tributary between Cochrane Pond Brook and Doyle’s River are in the range 0.1m to 0.2m
- Depths along the Doyle’s River are in the range 0.1m to 0.2m

Data from Previous Hydrotechnical Studies

Cross sections from the 1996 flood study were not verified. As such, no cross sections were reused from the 1996 flood study for the current assessment in compliance with the project terms of reference.

3.2.2 Structures

The field program related to structures is also depicted in Figure 3-1. As illustrated, each structure has been assigned a number and the associated structure summary sheets are provided in Appendix B. Structure locations identified for survey were originally identified from the 1:50,000 NTS topographic maps.

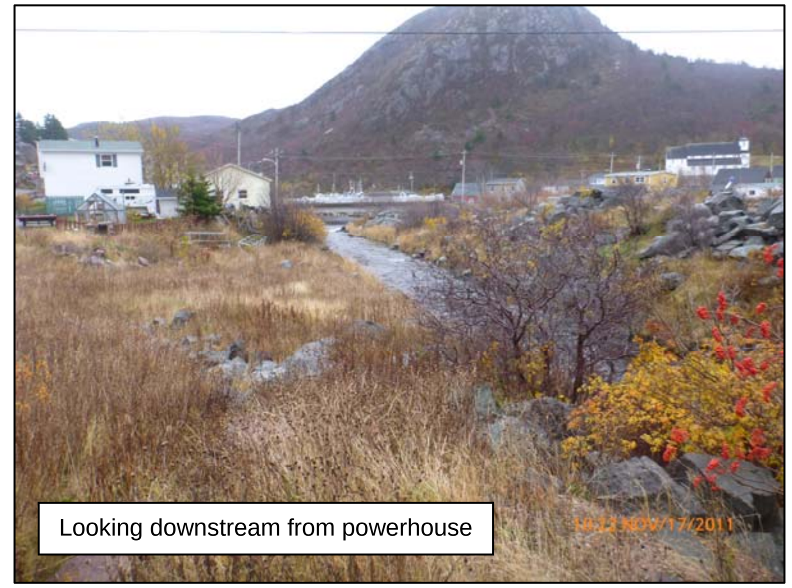
The overall field survey for the Goulds and Petty Harbour watershed area encompassed forty-two (42) structures. The structures identified for survey are listed in Table 3-1. Watercourse crossing summary sheets are available in Appendix B.

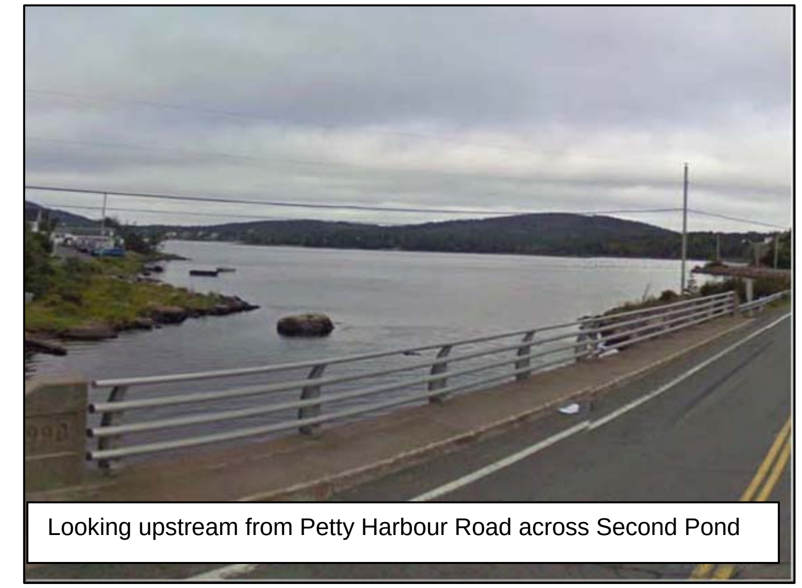
Photos Illustrating the Nature of the Subject Watercourses

Petty Harbour / Goulds Area

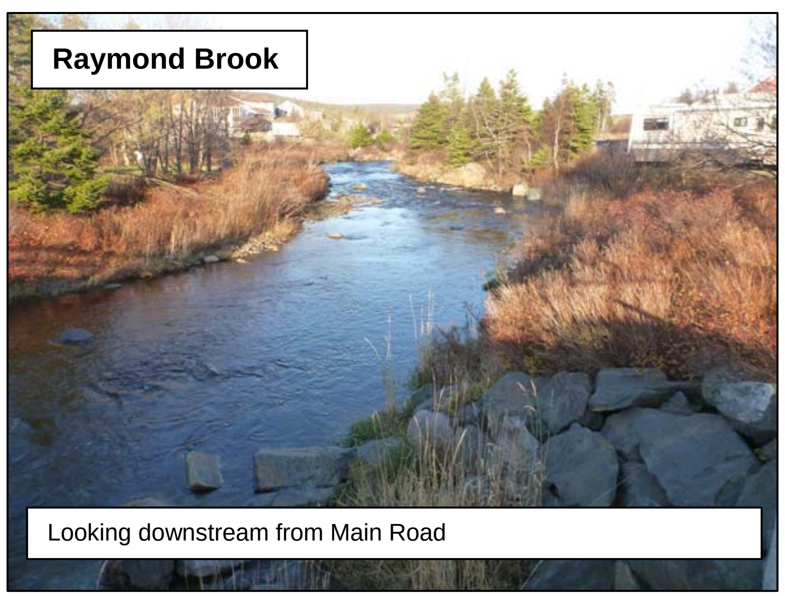
Petty Harbour River

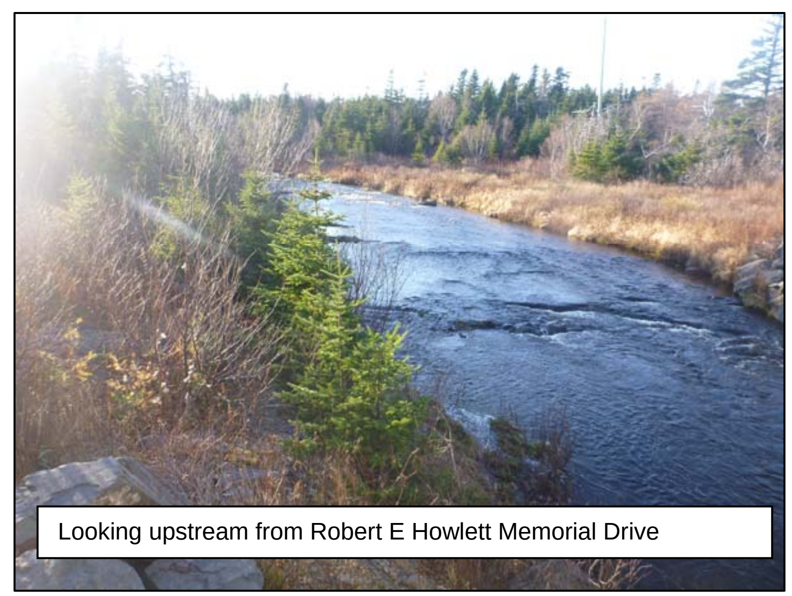
Looking towards Petty Harbour

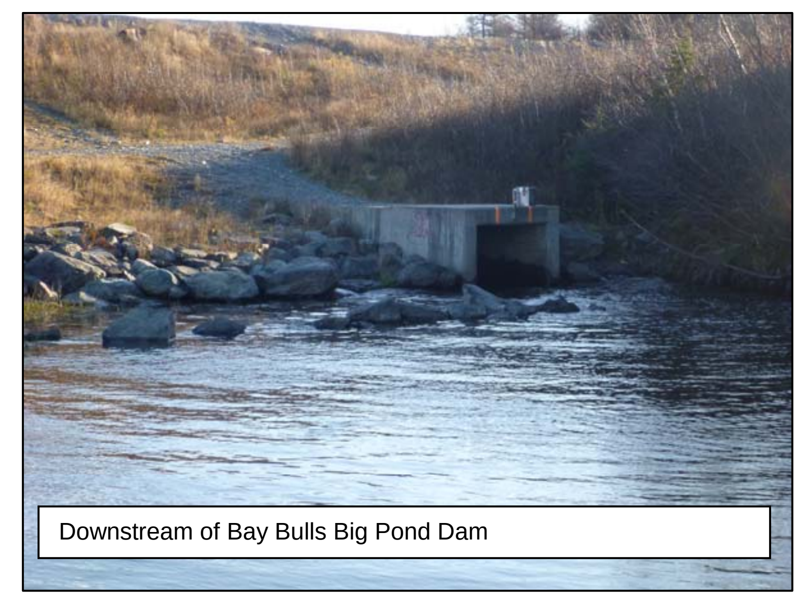

Looking downstream from powerhouse


Looking upstream from Petty Harbour Road across Second Pond

Raymond Brook

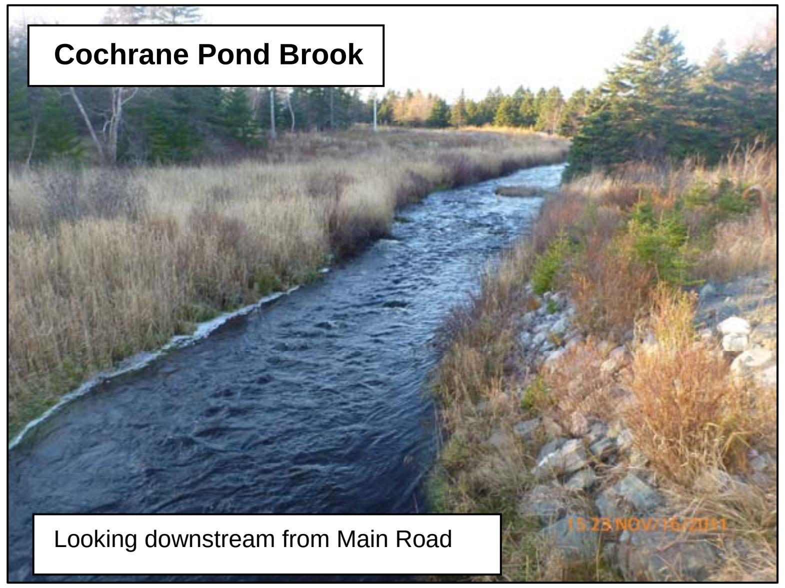

Looking downstream from Main Road


Looking upstream from Robert E Howlett Memorial Drive


Downstream of Bay Bulls Big Pond Dam

Photos Illustrating the Nature of the Subject Watercourses

Petty Harbour / Goulds Area


Cochrane Pond Brook
Looking downstream from Main Road


Looking downstream from Structure 3115 – just downstream from Robert E Howlett Drive


Looking downstream from Cochrane Pond Road


Doyles River
Looking downstream from Main Road near the intersection with Doyles Road


Upstream of Back Line at Structure #3131


Upstream of Robert E Howlett Memorial Drive

Figure 3-1: Petty Harbour River Field Survey Program

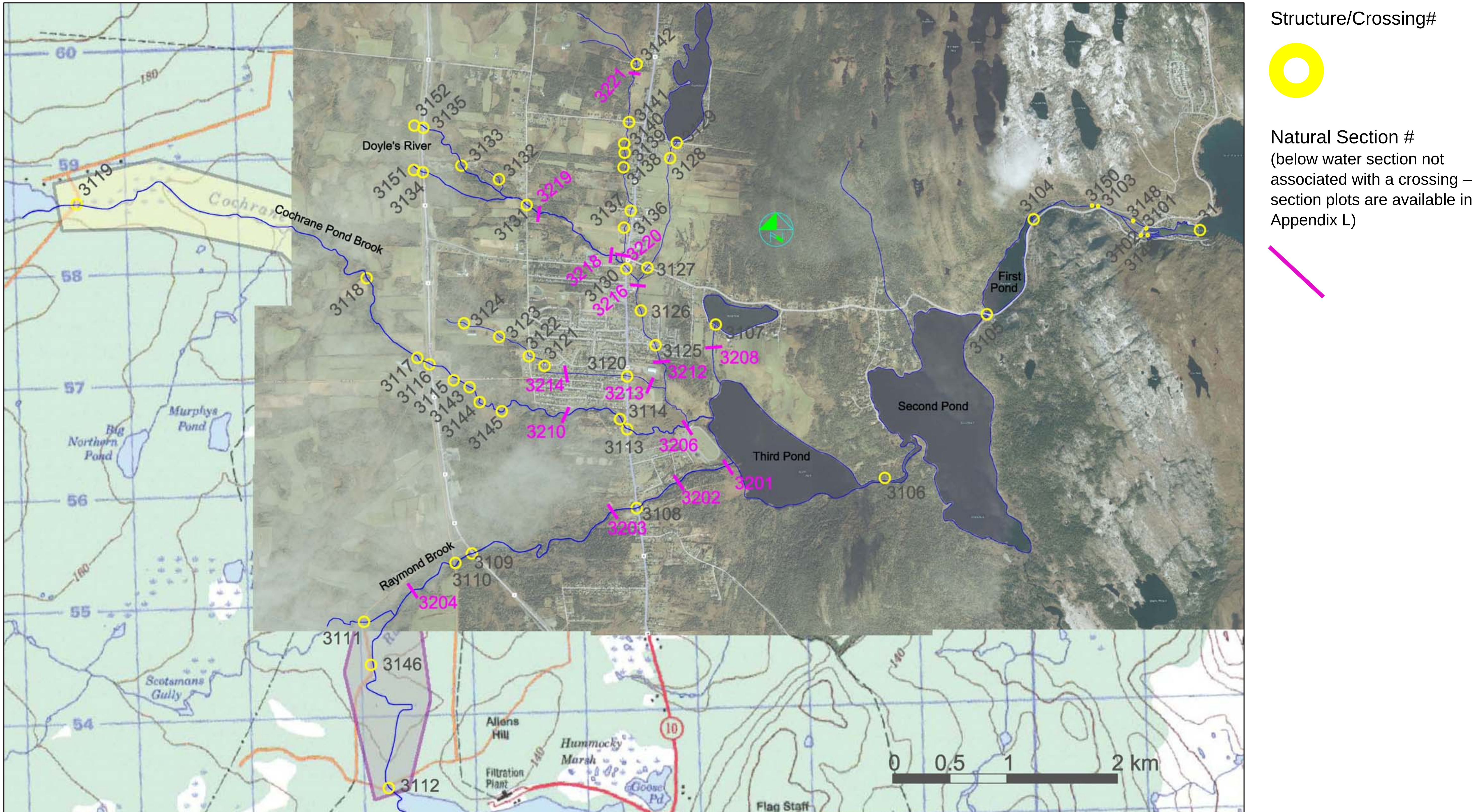


Table 3-1: Structure Survey Locations – Goulds and Petty Harbour Area

Structure#	Watercourse	Location/Description	Comments
3101	Petty Harbour River	Main Road	
3102	Petty Harbour River	Main Road	
3102a	Petty Harbour River	Main Road	
3103	Petty Harbour River	Power Plant Access Road	
3104	Petty Harbour River	Petty Harbour Dam	
3105	Petty Harbour River	Petty Harbour Road (btw 1st and 2nd Pond)	
3107	Petty Harbour River	Forest Pond Road	
3108	Raymond Brook	Main Road	
3109	Raymond Brook	Robert E Howlett Memorial Drive	
3110	Raymond Brook	Unnamed Road	
3112	Raymond Brook	Howlett's Line	
3113	Cochrane Pond Brook	Main Road	
3115	Cochrane Pond Brook	Powers Road	
3116	Cochrane Pond Brook	Robert E Howlett Memorial Drive	
3117	Cochrane Pond Brook	Unnamed Road	
3119	Cochrane Pond Brook	Cochrane's Pond Road	
3120	Dirty Bridge River	Main Road	
3121	Dirty Bridge River	Hannaford Place	
3122	Dirty Bridge River	Back Line	
3123	Dirty Bridge River	Access way	Not in the Survey
3124	Dirty Bridge River	Access way	Not in the Survey
3125	Fourth Pond Brook	Meadowbrook Drive	
3126	Fourth Pond Brook	Unnamed Road	Not in the Survey
3127	Fourth Pond Brook	Petty Harbour Road	
3129	Fourth Pond Brook	4th Pond Road	
3130	Doyles River	Doyles Road	
3131	Doyles River	Back Line	
3132	Doyles River	Trail	Not in the Survey
3133	Doyles River	Trail	Not in the Survey
3134	Doyles River	Robert E Howlett Memorial Drive	
3135	Doyles River	Robert E Howlett Memorial Drive	
3136	Goulds Stream	Driveway to commercial property	
3137	Goulds Stream	Unnamed Road	
3138	Goulds Stream	Trail	Not in the Survey
3139	Goulds Stream	Trail	Not in the Survey
3140	Goulds Stream	Trail	Not in the Survey
3141	Goulds Stream	Doolings Line	
3142	Goulds Stream	Viquers Road	

Table 3-1 (cont'd): Structure Survey Locations – Goulds and Petty Harbour Area

Structure#	Watercourse	Location/Description	Comments
3143	Cochrane Pond Brook	Walking Trail	
3144	Cochrane Pond Brook	Walking Trail	
3146	Raymond Brook	Walking Trail	
3147	Petty Harbour River	Entrance to Harbour	
3148	Petty Harbour River	Hydro plant Discharge	
3150	Petty Harbour River	Pipe crossing for hydro plant	
3151	Doyles River	Unnamed Road (west of Robert E Howlett Memorial Drive)	
3152	Doyles River	Unnamed Road (west of Robert E Howlett Memorial Drive)	
3153	Cochrane Pond Brook	Cochrane Pond Control Structure	

A number of locations were identified for field survey which were not found in the field, and as such, were not field surveyed. As noted previously, the field survey program for watercourse crossings was developed as a desktop exercise. It was found that the 1:50,000 scale mapping available at the outset of the project was not of sufficient resolution to clearly identify structures. As such, at a number of locations where it was identified on a map that a structure existed, it was determined in the field that no structure was at that location.

The late start to the project was key issue in this regard. Typically, the field survey program is developed as a component of the windshield survey effort. During the windshield survey the subject watercourse is walked and structures and notable other issues along the watercourse, that should be included in the field topographic survey effort, are identified. The late start to the project required that the windshield survey and field topographic survey were completed in parallel.

Forty-two (42) structures were originally identified from the 1:50,000 NTS topographic maps. Fourteen (14) additional structures were identified in the field and surveyed. It was determined during processing that two (2) of these structures were not located within the area for which flood plain mapping was required. Five (5) structures suggested by the 1:50,000 NTS topographic maps could not be located in the field. Survey at eight (8) structures was not completed as they are located within private property and authorized access could not be attained. Overall, forty-one (41) structures (relevant to the study) were surveyed.

Data from Previous Hydrotechnical Studies

The most recent previous flood risk report (BAE-Newplan Group, 1996) was reviewed to support the current study as detailed in Section 2.2. However, all structures to be included in the current hydraulic model were field surveyed as a component of the current study work effort. As such, no structure data was reused from the 1996 flood study for the current assessment.

3.2.3 Dams

As noted in Section 2.0 of this report, information related to dams, located within the reaches to be flood plain mapped, was obtained from the dam owner or abstracted from available literature and topographic mapping.

3.3 Bathymetry Survey of Petty Harbour

A bathymetry survey of Petty Harbour was completed using robotics technology (Clearpath Robotics).[4] Freeze up precluded use of this technology for the below waterline survey of the reservoirs (First, Second and Third Ponds) on the Petty Harbour River.

The Kingfisher M100 unmanned surface vehicle (USV), (see Figure 3-2) is designed for engineers, consultants and researchers who perform lengthy hydrological surveys in large, difficult to access or dangerous bodies of water. Developed in 2009 at the University of Waterloo, the first generation Kingfisher has been adopted for research at the University of Ottawa, University of Calgary, York University, MIT and Georgia Tech. This technology is also supported by the Water Survey of Canada.

Figure 3-2: Kingfisher M100 Unmanned Surface Vehicle

For the purposes of the this project, Clearpath mounted a Teledyne-RDI Riverray ADCP to the Kingfisher M100 USV. The ADCP is intended for collecting Depth/Bathymetric, Velocity Profile and Discharge measurements. It is rated for operability in streams, rivers, lakes, reservoirs, estuaries and irrigation canals and use cases including river hydrology, flood warning, irrigation monitoring, environmental impact studies, fishery studies and circulation studies. The Riverray ADCP is rated to deploy as float or boat mounted and specifications are as follows: 600 kHz, 1-2Hz data output rate, 10cm profile resolution, 200 cells, 0.4m minimum profiling range, 40m maximum profiling range, 16mb internal recording capability.

3.4 Water Surface Profiles

At study start-up, it was anticipated that streamflow monitoring would be undertaken by the Water Survey of Canada. Due to the late time of year start to this project and scheduling issues

[4] Freeze up precluded use of this technology waterbodies/reservoirs (First, Second and Third Ponds) on the Petty Harbour River.

at Water Survey of Canada, this aspect of the project could not be completed in advance of freeze up of watercourses in the project study areas. As such, this task of the project was not completed.

3.5 LiDAR Survey and Map Preparation

LiDAR data was collected by Leading Edge Geomatics Limited (LEGEO - Lincoln, New Brunswick) providing full coverage for the two subject watersheds. LEGEO used a Riegl LMS-680ii Airborne Scanner. This system makes use of a powerful laser source with multiple-time-around (MTA) processing and digital full waveform analysis. This combination allows for the operation at varying flight altitudes and is ideally suited for aerial survey of complex terrain. The LiDAR system was stabilized with the Applanix Position and Orientation system model 410.

The data deliverables from this effort were:

- Bare Earth DEM in both DWG and ESRI Grid format
    - Absolute Elevation precision : +/-15cm RMSEz
    - Horizontal accuracy 50cm RMSExy
    - Data collection density - 1 point per square meter
- Accuracy Report (provided in Appendix L)
- Tile Index
- Orthophotography was collected at a resolution of 15cm for the developed urban areas where floodplain mapping was to be produced.

Data collection in the Goulds and Petty Harbour study area was completed during the week of November 14, 2011.

The figures on the following pages illustrate the extent of the LiDAR and orthophoto data coverage across the subject watershed.

3.6 Data Gap Filling Related to Field Program

Data gaps were identified in the following areas:

- No as-built data was made available from municipalities for watercourse crossings. This gap did not pose any impact to the project as it was mitigated by in-field structure survey as required to support hydraulic modeling.

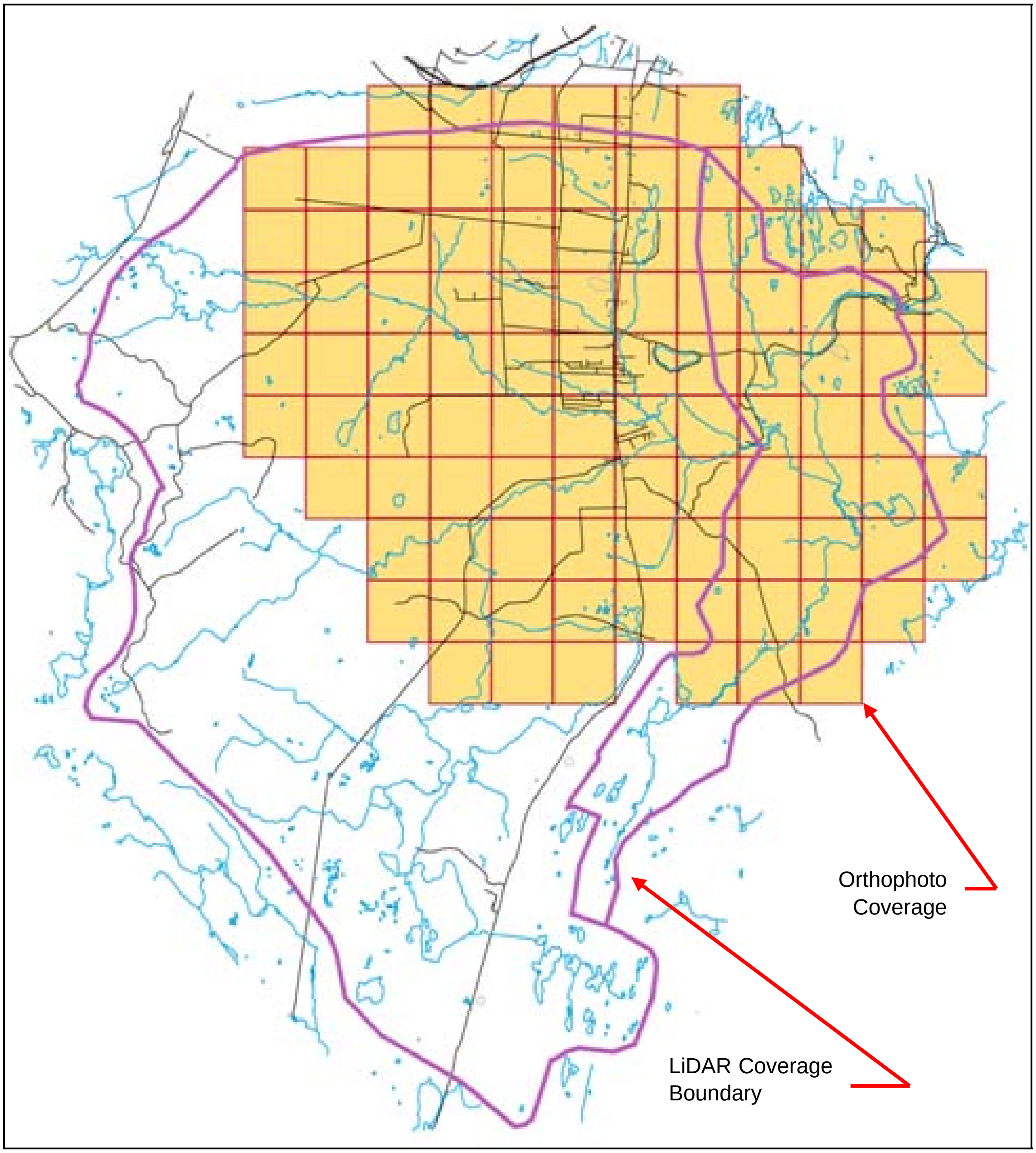


Figure 3-3: LiDAR and Orthophoto Coverage in the Goulds and Petty Harbour Area

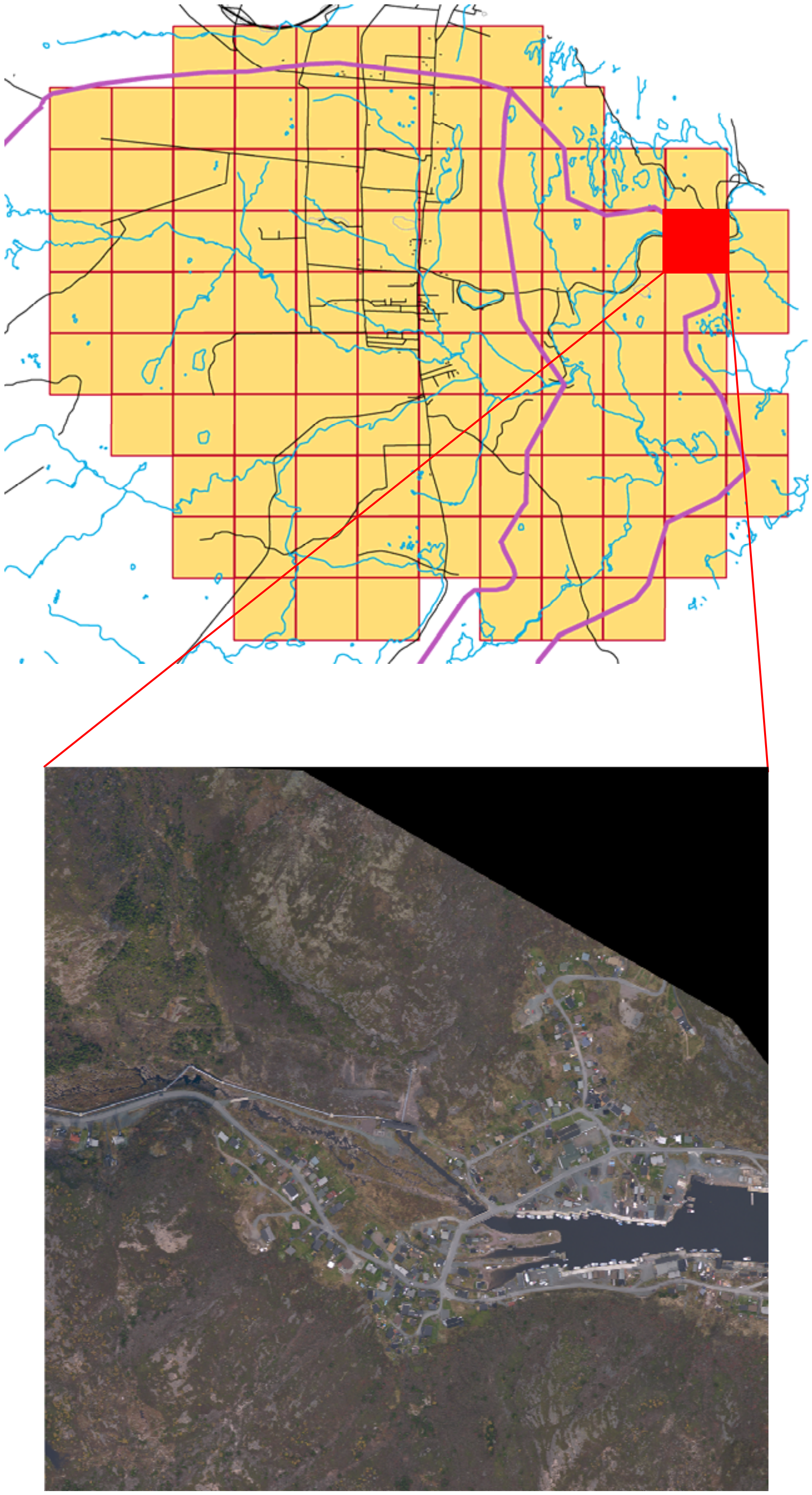


Figure 3-4: Sample Orthophoto from the Goulds and Petty Harbour Area

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4.0 HYDROLOGIC ANALYSIS

The purpose of the hydrologic analysis was to determine 1:20 year and 1:100 year annual exceedence probability (AEP) flow estimates for the Petty Harbour River Watershed. These flows were subsequently simulated in the hydraulic model to estimate flood levels across the study area.

Estimates of the 1:20 year and 1:100 year AEP flows were computed using both statistical methods and deterministic modelling. Given the uncertainty inherent in flood estimation, comparing results from alternative techniques enables an estimate to be adopted with greater confidence in its reliability and accuracy. In the case of this study, statistical estimates of flows were made by utilizing historical flow records from local hydrometric gauges. These statistical estimates were then used as the basis by which the deterministic hydrologic model HEC-HMS was calibrated. The following sections of this report detail this approach.

4.1 Statistical Analysis

4.1.1 Review of Data

The Petty Harbour River Watershed is located in tertiary drainage area 02ZM[5]. Environment Canada hydrometric gauges (ref. Table 4-1), located within the tertiary drainage area, and more specifically located within the study watershed, were evaluated for potential use in the streamflow estimation effort.

It is understood that the confidence in statistical estimates of streamflow increases with the length of the available streamflow historical record upon which to base the estimate. However, there is varied opinion as to the required length of record to support a 1:100 year AEP estimation with suggestions ranging from 18 years (WRMD, 1999), to 20 years (Alberta Transport, 2004), to 30-40 years (Watt et al, 1989; EC, 1976). The Institute of Hydrology (1999) suggests that a viable estimate of the 100 year AEP flow would require a 200 year streamflow record. The foregoing suggests a diversity of opinion on the minimum amount of data required. It can be concluded from the available references that the data records at gauges 02ZM022 - ‘Raymond Brook at outlet of Bay Bulls Big Pond’ (23 years) and 02ZM001 – ‘Petty Harbour River at Second Pond’ (49 years) are of sufficient length to use for estimation of the 1:20 year and 1:100 year AEP events.

The one limitation associated with the data from gauge 02ZM001 (at Petty Harbour Dam) is that the records provide estimates of maximum average daily flow only and not instantaneous maximum flow. Further, these flow values are measured downstream of a dam outlet and are thereby regulated flows rather than natural response of the watershed to rainfall events and therefore would not be appropriate for flow frequency analysis nor comparison with watershed response to frequency rainfall distributions.

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[5] The primary watershed (02) is the St. Lawrence River, the secondary watershed (02Z) is named ‘Southern Newfoundland’, the tertiary watershed (02ZM) is named ‘East Coast Newfoundland” (source: http://stds.statcan.gc.ca/sdac-ctad/sdacvar-ctadvar2-eng.asp?criteria=25)

WRMD also confirmed with EC that station 02ZM001 is not an EC hydrometric station. The available streamflow data was provided by Newfoundland Power Inc. and was estimated based on a power generation relationship specific to the turbines installed at the generating station. EC indicated that these data values are not calibrated and that they do not have much confidence in their accuracy. As they do not feel the data values are accurate, EC no longer published these values and, hence, the station is listed as discontinued.

The other Water Survey of Canada gauge located downstream of Bay Bulls Big Pond Dam on Raymond Brook has maximum instantaneous flow measurements for the period of 1988 to 2010 with a few gaps during this period. However, these flow values are also measured downstream of a dam outlet and are therefore not appropriate for flow frequency analysis nor comparison with watershed response to frequency rainfall distributions.

Table 4-1: Environment Canada Hydrometric Gauges in Tertiary Drainage Area 02ZM

Station Name	Station ID	Latitude Longitude	Data Type	Available Data Record	Drainage Area (km^2)
Petty Harbour River at Second Pond	02ZM001	47°27'27" N 52°43'47" W	Flow	1962 – 2010 (49 years)	134.0
Raymond Brook at outlet of Bay Bulls Big Pond	02ZM022	47°25'12"N 52°48'2" W	Flow	1988 – 2010 (23 years)	n/a[1.]

NOTES:

1. Drainage area not available from Water Survey of Canada, estimated from available mapping.

Notwithstanding the aforementioned limitations, these gauges provide the only recorded streamflows in the subject watershed. As such, these data still bear consideration in this assessment and were hence evaluated using single site frequency analysis.

The hydrologic data should satisfy certain assumptions, as follows, for the results of a statistical frequency analysis to be theoretically valid, namely;

- Randomness – variations in the flows should arise from natural causes.
- Independence – there should be no serial dependence between successive flows.
- Lack of trend – the series should display no long term trends over time, such as might be caused by changes in land use or climate.
- Homogeneity – All events should originate from a single population (i.e. represent similar hydrologic phenomena, be caused by a compatible flood-generating mechanism).

The streamflow data series was tested prior to the frequency analysis to ensure the data can be considered random and show no statistically significant serial dependence, trend or non-homogeneity. The Consolidated Frequency Analysis Package Version 3.1 (CFA_3) developed by Environment Canada was used to conduct the following screening tests;

- General Randomness Test,

- Spearman (independence) Test,
- Spearman (trend) Test, and the;
- Mann-Whitney split sample homogeneity test.

Additional details on the statistical screening analysis are provided in Appendix I. It should be noted that the data series for station 02ZM001 (Petty Harbour River at Second Pond) passed all of the screening tests, while the data series for station 02ZM022 (Raymond Brook at outlet of Bay Bulls Big Pond) did not.

4.1.2 Distribution Fitting and Quantile Analysis

The theoretical probability distributions generally considered for single site frequency analysis are the log-normal (LN) and three parameter log-normal (3PLN) distributions, and the Gumbel (EV-1) and Generalized Extreme Value (GEV). While all of these distributions have been historically recognized as possible flood frequency distributions in Newfoundland, streamflow estimates produced using these distributions typically lie within a narrow band. Further, other studies have concluded the 3PLN distribution to give the best overall fit to flood time series (WRMD, 1999; EC, 1985). Therefore, the 3PLN distribution was selected as the most appropriate statistical distribution for estimation of streamflow from the historical record for this project.

4.1.3 Regional Flood Frequency Analysis

The approach documented in the *Regional Flood Frequency Analysis for the Island of Newfoundland* (RFFA) (WRMD, 1999) was used for streamflow estimation in the Petty Harbour River watershed. The regional regression equations derived in this study are recommended for estimating return period flood flows on ungauged watersheds. However, these equations cannot be used on all watersheds as many ungauged watersheds have physiographic parameters which are outside the range of physiographic parameters which were used in the development of the regression equations.

The Goulds and Petty Harbour area is located within the South East Hydrological Region, as defined by the RFFA (see Figure 4.1 from the RFFA), therefore regression relationships developed for this region have been used to estimate 20 and 100 year flows. The minimum drainage area in the south-east region for which the regional equations are valid is 3.9 km^2 (see Table 5.1 from the RFFA).

$$Q_{20} = 2.366 \times (\text{Drainage Area})^{0.774}$$

$$Q_{100} = 3.020 \times (\text{Drainage Area})^{0.773}$$

The RFFA streamflow estimation results are provided in Table 4-8.

4.2 Deterministic Analysis

The 1:20 year and 1:100 year AEP flow estimates were simulated using a deterministic numerical model. There are several numerical models available for the analysis of the rainfall-runoff response of a watershed. The United States Army Corps of Engineers (USACE) HEC-HMS model was selected since it is a non-proprietary model which has been extensively used and tested (USACE, 2010). The numerical model includes a selection of methods to simulate watershed, channel and water control structure behaviour to predict flow, stage and timing. The advantages of a numerical model include the following:

- Synthesis and routing of flood hydrographs (quantifying basin response, flood volume and flow over time)
- Flow simulation distributed over several sub-watersheds and tributaries
- Simulation of reservoir routing
- Accounting for spatial variations in soil type and land cover, and
- Accounting for peak flow attenuation in channel and floodplain.

An advantage of this model is the HEC-GeoHMS tool which permits much of the model setup to occur within a GIS environment. This functionality was implemented for the current study and streamlined the model development process. The following sections describe the model inputs, calibration and verification of the model and the resulting flood flow estimates.

The HEC-GeoHMS model for Petty Harbour River was completed with the USACE HEC-GeoHMS v4.2.93 on ESRI ArcGIS 9.3.1 SP2.

4.2.1 Model Setup

Model Elements

The elements of the HEC-HMS model prepared for the current study were developed using the HEC-GeoHMS tool which allows one to process the watershed in ESRI ArcGIS 9.3/10.0 and develop the model for import into HEC-HMS. The parameters imported from HEC-GeoHMS include sub-basins, river reaches, and junctions. A Digital Elevation Model (DEM) raster network with a resolution of 1x1 m^2, provided from the LiDAR mapping was used for model set up and parameterization. Terrain pre-processing was applied to prepare the appropriate DEM for model set-up. HEC-GeoHMS recommended steps were followed and sub-basins were delineated. Figure 4-2a depicts the Petty Harbour River sub-basins respectively. Some of the delineated sub-basins were further discretized in HEC-GeoHMS to add a flow node at desired locations.

In the next step, several basin characteristics, including river length, river slope, basin slope, longest flow path, basin centroid, basin centroid elevation and centroidal flow path, were determined using HEC-GeoHMS.

The Muskingum-Cunge routing method was selected for simulation of routing in river reaches in the study area. The loss and transform method, selected to convert rainfall to runoff, was the

Soil Conservation Service[6] (SCS) method which requires several input parameters including Curve Number, initial abstraction and lag time for each sub-basin.

River reach routing was simulated for the study reach of Petty harbour River and associated tributaries. Channel shape, length, slope and roughness coefficients for the channel and overbanks were developed from survey cross sections along the reach in conjunction with the DEM for areas without survey.

In general, the available functionality within the HEC-GeoHMS tool facilitated all aspects of model development.

Sub-basin Inputs

SCS Curve Number (CN) is an index of basin's runoff generation potential and is a function of soil type and land use. National Resource Conservation Service (NRCS), known formerly as the US Soil Conservation Service (SCS), has tabulated Curve Numbers on the basis of soil type and land use. Four major hydrologic soil groups are defined which are briefly described as:

- Group A: Deep sand, deep loess aggregated soils
- Group B: Shallow loess, sandy loam
- Group C: Clay loams, shallow sandy loam, soils low in organic content and soils usually high in clay
- Group D: Soils that swell significantly when wet, heavy plastic clays, and certain saline soils

Soil information for the study area has been provided through Canadian Soil Information Service (CanSIS) of Agriculture Canada. The soils in the Petty Harbour River watersheds were defined using the *Soils of Avalon Peninsula*, 1981, Newfoundland Soil Survey, Report No. 3" [Agriculture Canada]. The detailed soil survey report along with a corresponding GIS soil layer have been provided from Agriculture Canada and used in the CN determination process. Hydrologic soil groups for different soil classes in the study area have been determined based on soil class descriptions in Ag Canada (1981). Petty Harbour River soil mapping, as well as SCS hydrological soil group classifications, are presented in Figure 4-2b.

From these reports (Ag Canada, 1981) and soil mapping, the following soil associations and their classified hydrologic soil group are as follows:

Series which are **Dominant** occupy over 50% of the Map Unit:

- Bauline – hydrologic soil group B
- Cochrane – hydrologic soil group C
- Colinet – hydrologic soil group C
- Patrick's Cove – hydrologic soil group BC
- Pouch Cove – hydrologic soil group BC

[6] The Soil Conservation Service is now known as the Natural Resources Conservation Service (NRCS)

- Red Cove – hydrologic soil group C
- Salmonier – hydrologic soil group C
- Torbay – hydrologic soil group C

The soil associations noted above represent the "Dominant Soil Association" which indicates the soil series which is dominant within the spatial polygon in the GIS database occupying over 50% of the polygon by area.

Land use classification was completed using remote sensing data (see Appendix C) as input and eight land use classes were identified for the subject watershed as outlined in Table 4-2. Land use class coverage for each HMS model sub-catchment is provided in Appendix I. Figure 4-2f illustrates the land cover across the Petty Harbour River basins.

Having both land cover and soil information in GIS form permitted efficient estimation of Curve Number values across the watershed for the hydrologic model. Table 4-2 presents Curve Numbers for some typical land covers and soil group based on values recommended in the current NRCS handbook for various hydrologic soil-cover complexes. Figure 4-2d illustrates the Curve Number grid for the modelled area.

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 II was used in this analysis representing average conditions.

Figure 4-2e illustrates the initial abstraction grid across the watershed. Initial abstraction is defined as losses from rainfall before runoff begins representing hydrologic elements such as infiltration, rainfall interception by vegetation, short term surface storage such as puddles, etc.

Table 4-3 summarizes the basin area, length, slope, weighted average CN and time lag for each of the model sub-basins.

Table 4-2: Curve Numbers for Typical Land Uses

	Soil Type			
Land Use	**A**	**B**	**C**	**D**
Forest	30	55	70	78
Developed	99	99	99	99
Fields/Pastures	39	61	74	80
Wetlands	46	66	78	83
Water	100	100	10	100
Barren/Soil	76	85	89	91
Open Space	49	69	79	84
Deforested	49	69	79	84

Table 4-3: Deterministic Model Basin Input Parameters
Petty Harbour River Watershed

Sub-basin	Basin Slope (%)	Initial Abstraction (mm)	CN	Lag Time (hr)	Area (Km^2)
W1390	7.1	4.3	81.6	0.46	0.59
W1140	10.4	4.7	78.8	0.49	0.81
W1090	6.3	4.8	71.6	1.59	5.63
W600	6.6	4.2	76.1	1.50	10.11
W1240	4.8	4.7	74.7	1.23	1.82
W1540	33.5	4.6	76.4	0.38	2.25
W640	5.4	4.6	73.8	1.56	2.69
W1040	7.1	4.0	81.9	0.85	1.57
W2050	13.2	3.1	82.8	0.42	1.94
W1590	15.9	4.1	76.8	0.97	10.35
W830	4.8	4.6	74.7	1.51	3.59
W840	5.3	4.8	73.8	1.48	4.27
W880	5.7	4.8	74.7	1.84	5.07
W1690	5.9	4.6	75.3	1.28	2.77
W2000	11.8	4.7	72.5	2.35	10.94
W920	4.9	4.7	73.2	1.77	4.17
W940	5.9	3.9	77.8	2.75	34.94
W1050	5.6	4.6	76.9	1.32	3.28
W1150	9.9	4.1	78.1	0.68	2.31
W1250	5.3	4.9	65.8	1.78	1.57
W1490	7.1	4.4	76.4	0.82	1.36
W1400	4.6	4.7	77.2	1.22	2.96
W1450	7.0	4.7	74.4	1.16	3.38
W1500	6.8	4.8	77.1	0.97	3.64
W1760	15.7	4.1	78.3	0.37	1.19
W1810	10.3	4.6	73.8	1.05	4.11
W1860	7.7	4.2	78.4	0.70	1.61
W2010	15.9	4.5	76.1	0.64	1.89
W2060	7.0	4.3	80.0	0.35	0.33
W2370	3.5	3.9	80.2	0.32	0.09
W2180	8.2	3.8	83.9	0.73	1.61
W2330	6.1	4.2	80.0	0.54	0.52
W2380	4.3	4.1	80.8	0.41	0.18

Reservoir Starting Water Levels

Section 2.0 provides details on each of the three dams located in the Petty Harbour River Watershed, namely Petty Harbour Dam, Bay Bulls Big Pond Dam, Cochrane Pond Dam and Outlet.

These dams operate differently during the different months of the year and it is understood that these facilities have the potential to exert influence on streamflows and flood management throughout the watershed. As such, a review of the temporal modelling basis for this project was completed. Re-stated, the single event modelling approach adopted for this project assumes the design rainfall event to occur during the warm period of the year when precipitation falls as rain and soils have completely thawed. Within this time frame, which lies approximately between late spring and early autumn (approximately May through October), the assumption of when the "single event" occurs is of importance in order that the modelling starting or boundary conditions can be defined in the context of starting water levels, gate settings and specific operational considerations for extreme weather at the dams.

As noted in Section 2.1, a review of the historical flood events in the study area was completed. Fifteen (15) documented and an additional twenty-one (21) potential flood events in the study area were identified (ref. Table 2-1). Floods are experienced throughout the year in the study area. Of the floods occurring in the winter months, ice jamming was the primary cause for five (5) documented flood events, while the remainder were rainfall on snowmelt caused flood events. During the open water season, a total of sixteen (16) flood events were identified. Of these floods, five (5) occurred in spring, three (3) in summer and six (6) in autumn.

A review of the climate normals for Petty Harbour and St. John's Airport (ref. Table 4-4) suggests a general trend to higher rainfall potential later in the year. However, from a review of the climate normals for the project area there is no clear evidence to select the summer or fall as the temporal modelling basis for this project.

Hurricanes represent a known flood producing threat to the Province. The Atlantic hurricane season begins in June and ends in late November[7], essentially representing the full temporal spectrum of the warm season rainfall period as defined above. As documented in the "Flood Risk and Vulnerability Analysis Project" completed for the Government of Newfoundland and Labrador (AMEC, 2012) the frequency of tropical storm occurrence in Newfoundland and Labrador, and the entire North Atlantic, can vary considerably from year to year and decade to decade. Tropical storm activity in Newfoundland and Labrador peaked in the 1960's and 1970's before reaching its lowest levels in the 1980's. But, activity in the past 20 years has increased considerably, especially over Eastern Newfoundland and the surrounding marine areas. 1997 was the last year where no tropical storms affected Newfoundland and Labrador. Since that time, an average of two or three storms have tracked across or near the province, including the peak year of 2006 when five (5) storms affected the region. The total number of tropical systems which have affected each region of Newfoundland and Labrador, by decade, is

[7] US National Weather Service, National Hurricane Center website at http://www.nhc.noaa.gov/

illustrated in Figure 4-1. The data indicates occurrence of tropical storms over the eastern part of Newfoundland over the months of June, July, August, September and October.

Table 4-4: Climate Normals – Petty Harbour and St. John's Airport
(source: Environment Canada[8])

Month	St. John's Airport			Petty Harbour		
	Rainfall (mm)	Snowfall (cm)	Extreme Daily Rainfall (mm)	Rainfall (mm)	Snowfall (cm)	Extreme Daily Rainfall (mm)
January	73.7	79.9	84.6	79.0	49.9	67.6
February	60.5	66.5	68.3	64.6	44.1	59.2
March	76.7	52.3	72.0	82.3	33.1	67.1
April	93.7	25.7	91.7	98.8	17.2	109.6
May	93.9	6.1	83.1	88.1	2.4	64.6
June	100.5	1.3	75.2	89.3	0.3	53.3
July	89.4	0	121.2	79.0	0	61.5
August	108.1	0	80.5	93.7	0	80.5
September	130.9	0	99.4	124.8	0	91.4
October	158.9	2.9	100.8	145.0	0.4	81.0
November	116.3	26.3	76.5	124.9	10.7	83.8
December	88.4	61.3	85.1	99.9	35.5	70.4

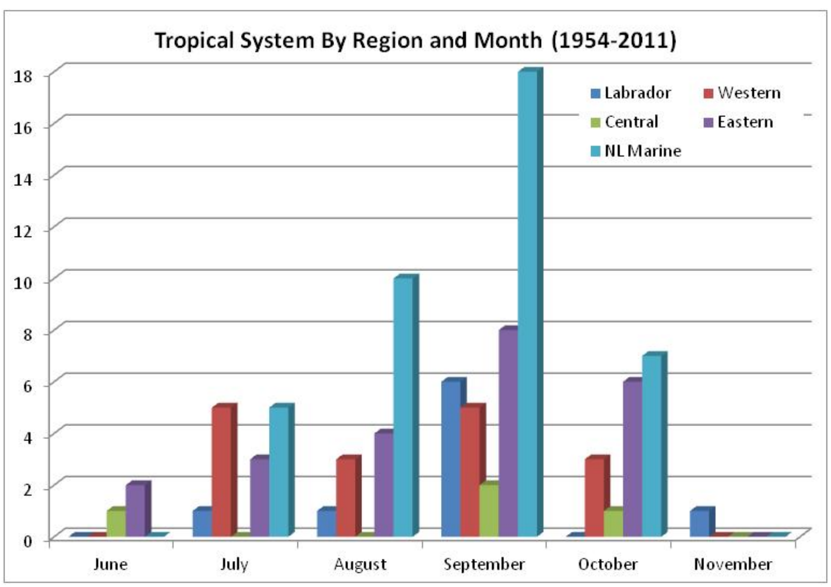

Figure 4-1: Tropical Storms by Region and Month (1954-2011)
(source: AMEC, 2012)

8 Canadian Climate Normals 1971-2000, St. John's Airport and Petty Harbour weather stations (#8403506 and #8402925, respectively)

Based on this review of hurricane occurrence near the study area, there is no clear evidence to select the summer or fall as the temporal modelling basis for this project.

The development of the temporal modelling scenarios started with a review of the flood operations at dams in the watershed. As noted in Section 2.0, Cochrane Pond Outlet (and dam) has no operational capability and is therefore not a consideration towards identification of the temporal modelling basis for this project. Similarly, Bay Bulls Big Pond Dam has a year round normal reservoir water level of 124.32m. This level was also identified by Newfoundland Power Inc. as the potentially worst case pre-flood condition. Further, normal operating practice (expected to be in effect at the outset of a potential flood event) is to have the gate opening set to the minimum required for downstream flow augmentation (or about 1.5 inches). This gate setting is also considered to be a reasonable pre-flood condition. These conditions at Bay Bulls Big Pond would be consistent throughout the year (from a modelling perspective) and as such, the assumed modelling conditions at Bay Bulls Big Pond Dam are also not a consideration towards identification of the temporal modelling basis for this project.

Petty Harbour Dam is the only dam in the system for which target water levels and turbine operations have some temporal definition as follows:

June 15 – September 15 (summer)	• *The overall target operating range between 62.64m and 63.55m* • *"Low Inflow" rules are followed for turbine operation*
September 16 – June 14 (non-summer)	• *The overall target operating range between 61.42m and 63.55m* • *"Normal Inflow" rules are followed for turbine operation*

Additionally, it was noted by Newfoundland Power Inc. that turbine operation may be "tripped" (i.e., turned off) during severe weather resulting in no flow through the penstock. This aspect was also integrated into the development of the modelling scenarios which are outlined in Table 4-5.

The results of the HEC-HMS modelling for the alternate operational scenarios are outlined in Table 4-6. Please note that this analysis was completed using the CBCL rainfall data for St. John's. A review of the results confirms that Operational Scenario #3 generates the most conservative peak flows. This scenario was therefore carried forward as the basis for modelling for determination of flood flows in the watershed.

Table 4-5: Dam Operation Scenarios

Operational Scenario #1 – summer with turbines operational

Dam	Operational Characteristics
Petty Harbour Dam	• Reservoir starting water level at 63.02m (represents average of operating water level range for June 15 to September 15) • Turbine operation following “Low Inflow” rules – integrated into rating curve
Bay Bulls Big Pond Dam	• Reservoir starting water level at 124.32m • Downstream flow augmentation - integrated into rating curve
Cochrane Pond Dam/Outlet	• Reservoir starting water level at 139.20m • No operational aspects to these structures

Operational Scenario #2 – non-summer with turbines operational

Dam	Operational Characteristics
Petty Harbour Dam	• Reservoir starting water level at 62.5m (represents average of operating water level range for September 16 to June 14) • Turbine operation following “Normal Inflow” rules – integrated into rating curve
Bay Bulls Big Pond Dam	• same as Operational Scenario #1
Cochrane Pond Dam/Outlet	• same as Operational Scenario #1

Operational Scenario #3 – summer with no turbines

Dam	Operational Characteristics
Petty Harbour Dam	• Reservoir starting water level at 63.02m (represents average of operating water level range for June 15 to September 15) • No turbine operation - integrated into rating curve
Bay Bulls Big Pond Dam	• same as Operational Scenario #1
Cochrane Pond Dam/Outlet	• same as Operational Scenario #1

Operational Scenario #4 – non-summer with no turbines

Dam	Operational Characteristics
Petty Harbour Dam	• Reservoir starting water level at 62.5m (represents average of operating water level range for September 16 to June 14) • No turbine operation - integrated into rating curve
Bay Bulls Big Pond Dam	• same as Operational Scenario #1
Cochrane Pond Dam/Outlet	• same as Operational Scenario #1

Table 4-6: Alternate Dam Operations - HEC-HMS Results

At Petty Harbour Dam	Computed Flows (m^3/s) associated with alternate Operational Scenarios			
	1 Summer with turbines	2 Non-Summer with turbines	3 Summer NO turbines	4 Non-Summer NO turbines
	1:100 Year (Existing)			
Inflow	242	242	242	*Scenario not assessed (see note #1)*
Outflow	103.8	87.3	110.5	
Maximum Head Pond Elevation	64.72	64.62	64.82	
	1:20 Year (Existing)			
Inflow	174	174	174	*Scenario not assessed (see note #1)*
Outflow	63	49.5	68.9	
Maximum Head Pond Elevation	64.32	64.22	64.52	

NOTES:

1. Please note that Operational Scenario #4 was not run as it represents a less conservative scenario when compared with Operational Scenario #3.

Using engineering judgment and maintaining the underlying theme of approaching flood plain designation in a conservative manner, it was concluded that modelling would be based on the assumption of all defined storage reservoirs being full. It must also be noted that this is only one possible condition. This modelling assumption does not preclude a change in the future in the operation or designation of the dams to include a flood control function and definition of protocols and guidelines that govern their operation during extreme flood conditions. This information, if available, can be integrated into future hydrotechnical studies.

Rainfall Inputs

Environment Canada publishes Intensity-Duration-Frequency (IDF) curves which are estimates of rainfall return period amounts in the form of design storm frequencies between 1:2 years and 1:100 years and for durations of 5 minutes to 24 hours. As noted in Section 2.3.1, CBCL updated the City of St. John's IDF relationship using data from Environment Canada and additional data from City of St. John's rain gauges. Hydrologic modelling was completed using the CBCL IDF data (ref. Table 4-7).

The 1:20 year precipitation amounts were estimated by interpolation (using the Power function trending option in Microsoft Excel™) from the 1:10 year and 1:25 year amounts. The IDF estimates for the project area are shown in Table 4-7.

The 1:20 year and 1:100 year precipitation hyetographs were estimated by using the Alternating Block method and the 5-minute to 24-hour durations for the 1:20 year and 1:100 year precipitation amounts respectively. Rainfall was input to the model in the form of a hyetograph (rainfall amount over time). Precipitation was assumed to be uniform over the watershed with no areal reduction. Figure 4-3 includes the 24 hour hyetographs for the 1:20 year and 1:100 year precipitation events.

Table 4-7: Return Period Rainfall Amounts (mm)

	Frequency			
	Environment Canada		CBCL	
Duration	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

4.2.2 Model Calibration and Validation

Model calibration and validation are required to ensure that generated peak flows from the HEC-HMS model are within an acceptable range. Unfortunately, it was not possible to conduct a conventional calibration process in this study due to insufficient measured flow, precipitation data and dam operation data for the study area on a storm by storm basis. Model calibration requires accurate measured flow data at points of interest within the watershed to be compared with corresponding computed flows from deterministic modeling.

Model calibration was completed using the CBCL rainfall data for St. John's.

The approach taken in this study to provide calibration for the HEC-HMS model has therefore been though comparison of model results with flows computed using the RFFA regression equations. As noted previously, the Goulds and Petty Harbour area is located in the RFFA South East hydrological Region in this study and therefore regression relationships developed for this region have been used to estimate 20 and 100 year flows. Unregulated sub-basins with a total drainage area larger than the RFFA minimum drainage area (3.9 km^2) were selected for comparison. HEC-HMS simulated peak flows were compared with RFFA estimates.

Initial model, results were deemed to be too high in comparison with the RFFA estimated streamflows. As such, model parameters Initial Abstraction and Lag Time were adjusted by factors of 0.85 and 1.8, respectively. With these changes, the HEC-HMS model computed streamflows were deemed to align with the RFFA estimates. The calibration results are presented in Table 4-8. Further, Figure 4-4 indicates that there is a strong correlation between simulated HEC-HMS model results and RFFA regression equation estimates for the watershed.

Table 4-8: Regional Flood Frequency Analysis Comparison with HEC-HMS Simulation

Sub-basin	Frequency Flow (yrs)	RFFA Regression Streamflow Estimation					HEC-HMS Simulation
		Drainage Area (km^2)	a1	C	log10(Q)	Q (m^3/s)	Q (m^3/s)
W840	20	4.27	0.774	2.366	0.86	7.3	9.2
	100		0.773	3.02	0.97	9.3	12.7
W880	20	5.07	0.774	2.366	0.92	8.3	9.8
	100		0.773	3.02	1.02	10.6	13.5
W1810	20	4.11	0.774	2.366	0.85	7.1	10.8
	100		0.773	3.02	0.95	9.0	15.1
W2000	20	10.94	0.774	2.366	1.18	15.1	17.1
	100		0.773	3.02	1.28	19.2	23.9
W920	20	4.17	0.774	2.366	0.85	7.1	7.9
	100		0.773	3.02	0.96	9.1	11
W830	20	3.59	0.774	2.366	0.80	6.7	7.8
	100		0.773	3.02	0.91	8.1	10.8

4.2.3 Comparison of Results with Previous Studies

A comparison of results was completed between the current study and the 1996 study of the watershed to further examine the hydrologic model validity. The comparison is based on the hydrologic modelling results from the current and previous studies as presented in Table 4-9 and Figure 4-5, respectively.

Although the contributing drainage areas are not significantly different between the two studies, the comparison has been conducted after normalizing the flows over contributing drainage area for accuracy purposes. Unitary flow comparison between the results of the two studies presented in Table 4-9 and Figure 4-4 indicate that the current study flow estimates are reasonable in comparison to previous study flow estimates with an approximately 8% difference. Over all, there is a strong correlation between the results of the two studies at the four (4) locations provided in the 1996 report. Considering the fact that these two studies have been conducted using different hydrological models (QUALHYMO vs. HEC-HMS) and design storms, the results between the two modelling efforts are considered highly comparable.

Table 4-9: Comparison of Computed Flows - 1996 and 2012 Study Results

Location	Drainage Area (km^2)		20 Year AEP				100 Year AEP			
			Peak Flow (m^3/s)		Unitary Flow ($m^3/s/km^2$)		Peak Flow (m^3/s)		Unitary Flow ($m^3/s/km^2$)	
	1996	2012	1996	2012	1996	2012	1996	2012	1996	2012
Cochrane Pond Brook	29.9	27.3	55.0	35.3	1.84	1.29	68.0	49.2	2.3	1.80
Doyles River	12.3	13.1	41.0	37.8	3.33	2.89	51.0	51.7	4.2	3.95
Raymond Brook	62.4	59.8	45.0	49.8	0.72	0.83	54.0	69.3	0.9	1.16
Third Pond	112.1	108.1	157.0	130.5	1.40	1.21	193.0	181.4	1.7	1.68

4.2.4 Comparison of Results with Measured Flows at Gauges

As previously described, there are limitations with the measured flow data for the two available gauges in the study area. Nonetheless, a comparison was completed between the 1:20 year and 1:100 year AEP flows at the two Water Survey of Canada gauges in the Petty Harbour River area and HEC-HMS computed flow results at corresponding nodes (see Table 4-10).

Table 4-10: Comparison of WSC Gauge Flows with HEC-HMS Flow Results

Station	Drainage Area (km^2)		20 Year Flow (m^3/sec)		100 Year Flow (m^3/sec)	
	WSC Station	2012	CFA*	2012	CFA*	2012
Raymond Brook at Bay Bull Big Pond	-	34.9	12.1	0.5	13.7	0.5
Petty Harbour at Second Pond	134.0	133.5	74.7	68.9	96.6	110.5

** CFA conducted using 3 Parameter Lognormal Distribution*

The current HEC-HMS modeling basis for Bay Bulls Big Pond Dam and head pond indicates that the reservoir can store the runoff from the entire 1:100 year event (based on the CBCL IDF relationship for St. John's) without overtopping of the emergency spillway when the gate is set to a 1 inch opening and a starting water level at 1.41m below the spillway crest. As a result the maximum flow from the dam is governed by the sluice gate setting, which for modelling purposes was set consistent with downstream flow augmentation requirements or about $0.5m^3/s$ (as indicated by Newfoundland Power). The inconsistency in these results prompted a more detailed review of the annual maximum instantaneous flows as available from Water Survey of Canada with rainfall data for St. John's as available from the *National Climate Data and Information Archive*[9].

[9] National Climate Data and Information Archive available at www.climate.weatheroffice.gc.ca

Table 4-11 provides a summary of the maximum instantaneous flows for streamflow station 02ZM022 (which lies below Bay Bulls Big Pond Dam) and commentary regarding total daily rainfall on and about the day of the maximum streamflow. Firstly, the majority of the occurrences of maximum instantaneous flow are in the winter and early spring months (December, January, February, March) when the ground may still be fully or partially frozen and runoff conditions may be influenced by snowmelt. This review also confirmed that the maximum instantaneous streamflows are generally measured a number of days after significant rainfall. The exceptions to this occur in 2001, 2005, 2010 when noticeable rain was measured on the day of the maximum instantaneous streamflow. However, even in these cases the rainfall is not significant in the context of the storage/flood mitigation potential of Bay Bulls Big Pond. This result is consistent with the flood operation of the dam. As outlined by Newfoundland Power flood management at the reservoir maximizes storage of the incoming flood with release of the stored flood days after the storm has passed so as not to negatively impact downstream areas. Also, the 1988 occurrence of a maximum instantaneous flow was recorded in a period with no significant rain on or before or after the date of the recorded maximum flow suggesting that this occurrence was the result of operation of the gate at the dam. Therefore, given the results of this review, comparison of the statistical estimates of the 1:20 year and 1:100 year AEP streamflows at station 02ZM022 and the hydrologic simulation results for the watershed is not considered valid. This review and comparison also reflects the limitations in using streamflows data recorded downstream of operational dams.

As noted previously, Newfoundland Power estimates streamflow at station 02ZM001 were based on a power generation relationship specific to the turbines installed at the generating station. However, EC have indicated that these data values are not calibrated and that they do not have much confidence in their accuracy. Given this, it is clear that the recorded streamflows at this station have limited value. Nonetheless, the HEC-HMS modelling results for this location are more consistent with the frequency analysis results conducted on WSC gauge located at this point.

Table 4-11: Peak Flow/Rainfall Comparison

Year	Maximum Instantaneous Flow (m3/s)	Date	Commentary regarding rainfall
1988	6.87	Jul 13	4mm total rain fell on the 13th with no significant rainfall for days before or after
1989	2.99	Dec 10	No rain on the 10th with no significant rainfall for days immediately before or after. 51.6 mm of rain on December 4th
1990	4.25	Dec 21	No rain on the 21st. 11.5mm of rain on the 19th, 30.0mm rain on the 17th, 10.8 mm rain on the 24th
1991	4.90	Nov 20	No rain on the 20th. Also no rain on the days before and after
1992	2.75	May 14	0.6mm rain on the 14th. 28.2mm of rain fell over over 9th and 10th
1993	2.34	Mar 23	No data available for the 23rd. 11.2mm of rain fell on the 22nd
1994	3.80	Apr 27	Trace rain only on the 27th. 15mm of rain fell on the 18th and 19th
1995	6.89	Dec 15	No rain on the 15th. 13.6mm of rain fell on the 11th
1996	10.10	Dec 18	4.9mm of rain fell on the 18th. 6.1mm of rain fell over the period from the 9th to the 11th.
1997	8.72	Nov 6	No rain on the 6th. About 10mm of rain fell over the 3rd, 4th and 5th. 32.4mm of rain fell on on Oct 28th
1998	----		No data
2000	8.21	Dec 28	Trace rain fell on the 28th and no significant rain on the days before or after
2001	12.70	Feb 28	15.7mm of rain fell on the 28th, 21.9mm fell on the 27th. It is also noted that this peak flow is not associated with Hurricane Gabrielle
2002	10.30	Dec 23	No rain fell on the 23rd. 8.6mm of rain fell on the 21st
2003	8.09	May 23	No rain on the 23rd, 55.8mm fell over the period May 9 - 11, and then again 15mm fell over the period May 14 - 15
2004	8.07	Nov 23	1.4mm of rain fell on the 23rd. 64.3mm fell on the 16th, 28.4mm over the period November 17-21
2005	12.30	Mar 2	30.1mm on the 2nd. 14mm of rain fell on the 26th
2006	7.20	Jan 24	No rain was measured on the 24th. 10.8mm of rain fell on the 16th, and then 16.2mm over the period January 19-22
2007	9.83	Dec 6	Minimal rain on and before the 6th. 50.8mm of rain fell on November 27th
2008	8.59	Dec 18	7mm of rain fell on the 18th. 157.6mm of rain fel over the period December 1-14.
2009	8.26	Nov 30	3.8mm of rain fell on the 30th. 28mm of rain fell in previous 4 days
2010	8.90	Dec 24	19.6mm of rain fell on the 24th. 167.5mm of rain fell over the period December 14-23.

4.3 Conclusions and Recommendations

4.3.1 Conclusions

The 1:20 year and 1:100 year AEP flood flows were estimated for the subject watershed, namely: Petty Harbour River, using both statistical and deterministic methodologies. Comparative assessment of the flow estimates over the range of methodologies concluded that the deterministic model results provided a reasonable estimate of streamflow for these watersheds.

The streamflow estimates generated through the deterministic analysis were carried forward for use in the hydraulic model.

4.3.2 Recommendations

The development of a deterministic watershed simulation model for Petty Harbour River was based on best available data, engineering judgment and parameterization founded upon field collected watershed data such as LiDAR and satellite and orthophoto imagery. The peak flows computed using the HEC-HMS model compared very well with independently determined peak flows at the Environment Canada gauge location in the watershed and using streamflow estimates based on application of the RFFA equations. It is, therefore, recommended that the streamflow estimates generated through the deterministic analysis be carried forward to the hydraulic analysis for computation of flood levels across the study areas.

The St. John's Airport rainfall station relative to the Petty Harbour River Watershed lies some distance away from the approximate centroid of the watershed. As such, it is recommended that a rainfall station local to the Goulds and Petty Harbour Area, which would support development of IDF relationships, be installed to support watershed analysis and give insight into local meteorological conditions specific to the area.

It is recommended that the City of St. John's engage in a field-based program to measure water levels at designated structures during flood events. This will provide for the development of a database of information which could be used to support both hydrologic and hydraulic modelling in the future.

It is also recommended that a program focused on unregulated streamflow data collection be developed for Petty Harbour River and its associated tributaries. Additional recording stations at strategic locations (e.g., outflow from each of the unregulated tributary areas) would provide a foundation of data that would enhance the hydrologic model calibration/validation process.

It is recommended that WRMD engage in a program to collect and develop stage-storage-discharge curves and operational data including rules curves, gates settings and reservoir water levels for all dams in the Province. Significant resources were utilized with the current project to first determine the ownership of the data (i.e. the contact person within the dam owner organization) and also to deal with delays that resulted from the time that was found to be necessary to obtain the information, once the most appropriate contact was established. If this information was already available through WRMD at the outset of the project, the development of the hydrologic model would have been more efficient.

Finally, it is recommended that that HEC-GeoHMS and HEC-HMS be used in future watershed and flood studies as these tools both simplify the development of deterministic models as well as provide for the generation of a significant warehouse of information that can be used for several of ancillary purposes, beyond hydrologic assessment.

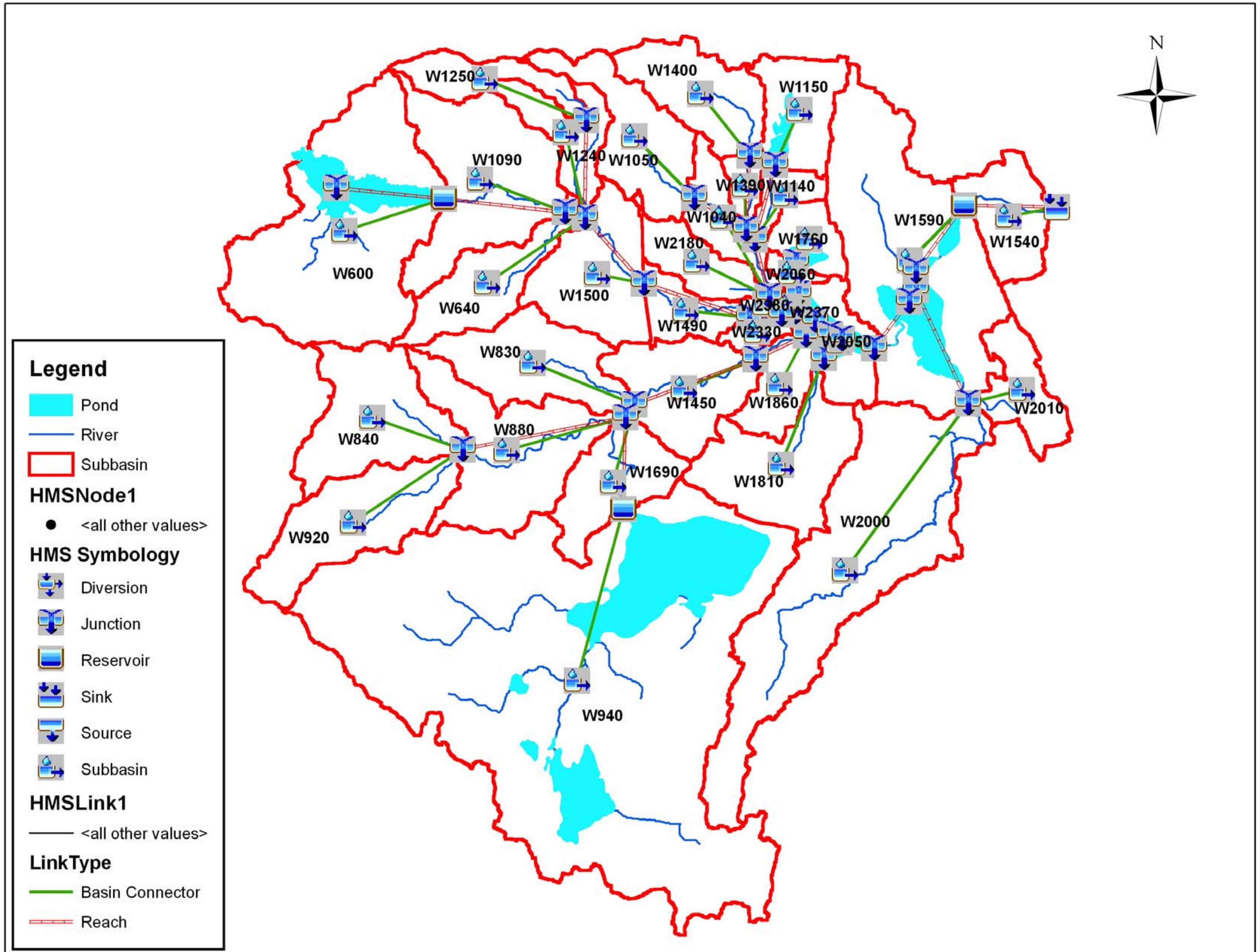

Figure 4-2a: Petty Harbour River HMS Model Schematic

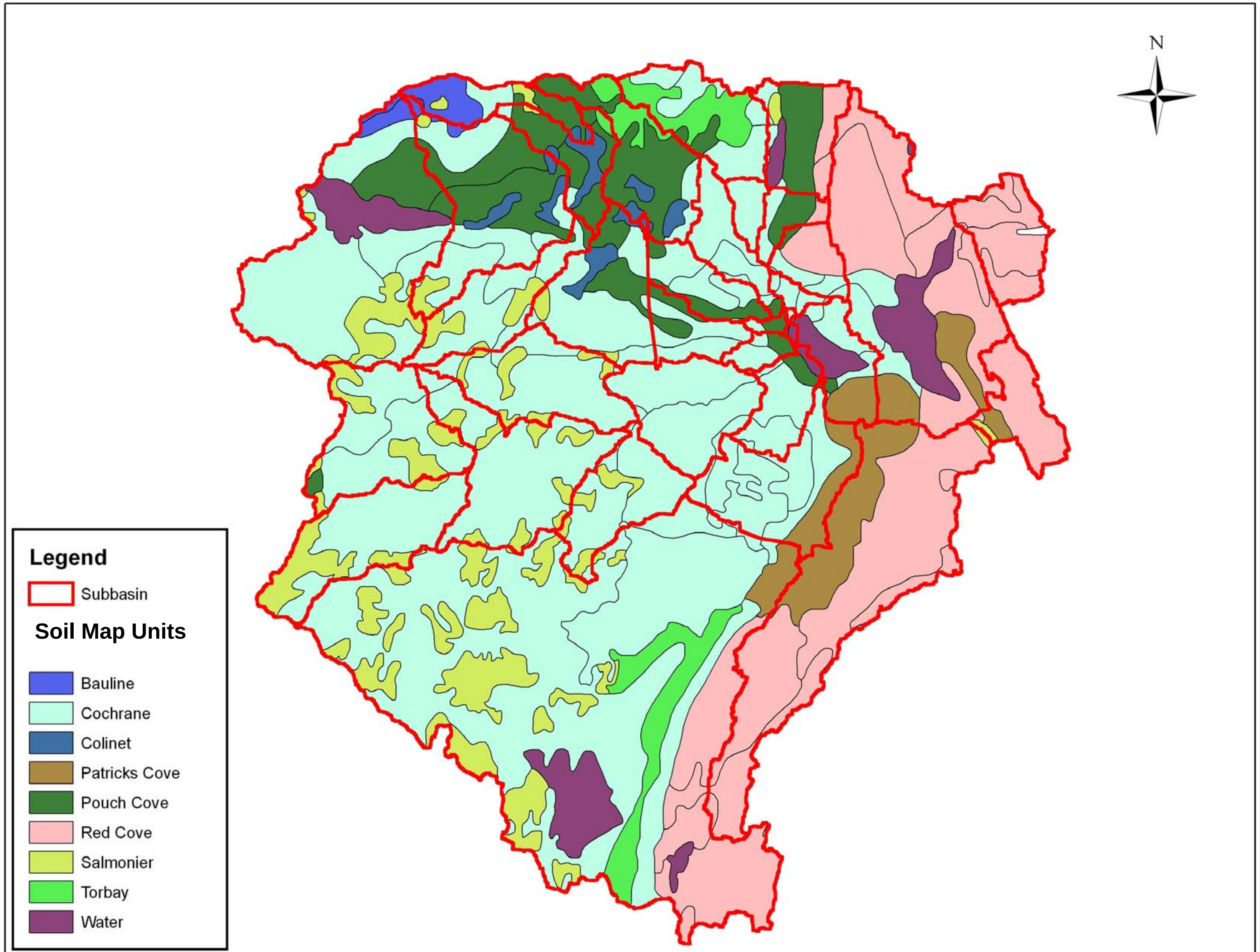

Figure 4-2b: Petty Harbour River Watershed Soils

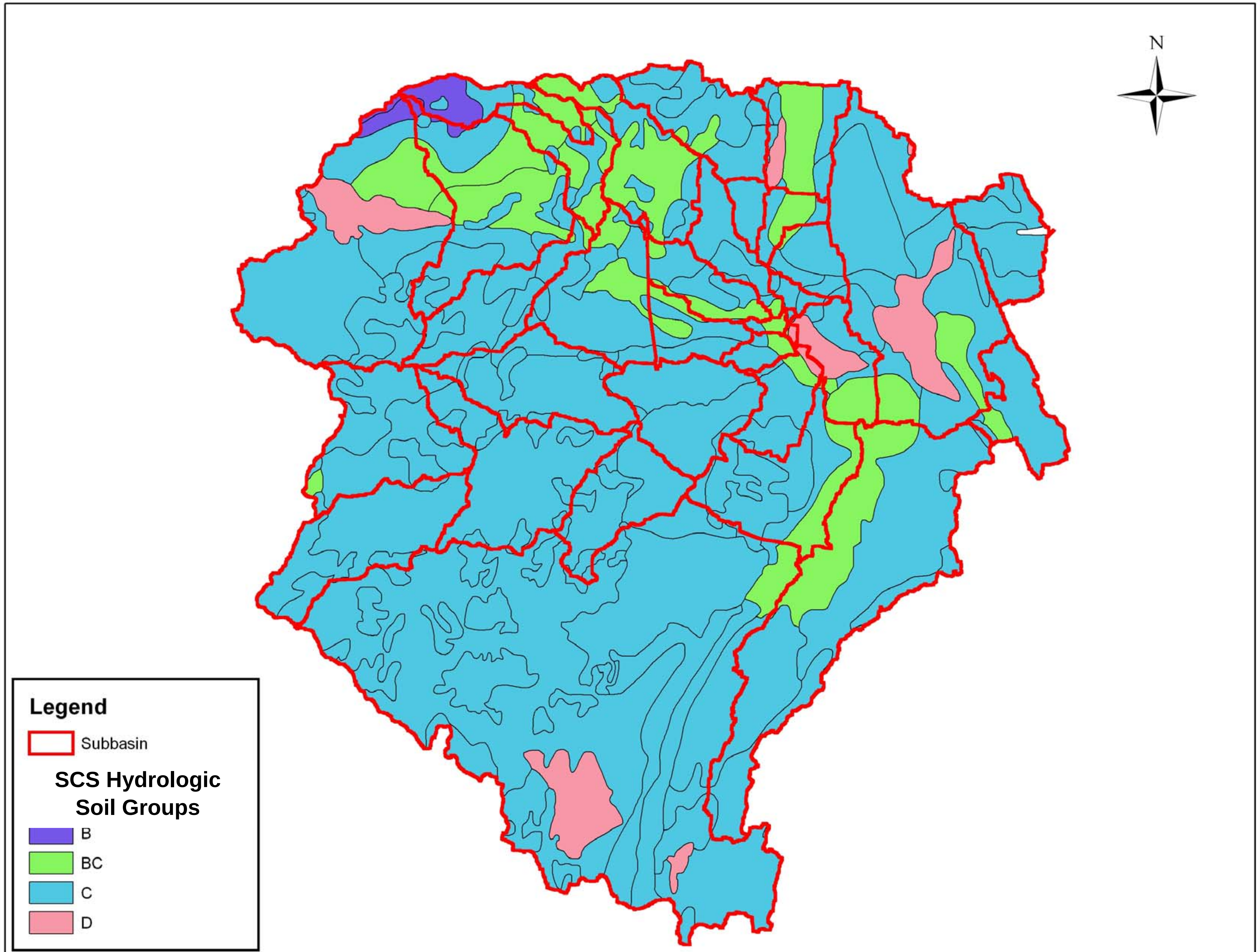

Figure 4-2c: Petty Harbour River Watershed – Hydrologic Soil Groups

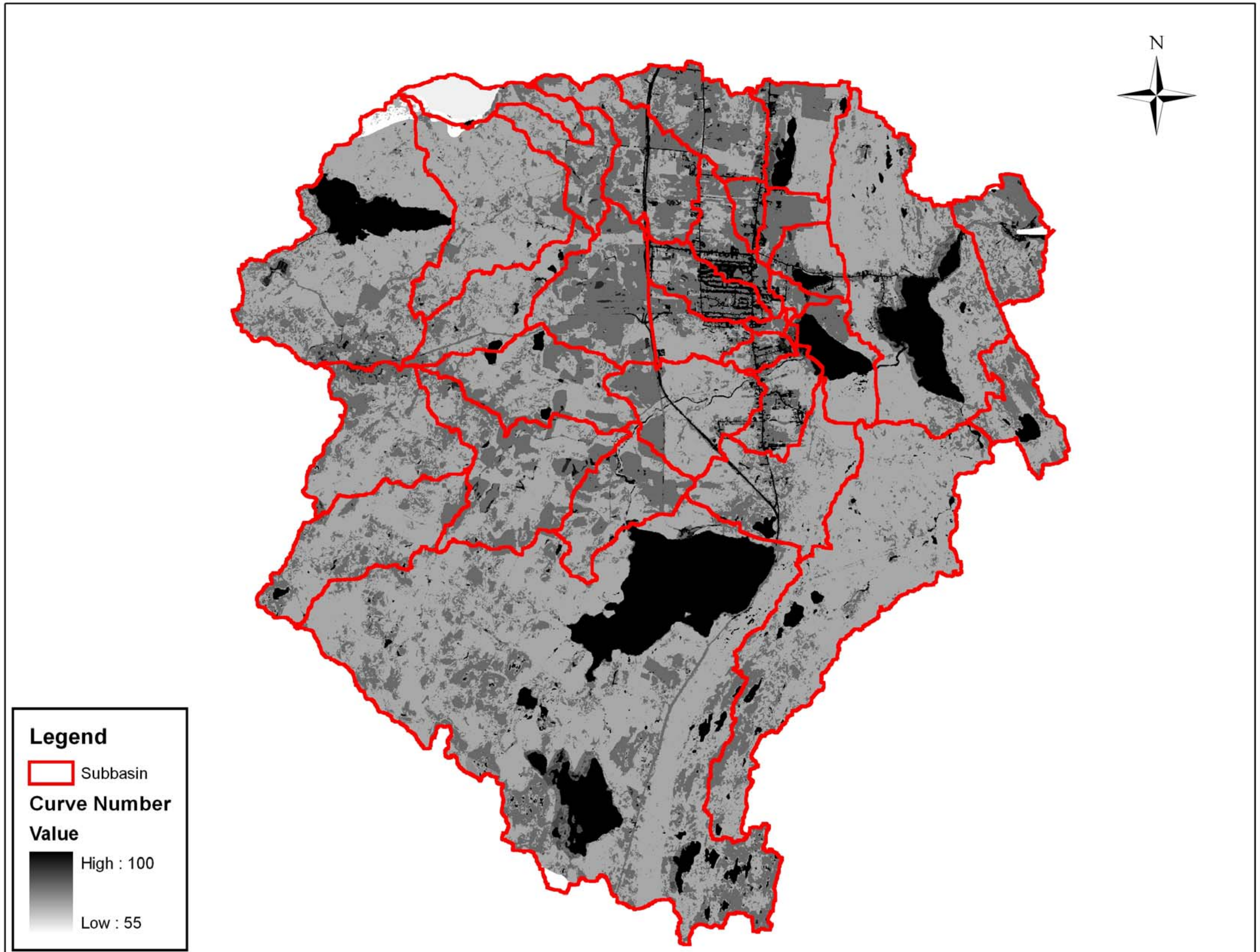

Figure 4-2d: Petty Harbour River Watershed – SCS Curve Number Grid

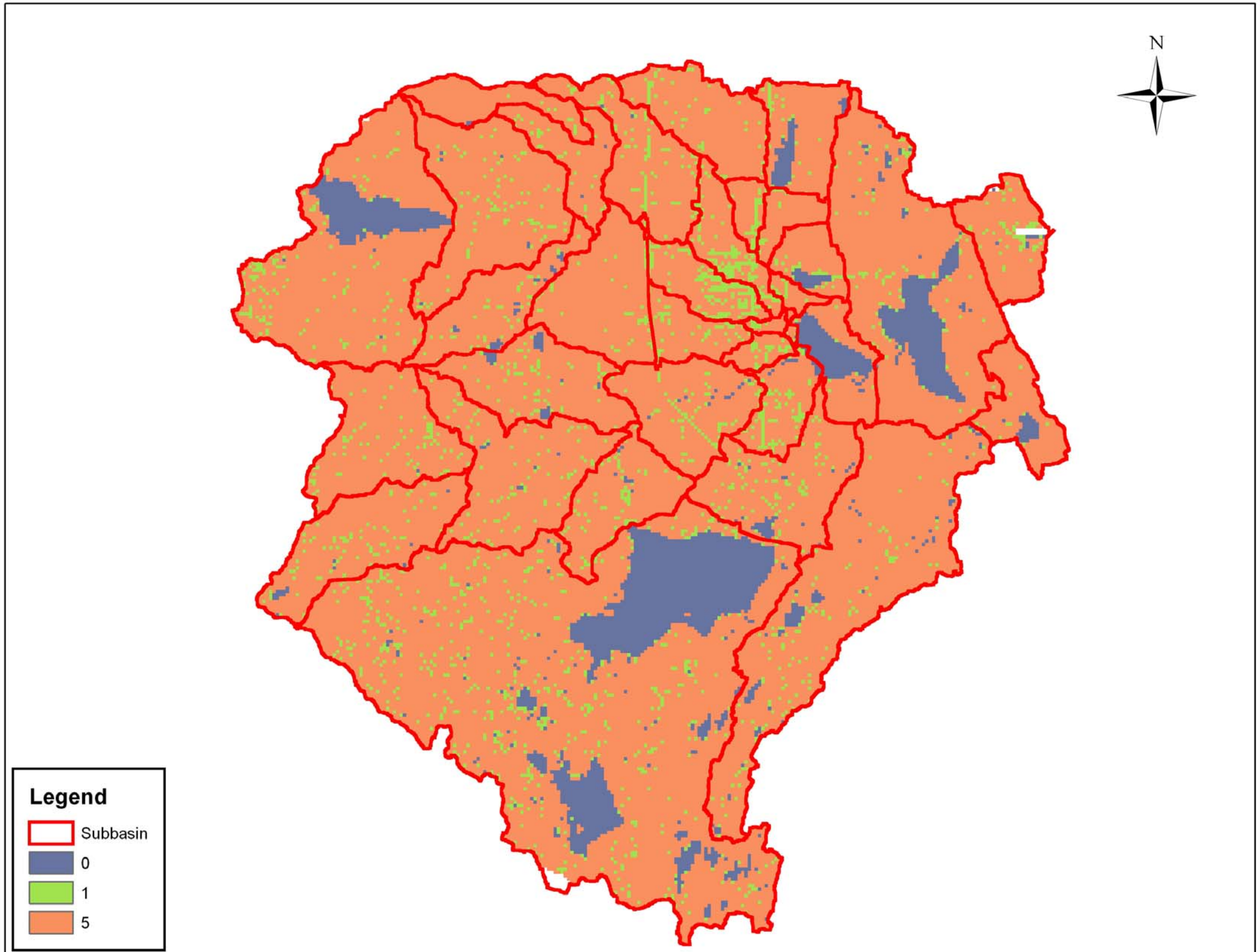

Figure 4-2e: Petty Harbour River Watershed – Initial Abstraction Grid

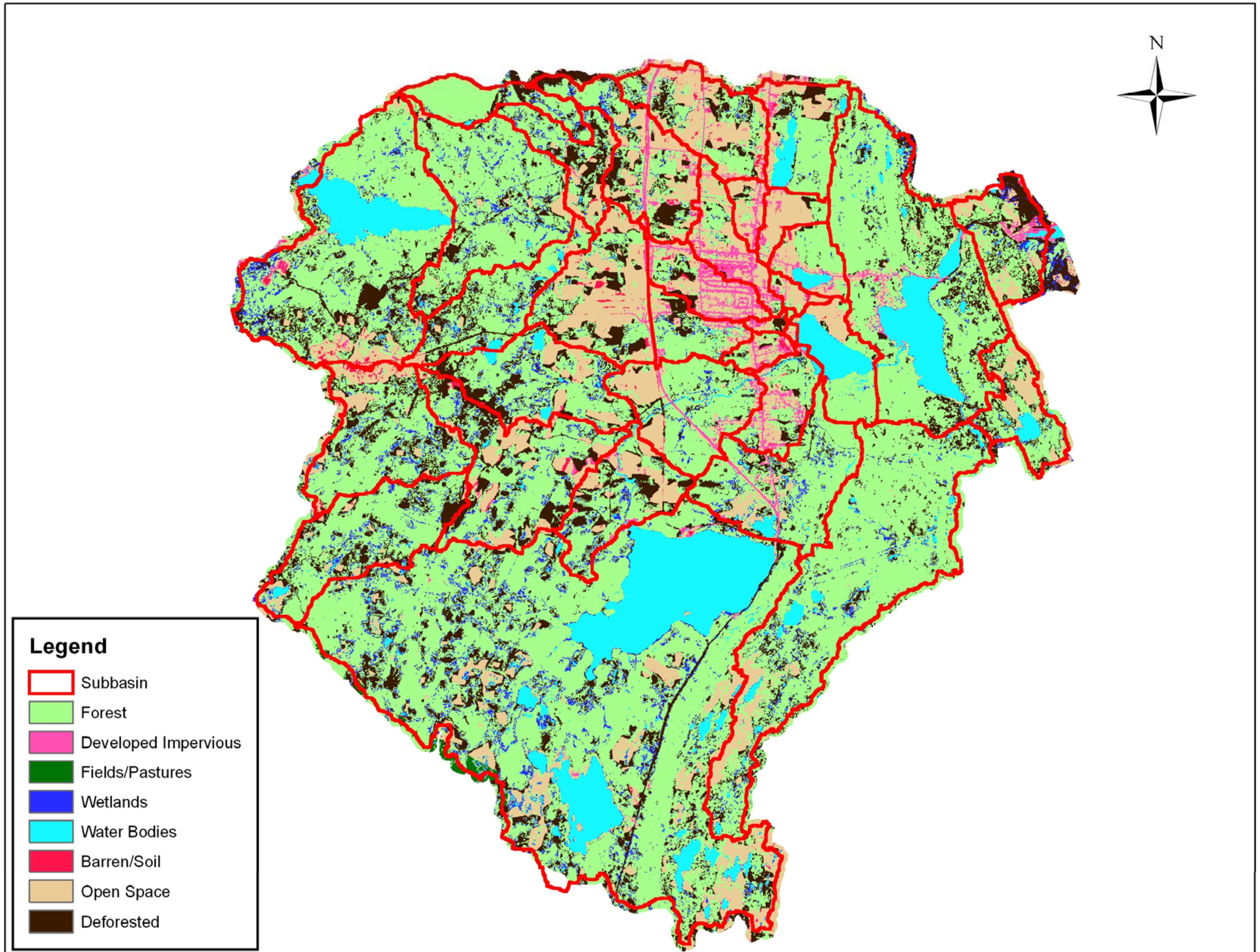

Figure 4-2f: Petty Harbour River Watershed – Land Cover

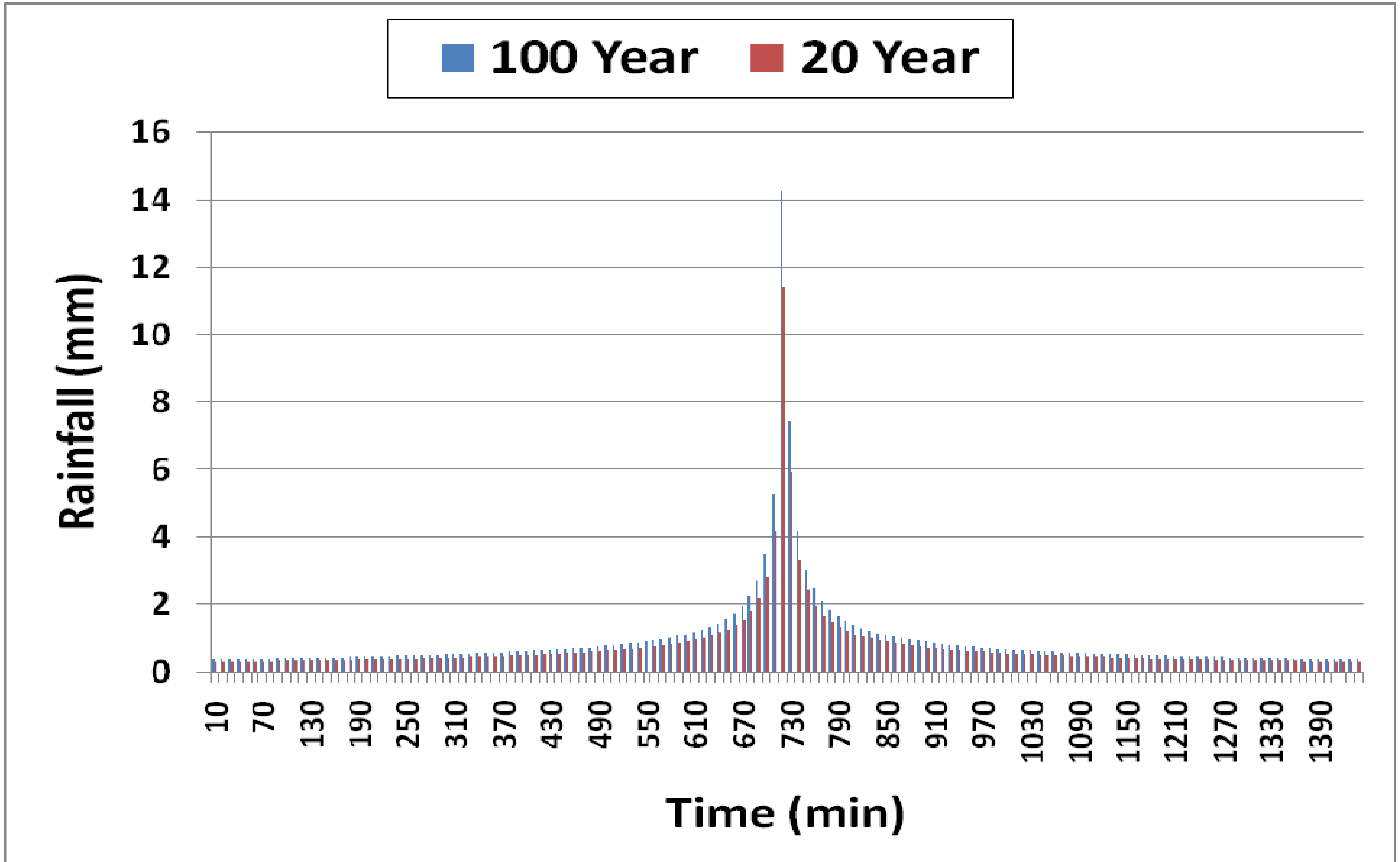

Figure 4-3: Existing Conditions Input Rainfall Distribution for Deterministic Modelling (10 minute time step)

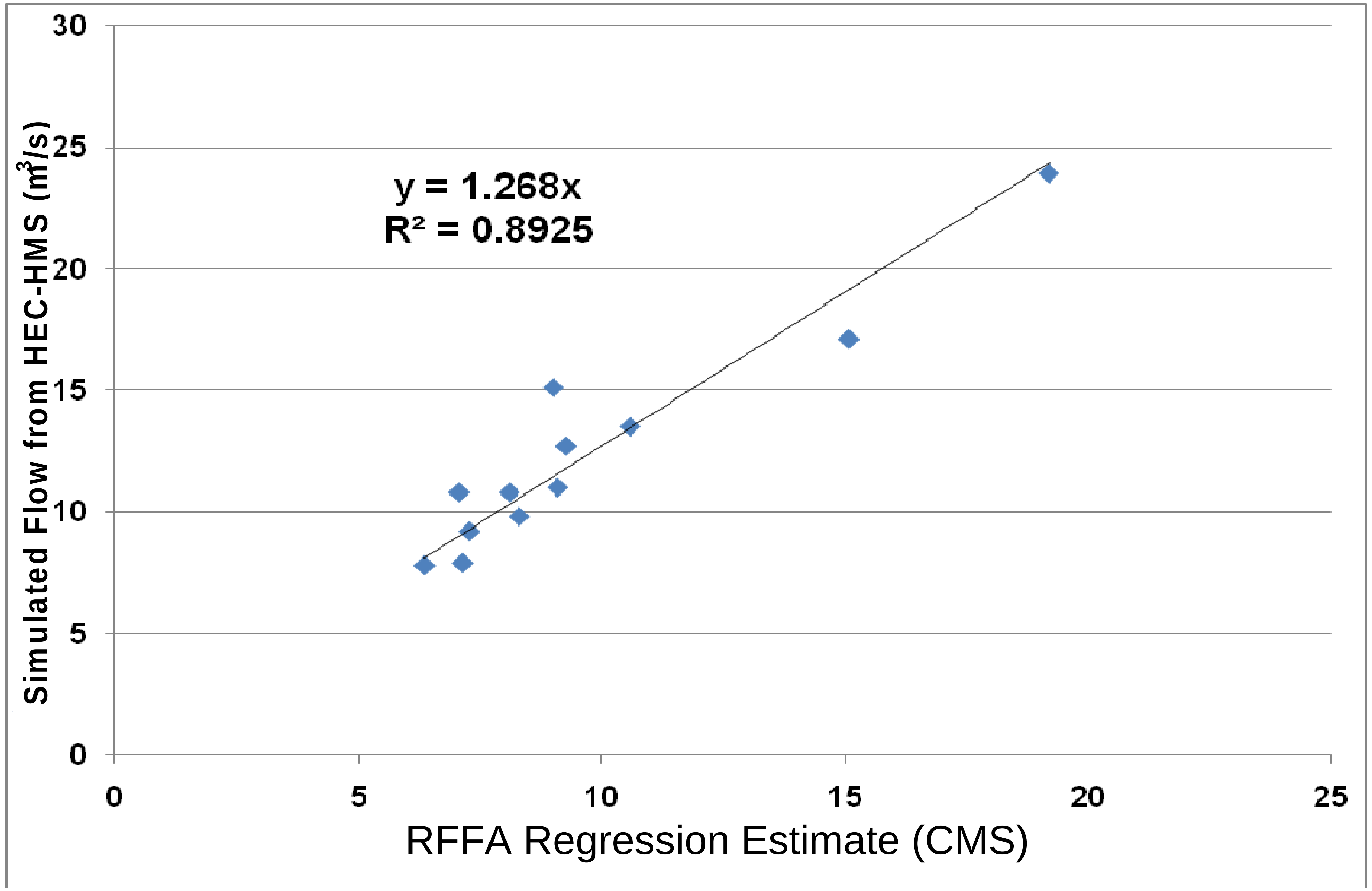

Figure 4-4: Comparison between RFFA Regression Estimates and HEC-HMS Simulated Flows for Selected Sub-basins in the Petty Harbour River Watershed

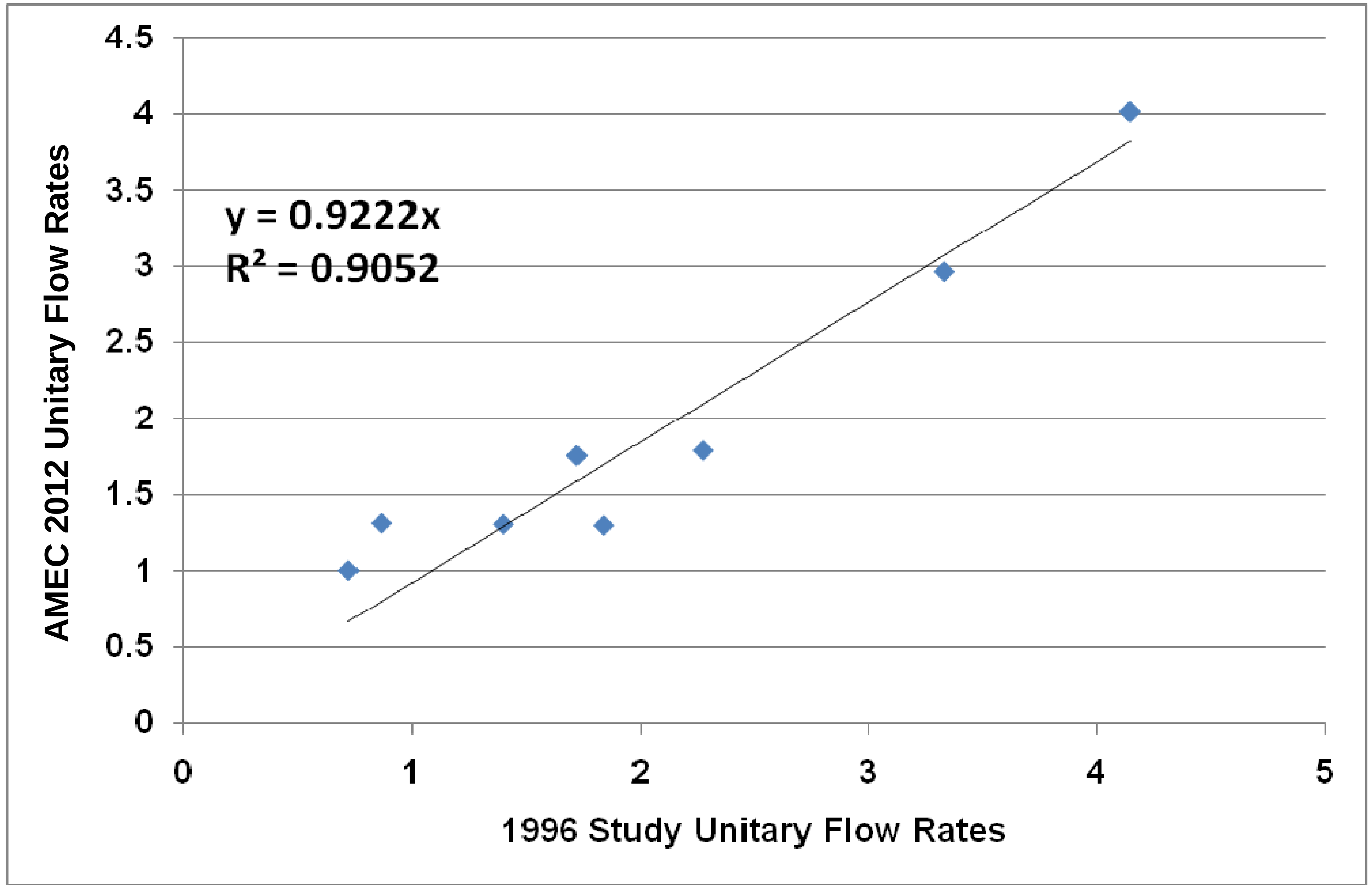

Figure 4-5: Comparison between Unitary Flow Rates for 1996 Flood Risk Mapping Study of Goulds, Petty Harbour and Ferryland and the Current Study

5.0 HYDRAULIC ANALYSIS

The collection and processing of data, computational procedures and analysis of computed profiles is compliant with criteria and guidelines published by the Hydrologic Engineering Center in the User's Manual and Training Documents (ref. USACE, 2010) and the *'Hydrologic and Hydraulic Procedures for Flood Plain Delineation'* (Environment Canada, 1976).

The objective of the hydraulic analysis was computation of water surface elevations resulting from the 1:20 year and 1:100 year AEP flow estimates. The computed water surface elevations are then used in conjunction with the LiDAR database or other mapping to visualize the limits of the flooding on flood risk maps. To determine the water surface profile for a given flood condition, a backwater analysis is generally necessary. The USACE HEC-RAS one-dimensional backwater model was selected for this analysis.

The following sections describe the development and calibration of the HEC-RAS hydraulic model, as well as the details associated with the results of the hydraulic simulation of various flood events.

5.1 Hydraulic Model Development

5.1.1 HEC-RAS

HEC-RAS (USACE, 2002), the successor to HEC-2, 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 computational procedures used by HEC-2 and HEC-RAS to model steady state flow are generally similar and are based on solving the one-dimensional energy equation. The HEC-RAS computational software estimates water surface elevation and related output along a channel reach under sub-critical, supercritical or mixed flow regimes. The program is capable of modelling complicated networks with multiple reaches and tributaries. Flow through culverts, bridges, weirs and gated spillways can also be accommodated. Levees, blocked obstructions and ineffective flow areas can also be modelled, as can ice jam and debris flow conditions.

In simple terms, the model uses surface water flow rates to predict water surface elevations. These elevations can then be transferred to a DTM or topographic map to identify the limits of flood-prone areas.

HEC-RAS requires a terrain model with three-dimensional attributes (x, y, and z) for the area of interest. The terrain model commonly used in hydrologic modeling is a DTM. HEC-GeoRAS is a pre- and post-processing program developed co-operatively by the Hydrologic Engineering Center (HEC) of the USACE and Environmental Systems Research Institute Inc. (ESRI) to:

- extract geometric data from a DTM for input into HEC-RAS, and;
- use output from the hydraulic model and generate a water surface elevation DTM that can be superimposed on the terrain DTM to identify flood-prone areas.

As noted previously, the DTM for this project was developed from the LiDAR database developed for this project, as described previously.

The HEC-GeoRAS 4.3.93 for ArcGIS 9.3 and HEC-RAS 4.1.0 were used to complete the one dimensional hydraulic modeling component of this project. HEC-RAS 4.1.0 represents the most up-to-date version of the software at the time of this project.

HEC-RAS is an approved model for flood plain calculations in Newfoundland and Labrador and was identified as the preferred modelling platform in the Terms of Reference for this project.

5.1.2 Cross Sections

Hydraulic sections were located in accordance with HEC-RAS modeling guidelines (USACE, 2010). Cross section data was abstracted from the LiDAR base mapping developed for this project supplemented with field surveyed cross-section data, as outlined in Section 3 of this report.

The locations of the sections are illustrated on the flood risk maps (see Appendices E, F, G, and H). The first cross-section of the hydraulic model is located at the entrance to Petty Harbour. This location was selected so as to ensure appropriate establishment of the downstream model boundary condition.

The LiDAR DTM developed for this project provides topographic information in a 1 m x 1 m grid to a vertical positional accuracy of +/- 0.1 m. Since the entire study watersheds were captured in the LiDAR survey, cross sections extending out past the floodplain extents were cut directly from the LiDAR without the need for supplementary field surveying.

As noted in Section 2, the below waterline survey data was integrated into the hydraulic models of the subject watercourses by adding a single cross-section X, Y point located at the centerline of the section along with a depth interpolated from the nearest surveyed cross-sections when compared with the LiDAR abstracted section elevation at that point.

An overview of the hydraulic models for each of the study watercourses follows:

- Overall study reach length of approximately 31.6 km
- 729 hydraulic sections across 19 reaches

- Minimum channel elevation -5.5 m at the start of the model
- Maximum channel elevation of about 125 m at the end of the model – Doyle's River
- Maximum channel elevation of about 141 m at the end of the model – Cochrane Pond Brook
- Maximum channel elevation of about 123 m at the end of the model – Raymond Brook

- Average inter-section reach length of about 50 m
- About 683 (or about 94%) of sections having inter-section reach length less than 100 m
- About 426 (or about 58%) of sections having inter-section reach length less than 50 m
- About 194 (or about 27%) of sections having inter-section reach length less than 25 m

5.1.3 Hydraulic Structures

Watercourse Crossings / Bridges

During the field survey, dimensions and elevations of each watercourse crossing listed in Table 3-1 were surveyed. This information is documented in the watercourse crossing sheets (available in Appendix B). Each of the surveyed structures was included in the hydraulic model. The rating curve, as generated by the hydraulic model, is included along with basic bridge survey data (invert, obvert, etc.) as components of the watercourse crossing information which allows for interpolation of bridge opening capacities (see Appendix B). Although the 1:20 year and 1:100 year AEP flows may exceed this value, the structure may still not be overtopped. This result is because the structures can become surcharged to gain additional head to pass the flow and/or there is a change in the flow regime whereby a higher flow results in a lower water level. Indication of overtopping of any watercourse crossing or bridge in the study reach is provided in Table 8-2.

Dams

Three dams are included in the hydraulic model of Petty Harbour River, namely Petty Harbour Dam, Bay Bulls Big Pond Dam and Cochrane Pond Outlet. The scenario around which the hydraulic modelling of the dams was developed was based on a variety of elements representing a reasonable worst case associated with the 1:20 year and 1:100 year rainfall events. The following elements, consistent between the two design rainfall events, were considered:

- No gate operation during the event at Bay Bulls Big Pond Dam. This is consistent with the information provided by Newfoundland Power Inc. whereby it was indicated that operation of the gates during extreme weather is done in a manner that does not worsen downstream flood conditions. This operation is typically enacted in the days after the event.
- The turbines are “tripped” at the Petty Harbour Generating Station resulting in no flow diversion via the penstock to the generating station.
- Head pond water levels were defined consistent with Operational Scenario #3 (ref. Section 4.0)

HEC-RAS provides functionality for modelling of in-line structures such as dams. However, the scenario (specific to dams) upon which the hydraulic model was based essentially removes gate operation from consideration. As such, the dam modelling functionality within HEC-RAS reverts to weir flow over the dam using a section defined across the dam crest, abutments and overbanks. Weir flow co-efficients are elements of the dam definition input which the program uses to determine a stage-discharge relationship for the dam.

As a stage-discharge relationship for the dams had already been independently defined (to support hydrologic modelling), the dam crests were modelled as cross-sections with rating curves. Cross-sections upstream and downstream of the dams were also defined consistent with the in-line structure coding requirements. This approach maintained consistency with the stage-discharge relationships used for the hydrologic modelling.

Water levels associated with dams are provided in Table 8-2.

5.1.4 Lateral Structures

No lateral structures (i.e., side weirs and similar) are located in the study reaches for this project. However, a rock containment berm is located in the south bank upstream of the Main Road bridge in Petty Harbour. This berm was included in the hydraulic model as a topographic feature through cross-section definition.

5.1.5 Energy Loss Coefficients

Energy loss coefficients are used in the HEC-RAS program to calculate changes in the water surface elevation between sections. The coefficients include Manning roughness coefficients, expansion and contraction coefficients, and weir and pressure coefficients for road / rail crossings. These coefficients were estimated based on published information, field reconnaissance and engineering judgment.

5.1.5.1 Expansion and Contraction Coefficients

Expansion and contraction coefficients for normal channel cross-section were set at 0.1 and 0.3, respectively, and 0.3 and 0.5 for cross-sections at hydraulic structures respectively. These ratios are used by HEC-RAS in the computation of energy losses due to flow contraction and expansion between adjacent cross sections. The noted values are consistent with those recommended in the HEC-RAS Technical Reference Manual.

5.1.5.2 Roughness Coefficients

Estimation of Manning roughness coefficients was based on field observation, review of satellite imagery (available via Google Maps™) and orthophotos, engineering judgment, previous modeling experience, and comparison of reach characteristics with the “Roughness Characteristics of Natural Channels” (Barnes, 1967). Images available via Google Streetview™ were also helpful in this regard.

Roughness coefficients used for the hydraulic model were in the range 0.035 to 0.050 for channels and 0.055 to 0.080 for overbank areas. Channels through the study area range from clean, gravel bottom to large boulders with debris (represented by the low and high range of roughness co-efficient). For the overbank areas the lower range represented grassed areas clear of significant vegetation and the upper range represented forested overbank areas.

5.1.5.3 Weir Flow Coefficients

HEC-RAS defaults to a generic weir coefficient of 1.4 for watercourse crossing (i.e.

bridge/culvert) modelling. For this project, weir flow coefficients were also estimated using the method outlined in the Connecticut Department of Transportation - Drainage Manual, Chapter 8, Section 8 (CONNDot, 2000) as a means of confirming this parameter value. Weir coefficient estimates were determined to be in the range of about 1.6 to 1.67 using this method. The final hydraulic models use the CONNDot method estimates given that they are linked to actual field conditions.

5.1.6 Starting Water Surface Elevations

Table 5-1 presents maximum tidal elevations for the study area. The sources of the values reported are noted at the bottom of the table. Tide table values are taken from the particular port (i.e. St. John's). For orientation, Figure 5-1 illustrates the relation between tidal surfaces (MWL[10], HHWMT[11], HHWLT[12]), charting datums, and physical features. 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 5-2. Future predictions for sea level rise are made based on predictions presented in Batterson and Liverman (2010) which include Intergovernmental Panel on Climate Change (IPCC) sea level predictions, potential accelerated ice melt, and regional trends of crustal rebound.

In the absence of an extremal analysis of water level measurements, it is noted that the HHWMT/LT (tidal water level, i.e. without surge) values quoted are generally representative of a 20 year return period (as they are based on 19 years of predictions) while the recorded extreme value (Recorded Extreme, HHW) from the Department of Fisheries and Oceans (DFO) tide tables are for the historical record at St. John's is reflective of a 100 year return period.

The guideline document for this study, *Hydrologic and Hydraulic Procedures for Flood Plain Delineation* (Environment Canada, 1976), provides no specific direction for establishing starting water levels for hydraulic modelling. For the purposes of this study, the starting water surface elevation was computed as the maximum high tide (large tide for higher high water - HHWLT) of 1.6m (geodetic) plus a storm surge of 0.95m for existing conditions. It should be noted that, in the absence of tide and surge observations specifically at the downstream limits of the hydraulic models, both parameters were assumed to be the same as observed by the Canadian Hydrographic Service (CHS) at the locations noted in Table 5-1. This provides a combined total of 2.55m which was used as the downstream boundary condition in the existing conditions hydraulic models for the 1:20 year and 1:100 year AEP flood simulations. This approach is consistent with previous hydrotechnical studies completed for WRMD such as the Flood Risk Mapping Project for Shearstown / Bay Roberts Area (Hatch 2012).

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[10] 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 (Forrester, 1983).

[11] HHWMT: is higher high water, mean tide, which is the average of all the higher high waters from 19 years of predictions (Forrester, 1983).

[12] HHWLT: is higher high water, large tide, which is the average of the highest high waters, one from each of 19 years of predictions (Forrester, 1983).

The future conditions models also incorporate a sea level rise component resulting in starting water surface elevations or 2.61m, 2.82m and 3.18m, respectively, for the 2020, 2050 and 2080 time frames.

Table 5-1: Tidal Elevations

Description	**Elevation (m)**
MWL (m)	0.8 [(1)]
HHWMT (m)	1.3 [(1)]
HHWLT (m)	1.6 [(1)]
Recorded Extreme, HHW (m)	2.5 [(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: St. John's (DFO, 2012a)
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

HWLT: 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

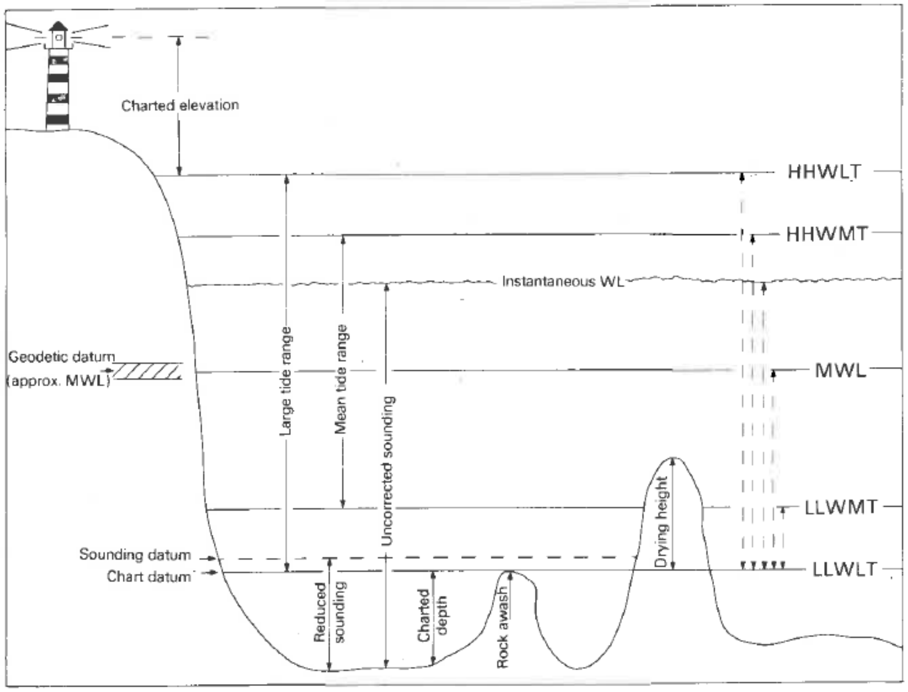

Figure 5-1: Relation between tidal surfaces, charting datums and physical features (Source: Forrester, 1983)

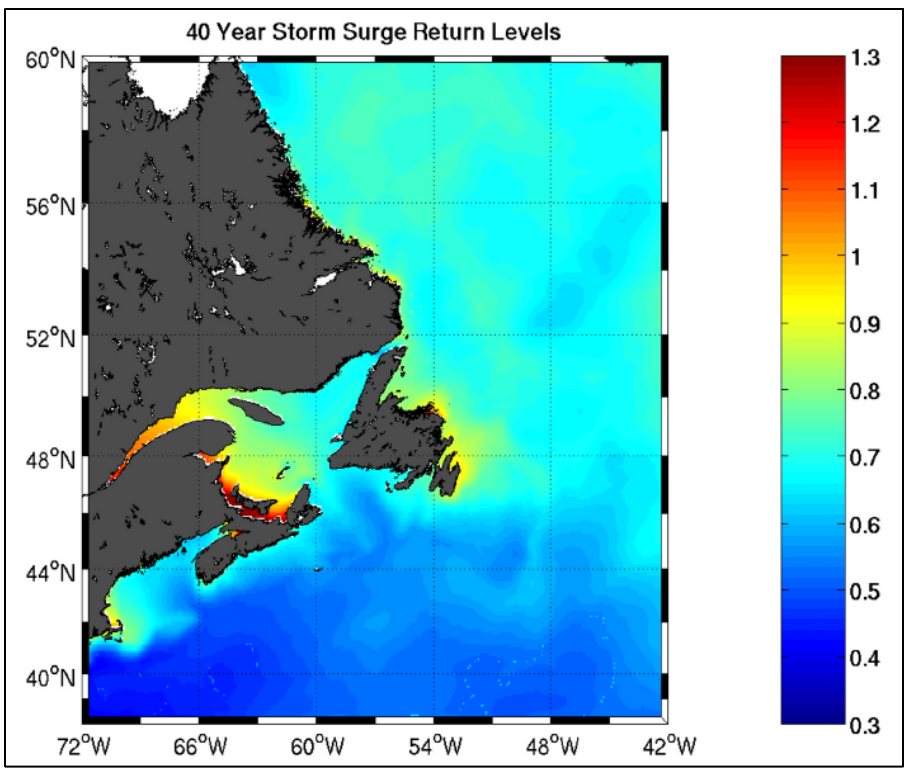

Figure 5-2: 40 year return level of extreme storm surges (Source: Bernier and Thompson, 2006)

5.1.7 Hydraulic Model Calibration/Validation

Hydraulic data to support calibration and validation was not available for this study. No hydrometric stations are in operation within the study reach which are effectively located to support model calibration. Further, issues previously noted (see Section 3.3) precluded point streamflow level data collection during the course of the study.

5.1.8 Simulation of the 1:20 year and 1:100 year AEP Flood Events

Peak flows through the study reaches were computed using the deterministic model developed for this project. These peak flows (determined in part using the CBCL IDF relationship) were input to a steady state hydraulic model for the purpose of estimating the water surface profiles corresponding to the 1:20 year and 1:100 year AEP flood events. The resultant water level output from the HEC-RAS model was used to delineate the extent of flooding on maps as discussed in Section 8.

HEC-RAS output defining computed water surface elevations for the 1:20 year and 1:100 year AEP events is provided in Appendix J. An outline of watercourse crossing / bridges and dams in the study reach and local computed water surface elevations, as a means of identifying which structures are overtopped, is provided in Table 8-2.

5.2 Ice Jam Assessment

Ice jams may develop when there is a rapid increase in discharge due to a rain or snowmelt event in winter that causes an intact ice cover to lift and break into pieces. The increased thickness and physical roughness of an ice jam often produces flood levels that exceed the 1:100 year open water flood level at considerably lower discharges. Available historical information related to ice jam occurrences in the Goulds and Petty Harbour flood risk mapping areas, along with the ice jam analysis approach employed, and the resulting flood levels, are described below.

5.2.1 Historical Context

As noted in Section 2.1, only the Goulds area has experienced ice jam flooding problems. Five (5) cases of ice jam flooding are presented in the Flood Risk Mapping Study for Goulds and Petty Harbour and Ferryland (BAE-Newplan Group, 1996). A summary of the dates and descriptions of these ice jam flood occurrences is contained in Table 2-2. These records indicate that the most frequent ice jam initiation sites are the outlets at Cochrane Pond Brook and Raymond Brook, at Third Ponds.

There is no historical evidence of ice jam flooding in the Petty Harbour area. Due to the flow regulation of the Petty Harbour Dam at the outlet of the First Pond and limited ice supply along the Petty Harbour River, it is assumed that the possibility of ice jamming is very low in the Petty Harbour area. Therefore, it was not considered in the ice jam modeling.

No ice thickness data were available along the study reaches; however, Water Survey of Canada (WSC) records and reports ice thickness at a nearby hydrometric station, St. Shott's River near Trepassey (02ZN002), which is located on the Avalon Peninsula, approximately 100 km southwest of the study site. Seven (7) years of ice thickness data (2001 to 2012) were available at this station, which had an average of 40 cm and a maximum of 80 cm. From the geometry of the HEC-RAS model channels, typical depths of Raymond Brook and Cochrane Pond Brook are on the order 0.4 m and 0.7 m respectively. Considering the relative size of these channels it was determined that 0.3 m is a more reasonable value for ice cover thickness at the study area than the range observed on the St. Shott's River. This estimate was used to determine the maximum ice supply available to form an ice jam within selected sub-reaches.

5.2.2 Analysis Approach

Several factors were considered when developing plausible ice jam scenarios for modelling. These included historic evidence of ice jam activity, geomorphic conditions, in-stream structures, and an appropriate peak discharge during breakup.

For the Goulds area, plausible ice jam initiation sites were considered along the Raymond and Cochrane Brooks, respectively. These sites included the inlets into Third Pond (i.e. outlets of Raymond and Cochrane Brooks), the sharp bends, the confluences of the tributaries to the main stream, and the constrictions at the highway bridges over Raymond and Cochrane Pond Brooks.

For the purpose of estimating the discharge during an ice jam event in winter, the meteorological data at the St. John's Airport station (#8403506) were assumed to be representative of the study area. Plausible rain-on-snow events occurring in the months from January to April were examined to estimate the direct runoff during the ice-affected season. The highest runoff value was selected in order to choose an event of similar rainfall volume from the IDF curve for St. John's. Then the HEC-HMS model developed for the study area was applied to simulate the peak discharges for the 1:20 year and 1:100 year winter flood events in the Goulds and Petty Harbour areas. Design discharges estimates generated for the HEC-RAS ice jam simulations are shown in Table 5-2.

The HEC-RAS model developed for the open water flood hazard mapping was applied directly for the ice jam modelling, using the same modelling parameters and boundary conditions, with the exception of inflow discharges, as noted earlier. Multiple ice jam locations and lengths, constrained by the limit of available ice volume were simulated using the adopted ice jam modelling parameters shown in Table 5-3. These values were selected based on experience at other sites and a review of relevant prior ice jam analyses conducted for Newfoundland flood hazard mapping studies. The sensitivity of ice roughness, friction angle, jam porosity, and stress ratio were investigated and found not to be significant to the predicted water surface profile along Raymond and Cochrane Pond Brooks.

Table 5-2: Winter Peak Discharges – Petty Harbour River

No.	River	Reach	Station	1:100 year (m^3/s)	1:20 year (m^3/s)
1	CochranePB	M002	8105.53	9.3	5.1
2	CochranePB	M002	5609.94	14	7.8
3	CochranePB	M002	5162.38	26.4	14.6
4	CochranePB	M002	3434.16	28.9	16.0
5	CochranePB	M002	1185.68	29.7	16.4
6	CochranePB	M001	381.27	59.6	33.0
7	DoylesR	TR019	260.51	0.2	0.1
8	DoylesR	TR018	302.01	0.9	0.5
9	DoylesR	TR017	4043.45	5.1	2.9
10	DoylesR	TR016	918.87	1.9	1.1
11	DoylesR	TR015	3230.71	11.2	6.3
12	DoylesR	TR014	2057.49	6.7	3.8
13	DoylesR	TR014	1384.41	7.7	4.4
14	DoylesR	TR012	1780.65	18.9	10.6
15	DoylesR	TR011	1552.47	27.2	15.2
16	DoylesR	TR013	2205.78	5.9	3.4
17	DoylesR	M001	484.12	33.1	18.5
18	ForestP	M001	1557.91	5.2	2.9
19	ForestP	M001	958.61	6.7	3.7
20	ForthP	M001	2616.9	7.4	4.2
21	ForthP	M001	1392.58	10.1	5.6
22	PettyHR	M002	6270.85	62	34.5
23	PettyHR	M001	5825.85	109.2	66.0
24	PettyHR	M001	5516.39	120.3	72.0
25	PettyHR	M001	4923.81	120.3	72.0
26	PettyHR	M001	3931.39	74.2	44.4
27	PettyHR	M001	1615.46	75.1	44.9
28	RaymondB	TR001	439.21	22.7	12.8
29	RaymondB	M002	6453.6	16	13.0
30	RaymondB	M001	4130.35	37.9	25.8
31	RaymondB	M001	3916.71	50	32.5
32	RaymondB	M001	1252.39	52.1	33.8

Table 5-3: Ice Jam Parameters Adopted for Petty Harbour River

Ice Jam Parameter	Value
Intact Ice Thickness	30 cm
Ice Jam Roughness	0.06
Friction Angle	45°
Porosity	0.4
Stress Ratio	0.33
Maximum Velocity	3.0 m/s
Ice Specific Gravity	0.92
Ice Cohesion	0.0 kPa

5.2.3 Ice Jam Flood Profiles

Based on the results, the highest water level value at each river station is considered as the ice jam flood level. The detailed modelling results are shown in Tables 5-4 and 5-5 for Raymond and Cochrane Pond Brooks, respectively.

Raymond Brook computed ice jam flood levels generally dominate over the open water flood levels over the entire reach with 129 of 144 sections with computed results indicating higher ice jam water levels. The 1:100 year maximum ice jam water level increase over the open water computations is 1.9m with an average of about 0.7m (when compared with open water computed water levels associated with the EC IDF relationship). Results are similar in comparison between the ice jam computed water levels and the CBCL IDF relationship associated computed open water levels. Results are also similar for the 1:20 year comparison.

Cochrane Pond Brook computed ice jam flood levels are generally higher than the open water flood levels over the entire reach with 124 of 190 sections with computed results indicating higher ice jam water levels. The 1:100 year maximum ice jam water level increase over the open water computations is 1.5m with an average of about 0.2m (when compared with open water computed water levels associated with the EC IDF relationship). Results are similar in comparison between the ice jam computed water levels and the CBCL IDF relationship associated computed open water levels. Results are also similar for the 1:20 year comparison.

Table 5-4: Comparison of Open Water and Ice Jam Flood Levels at Raymond Brook

Reach	River Station	1:100 year flood level (m)		1:20 year flood level (m)	
		ice jam	open water	ice jam	open water
M002	6453.6	125.18	**125.99**	125.12	**125.92**
M002	6405.52	125.18	**125.99**	125.12	**125.92**
M002	6347.04	125.12	**125.99**	125.07	**125.92**
M002	6247.26	**118.21**	117.99	**118.13**	117.92
M002	6230.1	**118.21**	117.98	**118.13**	117.90
M002	6205.35	**118.12**	117.96	**118.05**	117.88
M002	6143.06	**118.02**	117.85	**117.95**	117.79
M002	6105.93	**117.92**	117.71	**117.85**	117.64
M002	6026.35	**117.67**	117.47	**117.59**	117.38
M002	5933.71	**117.35**	117.15	**117.26**	117.05
M002	5883.92	**117.12**	116.93	**117.04**	116.84
M002	5785.15	**116.71**	116.50	**116.62**	116.41
M002	5732.44	**116.50**	116.19	**116.42**	116.10
M002	5631.37	**116.37**	115.56	**116.29**	115.53
M002	5500.15	**115.88**	115.36	**115.80**	115.28
M002	5485.46	**115.86**	115.35	**115.78**	115.27
M002	5411.19	**115.77**	115.25	**115.68**	115.15
M002	5338.54	**115.76**	115.25	**115.67**	115.15
M002	5278.93	**115.75**	115.24	**115.66**	115.15
M002	5226.6	**115.75**	115.23	**115.65**	115.14
M002	5171.66	**115.73**	115.22	**115.64**	115.12
M002	5111.5	**115.71**	115.19	**115.61**	115.09
M002	5077.48	**115.69**	115.17	**115.59**	115.07
M002	5017.13	**115.55**	115.06	**115.46**	114.96
M002	4967.43	**115.32**	114.80	**115.23**	114.70
M002	4943.43	**115.21**	114.69	**115.12**	114.59
M002	4910.95	**115.02**	114.55	**114.95**	114.45
M002	4851.17	**114.74**	114.29	**114.67**	114.22
M002	4783.73	**114.51**	114.02	**114.44**	113.96
M002	4732.68	**114.22**	113.72	**114.16**	113.65
M002	4681.76	**113.57**	113.04	**113.44**	112.94
M002	4673.96	**113.20**	112.36	**112.96**	112.27
M002	4644.78	**111.80**	111.13	**111.70**	111.07
M002	4625.72	**108.06**	107.14	**108.35**	107.08
M002	4615.42	**107.90**	106.37	**108.21**	106.32
M002	4607.06	**107.85**	106.74	**107.99**	106.66
M002	4597.34	**107.82**	106.69	**107.92**	106.61
M002	4575.18	**107.38**	106.38	**107.24**	106.32
M002	4545.57	**106.40**	105.43	**106.32**	105.33
M002	4522.17	**105.58**	104.20	**105.48**	104.13
M002	4511.42	**105.50**	104.56	**105.40**	104.45
M002	4489.94	**105.26**	104.26	**105.16**	104.17
M002	4466.63	**104.90**	104.04	**104.82**	103.91
M002	4439.46	**104.54**	103.74	**104.47**	103.63

Reach	River Station	1:100 year flood level (m)		1:20 year flood level (m)	
		ice jam	open water	ice jam	open water
M002	4415.77	**104.21**	103.33	**104.15**	103.30
M002	4385.89	**103.49**	102.42	**103.41**	102.30
M002	4354.01	**102.65**	101.95	**102.60**	101.88
M002	4341.66	**102.37**	101.60	**102.31**	101.48
M002	4332.73	**101.97**	100.87	**101.90**	100.79
M002	4308.73	**100.66**	99.40	**100.55**	99.35
M002	4294.65	**100.33**	99.09	**100.25**	99.01
M002	4263.86	**99.96**	98.40	**99.71**	98.62
M002	4235.25	**99.89**	98.85	**99.60**	98.61
M001	4130.35	**99.28**	97.98	**99.06**	97.80
M001	4076.94	**98.02**	97.62	**98.45**	97.46
M001	4023.53	**97.97**	97.08	**98.08**	96.93
M001	3978.2	**97.63**	96.70	**97.54**	96.52
M001	3916.71	**97.14**	96.38	**96.92**	96.19
M001	3863.3	**96.89**	96.25	**96.63**	96.03
M001	3809.58	**96.60**	95.76	**96.31**	95.59
M001	3756.26	**96.09**	95.01	**95.83**	94.80
M001	3694.37	**95.27**	94.07	**95.07**	93.88
M001	3569.22	**94.42**	94.07	**94.11**	93.70
M001	3527.64	**94.32**	94.05	**93.99**	93.67
M001	3446.51	**94.25**	94.02	**93.93**	93.64
M001	3421.64	**93.80**	93.40	**93.67**	93.28
M001	3410 BR U	**93.78**	93.03	**93.69**	93.16
M001	3410 BR D	**93.74**	93.07	**93.68**	93.00
M001	3404.11	**93.48**	93.03	**93.25**	92.71
M001	3380.48	**93.55**	92.85	**93.28**	92.67
M001	3308.92	**93.16**	92.46	**92.89**	92.10
M001	3216.73	92.15	**92.25**	91.69	**91.80**
M001	3200 BR U	92.15	**92.21**	91.69	**91.77**
M001	3200 BR D	**92.02**	91.91	**91.59**	91.49
M001	3188.78	**91.47**	91.46	91.21	**91.22**
M001	3015.18	**91.04**	90.80	**91.19**	90.60
M001	2876.32	**90.91**	90.20	**90.73**	90.08
M001	2789.37	**90.42**	90.00	**90.26**	89.87
M001	2681.85	**90.05**	89.47	**89.90**	89.39
M001	2604.32	**89.73**	89.30	**89.60**	89.15
M001	2531.57	**89.73**	89.27	**89.55**	89.11
M001	2385.34	**89.64**	89.19	**89.44**	89.00
M001	2245.72	**89.52**	88.63	**89.23**	88.37
M001	2194.47	**89.38**	88.71	**89.05**	88.43
M001	2140.67	**89.32**	88.60	**88.98**	88.27
M001	2087.6	**89.14**	88.46	**88.83**	88.15
M001	2069.54	**89.07**	88.41	**88.78**	88.11
M001	2040.77	**88.95**	88.35	**88.69**	88.04
M001	1989.18	**88.71**	88.25	**88.46**	87.79

Reach	River Station	1:100 year flood level (m)		1:20 year flood level (m)	
		ice jam	open water	ice jam	open water
M001	1950.22	**88.44**	87.45	**88.18**	87.35
M001	1885.73	**87.88**	87.05	**87.62**	86.86
M001	1835.84	**87.38**	86.44	**87.09**	86.17
M001	1784.61	**86.15**	85.26	**86.01**	85.18
M001	1733.38	**85.71**	84.77	**85.39**	84.50
M001	1682.15	**85.45**	84.40	**85.15**	84.16
M001	1630.92	**84.98**	83.89	**84.73**	83.64
M001	1584.49	**84.60**	83.68	**84.33**	83.44
M001	1536.01	**84.18**	83.21	**83.91**	83.00
M001	1477.96	**83.81**	82.79	**83.52**	82.59
M001	1425.99	**83.47**	82.52	**83.21**	82.30
M001	1372.05	**83.21**	82.36	**83.02**	81.80
M001	1323.54	**82.78**	81.65	**82.20**	81.21
M001	1315 BR U	**81.74**	81.15	**82.06**	80.77
M001	1315 BR D	**81.78**	79.86	**82.07**	79.55
M001	1303.54	**81.71**	80.49	**81.55**	80.32
M001	1252.39	**81.25**	80.38	**80.98**	80.05
M001	1216.25	**80.56**	79.72	**80.33**	79.60
M001	1189.75	**79.96**	79.04	**79.67**	78.90
M001	1166.2	**79.11**	78.20	**78.89**	78.08
M001	1148.7	**78.95**	78.00	**78.71**	77.89
M001	1116.25	**78.51**	77.12	**78.27**	76.95
M001	1066.09	**77.79**	76.76	**77.56**	76.47
M001	1035.68	**77.31**	76.27	**77.12**	76.10
M001	999.99	**76.19**	74.63	**75.96**	74.41
M001	976.73	**75.42**	74.49	**75.25**	74.35
M001	942.15	**73.92**	73.22	**73.75**	73.10
M001	916.7	**73.02**	72.13	**72.83**	72.00
M001	892.34	**72.81**	71.75	**72.61**	71.53
M001	866.25	**72.58**	71.81	**72.40**	71.60
M001	841.89	**72.30**	71.55	**72.14**	71.39
M001	816.68	**71.98**	71.35	**71.80**	71.19
M001	766.25	**71.20**	70.03	**70.98**	69.93
M001	716.25	**70.68**	69.86	**70.47**	69.70
M001	675	**70.38**	69.65	**70.16**	69.45
M001	615.86	**69.98**	69.59	**69.81**	69.25
M001	565.77	**69.73**	69.55	**69.54**	69.18
M001	520.15	69.47	**69.55**	69.18	**69.18**
M001	468.54	69.39	**69.55**	68.89	**69.18**
M001	429.18	69.38	**69.55**	68.88	**69.18**
M001	366.25	69.38	**69.55**	68.88	**69.18**
M001	341.79	69.38	**69.55**	68.88	**69.18**
TR001	439.21	106.49	**106.49**	106.24	**106.26**
TR001	409.42	**106.36**	106.23	**106.13**	105.88
TR001	380.07	106.09	**106.33**	105.67	**105.70**

Reach	River Station	1:100 year flood level (m)		1:20 year flood level (m)	
		ice jam	open water	ice jam	open water
TR001	360.64	105.51	**106.33**	105.33	**105.42**
TR001	350 BR U	105.16	**105.78**	104.76	**105.36**
TR001	350 BR D	105.02	**105.18**	104.36	**104.88**
TR001	344.45	**105.19**	104.98	**104.97**	104.84
TR001	334.13	**105.07**	104.29	**104.87**	104.19
TR001	272.22	**104.37**	103.97	**104.17**	103.80
TR001	218.5	**103.37**	102.67	**103.15**	102.51
TR001	163.97	**102.65**	101.84	**102.44**	101.60
TR001	115.54	**101.43**	101.31	**101.15**	101.09
TR001	56.65	**99.84**	98.45	**99.54**	98.34

NOTES:

1. The bold numbers indicate the higher levels between ice jam and open water conditions.

Table 5-5: Comparison of Open Water and Ice Jam Flood Levels in Cochrane Pond Brook

Reach	River Station	1:100 year flood level (m)		1:20 year flood level (m)	
		ice jam	open water	ice jam	open water
M002	8105.53	142.27	**142.47**	142.09	**142.20**
M002	8065.47	142.26	**142.46**	142.06	**142.18**
M002	8050.98	142.26	**142.45**	142.03	**142.18**
M002	8024.09	142.26	**142.45**	142.02	**142.18**
M002	8013.06	142.26	**142.45**	142.01	**142.17**
M002	8004.7	142.01	**142.45**	141.58	**142.17**
M002	7995.14	142.01	**142.44**	141.66	**142.17**
M002	7964.49	141.61	**141.89**	141.31	**141.70**
M002	7901.64	140.63	**140.76**	140.44	**140.64**
M002	7847.08	139.77	**139.82**	139.64	**139.74**
M002	7812.33	**139.36**	139.35	139.28	**139.30**
M002	7738.34	138.87	**138.96**	138.76	**138.88**
M002	7695.32	138.54	**138.63**	138.46	**138.56**
M002	7609.21	137.64	**137.79**	137.47	**137.69**
M002	7515.77	137.52	**137.65**	137.38	**137.56**
M002	7476.72	137.50	**137.63**	137.36	**137.54**
M002	7450.86	137.47	**137.59**	137.33	**137.51**
M002	7329.89	137.23	**137.30**	137.13	**137.22**
M002	7253.62	137.01	**137.13**	136.91	**137.05**
M002	7208.54	136.88	**137.03**	136.78	**136.95**
M002	7108.13	136.48	**136.53**	136.38	**136.43**
M002	7041.67	135.96	**136.11**	135.79	**136.07**
M002	6985.71	135.45	**135.67**	135.23	**135.47**
M002	6955.25	134.69	**134.71**	134.49	**134.61**
M002	6863.52	132.80	**132.88**	132.71	**132.81**
M002	6840.09	132.15	**132.15**	132.06	**132.09**
M002	6808.25	131.41	**131.41**	131.33	**131.41**
M002	6767.75	130.28	**130.36**	130.18	**130.26**
M002	6727.78	129.44	**129.48**	129.33	**129.43**
M002	6691.71	128.96	**129.07**	128.84	**128.99**
M002	6651.54	128.53	**128.67**	128.45	**128.61**
M002	6630.83	128.28	**128.44**	128.15	**128.35**
M002	6534.5	127.22	**127.30**	127.12	**127.24**
M002	6507.97	126.57	**126.58**	126.49	**126.53**
M002	6480.5	125.87	**125.93**	125.78	**125.87**
M002	6433.33	125.00	**125.07**	124.90	**125.01**
M002	6400.96	124.46	**124.61**	124.31	**124.50**
M002	6364.18	124.44	**124.62**	124.28	**124.50**
M002	6314.31	124.37	**124.55**	124.23	**124.44**
M002	6286.25	124.28	**124.47**	124.14	**124.36**
M002	6202.53	123.88	**124.06**	123.74	**123.98**
M002	6130	123.27	**123.36**	123.16	**123.26**
M002	6040.82	122.51	**122.62**	122.40	**122.54**
M002	6006.95	122.49	**122.58**	122.39	**122.52**

Reach	River Station	1:100 year flood level (m)		1:20 year flood level (m)	
		ice jam	open water	ice jam	open water
M002	5976.09	122.34	**122.41**	122.26	**122.37**
M002	5910.4	121.45	**121.53**	121.27	**121.43**
M002	5863.68	120.54	**120.54**	120.43	**120.50**
M002	5811.63	119.90	**119.98**	119.78	**119.84**
M002	5768.42	**118.41**	118.35	118.27	**118.34**
M002	5725.36	117.52	**117.63**	117.40	**117.55**
M002	5678.58	116.45	**116.48**	116.34	**116.40**
M002	5627.04	116.05	**116.22**	115.88	**116.10**
M002	5609.94	115.91	**116.02**	115.77	**115.91**
M002	5601	115.89	**116.01**	115.72	**115.86**
M002	5568.52	115.86	**115.98**	115.69	**115.82**
M002	5508.2	115.49	**115.66**	115.28	**115.53**
M002	5473.67	115.17	**115.33**	114.99	**115.22**
M002	5403.83	**112.10**	111.90	**111.89**	111.75
M002	5352.19	111.58	**111.67**	111.37	**111.55**
M002	5309.83	111.02	**111.10**	**111.22**	111.00
M002	5266.9	**111.00**	110.64	**110.91**	110.58
M002	5213.45	**110.55**	110.45	**110.33**	110.28
M002	5162.38	**110.39**	110.05	**110.17**	109.91
M002	5109.83	**109.81**	109.52	**109.63**	109.40
M002	5059.83	**109.55**	109.19	**109.38**	109.10
M002	5013.2	**109.25**	108.89	**109.10**	108.80
M002	4977.38	**108.98**	108.79	**108.80**	108.64
M002	4908.49	**108.88**	108.76	**108.66**	108.61
M002	4883.44	**108.84**	108.75	**108.62**	108.60
M002	4816.97	**108.80**	108.74	108.58	**108.58**
M002	4757.22	**108.79**	108.73	108.56	**108.57**
M002	4718.65	**108.78**	108.72	108.54	**108.56**
M002	4674.22	**108.75**	108.69	108.51	**108.53**
M002	4633.22	**108.73**	108.68	108.50	**108.52**
M002	4561.03	**108.66**	108.60	108.45	**108.45**
M002	4506.74	**108.36**	108.27	108.03	**108.15**
M002	4459.83	**107.59**	107.32	**107.28**	107.18
M002	4409.53	**106.54**	106.32	**106.32**	106.17
M002	4359.83	**105.64**	105.52	**105.36**	105.33
M002	4327.93	**105.15**	105.04	104.91	**104.92**
M002	4265.56	**104.45**	104.35	**104.16**	104.14
M002	4208.11	**103.56**	103.38	**103.28**	103.25
M002	4159.83	**102.41**	102.22	**102.30**	102.16
M002	4109.83	**100.80**	100.55	**100.43**	100.39
M002	4056	**99.28**	98.88	**98.91**	98.68
M002	4009.83	**98.48**	98.21	**98.30**	98.15
M002	3969.03	**97.70**	97.52	97.27	**97.28**
M002	3916.58	**97.19**	96.96	96.58	**96.89**
M002	3859.82	96.23	**96.28**	**96.06**	95.95
M002	3810.05	**95.82**	95.43	**95.90**	95.31

Reach	River Station	1:100 year flood level (m)		1:20 year flood level (m)	
		ice jam	open water	ice jam	open water
M002	3759.83	**95.24**	94.69	**95.10**	94.58
M002	3709.28	**94.64**	94.15	**94.50**	94.03
M002	3661.6	**94.22**	93.84	**94.09**	93.77
M002	3609.83	**93.86**	93.47	**93.71**	93.33
M002	3559.9	**93.33**	92.88	**93.14**	92.75
M002	3509.83	**92.86**	92.55	**92.40**	92.14
M002	3493.23	**92.73**	92.19	**92.21**	92.18
M002	3480 BR U	**92.74**	92.05	**92.21**	92.17
M002	3480 BR D	**92.74**	92.13	**92.22**	92.2
M002	3474.26	**92.74**	92.13	**92.21**	92.20
M002	3466.43	**92.74**	92.07	**92.15**	91.52
M002	3454.35	**92.67**	91.95	**92.12**	91.74
M002	3450 BR U	**92.67**	91.88	**92.12**	91.72
M002	3450 BR D	**92.67**	91.84	**92.12**	91.68
M002	3434.16	**92.63**	91.67	**92.10**	91.63
M002	3413.11	**92.03**	91.65	**91.83**	91.55
M002	3391.58	**91.85**	91.56	**91.66**	91.45
M002	3371.12	**91.69**	91.34	**91.53**	91.17
M002	3309.87	**91.04**	90.73	**90.88**	90.61
M002	3272.96	**90.73**	90.29	**90.56**	90.19
M002	3219.09	**90.19**	89.79	**90.02**	89.71
M002	3175.54	89.57	89.57	**89.51**	89.36
M002	3165 BR U	**89.59**	89.32	**89.53**	89.27
M002	3165 BR D	**89.59**	89.32	**89.53**	89.26
M002	3157.85	**89.54**	89.25	**89.50**	89.08
M002	3135.32	**89.43**	89.14	**89.32**	89.06
M002	3106.53	**89.19**	88.82	**89.08**	88.78
M002	3088.94	**88.78**	88.59	**88.67**	88.58
M002	3080 BR U	**88.76**	88.57	**88.60**	88.17
M002	3080 BR D	**88.77**	88.43	**88.60**	88.32
M002	3075.67	**88.75**	88.37	**88.59**	88.27
M002	3056.89	**88.65**	88.30	**88.47**	88.19
M002	3007.54	**88.30**	87.99	**88.15**	87.91
M002	2976.31	**88.19**	87.88	**88.09**	87.83
M002	2941.08	**88.07**	87.77	**88.04**	87.76
M002	2920.1	**88.03**	87.75	**88.03**	87.75
M002	2915 BR U	**88.03**	87.75	**88.03**	87.75
M002	2915 BR D	**88.03**	87.75	**88.03**	87.75
M002	2912.75	**88.03**	87.75	**88.03**	87.75
M002	2890.23	**87.50**	86.86	**87.29**	86.81
M002	2823.99	**87.37**	87.13	**87.15**	87.03
M002	2789.49	**87.33**	87.11	**87.12**	87.00
M002	2741.3	**87.20**	87.05	**87.05**	86.95
M002	2663.79	**87.14**	87.01	**86.99**	86.90
M002	2639.38	**87.10**	86.97	**86.94**	86.86
M002	2616.24	**87.09**	86.96	**86.92**	86.84

Reach	River Station	1:100 year flood level (m)		1:20 year flood level (m)	
		ice jam	open water	ice jam	open water
M002	2539.69	**87.03**	86.91	**86.85**	86.79
M002	2489.06	**86.98**	86.87	**86.78**	86.74
M002	2438.28	**86.91**	86.81	**86.69**	86.68
M002	2384.01	**86.82**	86.71	**86.60**	86.59
M002	2310.47	**86.54**	86.48	86.30	**86.32**
M002	2284.63	**86.46**	86.43	86.18	**86.26**
M002	2240.58	**86.06**	85.98	85.74	**85.86**
M002	2170.94	**84.33**	84.05	**83.98**	83.80
M002	2136.89	**83.75**	83.44	**83.48**	83.31
M002	2100.01	**83.35**	83.25	83.08	**83.14**
M002	2046.85	82.74	82.74	**82.98**	82.50
M002	2014.35	**82.34**	81.96	**82.58**	81.88
M002	1990.76	**82.02**	81.94	**82.21**	81.82
M002	1945.2	**81.76**	81.63	**81.73**	81.44
M002	1916.67	**81.71**	81.47	**81.56**	81.30
M002	1871.48	**81.68**	81.32	**81.39**	81.16
M002	1836.03	**81.52**	81.27	**81.24**	81.11
M002	1788.63	**81.13**	80.85	**80.84**	80.51
M002	1768.08	**80.86**	80.12	**80.59**	79.94
M002	1733.19	**80.47**	79.99	**80.29**	79.86
M002	1694.89	**79.80**	79.38	**79.44**	79.15
M002	1685.79	**79.75**	79.32	**79.39**	79.09
M002	1640.51	**79.18**	78.81	**78.92**	78.58
M002	1602.99	**77.49**	76.52	**77.20**	76.35
M002	1585.52	**76.68**	75.35	**76.34**	75.26
M002	1569.47	**76.19**	75.03	**75.92**	74.96
M002	1541.97	**75.17**	74.16	**74.82**	73.97
M002	1510.83	**74.06**	72.57	**73.79**	72.48
M002	1495.27	**73.70**	73.06	**73.37**	72.90
M002	1458.62	**73.08**	72.68	**72.77**	72.56
M002	1419.61	**72.82**	72.06	**72.50**	71.86
M002	1384.56	**72.43**	71.92	**72.20**	71.75
M002	1351.51	**72.13**	71.67	**71.95**	71.64
M002	1311.78	**71.74**	71.41	**71.61**	71.29
M002	1291.19	**71.53**	71.02	**71.38**	70.91
M002	1217.75	**71.38**	70.99	**70.99**	70.71
M002	1200.39	**71.18**	70.63	**70.79**	70.50
M002	1190 BR U	**71.22**	70.51	**70.82**	70.47
M002	1190 BR D	**71.22**	70.54	**70.83**	70.49
M002	1185.68	**71.07**	70.52	**70.77**	70.48
M002	1164.32	**70.83**	70.45	**70.65**	70.33
M002	1132.6	**70.31**	69.84	**70.18**	69.77
M002	1111.68	**70.18**	69.87	**70.00**	69.75
M002	1059.95	**70.02**	69.69	**69.83**	69.60
M002	998.99	**69.67**	69.57	**69.52**	69.22
M002	955.54	69.43	**69.57**	69.19	**69.21**

Reach	River Station	1:100 year flood level (m)		1:20 year flood level (m)	
		ice jam	open water	ice jam	open water
M002	851.16	69.39	**69.56**	68.96	**69.19**
M002	798.06	69.39	**69.56**	68.90	**69.19**
M002	746.77	69.39	**69.55**	68.89	**69.19**
M002	694.52	69.39	**69.55**	68.89	**69.19**
M002	669.16	69.39	**69.55**	68.89	**69.19**
M002	590.19	69.39	**69.55**	68.89	**69.19**
M002	539.85	69.38	**69.55**	68.88	**69.19**
M002	488.4	69.38	**69.55**	68.88	**69.19**

NOTES:

1. The bold numbers indicate the higher levels between ice jam and open water conditions.

5.3 Conclusions and Recommendations

5.3.1 Conclusions

A hydraulic model based on the USACE program HEC-RAS was developed for reaches of the Petty Harbour River covering a linear distance of approximately 31.6km (with 729 cross-sections).

The model was developed based on field surveyed bathymetric data and a LiDAR survey conducted in November and December of 2011. Field survey of water levels specifically to form a database upon which the hydraulic model could be calibrated/validated was not completed due to late season project start and freeze up of the waterways in the study area. As such, the hydraulic model has not been calibrated/validated, however, due care was taken during model development to accurately establish model parameterization.

The hydraulic model developed for this study was also used to evaluate the potential flood conditions (i.e. resultant water levels) associated with ice jamming events. The evaluation along Cochrane Pond Brook and Raymond Brook confirmed that along limited reaches of the watercourse, computed water levels associated with ice jams have the potential to generate water levels exceeding 1:100 year AEP open water event levels.

The hydrologic and hydraulic models developed for this study and relevant support data are included with the Project CD materials attached to this report. The models may be used in the future to evaluate the impact on water levels resulting from any structural changes to the subject watercourses, structures, or floodplain / overbank areas.

5.3.2 Recommendations

It is recommended that the City of St. John's engage in a field-based program to measure water levels at designated structures within the subject watershed during flood events. This data gathering effort will provide a basis for future calibration/validation of the models developed for this study.

It is recommended that a program focused on unregulated streamflow data collection be developed for unregulated tributaries of Petty Harbour River. The only hydrometric stations presently recording streamflow in the Petty Harbour River watershed are located downstream of dams. Additional recording stations at strategic locations (e.g., large unregulated tributary areas) would provide a foundation of data that would enhance the hydrologic model calibration/validation process.

In concert with the implementation of streamflow data collection, a program focused on field-based collection of ice thickness/accumulation data should be implemented in areas identified as ice jam prone. It was noted previously that no ice thickness data was available for the study area. A database of ice thickness/accumulation data would enhance and provide additional confidence the ice modelling process and results.

It is recommended that the water levels for existing conditions for the 1:20 year and 1:100 year AEP water surface profiles as defined on the flood plain maps and provided in tabular form in Appendix J, be adopted for regulatory and management purposes.

It is recommended that special consideration be given to higher water levels (than those based on the 1:100 year AEP flow) associated with ice jam conditions. A consideration may be to designate the "ice jam" flood inundated area as a special policy area which will allow the City of St. John's to enact specific policies/guidelines regarding development while recognizing the low expectation of ice jam influenced flooding.

It is recommended that HEC-GeoRAS and HEC-RAS be used in future watershed and flood studies as it both simplifies the development of deterministic models as well as provides for the generation of a significant warehouse of information that can be used for several ancillary purposes beyond hydraulic assessment.

6.0 SENSITIVITY ANALYSIS

The hydrologic and hydraulic models were developed based on a review of available data and selection of appropriate input data. However, as is the case in all numerical modelling of physical processes, there is the inherent potential for errors or uncertainty to be associated with the selection of input variables which could affect the resulting flood flows and subsequent computation of associated water levels. Sensitivity analysis can, hence, be useful for a range of purposes, including:

- Testing the robustness of simulation model results in the presence of uncertainty.
- Increasing the understanding of the relationships between input and output variables in the simulation models.
- Increasing confidence in simulation model results by identifying model inputs that cause significant uncertainty in the output. Increased attention to these specific model inputs can then be applied to ensure proper definition and/or parameterization.
- Ensuring the model is accurately reflecting watershed conditions and responses by identifying errors in the model output as reflected by unexpected relationships between inputs and outputs.

A sensitivity analysis of the hydrologic and hydraulic model inputs was completed to determine the effects of changing model parameters on the resulting flood flows and levels. This analysis was completed using the EC IDF relationship as the basis for rainfall (as this aspect of the project was completed during model development), however, it would be expected that the results would be comparable for the CBCL IDF relationship results also. The results of the sensitivity analyses are summarized below.

6.1 Sensitivity to Hydrologic Model Inputs

6.1.1 SCS Curve Numbers

As previously described, a SCS Curve Number is required for each sub-basin within the hydrologic model. The Curve Number for a particular sub-basin is a function of soil type, land use, and antecedent runoff conditions. The Curve Number defines the amount of runoff and infiltration based on a given rainfall amount. The Curve Numbers for each sub-basin within the HEC-HMS model were increased and decreased by 10 percent for the 1:20 year and 1:100 year AEP events. The results of this analysis are presented in Table 6-1. As suggested by the results, the generated peak flows are very sensitive to the selection of an appropriate Curve Number; as demonstrated by a 10 percent change in Curve Number resulting in a change in peak flow of 20 to 25 percent.

Given this result, the input variables, associated with generation of the Curve Number grid (soils and land use), developed for the HEC-HMS model were reviewed. This review confirmed that the soils information used for model development was the best currently available; sourced from the Government of Canada. The land use data was based on the land classification project completed for this project. This assessment was based on 10-meter resolution SPOT imagery. With coarser resolution, spectral mixing exists meaning some pixels contain a mixture of different features and cover types, compared to higher resolution images where individual pixel values represent more homogenous materials. The overall impact of the satellite imagery resolution on land use classification is difficult to quantify. Impacts in sub-catchment where the predominant land forest is forest (which represents a significant portion of the watershed) would not be expected to be significant. However, a greater degree of impact may be anticipated in urban areas where the 10 m resolution may not adequately capture impervious areas, resulting in potentially lower Curve Numbers, potentially leading to under-estimation of runoff.

Table 6-1: SCS Curve Number Sensitivity Analysis

Study Area	Event (AEP) (yr)	Base Case Flow (m^3/s)	Curve Number +10%		Curve Number -10%	
			Flow (m^3/s)	% Difference	Flow (m^3/s)	% Difference
Petty Harbour River	1:20	57.4	71.8	25.0%	46.1	-19.7%
	1:100	89.6	109.8	18.4%	73.0	-18.5%

6.1.2 River Reach Roughness

The river reach roughness is an input into the hydrologic model which is used to determine the shape of the resulting hydrograph through the effect of channel routing from one basin to the next downstream computational node. The Manning’s Roughness coefficients were increased and decreased by 10 percent for the 1:20 year and 1:100 year AEP events. The results of the analysis are presented in Table 6-2. As suggested by the results, the selection of the river reach roughness coefficient does not have a significant impact on the resulting peak flows.

Table 6-2: River Reach Roughness Sensitivity Analysis

Study Area	Event (AEP)	Base Case Flow (m^3/s)	Manning Coefficient +10%		Manning Coefficient -10%	
			Flow (m^3/s)	% Difference	Flow (m^3/s)	% Difference
Petty Harbour River	1:20	57.4	57.3	-0.1%	57.5	0.1%
	1:100	89.6	89.5	-0.1%	89.7	0.1%

6.1.3 IDF Estimate Uncertainty

The 1:100 year AEP rainfall events that were simulated in the hydrologic model were taken directly from the EC Short Duration Rainfall Intensity-Duration-Frequency Data, published April 13, 2010 for St. John’s Airport. The 1:20 year AEP rainfall was estimated from the EC IDF data and as such, confidence limits were not available. The 95% Confidence limits estimates provided with the rainfall intensity data were used to establish upper and lower bounding 1:100 year AEP rainfall hyetographs (see Table 6-3) developed using the Alternating Block method.

Table 6-3: Sensitivity Analysis of IDF Rainfall

Duration	***1:100 year AEP Rainfall Intensity (mm/hr)***		
	Lower Bound	**EC IDF Base**	**Upper Bound**
5 min	103.3	133.9	164.5
10 min	73.5	94.2	114.9
15 min	62.0	79.7	97.4
30 min	42.8	54.4	66.0
1 hr	28.5	35.5	42.5
2 hr	20.8	26.5	32.2
6 hr	10.6	13.1	15.6
12 hr	6.5	7.9	9.3
24 hr	3.8	4.6	5.4

Table 6-4 summarizes the results of the impact of varied rainfall inputs on computed peak flows. As suggested from the results, the model is sensitive to rainfall input within the confidence limits specified by EC in an amount equal to the change in rainfall on a percentage basis. Although confidence limits were not available for the 20 year AEP rainfall, a similar result would be expected.

Table 6-4: Results of Rainfall Sensitivity Analysis

Watershed	**1:100 year AEP Flow (m^3/s)** ***% change from base estimate***		
	Lower Bound	**Base**	**Upper Bound**
Petty Harbour River	64.0	89.6	118.5
	-29%	*0*	*32%*

The climate change analysis provided in Section 7 provides additional information outlining the sensitivity of peak flow estimates to additional variations in precipitation input.

6.1.4 Summary of Hydrologic Model Sensitivity

A sensitivity analysis of the hydrologic model inputs was completed to determine the effects of changing model parameters on the resulting flood flows. It was determined that peak flows are very sensitive to the selection of Curve Number but are not sensitive to changes in river reach roughness estimates. It was also determined that the hydrologic model is sensitive to variations in rainfall inputs within the confidence limits specified by EC.

It was noted that better estimates of Curve Number may be possible with the use of higher resolution satellite imagery to support the classification of land cover in the watersheds. This should be a consideration for future watershed modeling efforts.

6.2 Sensitivity of Hydraulic Model Inputs

6.2.1 Manning's Roughness

The Manning's Roughness input parameter of the hydraulic model defines the relative roughness of the main channel and floodplain areas. A higher Manning's Roughness coefficient will increase flooding levels and reduce velocities. The Manning's Roughness for the channel and overbank at each cross section were increased and decreased by 20 percent for the 1:20 year and 1:100 year AEP events. The results of the analysis are presented in Table 6-5.

The selection of Manning's Roughness coefficient generally has a limited overall impact. However, significant impacts in localized reaches is demonstrated through this analysis where changes in flow regime occur as a result of roughness variation (i.e. from supercritical to subcritical or vice-versa). Large changes in water surface can also occur in cross-sections near (typically upstream) critical culvert and bridge locations where flow changes from open surface flow to surcharged or overtopping situations. The analysis has demonstrated that altering of Manning's Roughness coefficient by 20% (positive or negative) results in an average changes in computed water surface elevation of between 1 cm to 7 cm.

Table 6-5: Manning's Roughness Sensitivity Analysis

Study Area	Event (AEP) (yr)	Manning's n + 20%			Manning's n - 20%		
		Average Change in WL[1] (m)	Maximum Increase in WL (m)	Maximum Decrease in WL (m)	Average Change in WL (m)	Maximum Increase in WL (m)	Maximum Decrease in WL (m)
Petty Harbour River	1:20	+ 0.01	+ 0.55	- 0.05	- 0.06	+ 0.09	- 1.83
	1:100	+ 0.06	+ 1.06	- 0.27	- 0.07	+ 0.13	- 0.95

1. Water Level

6.2.2 Peak Discharge

To determine the impact of the changes in peak flows on the resulting water surface profile, the peak flows for the 1:20 year and 1:100 year AEP events were increased/ decreased by 10, 20, and 30 percent. Tables 6-6, 6-7, and 6-8 summarize the changes in water levels for the 1:20 year and 1:100 year AEP events associated with the varying peak flow conditions.

As for Manning's Roughness, the selection of peak discharge generally has a limited impact on average (<0.2 m). However, significant impacts in localized reaches is demonstrated through this analysis where changes in flow regime occur (i.e. from supercritical to subcritical or vice-versa). Large changes in water surface can also occur in cross-sections near (typically upstream) critical culvert and bridge locations where flow changes from open surface flow to surcharged or overtopping situations.

Table 6-6: Peak Discharge Sensitivity Analysis (+/- 10%)

Study Area	Event (AEP) (yr)	Inflow + 10%			Inflow - 10%		
		Average Change in WL (m)	Maximum Increase in WL (m)	Maximum Decrease in WL (m)	Average Change in WL (m)	Maximum Increase in WL (m)	Maximum Decrease in WL (m)
Petty Harbour River	1:20	+ 0.05	+ 0.34	- 0.06	- 0.05	+ 0.47	- 0.72
	1:100	+ 0.05	+ 0.09	- 0.32	- 0.06	+ 0.35	- 0.47

Table 6-7: Peak Discharge Sensitivity Analysis (+/- 20%)

Study Area	Event (AEP) (yr)	Inflow + 20%			Inflow - 20%		
		Average Change in WL (m)	Maximum Increase in WL (m)	Maximum Decrease in WL (m)	Average Change in WL (m)	Maximum Increase in WL (m)	Maximum Decrease in WL (m)
Petty Harbour River	1:20	+ 0.09	+ 0.64	- 0.45	- 0.10	+ 2.22	- 1.93
	1:100	+ 0.11	+ 1.36	- 0.25	- 0.11	+ 0.56	- 0.78

Table 6-8: Peak Discharge Sensitivity Analysis (+/- 30%)

Study Area	Event (AEP) (yr)	Inflow + 30%			Inflow - 30%		
		Average Change in WL (m)	Maximum Increase in WL (m)	Maximum Decrease in WL (m)	Average Change in WL (m)	Maximum Increase in WL (m)	Maximum Decrease in WL (m)
Petty Harbour River	1:20	+ 0.13	+ 0.86	- 0.43	- 0.16	+ 1.32	- 2.03
	1:100	+ 0.15	+ 1.54	- 0.16	- 0.18	+ 0.45	- 1.01

6.2.3 Tidal and Surge Influence

The downstream boundary condition was assumed to be a water level of 2.55 m for the hydraulic model (for existing conditions). This water level is comprised of the maximum high tide and storm surge as previously documented in Section 5.1.6. The downstream boundary condition was increased by 1.0 m for the 1:20 year and 1:100 year AEP events (3.55 m total for Petty Harbour River. The results of the analysis are presented in Table 6-9.

The resulting increase in water level is consistent with the incremental increase in the downstream boundary condition of 1.0 m. The maximum increase in water level is 1.09 m. In all results, the impact of the increase in the downstream boundary condition is relatively localized. The changes in computed water surface elevations were limited to areas below cross-section 577.97 (approximately 90 m upstream of the Main Road Bridge).

Table 6-10 details a comparative assessment of computed water surface elevations for existing conditions based on three scenarios, namely:

- the starting water surface elevation described in Section 5.1.6 (HHWLT plus storm surge = 2.55m) and adopted for this study,
- a starting water surface elevation based on HHWMT plus storm surge of 2.25m, and;
- a starting water surface elevation based on MWL plus storm surge of 1.75m

For both the 1:20 year and 1:100 year AEP floods, the change in starting water level does not influence computed water levels a significant distance upstream. The influence extends only to about section 577.97 (no change in computed water levels) for the 1:20 year and 1:100 year AEP scenarios.

Table 6-9: Starting Water Level Sensitivity Analysis (+ 1 m)

Study Area	Event (AEP)	Starting Water Surface Elevation + 1 m		
		Average Change in WL (m)	Maximum Increase in WL (m)	Maximum Decrease in WL (m)
Petty Harbour River	1:20	+ 0.02	+ 1.09	0.00
	1:100	+ 0.02	+ 1.03	0.00

Table 6-10: Starting Water Level Sensitivity Analysis (Boundary Water Levels)

Reach	Section	Starting Water Levels					
		20 Year AEP Flood			100 Year AEP Flood		
		1.7m	2.25m	2.55m	1.7m	2.25m	2.55m
M001	12.04	1.75	2.25	2.55	1.75	2.25	2.55
M001	20.04	1.77	2.27	2.56	1.8	2.28	2.57
M001	91.29	1.77	2.27	2.56	1.8	2.28	2.57
M001	105.74	1.77	2.27	2.56	1.8	2.28	2.57
M001	150.84	1.77	2.27	2.56	1.8	2.28	2.57
M001	201.25	1.77	2.27	2.56	1.8	2.28	2.57
M001	238.18	1.77	2.26	2.56	1.78	2.27	2.56
M001	269.29	1.77	2.27	2.56	1.8	2.28	2.57
M001	305.38	1.78	2.27	2.56	1.8	2.29	2.57
M001	338.85	1.78	2.27	2.56	1.81	2.29	2.58
M001	367.8	1.78	2.27	2.56	1.81	2.29	2.58
M001	405.68	1.77	2.27	2.56	1.78	2.28	2.57
M001	439.27	1.78	2.27	2.57	1.81	2.29	2.58
M001	478.84	1.75	2.24	2.54	1.75	2.24	2.54
M001	485	Bridge					
M001	491.94	2.15	2.07	2.47	2.91	2.91	2.41
M001	525.57	2.6	2.57	2.76	3.15	3.15	3.08
M001	577.97	3.69	3.69	3.69	3.79	3.79	3.79
M001	622.83	5.13	5.13	5.13	5.30	5.30	5.30
M001	633.61	5.33	5.33	5.33	5.43	5.43	5.43
M001	652.51	6.02	6.02	6.02	6.32	6.32	6.32

6.2.4 Summary of Hydraulic Model Sensitivity

Average changes in computed water levels resulting from the sensitivity runs were close to base case results. More significant changes in computed water levels were attributed to changes in flow regime (i.e. from supercritical to subcritical or vice-versa) or changes in flow conditions around bridges and culverts (i.e. changes from open surface flow to surcharged or overtopping situations).

Standard HEC-RAS output tables, associated with hydraulic computations detailed for the hydraulic model sensitivity analysis, are provided in Appendix N.

6.3 Sensitivity Analysis Conclusions

As noted previously, sensitivity analysis is used to:

- *Test the robustness of simulation model results in the presence of uncertainty and increasing the understanding of the relationships between input and output variables in the simulation models.*

  Three input variables were tested with the following results:

  - Sensitive to changes in Curve Number,
  - Not sensitivity to river reach roughness, and;
  - Sensitivity to rainfall estimates within the confidence limits specified by Environment Canada.

  Some benefit may be gained regarding improved confidence in Curve Number estimation through the use of higher resolution satellite imagery for land classification. However, the difference between the two methods (i.e. use of low or high resolution data) in terms of Curve Number estimation cannot be quantified without parallel assessments.

- *Increasing confidence in simulation model results by identifying model inputs that cause significant uncertainty in the output thereby focusing increased attention towards estimation of these specific model inputs.*

  The sensitivity analysis results associated with river reach roughness and rainfall estimates did not justify any additional effort towards refining initial model estimates for these parameters.

- *Ensuring the model is accurately reflecting watershed conditions and responses by identifying errors in the model output as reflected by unexpected relationships between inputs and outputs.*

  The sensitivity analysis results did not demonstrate any unexpected relationships or model errors.

Overall, the hydrologic model input parameters were selected based on reliable background information, engineering judgment and field measured data and are considered to be a good and supportable reflection of watershed conditions. The sensitivity analysis results of the hydrologic models did suggest opportunities for future potential enhancement with regard to Curve Number estimation but, overall, did not suggest a need to alter the parameterization of the hydrologic models for the present study.

The sensitivity analysis results associated with the hydraulic model indicate a general insensitivity to changes in input parameters when viewed as average changes to computed water surface elevations. Some specific locations do experience larger variation in computed water levels but these are associated with changes in the flow regime between sub-critical flow and super-critical flow (and vice versa) and changes in bridge hydraulics associated with open water to pressure flow situations (and vice versa).

The sensitivity analysis results of the hydraulic models did not suggest a need to alter the parameterization of the hydraulic models for the present study.

6.4 Sensitivity Analysis Recommendations

It was noted that better estimates of Curve Number may be possible with the use of higher resolution satellite imagery to support the classification of land cover in the watersheds. This should be a consideration for future watershed modeling efforts.

7.0 CLIMATE CHANGE ANALYSIS

Newfoundland and Labrador is expected to experience changes in temperature, precipitation, sea level and other factors in the future as a result of climate change. These factors can influence the flood risk faced by a community directly or indirectly. Climate change may result in communities which are not presently at risk of flooding being included in the list of potential candidates for new flood plain mapping.

The climate change assessment for this project focused on the development of flood plain mapping for three future periods, namely: 2020, 2050 and 2080. It should be noted that the previously noted periods are not meant to represent exactly these years but the more general time frames of today through to 2035, 2036 to 2065, 2066 to 2095.

The HEC-HMS model of Petty Harbour River, developed for this project, was used to assess the impact of climate change by using projected rainfall data for the target periods. It can be argued that other parameters are also relevant in this analysis such as continued urban development and change of land cover.

Population statistics available through Newfoundland and Labrador Statistics Agency[13] were reviewed as an indication of potential future population growth. The available census data for St. John’s (Goulds and Petty Harbour data was not available separately) is outlined in Table 7-1. The data suggest that the population of St. John’s has not changed substantially over the past 20 years.

Table 7-1: Population Data for St. John’s

Population by Year				
1991	**1996**	**2001**	**2006**	**2011**
104,659	101,936	99,182	100,646	106,172

Watershed runoff response will also be influenced by changes in land cover that may result from community development (increased imperviousness as a result of new roads, buildings, paved areas, etc.) or changes in terrestrial communities (such as a forest changing to an open meadows or vice versa). The land cover analysis completed as a component of this project was focused on one time period only. A land cover change detection analysis of at least two periods, if not more, would be required to determine if any trends in changes in land cover over the watershed were identifiable.

Broader changes in land cover as a result of changing terrestrial communities due to climate change are addressed in Vasseur and Catto (2008). However, the sensitivity and vulnerability of forest communities in Atlantic Canada is considered to be low to moderate. Further, given that the Vasseur and Catto (2008) assessment of climate change influences on forest systems provided no specific guidance on regional variation of potential impacts across the Province,

[13] http://www.stats.gov.nl.ca/

there was not any means making projections regarding hydrologic model parameterization for future periods to reflect potential land cover changes.

The review of potential changes in population and land cover provided no definitive guidance towards alteration of the HEC-HMS to reflect future watershed characteristics. As such, the existing conditions HEC-HMS model was used for this assessment.

The estimates of future rainfall data were taken from three separate sources as outlined below:

- AMEC - Development of Projected Intensity-Duration-Frequency Curves

  In 2010, Environment Canada developed updated IDF curves based on historical observations from the stations at St. John's Airport (last rainfall data 1996). The documentation for this historical IDF curve included the record of the intensity of annual extreme precipitation events for nine event durations ranging from five minutes to 24 hours. To obtain projected IDF curves, the precipitation intensities in the historical IDF curve were adjusted to reflect projected changes in climate using a statistical modeling technique that is described briefly in the following paragraphs. A detailed report outlining the techniques used and outcomes from this analysis is provided in Appendix D.

  The approach selected for this analysis uses a statistical model that derives the sensitivity of extreme precipitation to climate conditions from the historical climate information for a site. In this case the historical climate was characterized by observations of monthly average temperature and monthly total precipitation at the St. John's Airport weather station. The statistical model was fitted to the local climate data and the historical monthly precipitation maxima using a form of regression. Information about future monthly average temperature and monthly total precipitation was obtained from the output of 48 runs of Global Climate Models (GCMs). Each GCM run was compared to establish a projected future change in temperature and precipitation. These changes were used to adjust the historical record of temperature and precipitation to reflect future conditions, which resulted in 48 future climate scenarios that were based on the historical record but which reflected the projected future change in climate. This approach, which is referred to as the delta approach, is used to reduce some of the inevitable bias inherent in projections of future climate.

  The statistical model of extreme precipitation was then run against each of these adjusted records to obtain estimates of climate-impacted extreme precipitation intensities for each of the nine durations and six return intervals. These estimates reflect the bias in the statistical model, so one more run of the statistical model was made against the average historical climate conditions to provide a baseline set of extreme precipitation intensities and this set of baseline intensities was compared against each of the 48 estimates of climate-impacted intensities to determine the change in intensity attributable to the change in climate. These changes were then used to adjust the values in the historical IDF curve to obtain the final projected values of precipitation intensity. (This is another application of the delta approach.)

The 48 projections used to characterize future climate conditions produced an equal number of estimates of projected precipitation intensities for each duration and return interval. For reporting purposes, these results were aggregated into the mean, maximum and 90th percentile non-exceedence value of precipitation intensity for each duration and return interval.

WRMD also provided AMEC with a revised estimate of the IDF curve for St. John's as computed by CBCL (see Section 2 of this report). The development of projected IDF curves for the future periods was based on CBCL IDF relationship for St. John's Airport.

The estimates of projected rainfall, for the St. John's Airport station, determined through this assessment are presented in Table 7-2 and Appendix D.

- Joel Finnis, Associate Professor, Department of Geography, Memorial University, 2012

As described by Dr. Finnis:

"The estimates were extrapolated from available observations and climate simulations from the North American Regional Climate Change Assessment Project (NARCCAP). To ensure an appropriate baseline, probability distributions were first fitted to Environment Canada IDF curves. Projected changes in the distribution parameters were then calculated from the model data; these changes were then applied to the observed distributions, giving an estimated distribution for the mid-21st century. A mixed probability model was used, in which the probability of daily precipitation was first calculated, and a gamma distribution was then fitted to daily precipitation amounts for days in which precipitation occurs. The model uses three parameters; the probability of no precipitation, and the gamma shape and scale parameters.

The projections are for 12 hour and 24 hour return periods for the mid-21st century ~2040-2060), using the official Environment Canada numbers as a baseline. There are two predictions: a 'fitted' value, which applies projected changes in the precipitation distribution to the 20th century baseline, and a 'raw model' value, which just applies the un-adjusted model projected change to the baseline. The fitted is a better assessment, and better accounts for model biases. The raw model is less useful, but could be taken as a low-end estimate of change."

The Finnis projected rainfall estimates (provided for 2050 only) were provided (see Table 7-3) as event totals only (12 hour and 24 hour only). As such, hyetographs for the purposes of HEC-HMS modeling were generated using the alternating block method using the 2050 projected IDF data (produced by AMEC) and applying the resultant mass rainfall curve to the Finnis data.

Dr. Finnis also revised initial future rainfall estimates based on the updated CBCL IDF relationship for St. John's at the request of WRMD.

Table 7-2: AMEC Projected Rainfall Estimates for St. John's Airport

		Rainfall Totals (mm) - Maximum, 2020 timeframe					
		Return period (years)					
		2	5	10	20	50	100
Storm Duration	5 min	4.8	6.4	7.4	8.3	9.6	10.6
	10 min	7.6	9.7	11.0	12.4	14.2	15.4
	15 min	9.8	12.5	14.3	15.9	18.2	19.9
	30 min	14.8	18.9	21.5	24.1	27.3	29.9
	1 hr	21.7	27.7	31.6	35.3	40.1	43.6
	2 hr	29.8	39.2	45.2	51.1	58.5	64.1
	6 hr	49.6	62.8	71.9	80.3	91.1	99.5
	12 hr	63.8	80.8	93.0	103.9	118.4	129.3
	24 hr	76.7	96.2	108.3	120.4	134.9	147.0

		Rainfall Totals (mm) - Maximum, 2050 timeframe					
		Return period (years)					
		2	5	10	20	50	100
Storm Duration	5 min	5.0	6.5	7.5	8.5	9.7	10.6
	10 min	7.9	9.9	11.3	12.6	14.4	15.6
	15 min	10.2	12.9	14.6	16.3	18.5	20.2
	30 min	15.6	19.6	22.2	24.8	28.0	30.5
	1 hr	23.1	29.1	32.9	36.6	41.4	44.9
	2 hr	31.7	41.1	47.1	53.0	60.4	66.0
	6 hr	52.4	65.7	74.7	83.2	94.0	102.5
	12 hr	67.9	85.0	97.2	108.2	122.8	133.7
	24 hr	82.0	101.7	113.8	126.0	140.4	152.5

		Rainfall Totals (mm) - Maximum, 2080 timeframe					
		Return period (years)					
		2	5	10	20	50	100
Storm Duration	5 min	5.2	6.7	7.7	8.6	9.9	10.8
	10 min	7.9	10.0	11.3	12.7	14.4	15.7
	15 min	10.2	12.9	14.7	16.3	18.6	20.2
	30 min	15.7	19.7	22.3	24.9	28.1	30.6
	1 hr	23.2	29.2	33.0	36.8	41.5	45.0
	2 hr	31.9	41.3	47.4	53.2	60.6	66.3
	6 hr	56.0	69.3	78.4	86.9	97.7	106.2
	12 hr	73.1	90.2	102.6	113.6	128.3	139.4
	24 hr	88.6	108.5	120.7	132.9	147.4	159.6

Table 7-3: Finnis 2050 Total Rainfall Estimates (mm) for St. John's

Location	Return Period	2050 Predictions	
		24 hour event	12 hour event
St. John's	2 yr	77.6	67.2
	5 yr	94.2	82.5
	10 yr	107.1	94.3
	20 yr	120.0	106.3
	50 yr	137.4	122.4
	100 yr	150.6	134.7

- Climate Change Scenarios for Atlantic Canada Utilizing a Statistical Downscaling Model Based on Two Global Climate Models, Gary S. Lines, Michael Pancura, Chris Lander, Lee Titus, Meteorological Service Of Canada, Atlantic Region, Science Report Series 2009-01, July 2008

  The project Terms of Reference required the use of the estimates outlined in the report above as one of the climate change scenarios to be evaluated for the purposes of determining flood plains in the subject watersheds. It was subsequently deemed by WRMD that the projected rainfall estimates determined by Lines et al (2008) were inappropriate for use by this project and assessment should continue using the AMEC and Finnis projected rainfall estimates only.

7.1 Hydrologic Summary

As noted previously, the existing conditions HEC-HMS model for Petty Harbour River was used to determine peak flows for the three future periods, namely: 2020, 2050 and 2080, based on rainfall estimates for these future periods as determined by AMEC and Dr. Joel Finnis. Table 7-4 provides a summary of the calculated flows.

Table 7-4: Streamflow Summary for Existing and Future Conditions

Scenario	Streamflow (m^3/s)	
	Petty Harbour River (at the outlet)	
	1:20 AEP	1:100 AEP
Existing Conditions	70.1	112.2
2020 (AMEC)	85.3	130.3
2050 (Finnis)	84.6	136.8
2050 (AMEC)	94.4	140.2
2080 (AMEC)	103.9	152.1

7.2 Hydraulic Summary

The flows determined for the future periods were then input to the HEC-RAS hydraulic model to evaluate the potential impact of climate change on computed water levels in the study reaches. Table 7-5 provides a summary of the changes in computed water surface elevations (from existing conditions) associated with each of the future conditions.

Table 7-5: Comparison of Existing and Future Computed Water Surface Elevations

Scenario	Changes in Computed Water Surface Elevation from Existing Conditions (m)	
	Average Change	Maximum/Minimum Change
1:20 year AEP Flood		
2020 (AMEC)	0.19	1.02 / -0.44
2050 (AMEC)	0.23	1.19 / -0.53
2050 (Finnis)	0.13	0.86 / -0.44
2080 (AMEC)	0.27	1.45 / -0.58
1:100 year AEP Flood		
2020 (AMEC)	0.22	1.45 / -1.04
2050 (AMEC)	0.26	1.59 / -0.97
2050 (Finnis)	0.17	1.19 / -1.16
2080 (AMEC)	0.29	2.05 / -0.89

The maximum changes in computed water surface elevations are typically experienced on the upstream side of culverts or where the flow regime changes from super-critical to sub-critical (and vice versa).

Table 7-6 provides a comparison between the 2050 computed water surface elevations associated with the Finnis and AMEC rainfall estimates.

The maximum changes in computed water surface elevations remain consistent to those described for Table 7-5.

A full listing of the HEC-RAS results for the future periods (and existing) is provided in Appendix J).

Table 7-6: Comparison of 2050 Computed Water Surface Elevations

Statistic	1:20 year AEP Flood	1:100 year AEP Flood
Average	0.10	0.09
Maximum	0.58	1.46
Minimum	-0.33	-0.85

7.3 Conclusions and Recommendations

An evaluation of the potential impacts of climate change on flood risk was completed. Estimates of flood plains for the periods 2020, 2050 and 2080 were computed and delineated. Two sources of rainfall estimates for these future periods were determined. Dr. Joel Finnis, an Associate Professor in the Department of Geography at Memorial University provided one set of estimates (12 hour and 24 hour durations) for St. John's. AMEC, as a component of the current project, developed projected IDF relationships for St. John's based on the CBCL IDF relationship.

It should be noted that there is a great deal of uncertainty with all climate models, statistical downscaling and projection of rainfall to point locations. The quantification of rainfall and, subsequently, flood plain estimates should not be interpreted as an accurate portrayal of possible future events. These estimates provide a good indication of upward and downward trends and general sense of the magnitude of the potential change but should not be considered absolute.

7.3.1 Conclusions

It is concluded from this assessment that climate change has the potential to increase flood risk in the Goulds and Petty Harbour Area.

7.3.2 Recommendations

It is recommended that meteorological conditions in the Goulds and Petty Harbour area be monitored towards determination of changing trends in rainfall and generally extreme weather.

It is further recommended that climate change be integrated into municipal planning in those areas where increasing flood risk is relevant such as infrastructure and emergency planning.

8.0 FLOOD RISK MAPPING

The 1:20 year and 1:100 year AEP water surface profiles were used to develop flood risk mapping using the outputs of the hydraulic model. Flows used as input to the hydraulic model are based on outputs from the hydrologic model, which used, in part, the CBCL IDF relationship as input to define rainfall. Flood risk maps illustrate the extent of flooding that is expected under the 1:20 year and 1:100 year AEP flood events and are available for use by all levels of government, private companies and other stakeholders. Additionally, climate change analyses were carried out for the 2020, 2050 and 2080 tri-decades, as outlined in Section 7, for both the 1:20 year AEP and 1:100 year AEP scenarios. Associated flood risk mapping was prepared for the most severe climate change water levels anticipated for the 1:20 year and 1:100 year AEP flood events.

HEC-GeoRAS enables the conversion of HEC-RAS results into GIS-based flood risk mapping. The program creates a polyline feature class to which the maximum water surface elevations are attributed. From this, triangulation is carried out which interpolates the water surface elevation between adjacent cross sections. A volumetric cut-fill analysis is then performed between the water surface and the topography to arrive at the resultant inundated area. The generic functionality for the automated flood line generation routines within HEC-GeoRAS is based on a gridded approach to DTM processing. The gridded approach attempts to represent the terrain using a “smooth” mathematical model across the entire terrain surface. Gridded DTM processing has a tendency for over- and under-shoot (i.e. the grid elevation at a point is over or under the known elevation at that same point) in zones of rapidly changing terrain. The terrain in the subject watersheds, particularly in the flood plain, is considered rapidly varying. Initial results with the generic automated flood plain functionality yielded less than desirable results. As such, a manual procedure mimicking the generic HEC-GeoRAS functionality was employed with the exception that the DTM processing was based on the Triangulated Irregular Network representation of the terrain. The resultant flood lines were significantly improved in terms of the accuracy of their placement relative to the known terrain and associated elevations.

LiDAR was acquired for the entire study area and, as such, was used to accurately represent basin topography for the purposes of flood mapping development.

It should be noted that, although the automatically generated inundation polygon provides reliable inundation at each cross section location, manual post-processing is required to ensure that the water surface elevation between cross sections is represented properly. The following issues were noted as requiring manual post-processing:

- In areas where a tributary enters the main watercourse between cross sections, the triangulated water surface often overestimates the extent of flooding up the tributary which is caused by an increase in water level along the main watercourse.
- It is also common for low lying areas, which are located off the main watercourse and which would not realistically be inundated, to appear inundated as a result of the cut-fill analysis.
- Similarly, backwater areas, where flooding of low lying areas located off the main

watercourses is reasonable, can be falsely extended if the extent of the backwater area traverses upstream sections beyond the point connection to the main watercourse.

- For the purpose of post processing, 0.5m contours were created from the LiDAR so that post processing in these areas can be carried out to approximately the same level of accuracy as is inherent in the LiDAR DTM.

Flood plain maps illustrating the extent of flooding expected under the 1:20 year and 1:100 year AEP flood events for Petty Harbour River (and tributaries) are available in Appendix E (existing conditions). Two versions of the maps have been produced. One set uses the community scale (1:2,500) digital topographical (vector) mapping as the backdrop. The second set uses the 2011 orthophotos, captured as a component of the overall LiDAR data collection effort, as the backdrop.

Flood Risk Mapping was produced also for the most severe 1:20 year and 1:100 year AEP climate change precipitation scenarios for the 2020, 2050 and 2080 periods. These flood plain maps are available in Appendices F, G and H, respectively. Table 8-1 presents a list of the climate change scenarios for which flood lines were delineated, in addition to the percentage increase in area over each respective base case scenario.

It should be noted that hydraulic structures that are overtopped are covered by the flood polygon on the flood plain maps. If the structure is not overtopped, there is a break in the flood polygon so that the bridge deck is visible. Tables 8-2 and 8-3 detail flood levels at all structures included in the modelling and provide additional details regarding structure overtopping (where this occurs).

The information on the flood plain maps provides explicit cross-section referencing for each section in the HEC-RAS model. Using this cross-section reference, flood plain map users can access secondary hydraulic data, provided in Appendix J, for all hydraulic sections which comprise the overall HEC-RAS model. The flood plain maps also identify the 1:20 year and 1:100 year AEP water levels at each cross-section.

Flood depth maps are also provided in Appendices E, F, G, and H, as appropriate for each modelled scenario. Figures 8-1 through 8-8 provide overviews of the flood depth maps for the different scenarios included with this study.

Table 8-1: Climate Change Influence on Flood Inundation Area

Scenario	1:20 Year AEP		1:100 Year AEP	
	Flooded Area (km^2)	% Change	Flooded Area (km^2)	% Change
Existing	5.1	4.5%	5.4	4.9%
2020	5.2	6.5%	5.5	6.5%
2050	5.3	7.3%	5.6	7.5%
2080	5.3	8.2%	5.6	8.1%

Notes:

1. Please refer to Section 7 [Climate Change] for details specific to hydrologic and hydraulic modelling that differentiates between the various modelling scenarios.

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Table 8-2: Watercourse Crossings - Overtopping Summary – Petty Harbour River – 1:100 Year AEP Flood

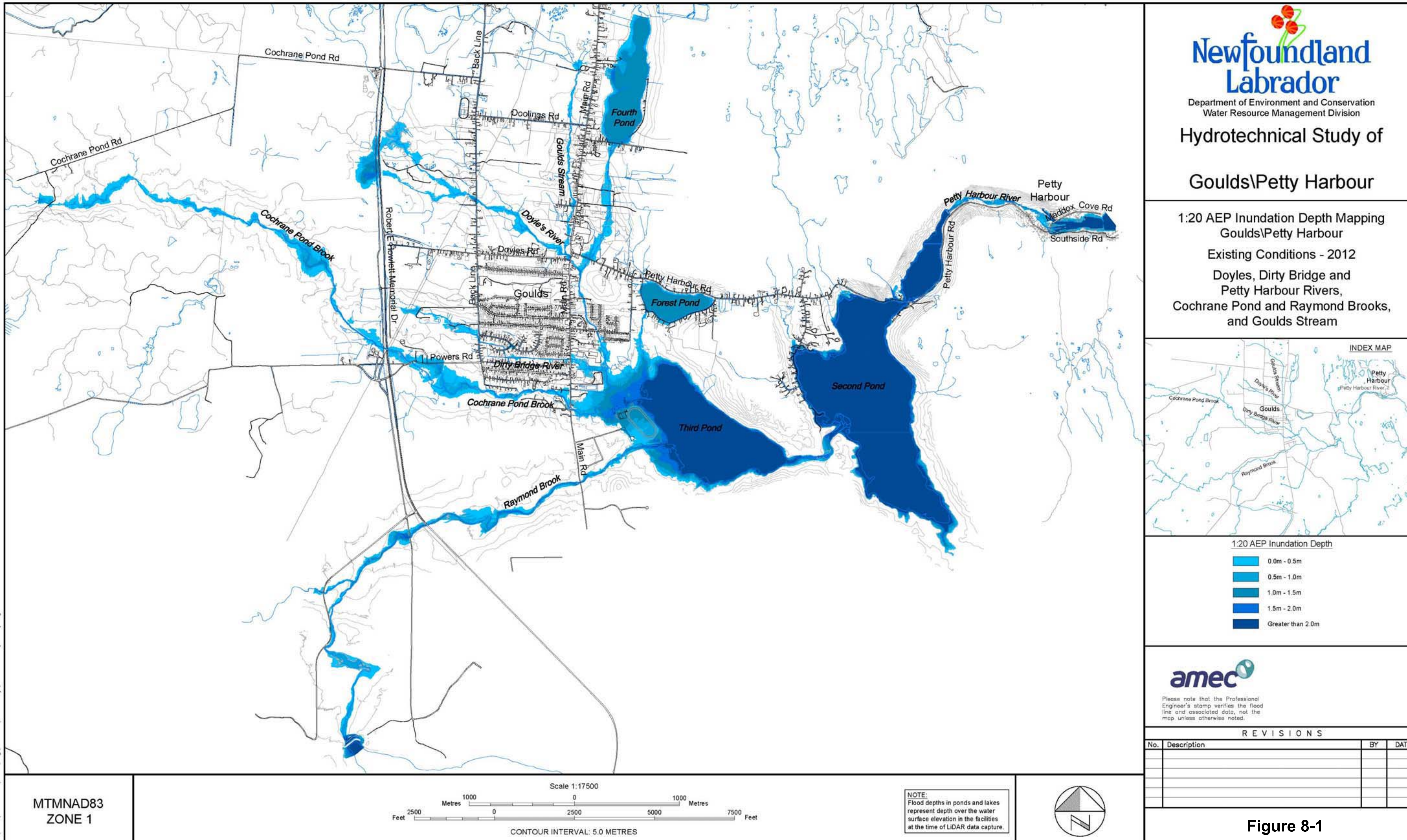
Structure #	Structure Name / Location	Structure Type	Watercourse	HEC-RAS Tributary	HEC-RAS Structure Number	Low Chord	Top of Road	Computed Water Surface Elevation by Scenario[1]				*Overtopping Depth* / Freeboard Available[2]			
								Existing	2020	2050	2080	Existing	2020	2050	2080
	100 Year AEP Flood														
3101	Main Road	Bridge	Petty Harbour River	PettyHR-M001	485	2.13	4.00	2.38	2.37	3.34	3.50	-1.62	-1.63	-0.66	-0.50
3103	Power Plant Access Road	Bridge	Petty Harbour River	PettyHR-M001	875	20.24	20.68	21.18	21.51	21.42	21.52	*0.50*	*0.83*	*0.74*	*0.84*
3104	Petty Harbour Dam	Dam	Petty Harbour River	PettyHR-M001	1631.79	63.55	65.02	64.81	64.94	65.01	64.99	-0.21	-0.08	-0.01	-0.03
3105	Petty Harbour Road (btw 1st and 2nd Pond)	Bridge	Petty Harbour River	PettyHR-M001	2580	63.10	64.50	65.26	65.41	65.48	65.55	*0.76*	*0.91*	*0.98*	*1.05*
3107	Forest Pond Road	Circular Culvert	Petty Harbour River	ForestP-M001	955	79.66	79.67	79.71	79.72	79.73	79.74	*0.04*	*0.05*	*0.06*	*0.07*
3108	Main Road	Bridge	Raymond Brook	RaymondB-M001	1315	82.22	82.63	81.65	81.82	81.91	82.55	-0.98	-0.81	-0.72	-0.08
3109	Robert E Howlett Memorial Drive	Bridge	Raymond Brook	RaymondB-M001	3200	93.17	93.95	92.25	92.40	92.48	92.53	-1.70	-1.55	-1.47	-1.42
3110	Unnamed Road	Bridge	Raymond Brook	RaymondB-M001	3410	93.50	94.50	93.40	93.55	93.62	93.67	-1.10	-0.95	-0.88	-0.83
3112	Bay Bulls Big Pond Dam	Dam	Raymond Brook	RaymondB-M002	6347.04	124.73	126.84	125.99	126.01	126.02	126.02	-0.85	-0.83	-0.82	-0.82
3113	Main Road	Bridge	Cochrane Pond Brook	CochranePB-M002	1190	71.13	71.52	70.63	70.70	70.75	70.79	-0.89	-0.82	-0.77	-0.73
3115	Powers Road	Bridge	Cochrane Pond Brook	CochranePB-M002	3165	89.98	90.13	89.57	89.63	89.69	89.67	-0.56	-0.50	-0.44	-0.46
3116	Robert E Howlett Memorial Drive	Bridge	Cochrane Pond Brook	CochranePB-M002	3450	93.11	94.50	91.95	92.05	92.09	91.82	-2.55	-2.45	-2.41	-2.68
3117	Unnamed Road	Bridge	Cochrane Pond Brook	CochranePB-M002	3480	94.11	94.26	92.19	92.27	92.33	92.46	-2.07	-1.99	-1.93	-1.80
3119	Cochrane's Pond Road	Circular Culvert	Cochrane Pond Brook	CochranePB-M002	8000	141.58	141.80	142.45	142.52	142.56	142.58	*0.65*	*0.72*	*0.76*	*0.78*
3120	Main Road	Circular Culvert	Dirty Bridge River	DoylesR-TR013	360	71.64	71.91	71.90	72.17	72.17	72.18	-0.01	*0.26*	*0.26*	*0.27*
3121	Hannaford Place	Circular Culvert	Dirty Bridge River	DoylesR-TR013	1050	83.74	84.55	84.49	84.51	84.50	84.50	-0.06	-0.04	-0.05	-0.05
3122	Back Line	Circular Culvert	Dirty Bridge River	DoylesR-TR013	1375	88.28	88.81	89.09	89.09	89.10	89.10	*0.28*	*0.28*	*0.29*	*0.29*
3125	Meadowbrook Drive	Bridge	Fourth Pond Brook	DoylesR-TR011	960	75.70	76.50	74.19	74.24	74.27	74.29	-2.31	-2.26	-2.23	-2.21
3127	Petty Harbour Road	Circular Culvert	Fourth Pond Brook	ForthP-M001	150	91.44	91.67	91.91	91.93	91.95	91.95	*0.24*	*0.26*	*0.28*	*0.28*
3129	4th Pond Road	Pipe Arch	Fourth Pond Brook	ForthP-M001	1390	123.88	124.15	124.37	124.38	124.39	124.39	*0.22*	*0.23*	*0.24*	*0.24*
3130	Doyles Road	Bridge	Doyles River	DoylesR-TR012	1745	90.90	91.59	92.34	92.42	92.50	92.56	*0.75*	*0.83*	*0.91*	*0.97*
3131	Back Line	Circular Culvert	Doyles River	DoylesR-TR015	3000	107.23	108.65	109.16	109.18	109.20	109.20	*0.51*	*0.53*	*0.55*	*0.55*
3134	Robert E Howlett Memorial Drive	Circular Culvert	Doyles River	DoylesR-TR016	850	127.38	128.70	128.74	128.74	128.74	128.75	*0.04*	*0.04*	*0.04*	*0.05*
3135	Robert E Howlett Memorial Drive	Circular Culvert	Doyles River	DoylesR-TR018	200	126.66	128.50	128.51	128.51	128.51	128.52	*0.01*	*0.01*	*0.01*	*0.02*
3136	Driveway to commercial property	Bridge	Goulds Stream	DoylesR-TR014	335	98.30	98.80	98.31	98.43	98.48	98.51	-0.49	-0.37	-0.32	-0.29
3137	Unnamed Road	Bridge	Goulds Stream	DoylesR-TR014	510	101.38	101.61	101.78	101.80	101.81	101.81	*0.17*	*0.19*	*0.20*	*0.20*
3141	Doolings Line	Bridge	Goulds Stream	DoylesR-TR014	1400	118.41	119.50	118.13	118.20	118.23	118.25	-1.37	-1.30	-1.27	-1.25
3142	Viquers Road	Circular Culvert	Goulds Stream	DoylesR-TR014	1980	126.21	126.27	126.46	126.49	126.51	126.56	*0.19*	*0.22*	*0.24*	*0.29*
3143	Walking Trail	Bridge	Cochrane Pond Brook	CochranePB-M002	3080	88.02	88.46	88.59	88.61	88.61	88.62	*0.13*	*0.15*	*0.15*	*0.16*
3144	Walking Trail	Bridge	Cochrane Pond Brook	CochranePB-M002	2915	87.57	87.87	87.75	87.76	87.76	87.76	-0.12	-0.11	-0.11	-0.11
3146	Walking Trail	Bridge	Raymond Brook	RaymondB-TR001	350	105.33	106.33	106.33	106.55	106.66	106.74	0.00	*0.22*	*0.33*	*0.41*
3151	Unnamed Road (west of REH Mermorial Drive)	Circular culvert	Doyles River	DoylesR-TR016	890	127.81	128.76	128.80	128.79	128.79	128.80	*0.04*	*0.03*	*0.03*	*0.04*
3152	Unnamed Road (west of REH Mermorial Drive)	Circular culvert	Doyles River	DoylesR-TR018	260	127.16	128.00	128.51	128.51	128.51	128.52	*0.51*	*0.51*	*0.51*	*0.52*
3153	Cochrane Pond Control Structure	Wooden Structure	Cochrane Pond Brook	CochranePB-M002	8050.98	140.00	141.00	142.45	142.52	142.56	142.55	*1.45*	*1.52*	*1.56*	*1.55*

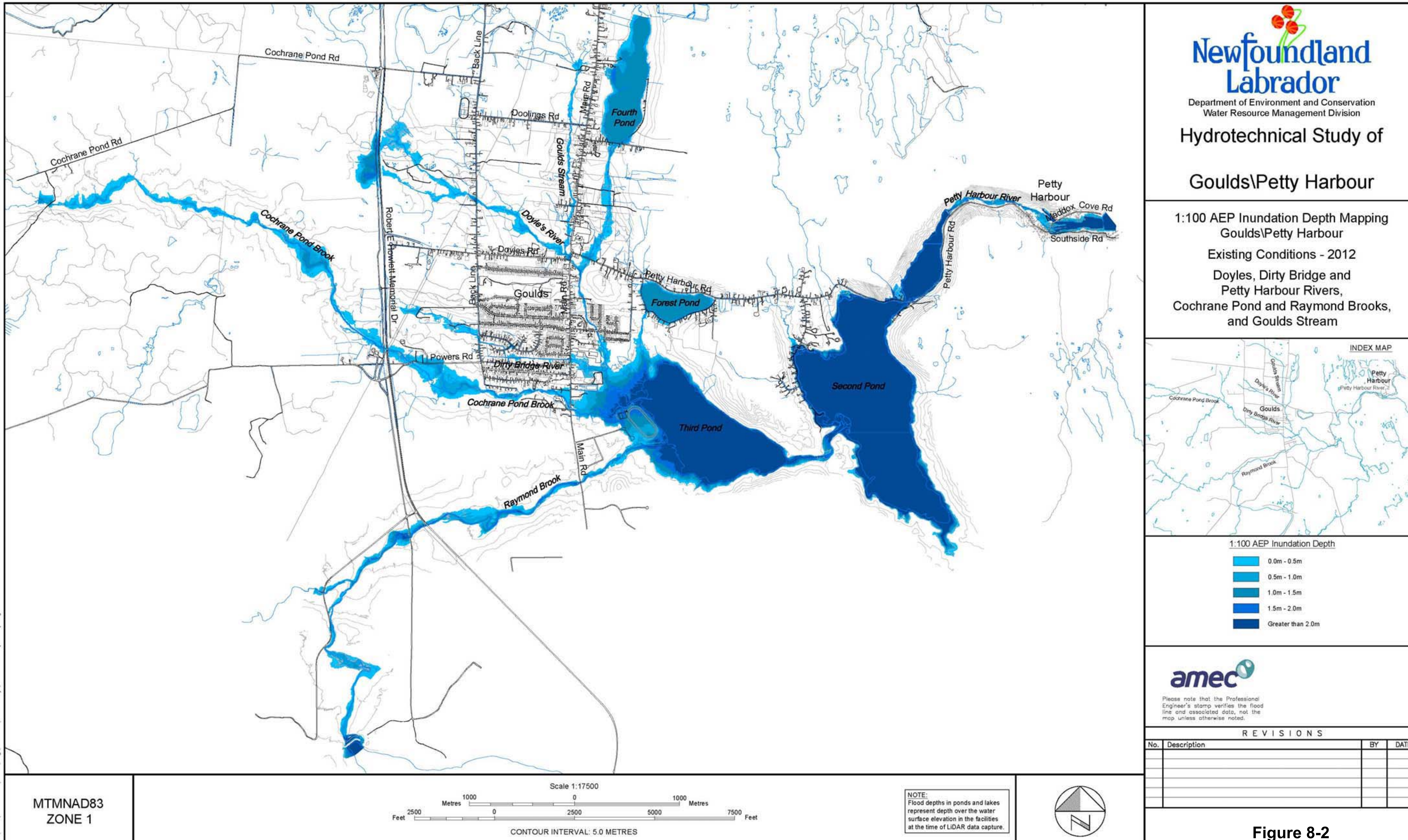
Notes:
1. Computed Water Surface Elevation by Scenario – colour coding of table cells – clear cells indicate no overtopping; blue cells indicate a water level causing surcharged flow; red cells indicate structures which are overtopped.
2. Overtopping Depth / Freeboard Available - colour coding of table cells – black text entries indicate that freeboard is available above the computed water level, red italic text entries indicate overtopping depth.

Table 8-3: Watercourse Crossings - Overtopping Summary – Petty Harbour River – 1:20 Year AEP Flood

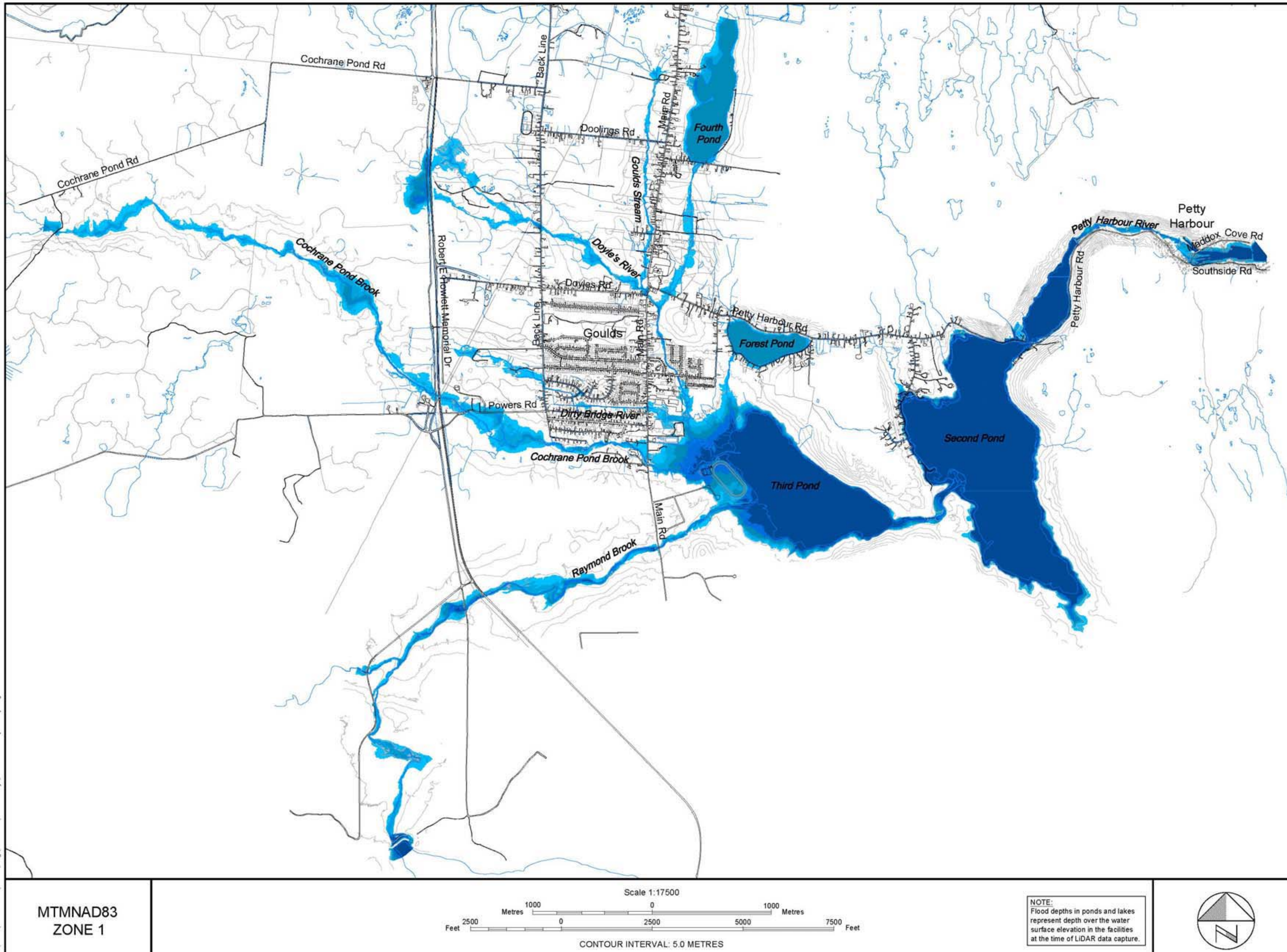
Structure #	Structure Name / Location	Structure Type	Watercourse	HEC-RAS Tributary	HEC-RAS Structure Number	Low Chord	Top of Road	Computed Water Surface Elevation by Scenario[1]				*Overtopping Depth* / Freeboard Available[2]			
								Existing	2020	2050	2080	Existing	2020	2050	2080
	20 Year AEP Flood														
3101	Main Road	Bridge	Petty Harbour River	PettyHR-M001	485	2.13	4.00	2.55	2.65	2.94	3.25	-1.45	-1.35	-1.06	-0.75
3103	Power Plant Access Road	Bridge	Petty Harbour River	PettyHR-M001	875	20.24	20.68	20.35	20.67	20.85	21.03	-0.33	-0.01	*0.17*	*0.35*
3104	Petty Harbour Dam	Dam	Petty Harbour River	PettyHR-M001	1631.79	63.55	65.02	64.48	64.61	64.68	64.61	-0.54	-0.41	-0.34	-0.41
3105	Petty Harbour Road (btw 1st and 2nd Pond)	Bridge	Petty Harbour River	PettyHR-M001	2580	63.10	64.50	64.77	64.97	65.08	65.18	*0.27*	*0.47*	*0.58*	*0.68*
3107	Forest Pond Road	Circular Culvert	Petty Harbour River	ForestP-M001	955	79.66	79.67	79.67	79.68	79.71	79.70	0.00	*0.01*	*0.04*	*0.03*
3108	Main Road	Bridge	Raymond Brook	RaymondB-M001	1315	82.22	82.63	81.21	81.39	81.48	81.54	-1.42	-1.24	-1.15	-1.09
3109	Robert E Howlett Memorial Drive	Bridge	Raymond Brook	RaymondB-M001	3200	93.17	93.95	91.80	91.99	92.08	92.13	-2.15	-1.96	-1.87	-1.82
3110	Unnamed Road	Bridge	Raymond Brook	RaymondB-M001	3410	93.50	94.50	93.28	93.45	93.56	93.62	-1.22	-1.05	-0.94	-0.88
3112	Bay Bulls Big Pond Dam	Dam	Raymond Brook	RaymondB-M002	6347.04	124.73	126.84	125.92	125.95	125.96	125.95	-0.92	-0.89	-0.88	-0.89
3113	Main Road	Bridge	Cochrane Pond Brook	CochranePB-M002	1190	71.13	71.52	70.50	70.55	70.55	70.57	-1.02	-0.97	-0.97	-0.95
3115	Powers Road	Bridge	Cochrane Pond Brook	CochranePB-M002	3165	89.98	90.13	89.36	89.42	89.48	89.50	-0.77	-0.71	-0.65	-0.63
3116	Robert E Howlett Memorial Drive	Bridge	Cochrane Pond Brook	CochranePB-M002	3450	93.11	94.50	91.74	91.82	91.87	91.89	-2.76	-2.68	-2.63	-2.61
3117	Unnamed Road	Bridge	Cochrane Pond Brook	CochranePB-M002	3480	94.11	94.26	92.18	92.42	92.09	92.09	-2.08	-1.84	-2.17	-2.17
3119	Cochrane's Pond Road	Circular Culvert	Cochrane Pond Brook	CochranePB-M002	8000	141.58	141.80	142.17	142.28	142.34	142.38	*0.37*	*0.48*	*0.54*	*0.58*
3120	Main Road	Circular Culvert	Dirty Bridge River	DoylesR-TR013	360	71.64	71.91	71.90	71.90	71.90	71.90	-0.01	-0.01	-0.01	-0.01
3121	Hannaford Place	Circular Culvert	Dirty Bridge River	DoylesR-TR013	1050	83.74	84.55	84.46	84.47	84.48	84.48	-0.09	-0.08	-0.07	-0.07
3122	Back Line	Circular Culvert	Dirty Bridge River	DoylesR-TR013	1375	88.28	88.81	89.05	89.07	89.07	89.07	*0.24*	*0.26*	*0.26*	*0.26*
3125	Meadowbrook Drive	Bridge	Fourth Pond Brook	DoylesR-TR011	960	75.70	76.50	74.07	74.12	74.14	74.15	-2.43	-2.38	-2.36	-2.35
3127	Petty Harbour Road	Circular Culvert	Fourth Pond Brook	ForthP-M001	150	91.44	91.67	91.83	91.87	91.88	91.88	*0.16*	*0.20*	*0.21*	*0.21*
3129	4th Pond Road	Pipe Arch	Fourth Pond Brook	ForthP-M001	1390	123.88	124.15	124.32	124.34	124.35	124.36	*0.17*	*0.19*	*0.20*	*0.21*
3130	Doyles Road	Bridge	Doyles River	DoylesR-TR012	1745	90.90	91.59	92.09	92.14	92.15	92.24	*0.50*	*0.55*	*0.56*	*0.65*
3131	Back Line	Circular Culvert	Doyles River	DoylesR-TR015	3000	107.23	108.65	109.09	109.12	109.13	109.14	*0.44*	*0.47*	*0.48*	*0.49*
3134	Robert E Howlett Memorial Drive	Circular Culvert	Doyles River	DoylesR-TR016	850	127.38	128.70	128.73	128.73	128.73	128.73	*0.03*	*0.03*	*0.03*	*0.03*
3135	Robert E Howlett Memorial Drive	Circular Culvert	Doyles River	DoylesR-TR018	200	126.66	128.50	127.64	127.92	128.22	128.50	-0.86	-0.58	-0.28	0.00
3136	Driveway to commercial property	Bridge	Goulds Stream	DoylesR-TR014	335	98.30	98.80	98.08	98.18	98.22	98.25	-0.72	-0.62	-0.58	-0.55
3137	Unnamed Road	Bridge	Goulds Stream	DoylesR-TR014	510	101.38	101.61	102.08	102.30	101.76	101.76	*0.47*	*0.69*	*0.15*	*0.15*
3141	Doolings Line	Bridge	Goulds Stream	DoylesR-TR014	1400	118.41	119.50	117.95	118.02	118.06	118.08	-1.55	-1.48	-1.44	-1.42
3142	Viqures Road	Circular Culvert	Goulds Stream	DoylesR-TR014	1980	126.21	126.27	126.38	126.43	126.43	126.44	*0.11*	*0.16*	*0.16*	*0.17*
3143	Walking Trail	Bridge	Cochrane Pond Brook	CochranePB-M002	3080	88.02	88.46	88.58	88.76	88.59	88.58	*0.12*	*0.30*	*0.13*	*0.12*
3144	Walking Trail	Bridge	Cochrane Pond Brook	CochranePB-M002	2915	87.57	87.87	87.75	87.75	87.75	87.75	-0.12	-0.12	-0.12	-0.12
3146	Walking Trail	Bridge	Raymond Brook	RaymondB-TR001	350	105.33	106.33	105.42	105.63	105.77	106.17	-0.91	-0.70	-0.56	-0.16
3151	Unnamed Road (west of REH Memorial Drive)	Circular culvert	Doyles River	DoylesR-TR016	890	127.81	128.76	128.79	128.80	128.80	128.80	*0.03*	*0.04*	*0.04*	*0.04*
3152	Unnamed Road (west of REH Memorial Drive)	Circular culvert	Doyles River	DoylesR-TR018	260	127.16	128.00	128.01	128.02	128.26	128.50	*0.01*	*0.02*	*0.26*	*0.50*
3153	Cochrane Pond Control Structure	Wooden Structure	Cochrane Pond Brook	CochranePB-M002	8050.98	139.20	141.00	142.18	142.28	142.34	142.28	*1.18*	*1.28*	*1.34*	*1.28*

Notes:
1. Computed Water Surface Elevation by Scenario – colour coding of table cells – clear cells indicate no overtopping; blue cells indicate a water level causing surcharged flow; red cells indicate structures which are overtopped.
2. Overtopping Depth / Freeboard Available - colour coding of table cells – black text entries indicate that freeboard is available above the computed water level, red italic text entries indicate overtopping depth.


Newfoundland Labrador
Department of Environment and Conservation
Water Resource Management Division
Hydrotechnical Study of
Goulds\Petty Harbour
1:20 AEP Inundation Depth Mapping
Goulds\Petty Harbour
Existing Conditions - 2012
Doyles, Dirty Bridge and
Petty Harbour Rivers,
Cochrane Pond and Raymond Brooks,
and Goulds Stream
INDEX MAP
1:20 AEP Inundation Depth
0.0m - 0.5m
0.5m - 1.0m
1.0m - 1.5m
1.5m - 2.0m
Greater than 2.0m
amec
Please note that the Professional Engineer's stamp verifies the flood line and associated data, not the map unless otherwise noted.
REVISIONS
No. Description BY DATE
Figure 8-1
MTMNAD83
ZONE 1
Scale 1:17500
CONTOUR INTERVAL: 5.0 METRES
NOTE: Flood depths in ponds and lakes represent depth over the water surface elevation in the facilities at the time of LiDAR data capture.
Cochrane Pond Rd
Back Line
Doolings Rd
Main Rd
Fourth Pond
Goulds Stream
Doyle's River
Doyles Rd
Goulds
Forest Pond
Petty Harbour Rd
Petty Harbour River
Petty Harbour
Maddox Cove Rd
Southside Rd
Cochrane Pond Brook
Robert E Howlett Memorial Dr
Powers Rd
Dirty Bridge River
Third Pond
Second Pond
Raymond Brook


Newfoundland Labrador
Department of Environment and Conservation
Water Resource Management Division
Hydrotechnical Study of
Goulds\Petty Harbour
1:100 AEP Inundation Depth Mapping
Goulds\Petty Harbour
Existing Conditions - 2012
Doyles, Dirty Bridge and
Petty Harbour Rivers,
Cochrane Pond and Raymond Brooks,
and Goulds Stream
INDEX MAP
1:100 AEP Inundation Depth
0.0m - 0.5m
0.5m - 1.0m
1.0m - 1.5m
1.5m - 2.0m
Greater than 2.0m
amec
Please note that the Professional Engineer's stamp verifies the flood line and associated data, not the map unless otherwise noted.
REVISIONS
No.
Description
BY
DATE
Figure 8-2
MTMNAD83
ZONE 1
Scale 1:17500
Metres
Feet
CONTOUR INTERVAL: 5.0 METRES
NOTE:
Flood depths in ponds and lakes represent depth over the water surface elevation in the facilities at the time of LiDAR data capture.
Cochrane Pond Rd
Back Line
Doolings Rd
Main Rd
Fourth Pond
Goulds Stream
Doyle's River
Cochrane Pond Brook
Robert E Howlett Memorial Dr
Doyles Rd
Goulds
Petty Harbour Rd
Forest Pond
Powers Rd
Dirty Bridge River
Third Pond
Second Pond
Raymond Brook
Petty Harbour River
Petty Harbour
Maddox Cove Rd
Southside Rd
Petty Harbour Rd

amec



Newfoundland Labrador
Department of Environment and Conservation
Water Resource Management Division

Hydrotechnical Study of

Goulds\Petty Harbour

1:20 AEP Inundation Depth Mapping
Goulds\Petty Harbour

Climate Change Scenario - 2020
AMEC (Maximum IDF Relationship)

Doyles, Dirty Bridge and
Petty Harbour Rivers,
Cochrane Pond and Raymond Brooks,
and Goulds Stream

1:20 AEP Inundation Depth

- 0.0m - 0.5m
- 0.5m - 1.0m
- 1.0m - 1.5m
- 1.5m - 2.0m
- Greater than 2.0m

amec

Please note that the Professional Engineer's stamp verifies the flood line and associated data, not the map unless otherwise noted.

REVISIONS

No.	Description	BY	DATE

MTMNAD83
ZONE 1

NOTE:
Flood depths in ponds and lakes represent depth over the water surface elevation in the facilities at the time of LiDAR data capture.

Figure 8-3

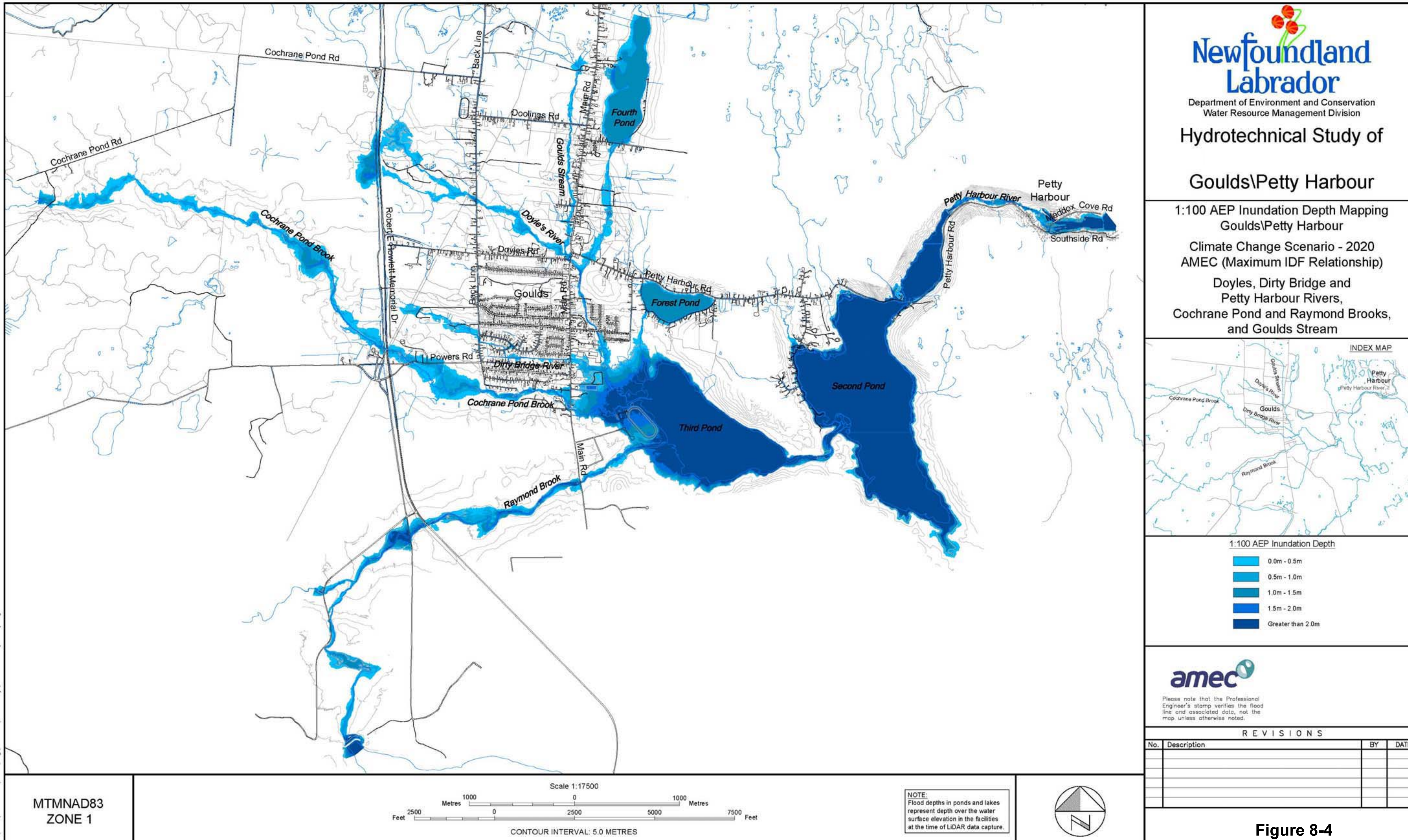
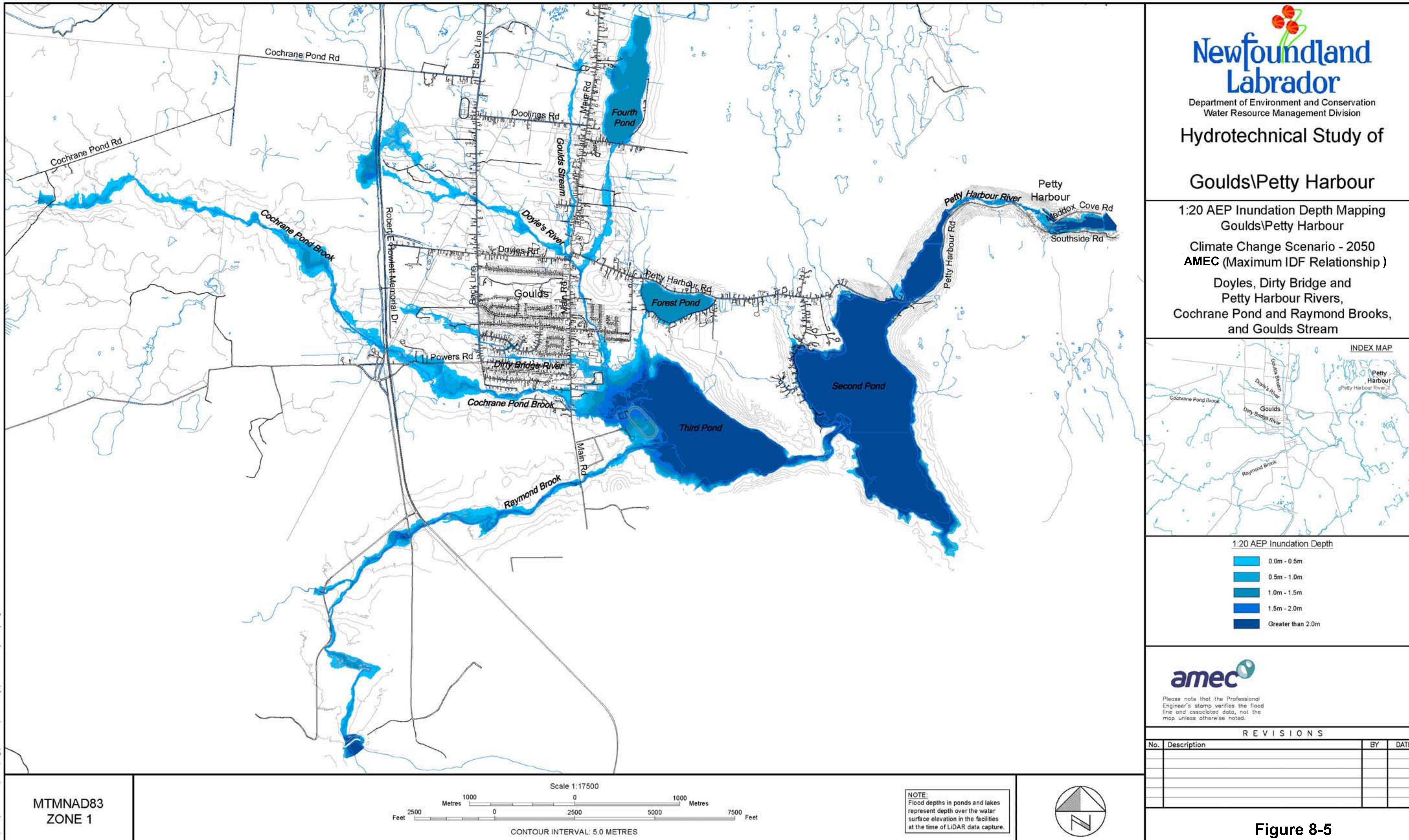
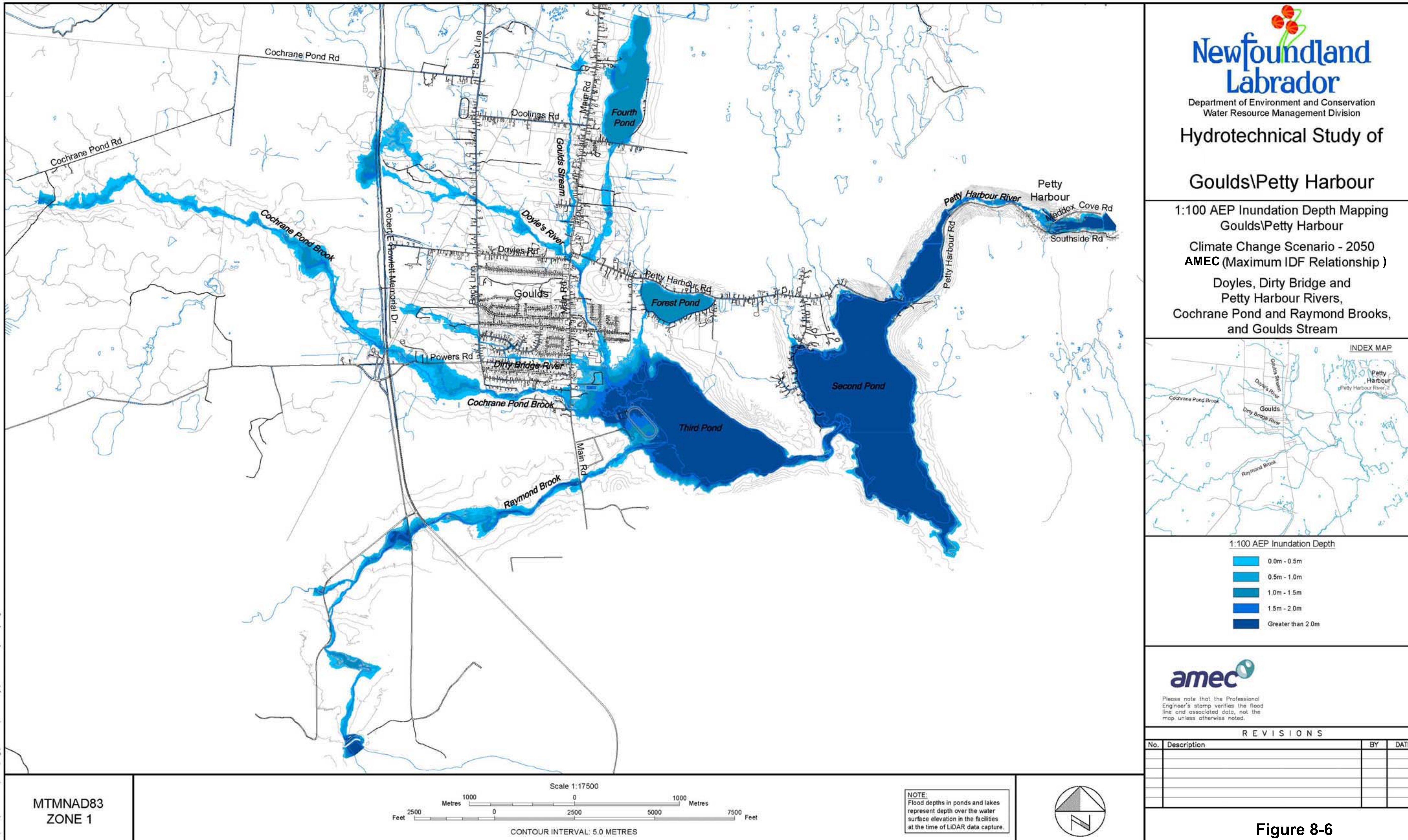

Cochrane Pond Rd
Cochrane Pond Rd
Cochrane Pond Brook
Back Line
Doolings Rd
Main Rd
Fourth Pond
Goulds Stream
Doyle's River
Doyles Rd
Robert E Howlett Memorial Dr
Goulds
Back Line
Main Rd
Petty Harbour Rd
Forest Pond
Powers Rd
Dirty Bridge River
Cochrane Pond Brook
Third Pond
Second Pond
Main Rd
Raymond Brook
Petty Harbour River
Petty Harbour
Maddox Cove Rd
Southside Rd
Petty Harbour Rd
Newfoundland Labrador
Department of Environment and Conservation
Water Resource Management Division
Hydrotechnical Study of
Goulds\Petty Harbour
1:100 AEP Inundation Depth Mapping
Goulds\Petty Harbour
Climate Change Scenario - 2020
AMEC (Maximum IDF Relationship)
Doyles, Dirty Bridge and
Petty Harbour Rivers,
Cochrane Pond and Raymond Brooks,
and Goulds Stream
INDEX MAP
1:100 AEP Inundation Depth
0.0m - 0.5m
0.5m - 1.0m
1.0m - 1.5m
1.5m - 2.0m
Greater than 2.0m
amec
Please note that the Professional Engineer's stamp verifies the flood line and associated data, not the map unless otherwise noted.
REVISIONS
No. Description BY DATE
MTMNAD83
ZONE 1
Scale 1:17500
Metres 1000 0 1000 Metres
Feet 2500 0 2500 5000 7500 Feet
CONTOUR INTERVAL: 5.0 METRES
NOTE:
Flood depths in ponds and lakes represent depth over the water surface elevation in the facilities at the time of LiDAR data capture.
N

Figure 8-4


Cochrane Pond Rd
Cochrane Pond Rd
Back Line
Doolings Rd
Main Rd
Fourth Pond
Goulds Stream
Cochrane Pond Brook
Robert E Howlett Memorial Dr
Doyle's River
Doyles Rd
Goulds
Back Line
Main Rd
Petty Harbour Rd
Forest Pond
Powers Rd
Dirty Bridge River
Cochrane Pond Brook
Third Pond
Second Pond
Petty Harbour Rd
Petty Harbour River
Petty Harbour
Maddox Cove Rd
Southside Rd
Main Rd
Raymond Brook
Newfoundland Labrador
Department of Environment and Conservation
Water Resource Management Division
Hydrotechnical Study of
Goulds\Petty Harbour
1:20 AEP Inundation Depth Mapping
Goulds\Petty Harbour
Climate Change Scenario - 2050
AMEC (Maximum IDF Relationship)
Doyles, Dirty Bridge and
Petty Harbour Rivers,
Cochrane Pond and Raymond Brooks,
and Goulds Stream
INDEX MAP
1:20 AEP Inundation Depth
0.0m - 0.5m
0.5m - 1.0m
1.0m - 1.5m
1.5m - 2.0m
Greater than 2.0m
amec
Please note that the Professional Engineer's stamp verifies the flood line and associated data, not the map unless otherwise noted.
REVISIONS
No. Description BY DATE
MTMNAD83
ZONE 1
Scale 1:17500
Metres 1000 0 1000 Metres
Feet 2500 0 2500 5000 7500 Feet
CONTOUR INTERVAL: 5.0 METRES
NOTE:
Flood depths in ponds and lakes represent depth over the water surface elevation in the facilities at the time of LiDAR data capture.
N
Figure 8-5


Cochrane Pond Rd
Cochrane Pond Rd
Back Line
Doolings Rd
Main Rd
Fourth Pond
Goulds Stream
Cochrane Pond Brook
Doyle's River
Doyles Rd
Robert E Howlett Memorial Dr
Goulds
Petty Harbour Rd
Forest Pond
Powers Rd
Dirty Bridge River
Cochrane Pond Brook
Third Pond
Second Pond
Raymond Brook
Petty Harbour River
Petty Harbour
Maddox Cove Rd
Southside Rd
Petty Harbour Rd
Newfoundland Labrador
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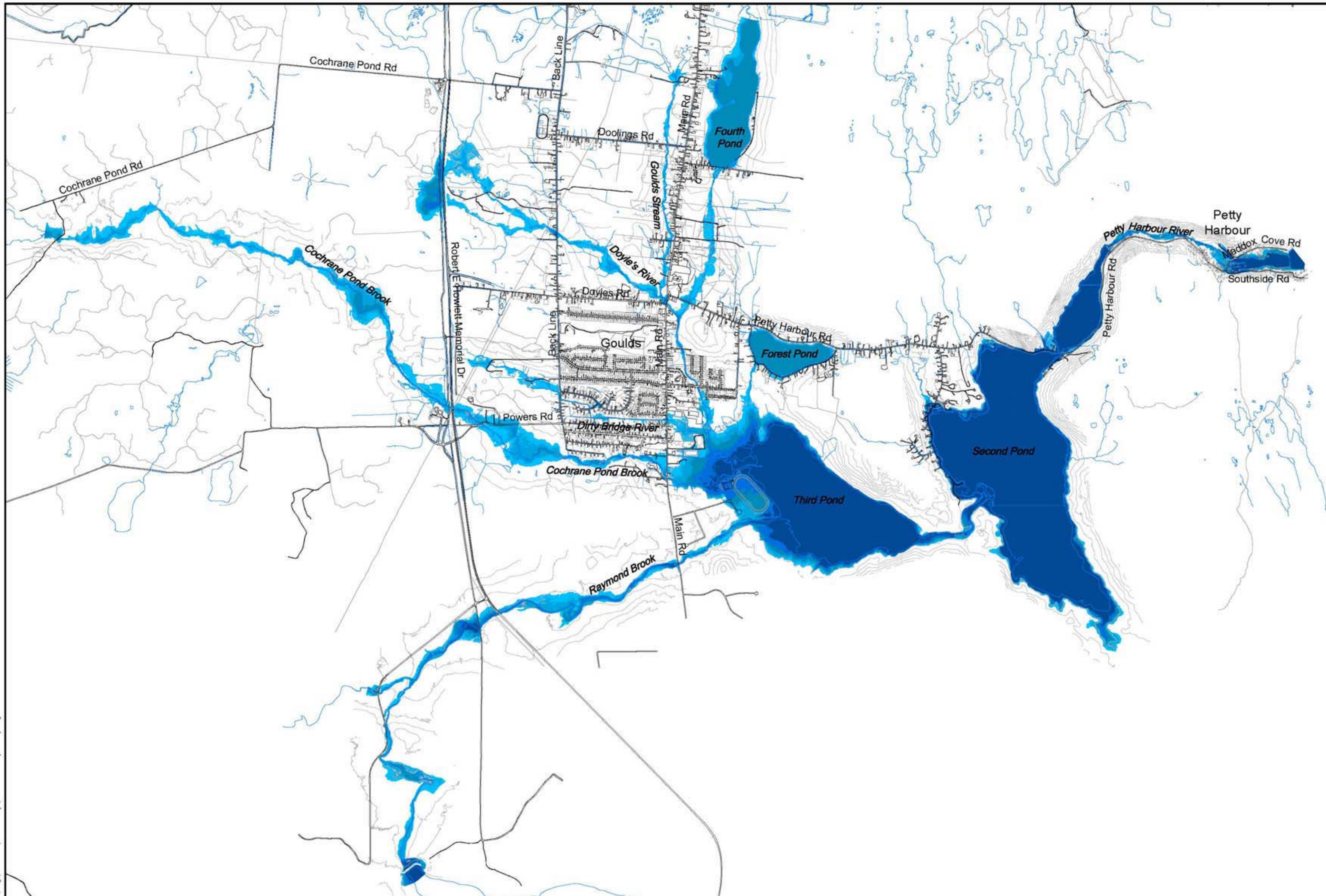
Goulds\Petty Harbour
1:100 AEP Inundation Depth Mapping
Goulds\Petty Harbour
Climate Change Scenario - 2050
AMEC (Maximum IDF Relationship)
Doyles, Dirty Bridge and
Petty Harbour Rivers,
Cochrane Pond and Raymond Brooks,
and Goulds Stream
INDEX MAP
1:100 AEP Inundation Depth
0.0m - 0.5m
0.5m - 1.0m
1.0m - 1.5m
1.5m - 2.0m
Greater than 2.0m
amec
Please note that the Professional Engineer's stamp verifies the flood line and associated data, not the map unless otherwise noted.
REVISIONS
No. Description BY DATE
MTMNAD83
ZONE 1
Scale 1:17500
Metres 1000 0 1000 Metres
Feet 2500 0 2500 5000 7500 Feet
CONTOUR INTERVAL: 5.0 METRES
NOTE:
Flood depths in ponds and lakes represent depth over the water surface elevation in the facilities at the time of LiDAR data capture.
N

Figure 8-6

amec



Newfoundland Labrador
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Hydrotechnical Study of

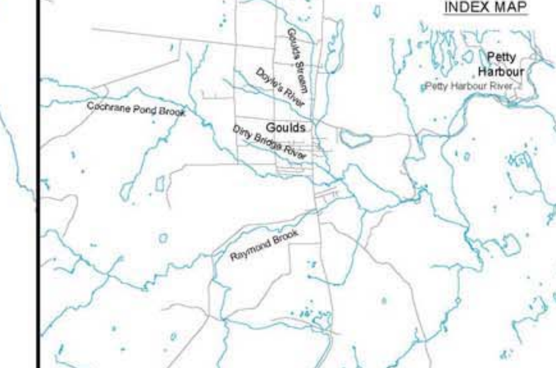
Goulds\Petty Harbour

1:20 AEP Inundation Depth Mapping
Goulds\Petty Harbour

Climate Change Scenario - 2080
AMEC (Maximum IDF Relationship)

Doyles, Dirty Bridge and
Petty Harbour Rivers,
Cochrane Pond and Raymond Brooks,
and Goulds Stream

INDEX MAP



1:20 AEP Inundation Depth

- 0.0m - 0.5m
- 0.5m - 1.0m
- 1.0m - 1.5m
- 1.5m - 2.0m
- Greater than 2.0m

amec

Please note that the Professional Engineer's stamp verifies the flood line and associated data, not the map unless otherwise noted.

REVISIONS

No.	Description	BY	DATE

MTMNAD83
ZONE 1

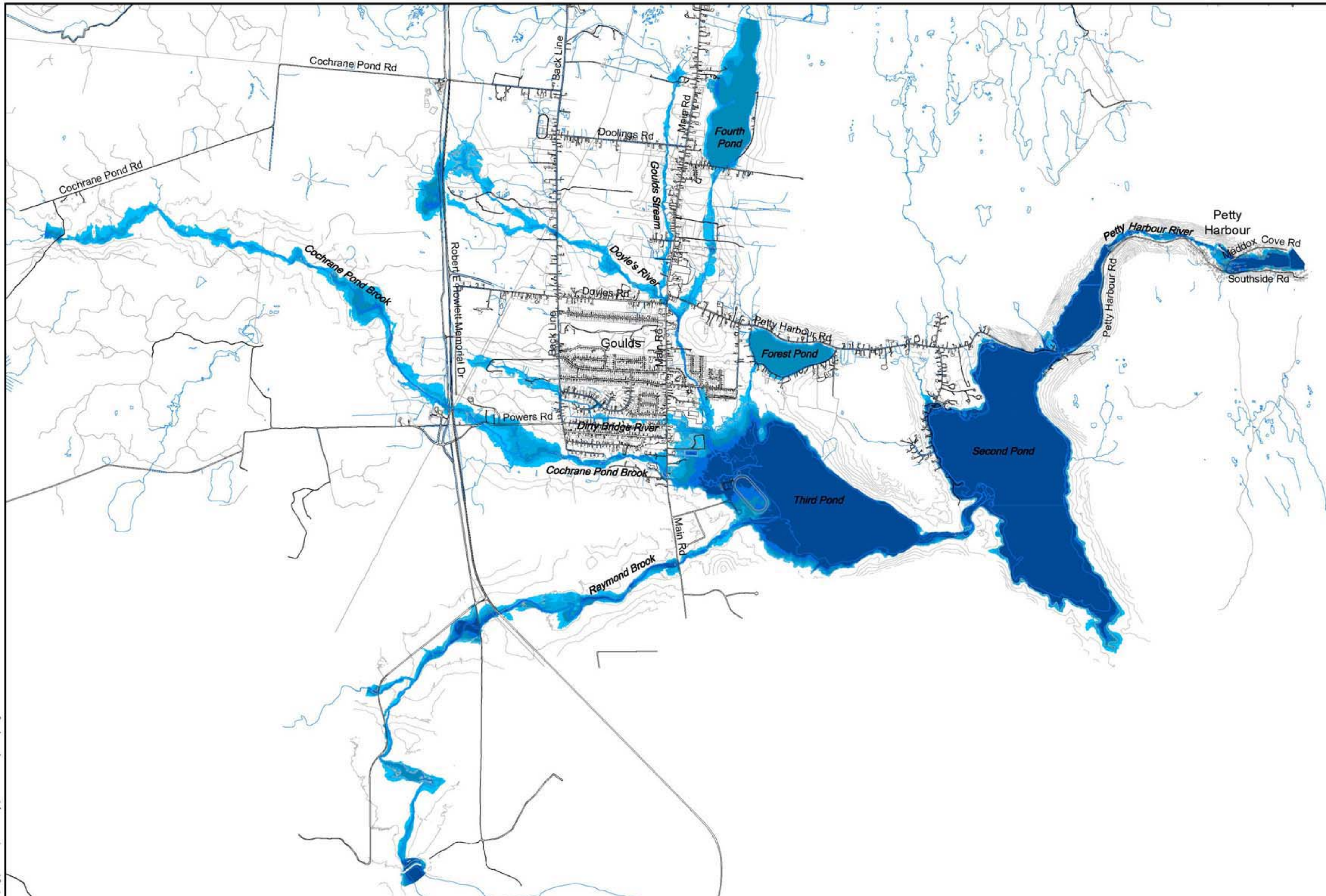
Scale 1:17500
Metres 1000 0 1000 Metres
Feet 2500 0 2500 5000 7500 Feet
CONTOUR INTERVAL: 5.0 METRES

NOTE:
Flood depths in ponds and lakes represent depth over the water surface elevation in the facilities at the time of LiDAR data capture.

N

Figure 8-7

amec



Newfoundland Labrador
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Water Resource Management Division

Hydrotechnical Study of

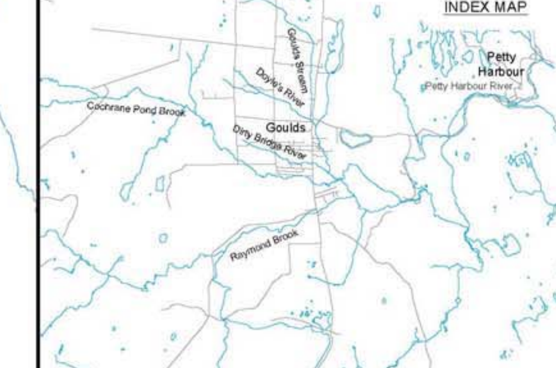
Goulds\Petty Harbour

1:100 AEP Inundation Depth Mapping
Goulds\Petty Harbour

Climate Change Scenario - 2080
AMEC (Maximum IDF Relationship)

Doyles, Dirty Bridge and
Petty Harbour Rivers,
Cochrane Pond and Raymond Brooks,
and Goulds Stream

INDEX MAP



1:100 AEP Inundation Depth

- 0.0m - 0.5m
- 0.5m - 1.0m
- 1.0m - 1.5m
- 1.5m - 2.0m
- Greater than 2.0m

amec

Please note that the Professional Engineer's stamp verifies the flood line and associated data, not the map unless otherwise noted.

REVISIONS

No.	Description	BY	DATE

MTMNAD83
ZONE 1

Scale 1:17500

Metres 1000 0 1000 Metres
Feet 2500 0 2500 5000 7500 Feet

CONTOUR INTERVAL: 5.0 METRES

NOTE:
Flood depths in ponds and lakes represent depth over the water surface elevation in the facilities at the time of LiDAR data capture.

N

Figure 8-8

9.0 CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

Key outcomes of the study are described below:

Background Information

- A review of the historical flooding events in the study watershed identified thirty-eight (38) flood events in the Goulds and Petty Harbour area. Flood events have occurred in this area in all seasons of the year and all months of the year except June. Of the noted flood events, five (5) identified ice jamming as the primary cause.
- The Goulds and Petty Harbour study area has been previously assessed for flood risk in 1996 by BAE-Newplan Group.

Field Program

- Fifteen (15) cross sections along each watercourse were field surveyed for below waterline information.
- It was anticipated that streamflow monitoring would be undertaken by the Water Survey of Canada. The late season start to this project and scheduling issues at Water Survey of Canada did not provide any opportunity for this aspect of the project to be completed in advance of freeze up of watercourses in the project study areas. As such, this task of the project was not completed and this limited the data available to this project for calibration and validation.
- A high-resolution LiDAR DTM of the entire Petty Harbour River watershed was collected in November and December of 2011.
- The field survey for Petty Harbour River included forty-one (41) structures.
- The Petty Harbour River watershed is influenced by three (3) dams.

Hydrologic Assessment

- The 1:20 year and 1:100 year AEP streamflows were estimated for the Petty Harbour River watershed using both statistical and deterministic methodologies. The CBCL IDF relationship was used for modelling of streamflows for the purposes of flood limit determination. Comparative assessment of the flow estimates over the range of methodologies concluded that the deterministic model results provided a good estimate of streamflow for these watersheds. The methods used in the current study led to comparable flood flow estimates which provide confidence in the results.
- The HEC-GeoHMS and HEC-HMS models developed for this study are included with the Project CD materials attached to this report. These models may be used in the future to evaluate the impact on streamflows resulting from changes to the watershed.

Hydraulic Assessment

- A hydraulic model based on the USACE program HEC-RAS was developed for reaches of the Petty Harbour River covering a linear distance of approximately 31.6km (with 729 cross-sections).
- The model was developed based on field surveyed bathymetric data and a LiDAR survey conducted in November and December of 2011. Field survey of water levels specifically to form a database upon which the hydraulic model could be calibrated/validated was not completed due to late season project start and freeze up of the waterways in the study area. As such, the hydraulic model has not been calibrated/validated, however, due care was taken during model development to accurately establish model parameterization.
- The hydraulic model developed for this study was also used to evaluate the potential flood conditions (i.e. resultant water levels) associated with ice jamming events. The evaluation along Cochrane Pond Brook and Raymond Brook confirmed that along limited reaches of the watercourse, computed water levels associated with ice jams have the potential to generate water levels exceeding 1:100 year AEP open water event levels.
- The HEC-GeoRAS and HEC-RAS models developed for this study is included with the Project CD materials attached to this report. The model may be used in the future to evaluate the impact on water levels resulting from any structural changes to the subject watercourses or floodplain / overbank areas.

Sensitivity Analysis

- It is understood that the hydrologic model is sensitive to a variety of input parameters including rainfall and Curve Number. These parameters have been developed upon the best available information from Environment Canada, as well as, soils and land cover data; the latter reflecting current conditions in late 2011.
- Since all hydraulic input parameters were selected based on reliable background information, it is expected that the error and uncertainty associated with model output is minimal.

Climate Change Assessment

- Climate change analysis was completed using two estimates of future rainfall for three tri-decades (i.e. 2020s, 2050s and 2080s) for both the 1:20 year and 1:100 year AEP flood events. Rainfall estimates of 12 hour and 24 hour duration events were provided to this project by Dr. Joel Finnis Associate Professor in the Department of Geography at Memorial University for St. John’s for the 2050 period. AMEC, as a component of this project, calculated estimates of future IDF relationships for the three subject periods for the St. John’s Airport station.
- It is concluded from this assessment that climate change has the potential to increase flood risk in the Goulds and Petty Harbour area.

Flood Risk Mapping

- All information necessary to complete the Flood Risk Mapping Project for this project was available either through information provided by the WRMD, available background reports, contact with local municipalities or based on the comprehensive field data collection program.
- Flood risk mapping was developed using the LiDAR DTM, 1:2,500 scale community mapping, 1:50,000 topographic maps, and orthophoto imagery. These maps were based on the results of both the hydrologic and hydraulic analysis, and can be used by both the communities located within the study watershed for municipal planning and the WRMD for flood risk identification.
- Climate change flood lines were delineated for the most severe climate change precipitation scenarios and mapping was developed using 1:2,500 scale community mapping in combination with the LiDAR DTM. The flooded area associated with the 2080 period represented an increase area of about 0.4% above flooded area associated with the existing conditions (CBCL) scenario for both the 1:20 year and 1:100 year AEP climate change scenarios.

9.2 Recommendations

Key recommendations stemming from the assessments completed for this study are outlined below:

1. It is recommended that the municipalities located within the study area adopt the flood lines developed by the current study for its municipal plan and development regulations.
2. It is recommended that the municipalities located within the study area and their partners make use of the up-to-date LiDAR topographic data and orthophotography which was collected for this study for relevant municipal initiatives.
3. The St. John's Airport rainfall station relative to the Petty Harbour River Watershed lies some distance away from the approximate centroid of the watershed. As such, it is recommended that a rainfall station local to the Goulds and Petty Harbour Area, that would support assessment of IDF relationships, be installed to support watershed analysis and give insight into local meteorological conditions specific to the area.
4. It is recommended that the municipalities located within the study area engage in a program to measure water levels at designated watercourse crossing structures during flood events. This will provide a database of information which could be used to support both hydrologic and hydraulic modelling in the future.
5. It is also recommended that a program focused on unregulated streamflow data collection be developed for Petty Harbour River and its associated tributaries. Additional recording stations at strategic locations (e.g., outflow from each of the unregulated tributary areas) would provide a foundation of data that would enhance the hydrologic model calibration/validation process.

6. It is recommended that that HEC-GeoHMS, HEC-HMS, HEC-GeoRAS and HEC-RAS be used in future watershed and flood studies as their use both simplifies the development of deterministic models, as well as provides for the generation of a significant warehouse of information that can be used for other ancillary purposes beyond hydrologic assessments.
7. It is recommended that special consideration be given to higher water levels (than those based on the 1:100 year AEP flow) associated with ice jam conditions. For instance, the community can opt to designate the “ice jam” flood inundated area as a special policy area which will allow the community to enact specific policies/guidelines regarding development while recognizing the local expectation (base on historical occurrence) of ice jamming.
8. It is recommended that the municipalities located within the study area consider stream and/or structure rehabilitation in the areas where water levels exceed the river banks during the 1:100 year AEP flood and spill over land. This will confine extreme flood flows to the river channel and avoid the risk of overland flooding.
9. It is recommended that meteorological conditions in the Goulds and Petty Harbour area be monitored towards determination of increasing trends in rainfall and generally extreme weather.
10. It is recommended that climate change be integrated into municipal planning in those areas where increasing flood risk is relevant such as infrastructure and emergency planning.
11. It is recommended that this study should be revisited in approximately ten (10) years, after which time additional detail may be available from rainfall and streamflow gauges in the basin.
12. It is recommended that flood studies be initiated in early spring or sooner. Starting these projects in early spring will provide the time necessary to better plan field programs that can be conducted over the summer months. This allows surveying to be conducted during low flow conditions and allows for easier and safer access during summer months. Another benefit is that it potentially allows for the collection of more model calibration data. Flow metering (when required) and water surface profiles can be conducted in the spring when river levels are typically high, and also in the late summer when river levels are low. This would help to provide a good range of model calibration and validation data.
13. It is recommended that LiDAR topographic survey and orthophoto databases continue to be used for future flood risk mapping studies as they provide an accurate means of collecting high quality topography information over large areas.
14. It is recommended, although fundamental principles remain the same, that the “1976 Hydrologic and Hydraulic Procedures for Flood Plain Delineation” be updated to reflect current technological and engineering practices in regards to flood plain delineation and development of flood plain mapping.

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