

**HEALTH SCIENCES CENTRE  
FLOOD PLAIN STUDY  
REPORT**

**Prepared for:**

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**Date:** August 6, 2014

**AMEC Ref. No.:** 174811-0000-CD10-RPT-0001

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REPORT  
FOR  
**HEALTH SCIENCES CENTRE  
FLOOD PLAIN STUDY**

FOR  
**EASTERN HEALTH**

**PROVINCE OF NEWFOUNDLAND AND LABRADOR**

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B	August 6, 2014	Report	<i>TW</i>	<i>GM</i>	<i>KK</i>		
A	July 21, 2014	DRAFT Report - Issued for Review	TW	GM	LR		
REV.	DATE	REVISION(S)	PREPARED BY	CHECK	APP	CLIENT	
		<b>REPORT FOR HEALTH SCIENCES CENTRE FLOOD PLAIN STUDY</b>	AMEC JOB NO. 174811				
			REPORT NO. 174811-0000-CD10-RPT-0001			REV. B	
			PAGE 1 OF 1				

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## 1.0 INTRODUCTION

The Health Sciences Centre is located on Clinch Crescent (off of Prince Phillip Drive) in St. John's NL and is located immediately adjacent to Leary's Brook. Leary's Brook, which is located within the Rennie's River watershed, is immediately upstream of Long Pond. The area along Leary's Brook adjacent to the Health Sciences Centre is prone to flooding and has flooded several times in the past. Over the last number of years there have been flooding events which have covered Prince Phillip Drive (in the area between the Health Sciences Centre and the CBC building) and have caused road closures of both Prince Phillip Drive and Clinch Crescent west. The general topography of the Health Sciences Centre site is sloped in a west to east direction towards Leary's Brook, with the portions of the Health Sciences Centre consisting of the ICU and CICU and the Agnes Cowan Hostel set at the lowest elevations. The Utility Annex is set at a slightly higher elevation and the underground utility tunnel (running between the Utility Annex and the HSC) and the associated utility tunnel access set at the lowest elevation of the various structures.

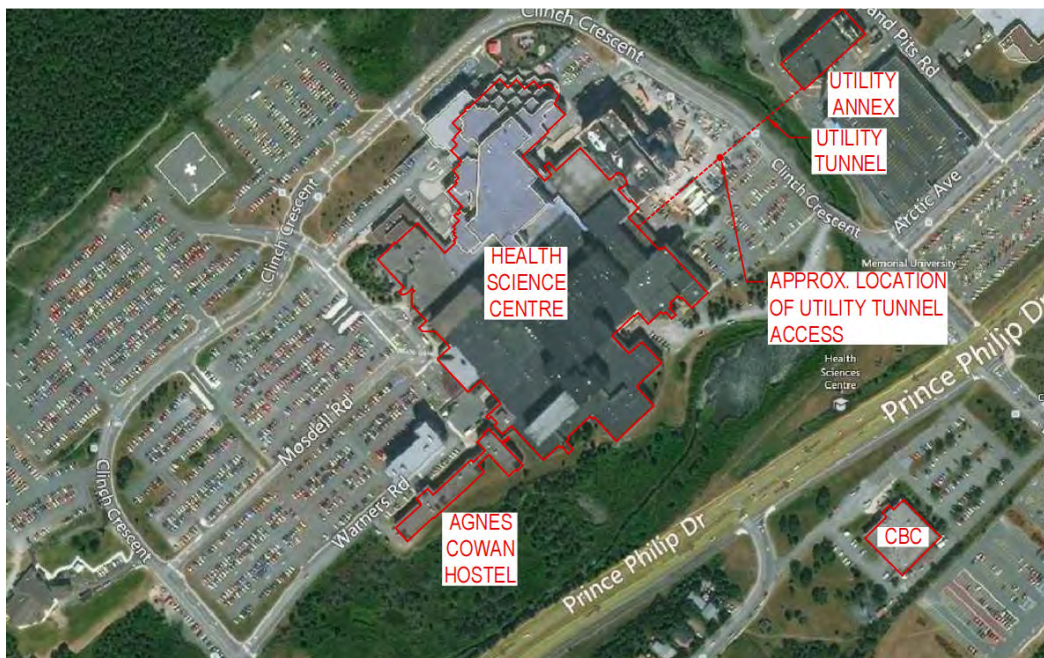


Figure 1 – Health Sciences Centre Site

Given the history of flooding in the vicinity, Eastern Health requested AMEC conduct a flood plain assessment to identify risks and future flooding issues.

## 1.1 STUDY SCOPE

The scope outlined in the proposal for this project was twofold. The first part of this project involved developing a concept design and cost estimate for an emergency access to the HSC site from Larkhall Street, where the second part of the project involved performing a floodplain study which is the subject of this report. The scope items specifically covered in this report are as follows:

1. Review of available documents including site civil drawings, aerial photos, and available flood plain analysis data provided by the City of St. John's to determine current conditions at the site.
2. Review flood plain data and conduct a desktop assessment to determine possible risk points in the present flood plain. Determine options for improving the protection measures along with associated costs and recommend preferred alternative.
3. Prepare a draft concept design report outlining the findings of the analysis conducted along with recommended solutions.

After discussions with Eastern Health regarding the potential impact of wind on flood water levels it was agreed that a wave setup and runoff analysis would be added to the scope of this project.

## 2.0 BACKGROUND INFORMATION

### 2.1 MAPS AND DRAWINGS

The following is a list of maps and drawings provided to AMEC for the purposes of this study. All of the following maps and drawings, with the exception of the first, were provided by the City of St. John's, where the first was provided by Eastern Health.

1. Document Type: Electronic .dwg (CAD) file  
Title: Proposed Site Development Plan – Health Sciences Centre Extension Janeway Children's Health and Rehabilitation Centre.  
Drawing Number: W-C\_1.1 R2  
Date: October 9, 1997 (rev 2)  
Brief Description: proposed site plan of entire Health Sciences Centre for Janeway extension.
2. Document Type: Electronic .jpg file  
Name: 300 Kenmount Rd. Floodplain  
From: City of St. John's  
Date: document creation date unknown (provided via. Email Nov 4, 2013).  
Brief Description: aerial photo of Health Sciences Centre with an unlabelled blue line representing flood plain.
3. Document Type: Electronic .pdf file  
Title: 220-212  
Date: 1983  
Brief Description: 1:500 City of St. John's map with 100 year and 1993 100 year flood plain indicated
4. Document Type: Electronic .pdf file  
Title: 221-212  
Date: 1983  
Brief Description: 1:500 City of St. John's map with 100 year and 1993 100 year flood plain indicated
5. Document Type: Electronic .pdf file  
Title: 220-213  
Date: 1983  
Brief Description: 1:500 City of St. John's map with 100 year and 1993 100 year flood plain indicated
6. Document Type: Electronic .pdf file  
Title: 221-213  
Date: 1983  
Brief Description: 1:500 City of St. John's map with 100 year and 1993 100 year flood plain indicated

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8. Document Type: Electronic .dwg (CAD) file  
Name: 3235270  
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Brief Description: CAD map of City of St. John's (area south of Health Sciences Centre)
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Date: unknown  
Brief Description: CAD map of City of St. John's (includes area southwest of Health Sciences Centre)
10. Document Type: Electronic .dwg (CAD) file  
Name: Kav & Assoc 3245270  
Date: unknown  
Brief Description: CAD map of City of St. John's (includes area to south east of Health Sciences Centre)
11. Document Type: Electronic .dwg (CAD) file  
Name: Kav & Assoc 3245271  
Date: unknown  
Brief Description: CAD map of City of St. John's (includes Health Sciences Centre and University)
12. Document Type: Electronic .dwg (CAD) file  
Name: cont3245270  
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Brief Description: CAD file containing contours for area east of University
13. Document Type: Electronic .dwg (CAD) file  
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Date: unknown  
Brief Description: CAD file containing contours for north east end of Health Sciences Centre
14. Document Type: Electronic .dwg (CAD) file  
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Date: unknown  
Brief Description: CAD file containing contours for area south of Health Sciences Centre

15. Document Type: Electronic .dwg (CAD) file  
Name: cont3235271  
Date: unknown  
Brief Description: CAD file containing contours for area west of Health Sciences Centre
16. Document Type: .shp  
Name: New\_Floodplain\_-\_Health\_Sciences.shp  
Date: unknown  
Brief Description: shp file containing extents of 1:100 flood plain

## 2.2 PREVIOUS STUDIES

The City of St. John's employed CBCL to develop the Rennies River Catchment Stormwater Management Plan (RRCSMP). This Plan involved reviewing previous studies, developing a calibrated hydraulic model of the Rennies river watershed allowing for effects of future development and climate change and developing flood plain mapping and preliminary designs and estimates for flood control. A copy of the report is located in Appendix B.

## 2.3 GOVERNMENT POLICIES

The following Government of Newfoundland and Labrador documents provide guidance regarding development within and adjacent to floodplains and flood risk areas:

1. Newfoundland and Labrador Provincial Land Use Policy Flood Risk Areas (LUPFRA), which is published by the Fire and Emergency Services Agency.
2. Policy for Flood Plain Management (PFPM), which is published by the Department of Environment and Conservation.

Copies of each policy are located in Appendix D. These policies define the floodplain, floodway fringe and climate change flood zone and indicate which types of developments are allowed within each area with the expressed goal, among others, of reducing property damages and increasing public safety. The LUPFRA indicates that structures such as hospitals are not permitted within the floodway fringe and defines the floodway fringe as the "flood risk area, where the risk of flooding is lower, on average once in one hundred years", the PFPM uses a similar definition of floodway fringe. The PFPM indicates that structures such as hospitals are not permitted with the climate change flood zone. Where the climate change zone is defined as an extension of the floodway fringe due to latest forecasted effects of climate change.



## 2.4 CRITICAL STRUCTURE ELEVATIONS

In order to assess the impact of flood events on the HSC it was necessary to establish the finished floor elevations for critical structures, such as the lowest floor elevations for the HSC, Agnes Cowan Hostel and Utility Annex (Memorial University). The finished floor elevations for each of these structures are:

1. Agnes Cowan Hostel = 58.12m
2. HSC (lowest floor) = 58.20m
3. Utility Annex = 58.22m

In addition there is a utility tunnel that runs between the Utility Annex and the HSC. This tunnel provides all of the steam and hot water used in the HSC complex. The tunnel has surface access and ventilation located at the midpoint. The surface elevation at this point is estimated to be approximately 57.15m. These various elevations were obtained from existing record drawings and survey data (in the case of the utility annex). Refer to Figure A1 in appendix A for approximate location of utility tunnel and utility tunnel access.

## **3.0 ANALYSIS**

### **3.1 FLOOD PLAIN**

Floodplain mapping and floodwater elevations provided by the City were used to establish the extents of the flooding, and to assess the flood risks associated with the 1:100 Annual Exceedance Probability (AEP) event. Both the flood plain mapping and corresponding floodwater elevations were established and provided by the City using the hydraulic model developed by for the RRCSMP. The flood plain extents and floodwater elevations illustrated in Figures A1 and A2 (located in Appendix A) reflect the 1:100 AEP and account for future developments and forecasted climate change effects (refer to RRCSMP located in Appendix B for further details). The 1:100 AEP elevations were given for three different points adjacent to the HSC and are as follows:

1. PT1 – 56.95m (corresponds to section C on Figure A2 located in Appendix A)
2. PT2 – 57.40m (corresponds to section B on Figure A2 located in Appendix A)
3. PT3 – 57.56m (corresponds to section A on Figure A2 located in Appendix A)

For the purposes of this study the highest elevation (PT3) was used as the 1:100 AEP flood water level in this report.

### **3.2 WAVE SETUP AND RUNUP ANALYSIS**

An analysis was undertaken to determine the maximum inundation levels due to wind induced setup and wave runup on the large flooded area adjacent to the HSC for the 1:100 AEP. The wave induced setup was found to be insignificant while the combined maximum wind and wave induced setup was found to be 0.32m (refer to report located in Appendix E for report).

### **3.3 STRUCTURAL FLOOD CONTROL OPTIONS**

Structural measures refer to flood control facilities that consist of engineering structures or modifications as opposed to non-structural measures which modify the damage potential for permanent facilities such as flood proofing, flood warning and land-use controls. Examples of Structural modifications include:

1. Flood – Control Reservoir
2. Diversion
3. Channel Modifications
4. Levee

#### **3.3.1 Flood - Control Reservoir**

Flood-Control Reservoirs are used to store flood waters for a specified period of time and release them after the flood event, reducing the magnitude of the peak discharge and consequently peak stage (downstream of reservoir) for any given return event. Among the many requirements that are necessary for a flood-control reservoir to be suitable for flood control is that there be sufficient area available to locate a flood-control reservoir. As there is no available area upstream of the

Health Sciences Centre to allow for the construction of a Flood-Control Reservoir, this option was given no further consideration.

### **3.3.2 Diversion**

Diversions are used to bypass flood flows around areas vulnerable to flooding, thus reducing the magnitude of the peak discharge and consequent stage at the vulnerable area for any given return event. Among the many requirements that are necessary for a diversion to be suitable for flood control is that there be sufficient area for both the diversion channel and diversion inlet works. As there is no available area for the construction of diversion works this option was given no further consideration.

### **3.3.3 Channel modification**

Channel modifications are used to improve the conveyance capacity of a stream channel, thus reducing the peak stage at the area of concern, for any given event. This option would present several different obstacles to implementation. Channel modifications to Leary's Brook would take the form of stream dredging and widening. Modifications such as these would result in major impacts to the aquatic and riparian zones of the brook and would require various permits from several different provincial and federal departments. Additionally the RRCSMP Report indicates that the bridge at Clinch Crescent East acts as a flow restriction and does not have the capacity to pass the 1:100 AEP flow and has a dominant role in generating the peak water levels observed between the HSC and Prince Philip Drive. Therefore channel modifications alone would have little to no effect on the 1:100 AEP water levels between the HSC and Prince Philip Drive, without replacing the bridge at Clinch Crescent East.

In any event it is expected that channel modifications would encounter such significant objections, from authorities, due to the environmental effects and damages to fish habitats, as to be deemed impractical and was given no further consideration.

### **3.3.4 Levee**

Levees are essentially berms or earth dams used to confine flood water to a smaller area than would otherwise be flooded without their presence. This option is the only practical option among the structural options discussed and was also the flood control approach recommended in the Rennies River Catchment Stormwater Management Plan for the area along Leary's Brook in front of the HSC.

## 4.0 CONCLUSION

Comparing the 1:100 AEP flood elevations, discussed in section 3.1 with the critical structure elevations, discussed in section 2.4 it is evident that the finished floor elevations of each structure is higher than the 1:100 AEP flood elevations, with the exception of the utility tunnel access (refer to figures A1 and A2 located in Appendix A for location of 1:100 AEP flood elevations relative to the HSC and the Agnes Cowan Hostel). Given that the 1:100 AEP flood elevations were lower than the floor elevations of the critical structures examined it is not immediately apparent that it is necessary to provide a Levee, however, due to the large flooded area created adjacent to the HSC in a 1:100 AEP flood, it was considered prudent to consider the effects of wind and wave action on the water surface of the flooded area and the corresponding inundation that would be observed. Therefore a wind and wave analysis was conducted, as discussed in section 3.2, to determine the increase in water elevation due to wave setup and runup. This analysis yielded a total maximum wind and wave induced runup that when added to the 1:100 AEP flood elevations give a maximum water level of 57.88m. This gives an inherent freeboard ranging between 0.24m and 0.34m from the maximum water level and the critical structure elevations, with the exception of the utility tunnel access.

Regardless of the inherent freeboard that currently exists it is recommended that the approach promulgated by the US FEMA (Federal Emergency Management Agency), regarding freeboard for levees, be followed. FEMA suggest that the top of levees should be set as the greater of either 1.) Flood level plus the computed wind and wave setup or 2.) Flood level plus 3 feet (900mm). Following the later criteria the top of levee should be set at 58.46m. Figure A3 and A4 located in Appendix A illustrates the location of the proposed levee along with several sections. Strictly speaking this levee would not confine flood waters and would primarily provide protection to the HSC and the Agnes Cowan Hostel from inundation resulting from wave action due to wave setup and runup, and would act as a breakwater rather than a levee. As current topography of the HSC site slopes towards Leary's brook a berm could actually create flooding issues, as the runoff from the HSC site would be retained by a levee and held immediately adjacent to the HSC. Therefore it is recommended that culverts be placed at intervals along the length of the berm to allow runoff from the HSC site to continue towards Leary's Brook (Figure A3 and A4 located in Appendix A illustrates the proposed berm location along with several berm sections).

As mentioned the 1:100 AEP flood elevation is above the estimated utility tunnel access elevation and therefore the potential to flood the tunnel exists. It should also be noted that this berm would require the placement of fill in the floodway fringe which will impact the 1:100 AEP flood levels.

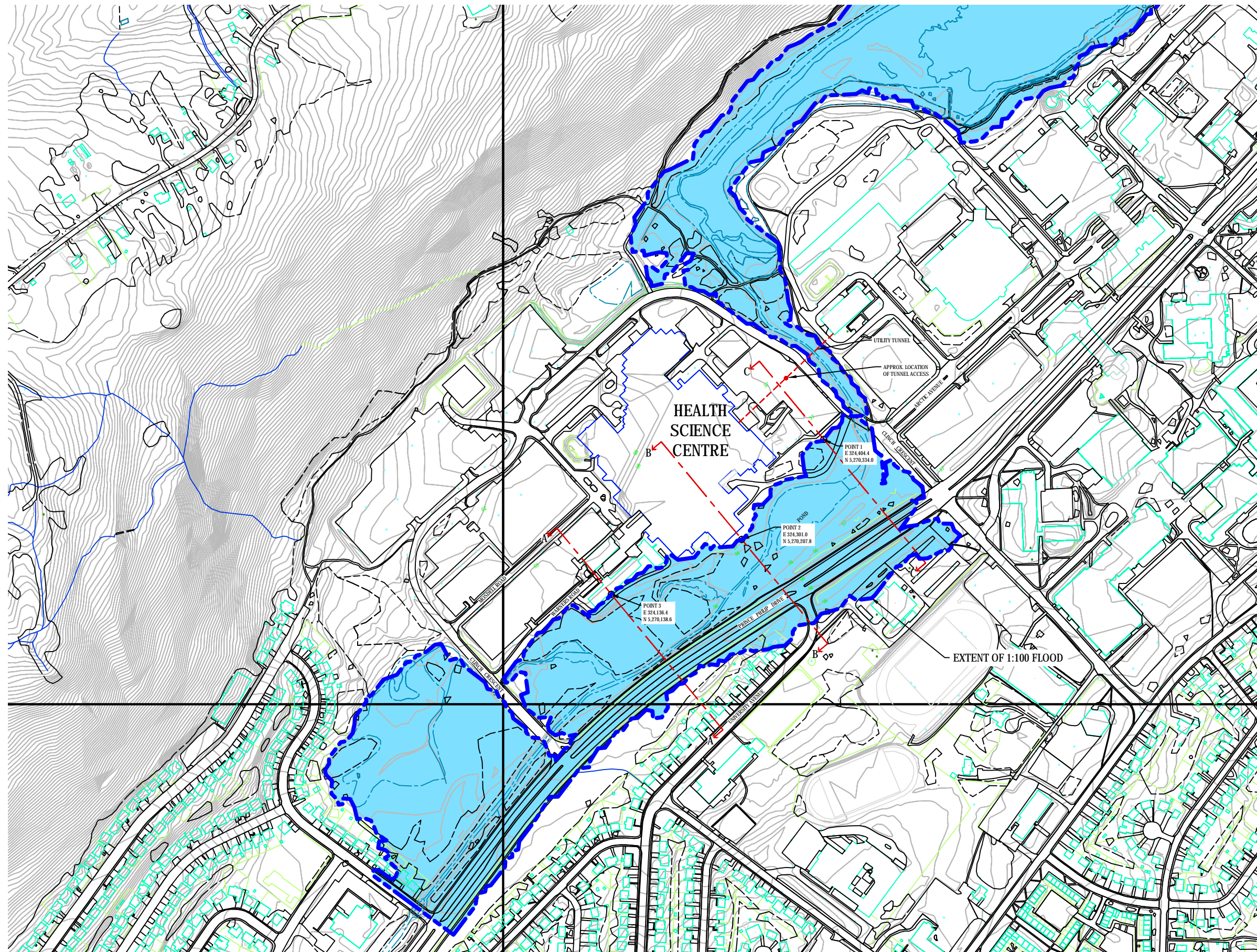
## 5.0 RECOMMENDATIONS

The following paragraphs summarize the recommendations:

1. The proposed berm as discussed in the previous section should be constructed of imported select fill and finished with topsoil and hydroseeded. Several culverts are also included to allow for drainage (as discussed in Section 4.0). In addition a paved access road, to allow emergency vehicle access to the east side of the HSC, is included. The estimated cost of the berm and road work is \$283,400. An item by item breakdown of the estimate is located in Appendix F. As the construction of a berm in the floodway fringe would impact the observed 1:100 AEP flood elevations, an evaluation, to determine those impacts, should be conducted during the detailed design stage.
2. This study did not include examining storm drains and sanitary sewers that service the hospital. It is recommended that impact of the 1:100 AEP flood level with respect to these services be examined.
3. It is also recommended that the potential impacts of flooding the utility tunnel between the utility annex and the HSC be examined more closely. If flooding of the utility tunnel is found to pose significant risks then flood proofing measures for the tunnel/tunnel access should be considered.
4. As discussed, in section 3.3.3, the bridge at Clinch Crescent East plays a dominant role in generating the 1:100 AEP flood levels observed in front of the HSC. It is recommended that hydraulic modeling be conducted to determine the potential reductions in the 1:100 AEP flood levels that could be realized by replacing the Clinch Crescent East Bridge.

## **APPENDIX A**

## **DRAWINGS**



**NOTES:**

1. 100 YEAR FLOOD AS IDENTIFIED ON PLANS IS BASED ON NEW\_FLOODPLAIN\_HEALTH\_SCIENCES.SHP OBTAINED FROM THE CITY OF ST. JOHN'S.
2. 100 YEAR FLOOD ELEVATIONS IDENTIFIED ON PROFILES ARE BASED ON ELEVATIONS PROVIDED BY THE CITY OF ST. JOHN'S.
3. GRADE INDICATED ON PROFILES BASED ON CAD DRAWINGS CONT3235270, CONT3235271, CONT3245270 AND CONT3245271 OBTAINED FROM THE CITY OF ST. JOHN'S.



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AMEC Americas Limited



DRAWN BY :

V.A.H.

SCALE :

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DATE :

2014/07/18

PROJECT : HEALTH SCIENCES CENTRE  
ST. JOHN'S, NL  
FLOOD PLAIN STUDY

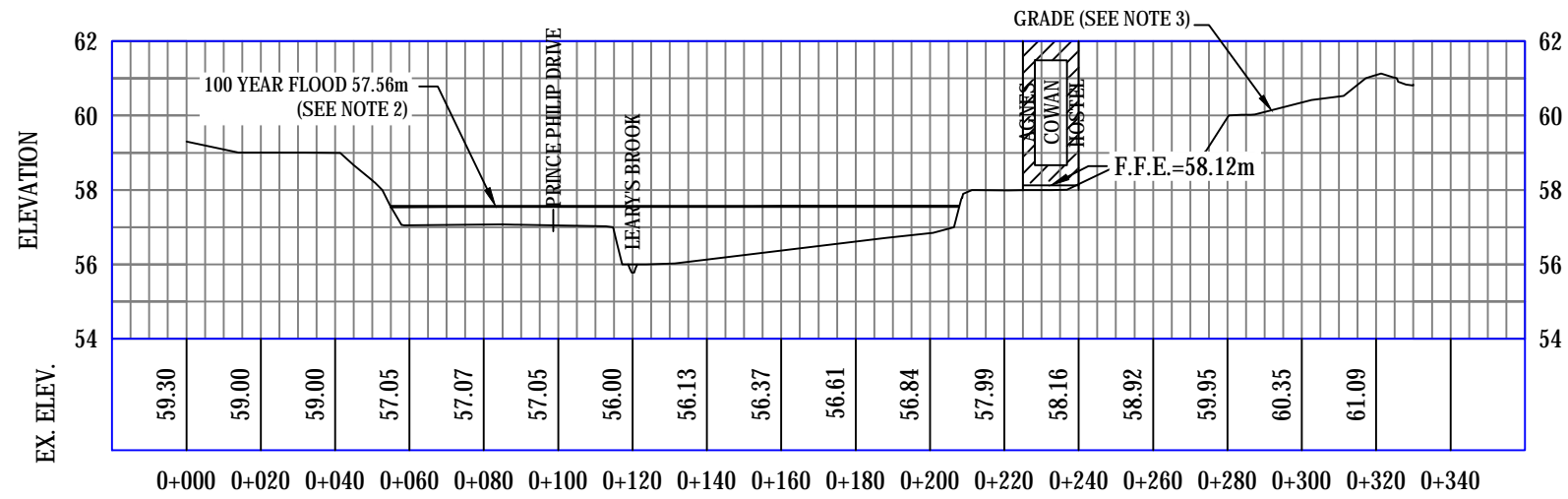
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100 YEAR STORM BOUNDARIES  
AND PROFILE LOCATIONS

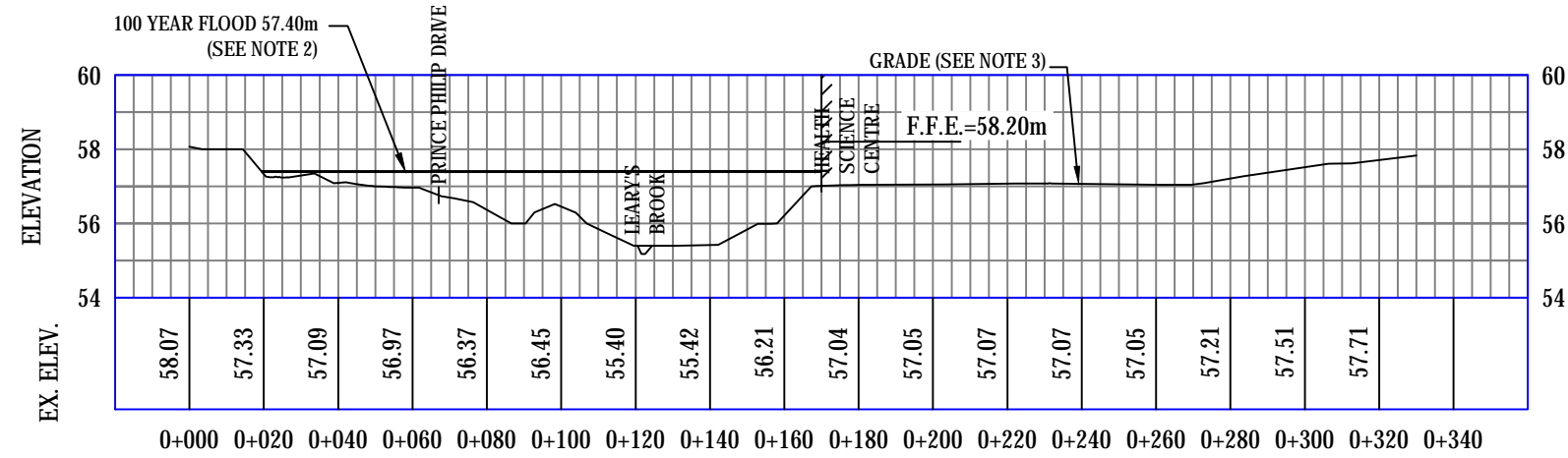
FIGURE :

A1

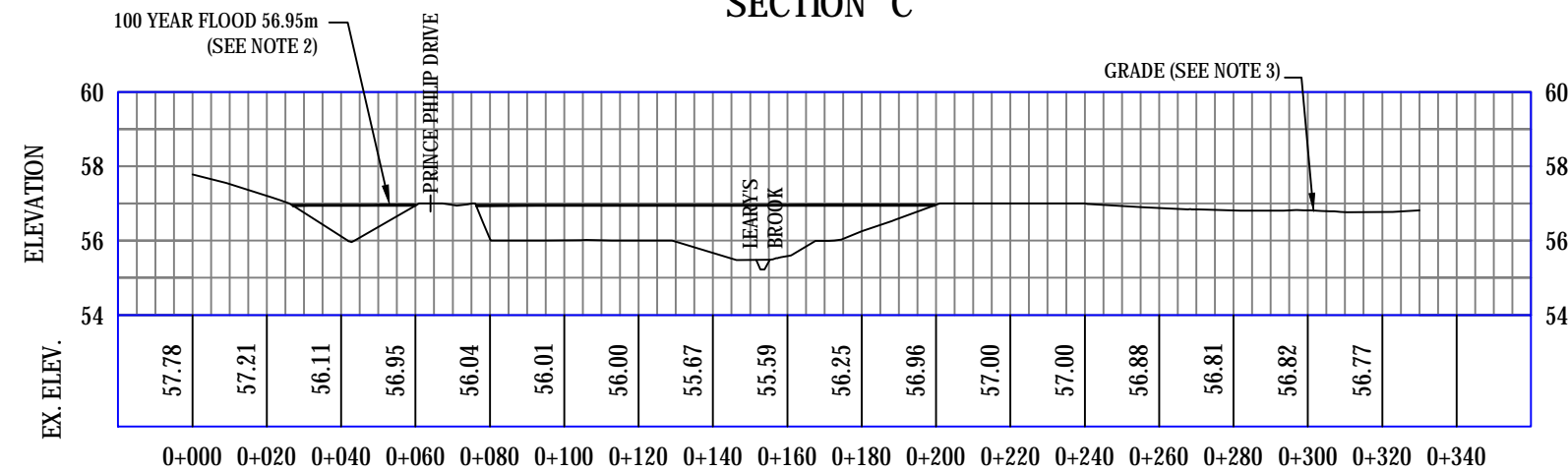
### SECTION 'A'



### SECTION 'B'



### SECTION 'C'



#### NOTES:

- 100 YEAR FLOOD AS IDENTIFIED ON PLANS IS BASED ON NEW\_FLOODPLAIN\_HEALTH\_SCIENCES.SHP OBTAINED FROM THE CITY OF ST. JOHN'S.
- 100 YEAR FLOOD ELEVATIONS IDENTIFIED ON PROFILES ARE BASED ON ELEVATIONS PROVIDED BY THE CITY OF ST. JOHN'S.
- GRADE INDICATED ON PROFILES BASED ON CAD DRAWINGS CONT3235270, CONT3235271, CONT3245270 AND CONT3245271 OBTAINED FROM THE CITY OF ST. JOHN'S.



DRAWN BY :

V.A.H.

SCALE :

HOR. 1 : 2000  
VER. 1 : 200

DATE :

2014/07/18

PROJECT : HEALTH SCIENCES CENTRE  
ST. JOHN'S, NL  
FLOOD PLAIN STUDY

SUBJECT :

100 YEAR STORM  
SECTIONS A, B and C


FIGURE :

A2

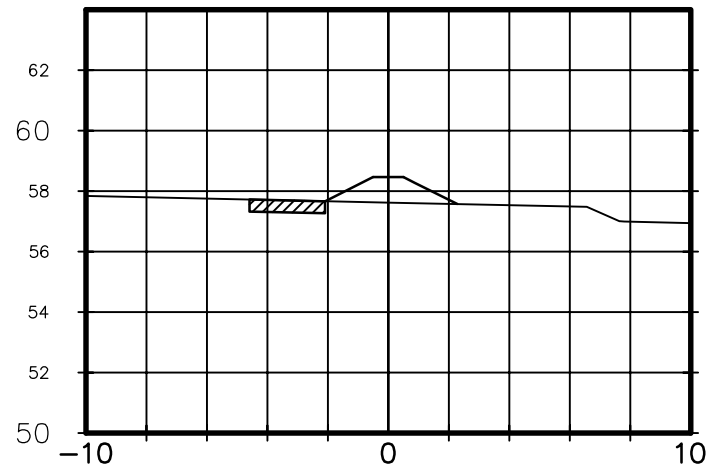




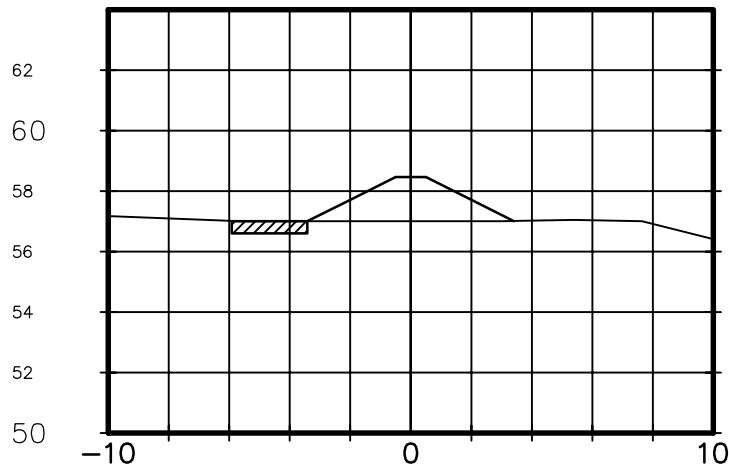
P:\Dwg\CM\Floodplain Study\FIGURE-A2-A3 - 11x17 - Aug 6 14.dwg  
 Aug 06 2014 11:14pm jallcock@tdsbn

 AMEC Americas Limited	DRAWN BY : V.A.H. / J.L.K.	SCALE : 1 : 1000	DATE : 2014/08/06	PROJECT : HEALTH SCIENCES CENTRE ST. JOHN'S, NL FLOOD PLAIN STUDY	SUBJECT : ROAD / BERM LOCATION	FIGURE : A3
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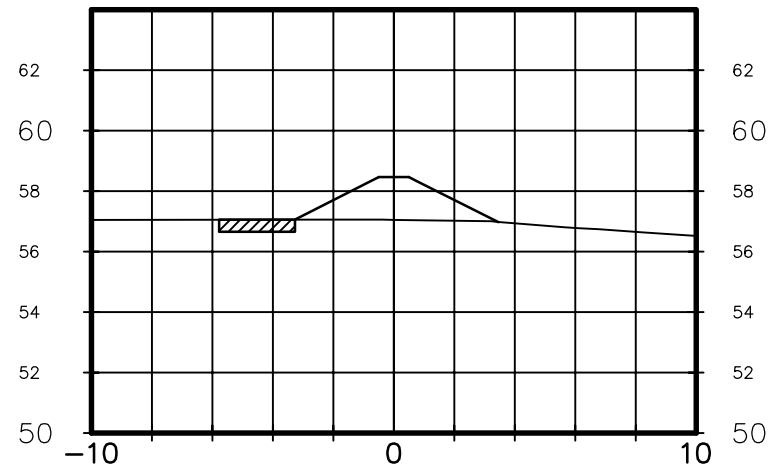
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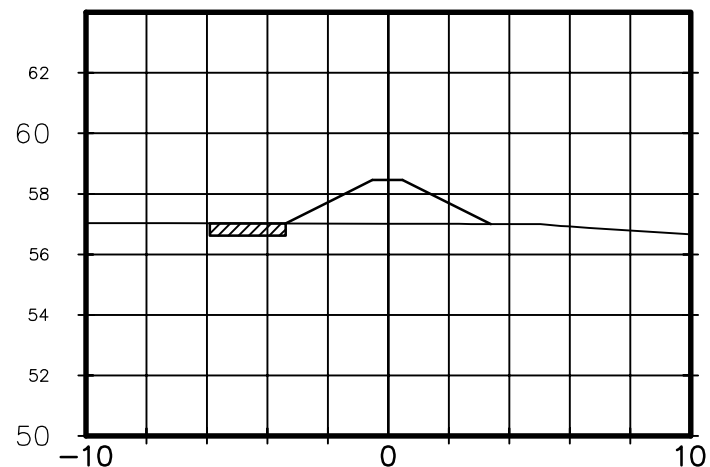
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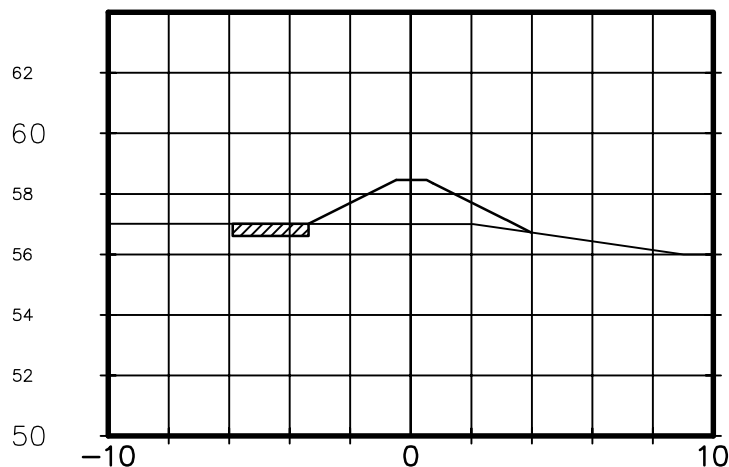
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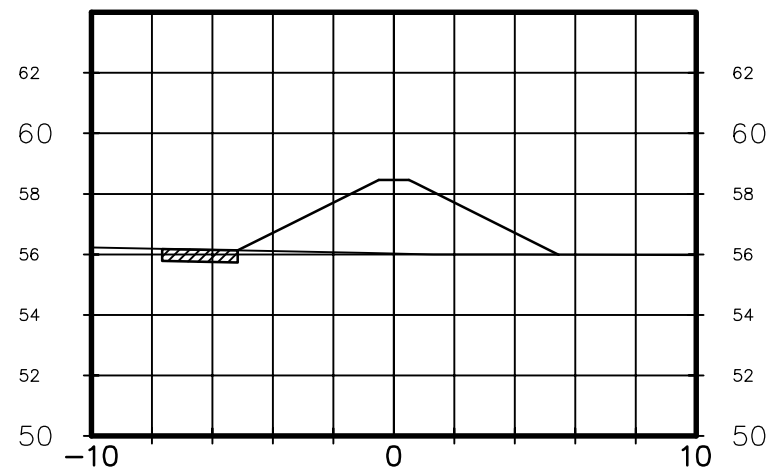
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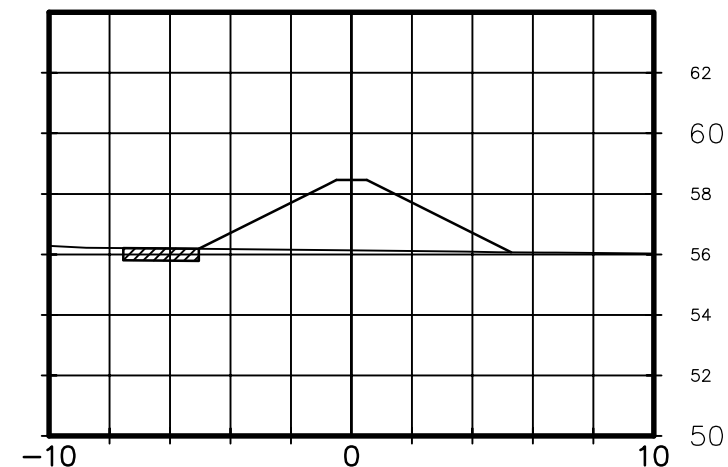
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P:\Dwg\CV\Floodplain Study\FIGURE-A2-A3 - 11x17 - Aug 6 14.dwg  
Aug 06 2014 11:19pm jalkobktdson

AMEC Americas Limited



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J.L.K.

SCALE :

1 : 250

DATE :

2014/08/06

PROJECT : HEALTH SCIENCES CENTRE  
ST. JOHN'S, NL  
FLOOD PLAIN STUDY

SUBJECT :

ROAD / BERM SECTIONS

FIGURE :

A4

**APPENDIX B**

**RENNIES RIVER CATCHMENT STORMWATER PLAN**

# Rennies River Catchment Stormwater Management Plan

## Final Report





123097.00 • Final Report • April 15, 2014

Prepared for:

**ST. JOHN'S**  
NEWFOUNDLAND AND LABRADOR, CANADA

Prepared by:

**CBCL**  
**CBCL LIMITED**  
Consulting Engineers

Final	Alexander Wilson	04/15/2014	Greg Shepapr
Draft Final	Alexander Wilson	01/29/2014	Greg Sheppard
Draft	Alexander Wilson	11/04/2013	Greg Sheppard
<b><i>Issue or Revision</i></b>	<b><i>Reviewed By:</i></b>	<b><i>Date</i></b>	<b><i>Issued By:</i></b>
 <p><b>CBCL LIMITED</b> Consulting Engineers</p> <p>This document was prepared for the party indicated herein. The material and information in the document reflects CBCL Limited's opinion and best judgment based on the information available at the time of preparation. Any use of this document or reliance on its content by third parties is the responsibility of the third party. CBCL Limited accepts no responsibility for any damages suffered as a result of third party use of this document.</p>			

April 15, 2014

City of St. John's  
PO Box 908  
10 New Gower Street  
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Attention: Mr. Dave Wadden, M.Eng. P.Eng.

RE: Rennie's River Catchment Stormwater Management Plan  
CBCL Project No. 123097

Dear Mr. Wadden:

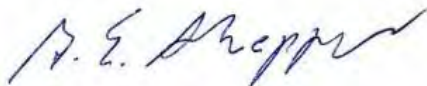
Please find enclosed six copies of the final report for the above noted study as well as a compact disk containing a pdf copy of the report. Due to the model file sizes, the remaining report deliverables will follow on a USB flash drive. Further, as we have yet to present the final report, a copy of the final report presentation will follow at a later date.

We have enjoyed working on this challenging assignment and have appreciated the feedback provided by the City throughout the project. We look forward to continuing to build on our working relationship with the City on future projects.

Please contact me with any questions related to this final study report.

Sincerely,

**CBCL LIMITED**  
**Consulting Engineers**



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www.cbcl.ca

**Solving  
today's  
problems  
with  
tomorrow  
in mind**

# Contents

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## EXECUTIVE SUMMARY

The Rennies River watershed has an area of approximately 32 km<sup>2</sup> and contains several major water courses, including Yellow Marsh Stream, Ken Brook, Leary's Brook and Rennies River. Runoff from this catchment ultimately discharges to Quidi Vidi Lake. During significant rainfall events, flooding has occurred at locations along Ken Brook, Leary's Brook and Rennies River. Flooding has, at a minimum, been inconvenient for the residents of the City of St. John's and, at other times, has resulted in major public and private property damage. Consequently, the City has identified a need for an overall plan to address flooding issues in the Rennies River catchment. One of the key components of this plan is a prioritized list of flood protection infrastructure improvements.

The detailed scope of work for this study is as follows:

- Carry out field surveys to obtain structure data;
- Update intensity-duration-frequency curves to include most recent rainfall data and estimate hyetographs to reflect changing climate conditions;
- Determine 1:20 and 1:100 annual exceedance probability flood flows by using statistical analysis;
- Assemble hydrologic and hydraulic models of the study areas;
- Calibrate the hydrologic and hydraulic models using available data;
- Prepare hydrologic and hydraulic models of the study areas to reflect potential future land uses;
- Complete sensitivity analysis on the hydrologic and hydraulic models;
- Prepare floodplain and flood hazard maps for the 1:20 and 1:100 AEP events for existing development conditions;
- Prepare floodplain and flood hazard maps for the 1:20 and 1:100 AEP event for future development conditions;
- Develop preliminary designs for methods of flood control;
- Identify areas with erosion problems and develop remedial plans; and
- Prepare preliminary cost opinions and designs for the optimum flood and erosion control methods selected.

Several flood protection approaches were evaluated using the hydrologic and hydraulic models developed for this study, and the most optimum flood protection measures have been recommended for the City's consideration. In terms of overall impact on the study area, the most significant

recommended flood protection improvement is a weir proposed for the east end of Long Pond. The construction of a weir at this location will result in reduced flooding downstream of Long Pond.

The flood control improvements recommended for downstream of Long Pond have been designed to function with the weir at Long Pond in place. Consequently, the weir at Long Pond must be constructed before the downstream improvements can be implemented. Given the extents of the flooding experienced at locations downstream of Long Pond, these areas are considered by the City to be high priority areas. Therefore, it is recommended that the weir at Long Pond be given first priority, and the two problem areas located downstream of Long Pond be given second priority. It is recommended that the remaining flood improvement recommendations be implemented in order from downstream to upstream.

The following table summarizes the recommended flood improvement measures. Only one of the options presented for location 1 needs to be implemented. The final decision regarding which of the location 1 options to implement will be made by the Department of Planning, Development and Engineering's senior management in consultation with Council.

Priority	Description of Location	Cost Opinion
1	Location 3: Weir at outlet of Long Pond	\$1,979,000
2	Location 1, Option A: Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road – Berms & Walls only	\$1,173,000
	Location 1, Option B: Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road – New Channel and bridge	\$3,891,000
	Location 1, Option C: Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road – Raised parking lot	\$1,379,000
2	Location 2: Upstream of Carpasian Road Bridge	\$27,000
3	Location 4: Clinch Crescent East to Clinch Crescent West	\$342,000
4	Location 5: Wicklow Street to Thorburn Road	\$294,000
5	Location 7: O'Leary Avenue Bridge	\$847,000
6	Location 8: Downstream of Mews Place	\$38,000

Erosion control improvements along Rennie's River and Leary's Brook can be accomplished using a cellular confinement system as described in the report. It is estimated that approximately 4000 m<sup>2</sup> of the river banks need to be rehabilitated. Based on using a cellular confinement system, the cost opinion to do this work is \$567,000. All cost opinions presented in the report include engineering, construction and HST.

CBCL recommends that the City move forward with the design and implementation of the proposed flood and erosion control improvements. Further, CBCL recommends that the Provincial Department of Environment and Conservation and the Department of Fisheries and Oceans be consulted during the design of the proposed infrastructure improvements.

## CHAPTER 1 INTRODUCTION

The Rennies River watershed has an area of approximately 32 km<sup>2</sup> and contains several major water courses, including Yellow Marsh Stream, Ken Brook, Leary's Brook and Rennies River. Runoff from this catchment ultimately discharges to Quidi Vidi Lake. During significant rainfall events, flooding has occurred at locations along Ken Brook, Leary's Brook and Rennies River. Flooding has, at a minimum, been inconvenient for the residents of the City of St. John's (City) and, at other times, has resulted in major public and private property damage. Consequently, the City has identified a need for an overall plan to address flooding issues in the Rennies River catchment. One of the key components of this plan is a prioritized list of flood protection infrastructure improvements.

In October 2012, the City issued a Request for Proposals (RFP) for a stormwater management study for the Rennies River drainage catchment. CBCL Limited (CBCL) was awarded this study in November 2012. Our findings are presented in this report.

### 1.1 Study Scope

The scope of work includes the following tasks:

- Carry out field surveys to obtain structure data;
- Update intensity-duration-frequency (IDF) curves to include most recent rainfall data and estimate hyetographs to reflect changing climate conditions;
- Determine 1:20 and 1:100 annual exceedance probability (AEP) flood flows by using statistical analysis;
- Assemble hydrologic and hydraulic models of the study areas using XPSWMM;
- Calibrate the hydrologic and hydraulic models using available data;
- Prepare hydrologic and hydraulic models of the study areas to reflect potential future land uses;
- Complete sensitivity analysis on the hydrologic and hydraulic models;
- Prepare floodplain and flood hazard maps for the 1:20 and 1:100 AEP events for existing development conditions;
- Prepare floodplain and flood hazard maps for the 1:20 and 1:100 AEP event for future development conditions;
- Develop preliminary designs for methods of flood control;
- Identify areas with erosion problems and develop remedial plans; and
- Prepare preliminary cost opinions and designs for the optimum flood and erosion control methods selected.

## CHAPTER 2 BACKGROUND INFORMATION

### 2.1 Historical Flooding

A search of online news articles and the provincial government website yielded a list of past flood events in the Rennies River catchment caused by river flooding. These events are summarized below:

- April 11, 1986: Rainfall of 110 mm caused flooding along Leary's Brook and Rennies River. The Avalon Mall parking lot flooded, and there was an estimated 30 cm of water covering Prince Philip Drive between the entrance to the Health Sciences Centre and the CBC building. The water level in Rennies River reportedly rose 1.8 m above the normal water level, destroying approximately 100 m of walking trail and causing severe flooding at Pringle Place.
- September 19-20, 2001: Post-tropical storm Gabrielle deposited 175 mm of rain in the city of St. John's, much of which fell within 6 hours or less, according to Environment Canada (EC). Flooding caused road closures on Kenmount Road, The Boulevard, Portugal Cove Road, Prince Philip Parkway and Clinch Crescent West. Carnell Drive was flooded, as was the Avalon Mall parking lot. As well, forty-five stores located in the Avalon Mall sustained flood damage.
- November 16, 2004: Rainfall caused minor flooding in St. John's. For example, water built up on Prince Philip Drive near the west entrance to the Health Sciences Centre, at Clinch Crescent West.
- April 11-12, 2005: Rainfall of 70 mm caused flooding along Leary's Brook, both upstream and downstream of the Avalon Mall, the Clinch Crescent West entrance to the Health Sciences to be temporarily closed, and the normal water level of Long Pond to rise by between 1 and 2 m.
- November 29, 2008: This storm dropped 100 mm of rain on the Northeast Avalon, most of which fell in a 3 hour period, according to a CBC News report. The storm caused Rennies River to overtop its banks near the entrance to Quidi Vidi, flooding the King George V Soccer Pitch, causing an estimated \$500,000 in damages to the artificial turf. Since the incident, a berm has been constructed between Rennies River and the field, near the shoreline of Quidi Vidi Lake.



- September 20-24, 2010: Rainfall associated with Hurricane Igor resulted in flooding at several locations along Rennies River and Leary's Brook, including Fieldian Grounds, Pringle Place, Vaughan Place and the Prince Phillip Parkway in the vicinity of the CBC Building.

## 2.2 Previous Studies

A literature review of previous flood studies was conducted to assess the underlying mechanisms of flooding, as well as to identify any areas which experience frequent flooding. In 2002, H.T. Kendall and Associates Ltd. completed a flood study titled *Ken Brook and Leary's Brook Floodplain Delineation Study*, and in 2006 Kendall Engineering Ltd. completed a floodplain mapping study of Rennies River, Virginia River and Quidi Vidi River, titled *Quidi Vidi Lake Tributary Flood Plain Delineation*. The findings of these two studies are summarized in the following sections.

### 2.2.1 Ken Brook and Leary's Brook Floodplain Delineation Study

In October 2002, H.T. Kendall and Associates Ltd. completed a floodplain mapping study for the City that examined the extent of flooding, identified flood hazard areas and proposed flood mitigation strategies for Ken Brook and Leary's Brook to the entrance to Long Pond. The study included estimating the 1:100 AEP flows using HEC-HMS and statistical techniques and delineating the resulting floodlines by transposing the water surface elevations determined from the HEC-RAS model on to City mapping.

The results indicated that flooding is common along Ken Brook, especially in the vicinity of undersized culverts located on private properties. It was also noted that the culverts located at Clinch Crescent West and Pippy Place, as well as bridges located at Thorburn Road, Oxen Pond Foot Bridge, Wicklow Street and Clinch Crescent East do not have capacity to pass the 1:100 AEP flow. In addition, flood mitigating measures were advised in the river banks downstream of the Pippy Place culverts, behind the Seaboard Building, downstream of the O'Leary Avenue Bridge, upstream and downstream of the Wicklow Street Bridge, along Prince Philip Drive and upstream of the Clinch Crescent East Bridge.

### 2.2.2 Quidi Vidi Lake Tributary Flood Plain Delineation

Kendall Engineering Ltd. completed the Quidi Vidi Lake Tributary Flood Plain Delineation study in August 2006. The study used HEC-HMS to estimate flood flows along the rivers, then modelled river cross sections in HEC-RAS to determine the extent of flooding. The hydraulic model for Rennies River extended 1,300 m from the entrance to Quidi Vidi Lake and included 42 cross sections and five river crossings, namely: Carnell footbridge, Carnell Bridge, footbridge at Loblaws, Kings Bridge Road Bridge, and Portugal Cove Road Bridge.

The study found that two large areas are prone to flooding during the 1:100 AEP flood; Portugal Cove Road Bridge and the floodplain immediately upstream and downstream, as well as the floodplain from Kings Bridge Road Bridge to Quidi Vidi Lake. To mitigate flooding near the Portugal Cove Road Bridge, the study recommends alterations to the bridge, which include removing sediment beneath the bridge, removing concrete obstructions in the downstream channel and raising the north bank of Rennies River for approximately 150 m upstream of the bridge. However,

even with these modifications, a large portion of the soccer pitch at Fieldian Grounds and the Riverdale Tennis Club grounds would still be flooded. To minimize the extend of flooding between Kings Bridge Road Bridge and Quidi Vidi Lake, the report suggests constructing berms or levees along the north bank of Rennies River from Kings Bridge Road Bridge to Carnell Bridge and raising the footbridge at Loblaws. However, these alterations will not prevent all the flooding problems; a large portion of the Loblaws parking lot as well as sections of Carnell Drive and Lake Avenue will still be within the flood limits.

## CHAPTER 3 DATA COLLECTION AND ANALYSIS

### 3.1 Data Collection

Several sources of data were required to accurately assemble the hydrologic and hydraulic models. Items included in the data collection process are as follows:

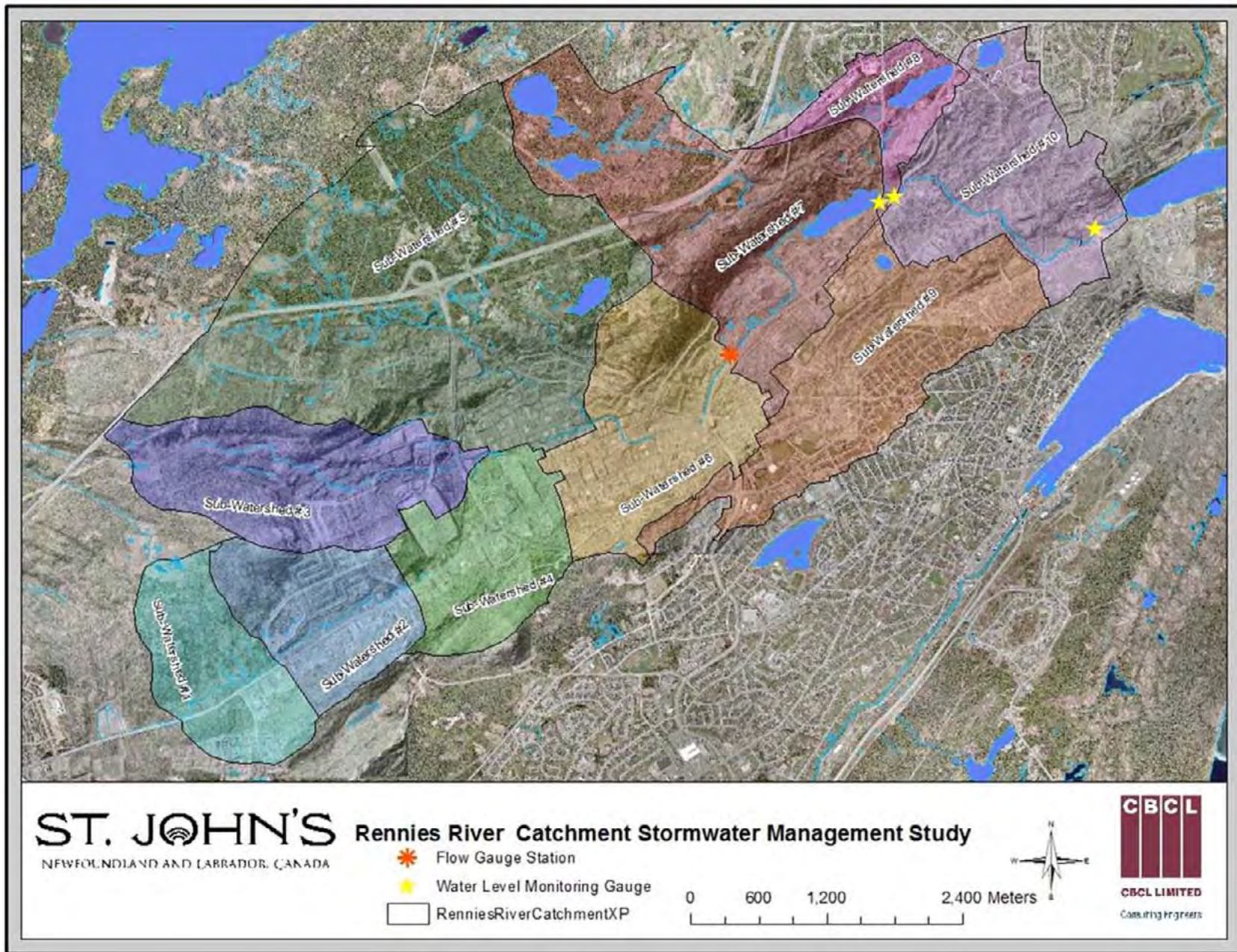
- Aerial photography of the study area provided by the City;
- LiDAR data of the study area provided by the City;
- Zoning and property mapping provided by the City;
- Future development data provided by the City;
- Bridge construction drawings provided by the City;
- Construction drawings of flood control structure for Quidi Vidi Lake provided by the City;
- Watershed delineations provided by the City;
- Water levels measurement provided by the City;
- Flow gauging data provided by Environment Canada (EC);
- Precipitation data provided by EC; and
- Hydraulic structure details obtained from field investigation.

#### 3.1.1 Calibration Data

Meteorological data and hydrologic data, including flows and water levels, were obtained for use in this study.

There is one long-term flow gauge in operation on Leary's Brook. The gauge is located upstream of Wicklow Street and is operated by the Water Survey of Canada under the name Leary's Brook at Prince Philip Drive (EC #02ZM02). The flow gauge has a contributing drainage area of approximately 17.8 km<sup>2</sup> and has been in operation since 1985. There is no flow gauge located within the lower reaches of the Rennies River. However, three water level monitoring gauges are in operation within the Rennies River study area including: Long Pond Bridge, Kings Bridge Road, and Prince Philip Drive at Rennies River. Figure 3-1 shows the location of these gauging stations.

FIGURE 3-1 FLOW AND WATER LEVEL GAUGE STATIONS



The use of the above data in the model calibration is discussed further in Chapter 6.

### **3.1.2 Detailed Topographic Data**

To generate the topographic information for the model, LiDAR point data (with a 1-m grid resolution) provided by the City was used to develop a topographic grid system. LiDAR mapping of the study area was produced at 1-m resolution.

### **3.1.3 Hydraulic Structure Details**

Photos, measurements and notes from field investigations of the hydraulic structures were collected. Thirty structures in total were investigated and assessed for hydraulic capacity. The hydraulic structure capacity assessment is discussed further in Chapter 6.

Bridge data was collected for the eleven bridge crossings within the study area for the hydraulic model. These represent critical points of energy losses and have a dominant role in generating peak water levels. Bridge opening geometry and bridge deck elevations were compiled from construction drawings and field measurements.

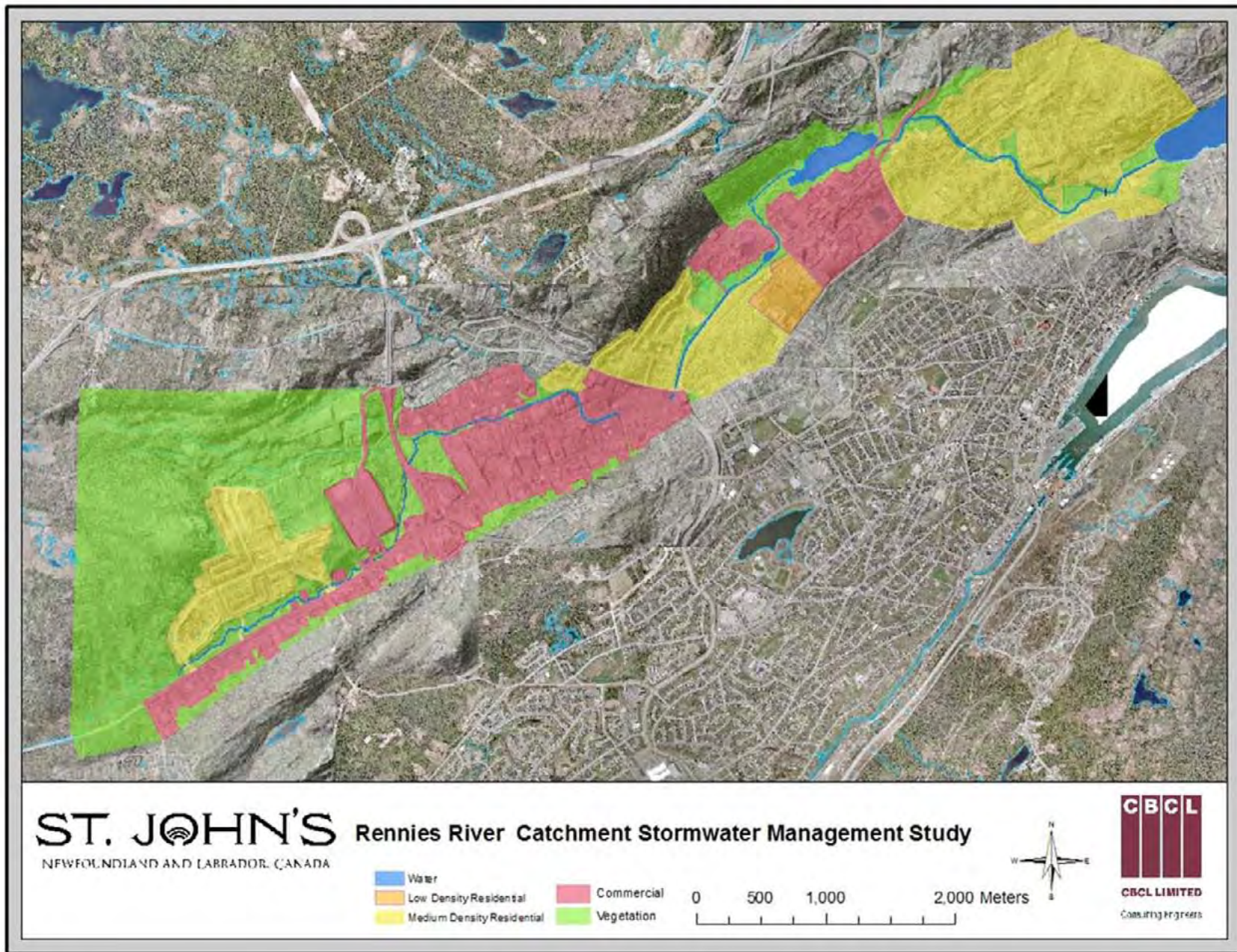
## **3.2 Data Analysis**

### **3.2.1 Land Use Mapping**

Land use mapping of the study area was developed using the aerial photography, zoning and property mapping and is presented in Figure 3-2. The land use mapping was used to estimate roughness coefficients for the sub-watersheds (this hydrologic parameter affects the time of concentration or lag time. As shown in Figure 3-2, land uses were divided into the following categories:

- Medium Density Residential;
- Low Density Residential;
- Commercial;
- Water; and
- Vegetation.

FIGURE 3-2 LAND USE MAPPING



### 3.2.2 Watershed Delineation and Watershed Properties

Watershed delineations provided by the City were reviewed and refined using the LiDAR data to delineate the sub-watersheds along the Rennies River catchment. Maps of the watershed delineations are presented in Figure 3-1.

Watershed characteristics were estimated for each sub-watershed and are presented in Table 3-1. The watershed characteristics were estimated using the LiDAR data, aerial photography, land use mapping, and the City's Subdivision Design Manual.

**TABLE 3-1 WATERSHED CHARACTERISTICS**

Name	Area (ha)	Subbasin Width (m)	Slope (%)	Percent Impervious Area (%)	Manning's Roughness Values		Depression Storage (mm)		Average Capillary Suction (mm)	Saturated Hydraulic Conductivity (mm/h)	Initial Moisture Deficit
					Imperv. Area	Perv. Area	Imperv. Area	Perv. Area			
Sub-Watershed1	179.69	595.83	3.1	9.4	0.02	0.5	1	2.5	200	0.001	0.3
Sub-Watershed2	183.28	477.92	4.8	20.6	0.02	0.5	1	2.5	200	0.001	0.3
Sub-Watershed3	285.18	476.69	3.8	4.3	0.02	0.5	1	2.5	200	0.001	0.3
Sub-Watershed4	195.89	566.11	5.2	23.6	0.02	0.5	1	2.5	200	0.001	0.3
Sub-Watershed5	845.24	1086.96	2.5	9.8	0.02	0.5	1	2.5	200	0.001	0.3
Sub-Watershed6	284.48	560.43	4.9	39	0.02	0.5	1	2.5	200	0.001	0.3
Sub-Watershed7	488.66	779.78	6.8	12.7	0.02	0.5	1	2.5	200	0.001	0.3
Sub-Watershed8	96.15	316.31	8.8	31	0.02	0.5	1	2.5	200	0.001	0.3
Sub-Watershed9	336.99	469.90	4.7	43.1	0.02	0.5	1	2.5	200	0.001	0.3
Sub-Watershed10	288.66	702.95	7.6	37.2	0.02	0.5	1	2.5	200	0.001	0.3

## CHAPTER 4 UPDATE OF IDF CURVES AND DESIGN HYETOGRAPHS

### 4.1 IDF Curves

IDF curves describe rainfall patterns for a particular geographical area. They are created by performing statistical analysis on rainfall data recorded by a rain gauge. The result is a set of curves representing rainfall intensities for a range of storm durations for various return periods, typically the 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 AEP.

For this study, CBCL updated IDF curves for the rain gauge located at the St. John's International Airport in order to account for the significance of recent extreme rainfall events.

#### 4.1.1 Existing Data

EC maintains rain gauges throughout Newfoundland and Labrador and, traditionally, has been responsible for creating IDF curves from the recorded data. The rain gauge nearest to the study area is located at the St. John's International Airport (EC #8403506) at an elevation of 131 m. A tipping bucket rain gauge was used to recorded data at 5-minute intervals until the end of 1996. In 1997, the tipping bucket was replaced with a Fisher and Porter rain gauge which archives data every six hours. The IDF curves for the Airport gauge were last updated by EC in 1996 and are found in Appendix A.

#### 4.1.2 Additional Data

EC continues to record rainfall amounts at the St. John's Airport rain gauge; however, intensities are not currently being recorded.

The City owns and operates three rain gauges, located at Ruby Line, Blackler Avenue and Windsor Lake. The Windsor Lake gauge is located approximately 1.6 km southwest of the previously operated St. John's Airport gauge. It observes rainfall over the Windsor Lake and Broad Cove River watershed, as well as parts of Outer Cove Brook, Stick Pond Brook, Coaker's Meadow Brook, Virginia River and Rennie's River. The Windsor Lake gauge is a Met One tipping bucket gauge, installed at an elevation of 159 m. It was installed in December of 1998 and records data at 1-minute intervals. The close proximity of the two gauges gives an initial indication that the two data sets can be



combined. The report titled *Rainfall Distribution in the City of St. John's: Temporal Distribution, Spatial Variation, Frequency Analysis, and Tropical Storm Gabrielle* examined the appropriateness of combining the two data sets by comparing overlapping data recorded between 1999 and 2001 at the two gauges. This study determined a correlation coefficient of 0.9 for the daily rainfall comparison, implying a strong relationship and suggesting that the observed rainfall at both locations is uniform.

#### 4.1.3 Update

Data from the Windsor Lake gauge was obtained from the City for 2001-2012 in 5-minute intervals. Annual maximums for 5, 10, 15, and 30-minute and 1, 2, 6, 12 and 24-hour intervals were extracted and combined with those data sets for the St. John's Airport gauge. Summaries of the annual maximum data for these durations are presented in Appendix B.

The largest 6, 12, and 24-hour rainfall maximums on record occurred in 2001 during Tropical Storm Gabrielle. The IDF update completed by the City in 2002 omitted this storm from the data series since at the time it was considered an outlier when compared to the remaining data set. Since 2002, there have been two additional rainfall events with recorded precipitation amounts that are larger than the remaining data sets. These events occurred in 2007 (Tropical Storm Chantal) and 2010 (Hurricane Igor). However, the data series indicates that Tropical Storm Gabrielle is the largest precipitation event recorded at the Windsor Lake gauge and could still be considered an outlier.

Through discussions with City officials, CBCL learned that the recorded rainfall during Hurricane Igor was likely underestimated at all of the City's gauges. The City indicated that flows recorded at hydrometric stations throughout the City were higher during Hurricane Igor than Tropical Storm Gabrielle. The actual 24-hour rainfall is estimated to be between 180 and 200 mm rather than the recorded 113.8 mm. This variation in actual and recorded precipitation can be attributed to the high winds experienced during the storm blowing rain out of the collection device. Considering this underestimation of the Hurricane Igor rainfall amount, we decided to retain Tropical Storm Gabrielle record for the statistical analysis, as omitting it would likely underestimate the return period rainfall amounts.

Statistical analyses were performed on each of the eight data sets to update the IDF curves. Several distributions were examined including the Lognormal, 3-Parameter Lognormal (3PLN), Log Pearson Type III and the Gumbel distributions. Each distribution was examined based on visual goodness-of-fit and several statistical tests. The 3PLN distribution was chosen for the IDF update. Distribution plots and screening tests are contained in Appendix C.

#### 4.1.4 Results

The updated IDF curves for the 1:2, 1:5, 1:10, 1:20, 1:50 and 1:100 AEP are presented in Figure 4-1. The intensities estimated for each return period and storm duration are presented in Table 4-1.

FIGURE 4-1 IDF CURVES

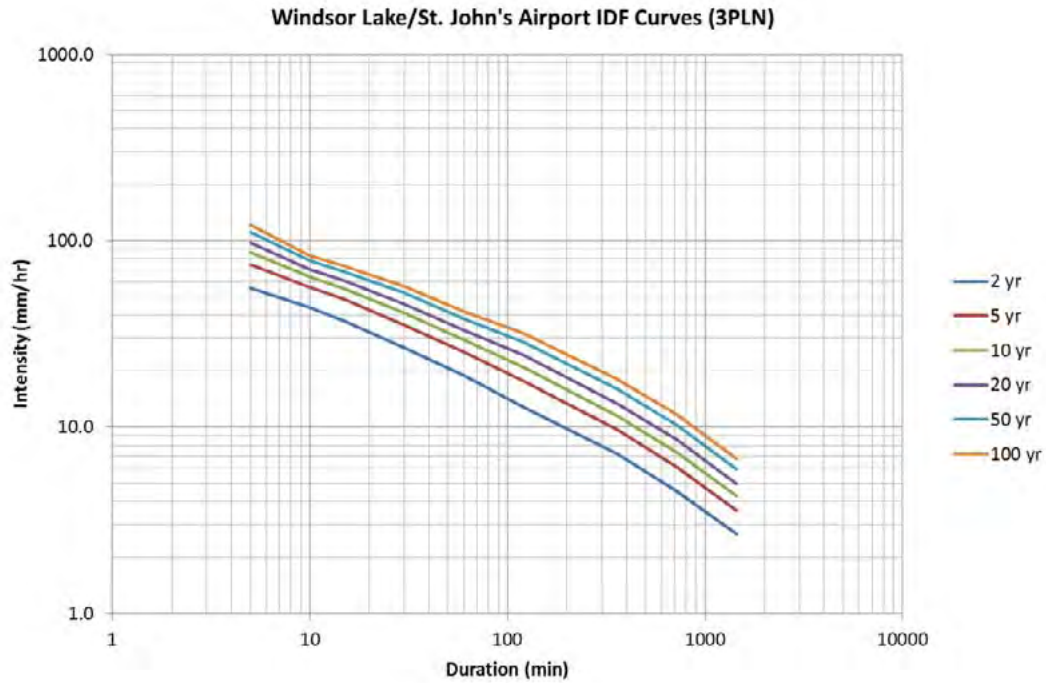


TABLE 4-1 UPDATED IDF RAINFALL INTENSITIES

Duration (min)	Intensity (mm/hr)																				
	2yr	L95%	U95%	5yr	L95%	U95%	10yr	L95%	U95%	20yr	L95%	U95%	25yr	L95%	U95%	50yr	L 95%	U95%	100yr	L 95%	U95%
5	55.44	50.12	60.76	74.28	68.96	79.60	86.16	80.84	91.48	97.08	91.76	102.40	100.56	95.24	105.88	110.88	105.56	116.20	121.20	115.88	126.52
10	43.68	39.99	47.32	56.52	52.87	60.20	64.20	60.29	67.63	70.20	66.77	74.11	72.60	68.69	76.03	78.00	74.45	81.79	83.40	79.85	87.19
15	36.92	32.90	40.91	48.40	44.43	52.45	55.20	51.07	59.09	60.80	56.95	64.97	62.80	58.75	66.77	68.00	63.99	72.01	72.80	68.91	76.93
30	26.60	23.43	29.77	35.20	32.03	38.37	40.80	37.63	43.97	45.80	42.63	48.97	47.40	44.23	50.57	52.20	49.03	55.37	56.80	53.63	59.97
60	18.90	17.07	20.73	25.30	23.47	27.13	29.30	27.47	31.13	33.10	31.27	34.93	34.30	32.47	36.13	37.90	36.07	39.73	41.50	39.67	43.33
120	12.75	11.11	14.39	17.60	15.96	19.24	20.90	19.26	22.54	24.20	22.56	25.84	25.25	23.61	26.89	28.50	26.86	30.14	31.85	30.21	33.49
360	7.20	6.08	8.32	9.65	8.53	10.77	11.47	10.35	12.58	13.32	12.20	14.43	13.93	12.82	15.05	15.90	14.78	17.02	18.00	16.88	19.12
720	4.53	3.75	5.30	6.12	5.35	6.89	7.32	6.55	8.09	8.58	7.81	9.35	9.00	8.23	9.77	10.33	9.56	11.10	11.75	10.98	12.52
1440	2.68	2.28	3.07	3.57	3.18	3.97	4.25	3.85	4.65	4.96	4.56	5.35	5.21	4.81	5.60	5.96	5.56	6.35	6.75	6.35	7.15

## 4.2 Design Hyetographs

Rainfall hyetographs show how the total depth (or intensity) of rainfall in a storm is distributed among time increments. Synthetic hyetographs, which are systematic, reproducible methods for varying rainfall over a period of time, are used as input in hydrologic modeling. The City has published a set of synthetic rainfall distributions, or design hyetographs, that are contained in the Subdivision Design Manual. The shape of the City’s design hyetographs is based on historical rainfall data.

For this study, CBCL developed design hyetographs based on the updated IDFs described above, and climate change projections prepared by Dr. Joel Finnis, Professor, Department of Geography, Memorial University of Newfoundland. Further, the design hyetographs were produced using the shape of the City’s design hyetographs contained in the Subdivision Design Manual, and the alternating block method. Using two different techniques to develop design hyetographs allowed for a more rigorous examination of peak flows. A discussion of the flows modeled using the various design hyetographs and the ultimate selection of design flows is presented in Chapter 6.

In summary, four sets of design hyetographs were developed using the following combinations:

- Updated IDFs and the City’s hyetograph shape;
- Updated IDFs and the alternating block method;
- Climate change projections and the City’s hyetograph shape; and
- Climate change projections and the alternating block method.

### 4.2.1 Updated IDFs

The 1:20 and 1:100 AEP design hyetographs based on the City’s shape are presented in Tables 4-2 and 4-3.

**TABLE 4-2 1:20 AEP RAINFALL HYETOGRAPHS – UPDATED IDFs AND CITY’S HYETOGRAPH SHAPE**

% Time	1:20 AEP Cumulative Rainfall (mm)					
	0.5 Hour	1 Hour	2 Hour	6 Hour	12 Hour	24 Hour
0.00%	0.0	0.0	0.0	0.0	0.0	0.0
8.33%	1.6	2.3	3.3	5.4	1.0	8.0
16.67%	3.9	5.6	8.2	13.6	2.1	20.2
25.00%	6.9	10.0	14.6	24.0	7.2	35.8
33.33%	10.8	15.6	22.9	37.8	18.5	56.3
41.67%	15.5	22.4	32.8	54.1	39.1	80.5
50.00%	19.1	27.6	40.4	66.7	64.9	99.3
58.33%	20.3	29.4	43.0	70.9	83.4	105.6
66.67%	21.3	30.7	44.9	74.1	93.7	110.4
75.00%	22.0	31.8	46.5	76.7	98.9	114.3
83.33%	22.5	32.6	47.6	78.5	100.9	117.0
91.67%	22.8	32.9	48.1	79.4	102.0	118.3
100.00%	22.9	33.1	48.4	79.9	103.0	119.0

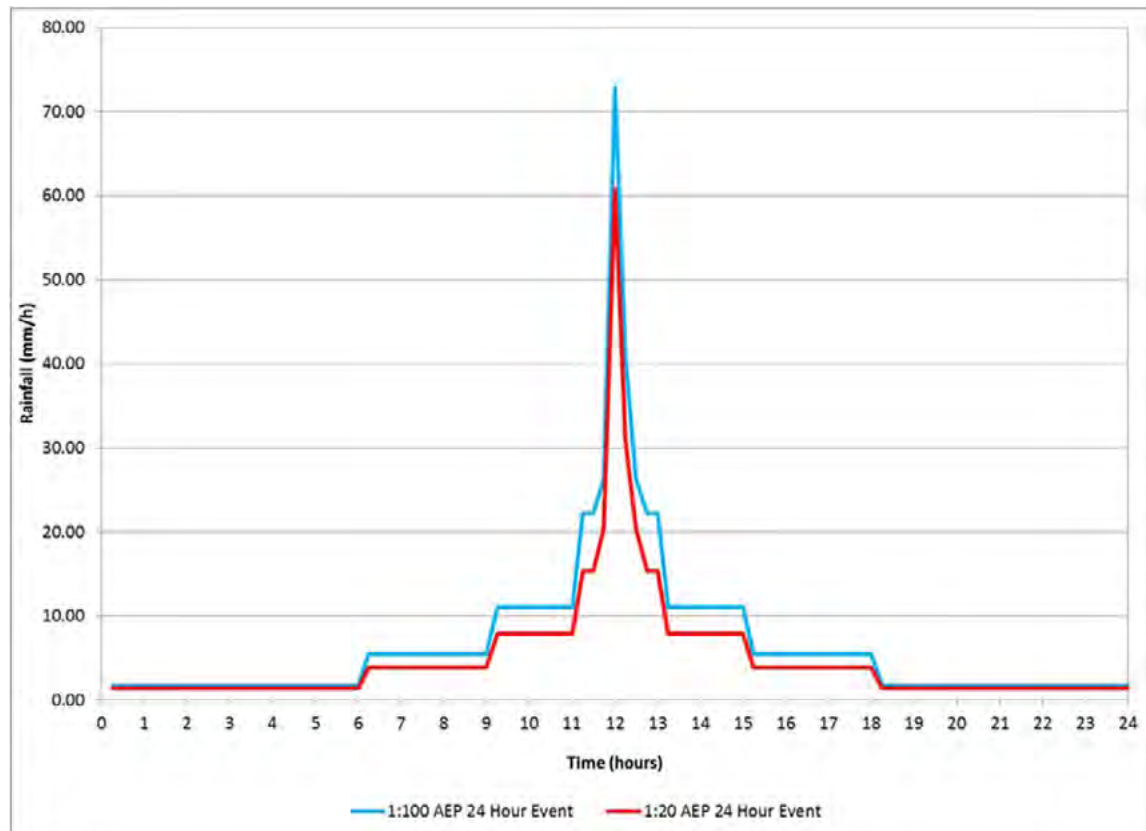
**TABLE 4-3 1:100 AEP RAINFALL HYETOGRAPHS – UPDATED IDF’S AND CITY’S HYETOGRAPH SHAPE**

% Time	1:100 AEP Cumulative Rainfall (mm)					
	0.5 Hour	1 Hour	2 Hour	6 Hour	12 Hour	24 Hour
0.00%	0.0	0.0	0.0	0.0	0.0	0.0
8.33%	1.9	2.8	4.3	7.3	1.4	10.9
16.67%	4.9	7.0	10.9	18.4	2.9	27.5
25.00%	8.5	12.5	19.2	32.4	9.9	48.7
33.33%	13.4	19.6	30.1	51.1	25.3	76.6
41.67%	19.2	28.0	43.1	73.1	53.5	109.6
50.00%	23.7	34.6	53.2	90.1	88.8	135.1
58.33%	25.2	36.9	56.6	95.9	114.2	143.7
66.67%	26.4	38.5	59.1	100.2	128.3	150.3
75.00%	27.3	39.9	61.2	103.7	135.4	155.6
83.33%	27.9	40.8	62.6	106.2	138.1	159.3
91.67%	28.2	41.2	63.3	107.4	139.6	161.0
100.00%	28.4	41.5	63.7	108.0	141.0	162.0

The complete set of updated design hyetographs (i.e. 1:2, 1:5, 1:10, 1:20, 1:25, 1:50 1:100 AEP) with the upper and lower confidence intervals are presented in Appendix D.

The hyetographs created for the 1:20 and 1:100 AEP return periods include the precipitation amounts for the 15 and 30-minute, and 1, 2, 6, 12 and 24-hour duration. Figure 4-2 illustrates the 1:20 and 1:100 AEP hyetographs.

**FIGURE 4-2 1:20 AND 1:100 AEP HYETOGRAPHS AND ALTERNATING BLOCK METHOD**



#### 4.2.2 Climate Change Projections

A report by Dr. Joel Finnis describing the climate change projections is included in Appendix E. The projections developed for the 2062 period are presented in Table 4-4.

**TABLE 4-4 RETURN PERIOD VALUES FOR 24-HOUR PRECIPITATION (MM) BASED ON ANALYSIS BY DR. JOEL FINNIS**

Return Period (yr)	Extreme 24 hour Precipitation Amounts (mm)	Ratio to Updated IDF
2	71.4	1.11
5	98.3	1.15
10	116.3	1.14
20	133.5	1.12
25	139.0	1.11
50	155.9	1.09
100	172.7	1.07

Design hyetographs based on the City's shape are presented in Tables 4-5 and 4-6.

**TABLE 4-5 1:20 AEP RAINFALL HYETOGRAPHS – CLIMATE CHANGE AND CITY'S HYETOGRAPH SHAPE**

% Time	1:20 AEP Cumulative Rainfall (mm)					
	0.5 Hour	1 Hour	2 Hour	6 Hour	12 Hour	24 Hour
0.00%	0.0	0.0	0.0	0.0	0.0	0.0
8.33%	2.0	2.7	3.5	5.8	1.1	9.0
16.67%	5.2	6.7	8.9	14.5	2.3	22.7
25.00%	9.1	12.0	15.8	25.6	7.8	40.2
33.33%	14.2	18.7	24.7	40.3	20.0	63.1
41.67%	20.3	26.8	35.4	57.7	42.2	90.3
50.00%	25.1	33.1	43.6	71.1	70.0	111.4
58.33%	26.7	35.2	46.4	75.6	90.0	118.5
66.67%	28.0	36.8	48.5	79.0	101.1	123.9
75.00%	29.0	38.1	50.2	81.8	106.7	128.3
83.33%	29.6	39.0	51.4	83.8	108.8	131.3
91.67%	29.9	39.4	52.0	84.7	110.0	132.7
100.00%	30.1	39.7	52.3	85.2	111.1	133.5

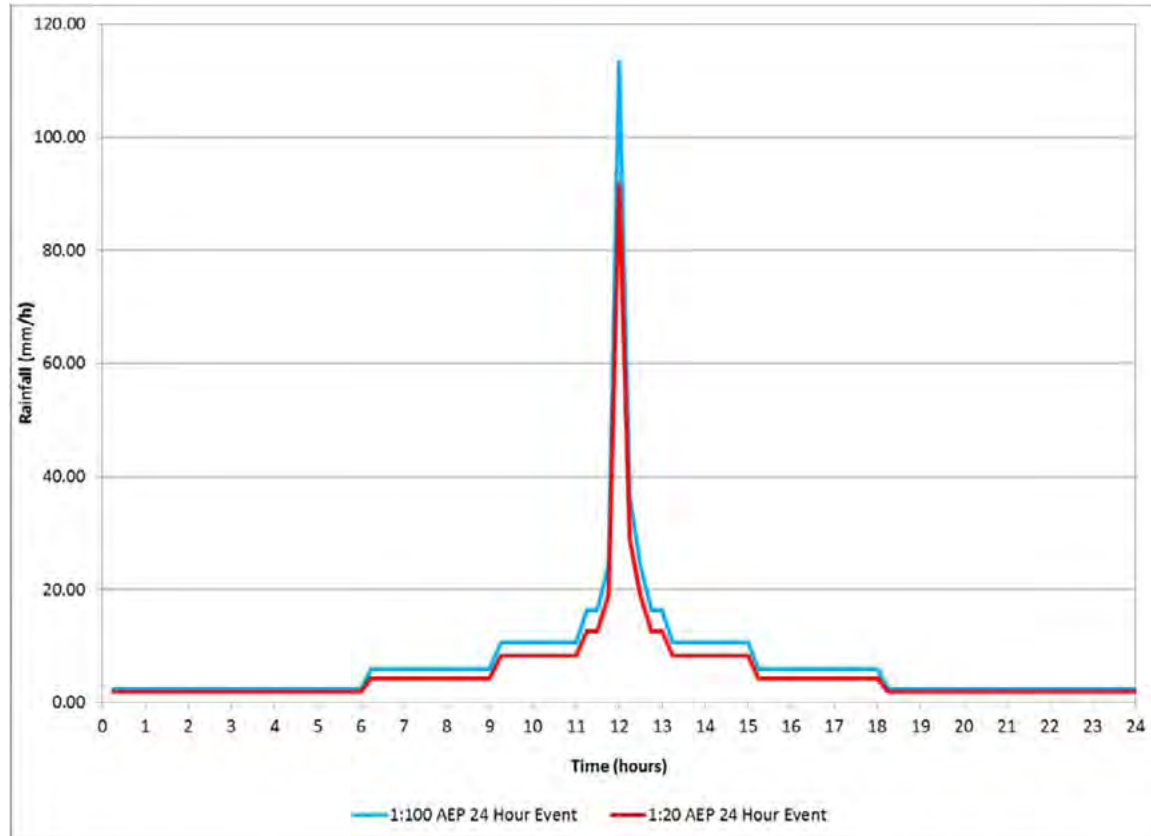
**TABLE 4-6 1:100 AEP RAINFALL HYETOGRAPHS – CLIMATE CHANGE AND CITY'S HYETOGRAPH SHAPE**

% Time	1:100 AEP Cumulative Rainfall (mm)					
	0.5 Hour	1 Hour	2 Hour	6 Hour	12 Hour	24 Hour
0.00%	0.0	0.0	0.0	0.0	0.0	0.0
8.33%	2.5	3.4	4.5	7.4	1.5	11.6
16.67%	6.4	8.4	11.2	18.4	2.9	29.3
25.00%	11.3	15.0	19.9	32.5	10.1	51.9
33.33%	17.7	23.5	31.2	51.1	25.8	81.6
41.67%	25.3	33.6	44.6	73.2	54.4	116.8
50.00%	31.3	41.4	55.0	90.2	90.3	144.1
58.33%	33.3	44.1	58.5	96.0	116.2	153.2
66.67%	34.8	46.0	61.1	100.2	130.5	160.3
75.00%	36.0	47.7	63.3	103.8	137.7	165.9
83.33%	36.8	48.9	64.8	106.3	140.5	169.8
91.67%	37.3	49.3	65.5	107.5	142.0	171.7
100.00%	37.5	49.7	65.9	108.1	143.4	172.7

The complete set of updated design hyetographs (i.e. 1:2, 1:5, 1:10, 1:20, 1:25, 1:50 1:100 AEP) with the upper and lower confidence intervals are presented in Appendix F.

The hyetographs created for the 1:20 and 1:100 AEP return periods include the precipitation amounts for the 15 and 30-minute, and 1, 2, 6, 12 and 24-hour duration. Figure 4-3 illustrates the 1:20 and 1:100 AEP hyetographs.

**FIGURE 4-3 1:20 AND 1:100 AEP HYETOGRAPHS – CLIMATE CHANGE AND ALTERNATING BLOCK METHOD**



## CHAPTER 5 STATISTICAL ANALYSIS

In accordance with the RFP, the 1:20 and 1:100 AEP flood flows at the EC gauge on Leary's Brook were estimated by performing a flood frequency analysis. Although there is no defined length of record that should be used to estimate flood flows, the *Regional Flood Frequency Analysis for the Island of Newfoundland* suggests a period of record exceeding 18 years to sufficiently estimate the 1:100 AEP flood. There are 26 years of annual instantaneous maximum data recorded at the Leary's Brook gauge. As such, this gauge can be used to estimate a 1:100 AEP flow, but the estimate should be used with caution.

The annual peak instantaneous flow series for the gauge is provided in Appendix G. At the time of this study only data from 1987 to 2010 was available on EC's website for the gauge. Therefore, EC was contacted to obtain the peak instantaneous flows for 2011 and 2012. These two data points are included with the data series; however, EC noted that the 2011 and 2012 data is preliminary only and subject to change. In addition, the data series for Leary's Brook gauge had one missing data point for 1991. The peak flow was estimated prior to conducting frequency analysis by estimating a peaking factor for the gauge. The peaking factor is calculated by dividing the peak instantaneous flow by the maximum daily flow for each annual pair and averaging the results. To estimate the absent peak instantaneous flow, the peaking factor is multiplied by the daily maximum value for that year. This estimated value is also included with the data series contained in Appendix G.

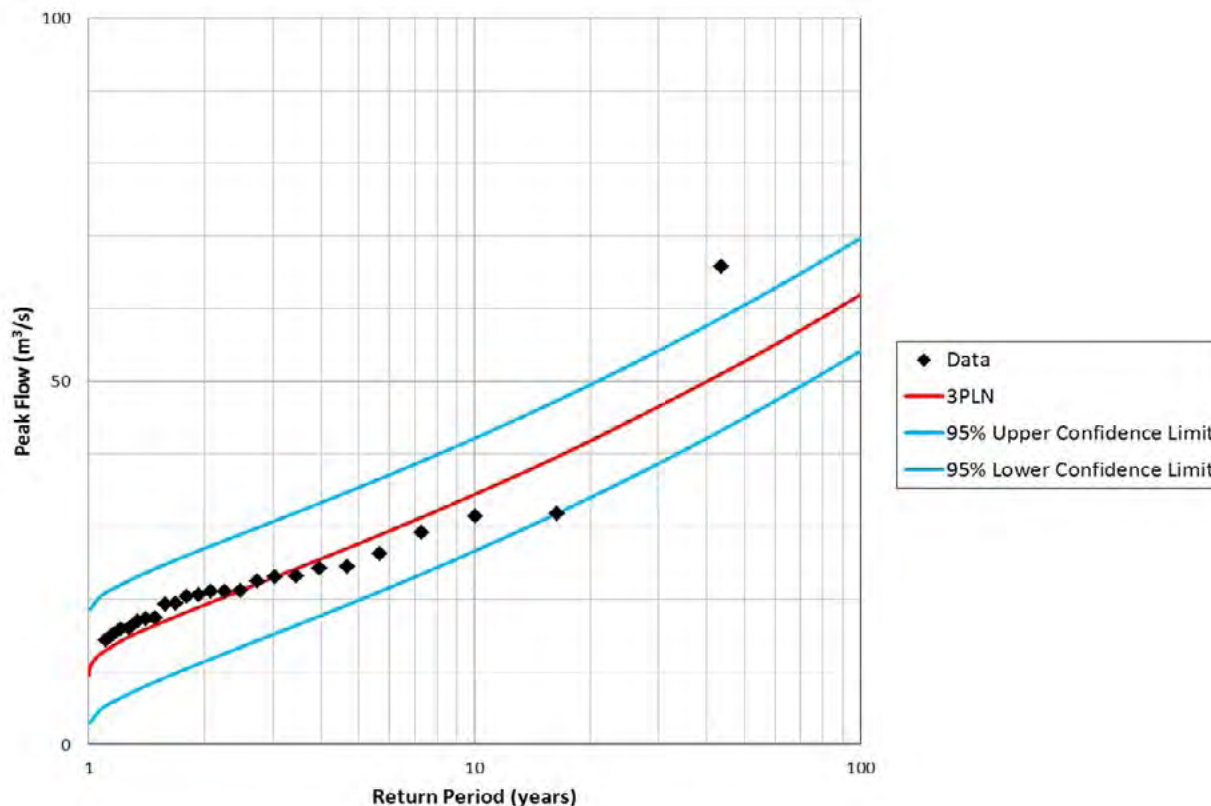
Prior to conducting the frequency analysis, several statistical screening tests were performed on the data. These tests include the following:

- Randomness: variations in the data set are a result of natural causes (i.e. the flow is not regulated)
- Independence: each recorded flow is independent of the other
- Stationarity: the data series does not display trend with respect to time
- Homogeneity: all the data points are derived from a single population

Plots of the distributions for the flow gauge data and the associated screening tests are included in Appendix G. The results indicate that the data is random, does not display dependence, does not display trend and does not display a significant difference in location.

Several statistical distributions were examined, including Gumbel, Generalized Extreme Value (GEV), Lognormal, 3-Parameter Lognormal and Log Pearson Type III. The most appropriate distribution was selected based on visual goodness-of-fit and statistical test. Figure 5-1 illustrates the selected distribution, along with the 95% confidence interval. The resulting AEP flow estimates are listed in Table 5-1.

**FIGURE 5-1 LEARY'S BROOK AT PRINCE PHILIP DRIVE—FREQUENCY ANALYSIS**



**TABLE 5-1 SINGLE STATION FREQUENCY ANALYSIS RESULTS**

Station Name	Distribution	1:20 AEP flood flow (m³/s)	1:100 AEP flood flow (m³/s)
Leary's Brook at Prince Philip Drive	3PLN	41.8	61.9



## CHAPTER 6 **HYDROLOGIC MODELING**

The 1:20 and 1:100 AEP flood flows for Rennies River were estimated using deterministic (modeling) approaches. The deterministic approach involved creating a hydrologic model of the watercourse to determine flood flows.

The modeling software XPSWMM, Version 13 (with Service Pack 1 installed), was used to create a hydrologic model of the study area. XPSWMM is a comprehensive software package used for dynamic modeling of stormwater, sanitary and river systems. The program was created by XP Software and uses a modified EPA SWMM engine for the runoff hydrograph simulation method. It uses the capabilities of SWMM and combines it with a user-friendly interface and the ability to link the 1D SWMM model to a 2D overland flow model. It simulates natural rainfall-runoff processes of the watershed systems, using climate data as dynamic inputs.

### 6.1 Model Development

All sub-watersheds in the study area were modelled with the characteristics shown in Table 3-1. The Green-Ampt Infiltration method was used for infiltration calculations. Rainfall hyetographs used for runoff calculations included the 1:20 and 1:100 AEP precipitation amounts. Peak flows were determined for existing and future development, and will serve as inputs to the hydraulic model to determine flood lines. For future flow calculation, the percent impervious area parameter was changed in the calibrated model to reflect future development. It was assumed 80% imperviousness for future phases based on the City’s Subdivision Design Manual. Table 6-1 shows the impervious area changes for the sub-watersheds with future development.

**TABLE 6-1 IMPERVIOUS AREA CHANGES FOR FUTURE DEVELOPMENT**

Name	Watershed Area (ha)	Future Impervious Area (ha)	Percent Impervious Area (%)	
			Existing	Future
Sub-Watershed1	179.69	99.45	9.4	53.7
Sub-Watershed2	183.28	65.39	20.6	49.1
Sub-Watershed3	285.18	42.03	4.3	16.1
Sub-Watershed4	195.89	28.58	23.6	35.3

It should be noted that while the RFP requires that three development scenarios be addressed – existing, future and ultimate – only the existing and future scenarios are discussed in this report. The reason for this deviation from the RFP is that with the adoption of the City’s Stormwater Detention Policy, there will be a net-zero-increase in stormwater runoff rendering the future and ultimate development scenarios to be identical.

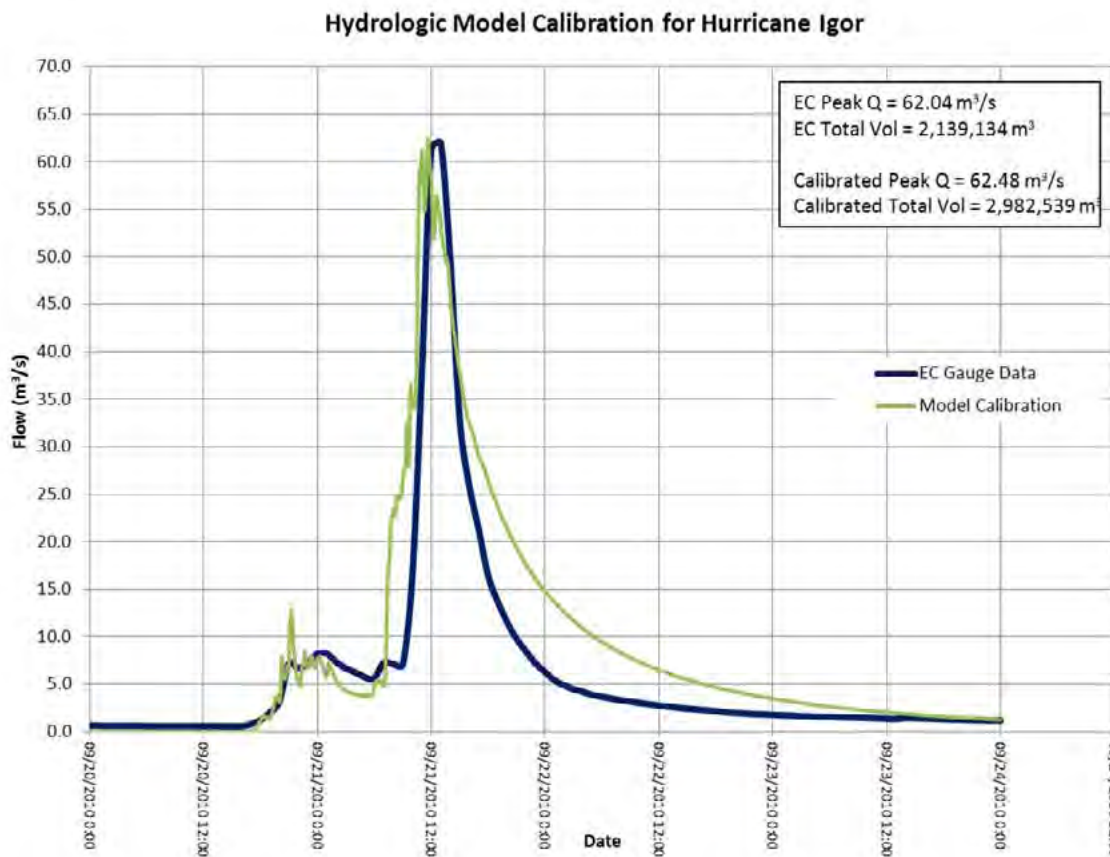
## 6.2 Model Calibration

Calibration of the Rennie’s River catchment XPSWMM model was undertaken through a comparison of simulated hydrographs and observed hydrographs at the location of the Leary’s Brook at Prince Philip Drive gauge.

A rainfall-runoff event from September 20-24, 2010 (during Hurricane Igor) was selected for calibration. Hourly flow data was obtained from the Water Survey of Canada for the above-noted gauge locations. Five-minute interval rainfall data was also obtained at the Windsor Lake rain gauge for this time period from the City.

Model parameters were adjusted until a reasonable calibration was achieved. Illustrated in Figure 6-1 is the result from the model calibration. As shown, the XPSWMM model provides a reasonably accurate estimation of flows.

FIGURE 6-1 HYDROLOGIC MODEL CALIBRATION

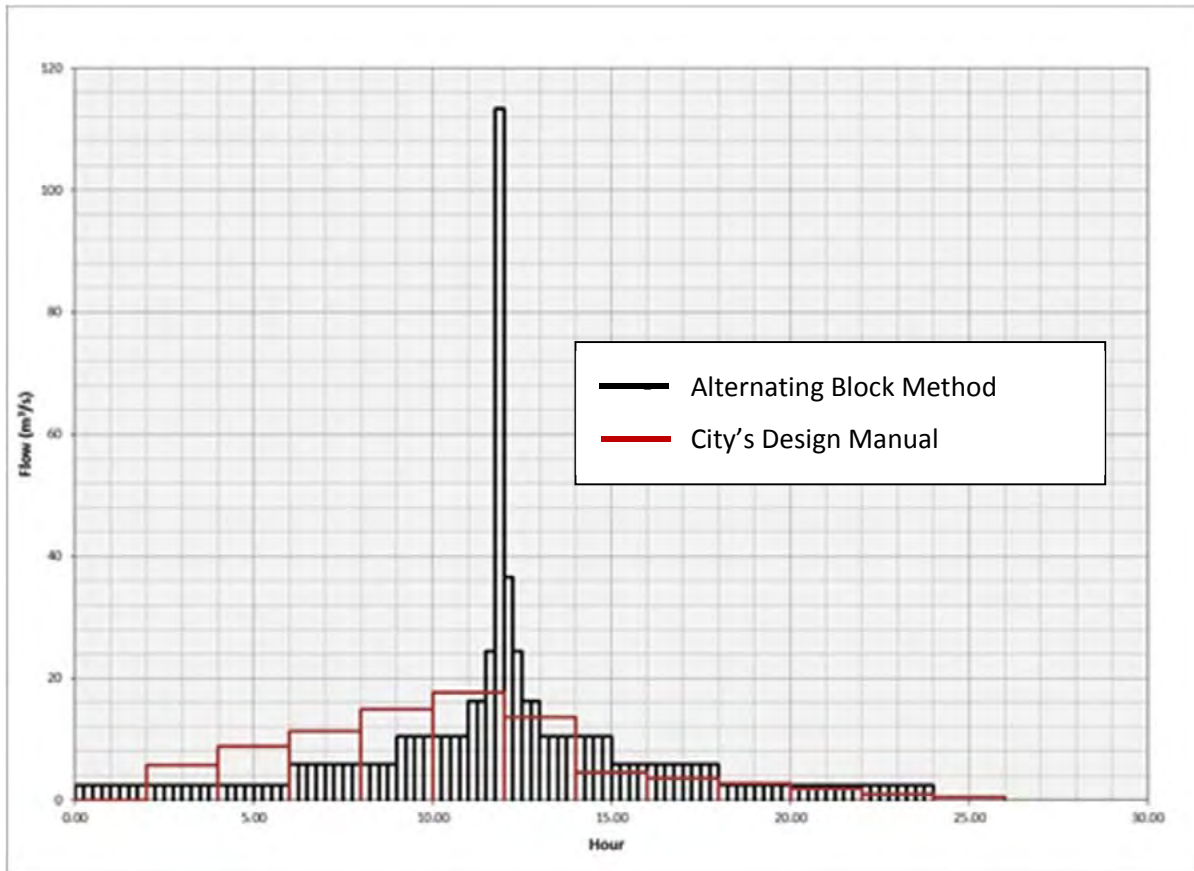


Since no flow data is available to calibrate the lower reach of Rennie's River (sub-watersheds 7-10) for Hurricane Igor, model parameters were determined by extrapolating the watershed parameters from the upstream areas to obtain the 1:20 and 1:100 AEP flows for these sub-watersheds.

### 6.3 Hyetograph Selection

Rainfall hyetographs and the resulting flows for both the alternating block method and the City Design Manual shapes were compared to identify the most suitable approach. These are shown below.

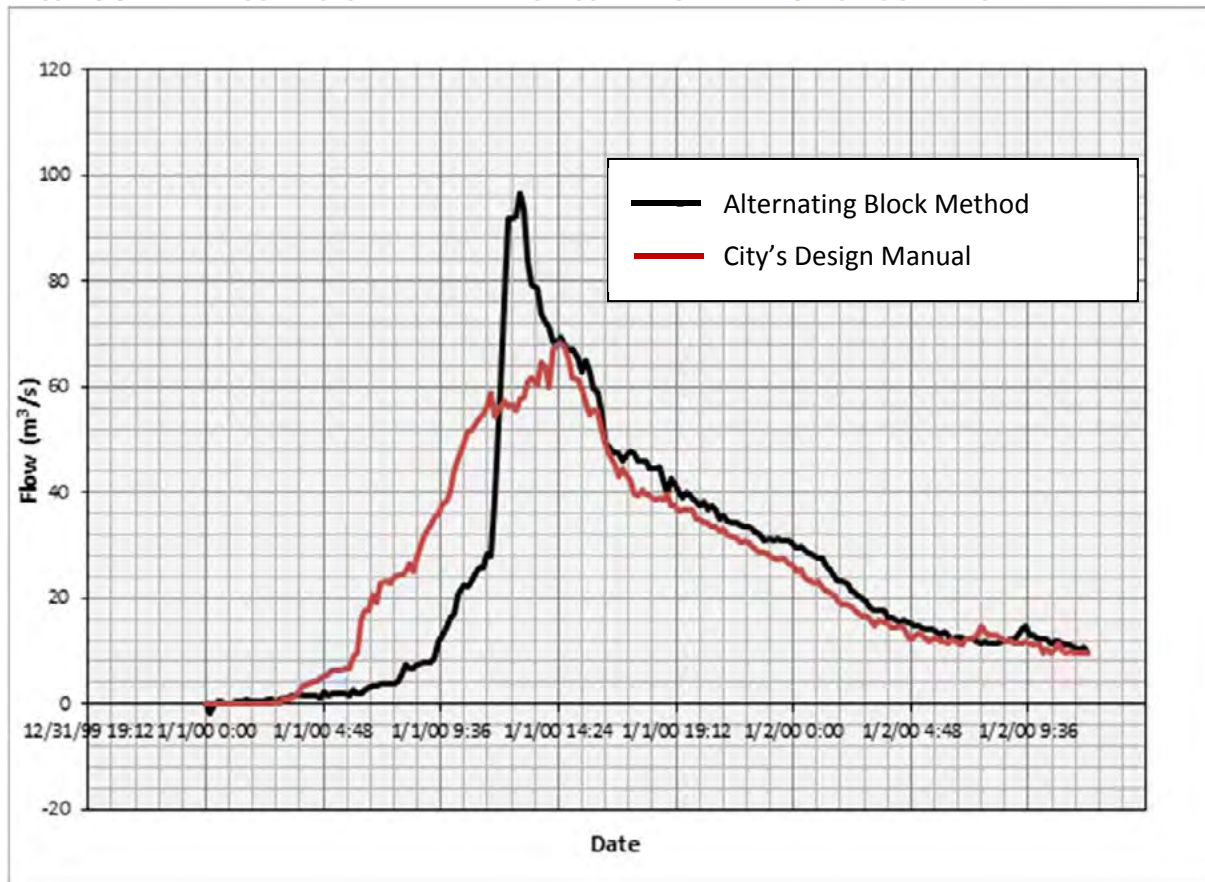
FIGURE 6-2 HYETOGRAPHS FOR THE ALTERNATING BLOCK METHOD AND THE CITY'S DESIGN MANUAL



As seen on Figure 6-2, the peak intensity of the alternating block method is almost 6 times the peak intensity of the City Design Manual method, even though the total volumes for the two methods are the same. The City's Subdivision Design Manual method works in 2-hours increments, whereas the alternating block method uses a time span of only 5 minutes at the peak intensity. Therefore, if the time of concentration (or lag time) of the watershed is less than 2 hours, there will be a difference in the flow calculation. Since this is indeed the case, it is normal to see a difference in flow calculation results. The more accurate method is the one that allows the watershed to see a peak intensity that corresponds to the same or a smaller duration than that of its time of concentration (or lag time).

The smaller duration storms (6-hour and 12-hour durations) of the City's Subdivision Design Manual method have correspondingly smaller increments, which help define the peak rainfall intensities better. However, the volume of rainfall is now smaller, which affects the flow buildup and results in smaller peaks.

FIGURE 6-3 HYDROGRAPHS FOR THE ALTERNATING BLOCK METHOD AND THE CITY'S DESIGN MANUAL



The hydrographs in Figure 6-3 show this phenomenon. In addition, the figure shows that the total volume of runoff looks equivalent for both methods.

The alternating block method was therefore used for floodplain delineation as well as for evaluating the efficiency of various flood mitigation measures.

#### 6.4 Simulated Flood Flows

The benefit of estimating flood flows using a deterministic method is that site-specific watershed characteristics are used to predict flood flows. As well, observed and planned changes within the basin can be simulated to determine impacts on flood flows.

Using information from the City's zoning maps, hydrologic parameters were altered in the models to reflect future development conditions. The 1:20 and 1:100 AEP hyetographs (described above) were simulated and the peak flood flows at the outlets were extracted.

The uncalibrated 1:20 and 1:100 AEP flow estimates at the outlets of Rennies River, for existing and future development conditions, are presented in Table 6-2. Existing flows were modeled using the hyetographs based on the updated IDFs and the alternating block method; whereas future flows were modeled using the hyetographs based on the climate change projections and the alternating block method.

**TABLE 6-2 1:20 AND 1:100 AEP FLOW ESTIMATES FOR EXISTING AND FUTURE DEVELOPMENT CONDITIONS**

Location	1:20 AEP Event Peak Flow (m <sup>3</sup> /s)		1:100 AEP Event Peak Flow (m <sup>3</sup> /s)	
	Existing Flow	Future Flow	Existing Flow	Future Flow
Great Eastern Ave. at Ken Brook	6.1	15.1	8.2	20.3
Lady Smith Dr. at Ken Brook	15.5	29.7	20.8	39.9
NL Power Yard at Yellow Marsh Brook	20.4	39.7	27.5	53.4
Pippy Place at Leary's Brook	31.9	53.8	42.9	72.4
O'Leary Ave. at Leary's Brook	50.3	72.2	67.8	97.3
Wicklow St. at Leary's Brook	68.1	90.1	91.9	121.4
Allandale Rd. at Rennies River	85.0	106.9	114.6	144.1
Prince Philip Dr. at Rennies River	92.6	114.6	124.7	154.2
Portugal Cove Rd. at Rennies River	111.0	133.0	149.8	179.3
Carnell Dr. at Rennies River	132.6	154.6	178.7	208.2

The flows presented in Table 6-2 for Rennies River (last four rows of the table) were adjusted during the hydraulic model calibration. These adjusted flows, which are presented in Chapter 7, were used in the creation of floodplain mapping and to develop the preliminary flood and erosion control designs.

## CHAPTER 7 HYDRAULIC MODELING

The purpose of the hydraulic analysis is to translate the 1:20 and 1:100 AEP flood flows, estimated during the hydrologic analysis, into floodplain mapping.

Hydraulic modeling was carried out using 2D XPSWMM model. The 2D module available for XPSWMM allows the user to utilize the 1D river modeling capability of the XPSWMM software with a 2D TUFLOW-based overland flow model. GIS data can be used to input a variety of information into the model including topography, land cover categories, model boundaries, and node and channel networks.

It was found that for conditions where the flows were contained within a well-defined channel, 1D hydrodynamic modelling is an effective method of representing flood characteristics. However, when flows become more complex 2D hydrodynamic modelling provides a more complete indication of flooding extents and other characteristics. 2D modeling has the advantage that it can resolve various surface water paths, with varying velocities, including splitting of flows, circulation and rejoining of various flow branches, which is typical in floodplains of urban areas.

### 7.1 Model Development

#### 7.1.1 Model Input

The approach adopted in this study is to model hydraulic structures as a 1D network nested within the 2D domain representing the floodplain. The first modelling step is to divide the catchment into a network of small cells, which form a grid. A 5-m grid was chosen considering the size of the channel and computational time required to run the model. The 5-m grid may appear to be “coarse” considering that the City provided 1-m LiDAR data to use in developing the models. To identify any potential shortcomings or advantages of using a coarse grid, CBCL carried out a comparative analysis between using a 5-m grid and 2-m grid. The analysis revealed that the 5-m grid produced acceptable results.

The second modelling step is to develop a ground model using the LiDAR data by transferring elevations to the centre of each cell. The extents of the 2D domain were defined based on the general land topography which included the low-lying floodplain areas that are likely to be flooded.

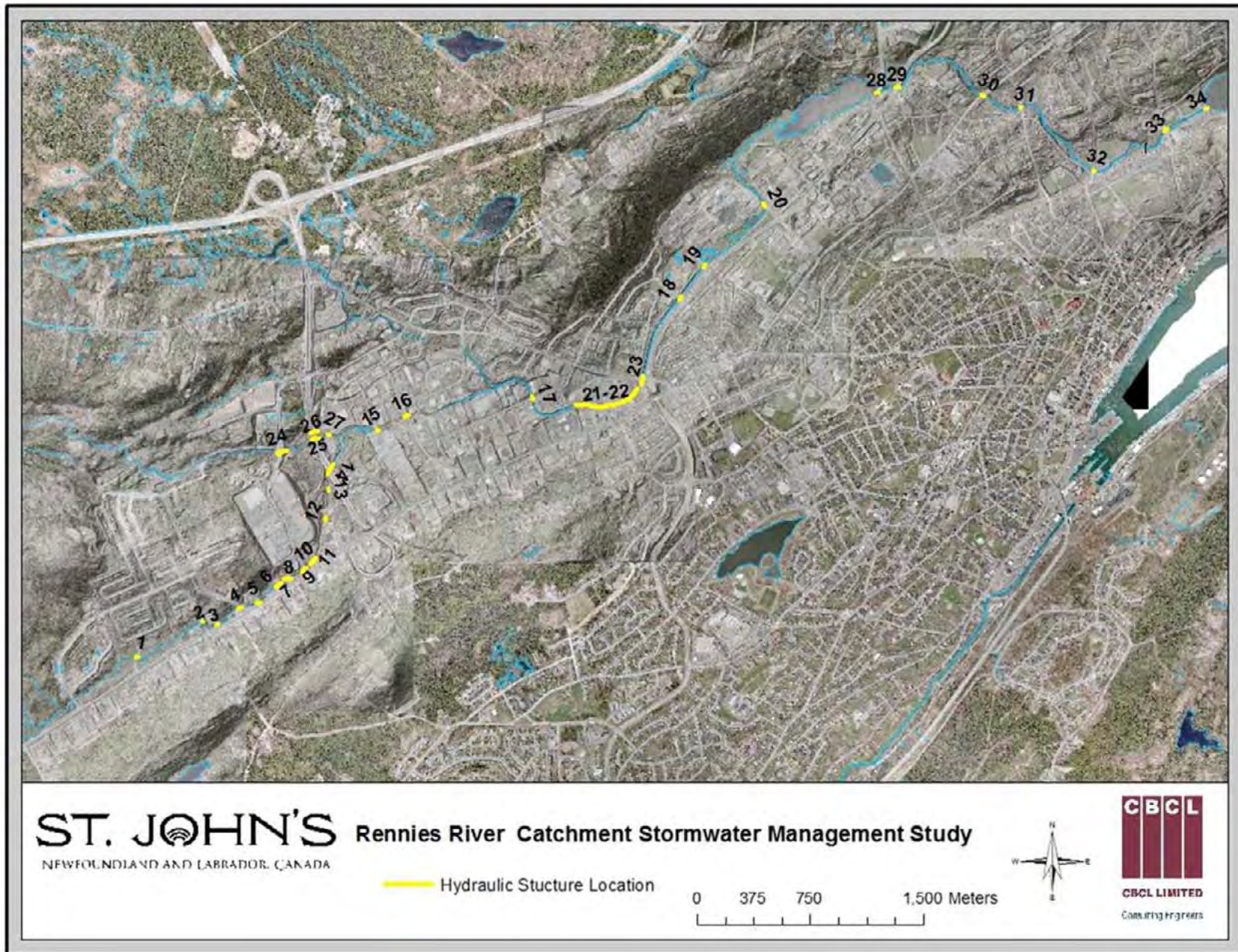
Hydraulic structures including culverts and bridges were then input into the model as a 1D network. The inverts, dimensions and channel cross sections for these structures were surveyed between April and August 2013 and used to define the hydraulic geometry of the 1D network. The channel roughness or bed resistance values were assigned based on the current land use.

An important aspect of the model is to define 1D/2D linkages using lines where there is flow interchange between 1D and 2D components of the model. These lines are located at the inlet and outlet of these structures to define where flow will interact between the 1D hydraulic structures and the 2D floodplain. The model upstream boundary was represented as flow boundary and the downstream as head boundary. The software version used in this study only allows for a maximum of 29,999 grid cells. There were seven models developed to cover the whole study area.

### **7.1.2 Structures**

Structures located along the river reach were entered in the hydraulic model. These structures are listed in the following table. The additional data required to effectively model the structures were collected during the field investigations. Hydraulic structure data sheets including photos and a description of each structure are provided in Appendix H. Structure Locations are showed in Figure 7-1 and listed in Table 7-1.

FIGURE 7-1 HYDRAULIC STRUCTURE LOCATIONS





**TABLE 7-1 HYDRAULIC STRUCTURES LOCATED ON MAIN RIVER REACHES**

Reach	ID	Structure	Hydraulic Structure Data Sheet
Ken Brook	1	Culvert at Great Eastern Ave.	Culvert No. 1
	2	Culvert Next to Tim Horton's	Culvert No. 2
	3	Culvert at NL Power Access	Culvert No. 3
	4	Culvert at Lady Smith Dr.	Culvert No. 4
	5	Culvert at Wing 'n it	Culvert No. 5
	6	Bridge at Private Driveway	Bridge No. 1
	7	Culvert at Keith Gordon Car Sales	Culvert No. 6
	8	Culvert at Discount Rentals	Culvert No. 7
	9	Culvert at Kelsey Dr.	Culvert No. 8
	10	Bridge at Private Driveway	Bridge No. 2
	11	Culvert at Personal Yard	Culvert No. 9
	12	Culvert at New Gushue Highway Ramp 1	Culvert No. 10
	13	Culvert at New Gushue Highway Ramp 2	Culvert No. 11
	14	Culvert at Existing Gushue Highway Ramp	Culvert No. 13
Leary's Brook	15	Culvert at Mews Place	Culvert No. 17
	16	Culvert at Pippy Place	Culvert No. 18
	17	Bridge at O'Leary Ave.	Bridge No. 3
	18	Bridge at Wicklow St.	Bridge No. 4
	19	Bridge at Clinch Crescent (W)	Bridge No. 5
	20	Bridge at Clinch Crescent (E)	Bridge No. 6
	21	Culvert1 at Avalon Mall	No Survey
	22	Culvert2 at Avalon Mall	No Survey
	23	Bridge at Thorburn Rd.	No Survey
Yellow Marsh Brook	24	Culvert at North on Kelsey Dr.	Culvert No. 12
	25	Culvert at Gushue Highway Crossing (S)	Culvert No. 14
	26	Culvert at Gushue Highway Crossing (N)	Culvert No. 15
	27	Culvert at NL Power Yard	Culvert No. 16
Rennies River	28	Bridge at Allandale Rd.	Bridge No. 7
	29	Bridge at Prince Philip Dr.	Bridge No. 8
	30	Bridge at Elizabeth Ave.	Bridge No. 9
	31	Bridge at Carpasian Rd.	Bridge No. 10
	32	Bridge at Portugal Cove Rd.	Bridge No. 11
	33	Bridge at Kings Bridge Rd.	Bridge No. 12
	34	Bridge at Carnell Dr.	Bridge No. 13

### 7.1.3 Manning's Roughness Coefficient

Perhaps the most sensitive parameter input in the hydraulic model is the Manning's n. During the field investigations, photos and notes were taken to aid the modeller in selecting appropriate Manning's n. Literature values for Manning's n for channels and flood plains are listed in Table 7-2<sup>1</sup>.

<sup>1</sup> Chow, V.T. 1959. *Open Channel Hydraulics*. McGraw-Hill, New York.

**TABLE 7-2 LITERATURE VALUES FOR MANNING’S N**

Natural Streams	Minimum	Normal	Maximum
Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.035
Same as above but more stones and weeds	0.030	0.035	0.040
Clean, winding, some pools and shoals	0.033	0.040	0.045
Same as above but some weeds and stones	0.035	0.045	0.050
Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
Very weedy reaches, deep pools, or floodways with heavy stands of timber and underbrush	0.075	0.100	0.150
Floodplains	Minimum	Normal	Maximum
Short grass	0.025	0.030	0.035
Tall grass	0.030	0.035	0.050
Scattered brush, heavy weeds	0.035	0.050	0.070
Light brush and trees, in summer	0.040	0.060	0.080
Medium to dense brush, in summer	0.070	0.100	0.160

## 7.2 Model Calibration

The development of hydraulic models across a large floodplain requires a rigorous calibration process to ensure the hydraulic model accurately reproduces the observed flooding behaviour. This process generally incorporates comparisons between observed flood levels and areas of inundation.

The focus of the floodplain 2D model calibration is the general flood behaviour during large flood events. Hence, the selection of calibration events reflects large flood events with adequate available observed flood data suitable for model calibration. A review of the available data on historical floods in the Rennies River catchment identified Hurricane Igor flood event suitable for the model calibration. Table 7-3 outlines the details of the selected calibration event.

**TABLE 7-3 SELECTED CALIBRATION EVENT**

Event	General Description	Available Observed Data
September, 2010	<ul style="list-style-type: none"> <li>- Significant flood event causing widespread inundation of the Rennies River floodplain between Portugal Cove Road and Kings Bridge Road, Allandale Road and Prince Philip Drive, and CBC Parking Lot.</li> <li>- Peak flow of the Leary’s Brook at Prince Philip Drive gauge (02ZM020): 62.5 m<sup>3</sup>/s</li> </ul>	<ul style="list-style-type: none"> <li>- Flood Levels: Long Pond Bridge</li> <li>- Stream flow Data: The Leary’s Brook at Prince Philip Drive gauge (02ZM020)</li> <li>- Observed flood extent by photos</li> </ul>

The model calibration was assessed through the comparison of observed and modelled flood levels, and flood extents. Model refinements to the Manning’s n and river bed terrain were undertaken to force the simulated water levels to match the measured water levels to an acceptable difference. All adjustments fall within the limits of the literature values for Manning’s n.

For the Rennies River at Long Pond Bridge, the observed flood level was 55.4 m and the modelled flood level was 55.6 m. The difference may be explained by errors in the measurement of the observed flood levels (e.g. not at flood peak) and/or errors in the specification of the model inflows from ungauged sub-catchment. The flood level difference is 0.2 m, which is reasonable. Such a comparison indicates that the hydraulic model is able to reflect the local hydraulic conditions.

The calibrated flood inundation maps for Hurricane Igor are shown in Appendix I. The City reviewed the floodplain maps to ensure consistency with their experience of the flood event.

### 7.3 Simulated Flood Flows

Flood flows for the 1:20 and 1:100 AEP events based on the calibrated hydrologic and hydraulic models at various locations along the river reach are presented in Table 7-4.

**TABLE 7-4 FLOOD FLOWS**

Location	1:20 AEP Event Peak Flow (m <sup>3</sup> /s)		1:100 AEP Event Peak Flow (m <sup>3</sup> /s)	
	Existing Flow	Future Flow	Existing Flow	Future Flow
Great Eastern Ave. at Ken Brook	4.2	6.1	5.8	8.2
Lady Smith Dr. at Ken Brook	10.6	15.5	14.5	20.8
NL Power Yard at Yellow Marsh Brook	14.1	20.4	19.3	27.5
Pippy Place at Leary's Brook	21.9	31.9	29.9	42.9
O'Leary Ave. at Leary's Brook	34.6	50.3	47.3	67.8
Wicklow St. at Leary's Brook	47.3	68.1	64.4	91.9
Allandale Rd. at Rennies River	38.7	46.4	59.7	65.7
Prince Philip Dr. at Rennies River	41.3	49.3	62.6	69.9
Portugal Cove Rd. at Rennies River	52.5	65.8	78.2	88.0
Carnell Dr. at Rennies River	58.4	73.1	87.9	97.9

The table shows a gradual increase in flows up to the Wicklow Street area, and then a clear drop, before rising steadily again up to the Carnell Drive area. This is related to the fact that the Environment Canada flow gauge is located by Wicklow Street. The calculated flows are calibrated on the gauged flows up to Wicklow Street. Downstream, the hydraulic model is used to estimate flows, and it takes into account the flow restrictions in the system, particularly at Long Pond, which has the ability to store a large portion of the peak flows, thereby reducing the peak flows notably.

The water levels resulting from the various flood flow scenarios were used in the creation of the floodplain maps presented in Chapter 9.

## CHAPTER 8 **SENSITIVITY ANALYSIS**

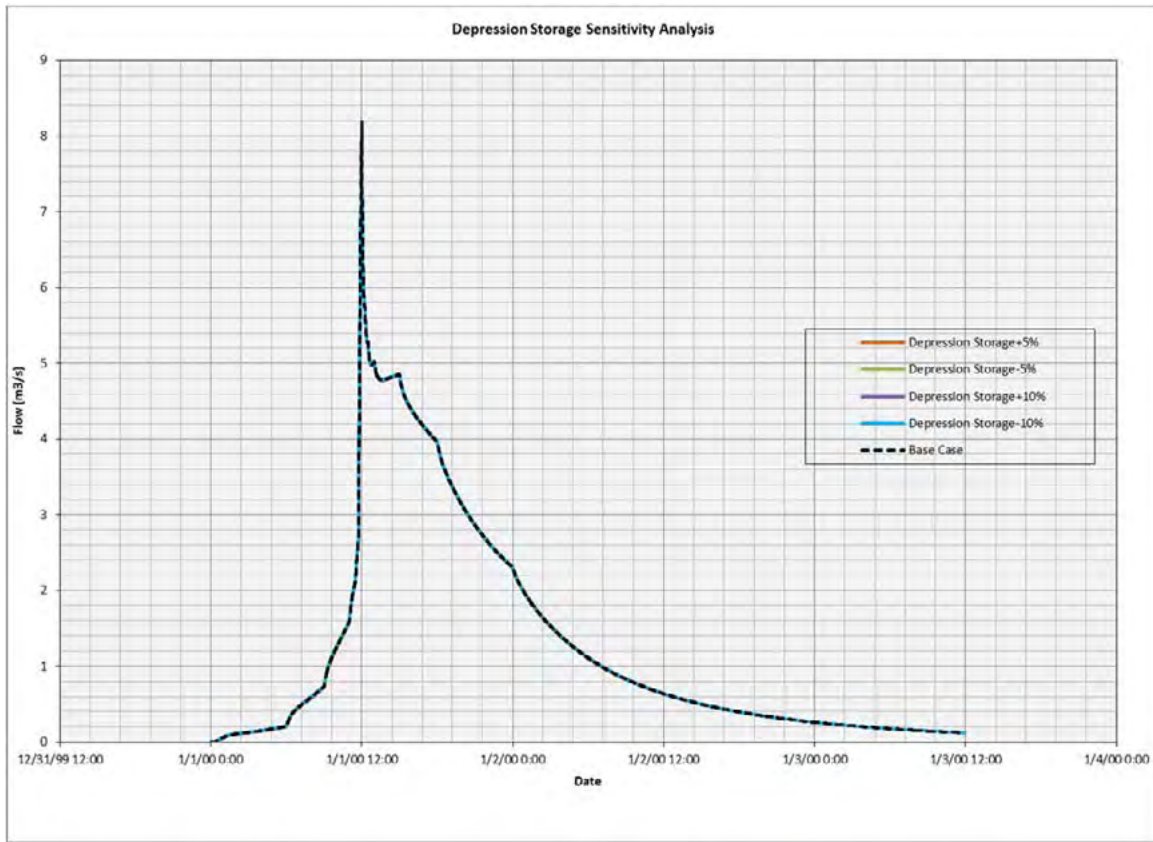
Sensitivity analyses were conducted on selected model parameters to assess the impact of changing these parameters on model results.

### **8.1 Hydrologic Model Sensitivity**

The hydrologic parameters selected for sensitivity analysis include depression storage, average capillary suction, initial moisture deficit, saturated hydraulic conductivity, subbasin width, percent impervious area and Manning's roughness values. The 1:100 AEP event for the existing development conditions was selected as a benchmark to evaluate the sensitivity of the flow to the variation of each parameter. Sensitivity analysis for the parameters was limited to  $\pm 5\%$  and  $10\%$ .

The results indicate that the hydrologic model is most sensitive to changing the percent impervious area. Decreasing the percent impervious area by  $10\%$  decreased peak flow at the outlet of sub-watershed 1 by  $5.31\%$  (over the base case). A close second were the Manning's  $n$  values and the Subbasin Width parameters, impacting the flows by  $3.83\%$  for a parameter change of  $10\%$ . Average capillary suction, initial moisture deficit and saturated hydraulic conductivity had the least effect on flow values. A decrease in these parameters of  $10\%$  increased the peak flow by only  $0.04\%$ . Graphs of the hydrologic model sensitivity analysis are presented in Figures 8-1 to 8-7. Table 8-1 shows the percent change in peak flows.

**FIGURE 8-1 DEPRESSION STORAGE SENSITIVITY ANALYSIS**



**FIGURE 8-2 AVERAGE CAPILLARY SUCTION SENSITIVITY ANALYSIS**

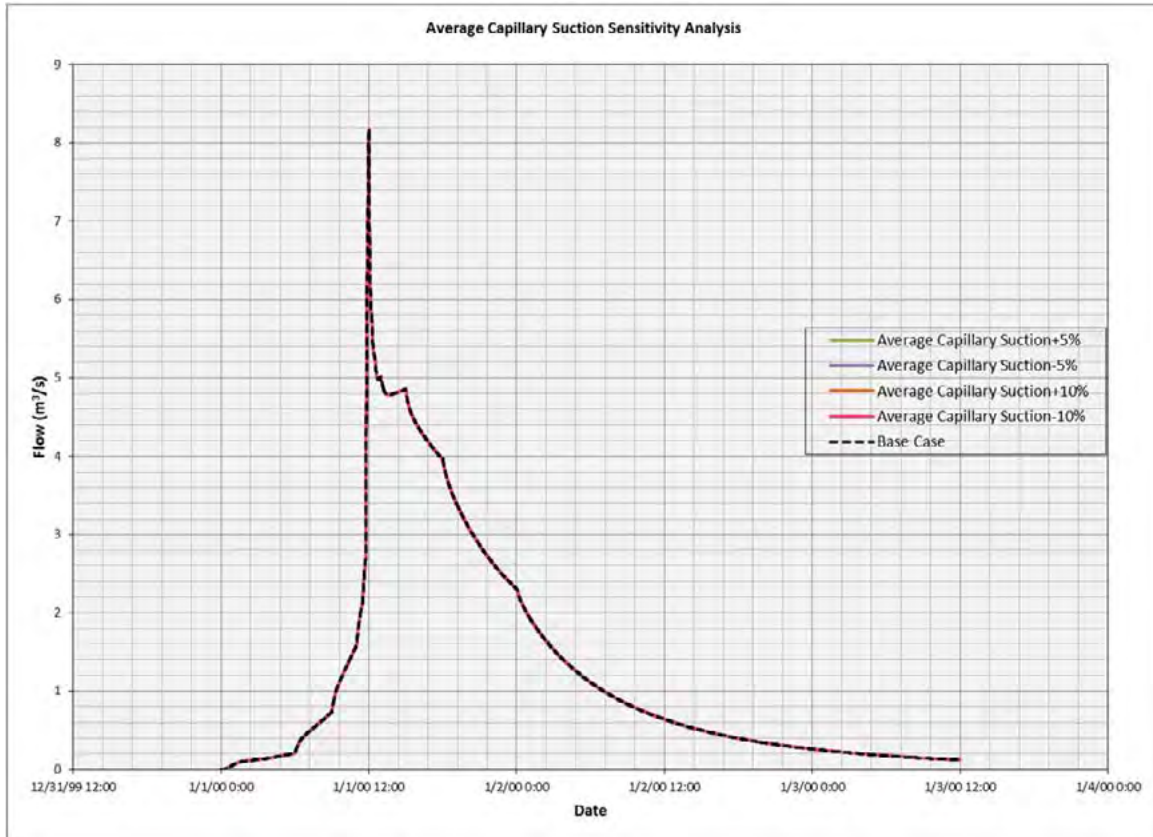


FIGURE 8-3 INITIAL MOISTURE DEFICIT SENSITIVITY ANALYSIS

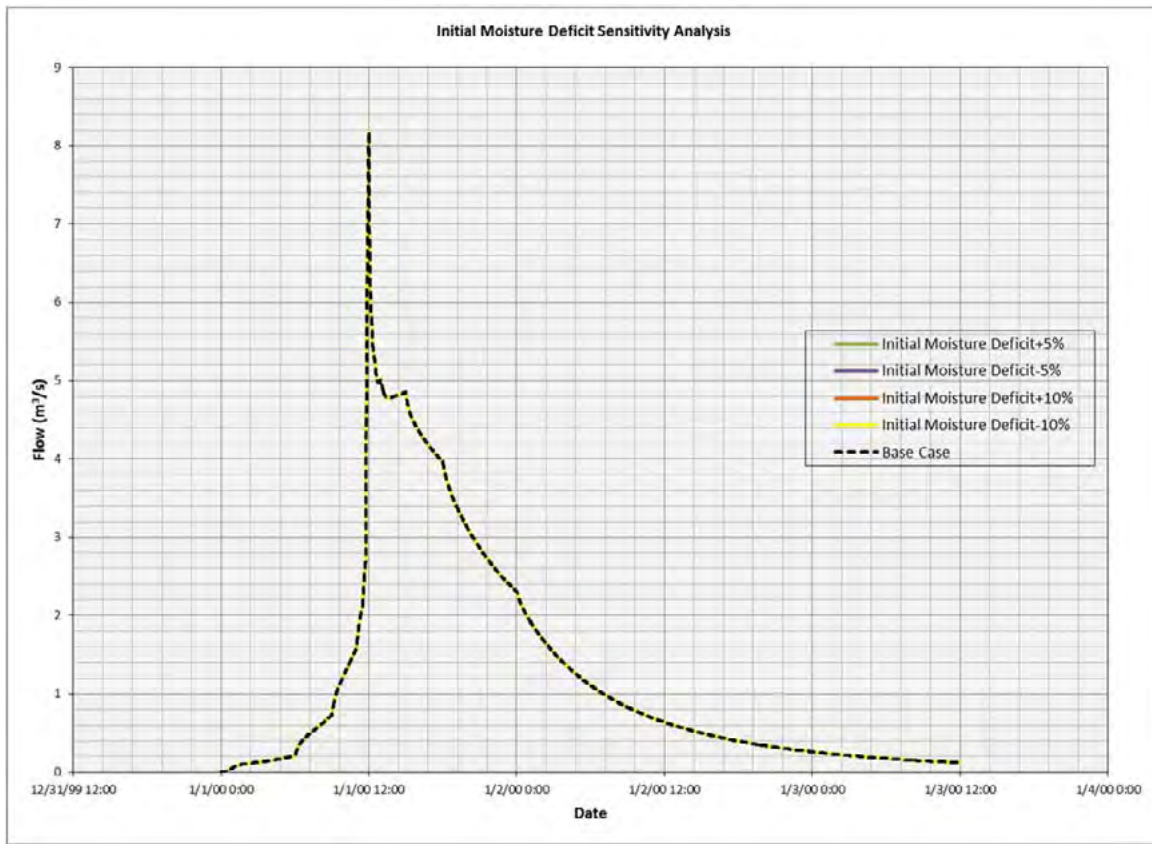


FIGURE 8-4 PERCENT IMPERVIOUS AREA SENSITIVITY ANALYSIS

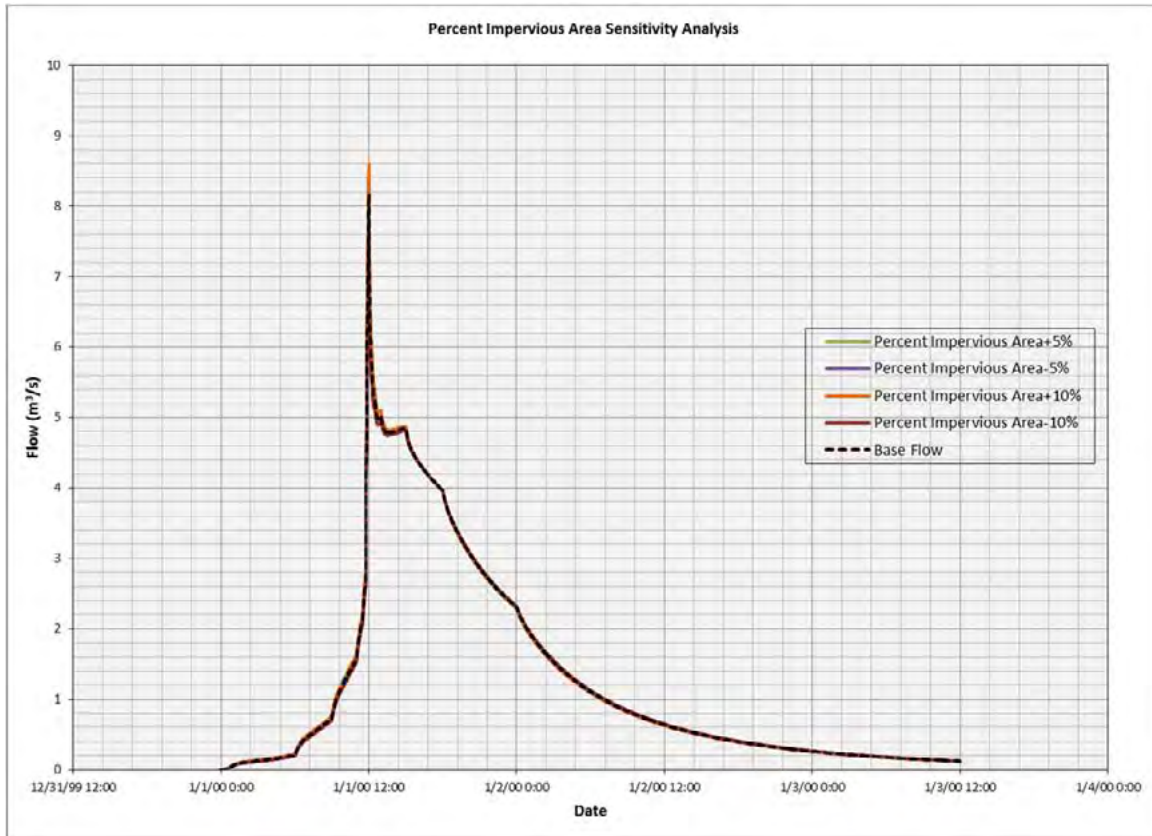


FIGURE 8-5 SUBBASIN WIDTH SENSITIVITY ANALYSIS

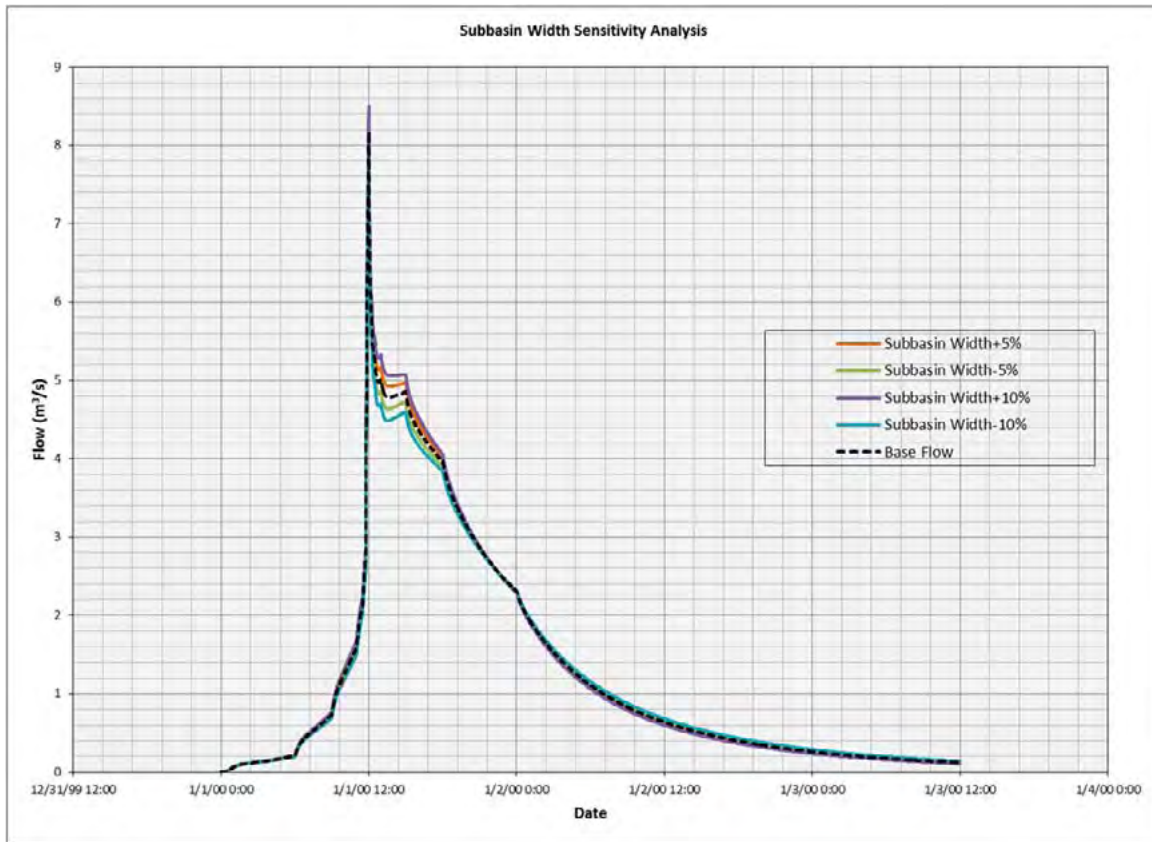
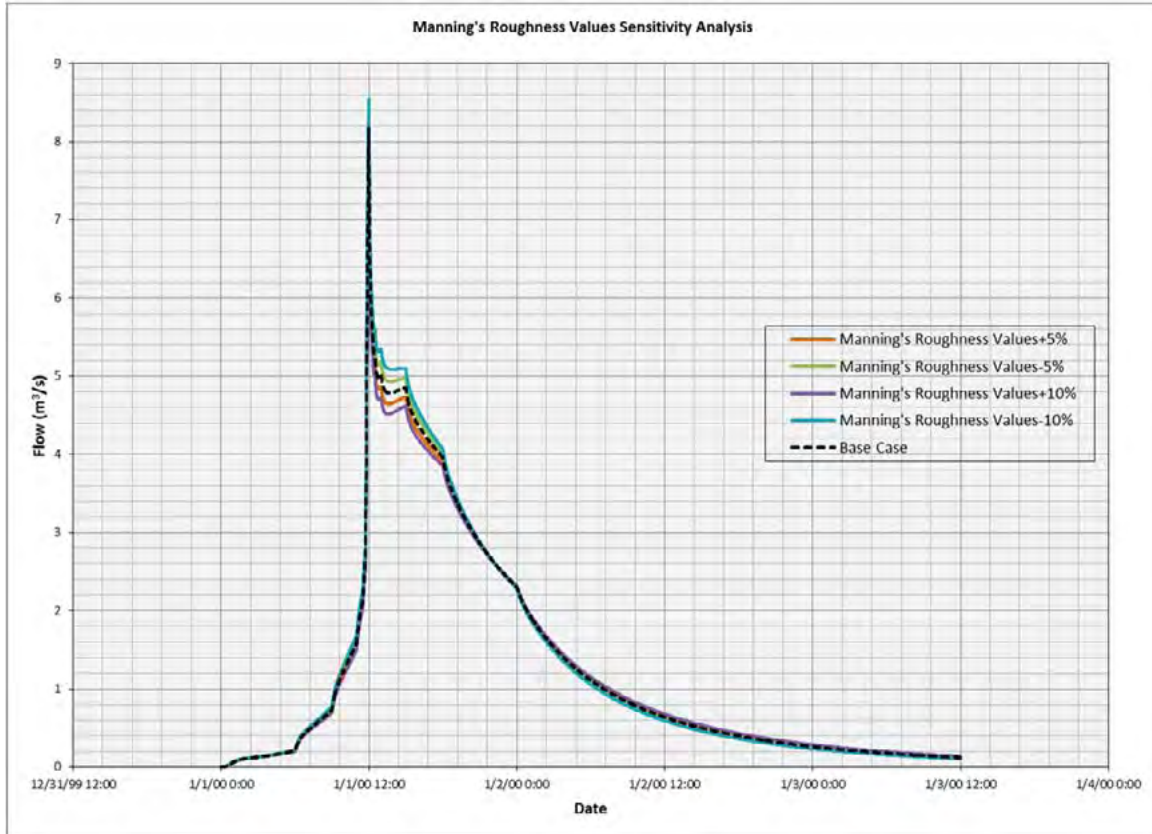
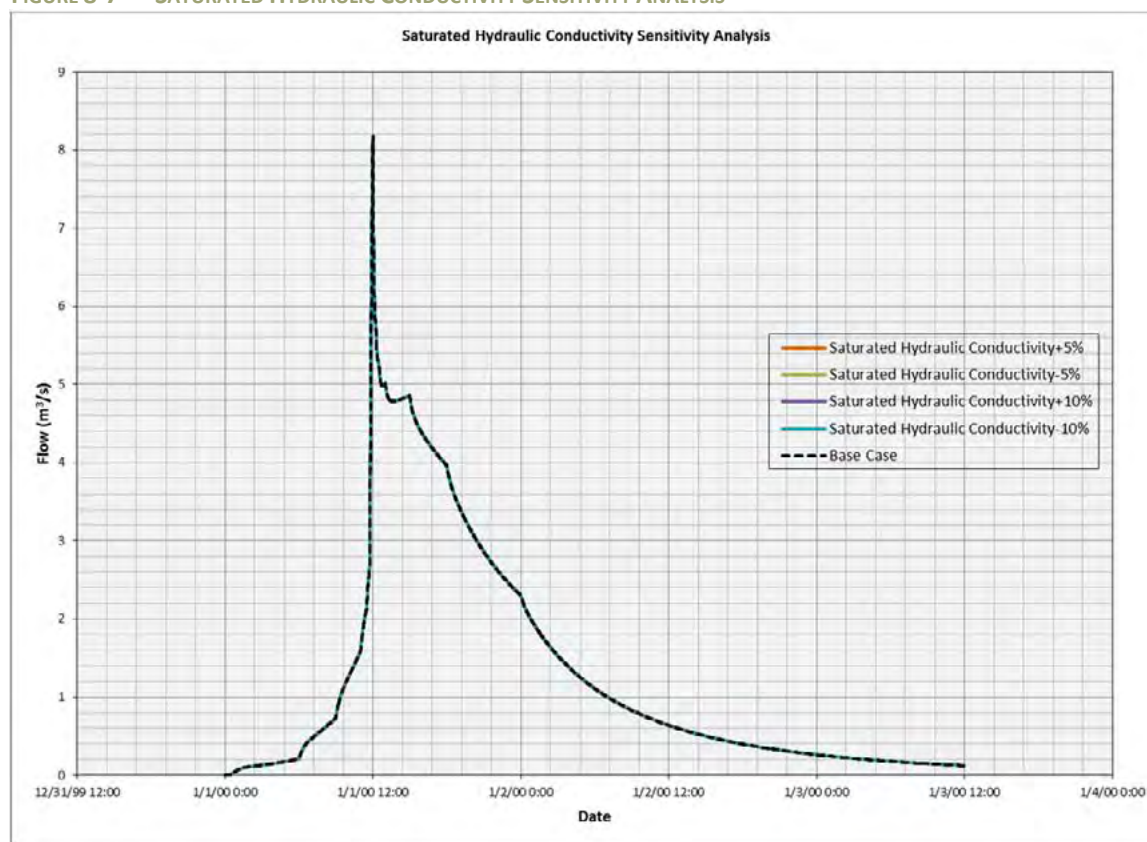


FIGURE 8-6 MANNING'S ROUGHNESS VALUE SENSITIVITY ANALYSIS



**FIGURE 8-7 SATURATED HYDRAULIC CONDUCTIVITY SENSITIVITY ANALYSIS**



**TABLE 8-1 VARIATIONS IN PEAK FLOWS AS A RESULT OF ADJUSTING HYDROLOGIC AND HYDRAULIC PARAMETERS**

Variation	Average Capillary Suction	Initial Moisture Deficit	Saturated Hydraulic Conductivity	Subbasin Width	Percent Impervious Area	Manning's Roughness Values	Depression Storage
10%	-0.04%	-0.04%	-0.04%	3.96%	5.15%	-3.83%	-0.12%
5%	-0.01%	-0.01%	-0.01%	2.01%	2.59%	-1.98%	-0.06%
0%	0%	0%	0%	0%	0%	0%	0%
-5%	0.01%	0.01%	0.01%	-2.08	-2.63%	2.12%	0.06%
-10%	0.04%	0.04%	0.04%	-4.23%	-5.31%	4.39%	0.12%

## 8.2 Hydraulic Model Sensitivity

A range of sensitivity tests were performed to ascertain how uncertainty in the model parameters impacts the robustness of the model output. The key parameters considered were 2D roughness coefficients and peak discharge rates. The 1:100 AEP event for the existing development conditions was selected as a benchmark. The robustness of the model output was assessed in terms of the change in water level at the upstream of Kings Bridge Road. Sensitivity analysis for the parameters was limited to  $\pm 5\%$  and  $\pm 10\%$ . Results of model sensitivity to changes in selected parameters are presented in Table 8.2.

It can be seen that generally, the variations of Manning’s n roughness and peak flow rate yield sensible and uniform variations in water levels. The results indicate that the hydraulic model is more sensitive to changes in the peak flow rates. Decreasing the peak flow rates by 10% decreased



peak water level at the upstream of Kings Bridge Road by 1.64%. The Manning's n values impacting the flows by 0.62% for a parameter change of 10%.

**TABLE 8.2 VARIATIONS IN PEAK WATER LEVEL AS A RESULT OF ADJUSTING HYDRAULIC PARAMETERS**

Variation	Water Level (m)		Water Level Variation	
	Manning's n	Peak Flow Rate	Manning's n	Peak Flow Rate
10%	14.63	14.69	0.46%	0.86%
5%	14.60	14.63	0.21%	0.44%
0%	14.57	14.57	0%	0%
-5%	14.53	14.50	-0.25%	-0.48%
-10%	14.48	14.33	-0.62%	-1.64%

**FIGURE 8-8 PEAK FLOW RATE SENSITIVITY ANALYSIS**

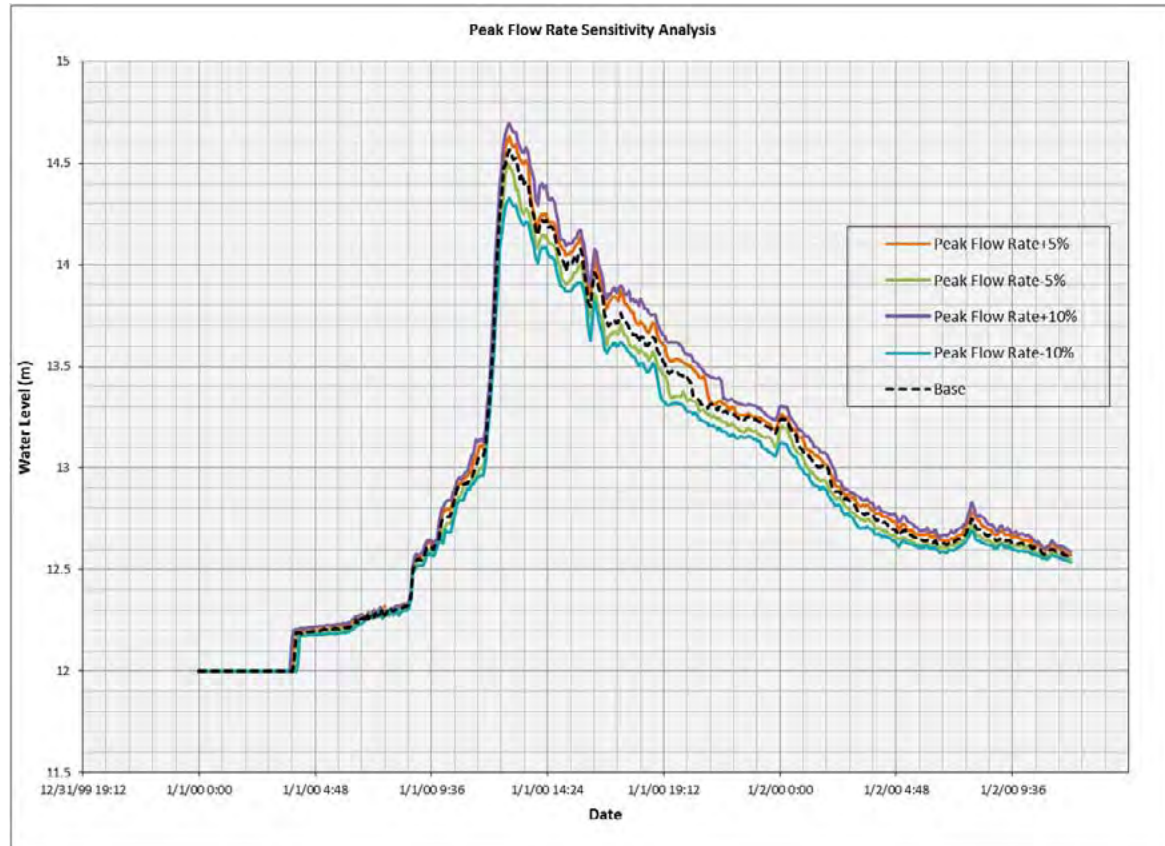
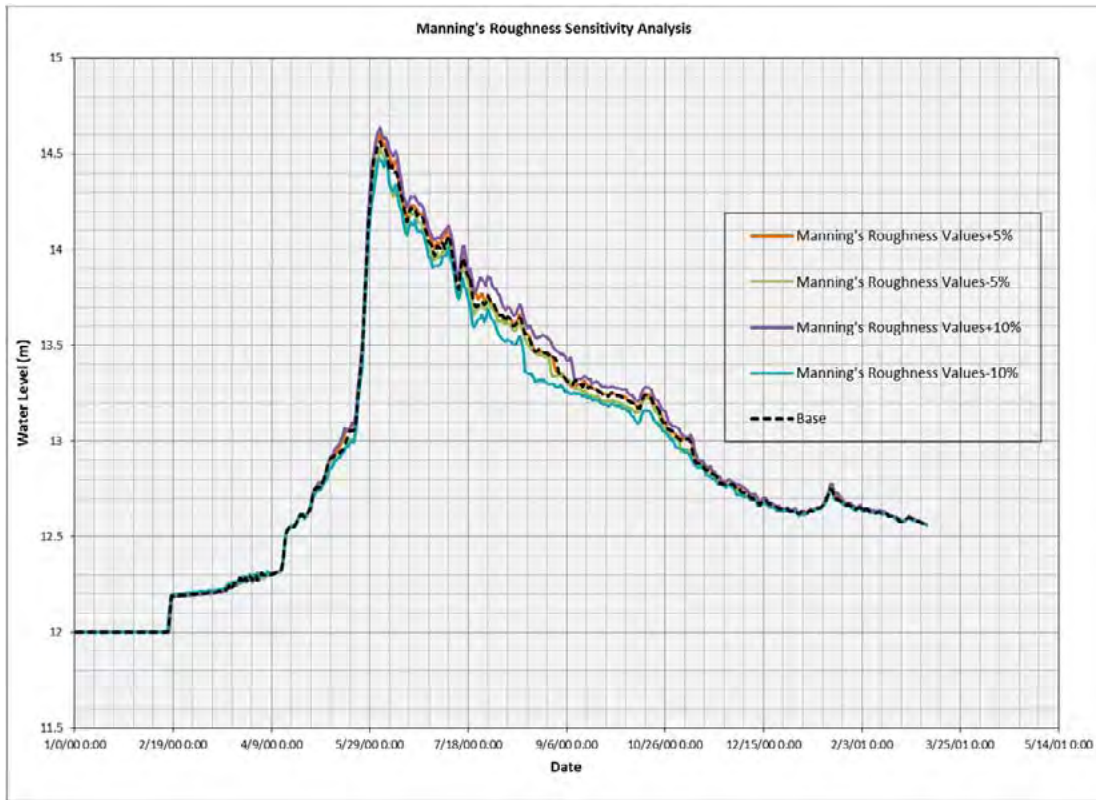


FIGURE 8-9: MANNING'S ROUGHNESS VALUE SENSITIVITY ANALYSIS



## CHAPTER 9 FLOODPLAIN AND FLOOD HAZARD MAPPING

Appendix J contains the floodplain mapping for the 1:20 and 1:100 AEP flood events for the existing and future development conditions.

The existing conditions mapping is based on the following:

- Existing land development conditions; and
- Flows developed using the updated IDFs and alternating block method.

The future conditions mapping is based on the following:

- Future land development conditions;
- Flows developed using the climate change projections and alternating block method; and
- City's Stormwater Detention Policy is in place.

Floodplain mapping for the future 1:100 AEP flood with the proposed flood control improvements is contained in Appendix K. The proposed improvements are discussed in Chapter 10. For the mapping, we have assumed that Option A at Location 1, which is discussed in Chapter 10, will be implemented.

Appendix L contains the flood hazard mapping for the 1:20 and 1:100 AEP flood events for the existing and future development conditions and for the future 1:100 AEP flood with the proposed flood control improvements.

## CHAPTER 10 PRELIMINARY DESIGN

### 10.1 Flood Control

Locations along Rennies River with known flooding problems include:

- Rear yards of properties located on Winter Avenue and Empire Avenue upstream from the Kings Bridge Road Bridge;
- Fieldian Grounds;
- Portugal Cove Road north of the Portugal Cove Road Bridge;
- Properties located on the south side of Pringle Place just upstream from the Portugal Cove Road Bridge;
- Properties located on the south side of Vaughan Place just upstream from the Carpasian Road Bridge;
- Prince Philip Drive east of the Prince Philip Drive Bridge;
- Prince Philip Drive and the CBC Building parking lot between Clinch Crescent East and Clinch Crescent West;
- Wicklow Street Bridge to Thorburn Road;
- Below O’Leary Avenue Bridge to the Avalon Mall Parking Lot culvert, and
- Local culverts on Ken Brook where the brook runs parallel to Kenmount Road.

For this study, we have considered the following typical flood control approaches:

- Conveyance capacity upgrades:
  - Culvert/bridge upgrades
  - Berms
  - Channel widening and deepening
- Storage to reduce flows:
  - Increase Storage in existing water bodies
  - Stormwater detention facilities
- Infiltration to reduce runoff:
  - Disconnection of roof downspouts
  - Perforated pipes installed in clearstone bedding
  - Rain gardens

These options have been investigated to assess their potential for flood mitigation. In general, all are feasible, with the exception of channel widening and deepening, which would not likely be acceptable to Department of Fisheries and Oceans (DFO). Dredging activities, if carried out along the entire river width, will harm fish habitat. The established natural balance of naturally graded rock and gravel will be removed by the dredging and is unlikely to be replaced with a similar mix. In addition, the natural low flow pool and riffle system, critical for fish survival during summer low flows, is also unlikely to be replaced.

Flood control approaches that involve infiltrating runoff to reduce peak flows face the challenges of low permeability soils and difficulty of implementation in urbanised areas. Nevertheless, infiltration measures can be implemented over the long term and even in low permeability soil conditions will provide a clear runoff reduction impact.

The immediate options envisaged to tackle the 1:100 AEP flood risks therefore include conveyance capacity upgrades as well as storage to reduce flows. Each part of the study area faces some challenges, but also offers opportunities with regards to the efficiency of each option considered. Each part of the study area will therefore have a customised best-fit recommendation for the flood mitigation measure recommended. In order to follow this approach, each type of option was evaluated using the hydrologic and hydraulic models. Further, the flood protection analysis was carried out based on the assumption that the City's Stormwater Detention Policy is implemented for any additional development in the Rennie's River watershed. The locations of the proposed flood control measures are shown on Figure 10-1 and described below.

### **Location 3 – Outlet of Long Pond**

In terms of overall impact on the study area, the most significant flood protection improvement is the weir located at the east end of Long Pond, which is noted as location 3 on Figure 10-1. Constructing a weir at this location will result in water being temporarily stored in Long Pond during a storm event and released at a lower flow rate than the flow rate would be without the weir in place. Due to the increased storage capacity, the level of Long Pond would increase for a short period of time during a storm and return to its normal level a short time after the end of a storm.

The overall increase in the storage capacity of Long Pond with the weir in place is in the order of 160,000 m<sup>3</sup>. The normal water level of Long Pond is approximately 53 m and will increase to approximately 55.7 m during the future 1:100 AEP flood event with the weir in place.

The major benefit of the weir is that the peak flows downstream of Long Pond will be reduced, resulting in reduced costs associated with the implementation of flood control options at locations downstream. For example, berms or walls proposed at locations downstream of Long Pond will not be as high with the weir in place because the peak flows will be reduced. In order to realize these benefits, the weir must be constructed before the other downstream improvements. A preliminary design of the weir is presented on Figures 10-2 and 10-3. Note that an opening in the weir will provide for the passage of fish.

### **Location 1 – Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road Bridge**

Three options for flood control are presented for the river section between Kings Bridge Road and Portugal Cove Road and immediately upstream from the Portugal Cove Road Bridge. This area is noted as location 1 on Figure 10-1. Along this river section, flooding has historically occurred at some of the rear yards along Winter Avenue and Empire Avenue, at Fieldian Grounds, and at some of the properties located on the south side of Pringle Place. Refer to Map 1 contained in Appendix I for the approximate extents of the flooding experienced during Hurricane Igor.

The options for flood control between Kings Bridge Road and Portugal Cove Road include:

#### **Option 1A – Construct berms and walls only along the river section**

This option involves constructing flood protection walls and earth berms such that flood water would be entirely contained within the river channel during a storm event. It does not include replacing the existing Portugal Cove Road Bridge. It should be noted that the earth berms proposed for the rear yards along Empire Avenue and the southeastern boundary of Fieldian Grounds may conflict with private property. Further, the construction of berms in these areas would result in a loss of mature trees located along the river banks, and temporary ponding on the property side of the berms during significant rainfall events.

#### **Option 1B – Realign the river to flow across Fieldian Grounds and construct berms and walls**

For this option, a new bridge would be required at Portugal Cove Road and the river would be realigned to flow through an existing property on Portugal Cove Road and through the tennis courts and soccer fields. For this option to proceed, consideration would have to be given to relocating the sports field, which is beyond the scope of this project. Also, the earth berms proposed for the rear yards along Empire Avenue may conflict with private property. In addition, the construction of berms in this area would result in a loss of mature trees located along the river banks, and temporary ponding on the property side of the berms during significant rainfall events.

#### **Option 1C – Raise the Riverdale Tennis Club parking lot and construct walls**

For this option, the parking lot would be raised; however, some portion of Fieldian Grounds would flood during storm events. The advantage of this option is that berms would not need to be constructed on the existing fields. Again, the berms proposed for the rear yards along Empire Avenue may conflict with private property. Further, the construction of berms in this area would result in a loss of mature trees located along the river banks, and temporary ponding on the property side of the berms during significant rainfall events.

Conceptual drawings of Options 1A, 1B and 1C are presented in Figures 10-4 to 10-7. The final decision regarding which of the above options to implement will be made by the Department of Planning, Development and Engineering's senior management in consultation with Council.

For the river section above Portugal Cove Road, the existing trail on the north side of the river will have to be raised in order to accommodate the flood protection wall; otherwise, property at the rear of the yards along Pringle Place would be required to allow for the construction of a wider earth berm. Photo renderings of the proposed wall are shown in Figures 10-8 to 10-11.

There is also significant erosion along the river banks throughout location 1. These areas are identified in section 10.2 and recommendations regarding remedial measures are presented. The increase in velocities in these areas with the flood control measures in place is not significant. The recommended erosion control measures will be adequate with or without the implementation of the flood control improvements.

#### **Location 2 – Upstream of Capasian Road**

An earth berm is recommended for the north side of the river section above Capasian Road, which is noted as location 2 on Figure 10-1. The preliminary design is shown on Figures 10-12 and 10-13.

#### **Location 4 – Clinch Crescent East to Clinch Crescent West**

Earth berms and a concrete wall are recommended for the river section from Clinch Crescent East to Clinch Crescent West. Only the improvements associated with the south side of this location are included in the cost opinion because the north side would be the responsibility of the Provincial Government. The preliminary design for this location is shown on Figure 10-14.

#### **Location 5 – Wicklow Street to Thorburn Road**

Earth berms and a concrete wall are recommended for the river section from Wicklow Street to Thorburn Road. The preliminary design for this location requires that the height of the headwall and wing walls of the existing bridge be increased by approximately 0.8 m. The berms proposed for the area located on the east side of Baird Place may conflict with private properties, and the construction of berms in this area would result in the loss of mature trees. The preliminary design is shown on Figure 10-15.

#### **Location 6 – Upstream from Avalon Mall Culverts**

At this location, it is recommended that the concrete headwall be raised. The total length is approximately 100 m. As this work would be the responsibility of the Avalon Mall, a cost opinion has not been included with this report. The preliminary design is shown on Figure 10-16.

#### **Location 7 – O’Leary Avenue Bridge**

It is recommended that the O’Leary Avenue Bridge be replaced to accommodate future flood flows. The cost opinion for this replacement includes pre-cast structural culvert sections similar to those used for the Pippy Place Culvert replacement. In addition, an earth berm is required for the left bank of the downstream side of the bridge. The preliminary design is shown on Figure 10-17.

#### **Location 8 – Downstream of Mews Place Culvert**

An earth berm is recommended for the right bank of the downstream side of the Mews Place Culvert. The location of the earth berm is shown on Figure 10-18.

#### **Ken Brook**

During the 1:100 AEP event under future conditions, there is localized flooding at private culverts located on Ken Brook; however, Kenmount Road does not flood. In most cases, the hydraulic

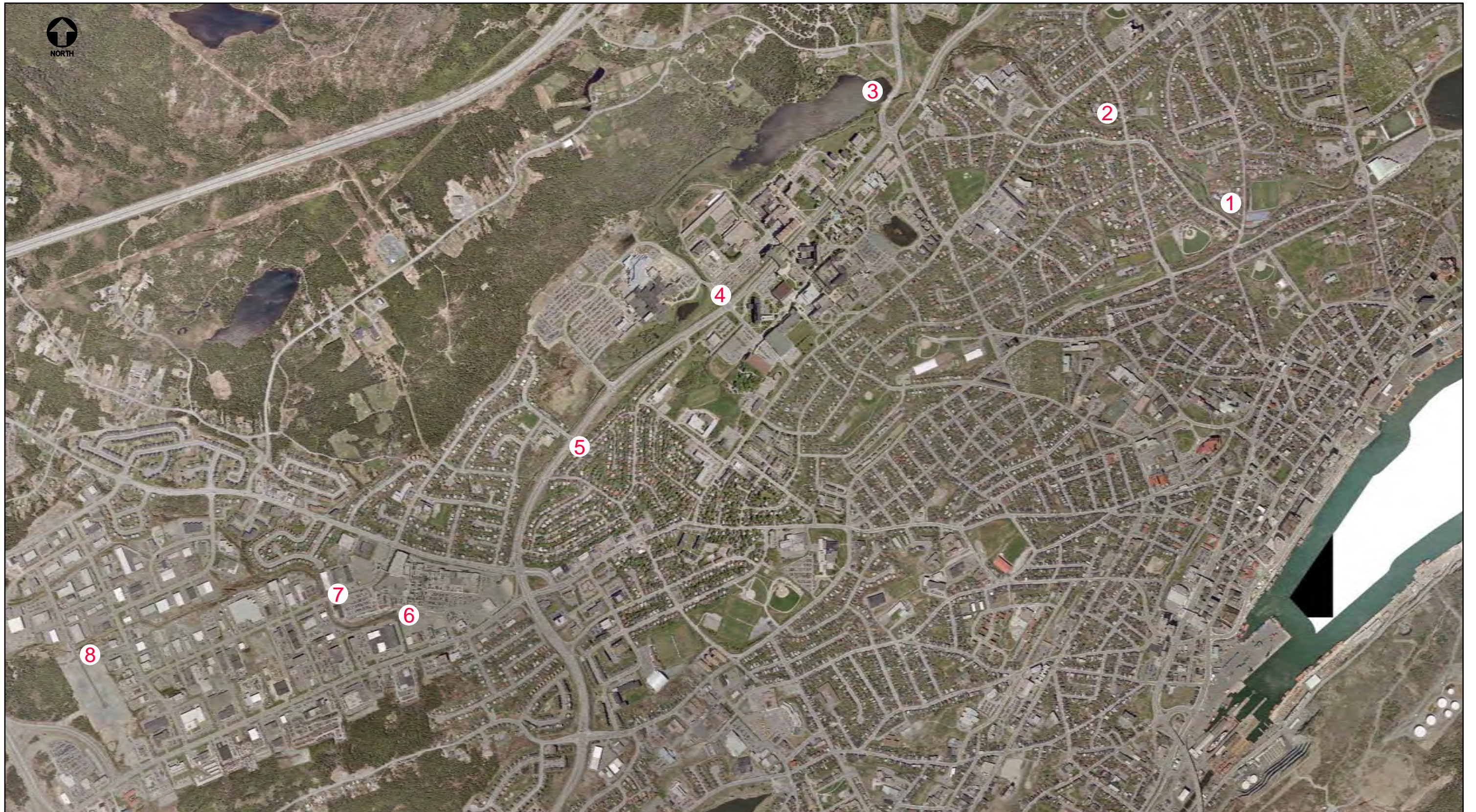
opening required to upgrade these culverts would result in the raising of private parking lots. Even then, there are some instances where cover for the culverts would not be adequate.

As noted, the construction of the weir at Long Pond will result in reduced flooding downstream of Long Pond. The flood control improvements located downstream of Long Pond at locations 2 and 3 have been designed to function with the weir at Long Pond in place. Consequently, the weir at Long Pond must be constructed before the downstream improvements can be constructed.

Given the extents of the flooding experienced at locations 2 and 3, these areas are considered by the City to high priority areas. Therefore, it is recommended that the weir at Long Pond be given first priority, and locations 2 and 3 be given second priority. It is recommended that the remaining flood improvement recommendations be implemented in order from downstream to upstream.

The recommended flood control measures are summarized in Table 10-1. Cost opinions include engineering, contingency and HST. Detailed breakdowns of the cost opinions are included in Appendix M.





PLAN

SCALE: 1:15000



DRAWN: J. LARACY	SCALE: 1:15000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO:	

**FIGURE 10-1**

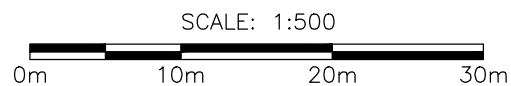
**CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN**

**FLOOD CONTROL IMPROVEMENTS  
LOCATION PLAN**

**CBCL** **CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627



PLAN



LEGEND:

 PROPOSED WEIR

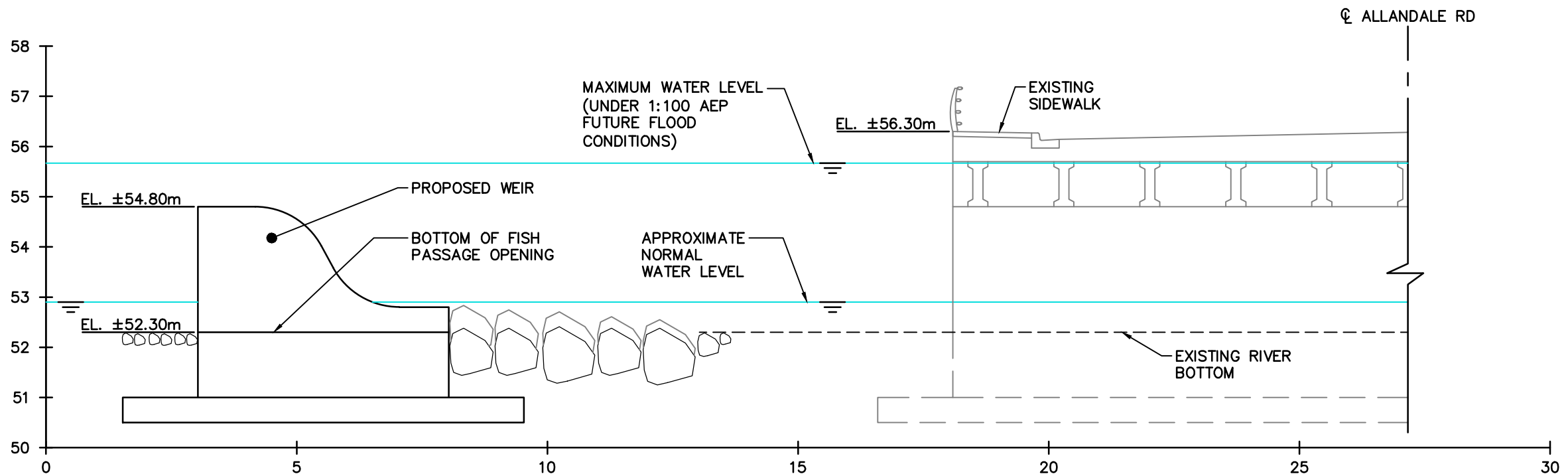
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DRAWING NO:	

FIGURE 10-2

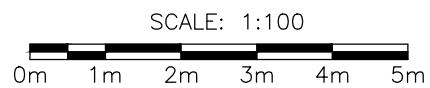
CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN

LOCATION 3  
LONG POND

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**SECTION A - A**



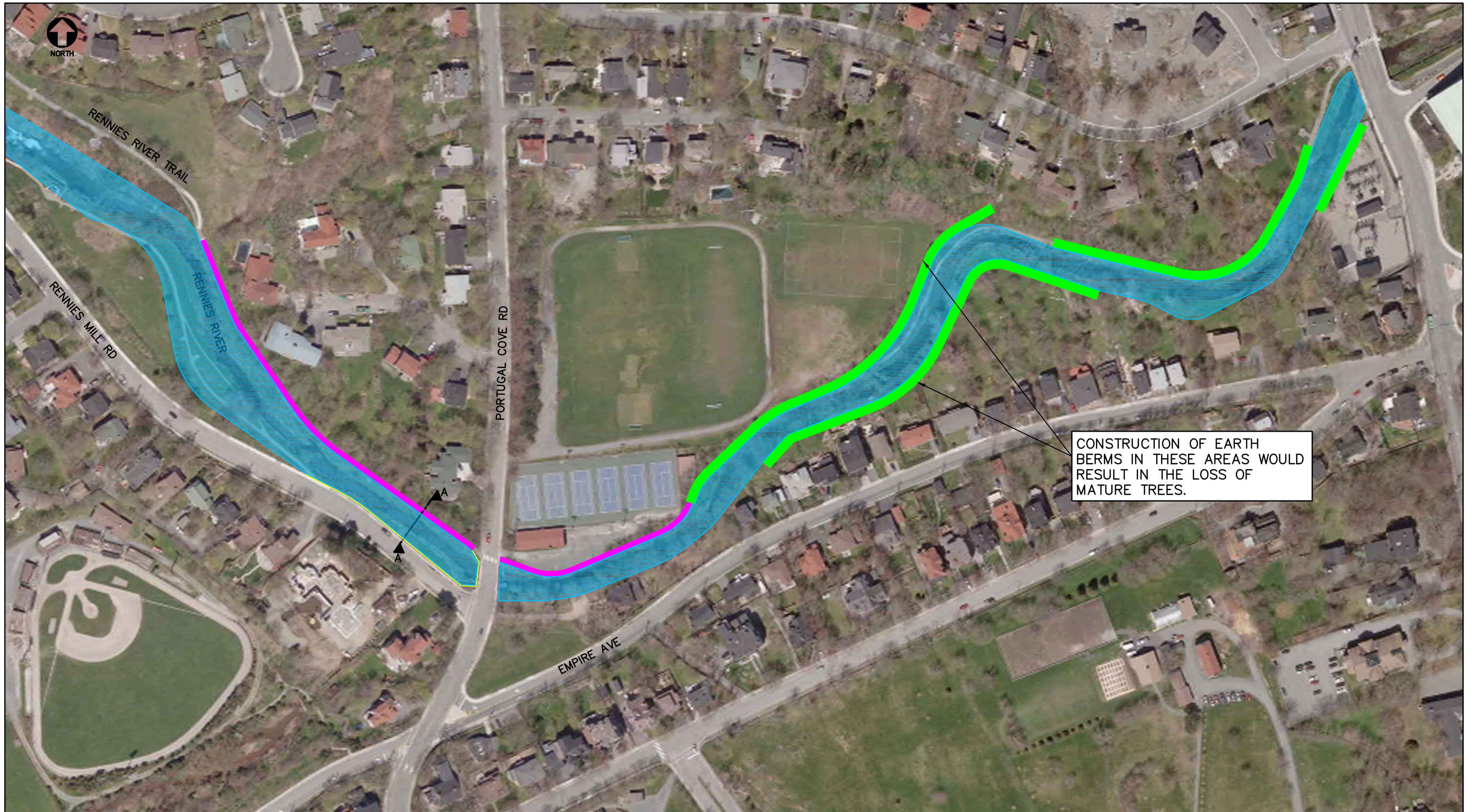
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PROJECT NO: <b>123097.00</b>	DATE: <b>APR 15, 2014</b>

**CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN**

DRAWING NO:  
**FIGURE 10-3**

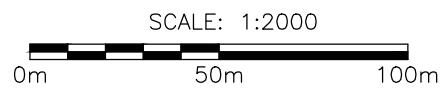
**LOCATION 3  
LONG POND**

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CONSTRUCTION OF EARTH BERMS IN THESE AREAS WOULD RESULT IN THE LOSS OF MATURE TREES.

PLAN



LEGEND:


-  PROPOSED SEGMENTAL CONCRETE BLOCK WALL
-  PROPOSED CAST-IN-PLACE CONCRETE WALL
-  PROPOSED EARTH BERM

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PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO:	

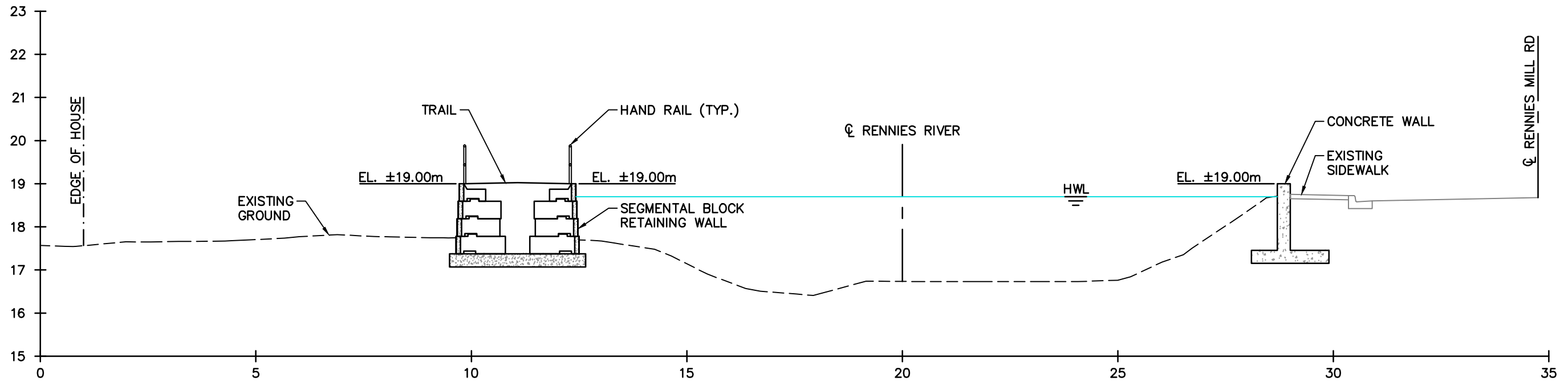
CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN

FIGURE 10-4

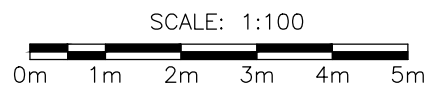
LOCATION 1 - OPTION A  
PORTUGAL COVE ROAD



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**SECTION A - A**



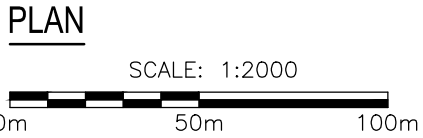
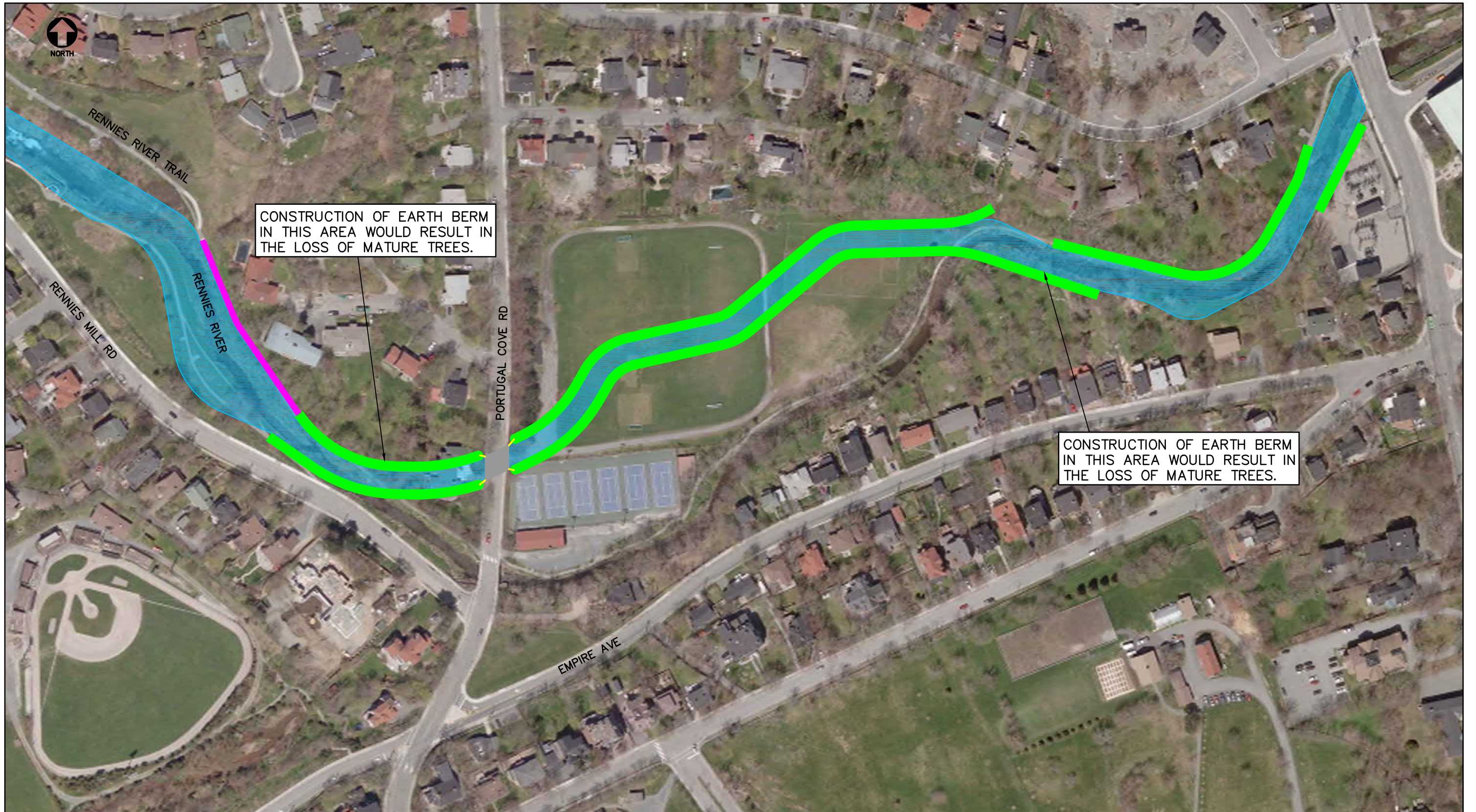
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PROJECT NO: 123097.00	DATE: APR 15, 2014

**CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN**

DRAWING NO:  
**FIGURE 10-5**

**LOCATION 1 - OPTION A  
PORTUGAL COVE ROAD**

**CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627



- LEGEND:**
- PROPOSED SEGMENTAL CONCRETE BLOCK WALL
  - PROPOSED CAST-IN-PLACE CONCRETE WALL
  - PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: 1:2000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO:	

**FIGURE 10-6**

**CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN**

**LOCATION 1 - OPTION B  
PORTUGAL COVE ROAD**

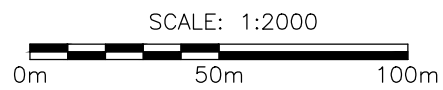
**CBCL** **CBCL LIMITED**  
Consulting Engineers

187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627






CONSTRUCTION OF EARTH BERM  
IN THIS AREA WOULD RESULT IN  
THE LOSS OF MATURE TREES.

PLAN



LEGEND:

-  PROPOSED SEGMENTAL CONCRETE BLOCK WALL
-  PROPOSED CAST-IN-PLACE CONCRETE WALL
-  PROPOSED EARTH BERM/  
RAISED PARKING LOT

DRAWN: J. LARACY	SCALE: 1:2000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO:	

**FIGURE 10-7**

**CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN**

**LOCATION 1 - OPTION C  
PORTUGAL COVE ROAD**

**CBCL** **CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627

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FIGURE 10-8 UPSTREAM OF PORTUGAL COVE ROAD BRIDGE – STREET VIEW – BEFORE IMPROVEMENTS



FIGURE 10-9 UPSTREAM OF PORTUGAL COVE ROAD BRIDGE – STREET VIEW – AFTER IMPROVEMENTS

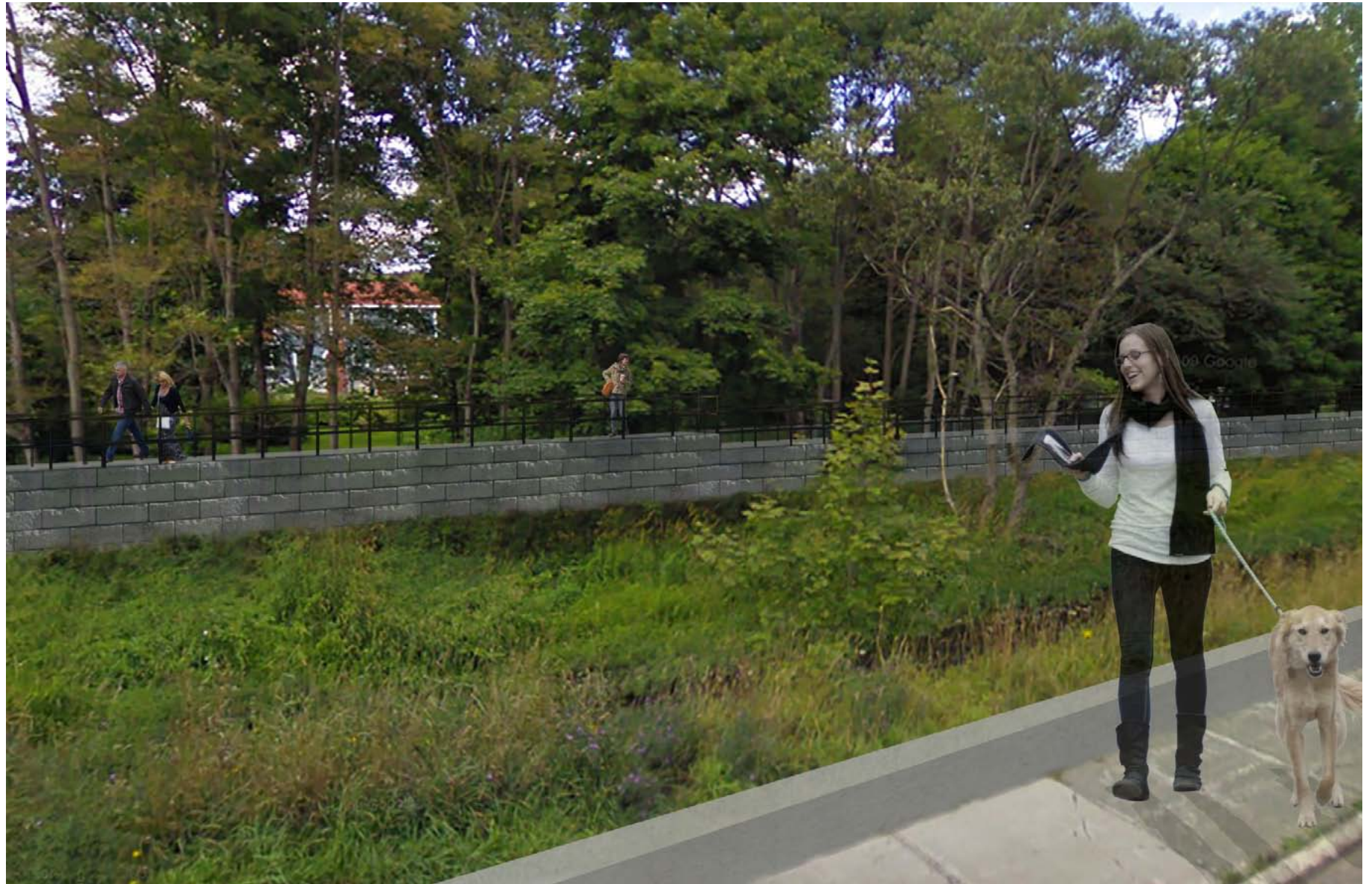
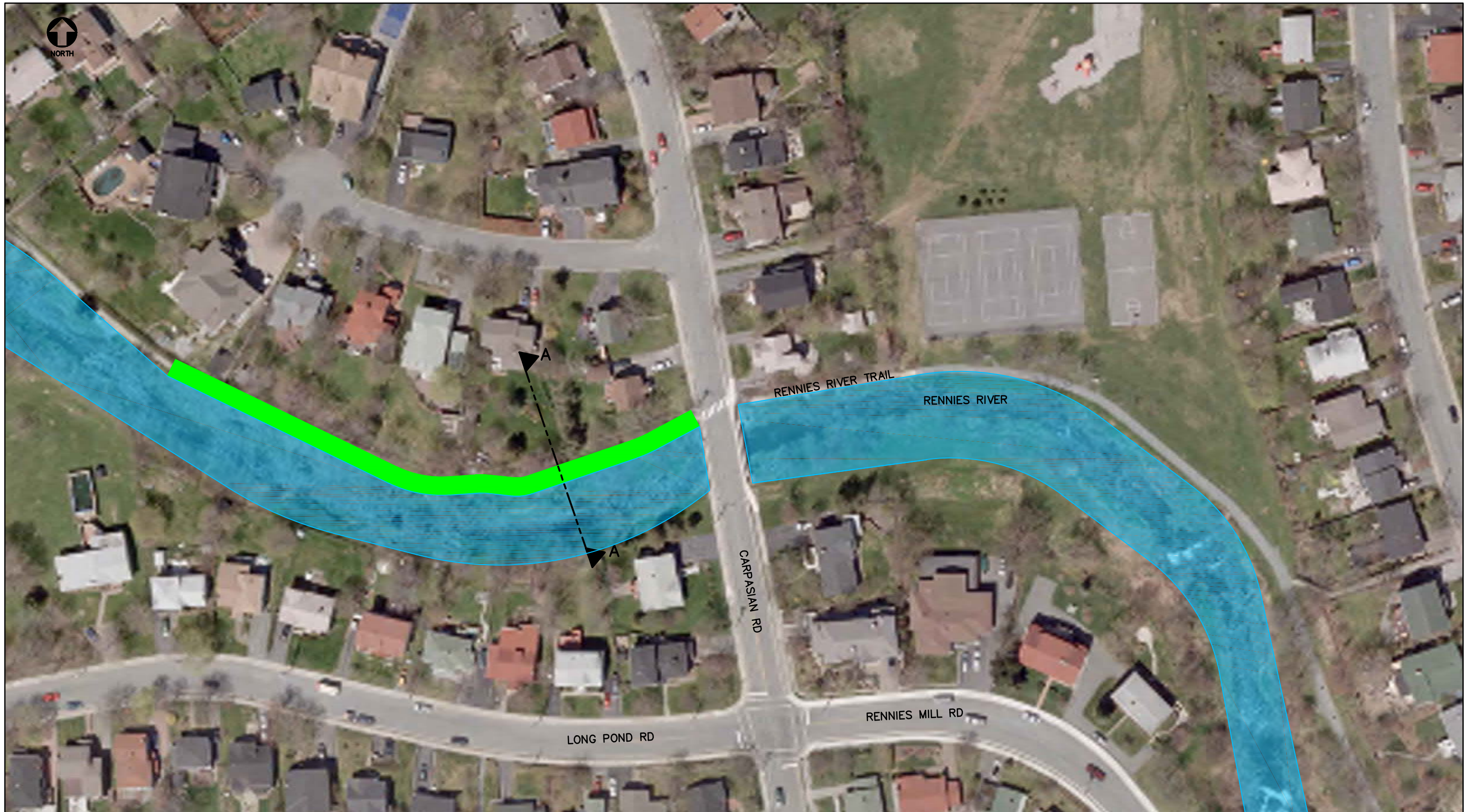


FIGURE 10-10 UPSTREAM OF PORTUGAL COVE ROAD BRIDGE – TRAIL VIEW – BEFORE IMPROVEMENTS



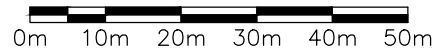
FIGURE 10-11 UPSTREAM OF PORTUGAL COVE ROAD BRIDGE – TRAIL VIEW – AFTER IMPROVEMENTS





PLAN

SCALE: 1:1000



LEGEND:

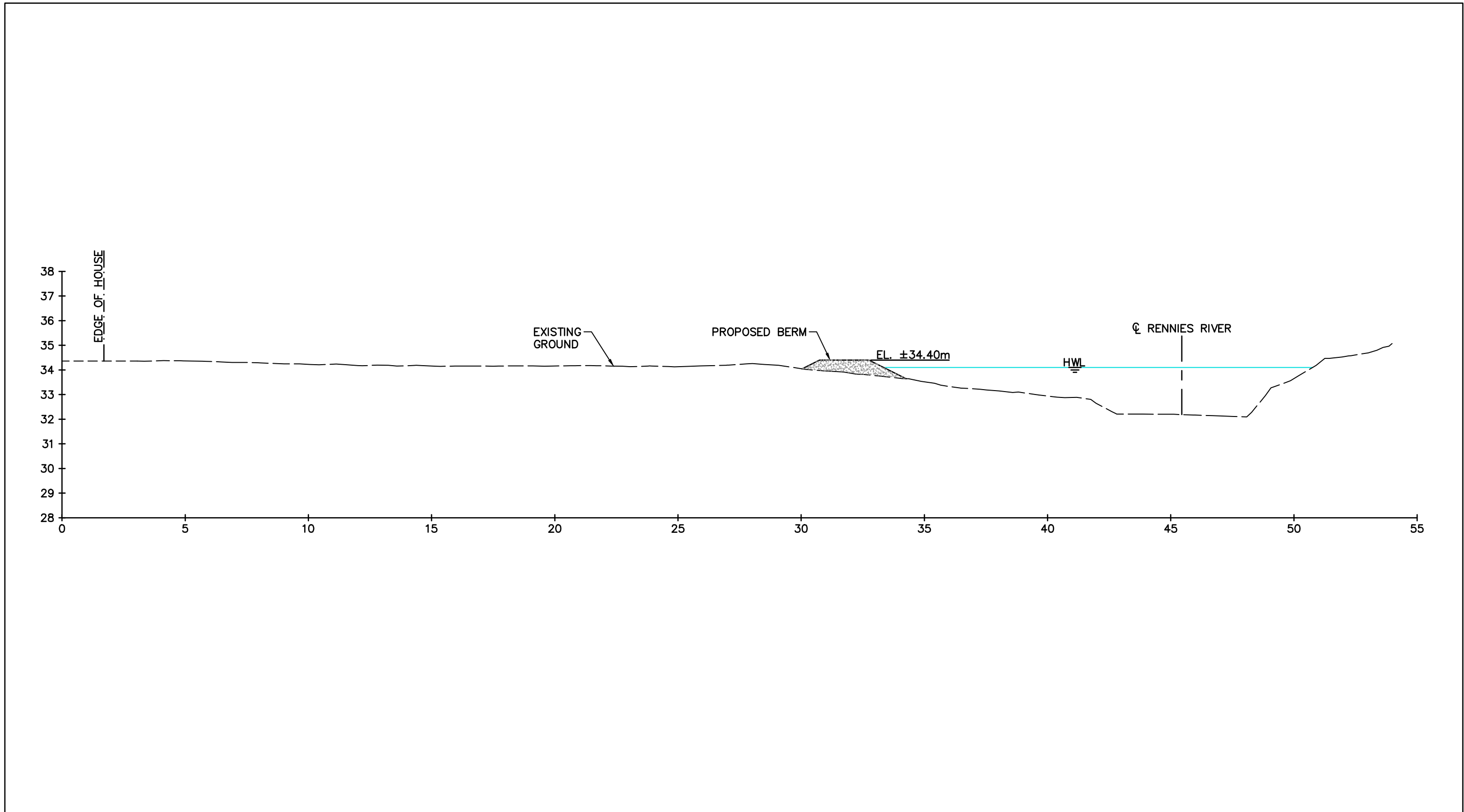
 PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: 1:1000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO: <b>FIGURE 10-12</b>	

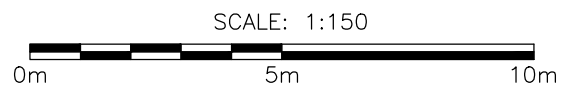
**CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN**

**LOCATION 2  
UPSTREAM OF CARPASIAN RD**

**CBCL** **CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627



SECTION A - A



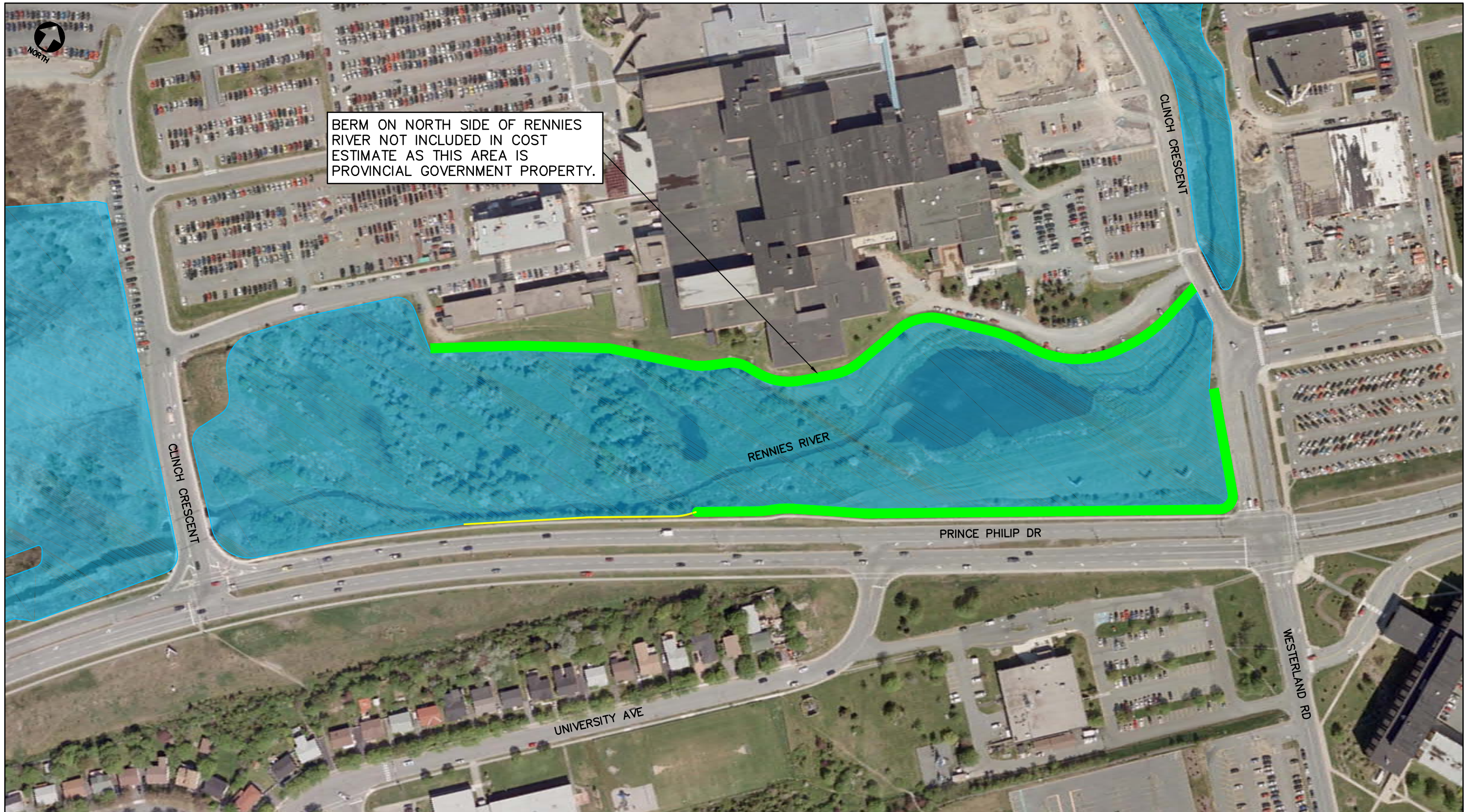
DRAWN: J. LARACY	SCALE: 1:150
PROJECT NO: 123097.00	DATE: APR 15, 2014

CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN

DRAWING NO:  
**FIGURE 10-13**

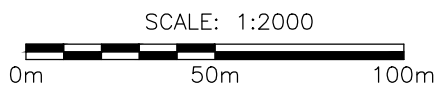
LOCATION 2  
UPSTREAM OF CARPASIAN RD

**CBCL** **CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627

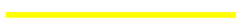



BERM ON NORTH SIDE OF RENNIES RIVER NOT INCLUDED IN COST ESTIMATE AS THIS AREA IS PROVINCIAL GOVERNMENT PROPERTY.

PLAN



LEGEND:


-  PROPOSED CAST-IN-PLACE CONCRETE WALL
-  PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: 1:2000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO:	

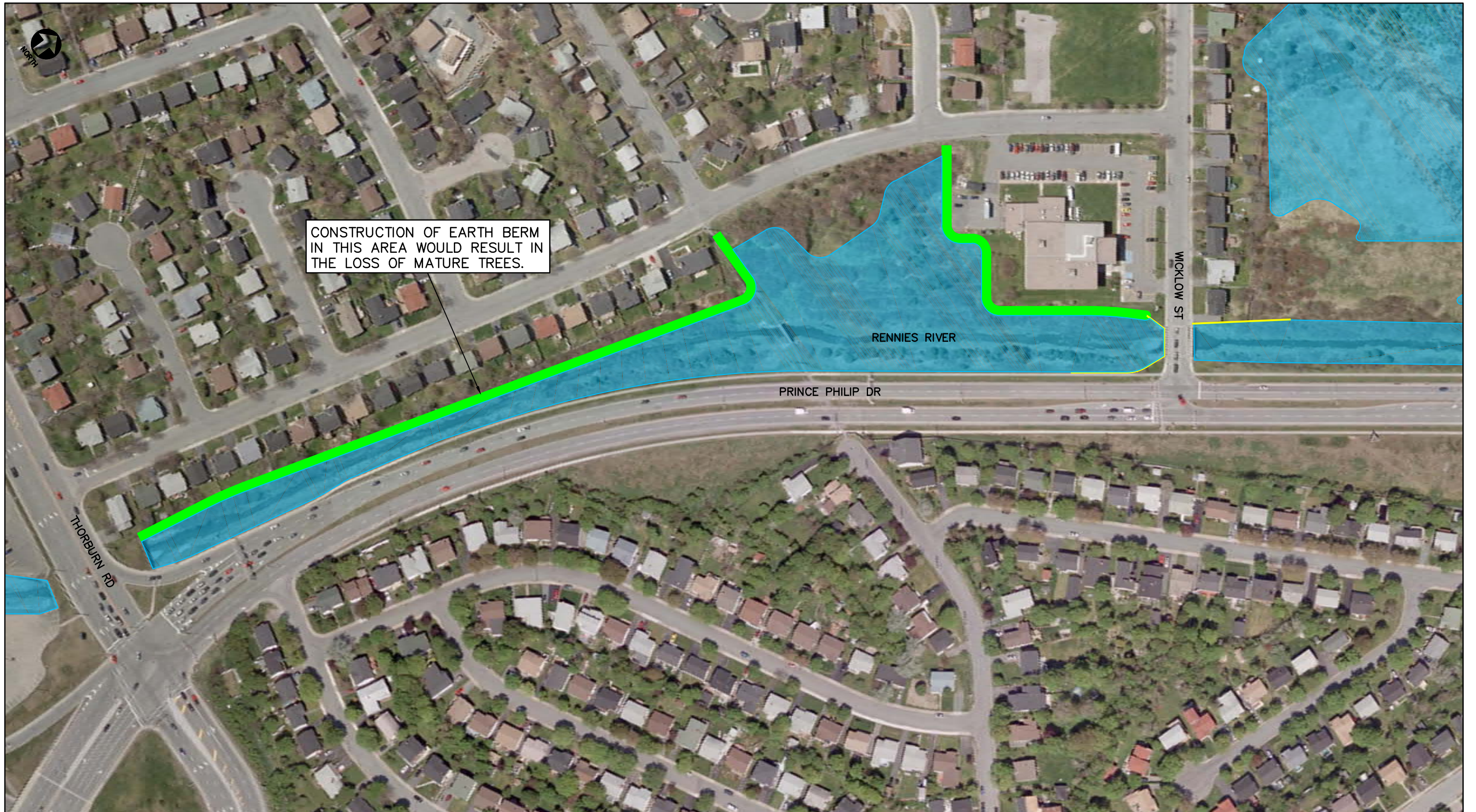
FIGURE 10-14

CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN

LOCATION 4  
CLINCH CRESCENT



**CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627





CONSTRUCTION OF EARTH BERM  
IN THIS AREA WOULD RESULT IN  
THE LOSS OF MATURE TREES.

PLAN

SCALE: 1:2000



LEGEND:


-  PROPOSED CAST-IN-PLACE CONCRETE WALL
-  PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: 1: 2000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO:	

FIGURE 10-15

CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN

LOCATION 5  
WICKLOW STREET



**CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627

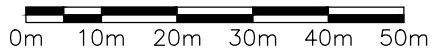





COST ESTIMATE FOR THIS IMPROVEMENT IS NOT INCLUDED IN REPORT AS THIS IMPROVEMENT WOULD BE RESPONSIBILITY OF AVALON MALL.

PLAN

SCALE: 1:1000



LEGEND:

 PROPOSED CAST-IN-PLACE CONCRETE WALL

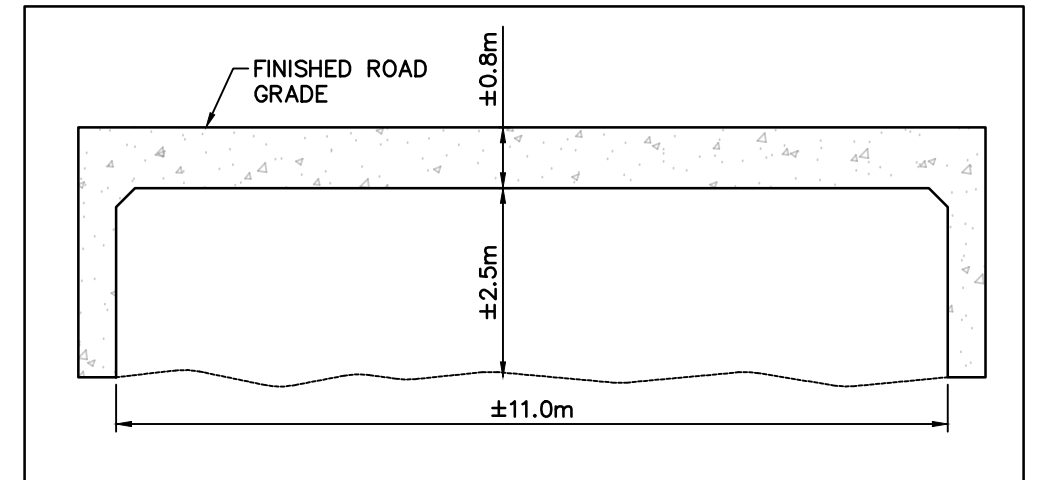
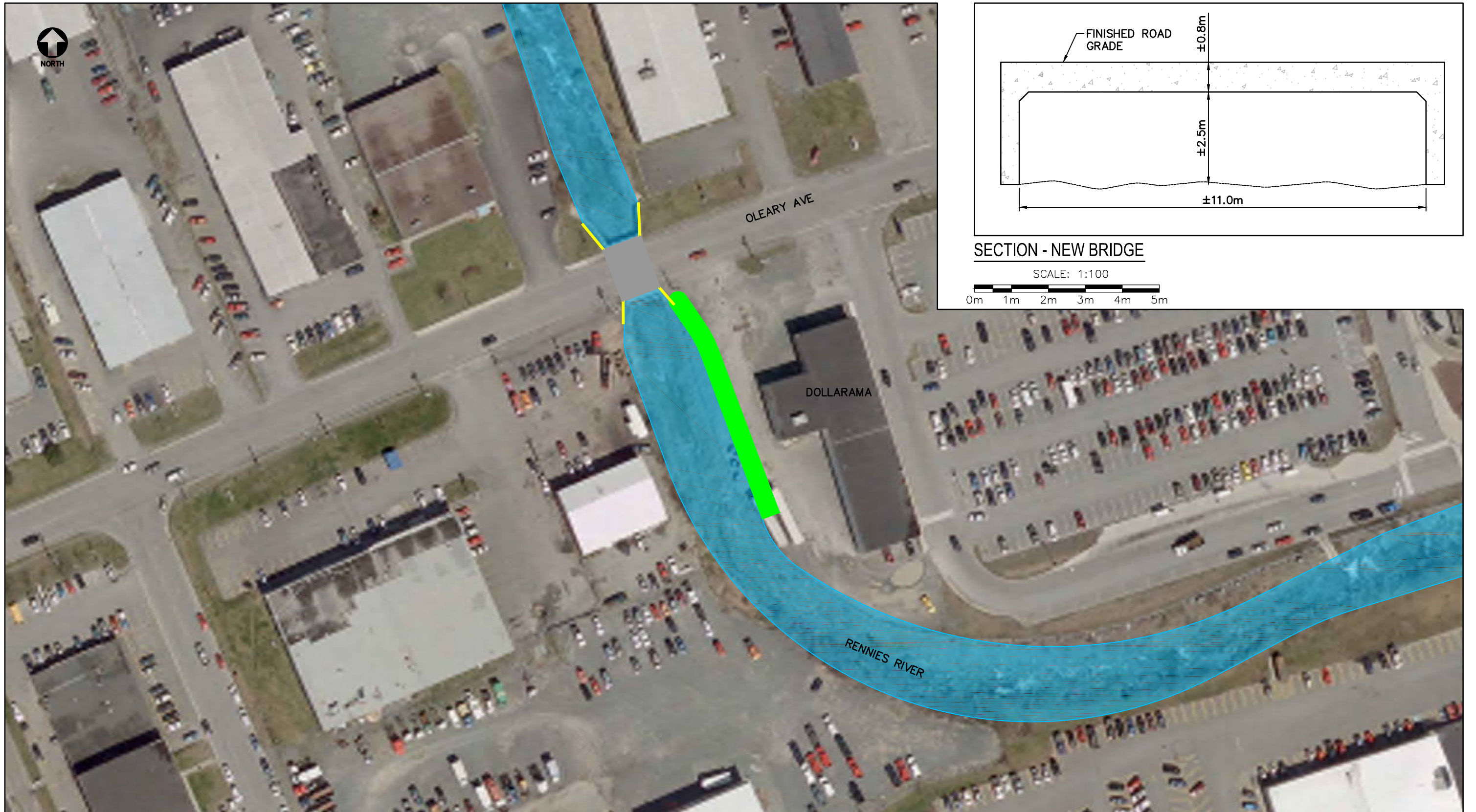
DRAWN: J. LARACY	SCALE: 1:1000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO:	

CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN

FIGURE 10-16

LOCATION 6  
AVALON MALL CULVERT

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Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627



SECTION - NEW BRIDGE

SCALE: 1:100

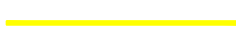

0m 1m 2m 3m 4m 5m

PLAN

SCALE: 1:1000

0m 10m 20m 30m 40m 50m

LEGEND:

-  PROPOSED CAST-IN-PLACE CONCRETE WALL
-  PROPOSED EARTH BERM

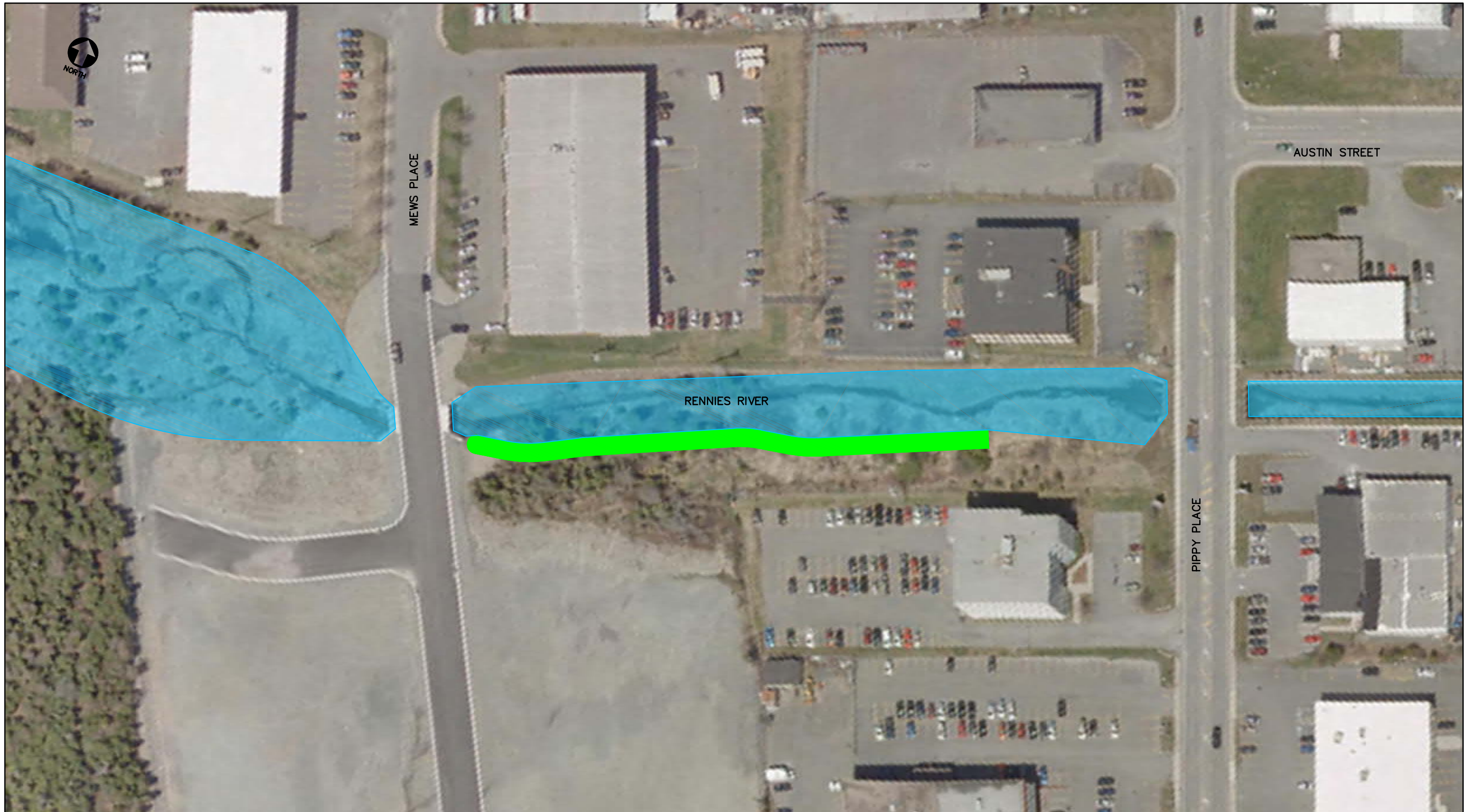
DRAWN: J. LARACY	SCALE: AS SHOWN
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO:	

FIGURE 10-17

CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN

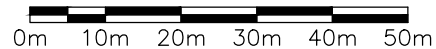
LOCATION 7  
O'LEARY AVENUE BRIDGE

**CBCL** **CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627



PLAN

SCALE: 1:1000



LEGEND:

 PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: 1:1000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO: <b>FIGURE 10-18</b>	

**CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN**

**LOCATION 8  
MEWS PLACE CULVERT**

**CBCL** **CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627

**TABLE 10-1 FLOOD PROTECTION IMPROVEMENT OPTIONS**

Priority	Location Number	Description of Location	Description of Improvement	Cost Opinion
1	3	Outlet of Long Pond	25m concrete weir and fish passage	\$1,979,000
2	1A	Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road	700m earth berm (avg. height = 1m); 340m segmental concrete block wall (avg. height = 1m); 130m cast-in-place concrete wall (avg. height = 0.4m)	\$1,173,000
	1B	Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road	300m earth berm (avg. height = 1m); 420m new channel (width = 12m); 110m segmental concrete block wall (avg. height = 1m); remove and replace bridge, remove house at 1 Portugal Cove Road	\$3,891,000
	1C	Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road	460m earth berm (avg. height = 1m); 230m segmental concrete block wall (avg. height = 1m); 130m cast-in-place concrete wall (avg. height = 0.4m); remove and replace tennis club building; raise parking lot and building pad	\$1,379,000
2	2	Upstream of Carpasian Road Bridge	150m earth berm at left bank (avg. height = 0.5m)	\$27,000
3	4	Clinch Crescent East to Clinch Crescent West	360m earth berm (avg. height = 1m); 120m cast-in-place wall (avg. height = 0.4m)	\$342,000
4	5	Wicklow Street to Thorburn Road	580m earth berm (avg. height = 1m); 120m cast-in-place wall (avg. height = 0.8m)	\$294,000
5	7	O'Leary Avenue Bridge	70m earth berm at left bank; remove and replace bridge	\$847,000
6	8	Downstream of Mews Place	140m earth berm at right bank (avg. height = 1.6m)	\$38,000

Notes:

1. For Priority 2, only one option will be implemented (i.e. one of 1A, 1B or 1C).
2. Right/Left banks are noted as looking downstream.

## 10.2 Erosion Control

Erosion control is a necessary part of the channel improvements needed to safely convey the future flows. The following areas, shown on Figures 10-19 to 10-21, have been identified as being particularly vulnerable:

1. The river section between Kings Bridge Road and Portugal Cove Road;
2. The river section between the inlet of the Avalon Mall Culvert and O'Leary Avenue;
3. The river section between O'Leary Avenue and Pippy Place; and
4. The river section upstream of Pippy Place.

The model results under the 1:100 AEP future flood condition with and without the recommended flood protection measures in place revealed that no new areas of erosion in addition to those listed above will be created through the implementation of the flood protection improvements.

The model results under the 1:100 AEP future flood condition with the recommended flood improvements in place indicate that the velocities in the bend outer banks in the section between O'Leary Avenue and Pippy Place can reach 5 to 8 m/s. Similarly, the model results indicate that the bend outer banks in the channel downstream of Portugal Cove Road could experience velocities in the order of 4 to 6 m/s.

Velocities in the order of 4-8 m/s are considered high and can only be protected with very large rip-rap sizes. Indeed, Table 10-2 below shows the rip-rap size that would be needed to protect the channel banks. In this calculation, 3 m of water depth, and a radius of bend centreline to width ratio of 3:1 is used.

**TABLE 10-2      CALCULATION OF RIP-RAP SIZES FOR VARIOUS FLOW VELOCITIES**

<b>Water Velocity:</b>	<b>V = 4 m/s</b>	<b>V = 5 m/s</b>	<b>V = 6 m/s</b>	<b>V = 7 m/s</b>	<b>V = 8 m/s</b>
<b>Percent Passing</b>	<b>Sieve Size (mm)</b>	<b>Sieve Size (mm)</b>	<b>Sieve Size (mm)</b>	<b>Sieve Size (mm)</b>	<b>Sieve Size (mm)</b>
10	500	870	1370	2010	2810
30	990	1730	2740	4020	5620
60	1490	2600	4110	6040	8430
100	1990	3470	5470	8050	11230

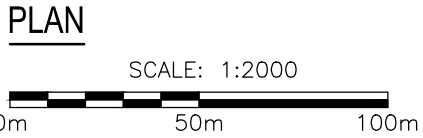
Based on this calculation, it is not recommended to use rip-rap for bank protection. Other systems such as high-performance cellular confinement systems may be better suited to this application. Other advantages of such systems include very low thickness which encroaches less on the river, as well as improved aesthetic appeal in residential areas. Product literature for one type of cellular confinement system is contained in Appendix N. This system can be used to protect embankments that experience velocities of up to 9 m/s and on embankments with a maximum slope of 1:1.

It is estimated that approximately 4000 m<sup>2</sup> of the river banks need to be rehabilitated. Based on using a cellular confinement system, the cost opinion to do this work is \$567,000, including engineering, contingency and HST. The detailed breakdown of the cost opinion is included in Appendix M.

Sediment depositions have accumulated at several bridges and culverts along Rennies River, including the following locations:

- Portugal Cove Road Bridge;
- Clinch Crescent East Bridge; and
- Wicklow Street Bridge.

The accumulation of rock and gravel at bridges and culverts can significantly reduce hydraulic capacities. Accordingly, this material should be removed periodically. The City has removed rock and gravel deposits at bridges and culverts in the past.



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CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN

**FIGURE 10-19**

**EROSION CONTROL IMPROVEMENTS  
LOCATION 1**

**CBCL** **CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627



DRAWN: J. LARACY	SCALE: 1: 3000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO: <b>FIGURE 10-20</b>	

<b>CITY OF ST. JOHN'S</b> <b>RENNIES RIVER CATCHMENT</b> <b>STORMWATER MANAGEMENT PLAN</b>
<b>EROSION CONTROL IMPROVEMENTS</b> <b>LOCATIONS 2-3</b>

	<b>CBCL LIMITED</b> <b>Consulting Engineers</b> 187 KENMOUNT ROAD ST. JOHN'S, NL, A1B 3P9 Phone: (709) 364-8623 Fax: (709) 364-8627
--	--





LOOSE BANK MATERIAL US OF BRIDGE ON RIGHT HAND SIDE LOOKING US. APPROX. LENGTH IS 200m.

4

3

PLAN

SCALE: 1:3000



DRAWN: J. LARACY	SCALE: 1: 3000
PROJECT NO: 123097.00	DATE: APR 15, 2014

CITY OF ST. JOHN'S  
RENNIES RIVER CATCHMENT  
STORMWATER MANAGEMENT PLAN

DRAWING NO:  
**FIGURE 10-21**

**EROSION CONTROL IMPROVEMENTS  
LOCATION 4**

**CBCL** **CBCL LIMITED**  
Consulting Engineers  
187 KENMOUNT ROAD  
ST. JOHN'S, NL, A1B 3P9  
Phone: (709) 364-8623  
Fax: (709) 364-8627

## CHAPTER 11 REGULATORY REQUIREMENTS

CBCL contacted the provincial Department of Environment and Conservation (DOEC) and DFO to establish each agency's regulatory requirements with respect to the proposed infrastructure improvements.

### 11.1 Department of Environment and Conservation

A Water Management Specialist with DOEC advised CBCL that DOEC does not review stormwater work unless it is incidental to water and/or sewer works (for example, a combined sanitary and storm sewer), or involves an outfall (for example, a storm sewer pipe discharging to a river). As such, DOEC considers stormwater management to fall under the jurisdiction of individual municipalities. While the province is in the process of creating a stormwater management policy directive, it is intended for the benefit of rural communities; major municipalities are expected to continue to oversee their own stormwater management activities.

In general, DOEC requires an Application for Permit to Alter a Body of Water and corresponding schedules to be submitted for approval. These documents are found on DOEC's webpage at <http://www.env.gov.nl.ca/env/waterres/regulations/appforms/index.html>. For the proposed detention facilities and outlet control structures, Schedules A and H are required along with the application.

### 11.2 Department of Fisheries and Oceans

When we contacted DFO to solicit their input on the proposed infrastructure improvements, they asked to see the report before providing any comments. We recommend that a copy of the final report be provided to DFO for their review.

## CHAPTER 12 CONCLUSIONS AND RECOMMENDATIONS

### 12.1 Conclusions

1. Flood protection infrastructure improvements are recommended for the following locations. The weir at the outlet of Long Pond must be implemented before the other recommended improvements for downstream locations. Only one of the options presented for location 1 needs to be implemented.

Priority	Description of Location	Cost Opinion
1	Location 3: Weir at outlet of Long Pond	\$1,979,000
2	Location 1, Option A: Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road – Berms & Walls only	\$1,173,000
	Location 1, Option B: Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road – New Channel and bridge	\$3,891,000
	Location 1 Option C: Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road – Raised parking lot	\$1,379,000
2	Location 2: Upstream of Carpasian Road Bridge	\$27,000
3	Location 4: Clinch Crescent East to Clinch Crescent West	\$342,000
4	Location 5: Wicklow Street to Thorburn Road	\$294,000
5	Location 7: O'Leary Avenue Bridge	\$847,000
6	Location 8: Downstream of Mews Place	\$38,000

2. Erosion control improvements can be accomplished using a cellular confinement system. It is estimated that approximately 4000 m<sup>2</sup> of the river banks need to be rehabilitated. Based on using a cellular confinement system, the cost opinion to do this work is \$567,000.
3. DOEC requires that the Application for Permit to Alter a Body of Water and corresponding schedules A and H be submitted for review and approval.

### 12.2 Recommendations

1. CBCL recommends that the City move forward with the design and implementation of the proposed flood and erosion control improvements. Further discussion regarding the preferred option for location 1 is required before moving ahead with the design.
2. CBCL recommends that DOEC and DFO be consulted during the design of the proposed infrastructure improvements.

**APPENDIX C**  
**CORRESPONDENCE**

## White, Tyler

---

**Subject:** FW: 174811 - HSC floodplain (AutoCAD Dwg)

**From:** Dave Wadden [<mailto:DWadden@stjohns.ca>]  
**Sent:** April-29-14 7:46 AM  
**To:** Reid, Lance  
**Subject:** Fw: 174811 - HSC floodplain (AutoCAD Dwg)

Lance:

Here are the elevations from our Consultant for the floodplain mapping:

- Point 1 – 56.95 m
- Point 2 – 57.40 m
- Point 3 – 57.56 m

Dave Wadden, M.Eng., P.Eng.  
Manager, Development - Engineering  
Planning, Development & Engineering  
City of St. John's  
Phone: (709)-576-8260  
Fax: (709)-576-8625  
e-mail: [dwadden@stjohns.ca](mailto:dwadden@stjohns.ca)

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----- Forwarded by Dave Wadden/CSJ on 2014/04/29 07:45 AM -----

From: Dave Wadden/CSJ  
To: "Reid, Lance" <[lance.reid@amec.com](mailto:lance.reid@amec.com)>  
Date: 2014/04/28 11:02 AM  
Subject: RE: 174811 - HSC floodplain (AutoCAD Dwg)

---

I've sent the request to the consultant that created the floodplain and I'm awaiting their response.

Dave Wadden, M.Eng., P.Eng.  
Manager, Development - Engineering  
Planning, Development & Engineering  
City of St. John's  
Phone: (709)-576-8260  
Fax: (709)-576-8625  
e-mail: [dwadden@stjohns.ca](mailto:dwadden@stjohns.ca)

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From: "Reid, Lance" <[lance.reid@amec.com](mailto:lance.reid@amec.com)>  
To: "[dwadden@stjohns.ca](mailto:dwadden@stjohns.ca)" <[dwadden@stjohns.ca](mailto:dwadden@stjohns.ca)>  
Cc: "White, Tyler" <[Tyler.White@amec.com](mailto:Tyler.White@amec.com)>  
Date: 2014/04/28 10:49 AM  
Subject: RE: 174811 - HSC floodplain (AutoCAD Dwg)

---

Good morning Dave,

When do you expect to have an opportunity to provide the requested information?

Regards,

**Lance Reid, P. Eng.**

Project Director

Power and Process Division - Atlantic Region

AMEC Americas Limited

Office (709) 724-3553

Cell: (902) 223-6077

**From:** White, Tyler

**Sent:** April-15-14 2:40 PM

**To:** [dwadden@stjohns.ca](mailto:dwadden@stjohns.ca)

**Cc:** Reid, Lance

**Subject:** 174811 - HSC floodplain (AutoCAD Dwg)

Dave,

As discussed earlier – attached is an AutoCAD file with three points where we would like to have the 1:100 year flood elevation.

Regards,

**Tyler White, P.Eng.**

**Municipal Engineer**

**AMEC**

Power & Process Americas

130 Eileen Stubbs Avenue, Suite 201

Dartmouth, Nova Scotia, Canada

B3B 2C4

Tel (902) 420 8941

Fax (902) 420 8949

[tyler.white@amec.com](mailto:tyler.white@amec.com)

[amec.com](http://amec.com)

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**APPENDIX D**  
**GOVERNMENT POLICIES**

# Policy for Flood Plain Management

---

## POLICY DIRECTIVE

Division:	Water Resources Management	P.D.	W.R. 96-1
Prepared By:	Amir Ali Khan, P. Eng	Issue Date:	May 13, 1996
Approved By:	Haseen Khan	Director Re-Issue Date:	March 19, 2014
Approved By:	Martin Goebel	ADM	Review Date:
Authorized By:	Jamie Chippett	DM	Superseded:
	Joan Shea	Minister	Cancelled:

## Subject:

Flood Plain Management

## 1.0 INTRODUCTION

Land use within flood plains involves trade offs between flood risk and development. Flood risk takes the form of danger to health and safety, financial costs associated with property damage and degradation of water resources and the environment. Some factors associated with flood risk such as flow velocity, upstream inundation, erosion potential or environmental impacts may be severe. Consequently, new land development should therefore be restricted or prohibited. However, where conditions are not as severe, some types of development and land use may occur safely provided certain terms and conditions apply.

## 2.0 OBJECTIVES

- to prevent loss of human life and avoid personal hardships,
- to minimize flood damage to properties, infrastructure and the environment,
- to restrict activities which would degrade water resources,
- to maintain the natural capability of waterways to convey flood flows,
- to minimize disruption of transportation, social and business activity, and,
- to minimize costs to the taxpayers of Newfoundland and Labrador.

The unwise development of land in flood plains has historically taken place in many areas of the province probably due to a natural tendency for settlers to utilize land that is near bodies of water. Unfortunately, the potential for flooding is often recognized only after it is too late. The basic operating premise of this policy is that these problems will not materialize if development takes place in a manner that does not place it at any risk of flooding.

The policy will address Crown land, developed land and undeveloped land. Where lands that are subject to periodic flooding are still directly owned by the Crown, those lands will not be transferred to private developers or municipalities. However, where land is already alienated, it is necessary to determine the risk of flooding and to discourage potential development by planning, zoning regulations and by removing any economic advantages or subsidies that would otherwise encourage such development. Finally, where development has already taken place or cannot be avoided, policy is intended to minimize potential flood damage by ensuring that flood proofing measures are implemented and that the development does not further exacerbate the flooding problem by impeding flows or by unduly constricting the flow channel. The policy also takes climate change into consideration.

## 3.0 BACKGROUND

### Canada - Newfoundland Flood Damage Reduction Program



Under the Canada - Newfoundland Flood Damage Reduction Program, both governments agreed that public funds would not be used or provided for development projects in flood risk areas. To identify these areas, hydrotechnical studies were carried out for 37 communities in the province. Without exception, the main recommendation in each study was that the implementation of proper flood plain management policies would minimize flood risk.

## 4.0 LEGISLATION

*Water Resources Act*, SNL 2002 cW-4.01, ("the Act") sections 30, 32, 33, 34, 35, 48, 64 and 90, the *Lands Act* SNL1991 CHAPTER 36 Section 7.

## 5.0 DEFINITIONS

Body of Water	(Statutory definition from the Act) "body of water" means a surface or subterranean source of fresh or salt water within the jurisdiction of the province, whether that source usually contains liquid or frozen water or not, and includes water above the bed of the sea that is within the jurisdiction of the province, a river, stream, brook, creek, watercourse, lake, pond, spring, lagoon, ravine, gully, canal, wetland and other flowing or standing water and the land usually or at any time occupied by that body of water;
Flood Plain	An area adjacent to a lake, river, seashore etc. which is inundated or covered with water on average at least once in 100 years. Note that a flood plain is considered to be an integral part of a body of water as defined above because it includes "the land usually or at a time occupied by that body of water" and "whether that source usually contains water or not".
Designated Area	A specific flood plain in a community for which a hydrotechnical study has determined the extent of flooding and for which flood risk maps are available. The designation is in accordance with the Canada - Newfoundland Flood Damage Reduction Program Agreements.
Floodway	The portion of a flood plain where the most frequent flooding occurs and where the flow of water is fastest. This area is determined on the basis of the 1 in 20 year (1:20) return period flood.
Floodway Fringe	The portion of a flood plain where less frequent flooding occurs and where the flow of water is considered to be tranquil. This area is where flooding occurs up to 1 in 100 years (1:100) on average.
Climate Change Flood Zone	Based on extension of the floodway fringe, this is the area which is likely to be impacted due to the latest forecasted affects of climate change.
Other Flood Risk Area	An area where flooding is known or has some probability to occur due to unique or unusual circumstances such as areas subject to shoreline recession, areas downstream of dams or areas adjacent to watercourses potentially prone to ice jams.
Flood Control Area	An area that is subject to periodic flooding which has been designated (by the Department) a control area in order to reduce the risks to public health and safety and property damages. This area shall normally be treated as a floodway zone (1:20), unless otherwise determined by the Department.
Buffer Zone	A zone of land that is in its natural state and that is intended to separate developed areas from bodies of water to provide basic protection of water resources. This zone may coincide with a Crown land reservation of a shoreline as prescribed by Section 7(1) of the <i>Lands Act</i> . In the absence of specific setback requirements (depending on the activity) the buffer is taken to be 15 metres measured from the high water mark which in turn is understood to be the 1 in 100 year (1:100) high water mark or the Climate Change Flood Zone, where they have been identified.
Coastal Area	The interface or transition area where the land meets the sea/ocean or large inland lakes. The coastal area can be flooded due to storm surges, high tides or waves, erosion, rising sea level, or reclaimed land.

## 6.0 POLICIES

### 6.01 Development Requires Written Approval

Development in a designated flood risk area, development in a flood plain and development in a climate change flood zone shall be subject to the prior written approval of the Minister of Environment and Conservation (the “Minister”) in accordance with the *Act*.

## 6.02 Project Categories

In general it is the policy of the Department of Environment and Conservation (“the Department”) that flood plains and the buffer zone be preserved and left in their natural state. Recognizing that this is an ideal that would hinder significant benefits that could be derived from certain development in a flood plain and outweigh all risk of loss, damage or peril, this policy for flood plain management views any application to avail of land in flood risk areas in decreasing order of preference. These preferences are referred to hereafter as project categories.

1. **Temporary alterations** in a buffer zone, a climate change flood zone, a designated floodway fringe, a flood plain, a designated floodway, and lastly, the body of water itself.
2. **Non-structural uses** such as open space recreation, pasture, and wildlife habitat enhancement.
3. **Structures related to use of water resources** such as wharves, slipways, boathouses, pumping stations, storm or sewerage discharges.
4. **Minor structural or other projects** where only soil disturbance is involved such as constructed trails, pipelines, transmissions lines, roads, etc., assuming there will be no change in the grade of the land.
5. **Other structures not used primarily for residential**, commercial, industrial or institutional purposes where there will be a change in grade but not a building.
6. **Industrial uses related to the marine shipping** or fishing industries.
7. **Other industrial and commercial** development.
8. **Institutional** developments such as hospitals, senior citizens homes, homes for special care or schools where flooding could pose a significant threat should evacuation become necessary.
9. **Residential and other institutional** development.

## 6.03 Hydraulic Structures

A special class of structures which includes most hydraulic structures such as dams, bridges, causeways, dykes, canals etc., are by their own needs and characteristics constructed in buffer zones and flood plains and consequently, no preference can be assigned. However, such structures are the subject of the *Act* and every effort must be made to ensure that such structures do not adversely affect the capability of the body of water to convey flow. In the case of dams, new areas of flooding and the impact of that flooding must be fully assessed by the proponent.

## 6.04 Project Classifications

Table 1 below indicates whether not project categories are permitted in each of the defined flood plains.

Category	Where Flood Plains are Designated			
	All Flood Plains	Floodway (1:20 year Zone)	Floodway Fringe (1:100 year Zone)	Climate Change Flood Zone
<b>Temporary alterations</b>	Permitted	Permitted	Permitted	Permitted
<b>Non-structural uses</b>	Permitted	Permitted	Permitted	Permitted
<b>Structures related to use of water resources</b>	Permitted	Permitted	Permitted	Permitted

<b>Minor structural or other projects</b>	Permitted	Permitted with conditions*	Permitted with conditions*	Permitted with conditions*
<b>Other structures not used primarily for residential</b>	Permitted with conditions*	Permitted with conditions*	Permitted with conditions*	Permitted with conditions*
<b>Industrial Uses related to shipping (marine only)</b>	Permitted with conditions*	Permitted with conditions*	Permitted with conditions*	Permitted with conditions*
<b>Other industrial and commercial</b>	Not Permitted	Permitted with conditions**	Permitted with conditions*	Permitted with conditions*
<b>Institutional</b>	Not Permitted	Not Permitted	Not Permitted	Not Permitted
<b>Residential and other institutional</b>	Not Permitted	Not Permitted	Permitted with conditions*	Permitted with conditions*
<b>Hydraulic Structures</b>	Permitted	Permitted	Permitted	Permitted

\* - See Section 6.05 for special terms and conditions related to necessary flood proofing measures.

\*\* - See Section 6.06 for special terms and conditions related to necessary flood proofing measures.

Note: All permits contain standard terms and conditions.

### 6.05 Projects Permitted Where Flood Plains Are Designated

In Table 1 where projects may be permitted with conditions, the following conditions will apply:

- i. the ground floor elevation of the structure is higher than the 1:100 year flood level and the climate change flood zone (where designated), and,
- ii. the structure will not interfere with the flow of water or displace water such that it creates a worse flooding situation for other properties, and,
- iii. the structure and the associated utilities must be designed and constructed in accordance with the approved flood proofing guidelines of the Department and entrances and exits from the building can be safely used without hindrance in the event of a flood, and,
- iv. the proposed use of the facility and site will not involve any storage of pollutants such as fuels, chemicals, pesticides etc., and,
- v. additional conditions which may be appropriate for specific projects and included in a permit issued under Section 48 of the Act.

### 6.06 Projects Permitted in Coastal Floodway Where Flood Plains Are Designated

In order to accommodate tourism activities in coastal areas, such as eateries, attractions, tourist information booths, tour headquarters etc., in Table 1 if the floodway (1:20 year zone) flooding in a coastal community is primarily due to backwater

effects of the ocean and extreme high tides and consequently the flow velocities in the floodway are low, the following conditions will apply:

- i. only a tourism related structure and the associated utilities are permitted. The tourism related structures and the associated utilities does not include accommodations such as motels or hotels, and,
- ii. the tourism related structure and the associated utilities would not be eligible for flood disaster compensation, and,
- iii. the ground floor elevation of the structure is higher than the 1:100 year flood level and the climate change flood zone (where designated), and,
- iv. the structure will not interfere with the flow of water or displace water such that it creates a worse flooding situation for other properties, and,
- v. the structure and the associated utilities must be designed and constructed in accordance with the approved flood proofing guidelines of the Department and entrances and exits from the building can be safely used without hindrance in the event of a flood, and,
- vi. the proposed use of the facility and site will not involve any storage of pollutants such as fuels, chemicals, pesticides etc., and,
- vii. additional conditions which may be appropriate for specific projects and included in a permit issued under Section 48 of the *Act*.

### **6.07 Additions and Modifications to Existing Development**

Additions, modifications, enhancements and improvements to existing structures where there is an increase in the floor area within the flood plain, will be assessed for suitability in the same way as the project category as a whole.

### **6.08 Use of Flood Risk Mapping in Municipal Plans**

Where flood risk mapping has been prepared for a community (or any city, town or area) the information in the flood risk maps must be incorporated in the Municipal Plan (if one exists) and the flood risk areas must be zoned so as to permit only those project categories specified by this policy. In the absence of official flood risk mapping, communities will be encouraged to determine flood risk areas in accordance with this Department's standard hydrotechnical methods for delineating flood risk zones and to zone those lands in accordance with this policy. Failing this, communities will be encouraged to at least make provisions in planning documents for minimum setbacks from watercourses to provide some margin of safety and to recognize potential flood susceptibility.

### **6.09 Eligibility for Flood Disaster Assistance**

Any vulnerable development placed in a flood plain or designated flood risk area after the designation and not in conformance with this policy or without approval as required by this policy, would not be eligible for flood disaster compensation if such a program of compensation were to become available through government. This policy provision does not apply to any development lawfully established in a flood plain prior to designation.

### **6.10 Use of Flood Disaster Compensation**

In the event that compensation by government is awarded to flood victims, it will be the policy of this Department to encourage victims to apply the compensation towards relocating rather than replacing or repairing damaged property in situ. If it is deemed acceptable by this Department to repair or replace damaged property in flood risk areas, then it will be required that the compensation be used firstly for appropriate flood proofing measures.

### **6.11 Flood Insurance**

Persons living or carrying out business in flood risk areas may not be able to purchase flood insurance and if available it may be very expensive. It is therefore recommended that those who are located in flood risk areas carry out flood proofing measures and have an emergency plan available.

### **6.12 Flood Control Projects**

Proposals for flood control measures such as construction of dykes, river diversions, retaining walls or flood control dams will only be considered where the alternative with the highest benefit/cost ratio is recommended. Alternatives considered may also include possible compensation for flood victims or the cost of relocating the inhabitants of the flood risk areas or maintaining the status quo.

After flood controls have been implemented, flood risk designations shall remain in effect until such time as new hydrotechnical studies have been undertaken and new flood risk areas delineated (in accordance with the Departments standards).

### **6.13 Role of Water Resources Management Division**

The Water Resources Management Division of this Department will continue as the lead agency with respect to flood plain management. This role will include but is not limited to:

- i. Evaluating all applications for approval under section 48 of the *Act* and making the appropriate recommendations in accordance with this policy.
- ii. Carrying out hydrotechnical studies, flood risk analyses and mapping to the extent possible with limited funds provided.
- iii. Continuing to monitor areas of flood risk such as Badger, Deer Lake and Steady Brook to provide flood warning and flood status reports.
- iv. Providing to the public information, data, maps, guidelines for flood proofing and other materials that will be useful in reducing flood damage.
- v. Providing technical expertise and assisting Fire and Emergency Services – Newfoundland and Labrador in the event of a flood emergency.
- vi. Continue with the ability to forecast flooding using computer models and real time data.

### **6.14 Offences**

A municipal authority or person that unlawfully alters a body of water by carrying out any development in a designated flood risk area or a flood plain without written approval from the Minister, thereby violates section 48 of the *Act* and commits an offence contrary to section 90 of the *Act*.

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## NEWFOUNDLAND AND LABRADOR PROVINCIAL LAND USE POLICY FLOOD RISK AREAS

Floods can cause substantial damage to property and threaten human life. After a flood, Government is often faced with providing emergency assistance, clean-up, remediation and, in some cases, compensation to affected residents and businesses. Government has also financially assisted towns in constructing engineered flood defences to protect areas exposed to flooding. The Province of Newfoundland and Labrador, through this provincial land use policy, is working to reduce damage to property caused by flooding, to protect public safety, and to reduce the requirements for flood defences and flood damage remediation.

Newfoundland and Labrador, in conjunction with the federal government, has formally identified and mapped areas across the province that are subject to flooding. These flood risk areas affect portions of 32 municipalities; 4 local service districts; 6 unincorporated communities; and an uninhabited area on the Trans Canada Highway. Other places across the province have also experienced localized flooding but the affected areas have not been formally mapped.

This provincial land use policy directs new buildings and land uses to areas that are not at a high risk of flooding. In lower risk areas, development needs to be of a design and with an appropriate level of protection to ensure that the risk of damage from flooding is minimized. Any development within a flood risk area should not impede water flows or exacerbate flood risk elsewhere.

<b>GOAL:</b> To protect public safety and property from the risk of flooding and to reduce the requirement for flood defences and flood damage remediation.
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1. The Province of Newfoundland and Labrador discourages the construction of new buildings and structures in areas at risk of flooding. All development in Newfoundland and Labrador must conform to the provisions of this Provincial Land Use Policy. It is the prime responsibility of the property owner to avoid development in areas of flood risk and, in instances where development already exists or is permitted, to undertake appropriate flood proofing. Municipal Councils and all other agencies with development control authority are responsible for strictly controlling development in line with this Provincial Land Use Policy and for ensuring that all owners and users of property are aware of the risks of developing in an area that is subject to flooding.
2. In this Provincial Land Use Policy,
  - (a) flood proofing means structural and/or non-structural measures incorporated in the design of a building or structure which reduce or eliminate the risk of flood damage by ensuring that the ground floor elevation is higher than the projected flood level and that the building can be exited without hindrance in the event of a flood.
  - (b) floodway means the inner portion of a flood risk area where the risk of flood is

- greatest, on average once in twenty years, and where the flood depths and water velocities are greatest.
- (c) floodway fringe means the outer portion of a flood risk area, between the floodway and the outer boundary of the flood risk area, where the risk of flooding is lower, on average once in one hundred years, and flood waters are shallower and slower.
  - (d) plan means a municipal plan, regional plan, protected road zoning plan, local area plan or protected area plan prepared under the Urban and Rural Planning Act.
  - (e) regulations means regulations made under the Urban and Rural Planning Act.
3. The flood risk areas mapped under the Canada-Newfoundland Flood Damage Reduction Program are listed in Schedule A.
- (a) Within a floodway,
    - (i) new development is restricted to non-building uses such as roads and associated structures, agriculture, open space and recreation, service corridors and to minor structures related to utilities and marine activities;
    - (ii) the placement of imported fill is prohibited unless it is specifically required as a flood proofing measure or for public infrastructure provided that appropriate studies are carried out and show that these structures will not be damaged by flooding, impede water flows or contribute to an increase in flood risk.
  - (b) Within a floodway fringe:
    - (i) new buildings and structures, provided they are floodproofed, may be permitted, except for
      - (1) residential institutions such as hospitals, senior citizen homes, homes for special care and any other use where flooding could pose a significant threat to the safety of residents if evacuation becomes necessary,
      - (2) police stations, fire stations and other facilities that may provide emergency services during a flood including government offices,
      - (3) schools, and
      - (4) uses associated with the storage, warehousing or the production of hazardous materials including gas stations.
    - (ii) the placement of imported fill shall be limited to that required for floodproofing, flood risk management or for public infrastructure provided that appropriate studies are carried out and show that these structures will not be damaged by flooding, impede water flows or contribute to an increase in flood risk.
4. Crown Land shall not be released for any development involving building in either the flood way or floodway fringe.
5. Within the floodway, existing buildings or structures damaged beyond economic repair shall not be replaced unless:
- (a) the replacement building or structure has no greater floor area than the original building or structure at or below the defined flood proofing elevation;

- (b) the replacement building or structure is flood proofed; and
  - (c) the replacement building or structure does not impede water flows or contribute to an increase in flood risk.
6. Within the floodway, existing uses, such as residential, commercial, industrial and institutional buildings, are encouraged to undertake flood proofing measures to reduce the risk of flood damage or relocate.
  7. Any expansion of existing buildings and structures must be balanced against the risks to human safety and property and the possibility of exacerbating upstream and downstream flooding. Any expansion of existing buildings and structures in the floodway must not increase the area of the structure at or below the defined flood proof elevation.
  8. Plans and regulations must identify flood risk areas mapped under the Canada-Newfoundland Flood Damage Reduction Program, as listed in Schedule A, and contain policy statements and regulations consistent with this Provincial Land Use Policy.
  9. There are areas in Newfoundland and Labrador that are known to be subject to localized flooding but have not been mapped under the Canada-Newfoundland Flood Damage Reduction Program. Development vulnerable to flood damage, such as houses, businesses and institutions, is discouraged in these areas. Plans and regulations must identify any locally known flood risk areas and contain policy statements and regulations consistent with this Provincial Land Use Policy.
  10. For those areas without a plan and/or regulations in which flood risk areas have been mapped under the Canada-Newfoundland Flood Damage Reduction Program, all applications for development within the floodway and floodway fringe shall be referred to the Minister of Municipal and Provincial Affairs in accordance with section 47 of the Urban and Rural Planning Act, 2000. The Minister shall determine the outcome of the application after evaluating it against this Provincial Land Use Policy and the risks to public safety and property. Any such development must not contribute to upstream or downstream flooding or result in a change to flood water flow patterns.
  11. Development in the floodway, floodway fringe and in a 15 metre buffer around these zones as well as any area known to be subject to flooding must obtain prior written approval of the Minister of Environment and Conservation in accordance with section 48 of the Water Resources Act.



**APPENDIX E**  
**WAVE ANALYSIS**

**DRAFT**

**Wind and Wave Setup and Runup during a 100-year flood event near the  
Health Sciences Centre in St. John's, Newfoundland**

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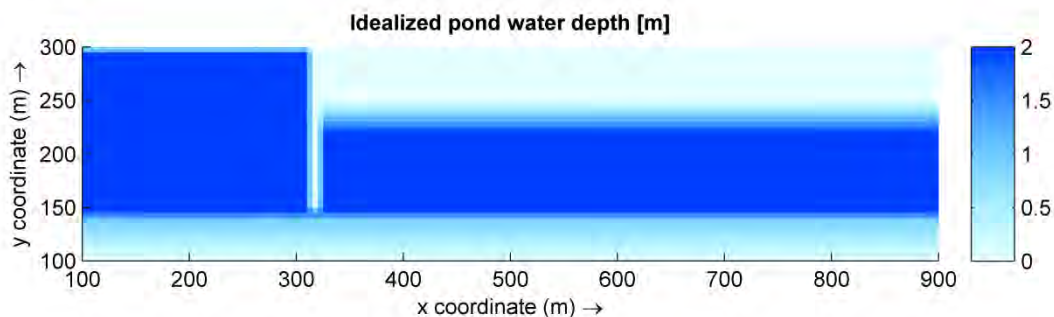
**July 2014**

## 1.0 WAVE SETUP AND RUNUP ANALYSIS

The aim of the following analysis is to estimate the wave setup and runup associated with the worst-case scenario during a 100-year flood event in the vicinity of the Health Sciences Centre (HSC) in St. John’s, Newfoundland. The floodplain extent used for wave generation and propagation is based on a separate analysis of the 100-year flood event that characterized the resulting flood plain, depicted in AMEC figures A1 and A2.

The present analysis is based on a fully-developed wave condition, generated using the SWAN (Simulating Waves in the Nearshore) model (Holthuijsen et al., 2000, The SWAN team, 2014). SWAN utilizes a finite difference scheme to compute random, short-crested, wind-generated waves and allows for spectral wave input at specified boundaries. The action density spectrum (equal to the energy spectrum divided by the relative frequency) is used since it is a quantity that is conserved in the presence of currents. In the current implementation, SWAN incorporates physical processes such as wave propagation, white-capping, shoaling, wave breaking, bottom friction, wave set-up and wave-wave interactions in its computations. SWAN computes the wave field and other wave parameters over a specified range of geographical space, time, wave frequencies and directions. The model inputs include the gridded bathymetry, stillwater level, and the prescribed winds associated with a 100-year storm event. The water depth throughout the pond was derived (Figure 1) based on an interpretation of the elevation profiles (A, B and C) in figures A1 and A2. The maximum water depth is assumed to be 2 m, and the idealized bathymetry was constructed with the assumption that this depth would occupy a conservatively large portion of the pond.

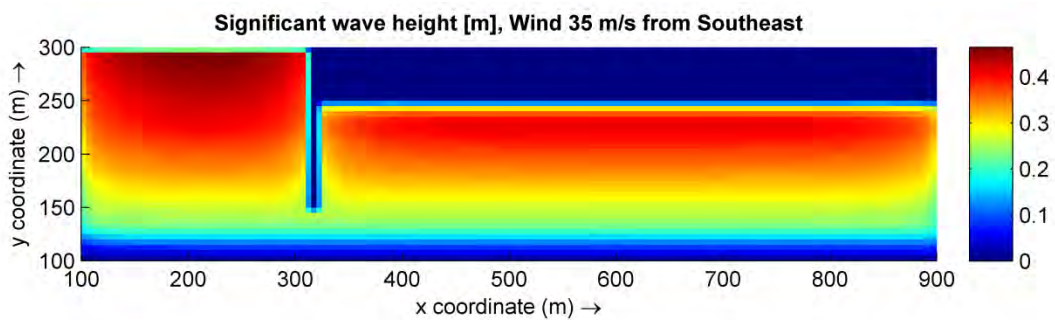
Over 60 years, the highest sustained wind speed recorded at St. John’s airport was 38 m/s in February 1959. However these winds were from the Northwest, away from the Health Sciences Centre. The strongest winds recorded toward the HSC were southwesterly winds of 33 m/s in January 1977. Considering the observation record of St. John’s airport covers 60 years, a representative 100 year wind toward the HSC was estimated at 35 m/s.



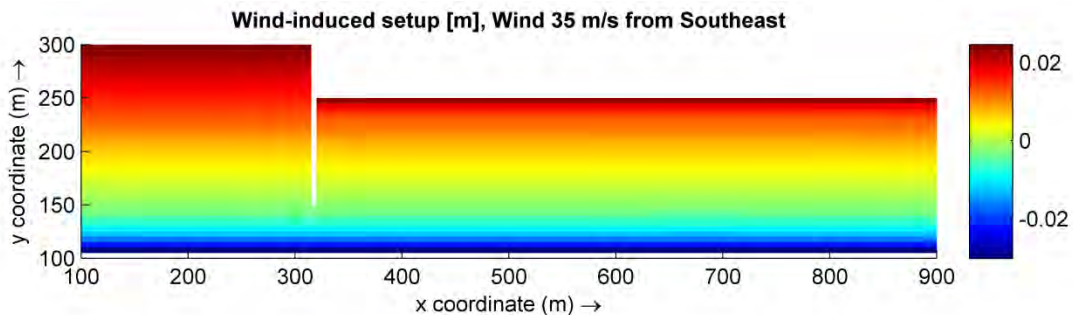
**Figure 1 Idealized bathymetry for the pond formed during a 100-year flood event in the vicinity of the Health Sciences Centre in St. John’s, NL, based on AMEC Drawings 174811-601 and 174811-602.**

The same model domain and wind condition was additionally prescribed in the Delft3D-FLOW model (Deltares, Delft University of Technology), and the model was allowed to reach a steady adjustment to the wind field in order to estimate the wind-induced setup of the water surface (Figure 3).

The maximum southeasterly winds, oriented directly toward the HSC, produce a wave condition characterized by a maximum significant wave height of 0.4 m (Figure 2), peak wave period of 1.5 s and mean wavelength of 2 m. The computed wave-induced setup was insignificant ( $<0.01$  m), while the wind-induced setup was 0.03 m.



**Figure 2 Significant wave height distribution in the pond formed during a 100-year flood event, based on assumed winds of 35 m/s from the southeast direction.**



**Figure 3 Wind setup in the pond formed during a 100-year flood event, based on assumed winds of 35 m/s from the southeast direction.**

For the purpose of estimating the highest wave runup levels on the northwest side of the pond, it is necessary to consider the wave runup in addition to the wind-induced setup. The wave runup was estimated from an empirical relation describing the maximum runup for irregular waves on an impermeable beach (USACE EM, 2008):

$$\frac{R_{max}}{H_0} = 2.32 * \xi^{0.77}$$

$$\frac{R_{2\%}}{H_0} = 1.86 * \xi^{0.71}$$

where:

$R_{max}$  = maximum wave runup (m)

$R_{2\%}$  = wave runup (m) exceeded by only 2% of waves

$H_0$  = Significant wave height (m)

$\xi_0$  = Surf similarity parameter (dimensionless)

The surf similarity parameter is calculated using the following equation (USACE EM, 2008):

$$\xi_0 = \tan\beta \left( \frac{H_0}{L} \right)^{-1/2}$$

where:

$\xi_0$  = Surf similarity parameter (dimensionless)

$H_0$  = Significant wave height (m)

$L$  = Wavelength (m)

$\tan\beta$  = Beach slope (Rise (m)/run (m))

The beach slope estimated at approximately 0.1 based on Section B in drawing 174811-602.

The results from the wave and flow model computations are summarized in Table 1.1. Based on the assumption that maximum wind setup and wave runup occur at the same time, the total inundation levels due to combined wind setup and wave runup are estimated to reach a maximum of 0.32 m above the 100-year flood still water level, although a relatively small percentage of the waves is expected to create this level of runup. Most of the individual waves would create inundation levels of less than 0.28 m above the 100-year flood still water level.

**Table 1.1 Model results for a worst-case scenario during a 100-year storm event**

<b>Parameter</b>	<b>Value</b>
Significant wave height	0.4 m
Peak Wave Period	1.5 s
Mean wave length	2 m
Wave-induced setup	<0.01 m
Wave runup (top 2% of waves)	0.25 m
Wave runup (maximum)	0.29 m
Wind-induced setup	0.03 m
Total wind and wave-induced runup (top 2% of waves)	0.28 m
Total wind and wave-induced runup (maximum)	0.32 m

## 2.0 REFERENCES

The SWAN team, *SWAN user manual*, TU Delft, The Netherlands, 2014.

Holthuijsen, L.H., Booij, N., Ris, R.C., Haagsma, IJ.G., Kieftenburg, A.T.M.M., Kriezi, E.E., 2000. *SWAN user manual*, TU Delft, The Netherlands.

United States Army Corps of Engineers, 2008. Engineering Publication EM 1110-2-1100

**APPENDIX F**  
**COST ESTIMATE**



Description	Quantity	Unit price	Amount
Mobilization and Demobilization (L.S.)	-	\$20,000.00	\$20,000.00
Erosion and Sediment Control (L.S.)	-	\$5,000.00	\$5,000.00
Clearing and Grubbing (/m2)	3,458	\$5.00	\$17,290.00
Topsoil Stripping and Stockpiling (/m2)	3,458	\$2.00	\$6,916.00
Supply Topsoil (/m2)	354	\$2.00	\$708.00
Common Excavation and Removal (/m3)	900	\$25.00	\$22,500.00
Select/Structural Fill (/m3)	4510	\$25.00	\$112,750.00
Pedestrian Gates (ea.)	4	\$1,500.00	\$6,000.00
Culvert (ea.)	6	\$2,000.00	\$12,000.00
Asphalt (50mm asp. Plus 300mm gravel base) (/m2)	860	\$80.00	\$68,800.00
Spreading Topsoil (/m2)	3,812	\$2.00	\$7,624.00
Hydroseeding (/m2)	3,812	\$1.00	\$3,812.00
	<b>Total</b>		<b>\$283,400.00</b>