

Town of Logy Bay – Middle Cove – Outer Cove Flood Risk Mapping Study Final Report



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Prepared for:
**Town of
Logy Bay – Middle
Cove – Outer Cove**

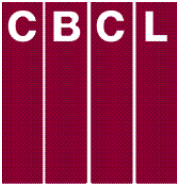
Prepared by:



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Consulting Engineers





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August 3, 2012

Adele Carruthers
Town Manager
Town of Logy Bay – Middle Cove – Outer Cove
744 Logy Bay Road
Logy Bay, NL A1K 3B5

Dear Ms. Carruthers:

RE: Flood Risk Mapping Study

We are pleased to submit ten copies of our final report for the above noted study. We have provided three copies to the Town of Logy Bay – Middle Cove – Outer Cove, and have forwarded seven copies to the Water Resources Management Division, Department of Environment and Conservation.

This report details all of the work carried out during the course of the study, the methods of analysis, results and recommendations. The flood risk mapping, which is the key project deliverable, is contained in Appendix P.

We would like to thank the Town and the Department for their cooperation throughout the execution of this study. We enjoyed working on this important project, and trust that the report findings will assist the Town in development planning and the Department in managing surface water resources.

Yours truly,

CBCL LIMITED
Consulting Engineers

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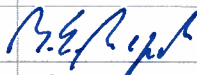

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today's
problems
with
tomorrow
in mind

Final Report	G. Sheppard		3-Aug-12	J. Bursey		3-Aug-12
Issue or Revision:	Name:	Signature:	Date:	Name:	Signature:	Date:
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Signed and Sealed:



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EXECUTIVE SUMMARY

In April 2011, the Department of Environment and Conservation, Water Resources Management Division solicited proposals from qualified consultants to undertake a flood risk mapping study for the Town of Logy Bay-Middle Cove-Outer Cove (LB-MC-OC). CBCL Limited was subsequently retained by the town to conduct a study of the four main watercourses in the town.

The main objectives of this study are to assess the hydraulic structures within the Town and to produce flood risk maps of the four study basins: Kennedys Brook, Outer Cove Brook, Coakers River and Drukens River. The results of the flood analysis will be used as a tool to assist in managing surface water resources.

A review of historical flooding was conducted by thoroughly reviewing background information regarding notable past flood events and through discussions with the Town Manager.

An extensive field program was carried out to collect cross section data, calibration data, and structure data for the four rivers. LiDAR data was also collected to aid in the development of the hydraulic models and flood risk and inundation maps.

To ensure the most up-to-date data was used in estimating the 1:20 and 1:100 annual exceedance probability (AEP) flood flows, the Intensity Duration Frequency (IDF) curves for the St. John's Airport station were updated. This updating involved obtaining current rainfall data from the Windsor Lake rain gauge, which is owned and operated by the City of St. John's. Annual maximums for 5, 10, 15 and 30-minute and 1, 2, 6, 12 and 24-hour intervals were extracted from the Windsor Lake gauge data and combined with the St. John's Airport gauge data. Statistical analysis was performed on each of the 8 data sets to update the IDF curves. A comparison of the updated curves to the previous (1996) curves indicated slight decreases in rainfall intensities for shorter duration storms and significant increases in rainfall intensities for longer duration storms. CBCL recommends that LB-MC-OC encourage Environment Canada to reinstate the measurement of rainfall intensity at the St. John's Airport rain gauge. Rainfall intensity data collected at this location is essential for carrying out flood risk mapping work in LB-MC-OC and surrounding areas.

The hydrologic analysis was conducted using both statistical and deterministic approaches. The statistical analysis included single station frequency analysis on nearby gauged basins, prorated to the study basins and a regional flood frequency analysis. Flood estimates determined using both of these statistical techniques should be used with caution; single station frequency analysis results are

influenced by the physical factors in the gauged basin that the study basin may not experience and many of the parameters calculated for the regional flood frequency analysis were outside the acceptable range or near the extreme. The deterministic analysis was performed using the modelling software HEC-HMS. 1:20 and 1:100 AEP flood estimates were determined for each river for the existing and ultimate development conditions.

The hydraulic analysis involved modeling and calibrating the four rivers using data collected during the field program in the modelling software HEC-RAS. The flood flows determined from the hydrologic analysis for the 1:20 and 1:100 AEP floods were simulated in the calibrated HEC-RAS models to produce water surface profiles, which were used in the creation of the flood risk and inundation maps. It is recommended that LB-MC-OC measure high water levels at designated structures on each river during future high flow events. This information could be used in further validation of the present models and in future studies.

Both the hydrologic and hydraulic models were tested for sensitivity to the model parameters. The hydrologic model was tested for sensitivity to Manning's n , hyetographs and curve number. The 1:100 AEP flow for existing conditions was used as a benchmark to test sensitivity. Results of the analysis indicated that the hydrologic model is most sensitive to curve number variation, while alterations to Manning's n values had the least effect on flow results. Parameters selected for sensitivity analysis of the hydraulic models included Manning's n , expansion and contraction coefficients and peak flow. The 1:100 AEP water surface profile for the existing condition scenario was used as a benchmark to test sensitivity of these parameters. Alterations to Manning's n and peak flow had the largest impact on resulting water levels; while changing the expansion and contraction coefficients had a near negligible effect on water levels.

An assessment of flow capacities and estimated remaining service life of various hydraulic structures within the study area was completed. Flow scenarios examined included 1:20 and 1:100 AEP flows for existing conditions, ultimate development, planned future development in LB-MC-OC with no future development in St. John's and Torbay, and planned future development in St. John's and Torbay with no future development in LB-MC-OC. The results of the analysis indicate which structures are currently undersized and which will become deficient as a result of planned development. Each structure was also visually inspected and an estimated timeframe given for when each should be considered for replacement. CBCL recommends that LB-MC-OC re-evaluate the condition of existing hydraulic structures in five to ten years. It is also recommended that LB-MC-OC develop an infrastructure renewal plan, in consultation with the Department of Municipal Affairs, to replace the under-sized culverts and bridges identified in this study. The 1:100 AEP flood flows under the ultimate development scenario should be used as the design flows for the design of new structures.

The impacts of changing climate conditions on flood flows and floodplains were also assessed. Climate change predictions developed by the Meteorological Services of Canada and by Dr. Joel Finnis, Professor, Department of Geography, Memorial University of Newfoundland were assessed. The predictions were simulated in the HEC-HMS models to estimate 1:20 and 1:100 AEP flood flows for each of the four rivers. The estimates prepared by Dr. Finnis were used to produce flood risk maps for the climate change scenario since these estimates were based on the most up-to-date IDF values.

Flood risk maps were generated for the 1:20 and 1:100 AEP floods for the existing development, ultimate development and climate change conditions and overlain on community mapping and

satellite imagery. Water levels generated by the HEC-RAS models for the 1:20 and 1:100 AEP floods for the existing, ultimate development and climate change conditions were used to develop flood risk maps for the four rivers. These maps were field verified through discussions with local residents and the Town Manager. It is recommended that LB-MC-OC adopt the ultimate development flood lines for its town plan and development regulations.

Similarly, inundation maps illustrating the depth of water within the floodplain extents were developed for the 1:20 and 1:100 AEP floods for existing development, ultimate development and climate change scenarios. These maps were overlain on community mapping. It is recommended the inundation maps be used with caution, since they are developed using bathymetric data and the only reliable areas of such data are at the locations of each surveyed cross section.

During the land classification exercise, required for the hydrologic modelling, environmentally sensitive areas such as wetlands and water bodies were identified. These areas were mapped and labeled to help the town identify locations they may want to consider preserving.

CHAPTER 1 INTRODUCTION

1.1 Background

The Town of Logy Bay-Middle Cove-Outer Cove (LB-MC-OC) is located on the north eastern portion of the Avalon Peninsula, as illustrated in Figure 1.1. LB-MC-OC is bordered by the City of St. John's to the southwest and the Town of Torbay to the northwest.

LB-MC-OC is rapidly growing, and is currently dealing with requests to develop land within its boundaries. Similarly, development in the adjacent municipalities of Torbay and St. John's is also occurring. Development results in changes in land use from natural open space and treed areas to impervious areas such as parking lots and buildings. Since precipitation cannot infiltrate impervious areas, the amount of rainfall converted directly to runoff is greater than that over the natural land types. Larger runoff leads to higher river flows.

In light of this continued development, LB-MC-OC is concerned about the ability of the existing hydraulic structures located throughout the town to accommodate flood flows. The four main watercourses and associated watersheds in LB-MC-OC are shown on Figure 1.2. As shown, the headwaters of the four streams originate within the limits of the City of St. John's and portions of Kennedys Brook drainage area lie within Torbay. It is expected that development activities in these areas will affect the river flows.

In April 2011, the Department of Environment and Conservation, Water Resources Management Division (WRMD) solicited proposals from qualified consultants to undertake a flood risk mapping study for LB-MC-OC. CBCL Limited (CBCL) was subsequently retained by LB-MC-OC to conduct a study of the four main watercourses in the town.

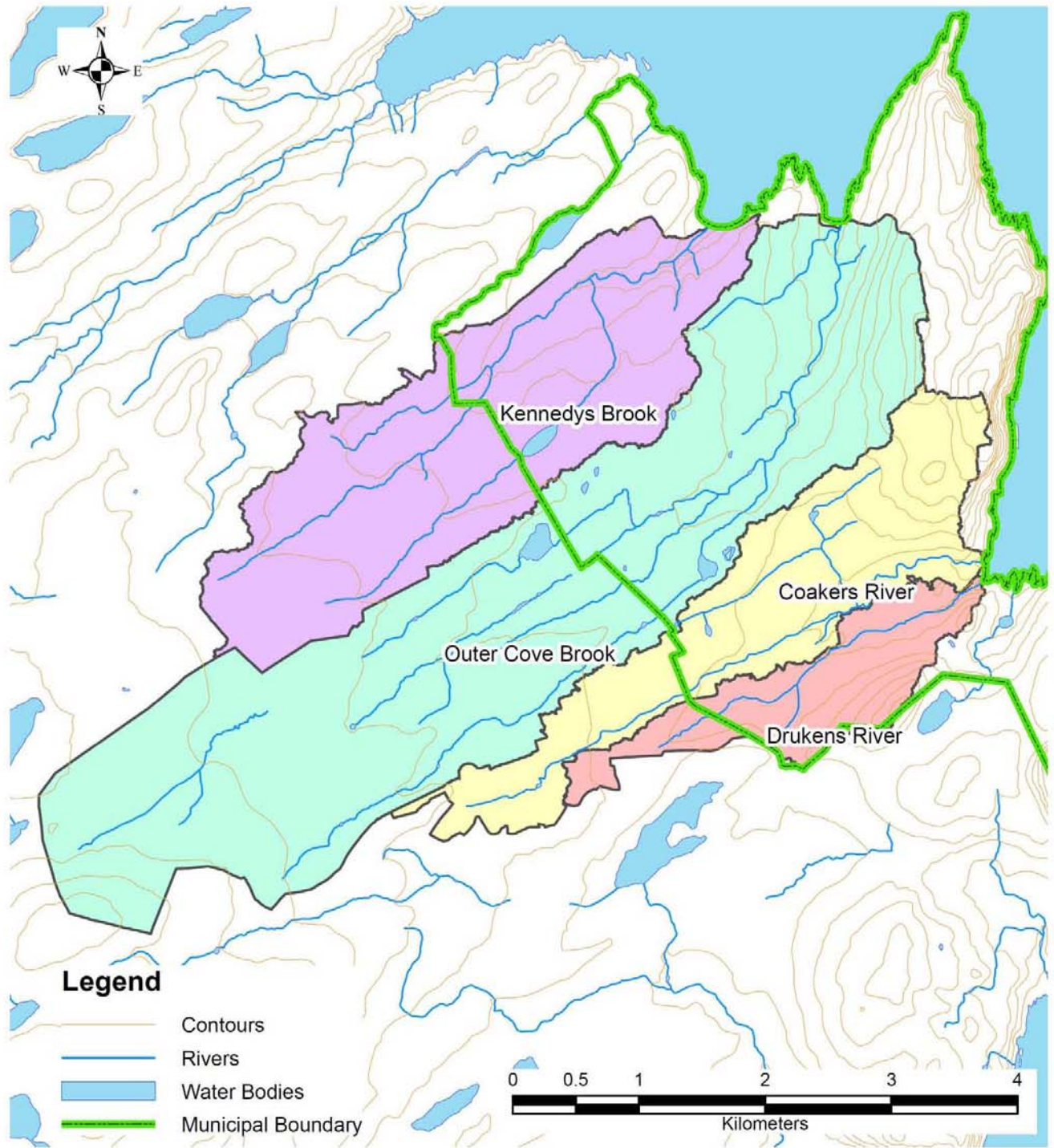
1.2 Objectives

The main objectives of this study are to assess the hydraulic structures within LB-MC-OC and to produce flood risk maps of the four study basins: Kennedys Brook, Outer Cove Brook, Coakers River and Drukens River. The results of the flood analysis will be used as a tool to assist LB-MC-OC in planning developments within town limits and WRMD in managing surface water resources.

1.3 Scope of the Study

The scope of work includes the following tasks:

- Obtain and review all available background material concerning the project, including previous reports, air photos, flow information, weather data, etc.;
- Survey cross-sections along Kennedys Brook, Outer Cove Brook, Coakers River and Drukens River;
- Update the intensity-duration-frequency (IDF) curves used in the study;
- Perform statistical analysis to estimate 1:20 and 1:100 annual exceedence probability (AEP) flood flows;
- Assemble hydrologic models of the study areas using HEC-geoHMS;
- Calibrate the HEC-HMS model using measured flow data;
- Prepare HEC-HMS model to reflect future changes to the watershed as a result of development within the LB-MC-OC, St. John's and Torbay;
- Complete sensitivity analysis on the hydrologic models;
- Assemble hydraulic models of the study areas using HEC-geoRAS;
- Calibrate the HEC-RAS models based on the measured flow and water level data obtained during survey;
- Complete sensitivity analysis on the hydraulic models;
- Perform peak water level calculations with the hydraulic models for the 1:20 and 1:100 AEP floods and climate change scenarios for the study areas as they currently exists as well as for planned future developments;
- Assess the flow capacities and current conditions of various hydraulic structures within the Town and estimate the remaining service life of each;
- Prepare flood risk maps to identify risks of flooding for the 1:20 and 1:100 AEP floods and climate change scenarios; and
- Identify environmentally sensitive areas.



Town of Logy Bay-Middle Cove-Outer Cove
 Flood Risk Mapping

Date: June 2012
 Proj. #: 113076.00

Basin Areas

Figure 1.2

CHAPTER 2 BACKGROUND INFORMATION

Sources of background information include discussions with the Town Manager and WRMD as well as literature review of flood studies previously completed.

2.1 Historical Flooding

Through discussions with the Town Manager, Ms. Adele Carruthers, it was learned that past flooding in the Town has been the result of major rainfalls. Past flood events that Ms. Carruthers noted include the following:

- Hurricane Gabrielle in September, 2001;
- November 29, 2008 rain storm which resulted in a State of Emergency being called when Outer Cove Brook Bridge at Logy Bay Road (also referred to as Savage Creek Bridge) overtopped;
- Hurricane Igor in September, 2010; and
- February 13, 2012 rain storm which resulted in many culverts and ditches surpassing their capacities and overtopping roads.

2.2 Previous Studies

A literature review of previous flood studies was conducted to assess the underlying cause of flooding in the Town, as well as to identify any areas which experience frequent flooding.

2.2.1 Flood Risk Mapping Study of Portugal Cove, St. Philips, and Outer Cove

In 1996, Coretech Inc., in association with Davis Engineering and Associates Ltd., completed a flood risk mapping study of Portugal Cove, St. Philips and Outer Cove under the Canada Newfoundland Water Resources Management Agreement. The areas studied in LB-MC-OC were on an unnamed river that is a tributary to Outer Cove Brook, near Collision Clinic (203 Lower Road), and on a tributary to Coakers River, crossing Caddigan's Road.

The report revealed that, during a significant rainfall event in 1986, a blocked culvert at Collision Clinic caused a portion of Lower Road to be washed away. Following the wash out, a new 600 mm culvert was installed to replace the previous structure.

The culvert on Caddigan's Road reportedly caused flooding to an adjacent property seven times in a 15 year period. The cause of the flooding was determined to be due to an undersized (900mm) culvert crossing Caddigan's road. In 1993, a hydrologic review revealed that a 2130 x 1400 mm pipe arch was needed to pass the expected flood and the 900mm culvert was subsequently replaced.

The Caddigan's Road culvert is included in the hydraulic assessment section of the current study.

2.2.2 Torbay Road North Commercial Area

In 2007, Kendall Engineering Ltd. conducted a study of the proposed Torbay Road North Commercial Area for the City of St. John's. The objective of the study was to examine the effects of rezoning two large parcels of land on Torbay Road to Commercial area on flow in Outer Cove Brook.

The study was carried out using the City of St. John's Subdivision Design Manual standards. The study revealed that several existing structures on Outer Cove Brook are not able to accommodate the 1:100 AEP flows for the existing conditions.

2.2.3 Vincent's Road Culvert Analysis

In 2009, LB-MC-OC contracted Hatch Mott MacDonald to carry out a study of a proposed culvert on Coakers River for the Vincent's Road subdivision. The purpose was to estimate the 1:100 AEP design flow, recommend a structure size and delineate flood lines for the pre and post development 1:100 AEP design flow.

The study was carried out using the design standards described in the City of St. John's Subdivision Design Manual. This analysis revealed that the proposed structure did not have adequate capacity to pass the post-development 1:100 AEP flood, and several acceptable structures were recommended.

CHAPTER 3 DATA COLLECTION

Several sources of data were required to accurately assemble the hydrologic and hydraulic models discussed in Sections 5 and 6. Items included in the data collection process are as follows:

- Channel cross sections
- Flow and corresponding water level measurements for calibration purposes
- Detailed terrain data
- Hydraulic structure details

3.1 Channel Cross Sections

3.1.1 Kennedys Brook

On October 7, 2011, two CBCL employees walked Kennedys Brook and identified cross section locations. During the site visit, they also made notes regarding the channel and overbank materials and photographed each cross section. Photos of the cross sections are included in Appendix A.

There were a total of 46 cross sections surveyed along Kennedys Brook at the locations illustrated on Figure 3.1. There are three hydraulic structures along Kennedys Brook. Appendix B contains structure data sheets, which provide photos and descriptions of the physical condition of the structures. The most upstream structure is located below residence 167 Middle Cove Road, on a gravel road constructed to access farm land. The structure is an 1880 x 1260 mm pipe arch culvert that is significantly damaged and has insufficient cover, as noted in the structure data sheet. The condition of the culvert significantly reduces the flow area, and therefore, the amount of flow the culvert can accommodate. The next structure is a one-lane concrete bridge, located on Pine River Road. The bridge is 3.1 m wide and rated for a maximum load of 10 tonnes. It is in good condition with only minor surface cracks of the concrete. The most downstream structure is located on Marine Drive near Middle Cove Beach. It is a 3100 x 1980 mm pipe arch culvert with gabion headwalls. The structure is in fair condition, with minor amount of rust on the bottom. The headwalls require some repair as some of the rocks have fallen into the river.

3.1.2 Outer Cove Brook

On October 13, 2011, two CBCL employees visited outer Cove Brook and marked cross section locations. They also collected photos (Appendix A) and made field notes regarding characteristics of the channel and overbanks.

A total of 45 cross sections were surveyed, illustrated in Figure 3.2. There are 7 structures located along Outer Cove Brook. Structure data sheets, describing the condition of these structures and the inlet and outlet channels are included in Appendix B. The most upstream structure is located at Clovelly Golf Course along the main road to the course clubhouse. Two 1600 mm circular culverts are located here. Headwalls and wingwalls constructed of mortar and stones are located at the entrance and exit of the culverts. A man-made pool is located at the outlet. The next structure is located near Hole 9 on the Osprey golf course. It is a 6.0 m wide wooden bridge with concrete abutments and riprap located in the channel banks upstream and downstream. The structure is in excellent condition. A near identical bridge is located slightly downstream on the Osprey Hole 11. Further downstream at the intersection of Lower Road, Logy Bay Road and Outer Cove Road is located Savage Creek Bridge. This bridge is 5.4 m wide and of concrete construction. In the past, this bridge has experienced extremely high water levels during heavy rainfall. During Hurricane Igor this bridge was closed due to flooding. There is currently an agreement in place between the City of St. John's and the province of Newfoundland and Labrador to replace this bridge.

Approximately 585 m downstream there is an abandoned spillway. The spillway is no longer operated and acts as an uncontrolled overflow weir. The concrete is in good condition.

Slightly downstream of the spillway is MacDonald's Road Bridge. It is a 3.6 m wide concrete bridge with concrete abutments. It is currently in very poor condition, with large pieces of concrete having fallen into the river.

The structure at the most downstream location is a new concrete bridge constructed in 2010 at Marine Drive.

3.1.3 Coakers River

On October 17, 2011, two CBCL employees visited Coakers River to mark cross section locations and take photos (Appendix A) and notes of the characteristics of the river and banks.

A total of 44 sections were surveyed, as illustrated in Figure 3.3. Seven structures are located along Coakers River. Descriptions and photos of the structures are presented in Appendix B. The most upstream structure is located on Ashkay Drive. This is a new concrete box culvert. The upstream and downstream channels are lined with riprap.

Approximately 560 m downstream, crossing at Logy Bay Road, are two culverts, one 1000 mm circular culvert and one 1240 x 840 mm pipe arch.

The next downstream structure is a single 600 mm circular culvert located at the dairy farm. This culvert is in poor condition and has insufficient cover. The channel and banks upstream and downstream of the culvert are tall grass and wetland. The culvert was flowing full and the road was overtopped on the day the high water levels and flows were collected (Figure 3.4).

Between the farm culvert and Marine Drive there is a significant amount of wetland. The depth of the wetland did not allow for two proposed sections to be surveyed, and therefore, they were estimated using LiDAR and bounding cross sections. These interpolated cross sections are located at sections CR23 and CR24 on Figure 3.3.

At Marine Drive there is a 2400 mm circular culvert that is in good condition.

Slightly downstream is a wooden bridge with concrete abutments connecting residence 320 Marine Drive to Marine Drive.

Further downstream are two circular culverts crossing at the driveway to residence 280 Marine Drive.

The most downstream structures are three 900 mm circular culverts. The channel upstream is obstructed by fallen trees.

3.1.4 Drukens River

On October 18, 2011, two CBCL employees walked Drukens River and marked cross section locations. During the visit, notes and photos were taken of the river channel and overbanks, and are included in Appendix A.

24 cross sections were surveyed, as illustrated in Figure 3.5. Two structures are located on the river. Photos and descriptions of the structures are presented in Appendix B. The most upstream structure is located at Logy Bay Road, consisting of a 600 mm circular culvert and a 900 mm circular culvert.

The downstream structure is located at 250 Marine Drive and consists of a single 1800 x 1200 mm pipe arch culvert.

3.2 Calibration Data

To aid in the calibration of the HEC-RAS models, flow and water level data was collected during two flow conditions: one normal flow (i.e. dry weather) and one flow following a heavy precipitation event. CBCL did not determine the return period of this rainfall event.

On October 19, 2011, an Environment Canada employee (Mr. Perry Pretty) collected flow measurements at the outlet of each of the four study rivers. While Mr. Perry was collecting these flows, a CBCL engineer marked water levels at various locations along each river. These locations are illustrated on Figure 3.6. A second set of calibration data was gathered on October 27th, following a heavy rainfall (the total rainfall over October 26th and 27th was approximately 54 mm). The measured flows and water levels are listed in Table 3-1 and 3-2.

TABLE 3-1 FLOW MEASUREMENTS

Basin	Measured Flow (m ³ /s)	
	19-Oct-11	27-Oct-11
Kennedys Brook	0.110	1.360
Outer Cove Brook	0.228	2.643
Coakers River	0.021	0.519
Drukens River	0.049	0.573

TABLE 3-2 WATER LEVEL MEASUREMENTS

Location	Measured WL (m)	
	19-Oct-11	27-Oct-11
1	3.9	4.1
2	39.7	40.0
3	45.5	45.7
4	3.0	3.2
5	5.5	N/A
6	64.5	64.7
7	87.6	87.9
8	32.3	32.5
9	52.8	N/A
10	76.8	76.9
11	80.5	80.6
12	38.5	38.8
13	68.0	68.2
14	73.0	73.2

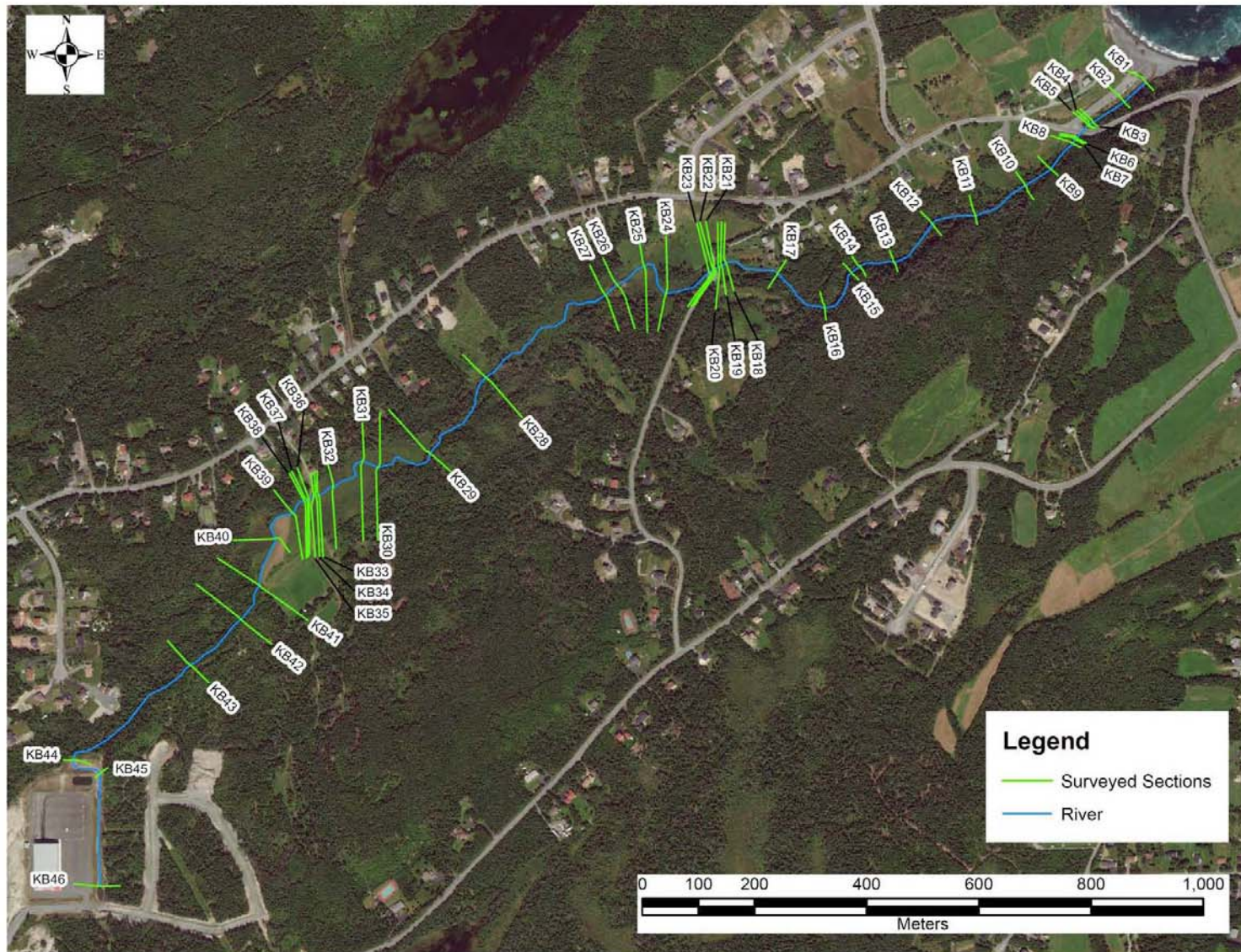
3.3 Detailed Terrain Data

CBCL retained Leading Edge Geomatics to produce detailed terrain data through Light Detection and Ranging (LiDAR). Leading Edge Geomatics used LiDAR to produce a digital elevation model (DEM) containing one elevation point per meter. One meter contours were created from the DEM. The accuracy of the LiDAR data is described in Appendix C.

The LiDAR data is used to create the overbank portions of the cross sections entered in the hydraulic models. It is also used in the flood risk mapping exercise. By intersecting the water surface, generated by the hydraulic model, on the LiDAR terrain surface, floodplain polygons are produced.

3.4 Hydraulic Structure Details

Photos, measurements and notes regarding hydraulic structure were collected during the field investigations and are presented in Appendix B. Twenty-two structures were examined in addition to those located on the four main channels. Hydraulic structure assessment is discussed further in Section 8.

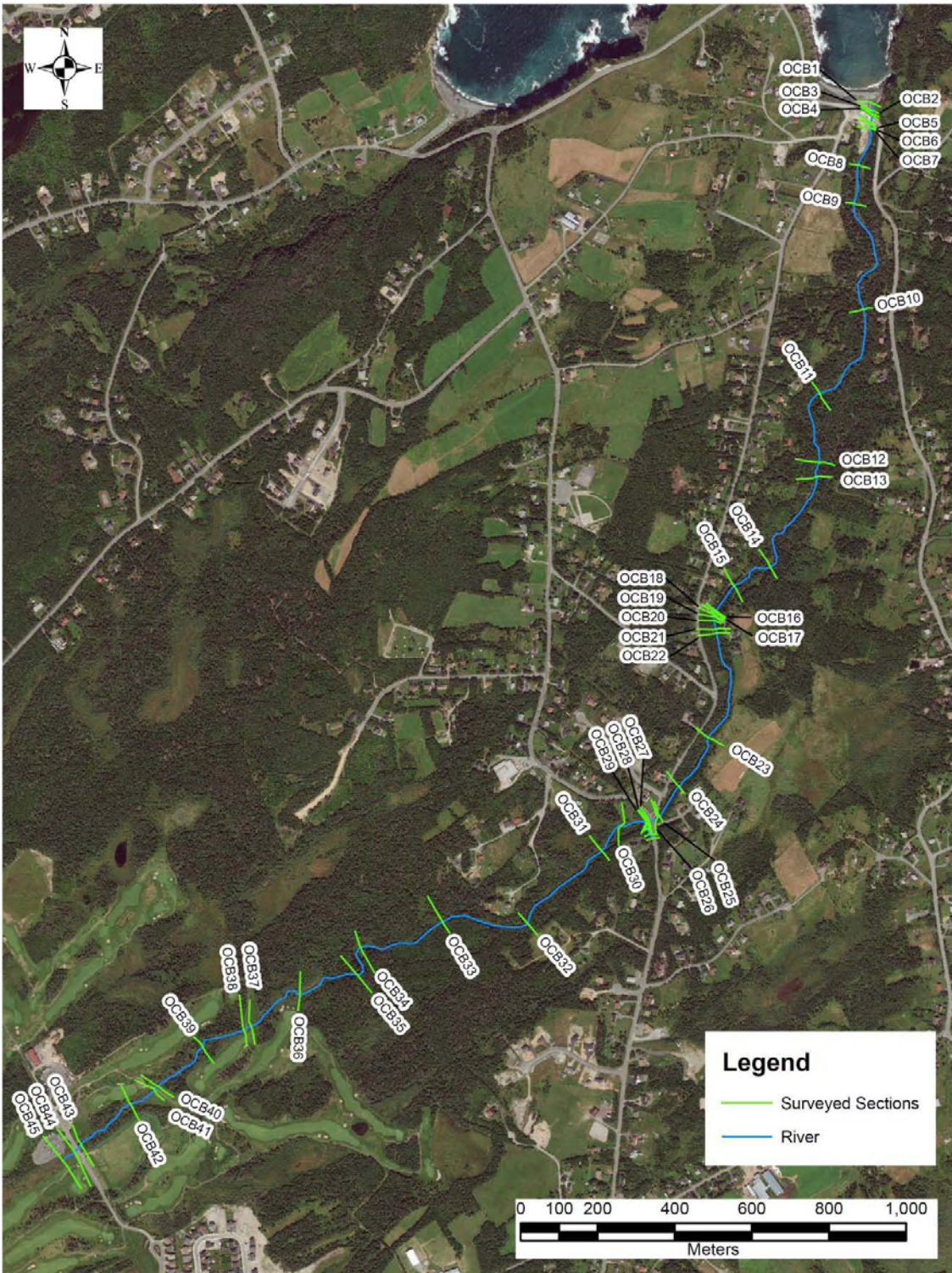


Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

Date: June 2012
Proj. #: 113076.00

Surveyed Sections – Kennedys Brook

Figure 3.1

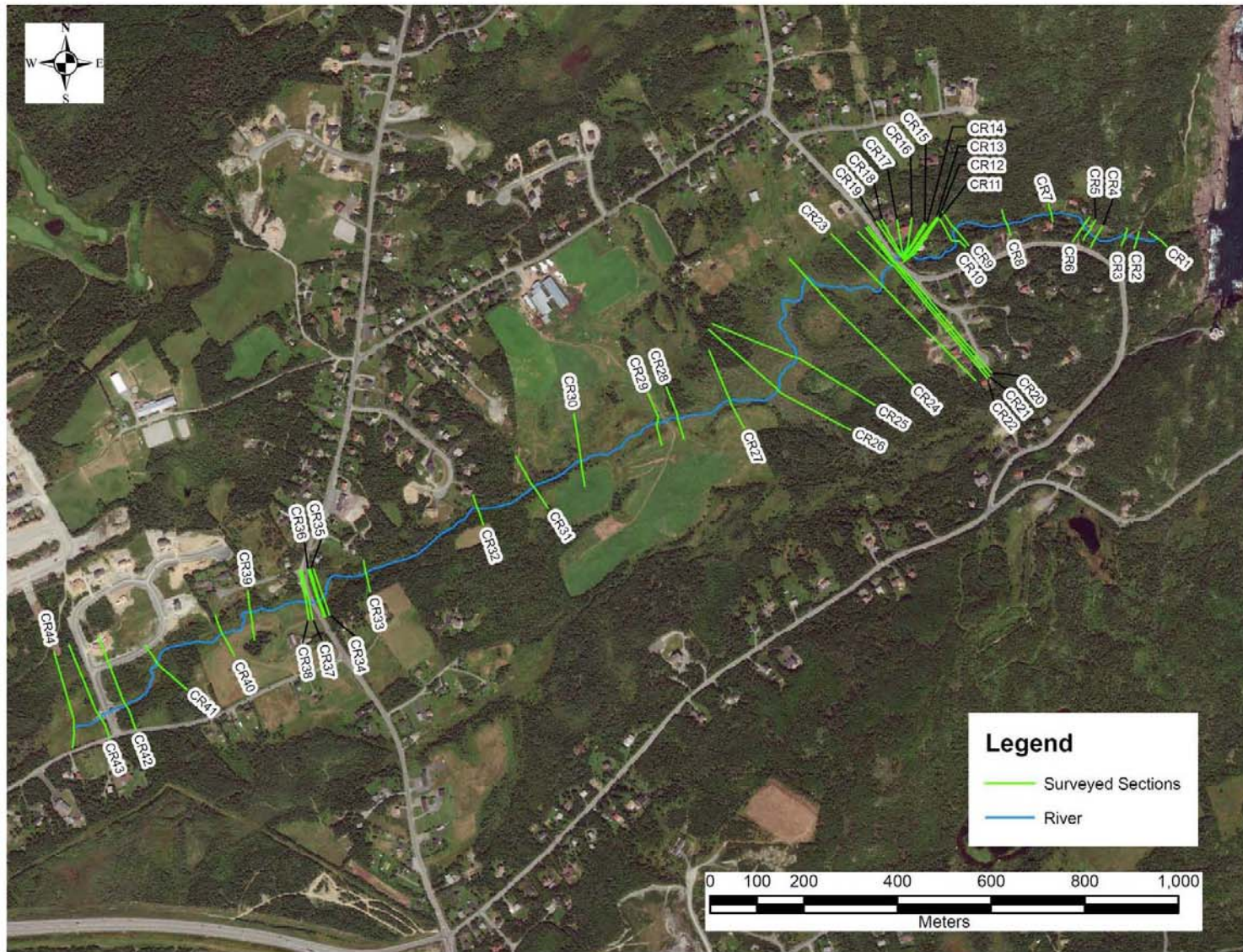


Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

Date: June 2012
Proj. #: 113076.00

Surveyed Sections – Outer Cove Brook

Figure 3.2



Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

Date: June 2012
Proj. #: 113076.00

Surveyed Sections – Coakers River

Figure 3.3

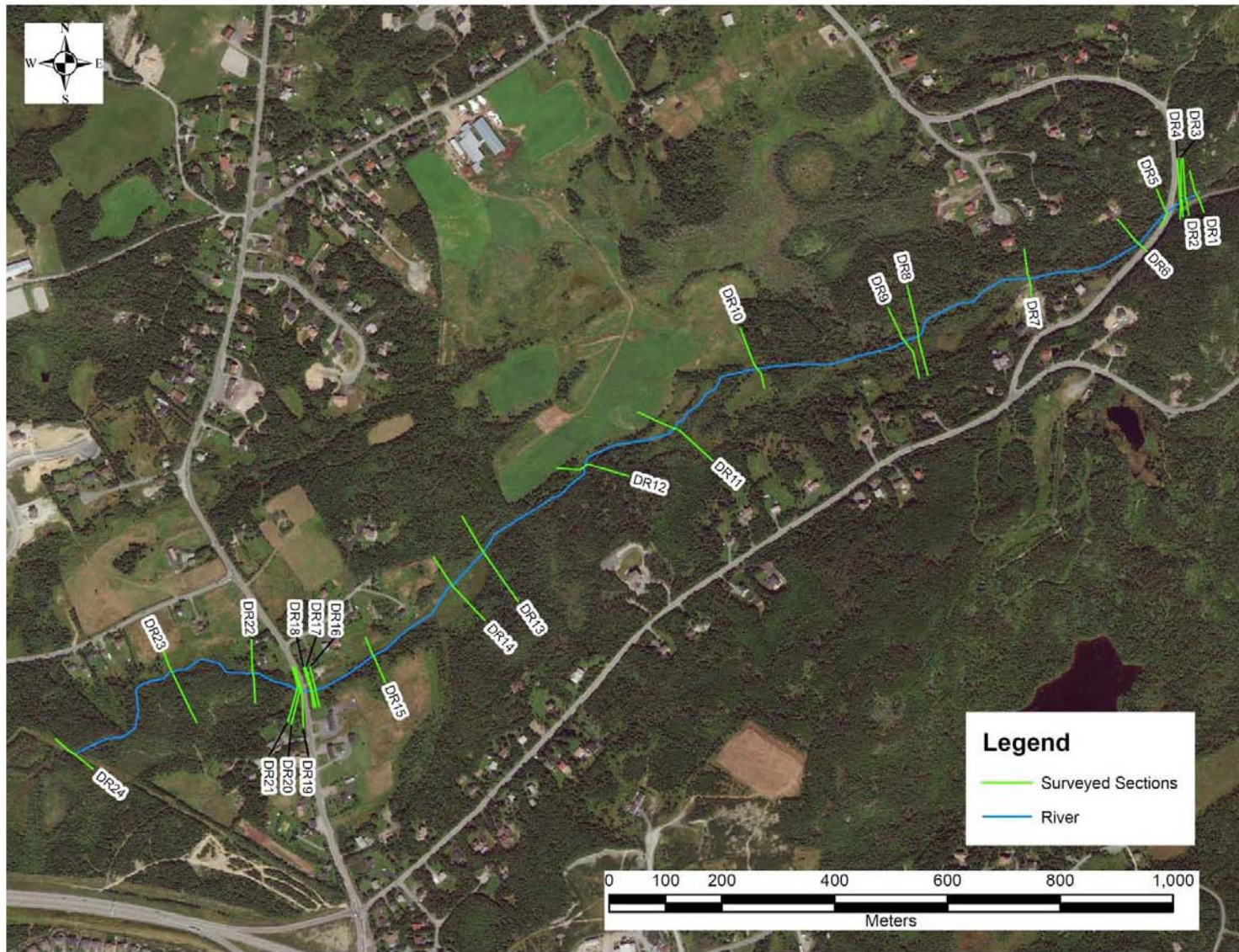


Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

Date: June 2012
Proj. #: 113076.00

Dairy Farm Culvert Overtopping on October 27, 2011

Figure 3.4

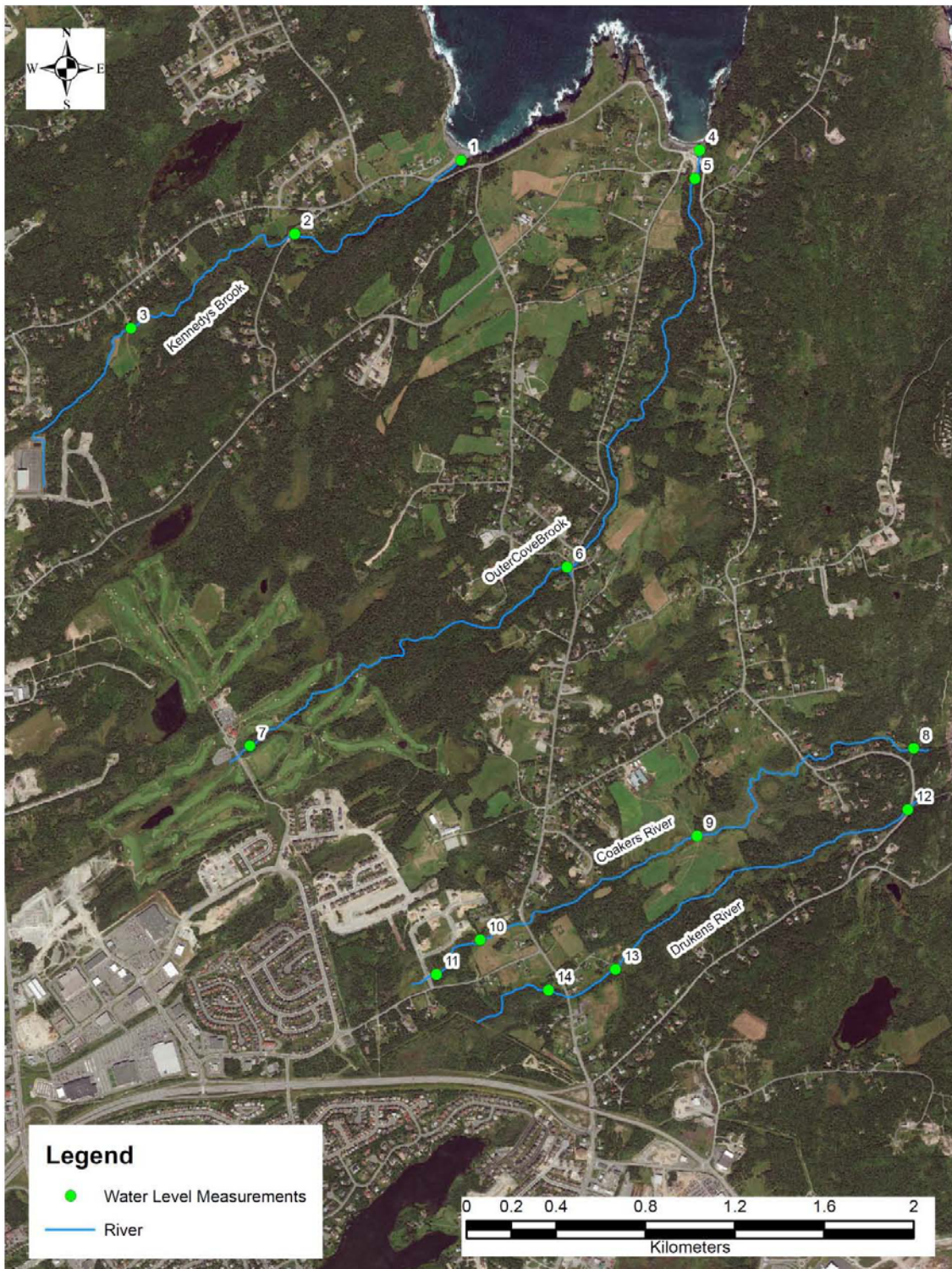


Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

Date: June 2012
Proj. #: 113076.00

Surveyed Sections – Druzens River

Figure 3.5



Town of Logy Bay-Middle Cove-Outer Cove
 Flood Risk Mapping

Date: June 2012
 Proj. #: 113076.00

Locations of Water Level Measurements

Figure 3.6

CHAPTER 4 **UPDATE OF IDF CURVES**

IDF curves describe rainfall patterns for a particular 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, generally the 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 AEP. For this study, CBCL updated the IDF curves used in the hydrological analysis in order to account for the significance of recent heavy rainfall events.

4.1 Existing Data

Environment Canada maintains rain gauges throughout the province 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 (gauge number 8403506) at an elevation of 131 m. The original instrument used to collect rainfall data was a tipping bucket rain gauge, which 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 Environment Canada in 1996 and are found in Appendix D.

Since the Airport gauge is located a short distance from LB-MC-OC, and is actually located within the drainage basin for Outer Cove Brook, it is reasonable to assume that precipitation events experienced at this gauge and within the study area are similar. Accordingly, the IDF curves for the St. John's Airport gauge can be used in the estimation of flood flows.

4.2 Additional Data

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

The availability of additional sources of data was explored and it was discovered that the City of St. John's 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 Rennies River¹. 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. The 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.

4.3 Update

Data from the Windsor Lake gauge was obtained from the City for 2001-2010 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 the durations are presented in Appendix E.

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 of St. John's 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 still the largest precipitation event recorded at the Windsor Lake gauge and could still be considered an outlier.

Through discussions with the City of St. John's Hydrological Engineer, CBCL learned that the recorded rainfall during Hurricane Igor was likely underestimated at all of the City's gauges. The City's Engineer indicated that flows recorded at hydrometric stations throughout the City were higher during Hurricane Igor than Tropical Storm Gabrielle. He estimated the actual 24-hour rainfall amount 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 analysis was performed on each of the eight data sets to update the IDF curves. Several distributions were examined including the Lognormal, 3-Parameter Lognormal, Log Pearson Type III and the Gumbel distributions. Each distribution was examined based on visual goodness-of-fit and several statistical tests. The choice of distribution also took into consideration the distribution preferred by Environment Canada for IDF generation, the Gumbel distribution. This distribution displayed visual goodness-of-fit comparable to the other distributions and above satisfactory results for the statistical tests. The Gumbel distribution was chosen for the IDF update, which is consistent with Environment Canada's practice.

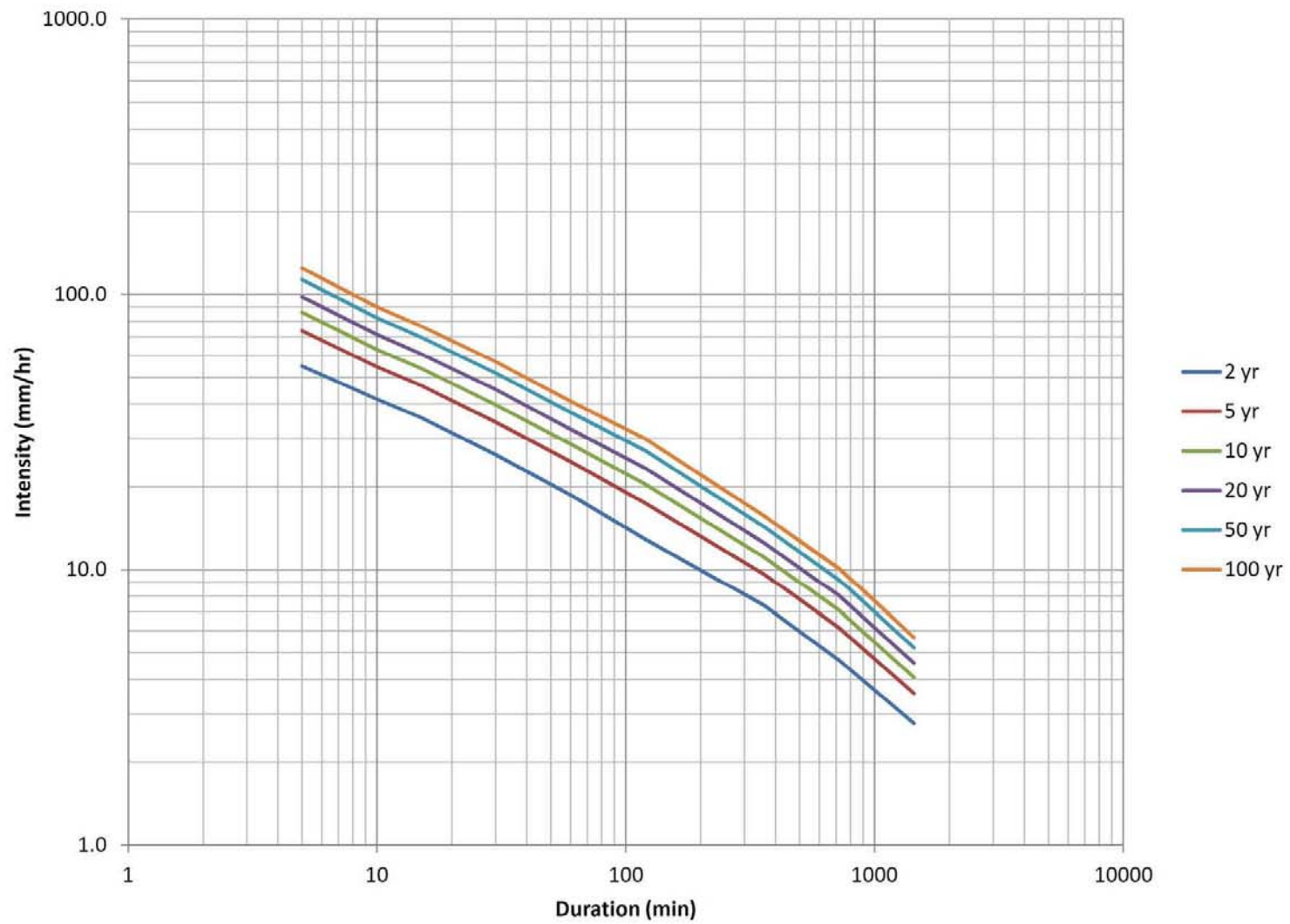
¹ Wadden, David. August, 2002. *Rainfall Distribution in the City of St. John's: Temporal Distribution, Spatial Variation, Frequency Analysis, and Tropical Storm Gabrielle*. Prepared for School of Graduate Studies, Faculty of Engineering and Applied Science, Memorial University of Newfoundland, St. John's, NL.

4.4 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. A comparison of the updated IDF to the 1996 IDF generally indicates slight decreases in rainfall intensity for the shorter duration storms and significant increases in intensity for the longer duration storms. For example, the 1:100 AEP 24-hr rainfall amount for the updated IDF is approximately 23% greater than that of the 1996 IDF.

TABLE 4-1 UPDATED IDF RAINFALL INTENSITIES

Duration (min)	Intensity (mm/hr)					
	2 yr	5 yr	10 yr	20 yr	50 yr	100 yr
5	55.0	73.7	86.0	97.9	113.3	124.8
10	41.6	54.6	63.0	71.4	82.2	90.0
15	35.7	46.8	54.0	60.8	70.0	76.8
30	26.2	34.4	39.8	45.2	51.8	57.0
60	18.7	24.7	28.6	32.4	37.3	40.9
120	12.9	17.5	20.5	23.4	27.1	29.9
360	7.4	9.6	11.1	12.5	14.3	15.7
720	4.7	6.1	7.1	8.0	9.2	10.1
1440	2.8	3.6	4.1	4.6	5.2	5.7



Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

Date: June 2012
Proj. #: 113076.00

Updated IDF Curves

Figure 4.1

CHAPTER 5 **HYDROLOGIC ANALYSIS**

The 1:20 and 1:100 AEP flood flows for Kennedys Brook, Outer Cove Brook, Coakers River and Drukens River were estimated using statistical and deterministic (modeling) approaches. The statistical approach includes single station frequency analysis and regression equations given in the Regional Flood Frequency Analysis report prepared by WRMD. The deterministic approach involved creating hydrologic models of the four watercourses to determine flood flows. The following sections discuss the methodology and results of each approach.

5.1 Statistical Analysis

5.1.1 Single Station Frequency Analysis

One method of estimating flood flows is accomplished by obtaining annual peak instantaneous flows from a hydrometric station and performing statistical analysis on the data set. Water Survey of Canada (WSC) operates and maintains hydrometric gauges throughout Newfoundland and Labrador, and Canada. Unfortunately, there are no WSC gauges located on any of the four streams, or their tributaries, located in the study area. A temporary hydrometric station, owned by WRMD, was installed on Outer Cove Brook, approximately 115 m downstream of Savage Creek Bridge. However, this gauge has only been recording data since April 2011. Therefore, the length of record is not long enough to estimate return period flood flows.

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.

In the absence of a useful flow record within the study area, single station frequency analysis was conducted on several nearby hydrometric station records. The results of this analysis were then transferred to each of the study basins by drainage area proration. The following criteria were defined in choosing appropriate gauges for single station frequency analysis:

- Gauged drainage area comparable to study drainage area
- In close proximity to the study area
- Long term periods of record (longer than 20 years)
- Have natural flow (i.e. non-regulated flow)

- Have up-to-date data available (i.e. include peak instantaneous maximum flows up to and including 2010)

Three stations were chosen based on these criteria. The following table describes each gauge. Figure 5.1 illustrates the location of the gauged basins in relation to the study area.

TABLE 5-1 ENVIRONMENT CANADA HYDROMETRIC GAUGES

Station Name	Station Number	Drainage Area (km ²)	Period of Record	# of Years of Data
Northeast Pond River at Northeast Pond	02ZM006	3.63	1970-2010	41
Virginia River at Pleasantville	02ZM018	10.7	1984-2010	25
Learys Brook at Prince Philip Drive	02ZM020	17.8	1987-2010	23

The annual peak instantaneous flow series for each gauge is provided in Appendix F. The data series for Learys Brook and Virginia River gauges have one and two missing data points, respectively. These peak flows were estimated prior to conducting frequency analysis by estimating a peaking factor for each 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. These estimated values are shown in Appendix F.

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

- Randomness: variations in the data set are a result of natural causes (ie. 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

The results of these screening tests for each hydrometric station are included in Appendix G. Although some of the screening tests failed, this does not necessarily mean the results of the frequency analysis are invalid. However, it does indicate that caution should be used in the interpretation and extrapolation of the results.

Several statistical distributions were examined, including Gumbel, Generalized Extreme Value (GEV), Lognormal, 3-Parameter Lognormal and Log Pearson Type III. The most appropriate distribution for each data set was selected based on visual goodness-of-fit and statistical test. The chosen distribution for each station, and the resulting AEP flow estimates are listed in table below. Figures 5.2 to 5.4 illustrate the selected distributions, along with the 95% confidence interval.

TABLE 5-2 SINGLE STATION FREQUENCY ANALYSIS RESULTS

Station Name	Distribution	Q20 (m ³ /s)	Q100 (m ³ /s)
Northeast Pond River at Northeast Pond	LN	6.6	8.5
Virginia River at Pleasantville	LN	16.0	19.9
Learys Brook at Prince Philip Drive	3PLN	42.6	63.0

The results of the frequency analysis were transferred to the study area by drainage area proration. The prorated flows for the 1:20 and 1:100 AEP flood at the outlet of each study area are presented in Table 5-3 and 5-4, respectively. Averages of the prorated flows are also presented.

TABLE 5-3 1:20 AEP FLOW ESTIMATES BASED ON DRAINAGE AREA PRORATION

Hydrometric Station	1:20 AEP Flow at Basin Outlet (m ³ /s)			
	Kennedys Brook	Outer Cove Brook	Coakers River	Drukens River
Northeast Pond River at Northeast Pond	11.5	22.3	6.7	3.3
Virginia River at Pleasantville	9.5	18.4	5.5	2.7
Leary Brook at Prince Philip Drive	15.1	29.4	8.8	4.3
Average	12.0	23.4	7.0	3.4

TABLE 5-4 1:100 AEP FLOW ESTIMATES BASED ON DRAINAGE AREA PRORATION

Hydrometric Station	1:100 AEP Flow at Basin Outlet (m ³ /s)			
	Kennedys Brook	Outer Cove Brook	Coakers River	Drukens River
Northeast Pond River at Northeast Pond	14.9	28.9	8.7	4.2
Virginia River at Pleasantville	11.8	22.9	6.9	3.3
Learys Brook at Prince Philip Drive	22.4	43.5	13.1	6.4
Average	16.3	31.8	9.5	4.6

These estimates should be used with caution since there are many physical factors that can influence flow in the gauged basins that the study basins may not experience. For example, the basin slope, the amount and intensity of the precipitation event, the shape of the basin, the soil type and land use, etc., all vary from basin to basin. If the temporary gauge on Outer Cove Brook had a longer period of record, the daily flows could be compared to the nearby gauged data to determine whether a correlation exists. This would help in the selection of the most representative gauge to use to estimate flood flows.

5.1.2 Regional Flood Frequency Analysis

As there is no flow data available for the study areas, peak flows for the watersheds were estimated using the *Regional Flood Frequency Analysis for the Island of Newfoundland* (RFFA) developed by WRDM. The Geographic Information System (GIS) software was used to extract RFFA specific watershed characteristics from 1:50,000 scale mapping obtained from Natural Resources Canada. These characteristics include percent of watershed area covered by lakes, swamps, trees and barren land as well as average slopes and a shape parameter.

According to Figure 4.1 of the RFFA manual, the study watersheds are located in the south east hydrological region. Using the RFFA spreadsheet parameters and the south east equations, the estimated 1:20 and 1:100 AEP flows and upper 95% confidence limits were estimated for each watershed as presented in Table 5-5. The RFFA spreadsheet outputs are shown in Appendix H. The estimates were based on the equations for drainage area only because the lake attenuation factor (LAF), the second variable added to the regression equations, was equal to zero for the four watersheds. According to the RFFA, the exemption of this variable is not expected to have a significant impact on the return period flow estimates because the drainage area is the most important variable in this region. In terms of the squared multiple R (SMR) statistics, the drainage area accounted for 90 – 94% of the variation in flood flows, while the addition of the LAF parameter only marginally increases the SMR to 93 – 97%.

TABLE 5-5 REGIONAL FLOOD FREQUENCY ANALYSIS ESTIMATES

Study Basin	RFFA Flood Flow Estimates (m ³ /s)	
	1:20 AEP	1:100 AEP
Kennedys Brook	9.9	12.6
Outer Cove Brook	16.5	21.0
Coakers River	7.0	9.0
Drukens River	3.8	4.8

5.1.2.1 APPLICABILITY OF ESTIMATES

As shown in Appendix H, several of the calculated factors are either outside the acceptable range or near the extreme. The RFFA manual cautions against use of the regression equations for watersheds that have physiographic parameters near extremes. It also states that small errors in extracting the physiographic parameters may lead to larger errors in flood flow estimation. For example, the manual states an error of 3% in drainage area can lead to a maximum anticipated error of 3.4% in the return period flow estimation. Similarly, a 3% error in the extraction of lakes and swamps can result in a flow error of 1.3%. In addition, to these cautionary items the report also states that these equations cannot be used on urban watersheds. Therefore, the RFFA method was examined, but the results are used as a comparison, or check, of the flow estimates determined from the single station frequency analysis and the hydrologic modelling exercises.

5.2 Deterministic Analysis

Hydrologic modeling was carried out using the Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) and its geospatial modelling extension (HEC-geoHMS). HEC-HMS was developed by the US Army Corp of Engineers (USACE) and is specifically geared towards the precipitation-runoff processes of watershed systems. It allows the user to select from a variety of methods to simulate ground infiltration, transformation of surplus precipitation, baseflow and open channel flow. HEC-GeoHMS is a GIS modeling extension that allows the modeller to extract watershed characteristics, define subbasins and streams, and assemble hydrologic model inputs to be used directly with HEC-HMS.

Prior to using HEC-geoHMS to extract watershed characteristics, terrain and basin pre-processing is completed to derive the drainage network, using the ArcHydro toolbar in GIS. Pre-processing includes determining flow direction, flow accumulation, stream definition and basin delineation. During basin pre-processing, the watershed is divided into sub-basins based on project specific requirements. For instance, part of this study includes assessing various hydraulic structures within each watershed. Using the basin pre-processing tools, the subbasin contributing to each individual structure was delineated. Upon running the HEC-HMS model, design flows for each structure can then be extracted from the results.

Following stream and subbasin delineation, physical characteristics such as stream length and slope, longest flow paths, centroidal flow lengths and basins slopes are extracted from the terrain data using the HEC-geoHMS extension. It is also used to extract hydrologic parameters such as curve numbers and basin lag time, and to specify the loss, transform, baseflow and river routing methods to be used in the hydrologic model.

5.2.1 Model Elements

5.2.1.1 SUBBASIN AREAS

Hec-geoHMS was used to delineate the watershed areas for each of the four rivers. The resulting delineations were then compared to those created by the City of St. John's for the upper portions of each river reach. Minor edits were made to the delineated watersheds to correspond to the City of St. John's areas. Figure 1.2 illustrates the watershed delineations for the four rivers in this study.

During the delineation process, it was discovered that Drukens River is connected to Coakers River through a bifurcation located in the wetland area between Marine Drive and the Dairy Farm culverts (cross sections CR20 to CR27 on Figure 3.3). As a result of the river split, HEC-geoHMS was not able to properly delineate the watersheds for Drukens River or Coakers River. The delineation of Drukens River would include all of Coakers River drainage area upstream of the wetland. No discernible river flowing out of Coakers River wetland could be located during a site visit. To accommodate the flow from Coakers River into Drukens River, the drainage areas were delineated as if no connection existed and an estimate of percentage flow from Coakers River to Drukens River was made. The HEC-HMS results for Drukens River and Coakers River were then altered manually, outside the program.

The percentage of flow from Coakers River to Drukens River was estimated using recorded flow measurements (Section 3). It was assumed that there was adequate rainfall on October 26th and 27th to saturate the wetland such that it was no longer acting to attenuate flow. It was also assumed that any additional overland flow from Coakers River and Drukens River watersheds, downstream of

the wetland, would be negligible. The percentage flow was then calculated using the measured flows and assuming the total flow out of the wetland (Coakers River measurement plus Drukens River measurement) was equal to the total flow into the wetland. The result being approximately 35% of the flow entering Coakers River wetland is discharged to Drukens River and the remaining 65% continues on Coakers River. These estimates are based on several assumptions and should be used with caution. Ideally, flow gauges should be installed on each river downstream of the wetland, and on at least one river upstream of the wetland to confirm the assumptions.

5.2.1.2 LOSS METHOD

Subbasin elements in HEC-HMS represent ground infiltration, surface runoff and subsurface processes. The infiltration calculations are performed through the selection of a loss method contained in the subbasin. The Soil Conservation Service (SCS) curve number loss method was used for this study, which is a well-established method, and is the method employed by the City of St. John's in performing hydrological analysis.

The SCS curve number loss method relates depth of runoff to rainfall, potential maximum soil moisture retention, and initial abstraction through the following equation.

$$Q = \frac{(P - I_a)^2}{P - I_a + S}$$

Where Q = Depth of water retained in the watershed

P = Depth of excess precipitation

I_a = Initial abstraction before ponding begins and

S = potential maximum retention

$$S = \frac{1000}{CN} - 10$$

Where CN = runoff curve number

Runoff curve numbers (CN) are determined through a combination of land use, antecedent soil moisture and soil type. Typical values of CN range between 30 and 100. A large value of CN represent impervious areas, for example a value of 98 corresponds to paved streets, while a CN of 39 represents open space with more than 75% grass cover.

Anderson and Yates Forest Consultants (AYFC) completed land use analysis using high resolution satellite imagery provided by WRMD. The land uses were classified in accordance with WRMD's requirements, and included the following classes:

- Forest
- Residential
- Commercial
- Open space (ex. Parks, cemeteries, golf course, grassed area)
- Swamps/wetlands
- Water bodies
- Deforested areas
- Barren land
- Fields/pastures

At CBCL’s request, AYFC included an additional classification called “Road”. This was done to simplify the process of accounting for public roads in areas where both commercial and residential developments exist. Appendix I contains AYFC’s report describing the classification process. The final land classification was presented in raster and shapefile format. Figure 5.5 shows the results of the land classification analysis.

Antecedent moisture conditions (AMC) are used to describe the moisture conditions of soil within the watershed preceding a precipitation event. There are three categories of AMC as described below:

- AMC I – Low moisture
- AMC II – Average moisture (generally used in estimating flood flows)
- AMC III – Excessive moisture (generally representing soil conditions resulting from heavy precipitation in the preceding days)

AMC II was assumed for the selection of curve numbers for this study, which is normally used for calculating flood estimates.

As mentioned above, curve number selection is also related to soil type. There are four hydrologic soil groups (A, B, C and D), which are characterized by drainage. Technical Reference-55 (TR-55) describes the drainage of each soil group as presented in the table below.

TABLE 5-6 HYDROLOGIC SOIL GROUPS DESCRIPTION

Soil Group	Drainage Description
A	Well to excessively drained (high infiltration rates)
B	Moderately well to well drained (moderate infiltration rates)
C	Imperfectly drained (low infiltration rates)
D	Poorly to very poorly drained (very low infiltration rates)

Soil information was obtained from the National Soil DataBase available through Agriculture and Agri-Foods Canada. The various soil types within the study area are described in the table below. These soil types were related to the hydrologic soil groups based on drainage, and are also listed in the table. Figure 5.6 illustrates the hydrologic soil groups within the study areas.

TABLE 5-7 SOIL TYPES

Map unit	Texture and Stoniness	Drainage	Soil Type
NFNFD021Cr/C3	Loam moderate to exceedingly stony	Moderately well drained	B
NFNFD021Cr/c4	Loam moderate to exceedingly stony	Moderately well drained	B
NFNFD021Cr/D4	Loam moderate to exceedingly stony	Moderately well drained	B
NFNFD021Cr/E3	Loam moderate to exceedingly stony	Moderately well drained	B
NFNFD021Cr/E3I	Loam moderate to exceedingly stony	Moderately well drained	B
NFNFD021Cr/F3	Loam moderate to exceedingly stony	Moderately well drained	B

TABLE 5-7 SOIL TYPES (CONTINUED)

NFNFD021Cr:O3/C4-2	Loam moderate to exceedingly stony - organic	Moderately well drained - Very poorly drained	BC
NFNFD021Cr:Pc/D4	Loam moderate to exceedingly stony - Loam very stony	Moderately well drained - Imperfectly to poorly drained	BC
NFNFD021Cr:Tb/c2	Loam moderate to exceedingly stony - Loam exceedingly stony	Moderately well drained - Poorly drained	BC
NFNFD021Cr:Tb/D2	Loam moderate to exceedingly stony - Loam exceedingly stony	Moderately well drained - Poorly drained	BC
NFNFD021Cr:Tb/D3	Loam moderate to exceedingly stony - Loam exceedingly stony	Moderately well drained - Poorly drained	BC
NFNFD021M8/B	Organic	Very poorly drained	D
NFNFD021O3/B	Organic	Very poorly drained	D
NFNFD021O3:Tb/B3	Organic - Loam exceedingly stony	Very poorly drained - Poorly drained	D
NFNFD021O3:Tb/B4	Organic - Loam exceedingly stony	Very poorly drained - Poorly drained	D
NFNFD021O5/B	Organic	Very poorly drained	D
NFNFD021O5:Tb/B4	Organic - Loam exceedingly stony	Very poorly drained - Poorly drained	D
NFNFD021Rc/H3IV	Sandy loam exceedingly stony	Moderately well drained	B
NFNFD021Rc:Cr/F4III	Sandy loam exceedingly stony - Loam moderate to exceedingly stony	Moderately well drained	B
NFNFD021Rc:Cr/G3IV	Sandy loam exceedingly stony - Loam moderate to exceedingly stony	Moderately well drained	B
NFNFD021Tb/B4	Loam exceedingly stony	Poorly drained	C
NFNFD021Tb:O3/B4	Loam exceedingly stony - Organic	Poorly drained - Very poorly drained	D
NFNFD021Tb:O5/B4	Loam exceedingly stony - Organic	Poorly drained - Very poorly drained	D
NFNFD021ZZ	Water	N/A	-

Table 5-8 presents the resulting curve numbers used in the analysis.

TABLE 5-8 CURVE NUMBERS (AMC II)

Land Use Description	Curve Number for Soil Group			
	A	B	C	D
Forest	25	55	70	77
Open spaces (lawns, parks, golf courses, etc.)	39	61	74	80
Fields pastures/good condition	39	61	74	80
Residential (2 acre lots)	46	65	77	82
Residential (1 acre lots)	51	68	79	84
Residential (1/2 acre lots)	54	70	80	85
Barren	76	85	89	91
Residential (1/8 acre lots)	77	85	90	92
Industrial	81	88	91	93
Roads (paved with open ditches)	83	89	92	93
Commercial	89	92	94	95
Water	100	100	100	100

5.2.1.3 TRANSFORM METHOD

The amount of surface runoff from excess precipitation is calculated using a transform method specified for each subbasin within the model. The SCS unit hydrograph transform method was selected for this study. This method was developed from a large number of recorded observations of rainfall and runoff on small watersheds. It assumes the watershed hydrograph is a single peaked hydrograph. The SCS unit hydrograph method was selected since the study basins are all relatively small areas, and the characteristics of the basins and past flood events suggest the rivers experience single peaks during a single event. Also, the SCS unit hydrograph method is used by surrounding municipalities, such as the City of St. John's, therefore selecting this method allows a meaningful comparison to be made against flood flows that have been estimated during previous studies of the area.

The SCS unit hydrograph method uses lag time, which is calculated by HEC-geoHMS using the following equation.

$$T_{lag} = \frac{L^{0.8} * (0.039 * S_{CN} + 1)^{0.7}}{(735 * Y^{0.5})}$$

Where L = maximum travel length from the most remote part of the basin (m)

Y = average slope of the drainage basin (%)

And

$$S_{CN} = 254 * \left(\frac{100}{CN} - 1 \right)$$

Where CN = Curve Number (described above)

5.2.1.4 ROUTING METHOD

Reach elements are used in HEC-HMS to represent streams, which connect upstream subbasins to downstream subbasins. This channel flow is modeled by selecting one of several routing methods. For this study the muskingum-cunge method was selected. Parameters required for the muskingum-cunge method include the channel length, slope, roughness and cross sectional shape. The length and slope of each reach were extracted by HEC-geoHMS from the pre-processed terrain data. This method allows actual surveyed river sections to be modeled. For reaches that were not surveyed, trapezoidal shapes were approximated based on detailed contours and photographs. Manning's roughness values for the channel and banks were determined from site photos.

5.2.2 Calibration

Calibration of a HEC-HMS model is achieved by simulating a recorded precipitation event and comparing the output hydrograph with measured flows.

Outer Cove Brook is the only river in this study with an active flow gauge. Accordingly, the HEC-HMS model for Outer Cove Brook was the only model that could be calibrated. The rainfall experienced on the 26th and 27th of October, 2011 was selected for this calibration exercise. Five minute rainfall data recorded at the Windsor Lake rain gauge for this time period was simulated in the Outer Cove Brook HEC-HMS model. The output hydrograph was plotted against the hourly flow gauge data and compared.

The comparison illustrated that the peak flows simulated by HEC-HMS were higher than the observed gauge flows but the modeled limbs of the hydrograph were much lower than the recorded. Attempts were made to increase the modeled hydrograph limbs by applying a baseflow in the model and running the preceding and following days. However, these efforts did not have a significant impact on the rising and falling hydrograph limbs. The runoff curve numbers were adjusted in an effort to decrease the peak of the model hydrograph to correspond with the recorded peak flow. A decreased of 4% in the subbasin curve numbers resulted in a simulated peak flow that closely matches the measured peak, as illustrated in Figure 5.7. Since the main objective of the hydrologic models is to determine peak flows for the 1:20 and 1:100 AEP it was decided that by matching the peaks of the simulated and recorded hydrographs the model was considered calibrated.

Since no flow data is available to calibrate Kennedys Brook, Coakers River or Drukens River, the model parameters as extracted by HEC-geoHMS were used in determining the 1:20 and 1:100 AEP flows. It is recommended that flow gauges be installed on these three streams to calibrate the hydrologic models.

5.2.3 Rainfall Input

HEC-HMS requires a rainfall hyetograph (time-series precipitation data) as input to simulate the rainfall to runoff process. Hyetographs representing the 1: 20 and 1:100 AEP precipitation amounts, as determined from the IDF update, were entered in HEC-HMS to estimate the corresponding 1:20 and 1:100 AEP flood flows. Generally, hyetograph shapes are not known and are approximated using documented methods. However, the City of St. John's has created hyetographs for storms with a 1:100 AEP and various storm durations from historical rainfall data. For this study, two sets of design hyetographs were examined, including those developed by the City of St. John's and synthetic hyetographs created using the alternating block method, as discussed below.

The alternating block method was used to estimate a synthetic hyetograph shape. This method incorporates the precipitation for various durations for a particular return period rainfall event into a single hyetograph. The maximum incremental precipitation is placed at the centre of the storm and the remaining incremental precipitation values are arranged in descending order alternating right and left of the centre. 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 5.8 illustrates the 1:20 and 1:100 AEP hyetographs.

The City of St. John’s “Subdivision Design Manual” specifies hyetographs to be used for modeling a 1:100 AEP event for 0.5, 1, 2, 6, 12 and 24-hour durations. These hyetograph shapes are based on recorded rainfall amounts, and therefore, represent actual observed storms. CBCL used the shape of the City’s hyetographs to model the 1:20 and 1:100 AEP precipitation amounts resulting from the IDF update. These hyetographs are presented in Tables 5-9 and 5-10 below.

TABLE 5-9 1:20 AEP RAINFALL HYETOGRAPHS

% 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%	1.5	2.2	3.2	5.1	1.0	7.4
16.67%	3.9	5.5	8.0	12.8	2.0	18.7
25.00%	6.8	9.8	14.1	22.5	6.8	33.1
33.33%	10.7	15.3	22.1	35.5	17.3	52.0
41.67%	15.3	21.9	31.6	50.8	36.5	74.4
50.00%	18.9	27.0	39.0	62.6	60.5	91.8
58.33%	20.1	28.8	41.5	66.6	77.9	97.6
66.67%	21.0	30.0	43.3	69.6	87.5	102.1
75.00%	21.7	31.1	44.9	72.0	92.3	105.7
83.33%	22.2	31.9	45.9	73.7	94.1	108.2
91.67%	22.5	32.2	46.4	74.6	95.1	109.3
100.00%	22.6	32.4	46.7	75.0	96.1	110.0

TABLE 5-10 1:100 AEP RAINFALL HYETOGRAPHS

% 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	4.0	6.4	1.2	9.2
16.67%	4.9	6.9	10.2	16.0	2.5	23.1
25.00%	8.6	12.3	18.0	28.3	8.5	40.9
33.33%	13.5	19.3	28.2	44.5	21.7	64.3
41.67%	19.3	27.6	40.4	63.7	45.9	92.0
50.00%	23.8	34.1	49.8	78.5	76.2	113.4
58.33%	25.3	36.3	53.0	83.5	98.0	120.6
66.67%	26.5	37.9	55.4	87.3	110.1	126.2
75.00%	27.4	39.3	57.3	90.4	116.2	130.6
83.33%	28.0	40.2	58.7	92.5	118.5	133.7
91.67%	28.3	40.6	59.3	93.6	119.8	135.2
100.00%	28.5	40.9	59.7	94.1	121.0	136.0

Table 5-11 presents the comparison of results for the 1:100 AEP rainfall event for existing basin conditions. The peak flows resulting from the City’s hyetographs are consistently lower than those determined from the alternating block method. For the Kennedys Brook and Outer Cove Brook these differences are minor. However, the alternating block method estimates significantly larger flows at the outlet of Coakers River and Drukens River.

TABLE 5-11 COMPARISON OF ALTERNATING BLOCK AND CITY DESIGN HYETOGRAPHS

Basin	1:100 AEP Flow at Outlet (m ³ /s)	
	Alternating Block Hyetograph	City's Design Hyetograph
Kennedys Brook	21.3	19.7
Outer Cove Brook	44.5	41.6
Coakers River	16.8	12.9
Drukens River	16.7	12.5

Since the City’s hyetographs are created using actual recorded rainfall events, the runoff amounts simulated using these hyetographs are considered more representative of actual conditions. Accordingly, CBCL decided to use the flow results from the City’s hyetographs in the hydraulic modeling (discussed in Section 6). The 12-hour storm duration produced the largest flood flow for Kennedys Brook and Outer Cove Brook and the 6-hour storm produced the largest flows for Coakers River and Drukens River. Flows at a number of smaller, upstream, subbasins were also compared to ensure that the storm duration that produced the largest flow at the outlet also produced the largest flow at points of interest throughout the entire basin.

5.2.4 Effects of Planned Development

The benefit of estimating flood flows using a deterministic method, as opposed to statistical analysis, 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. For example, an area of planned development can be modeled in HEC-HMS by altering the curve number (and consequently lag time) for that area, and the pre- and post-development flows can be compared.

This study examined the effects of planned developments in each watershed, including development areas planned in St. John’s and Torbay. This analysis illustrates the impacts planned developments have on existing hydraulic structures within LB-MC-OC. The LB-MC-OC zoning map is dated 2005 and includes the zoning plan for 2005-2015. The zoning map for St. John’s is frequently updated; the map used for this analysis was received in January 2012. Torbay’s zoning map was for the 2007-2017 municipal plan, and included amendments made in 2011. The three zoning maps are included in Appendix J.

Using zoning plans from the LB-MC-OC, St. John’s and Torbay, the curve numbers and lag times were altered in the hydrologic models to reflect ultimate 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 1:20 and 1:100 AEP flow estimates at the outlets of the four study areas, for existing and ultimate development conditions, are presented in Table 5-12.

TABLE 5-12 1:20 AND 1:100 AEP FLOW ESTIMATES FOR EXISTING AND ULTIMATE DEVELOPMENT CONDITIONS

Basin	Flow at Outlet (m ³ /s)			
	Existing Development Q20	Existing Development Q100	Ultimate Development Q20	Ultimate Development Q100
Kennedys Brook	12.6	19.7	17.3	25.6
Outer Cove Brook	27.0	41.6	39.5	56.7
Coakers River	8.4	12.9	9.5	14.3
Drukens River	7.9	12.5	9.8	14.8

5.3 Summary of Hydrologic Analysis

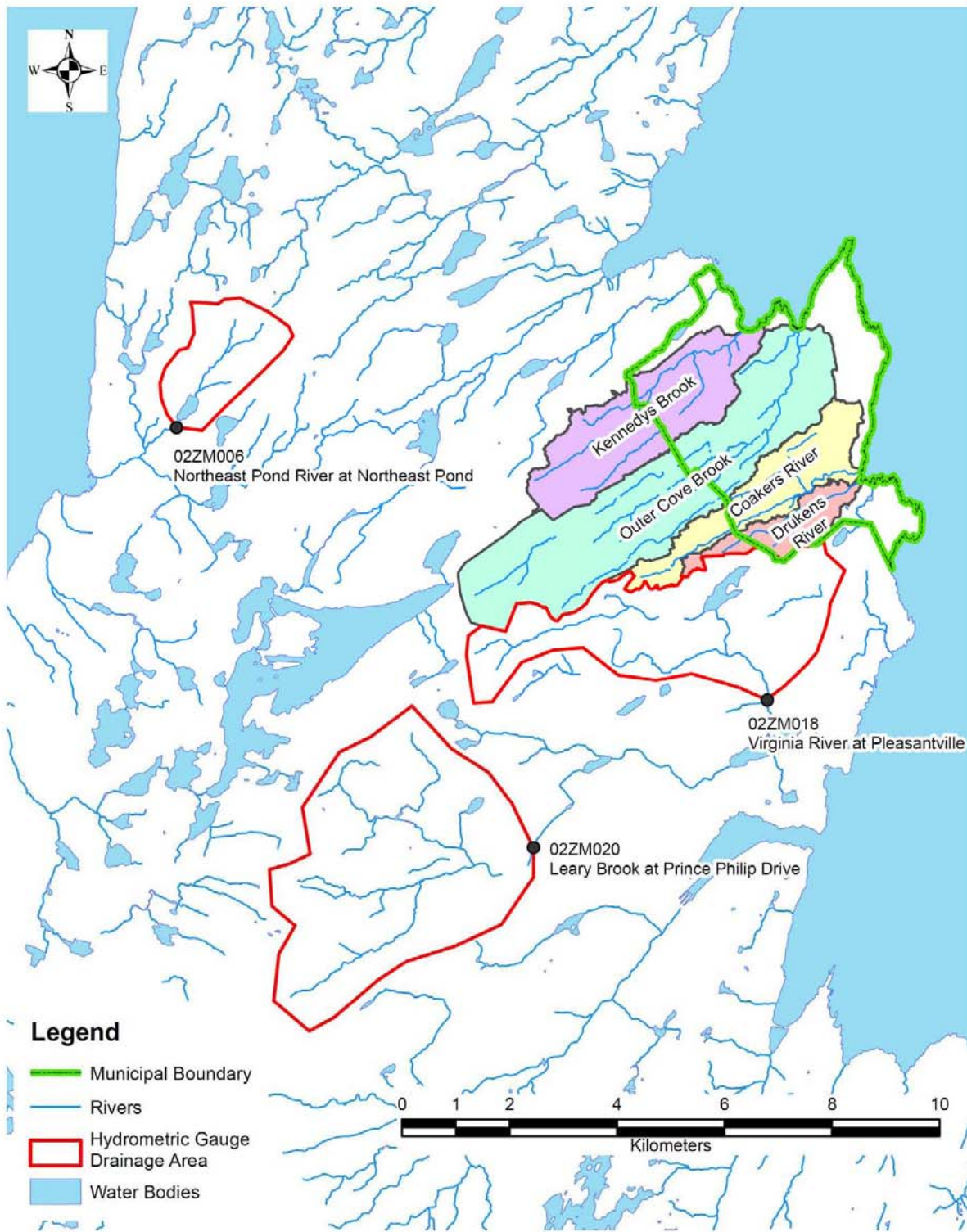
Flow estimates for the 1:20 and 1:100 AEP floods were determined for each of the four study areas using statistical and deterministic analysis as discussed above. Table 5-13 compares the results of the analysis.

This comparison indicates that the flows estimated using the deterministic approach fall within the range of flows computed using statistical analysis for all the rivers with the exception of Drukens River. In the case of Drukens River, the HEC-HMS modeled results are nearly double the upper value of the statistical analysis. This is because Drukens River receives flow from Coakers River, which is difficult to incorporate into the drainage area proration.

TABLE 5-13 SUMMARY OF FLOOD FLOW ESTIMATES

AEP	Method	Flow at Outlet (m ³ /s)			
		Kennedys Brook	Outer Cove Brook	Coakers River	Drukens River
1:20	Single Station (range)	9.5 – 15.1	18.4 – 29.4	5.5 – 8.8	2.7 – 4.3
	RFFA	9.9	16.5	7.0	3.8
	Deterministic	12.6	27.0	8.4	7.9
1:100	Single Station (range)	11.8 – 22.4	22.9 – 43.5	6.9 – 13.1	3.3 – 6.4
	RFFA	12.6	21.0	9.0	4.8
	Deterministic	19.7	41.6	12.9	12.5

The flood flows estimated using the deterministic method are used in the hydraulic analysis and flood risk mapping. Using the deterministic method provides a means to evaluate post-development scenarios and climate change scenarios.

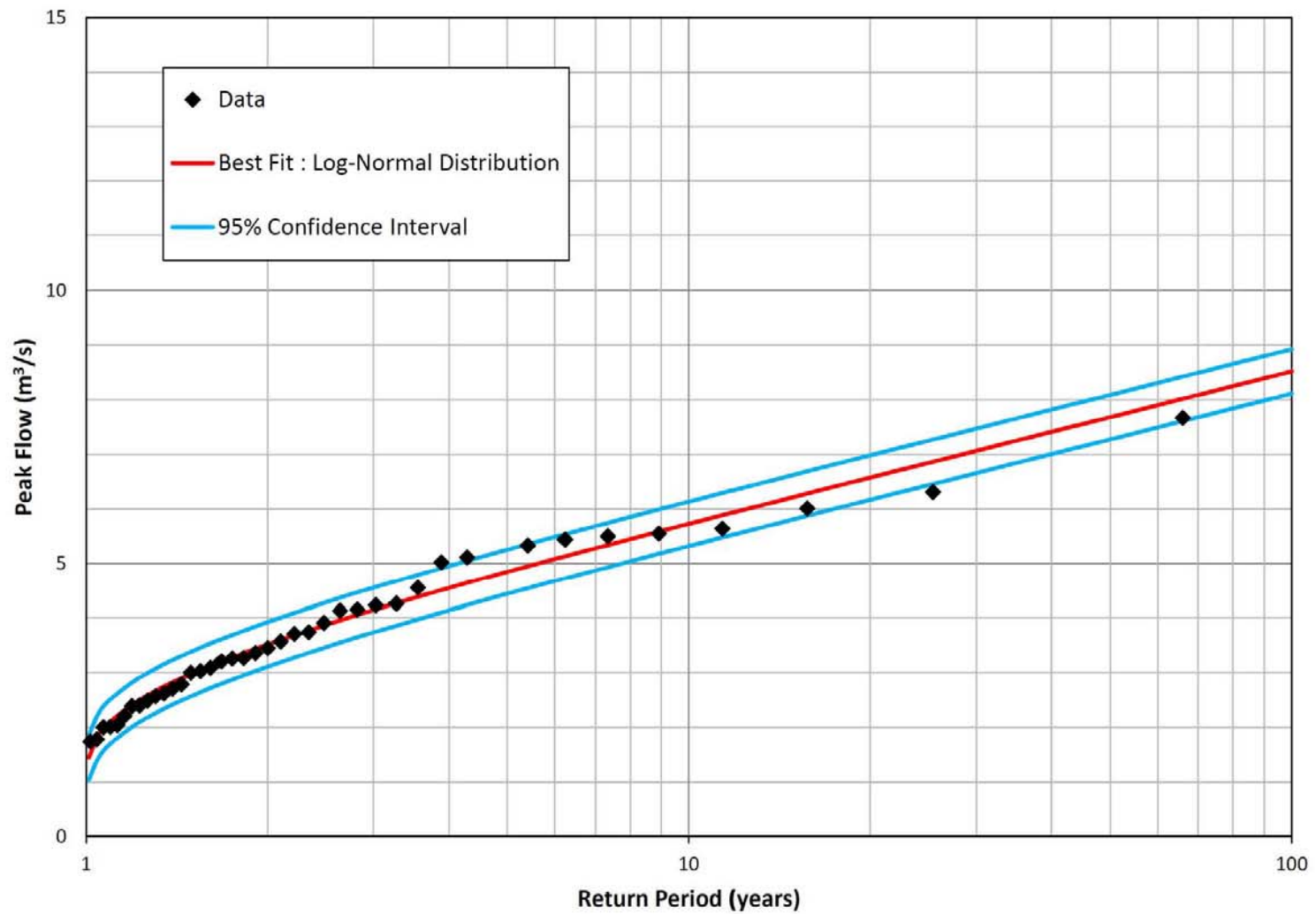


Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

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Hydrometric Gauge Locations

Figure 5.1

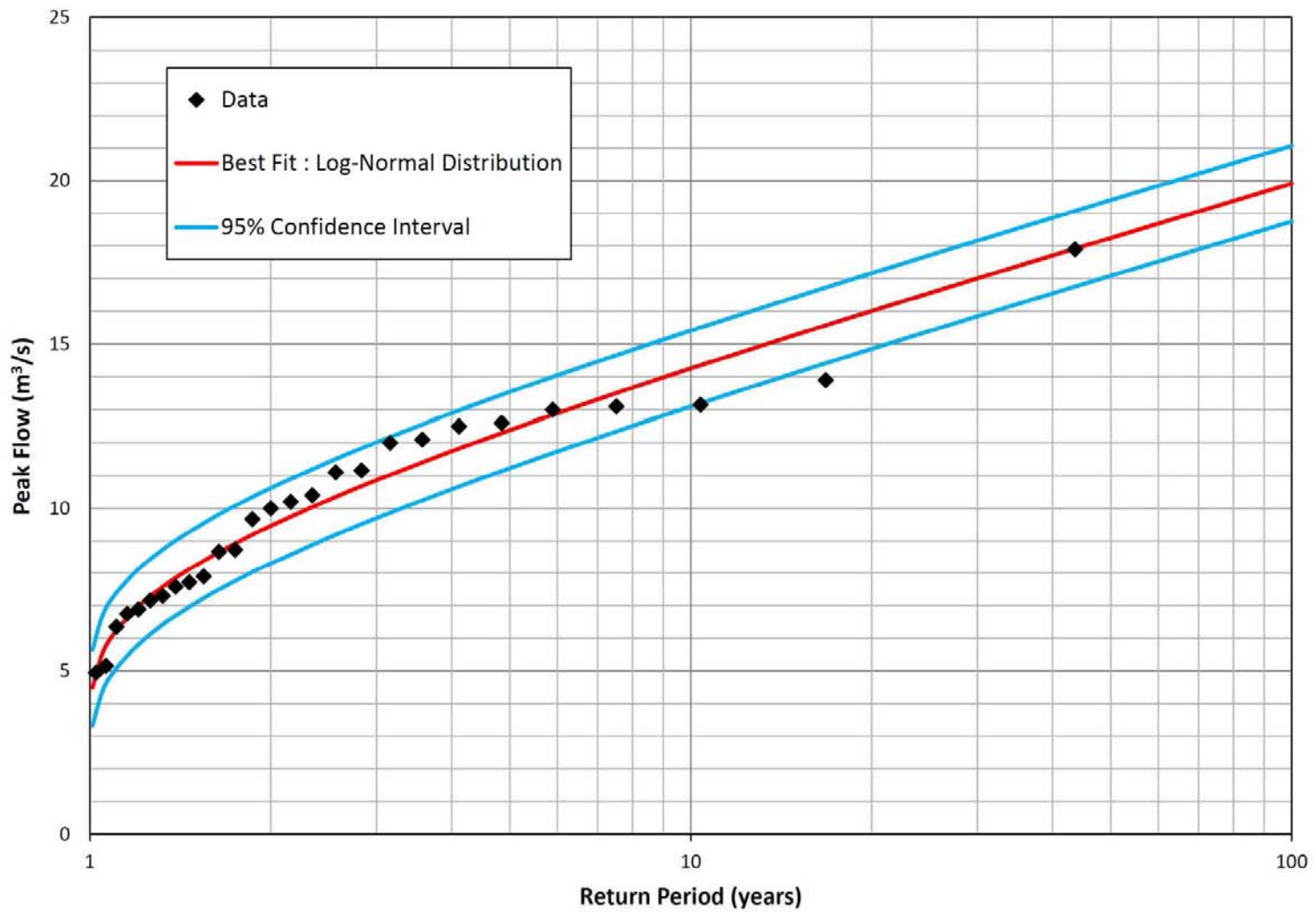


Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

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Northeast Pond River at Northeast Pond – Frequency Analysis

Figure 5.2

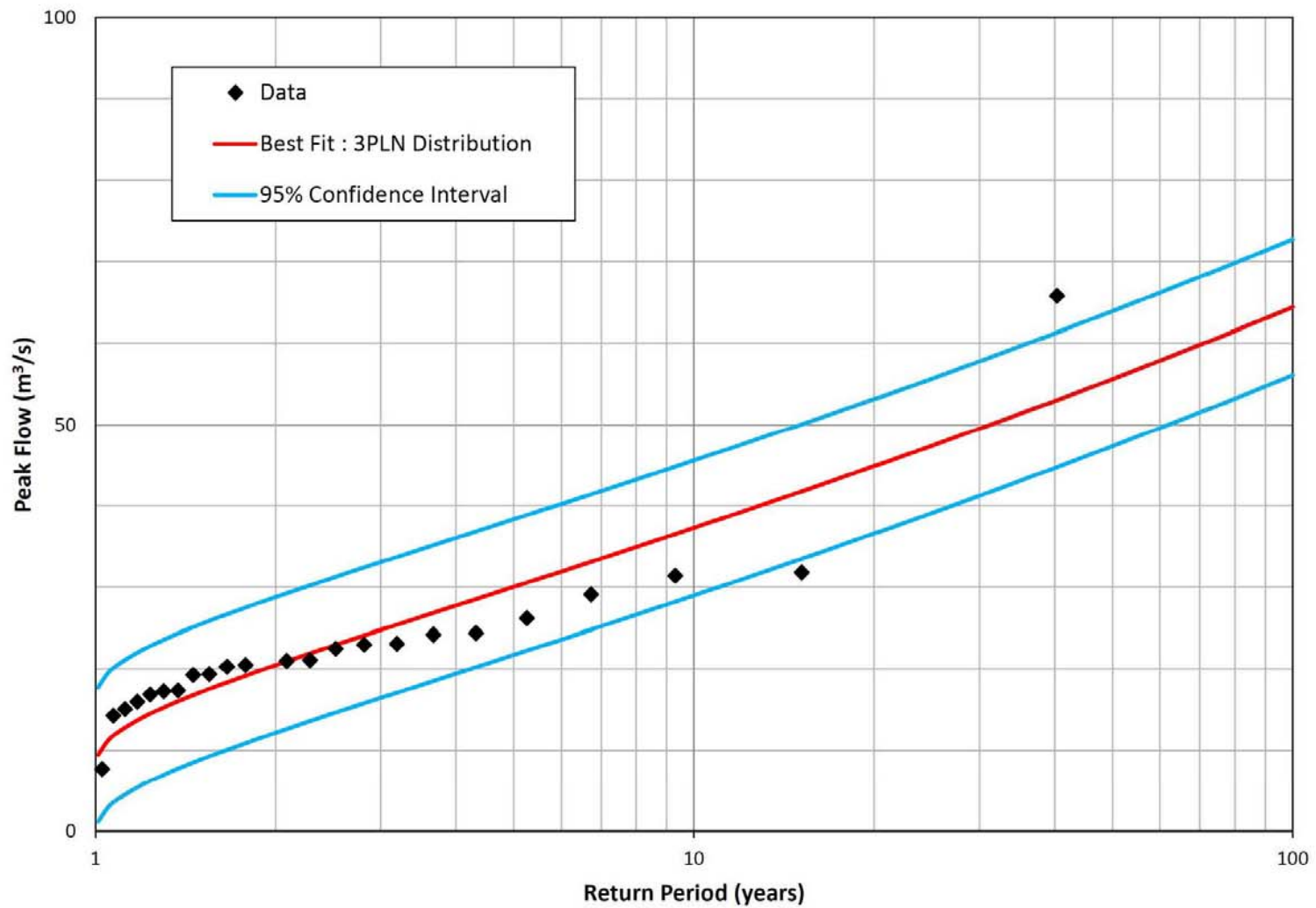


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Virginia River at Pleasantville – Frequency Analysis

Figure 5.3

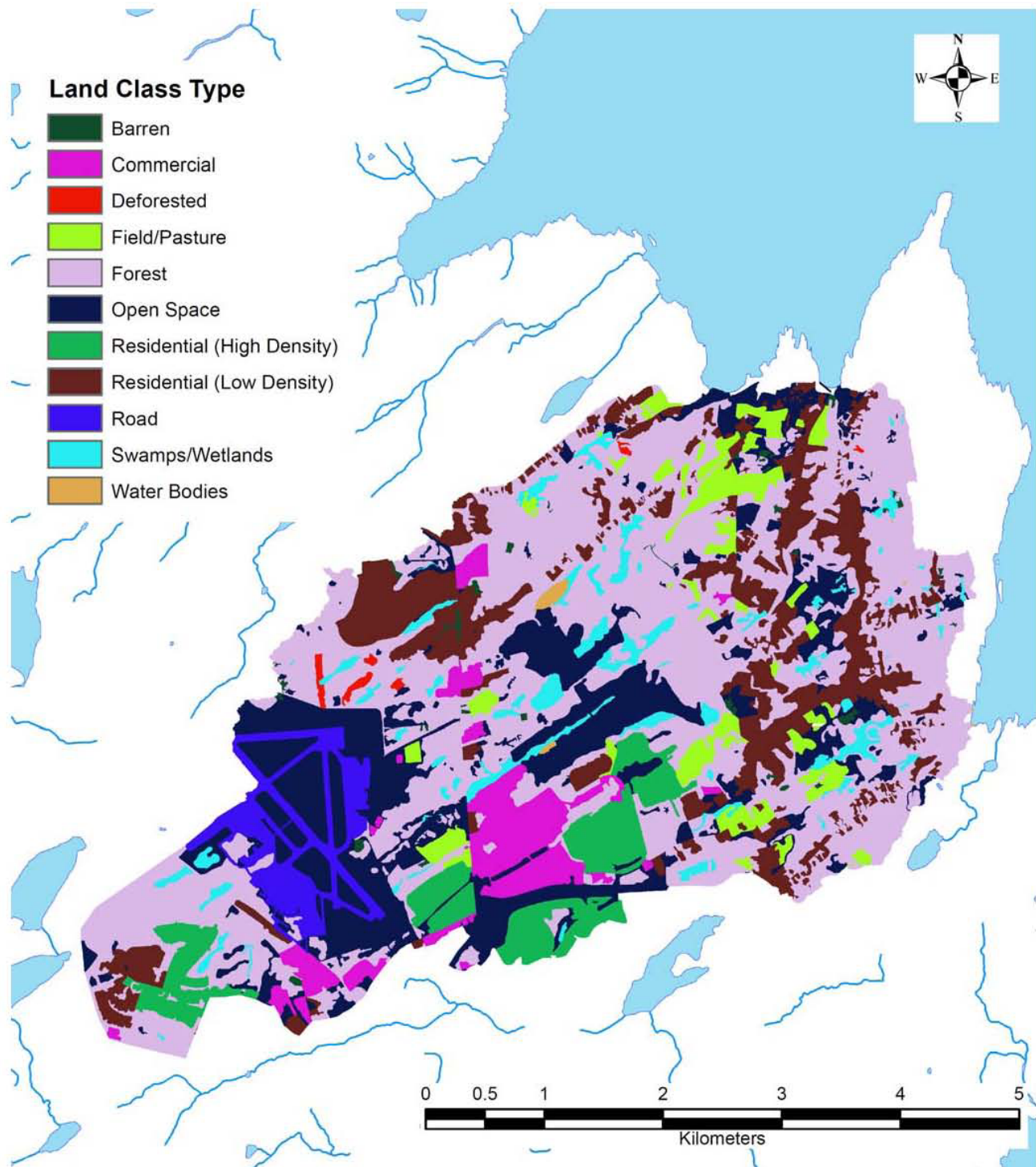


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Leary Brook at Prince Philip Drive – Frequency Analysis

Figure 5.4

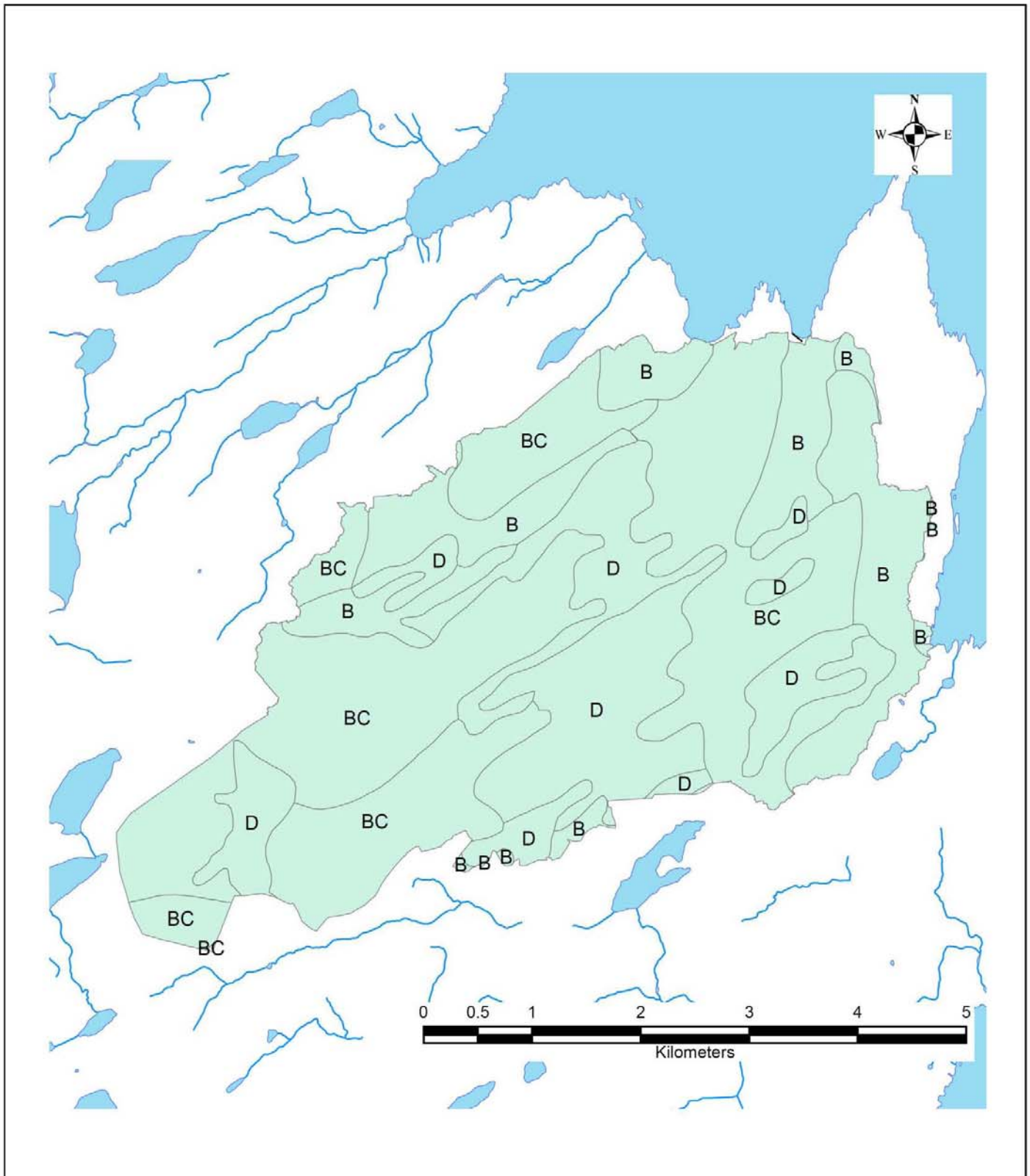



Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

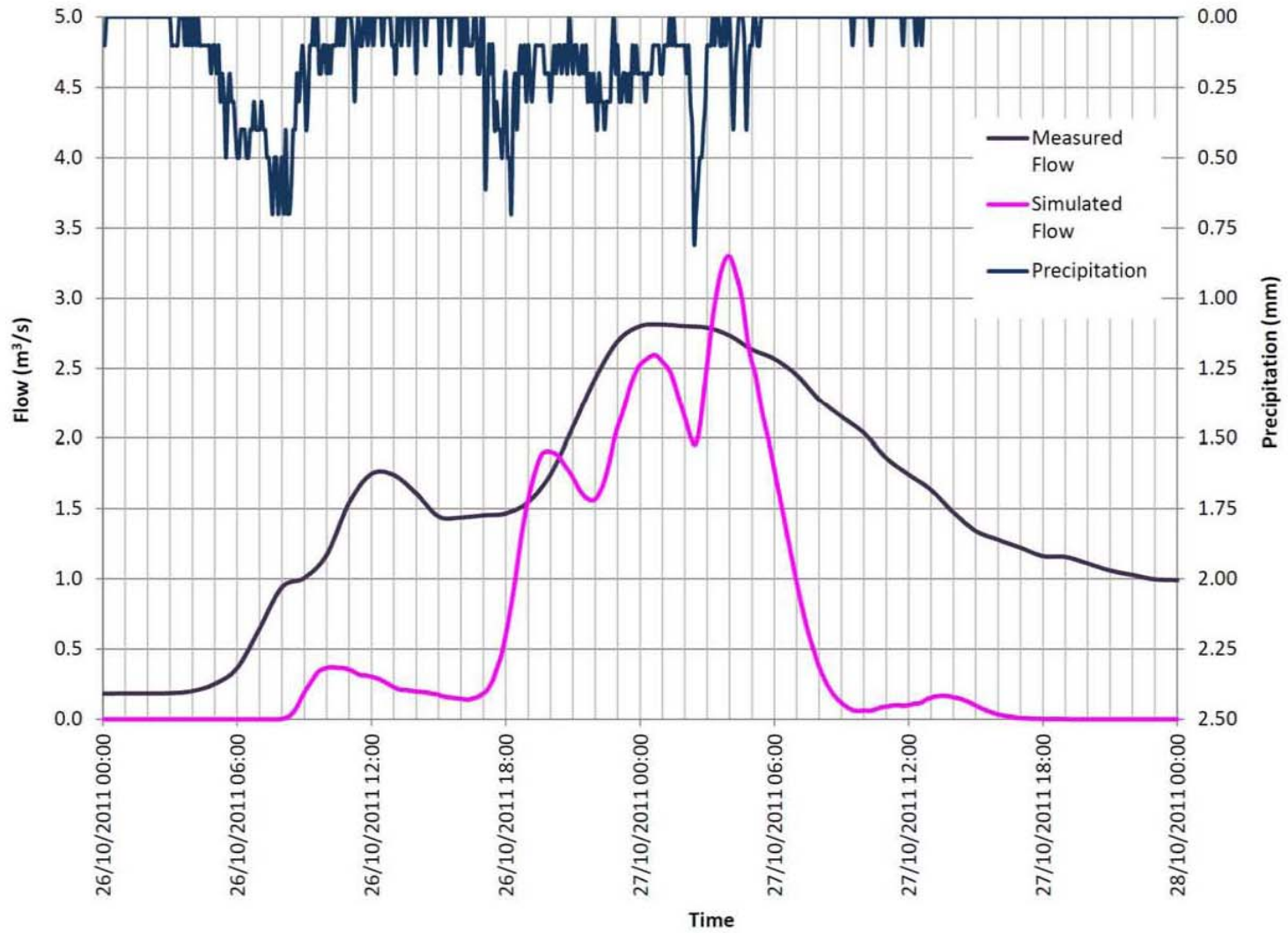
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Land Cover Classification

Figure 5.5



	<p>Town of Logy Bay-Middle Cove-Outer Cove Flood Risk Mapping</p>	<p>Date: June 2012 Proj. #: 113076.00</p>
	<p>Soil Classification</p>	<p>Figure 5.6</p>

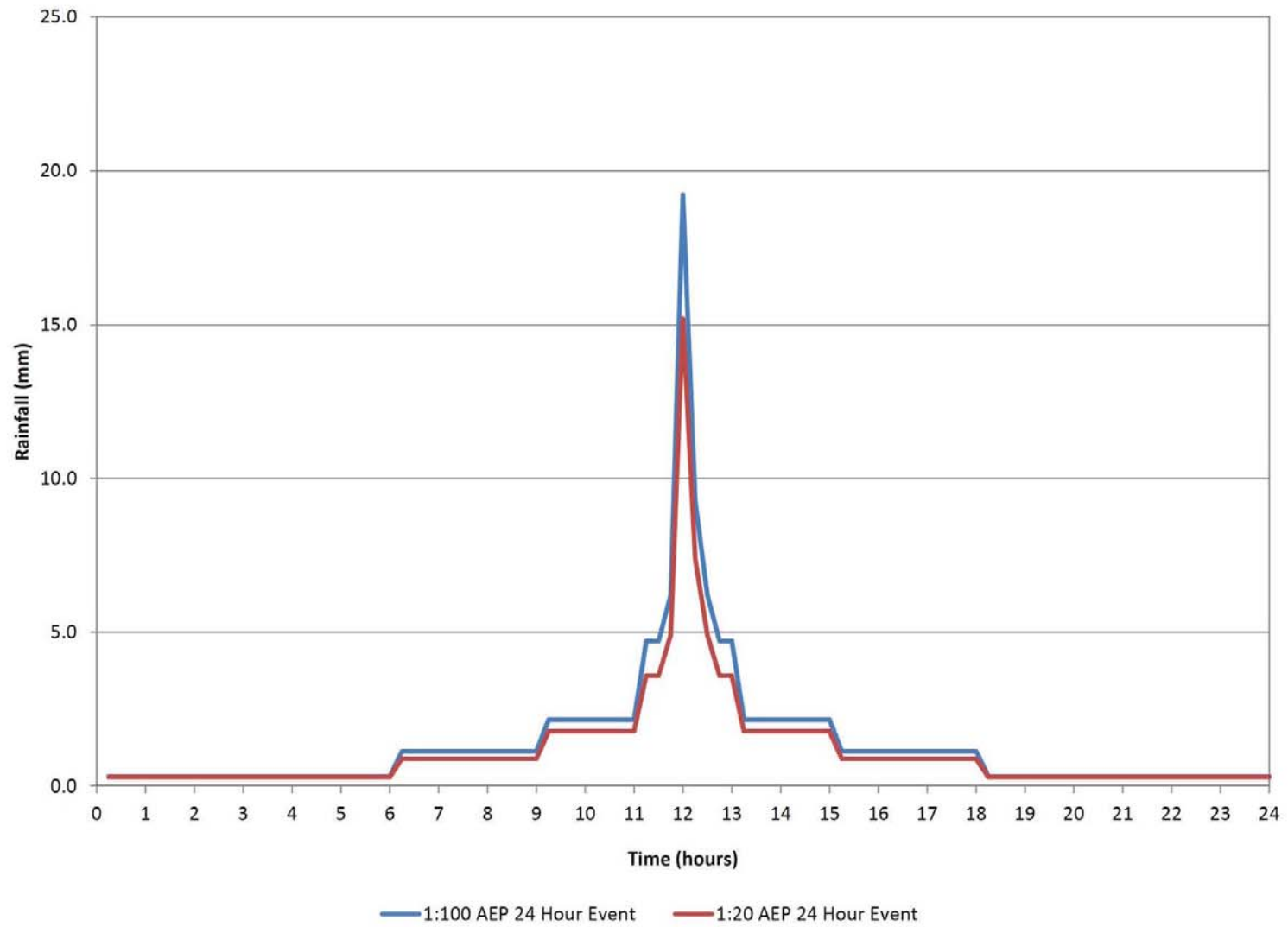


Town of Logy Bay-Middle Cove-Outer Cove
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Outer Cove Brook – Hydrologic Model Calibration at Temporary Hydrometric Gauge

Figure 5.7



Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

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1:20 and 1:100 AEP Rainfall Hyetographs Using Alternating Block Method

Figure 5.8

CHAPTER 6 **HYDRAULIC ANALYSIS**

The purpose of the hydraulic analysis is to translate the 1:20 and 1:100 AEP flood flows, estimated during the hydrologic analysis, into water levels that are used to create flood risk maps.

Hydraulic modeling was carried out using the Hydrologic Engineering Center's River Analysis System (HEC-RAS) and its geospatial modelling extension (HEC-geoRAS). HEC-RAS provides open channel solutions for one-dimensional steady and unsteady flow hydraulics. The basic steady flow computational procedure follows the solution of a one-dimensional energy equation through an iterative procedure (standard step method). Energy losses attributed to channel roughness are estimated by Manning's equation and contraction and expansion losses are estimated as a function of the rate of change in velocity head. HEC-RAS computations account for energy losses associated with common channel obstructions (ex. bridges, culverts, and weirs) and facilitates horizontal and vertical variations in channel roughness at each river cross section. HEC-geoRAS facilitates efficient model construction as well as floodplain mapping, through GIS.

Four models were developed for this study, one for Kennedys Brook, Outer Cove Brook, Coakers River and Drukens River. The extent of each model was contained within the town boundary.

6.1 Model Development

6.1.1 Cross Sections

As discussed in Section 3, cross sections were surveyed along each river reach. An engineer from CBCL walked each river, marking locations for cross sections to be surveyed. The following numbers of cross sections were surveyed:

- 46 on Kennedys Brook
- 45 on Outer Cove Brook
- 44 on Coakers River and
- 24 on Drukens River

Each cross section includes channel elevations below the water level as well as several elevation points in the channel overbanks. This information was combined with the LiDAR data and HEC-geoRAS was used to assemble cross sections for import to the HEC-RAS model.

Plots of each cross section are included in Appendix K.

6.1.2 Structures

Several structures, located along each reach, were also entered in the hydraulic models. These structures are listed in the following table. The additional data required to effectively model the structures was collected during the field investigations. Hydraulic structure data sheets including photos and a description of each structure are provided in Appendix B.

TABLE 6-1 STRUCTURES LOCATED ON MAIN RIVER REACHES

Reach	Structure	Hydraulic Structure Data Sheet
Kennedys Brook	Culvert near 167 Middle Cove Road	1
	Pine River Road Bridge	2
	Culvert on Marine Drive near the Entrance to Middle Cove Park	3
Outer Cove Brook	New Bridge on Marine Drive - Entrance to Outer Cove Beach	4
	MacDonald's Road Bridge	16
	Savage Creek Bridge	19
	Culverts on Access Road to Clovelly Golf Course – Near Clubhouse	23
	Osprey Golf Course - Hole 9 Bridge	24
	Osprey Golf Course – Hole 11 Bridge	25
Coakers River	Ashkay Drive Bridge	23
	Culverts on Logy Bay Road – South of Murphy's Lane	30
	Culvert on Dairy Farm Gravel Road	31
	Culvert on Marine Drive near 320 Marine Drive	32
	320 Marine Drive Driveway Bridge	33
	Culverts at 280 Marine Drive	34
	Culverts on Gravel Road East of 280 Marine Drive	35
Drukens River	Culverts on Logy Bay Road South of Kinsella's Lane	36
	Culvert at 250 Marine Drive	37

6.1.3 Manning's Roughness Coefficient

As noted above, energy losses at each cross section are calculated by the model using Manning's roughness coefficients (Manning's n) for the channel and overbanks. This is perhaps the most sensitive parameter input in the hydraulic model. During the field investigations, photos and notes were taken to aid the modeller with selecting appropriate Manning's n values. Literature values for Manning's n for channels and flood plains are listed in Table 6-2².

² Chow, V.T. 1959. *Open Channel Hydraulics*. McGraw-Hill, New York.

TABLE 6-2 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

6.1.4 Contraction and Expansion Coefficients

Energy losses due to changes between two cross sections are calculated using contraction and expansion coefficients. Where there are minor, or gradual, changes between cross sections typical values of contraction and expansion coefficients are 0.1 and 0.3, respectively. At a bridge or culvert, the change in effective cross section areas is generally abrupt and the contraction and expansion coefficients are 0.3 and 0.5, respectively. It was not necessary to adjust the contraction or expansion coefficients in the models.

6.2 Calibration

Calibration of a HEC-RAS model is achieved by simulating a recorded flow value and comparing the output water levels to measured water levels.

Calibration data was collected on two separate occasions, on October 19th, and October 27th, 2011. The data collection consisted of flow measurements taken near the outlet of each of the four rivers by an Environment Canada technician, while a CBCL employee surveyed water levels at various locations along the rivers.

The measured flows were simulated in the HEC-RAS models and adjustments were made to the Manning’s n values 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.

6.3 Simulated Flood Flows

Flood flows for the 1:20 and 1:100 AEP events were extracted from the HEC-HMS models at various locations along each river reach. These flows, presented in the following tables, were input in the hydraulic models to simulated water surface profiles to be used to create flood risk maps.

TABLE 6-3 KENNEDYS BROOK FLOOD FLOWS

Flow Change Location (X-Section)	Flows (m ³ /s)			
	Existing Q20	Existing Q100	Ultimate Q20	Ultimate Q100
KB46	4.64	7.22	7.05	10.27
KB43	5.15	8.08	7.81	11.36
KB36	6.36	10.03	9.06	13.33
KB26	10.94	16.90	14.81	21.65
KB20	11.26	17.47	15.27	22.46
KB10	12.41	19.49	17.10	25.31
KB5	12.55	19.72	17.29	25.62

TABLE 6-4 OUTER COVER BROOK FLOOD FLOWS

Flow Change Location (X-Section)	Flows (m ³ /s)			
	Existing Q20	Existing Q100	Ultimate Q20	Ultimate Q100
OCB45	12.22	18.545	18.145	25.681
OCB43	12.33	18.691	18.270	25.902
OCB40	12.43	18.83	18.39	26.13
OCB38	15.16	23.02	22.23	31.69
OCB32	17.40	26.37	25.57	36.30
OCB28	22.97	34.47	33.37	47.12
OCB17	23.79	35.86	34.80	49.24
OCB14	24.51	37.15	35.91	50.94
OCB13	25.62	39.23	37.47	53.46
OCB9	25.84	39.67	37.88	54.18
OCB4	26.99	41.63	39.52	56.71
4SP	5.41	7.85	7.60	10.43

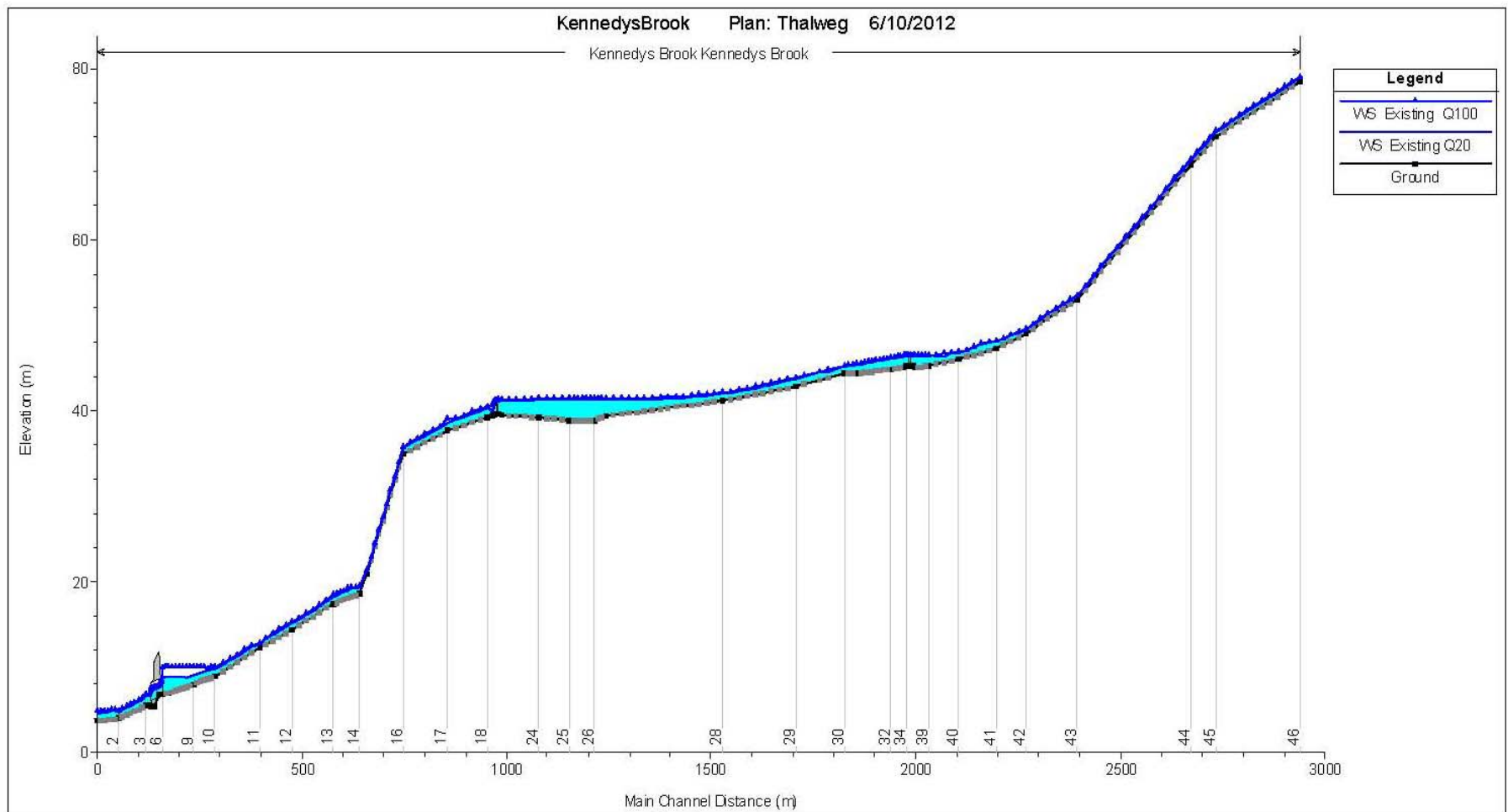
TABLE 6-5 COAKERS RIVER FLOOD FLOWS

Flow Change Location (X-Section)	Flows (m ³ /s)			
	Existing Q20	Existing Q100	Ultimate Q20	Ultimate Q100
CR44	5.20	7.27	5.93	8.08
CR42	5.75	8.11	6.48	8.92
CR36	7.14	10.17	7.99	11.17
CR28	7.24	10.32	8.09	11.35
CR27	7.55	11.32	8.33	12.28
CR19	7.56	11.32	8.33	12.29
CR13	8.31	12.71	9.28	13.94
CR5	8.32	12.74	9.29	13.97
CR2	8.41	12.92	9.46	14.27

TABLE 6-6 DRUKENS RIVER FLOOD FLOWS

Flow Change Location (X-Section)	Flows (m ³ /s)			
	Existing Q20	Existing Q100	Ultimate Q20	Ultimate Q100
DR24	1.95	2.93	2.47	3.55
DR18	3.87	6.18	5.09	7.69
DR9	7.74	12.04	9.51	14.21
DR4	7.95	12.49	9.82	14.81

The water levels resulting from the various flood flow scenarios were used in the creation of the flood risk maps presented in Section 10. Figures 6.1 to 6.4 illustrate the simulated 1:20 and 1:100 AEP water surface profiles, for existing conditions, along Kennedys Brook, Outer Cove Brook, Coakers River and Drukens River, respectively. Appendix L contain the water levels at each cross section for the 1:20 and 1:100 AEP flood flows for existing and ultimate development conditions.

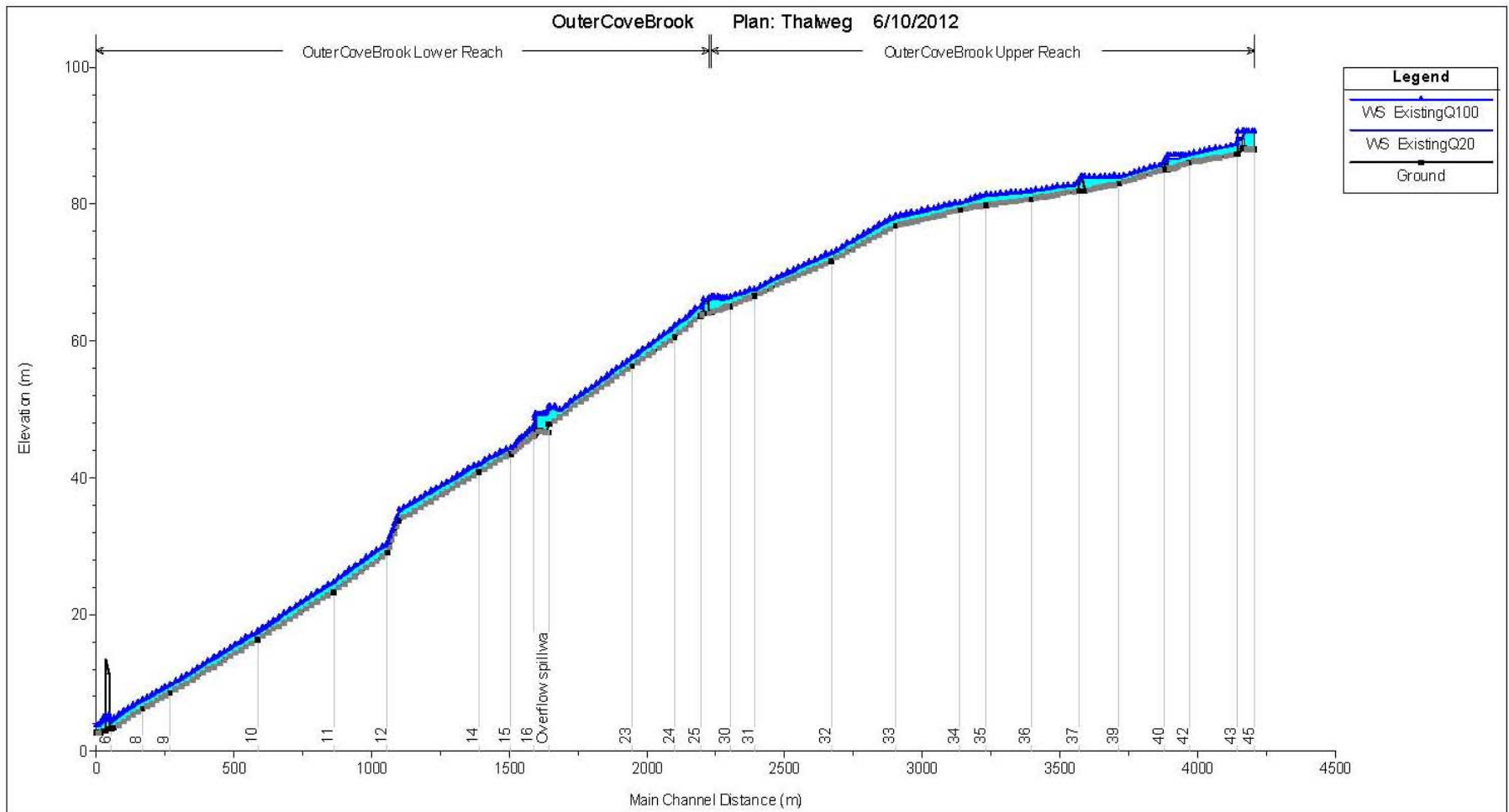


Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

Date: June 2012
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1:20 and 1:100 AEP Water Surface Profiles for Existing Conditions at Kennedys Brook

Figure 6.1

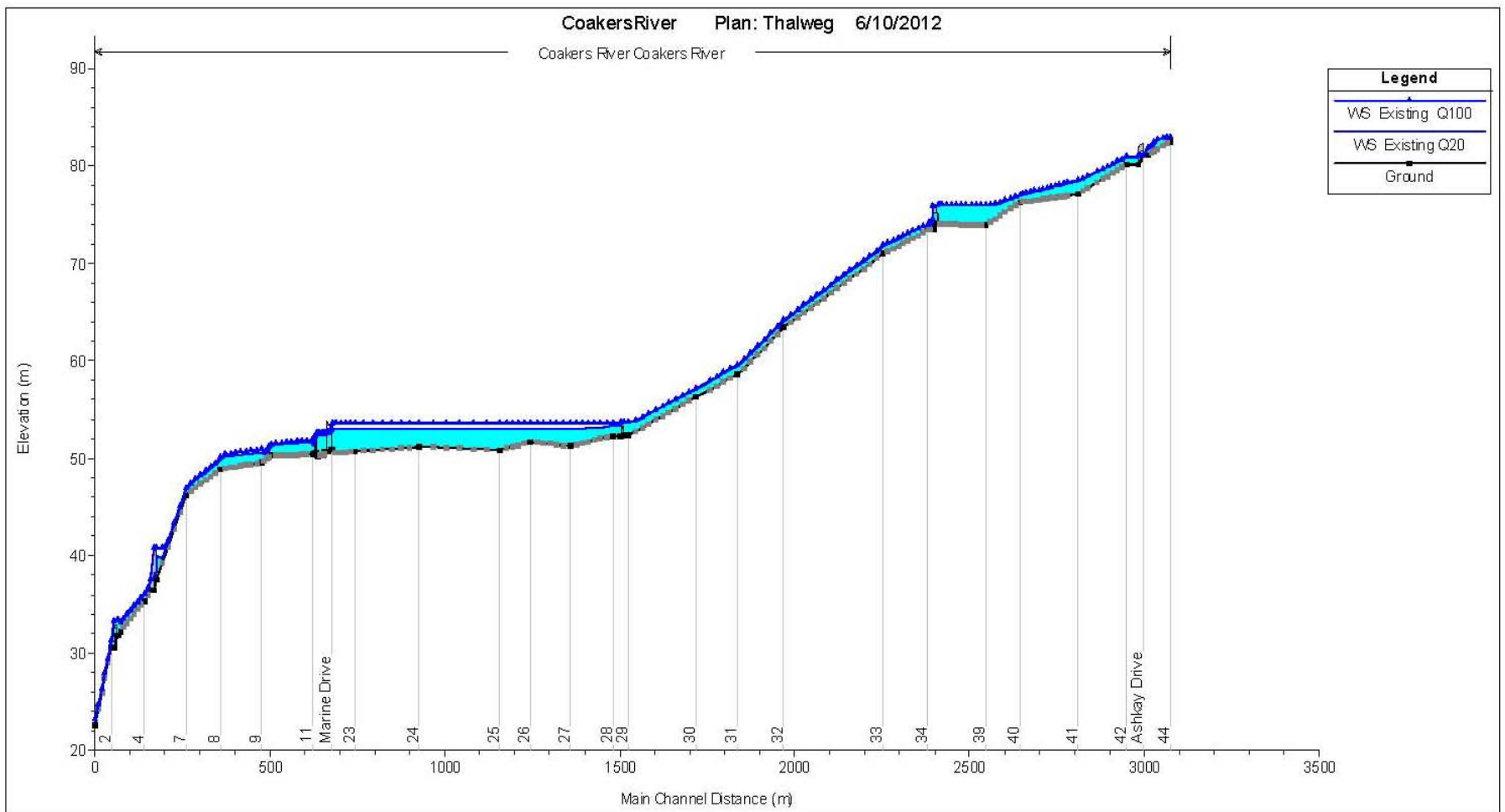


Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

Date: June 2012
Proj. #: 113076.00

1:20 and 1:100 AEP Water Surface Profiles for Existing Conditions at Outer Cove Brook

Figure 6.2

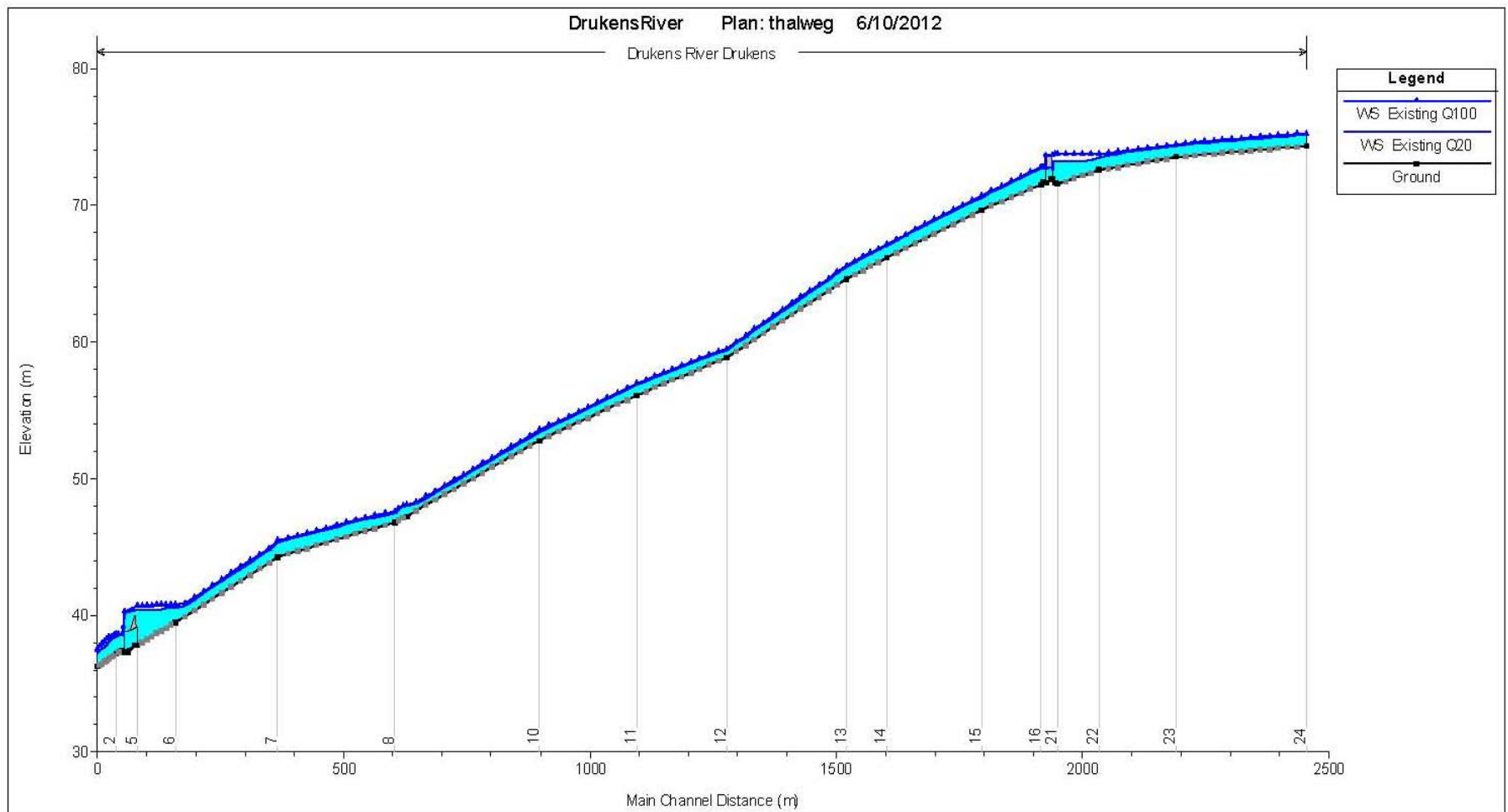


Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

Date: June 2012
Proj. #: 113076.00

1:20 and 1:100 AEP Water Surface Profiles for Existing Conditions at Coakers River

Figure 6.3



Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

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1:20 and 1:100 AEP Water Surface Profiles for Existing Conditions at Drukens River

Figure 6.4

CHAPTER 7 **SENSITIVITY ANALYSIS**

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

7.1 Hydrologic Model Sensitivity

The hydrologic parameters selected for sensitivity analysis include SCS curve numbers, design hyetographs and Manning's roughness values (entered in the river reach elements). The 1:100 AEP event for the existing development conditions was selected as a benchmark to test the sensitivity of the peak flow to the variation of each parameter. The hyetograph and Manning's n values were altered by $\pm 5\%$, 10% and 25%. Altering curve numbers by $\pm 25\%$ produced CN values that were outside the ranges of the literature values; therefore, sensitivity analysis for CN was limited to $\pm 5\%$ and 10%.

Sensitivity analyses were conducted on the Outer Cove Brook hydrologic model only, since it was the only model that could be calibrated. The results indicate that the hydrologic model is most sensitive to changing the curve number values. Increasing the curve numbers by 10% increased flow at the outlet of Outer Cove Brook by 36% (over the base case). Manning's n had the least affect on flow values. A decrease in Manning's n of 25% increased the flow by only 1%. Graphs of the curve number and hyetograph analysis are presented in Figures 7.1 and 7.2.

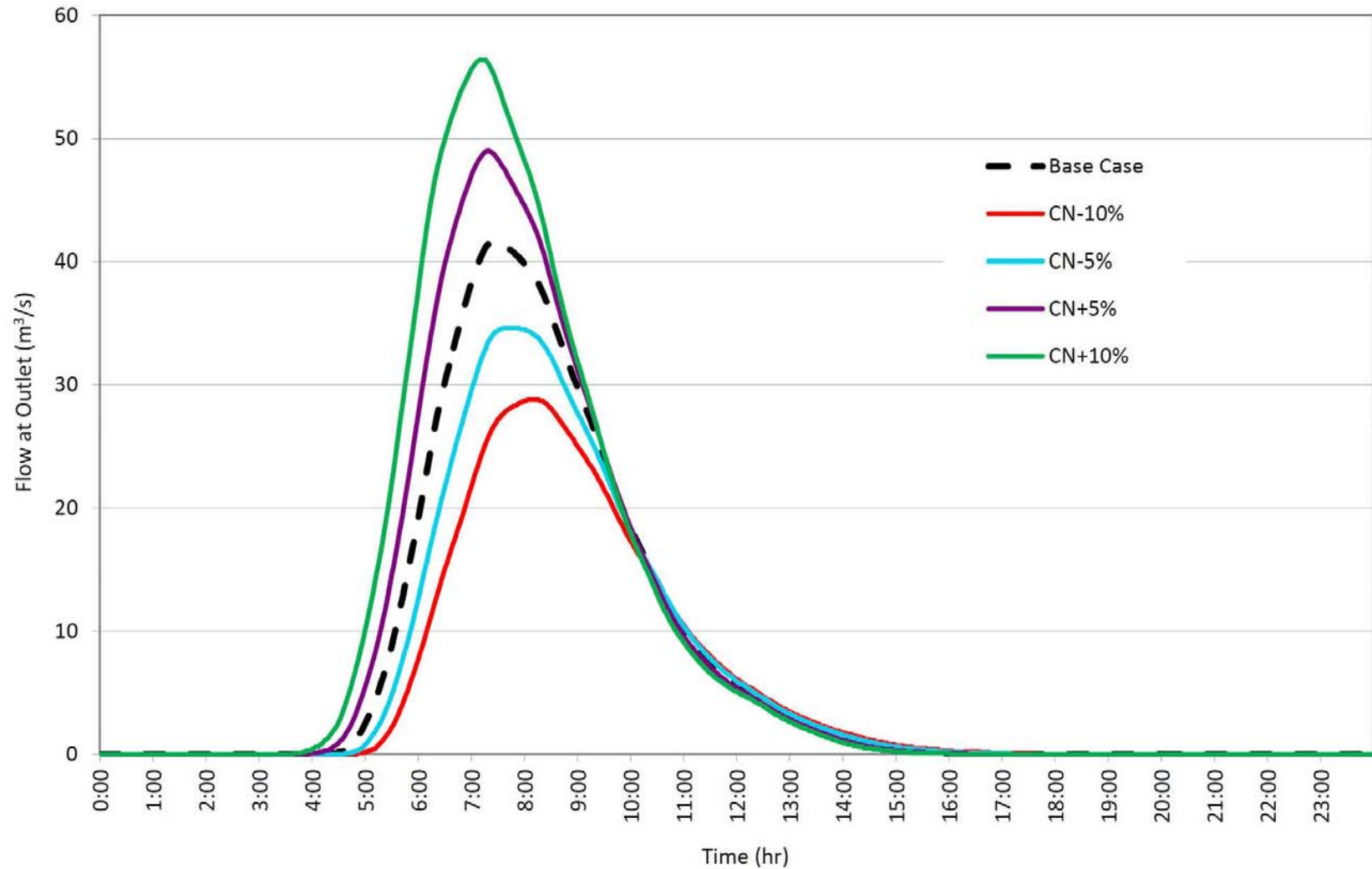
7.2 Hydraulic Model Sensitivity

Sensitivity analyses were conducted on each of the variable hydraulic parameters, including Manning's n value, expansion and contraction loss coefficients, and peak discharge rates. The 1:100 AEP event for the existing development conditions was selected as the benchmark to test the sensitivity of flood levels to the variation of each parameter. Each parameter was varied by $\pm 5\%$, 10% and 25%.

Results of the sensitivity analyses are presented in Appendix M. In general, Manning's n and peak flow rates have the greatest impact on the resulting water surface profiles. Using Outer Cove Brook as an example, decreasing Manning's n by 25% in the HEC-RAS model led to an average decrease in

water levels of approximately 10 cm. Decreasing the peak flow by 25% resulted in an average water level decrease of approximately 16 cm.

Altering the contraction and expansion loss coefficients had a negligible effect on water levels. Increasing the coefficients by 25% led to an increase in the average water level by less than 1 cm. Comparisons of the contraction and expansion loss coefficients were therefore omitted from the sensitivity results presented in Appendix M.

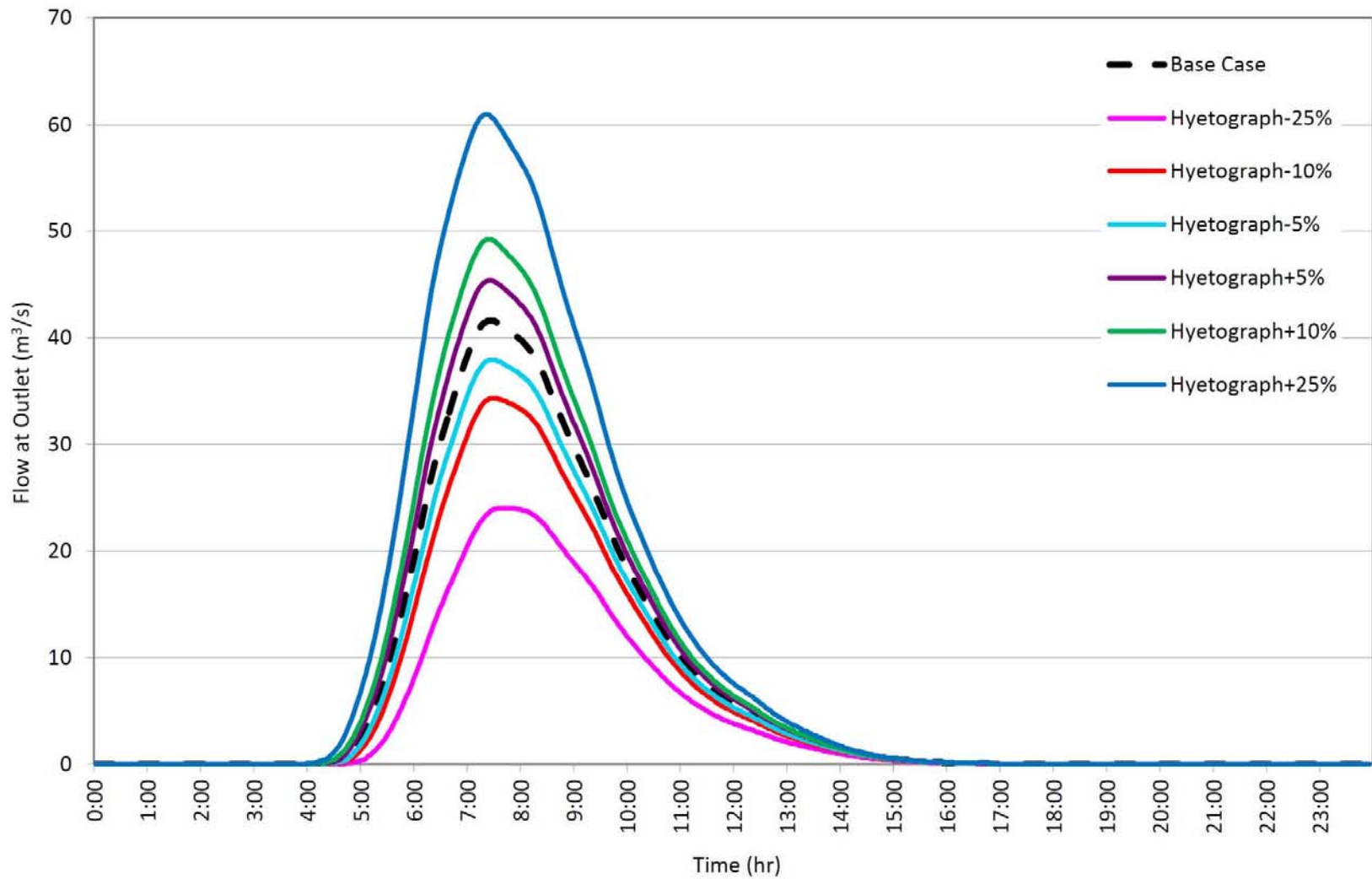


Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

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Outer Cove Brook Hydrologic Model – Curve Number Sensitivity

Figure 7.1



Town of Logy Bay-Middle Cove-Outer Cove
Flood Risk Mapping

Date: June 2012
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Outer Cove Brook Hydrologic Model – Hyetograph Sensitivity

Figure 7.2

CHAPTER 8 HYDRAULIC STRUCTURE ANALYSIS

Flow capacities and current conditions of various hydraulic structures within LB-MC-OC were assessed to estimate their remaining service life. Figure 8.1 illustrates the locations of the structures analyzed. Photos, measurements and notes regarding each structure were collected during the field investigations and are presented in Appendix B.

Flow capacities of structures located on the four main channels were assessed in the HEC-RAS models. Nomographs from the Federal Highway Administration’s Hydraulic Design of Highway Culverts were used to assess the capacity of the remaining structures. Using the available headwater to diameter (HW/D) value for each culvert the theoretical maximum capacity of the culvert is extracted from the nomograph. The maximum capacity was then compared to the return period flows extracted from the HEC-HMS models at the structure location. Table 8-1 presents the results of the capacity analysis. Estimates of remaining structure service life were made through visual inspection of the current structure conditions. Results of this analysis are also presented in the Table 8-1.

TABLE 8-1 SUMMARY OF HYDRAULIC STRUCTURES ANALYSIS

Structure #	Has Adequate Capacity to Pass Flow? (Y/N)				Estimated Remaining Service Life (Physical Condition)
	Q20 existing	Q100 existing	Q20 ultimate	Q100 ultimate	(Years)
1	N	N	N	N	0-5
2	N	N	N	N	10+
3	Y	Y	Y	Y	5-10
4	Y	Y	Y	Y	25+
5	Y	Y	Y	Y	10-15
6	N	N	N	N	0-5
7	Y	Y	Y	Y	10-15
8	Y	N	N	N	0-5
9	Y	Y	Y	Y	5-10
10N	Y	N	N	N	0-5
10S	Y	Y	Y	Y	0-5
11	Y	Y	Y	Y	10-15

TABLE 8-1 SUMMARY OF HYDRAULIC STRUCTURES ANALYSIS (CONTINUED)

12	N	N	N	N	10-15
13	Y	Y	Y	Y	15-20
14	Y	Y	Y	Y	5-10
15	Y	Y	Y	Y	10-15
16	Y	N	N	N	0-10
17E	Y	Y	Y	Y	10-15
17W	Y	Y	Y	Y	0-5
18	Y	Y	Y	Y	15-20
19	N	N	N	N	15+
20	N	N	N	N	25+
21	Y	Y	Y	N	10-15
22N	Y	Y	Y	Y	10-15
22S	Y	Y	Y	Y	0-5
23	Y	N	N	N	0-5
24	Y	N	N	N	10-20
25	Y	N	N	N	10-20
26N	N	N	N	N	10-15
26S	N	N	N	N	5-10
27	Y	Y	Y	Y	10-15
28	Y	Y	Y	Y	5-10
29	Y	Y	Y	Y	25+
30	N	N	N	N	10-15
31	N	N	N	N	5-10
32	Y	Y	Y	Y	15-20
33	Y	N	Y	N	15+
34	Y	N	N	N	0-5
35	N	N	N	N	10-15
36N	Y	N	Y	N	5-10
36S	Y	N	Y	N	25+
37	N	N	N	N	0-5

The structures were also assessed to determine their ability to pass the 1:20 and 1:100 AEP flows for two partial development scenarios. The scenarios included potential development in LB-MC-OC with no further development in St. John's and Torbay, and potential development in St. John's and Torbay with no further development in LB-MC-OC. The areas and types of potential developments were taken from the zoning maps for LB-MC-OC, St. John's and Torbay. The results of the analysis are presented below in Table 8-2.

TABLE: 8-2 SUMMARY OF HYDRAULIC STRUCTURE ANALYSIS FOR PARTIAL DEVELOPMENT SCENARIOS

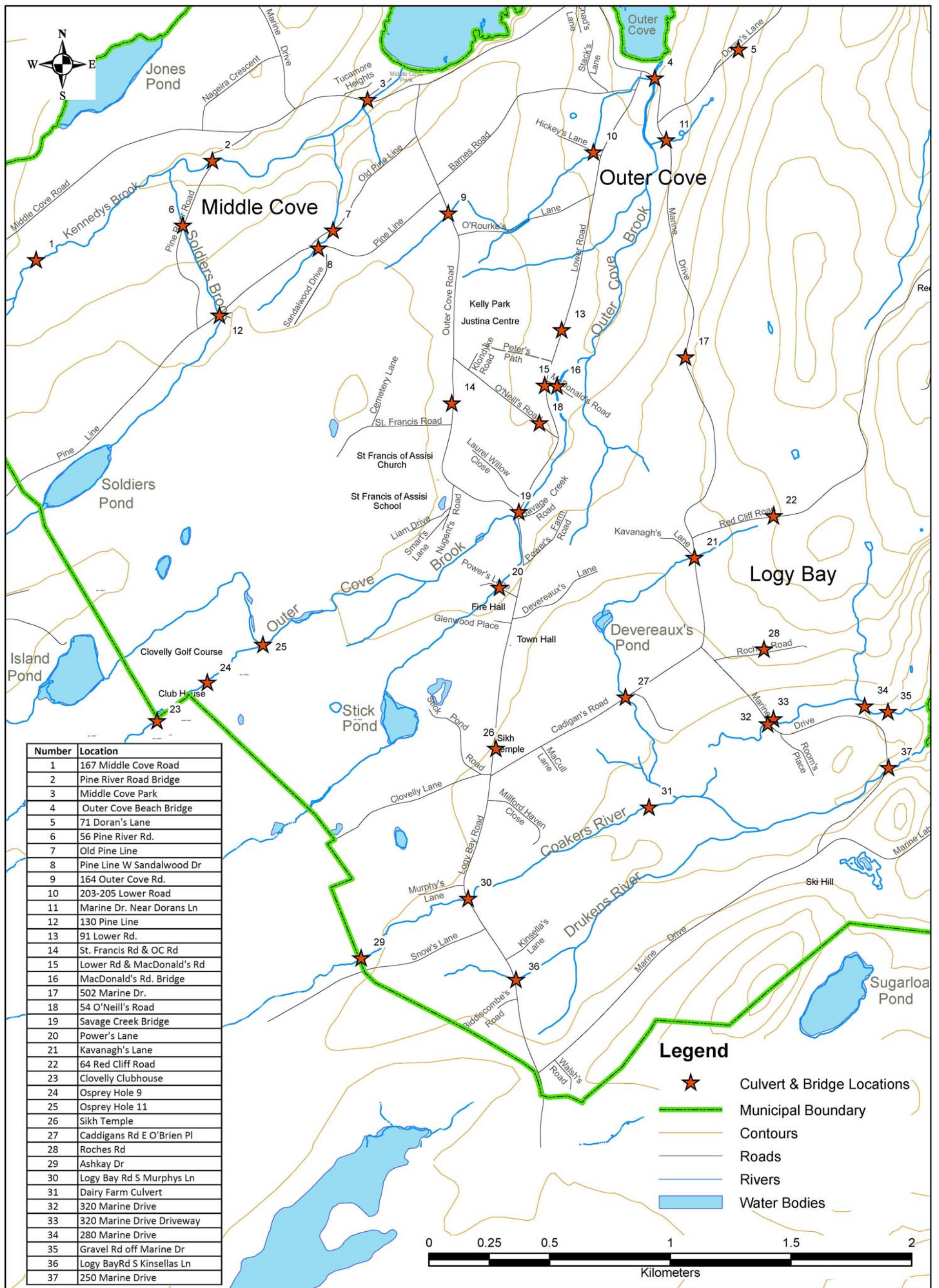
Structure #	Has Adequate Capacity to Pass Flow? (Y/N)			
	LB-MC-OC Development		St. John's/Torbay Development	
	Q20	Q100	Q20	Q100
1	N	N	N	N
2	N	N	N	N
3	Y	Y	Y	Y
4	Y	Y	Y	Y
5	Y	Y	Y	Y
6	N	N	N	N
7	Y	Y	Y	Y
8	N	N	Y	N
9	Y	Y	Y	Y
10N	N	N	N	N
10S	Y	Y	Y	Y
11	Y	Y	Y	Y
12	N	N	N	N
13	Y	Y	Y	Y
14	Y	Y	Y	Y
15	Y	Y	Y	Y
16	N	N	N	N
17E	Y	Y	Y	Y
17W	Y	N	Y	N
18	Y	Y	Y	Y
19	N	N	N	N
20	N	N	N	N
21	Y	N	Y	Y
22N	Y	Y	Y	Y
22S	Y	Y	Y	Y
23	Y	N	N	N
24	N	N	N	N
25	N	N	N	N
26N	N	N	N	N
26S	N	N	N	N
27	Y	Y	Y	Y
28	Y	Y	Y	Y
29	Y	Y	Y	Y
30	N	N	N	N
31	N	N	N	N
32	Y	Y	Y	Y
33	Y	N	Y	N
34	N	N	N	N
35	N	N	N	N

TABLE: 8-2 SUMMARY OF HYDRAULIC STRUCTURE ANALYSIS FOR PARTIAL DEVELOPMENT SCENARIOS (CONTINUED)

36N	Y	N	Y	N
36S	Y	N	Y	N
37	N	N	N	N

A comparison was made of the LB-MC-OC development results in Table 8-2 to the results in Table 8-1, for the Q20 and Q100 existing development. This comparison revealed that six of the structures (structure numbers 8, 10N, 16, 24, 25 and 34) which currently are able to pass the 1:20 AEP flood will no longer have adequate capacity as a result of planned development in LB-MC-OC. Also, structure numbers 17W and 21, which currently can pass the 1:100 AEP flood for existing conditions will have insufficient capacity to pass the 1:100 AEP flood flow resulting from development within LB-MC-OC.

Similarly, the St. John's and Torbay development results in Table 8-2 were compared to the results for the Q20 and Q100 existing development in Table 8-1. This comparison revealed that six of the structures (structure numbers 10N, 16, 23, 24, 25 and 34) which can accommodate the existing 1:20 AEP flood will not have adequate capacity to pass the 1:20 AEP flow resulting from planned development in St. John's and Torbay. Also, structure number 17W will no longer have capacity to pass the 1:100 AEP flood resulting from the St. John's and Torbay planned development.



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Hydraulic Structures Locations

Figure 8.1

CHAPTER 9 CLIMATE CHANGE

To assess the impacts of changing climatic conditions on flood flows and, consequently, the delineated floodplains, several sources of climate change predictions were examined.

One set of predictions came from the ‘Climate Change Scenarios for Atlantic Canada Utilizing a Statistical Downscaling Model Based on Two Global Climate Models’ report, prepared by the Meteorological Service of Canada – Atlantic Region. This report, completed in 2008, examined two Global Climate Models (GCM): the Canadian Coupled General Circulation Model version 2 (CGCM2), and the Hadley Research Center’s Hadley Climate Model version 3 (HadCM3).

Results from simulated changes in 24-hour precipitation amounts generated by two global climate change models were “down-scaled” for the St. John’s rain station by the Environment Canada team. The down-scaled results are presented in the following tables for St. John’s for three future periods centred on the years: 2020, 2050 and 2080 and for return period of 10, 50 and 100 years and are compared to return period values based on historical records.

TABLE 9-1 RETURN PERIOD VALUES FOR 24-HOUR PRECIPITATION (MM) BASED ON DOWNSCALED GLOBAL CLIMATE CHANGE MODEL-CGCM2

Period	10 Years	50 Years	100 Years	Ratio to Historical	Ratio to Updated IDF
HISTORICAL	75.9	92.2	99.1	1.00	-
2020's	113.2	149.2	164.4	1.66	1.21
2050's	118.5	160.3	178.0	1.80	1.31
2080's	107.2	134.5	146.0	1.47	1.07

TABLE 9-2 RETURN PERIOD VALUES FOR 24-HOUR PRECIPITATION (MM) BASED ON DOWNSCALED GLOBAL CLIMATE CHANGE MODEL-HADCM3

Period	10 Years	50 Years	100 Years	Ratio to Historical	Ratio to Updated IDF
HISTORICAL	75.9	92.2	99.1	1.00	-
2020's	103.5	128.6	139.1	1.40	1.02
2050's	139.1	199.0	224.3	2.26	1.65
2080's	110.8	147.1	162.4	1.64	1.19

Downscaled results from both global climate change models are similar in that they predict that extreme values of the 24-hour rainfall amount will increase and then decrease again over the next 70 years. The CGCM2 and HadCM3 models both predict that a maximum value for the 24-hour 1:100 AEP rain volume will be reached in the 2050 tri-decadal period. However, the models differ in the magnitude of the increase. The expected increase over the updated IDF is 1.31 and 1.65 for the CGCM2 and HadCM3 models, respectively.

Rainfall predictions for the 2050 tri-decadal period for the 24-hour 1:20 AEP were estimated by plotting the 10, 50 and 100-year estimates on Gumbel distribution paper and reading off the precipitation amount for the 20-year return period. This analysis is presented in Appendix N. The predicted rainfall amounts for the 2050 period are as follows:

- CGCM2 = 137 mm
- HadCM3 = 165 mm

Relative to the updated IDF curve for the 24-hour 1:20 AEP rainfall of 110 mm these predictions represent increases of 1.25 (CGCM2) and 1.50 (HadCM3).

During a meeting with WRMD on March 14, 2012, an additional set of climate change predictions, prepared by Dr. Joel Finnis, Professor, Department of Geography, Memorial University of Newfoundland, were presented to CBCL to be included in the climate change analysis.

These additional values were derived from high resolution regional climate model (RCM) simulations. Raw model data was used to fit a Bernoulli-gamma probability distribution to daily precipitation totals. The impacts of climate change were then estimated by comparing distribution parameters between 21st and 20th century simulations.

Projected parameter shifts were then applied to parameters fit to observed data, giving an estimated 21st century distribution in which internal RCM biases have been removed. In the current study, RCM were taken from the North American Regional Climate Change Assessment Project (NARCCAP), and historical parameters were fit to the updated IDF curve data provided by CBCL. The projections are valid for the 2041-2070 period. A report and email describing the process used to determine the climate change predictions was provided Dr. Finnis and is included in Appendix O.

The predictions developed for the 2050 tri-decadal period are presented in Table 9-3. This analysis indicates increases over the updated IDF of 1.09 and 1.11 for the 1:20 and 1:100 AEP rainfall amounts, respectively.

TABLE 9-3 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	77.6	1.16
5	94.2	1.10
10	107.1	1.10
20	120.0	1.09
50	137.4	1.10
100	150.6	1.11

Flood flows for the 1:20 and 1:100 AEP were estimated for the available precipitation predictions using the HEC-HMS models discussed in Section 5. The rainfall hyetographs were generated using the alternating block method, described in Section 5. This analysis indicates that the storm duration producing the largest flood flows at Kennedys Brook and Outer Cove Brook is the 12-hour storm and for Coakers River and Drukens River it is the 6-hour storm. Using the alternating block method hyetograph takes these smaller storm durations into account.

These hyetographs were simulated in the HEC-HMS models for the four study watercourses. The resulting 1:20 and 1:100 AEP flood flows at the outlet of each river are presented for comparison, in Table 9-4 and 9-5, respectively.

TABLE 9-4 1:20 AEP FLOWS (M³/S) BASED ON CLIMATE CHANGE ANALYSIS

Basin	1:20 AEP Flow at Basin Outlet			
	Existing Conditions	CGCM2	HadCM3	Dr. Joel Finnis
Kennedys Brook	12.6	21.2	29.5	16.2
Outer Cove Brook	27.0	44.2	60.6	34.4
Coakers River	8.4	16.8	22.8	13.1
Drukens River	7.9	16.6	22.6	13.2

TABLE 9-5 1:100 AEP FLOWS (M³/S) BASED ON CLIMATE CHANGE ANALYSIS

Basin	1:100 AEP Flow at Basin Outlet			
	Existing Conditions	CGCM2	HadCM3	Dr. Joel Finnis
Kennedys Brook	19.7	34.7	50.4	26.0
Outer Cove Brook	41.6	70.9	101.6	53.7
Coakers River	12.9	26.4	37.4	20.2
Drukens River	12.5	26.4	37.5	19.9

The main objective of the climate change analysis is to create flood risk maps for each river. During a meeting with WRMD on May 14, 2012, WRMD indicated that they have more confidence in the climate change precipitation estimates developed by Dr. Joel Finnis, and that the department intends to use Dr. Finnis' work in future studies. For the purpose of this study, the final flood risk maps developed for the climate change were determined using the results from Dr. Joel Finnis' work. Appendix L contain the water levels at each cross section for the 1:20 and 1:100 AEP climate change flows.

CHAPTER 10 FLOOD RISK MAPPING

The water levels generated by the HEC-RAS models for the 1:20 and 1:100 AEP flood events were used to develop flood risk maps for the four rivers. Flood lines for the existing, ultimate development and climate change conditions were created.

HEC-geoRAS was used to create the flood risk maps in GIS. Floodplain maps are produced in HEC-geoRAS by intersecting the water surface, generated by HEC-RAS, with the terrain surface (LiDAR data for this study). The HEC-geoRAS smooth floodplain delineation tool was used to remove jagged edges. Further post-processing of the floodplain polygons was completed to fill in any gaps in the data, and to ensure that the flood lines reflect unique field conditions, including the routing of flood lines around existing structures.

Appendix P contains the flood risk maps for the 1:20 and 1:100 AEP flood events for the existing conditions, ultimate development and climate change scenarios for Kennedys Brook, Outer Cove Brook, Coakers River and Drukens River. Overtopping of hydraulic structures is evident on the maps by the floodplain polygon covering the structure. For structures that are not overtopped, the floodplain polygon is split so that the top of the structure (road or bridge deck) is visible. Table 10-1 presents the percent increase in flooded areas when comparing the ultimate development and climate change floodplains to the existing condition floodplains for the 1:20 and 1:100 AEP flood events.

TABLE: 10-1 PERCENT INCREASE IN FLOODED AREA

Reach	Percent Increase in Flooded Area (%)					
	Q20			Q100		
	Existing	Ultimate	Climate Change	Existing	Ultimate	Climate Change
Kennedys Brook	-	15.7	15.4	-	11.7	12.9
Outer Cove Brook	-	20.7	17.6	-	11.7	11.1
Coakers River	-	4.9	14.8	-	2.9	6.9
Drukens River	-	13.5	26.3	-	4.5	13.6

The flood risk maps, located in Appendix P, show that several homes located near Coakers River, specifically near the culvert crossing Marine Drive and along Rooms Place, are located in, or very near, the floodplain for each flood scenario examined. A field survey was carried out by CBCL to estimate the basement floor elevation for each of the dwellings located in the floodplain. Table 10-2 presents the results of the survey. The elevations given for 8 Rooms Place and 320 Marine Drive are the estimated finished floor elevations since these homes do not have basements.

TABLE: 10-2 ESTIMATED BASEMENT FLOOR ELEVATIONS

Residence	Estimated Basement Elevation (m)
17 Rooms Place	53.39
14 Rooms Place	52.99
10 Rooms Place	51.87
8 Rooms Place	53.34
2 Rooms Place	52.68
320 Marine Drive	52.70

Field verification of the flood risk maps was completed by speaking with Town Manager, Ms. Carruthers, and some local residents. The caretaker for 250 Marine Drive revealed to Mr. Michael Colbert and Mr. Ali Khan of WRMD, that the culvert crossing Drukens River near the entrance to 250 Marine Drive had overtopped within the past 10 – 15 years. Mr. Berkshire of 14 Rooms Place told CBCL employees that he has not experience any problems with flooding in the past; however, he mentioned that the adjacent home, 10 Rooms Place, has had past flooding problems which required the use of a sump pump. During the cross section survey, Ms. Noseworthy of 320 Marine Drive indicated that the home has not experienced any recent flooding, however she did point to a past flood line near the fire pit, approximately 21 m from the home. Discussions with Town Manager, Ms. Carruthers, revealed that the bridge crossing Outer Cove Brook, at the intersection of Logy Bay Road, Lower Road and Outer Cove Road, overtopped with water flowing along Lower Road during a rainstorm in November of 2008. Ms. Carruthers also indicated that the town has had issues with water overtopping the culvert crossing Logy Bay Road on Coakers River, north of Snows Lane.

CHAPTER 11 INUNDATION MAPPING

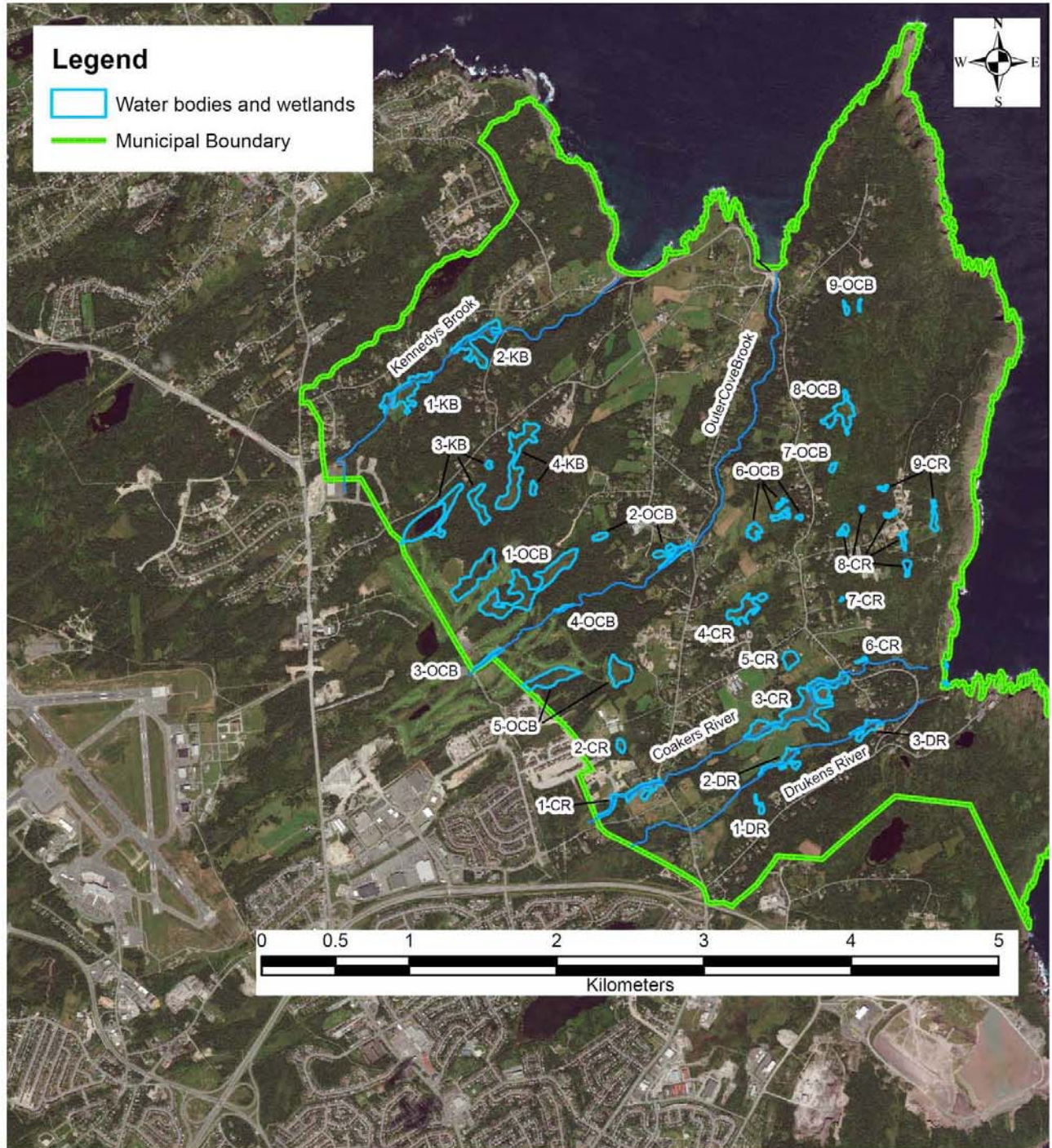
Inundation maps are used to illustrate the depth of water within the floodplain extents for a given flood event. HEC-geoRAS produces inundation maps by intersecting a water surface profile with the terrain data and then creating a depth grid where the water surface is higher than the terrain. HEC-geoRAS was used to create inundation maps for the 1:20 and 1:100 AEP flood events for existing conditions in GIS. These maps are presented in Appendix Q.

The creation of inundation maps requires detailed bathymetric (below water level) data to calculate water depth. For this study, bathymetric data has only been collected at the location of each cross section. Therefore, the inundation depth data can only be considered reliable at each cross section. The depth data shown between cross sections is produced by HEC-RAS interpolating between the cross sections. Therefore, these maps should be used with caution.

CHAPTER 12 ENVIRONMENTALLY SENSITIVE AREAS

For the purposes of this report, environmentally sensitive areas include wetlands and water detention bodies, including ponds. The environmentally sensitive areas identified by AYFC are shown on Figure 12.1. The areas have been labeled using a scheme that identifies the drainage area each is contained in, and groups the smaller/closely spaced areas. Wetlands and water bodies are located throughout LB-MC-OC, and, at some locations, coincide with floodplains.

LB-MC-OC has expressed interest in reserving some of the environmentally sensitive areas as conservation areas, and may wish to consider reserving some of the areas located outside of the floodplain limits.



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Environmentally Sensitive Areas

Figure 12.1

CHAPTER 13 CONCLUSIONS

The following are a list of key conclusions determined from this study.

1. Historical flooding
 - Through discussions with LB-MC-OC and a thorough review of background information notable past flood events which resulted in hydraulic structures meeting or exceeding their capacities were noted.
2. Data collection
 - A total of 159 cross sections were surveyed for this study; 46 cross sections were surveyed on Outer Cove Brook, 45 sections Kennedys Brook, 44 cross sections on Coakers River and 24 cross sections on Drukens River.
 - Two sets of calibration data were collected for each river. The calibration data consisted of water surface profiles and flow measurements during normal flow conditions and one high flow event.
 - Twenty-two hydraulic structures were measured and photographed.
 - LiDAR data was collected.
3. IDF update
 - The IDF curves for the St. John's Airport climate station were updated as part of this study. Additional data was obtained from the Windsor Lake rain gauge which is owned and operated by the City of St. John's. Annual maximums for 5, 10, 15 and 30 minute and 1, 2, 6, 12 and 24 hour intervals were extracted from the Windsor Lake gauge data and combined with the St. John's Airport gauge data. Statistical analysis was performed on each of the 8 data sets to update the IDF curves.
 - A comparison of the updated curves to the 1996 curves indicated slight decreases in rainfall intensities for shorter duration storms and significant increases in rainfall intensities for longer duration storms.
4. Hydrologic Analysis
 - Hydrologic analysis was conducted using both statistical and deterministic approaches.
 - The statistical analysis was performed using prorated single station frequency analysis and regional flood frequency analysis. Flood estimates determined using both of these statistical techniques should be used with caution; single station frequency analysis results are influenced by the physical factors in the gauged basin that the study basin

may not experience and many of the parameters calculated for the regional flood frequency analysis were outside the acceptable range or near the extreme.

- The deterministic analysis was performed using the modelling software HEC-HMS. The model developed for Outer Cove Brook was calibrated using 5-minute rainfall data and hourly flow measurements. The other rivers could not be calibrated due to lack of flow data.
- 1:20 and 1:100 AEP flood estimates were determined for each river for the existing and ultimate development conditions.

5. Hydraulic Analysis

- Hydraulic models were created using HEC-RAS for the four rivers.
- The models were calibrated using measured water level and flow data collected during the field program and by adjusting Manning's n values to force the simulated water levels to match the measured water levels.
- The flood flows extracted from the HEC-HMS models for the 1:20 and 1:100 AEP floods were simulated in the calibrated HEC-RAS models to produce water surface profiles, which were used in the creation of the flood risk maps.

6. Sensitivity Analysis

- The hydrologic model developed for Outer Cove Brook was tested for sensitivity to Manning's n, hyetographs and curve number. The 1:100 AEP existing condition result was used as a benchmark to test sensitivity. Results of the analysis indicated the hydrologic model is most sensitive to curve number, while alterations to Manning's n values had the least effect on flow results.
- Parameters selected for sensitivity analysis of the hydraulic models included Manning's n, expansion and contraction coefficients and peak flow. The 1:100 AEP existing condition result was used as a benchmark to test sensitivity of these parameters. Alterations to Manning's n and peak flow had the largest impact on resulting water levels; while expansion and contraction coefficients had a near negligible effect on water levels.

7. Hydraulic Structure Analysis

- Assessments were made of flow capacities and estimated remaining service life of various hydraulic structures.
- Flow scenarios examined included 1:20 and 1:100 AEP flows for existing conditions, ultimate development, planned future development in LB-MC-OC with no future development in St. John's and Torbay, and planned future development in St. John's and Torbay with no future development in LB-MC-OC.

8. Climate Change

- The impacts of changing climate conditions on flood flows and floodplains were assessed.
- Climate change predictions developed by the Meteorological Services of Canada and by Dr. Joel Finnis, Professor, Department of Geography, Memorial University of Newfoundland were assessed.
- The predictions were simulated in the HEC-HMS models to estimate 1:20 and 1:100 AEP flood flows for each of the four rivers.

- The estimates prepared by Dr. Finnis were used to produce flood risk maps for the climate change scenario.

9. Flood Risk Mapping

- Water levels generated by the HEC-RAS models for the 1:20 and 1:100 AEP floods for the existing, ultimate development and climate change conditions were used to develop flood risk maps for the four rivers.
- The flood risk maps were field verified through discussions with local residents and the Town Manager.

10. Inundation Maps

- Inundation maps illustrating the depth of water within the floodplain extents were developed for the 1:20 and 1:100 AEP floods for existing, ultimate development and climate change scenarios.
- These maps should be used with caution since they are developed using bathymetric data and the only reliable areas of such data are at the locations of each surveyed cross section.

11. Environmentally Sensitive Areas

- Environmentally sensitive areas are considered wetlands and water bodies.
- These areas were identified during the land classification exercise.
- The town may want to consider preserving some of these areas.

CHAPTER 14 RECOMMENDATIONS

Key recommendations of the study are presented below.

1. CBCL recommends that LB-MC-OC adopt the ultimate development flood lines for its town plan and development regulations. Appendix L contains tables of water levels at each cross section for the ultimate development scenarios.
2. CBCL recommends that LB-MC-OC encourage Environment Canada to reinstate the measurement of rainfall intensity at the St. John's Airport rain gauge. Rainfall intensity data collected at this location is essential for carrying out flood risk mapping work in LB-MC-OC and surrounding areas.
3. CBCL recommends that WRMD, in cooperation with Environment Canada and LB-OC-MC, continue to operate the hydrometric gauge on Outer Cove Brook. Over the long term, this hydrometric gauge will provide the following benefits:
 - Approximately 20 years of data is required before reliable flood flow estimates can be calculated using statistically based techniques. A gauge on Outer Cove Brook, together with the two gauges that were installed in the upper reaches of the basin, would collect data for use in future studies.
 - Information on future flood flows could be used to verify the current models or for future flood risk mapping work.
4. CBCL recommends that WRMD, in cooperation with Environment Canada, consider installing hydrometric gauges on Kennedys Brook, Coakers River and Drukens River.
5. CBCL recommends that LB-MC-OC measure high water levels at designated structures on each river during future high flow events. This information could be used in further validation of the present models and in future studies.
6. CBCL recommends that LB-MC-OC develop an infrastructure renewal plan in consultation with the Department of Municipal Affairs to replace the under-sized culverts and bridges identified in this study. The 1:100 AEP flood flows under the ultimate development scenario should be used as the design flows for the design of new structures.
7. CBCL recommends that LB-MC-OC re-evaluate the condition of existing hydraulic structures in five to ten years.
8. CBCL recommends that LB-MC-OC consider amending the town's municipal plan to designate wetland and water bodies identified in this study as conservation areas.

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