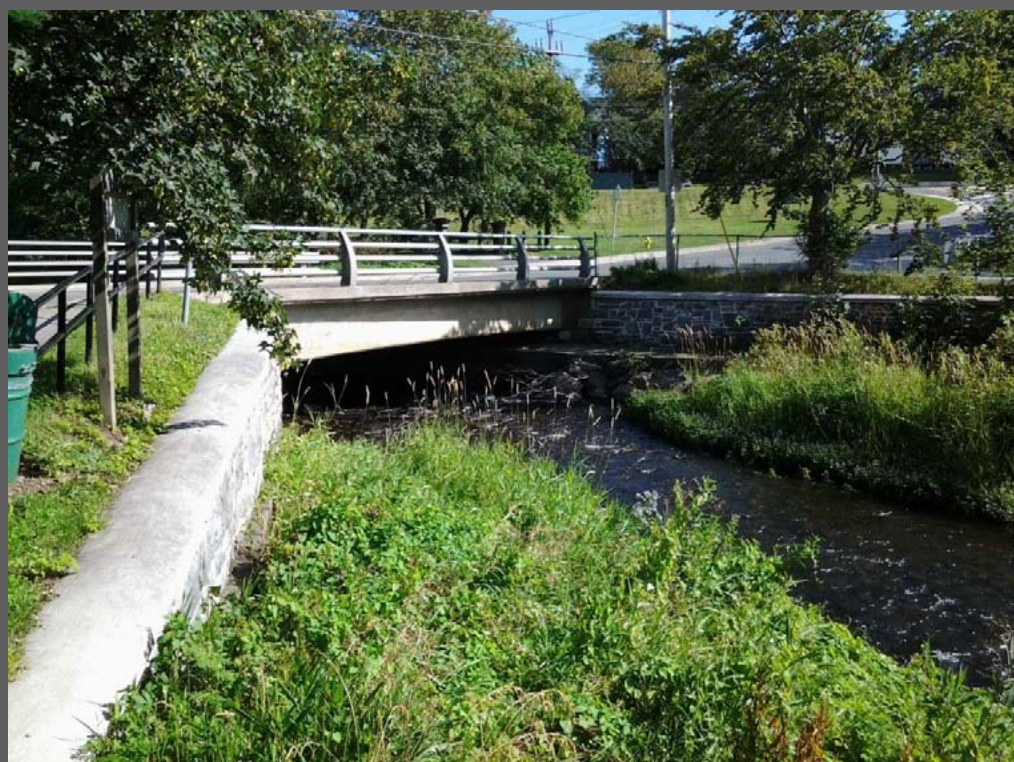
Rennies River Catchment Stormwater Management Plan

Final Report



123097.00 ● Final Report ● April 15, 2014

Prepared for:

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Prepared by:

CBCL LIMITED
Consulting Engineers

Issue or Revision	Reviewed By:	Date	Issued By:
Final	Alexander Wilson	04/15/2014	Greg Shepaprd
Draft Final	Alexander Wilson	01/29/2014	Greg Sheppard
Draft	Alexander Wilson	11/04/2013	Greg Sheppard

CBCL LIMITED
Consulting Engineers

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Solving today's problems with tomorrow in mind

April 15, 2014

City of St. John's
PO Box 908
10 New Gower Street
St. John's, NL
A1C 5M2

Attention: Mr. Dave Wadden, M.Eng. P.Eng.

RE: Rennies River Catchment Stormwater Management Plan
CBCL Project No. 123097

Dear Mr. Wadden:

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

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

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

Sincerely,

CBCL LIMITED
Consulting Engineers

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Project Manager
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Contents

Appendices

List of Tables

List of Figures

EXECUTIVE SUMMARY

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

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

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

Several flood protection approaches were evaluated using the hydrologic and hydraulic models developed for this study, and the most optimum flood protection measures have been recommended for the City's consideration. In terms of overall impact on the study area, the most significant recommended flood protection improvement is a weir proposed for the east end of Long Pond. The construction of a weir at this location will result in reduced flooding downstream of Long Pond.

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

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

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

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

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

CHAPTER 1 INTRODUCTION

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

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

1.1 Study Scope

The scope of work includes the following tasks:

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

CHAPTER 2 BACKGROUND INFORMATION

2.1 Historical Flooding

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

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

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

2.2 Previous Studies

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

2.2.1 Ken Brook and Leary's Brook Floodplain Delineation Study

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

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

2.2.2 Quidi Vidi Lake Tributary Flood Plain Delineation

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

The study found that two large areas are prone to flooding during the 1:100 AEP flood; Portugal Cove Road Bridge and the floodplain immediately upstream and downstream, as well as the floodplain from Kings Bridge Road Bridge to Quidi Vidi Lake. To mitigate flooding near the Portugal Cove Road Bridge, the study recommends alterations to the bridge, which include removing sediment beneath the bridge, removing concrete obstructions in the downstream channel and raising the north bank of Rennies River for approximately 150 m upstream of the bridge. However, even with these modifications, a large portion of the soccer pitch at Fieldian Grounds and the Riverdale Tennis Club grounds would still be flooded. To minimize the extend of flooding between Kings Bridge Road Bridge and Quidi Vidi Lake, the report suggests constructing berms or levees along the north bank of Rennies River from Kings Bridge Road Bridge to Carnell Bridge and raising the footbridge at Loblaws. However, these alterations will not prevent all the flooding problems; a large portion of the Loblaws parking lot as well as sections of Carnell Drive and Lake Avenue will still be within the flood limits.

CHAPTER 3 DATA COLLECTION AND ANALYSIS

3.1 Data Collection

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

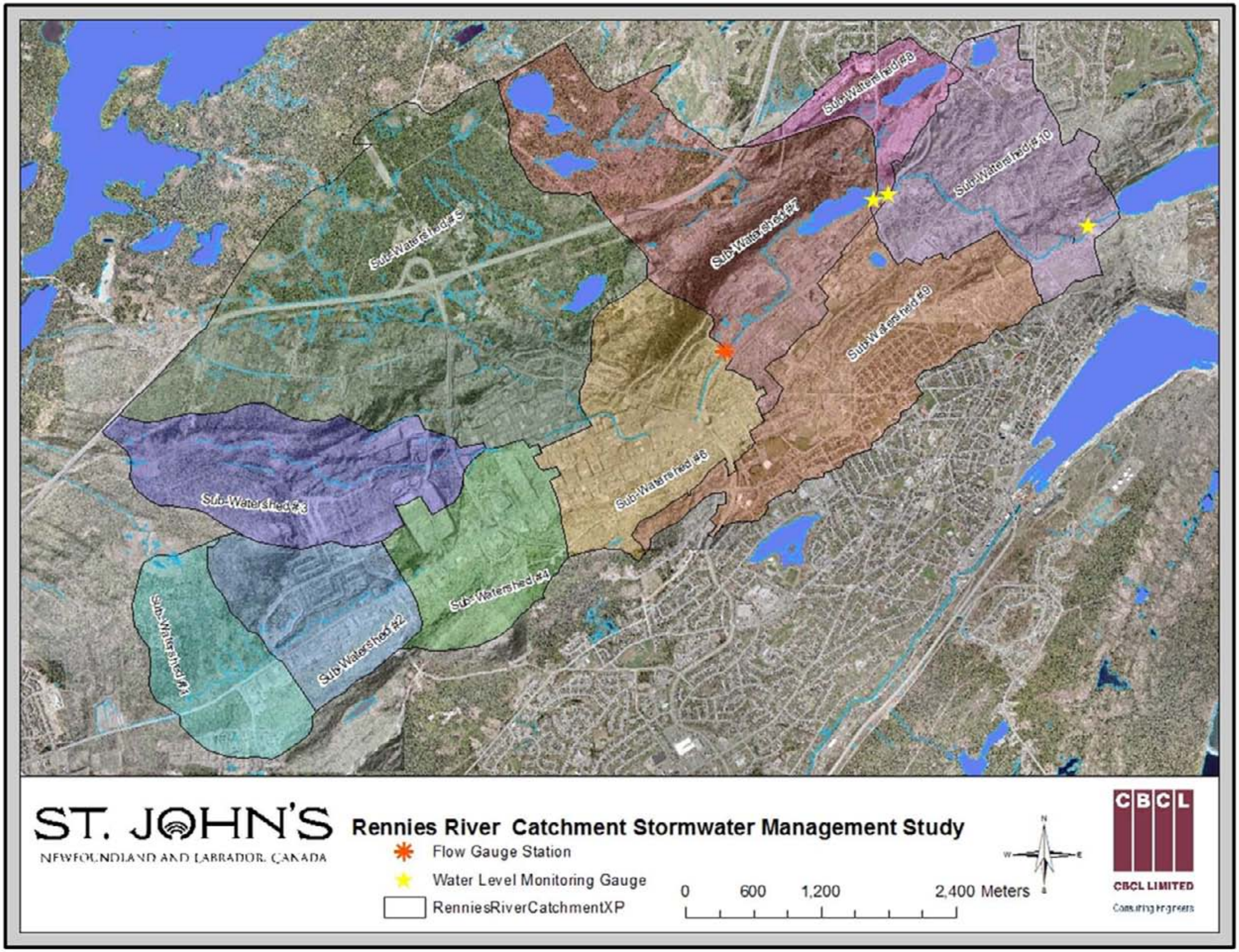
- Aerial photography of the study area provided by the City;
- LiDAR data of the study area provided by the City;
- Zoning and property mapping provided by the City;
- Future development data provided by the City;
- Bridge construction drawings provided by the City;
- Construction drawings of flood control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

3.1.1 Calibration Data

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

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

FIGURE 3-1 FLOW AND WATER LEVEL GAUGE STATIONS



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

3.1.2 Detailed Topographic Data

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

3.1.3 Hydraulic Structure Details

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

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

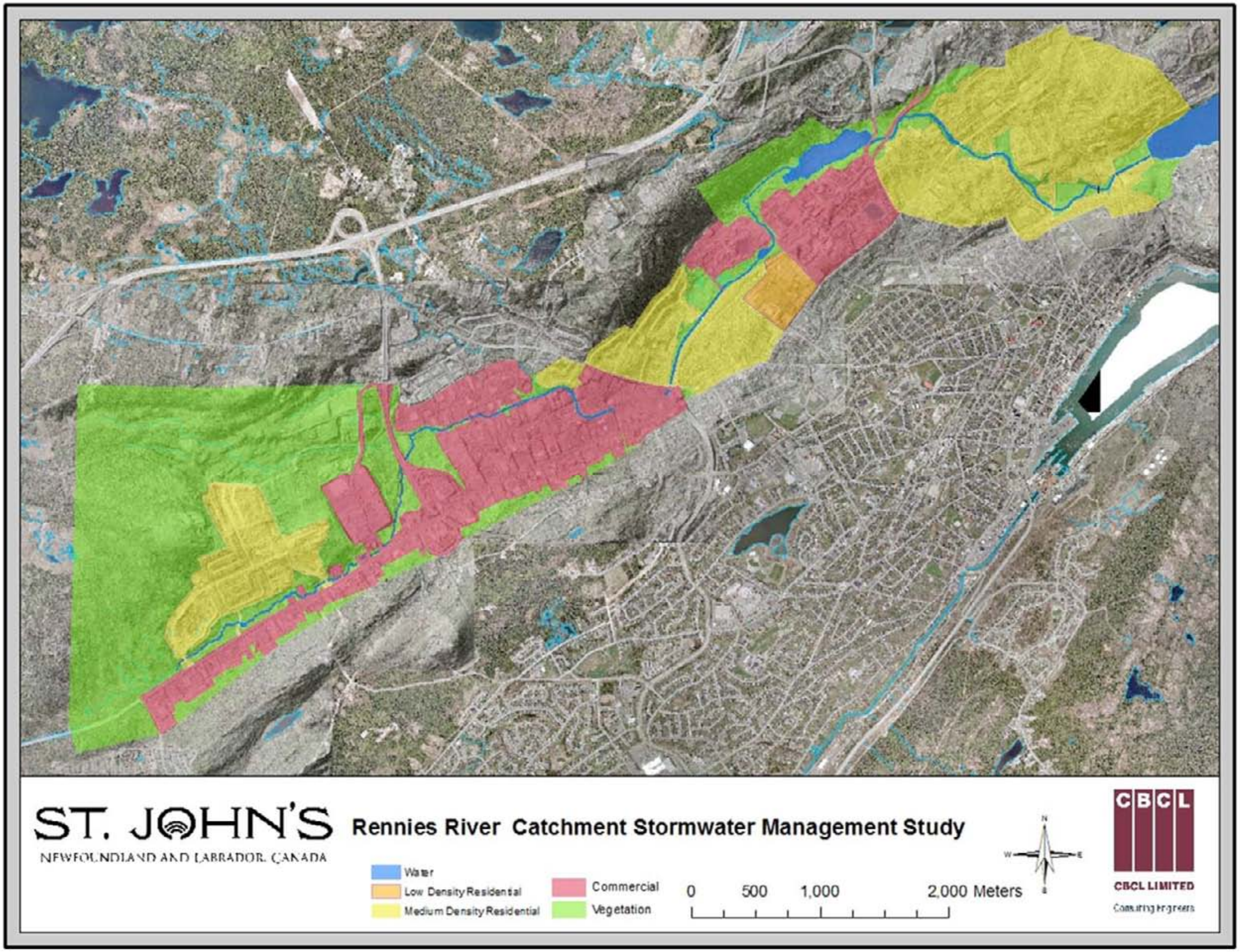
3.2 Data Analysis

3.2.1 Land Use Mapping

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

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

FIGURE 3-2 LAND USE MAPPING



3.2.2 Watershed Delineation and Watershed Properties

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

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

TABLE 3-1 WATERSHED CHARACTERISTICS

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

CHAPTER 4 UPDATE OF IDF CURVES AND DESIGN HYETOGRAPHS

4.1 IDF Curves

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

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

4.1.1 Existing Data

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

4.1.2 Additional Data

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

The City owns and operates three rain gauges, located at Ruby Line, Blackler Avenue and Windsor Lake. The Windsor Lake gauge is located approximately 1.6 km southwest of the previously operated St. John's Airport gauge. It observes rainfall over the Windsor Lake and Broad Cove River watershed, as well as parts of Outer Cove Brook, Stick Pond Brook, Coaker's Meadow Brook, Virginia River and Rennies River. The Windsor Lake gauge is a Met One tipping bucket gauge, installed at an elevation of 159 m. It was installed in December of 1998 and records data at 1-minute intervals. The close proximity of the two gauges gives an initial indication that the two data sets can be combined. The report titled *Rainfall Distribution in the City of St. John's: Temporal Distribution, Spatial Variation, Frequency Analysis, and Tropical Storm Gabrielle* examined the appropriateness of combining the two data sets by comparing overlapping data recorded between 1999 and 2001 at the two gauges. This study determined a correlation coefficient of 0.9 for the daily rainfall comparison, implying a strong relationship and suggesting that the observed rainfall at both locations is uniform.

4.1.3 Update

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

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

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

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

4.1.4 Results

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

FIGURE 4-1 IDF CURVES

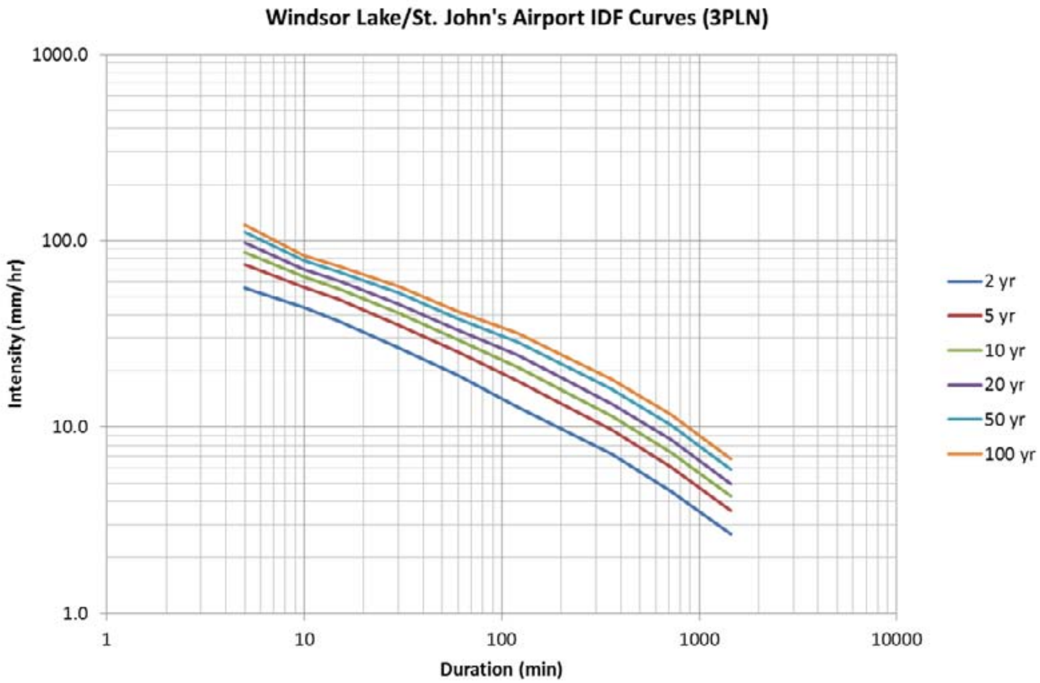


TABLE 4-1 UPDATED IDF RAINFALL INTENSITIES

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

4.2 Design Hyetographs

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

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

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

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

4.2.1 Updated IDFs

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

TABLE 4-2 1:20 AEP RAINFALL HYETOGRAPHS – UPDATED IDFS AND CITY'S HYETOGRAPH SHAPE

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

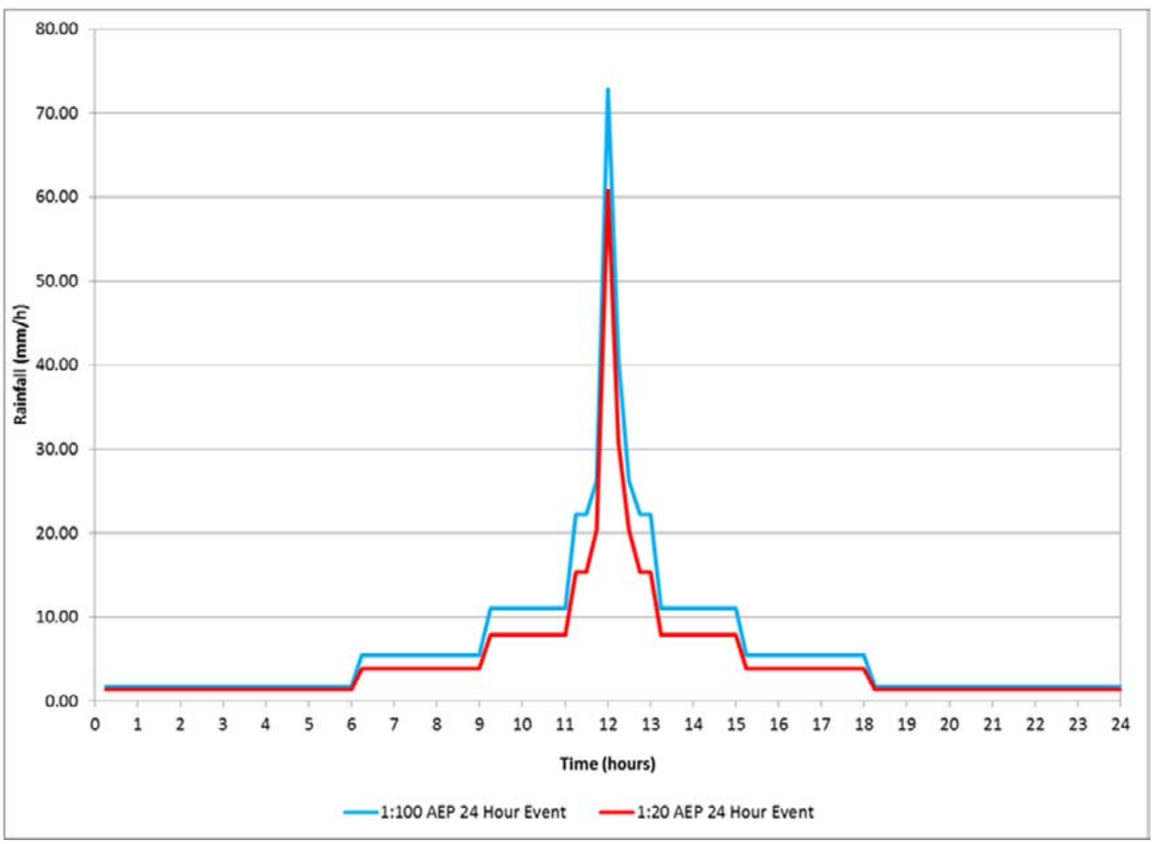
TABLE 4-3 1:100 AEP RAINFALL HYETOGRAPHS – UPDATED IDFS AND CITY'S HYETOGRAPH SHAPE

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

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

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

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



4.2.2 Climate Change Projections

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

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

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

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

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

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

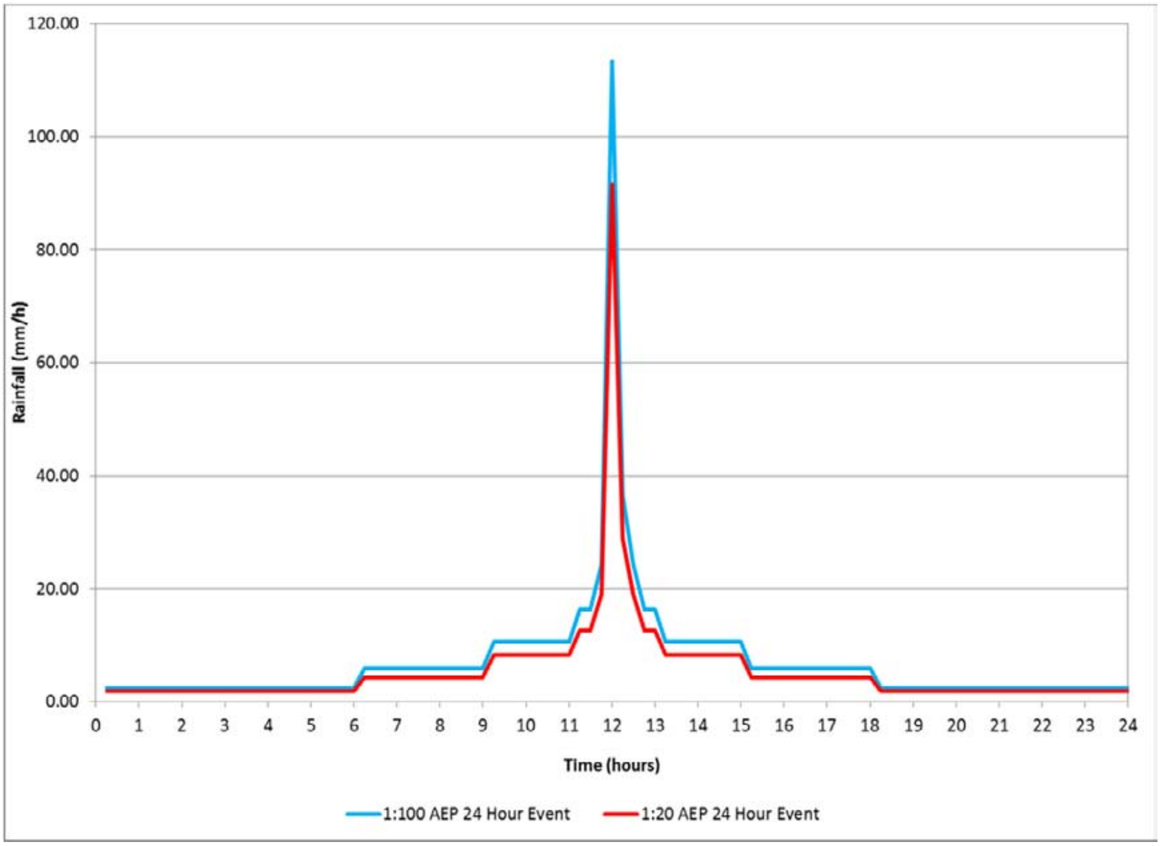
TABLE 4-6 1:100 AEP RAINFALL HYETOGRAPHS – CLIMATE CHANGE AND CITY'S HYETOGRAPH SHAPE

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

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

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

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



CHAPTER 5 STATISTICAL ANALYSIS

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

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

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

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

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

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

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

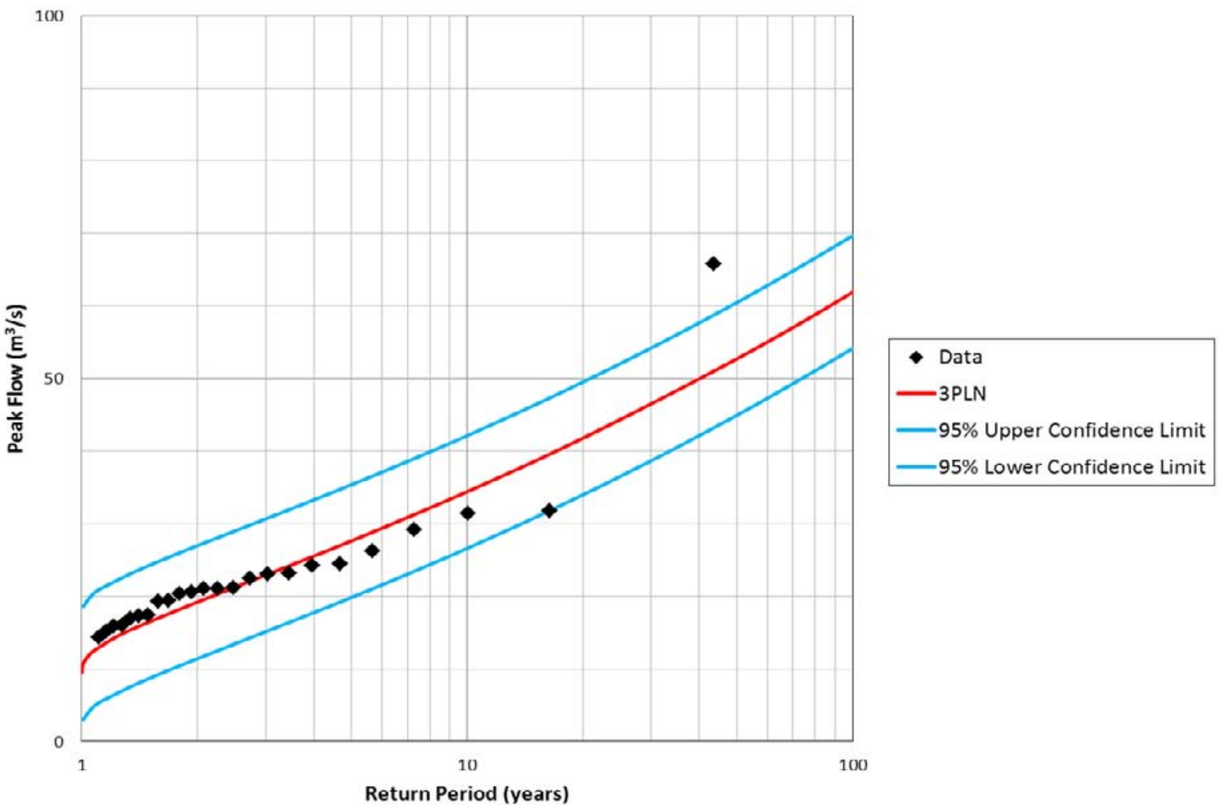


TABLE 5-1 SINGLE STATION FREQUENCY ANALYSIS RESULTS

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

CHAPTER 6 HYDROLOGIC MODELING

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

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

6.1 Model Development

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

TABLE 6-1 IMPERVIOUS AREA CHANGES FOR FUTURE DEVELOPMENT

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

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

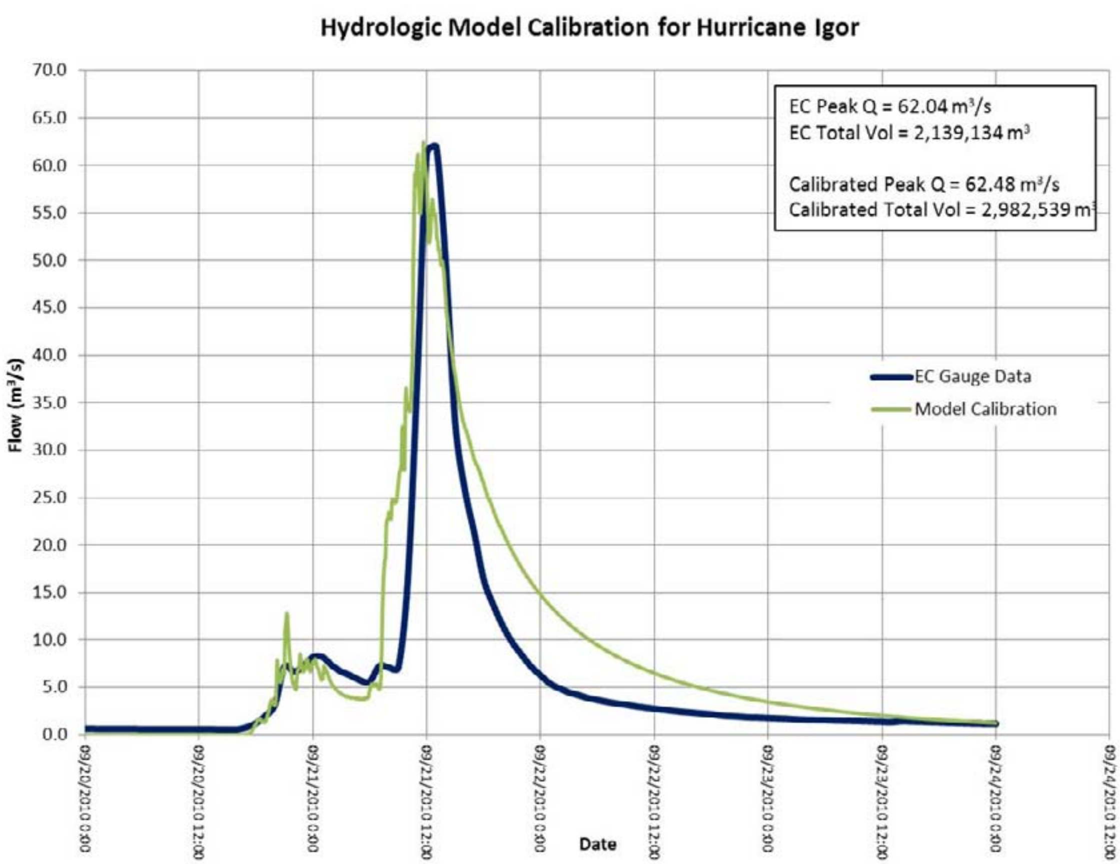
6.2 Model Calibration

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

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

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

FIGURE 6-1 HYDROLOGIC MODEL CALIBRATION

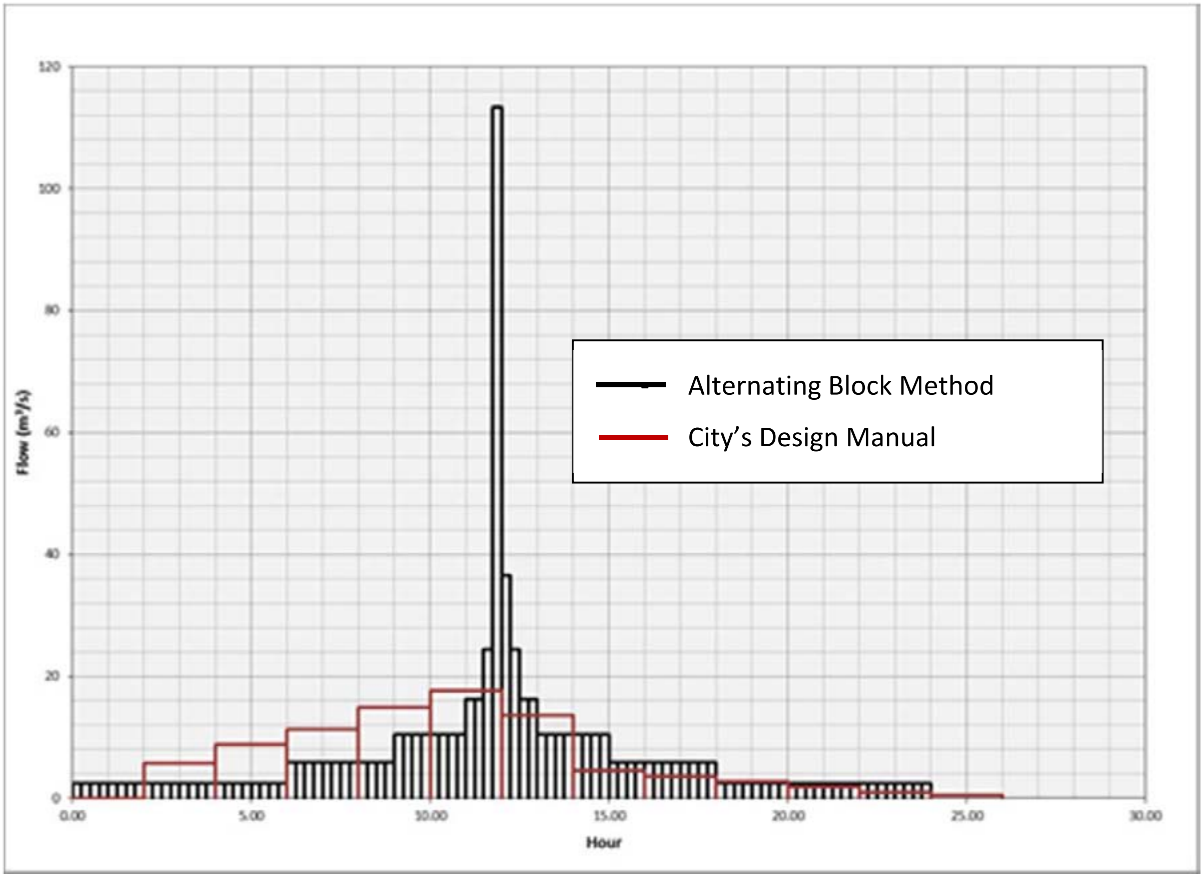


Since no flow data is available to calibrate the lower reach of Rennies River (sub-watersheds 7-10) for Hurricane Igor, model parameters were determined by extrapolating the watershed parameters from the upstream areas to obtain the 1:20 and 1:100 AEP flows for these sub-watersheds.

6.3 Hyetograph Selection

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

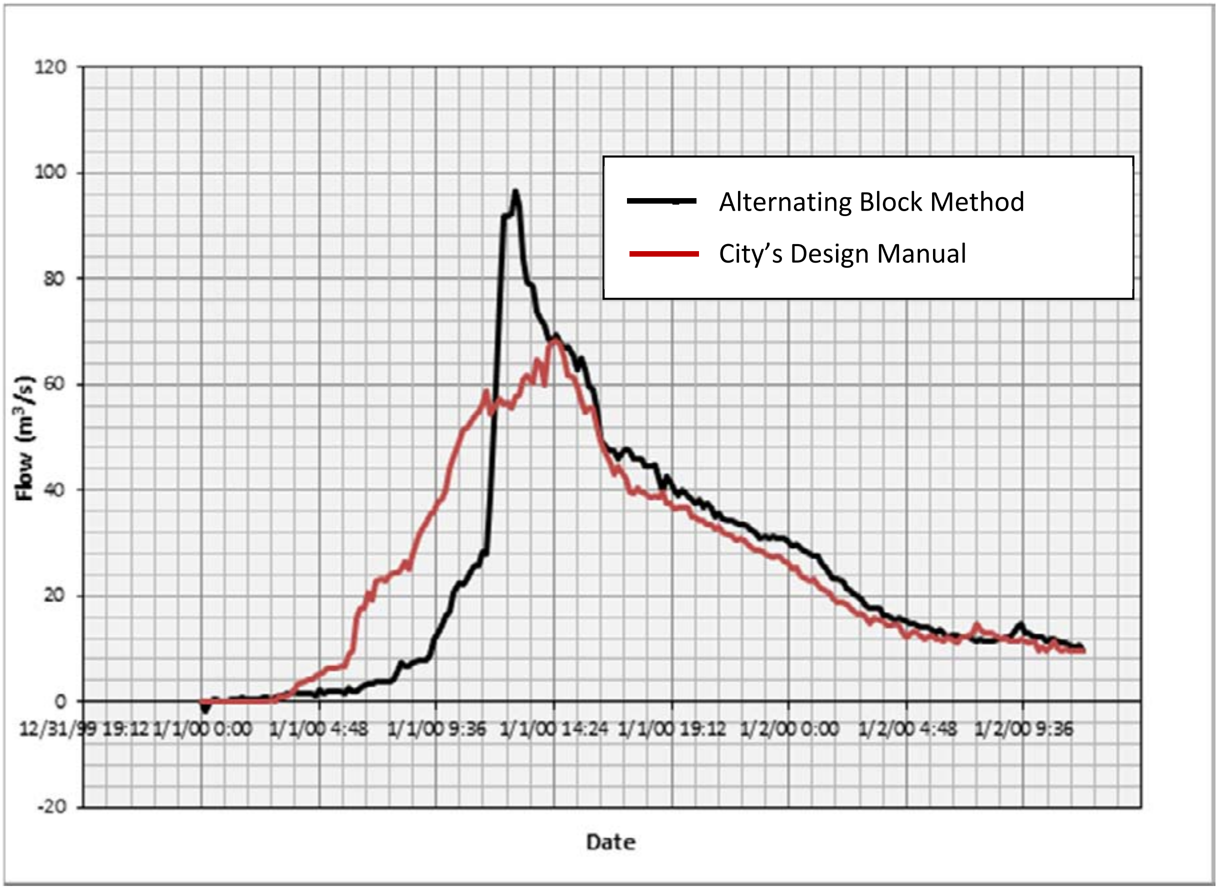
FIGURE 6-2 HYETOGRAPHS FOR THE ALTERNATING BLOCK METHOD AND THE CITY'S DESIGN MANUAL



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

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

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



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

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

6.4 Simulated Flood Flows

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

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

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

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

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

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

CHAPTER 7 HYDRAULIC MODELING

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

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

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

7.1 Model Development

7.1.1 Model Input

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

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

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

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

7.1.2 Structures

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

FIGURE 7-1 HYDRAULIC STRUCTURE LOCATIONS

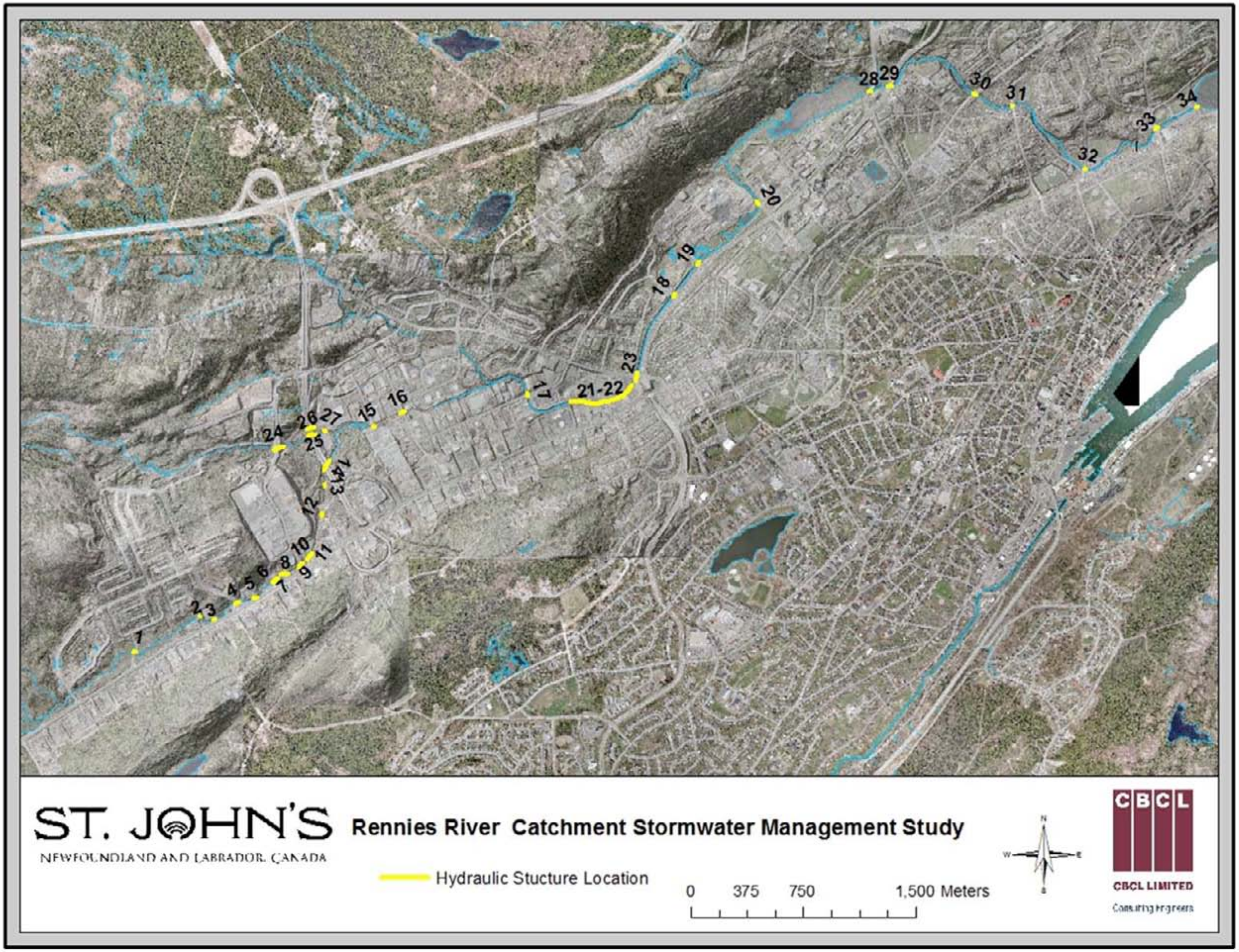


TABLE 7-1 HYDRAULIC STRUCTURES LOCATED ON MAIN RIVER REACHES

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

7.1.3 Manning's Roughness Coefficient

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

[1] Chow, V.T. 1959. *Open Channel Hydraulics.* McGraw-Hill, New York.

TABLE 7-2 LITERATURE VALUES FOR MANNING'S N

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

7.2 Model Calibration

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

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

TABLE 7-3 SELECTED CALIBRATION EVENT

Event	General Description	Available Observed Data
September, 2010	- Significant flood event causing widespread inundation of the Rennies River floodplain between Portugal Cove Road and Kings Bridge Road, Allandale Road and Prince Philip Drive, and CBC Parking Lot. - Peak flow of the Leary's Brook at Prince Philip Drive gauge (02ZM020): 62.5 m^3/s	- Flood Levels: Long Pond Bridge - Stream flow Data: The Leary's Brook at Prince Philip Drive gauge (02ZM020) - Observed flood extent by photos

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

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

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

7.3 Simulated Flood Flows

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

TABLE 7-4 FLOOD FLOWS

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

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

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

CHAPTER 8 SENSITIVITY ANALYSIS

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

8.1 Hydrologic Model Sensitivity

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

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

FIGURE 8-1 DEPRESSION STORAGE SENSITIVITY ANALYSIS

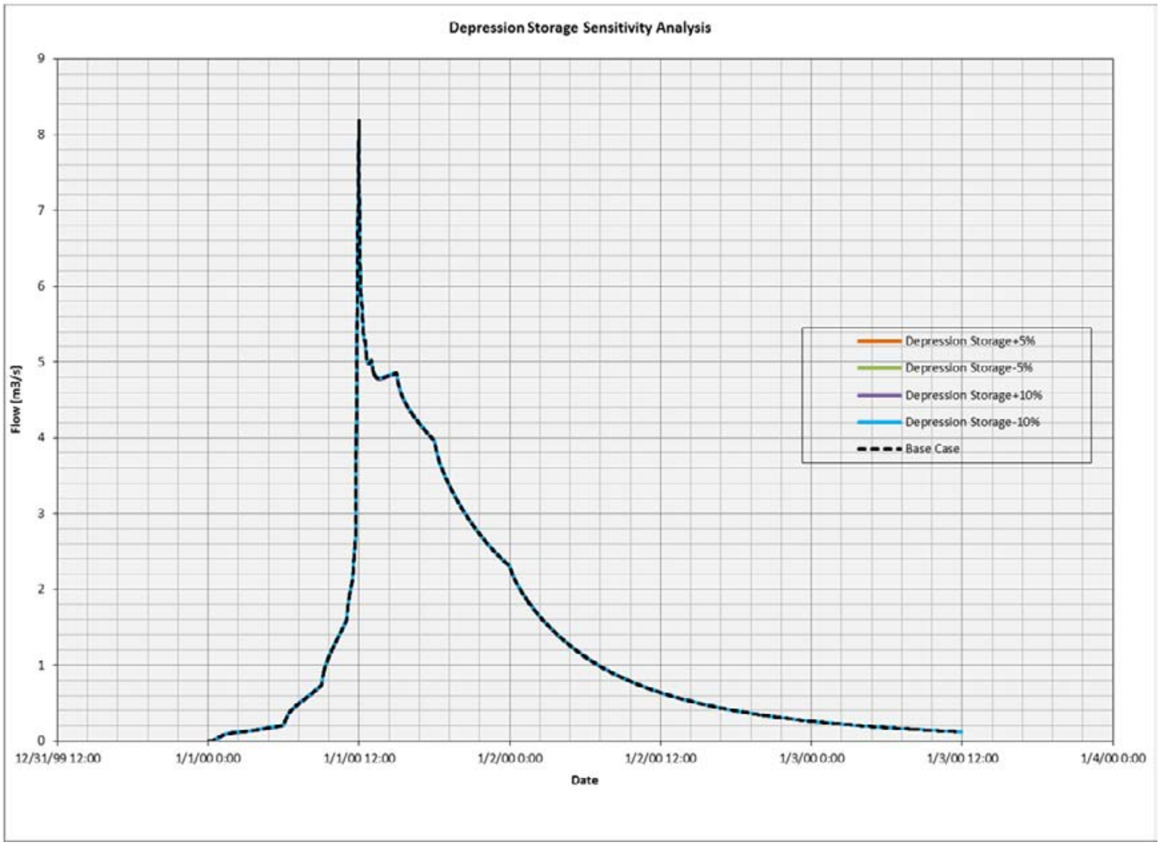


FIGURE 8-2 AVERAGE CAPILLARY SUCTION SENSITIVITY ANALYSIS

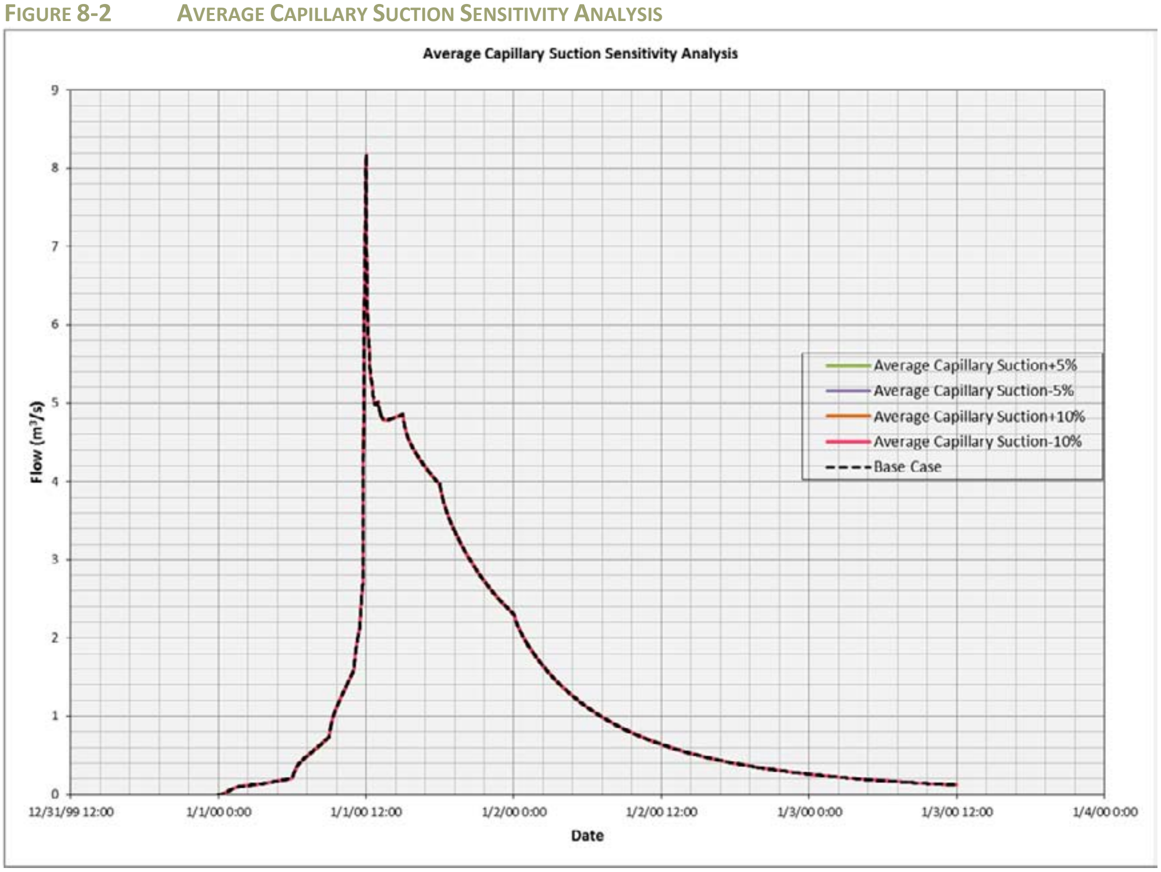


FIGURE 8-3 INITIAL MOISTURE DEFICIT SENSITIVITY ANALYSIS

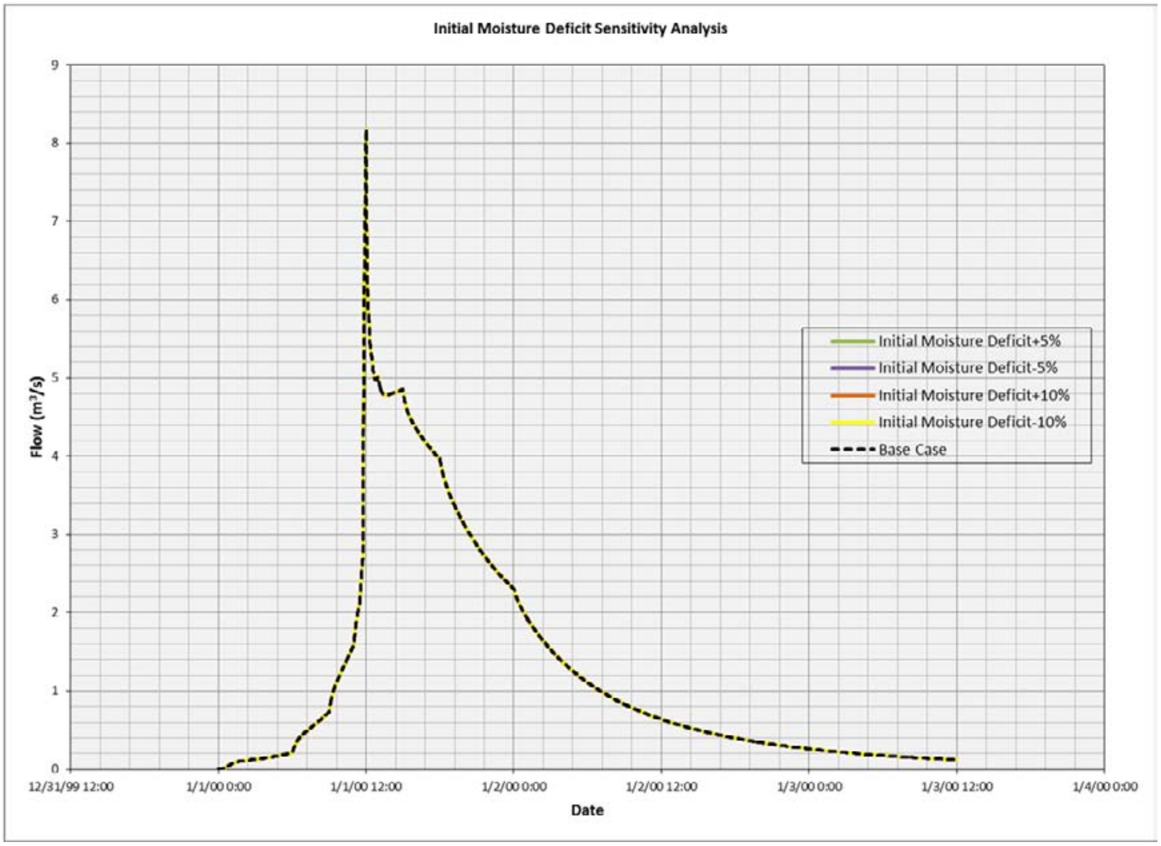


FIGURE 8-4 PERCENT IMPERVIOUS AREA SENSITIVITY ANALYSIS

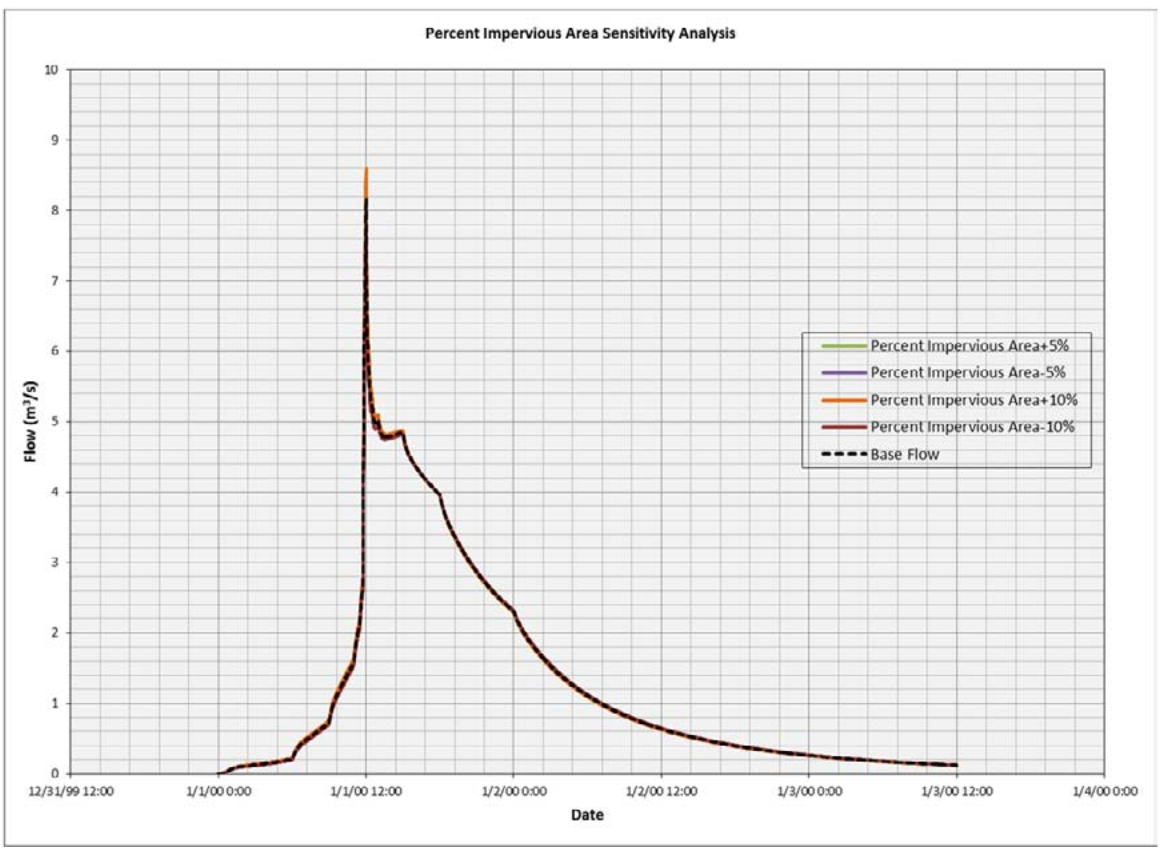


FIGURE 8-5 SUBBASIN WIDTH SENSITIVITY ANALYSIS

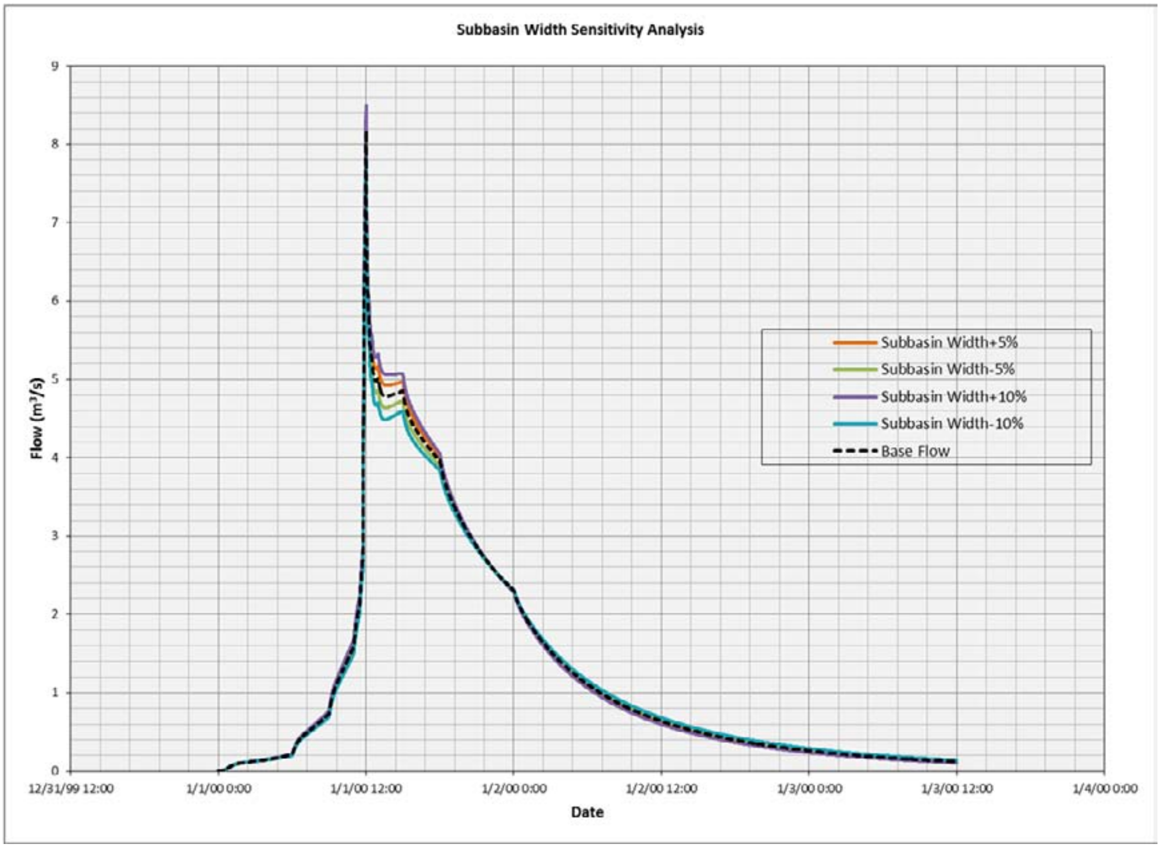


FIGURE 8-6 MANNING'S ROUGHNESS VALUE SENSITIVITY ANALYSIS

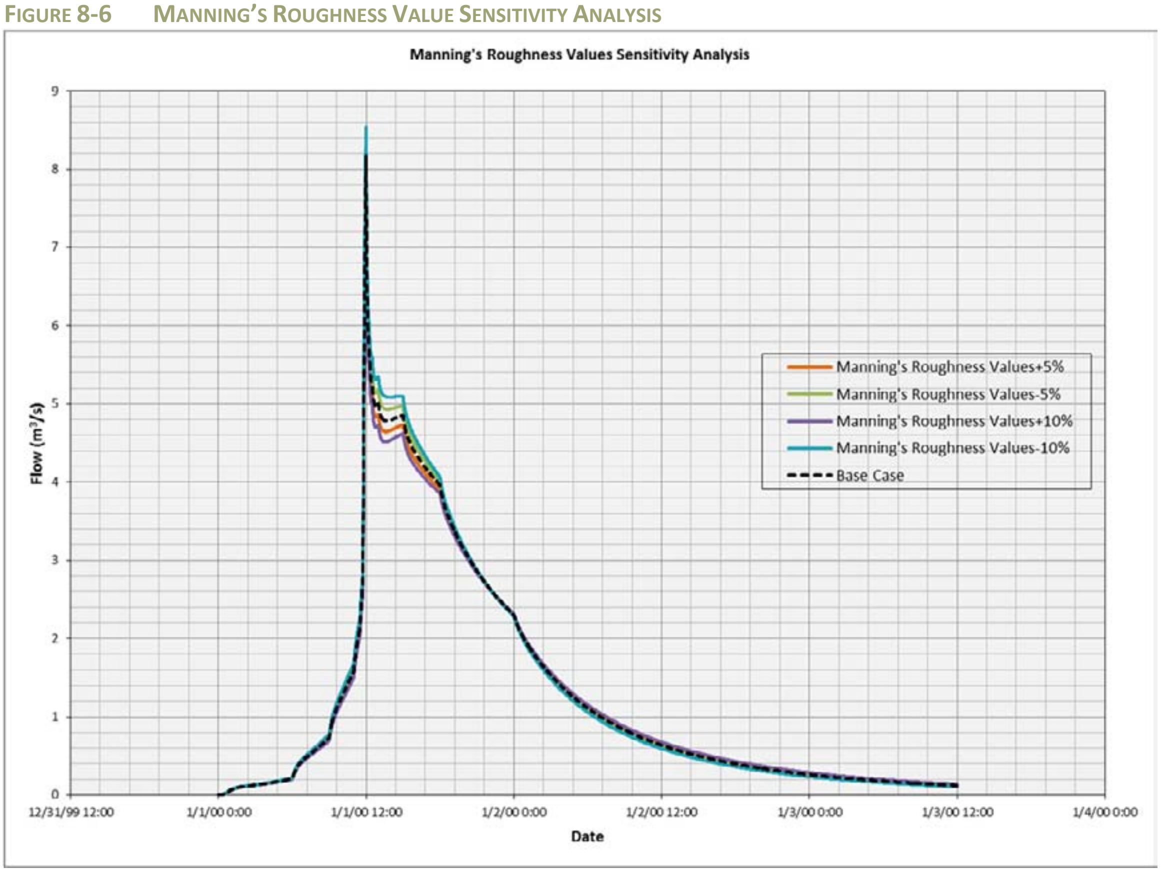


FIGURE 8-7 SATURATED HYDRAULIC CONDUCTIVITY SENSITIVITY ANALYSIS

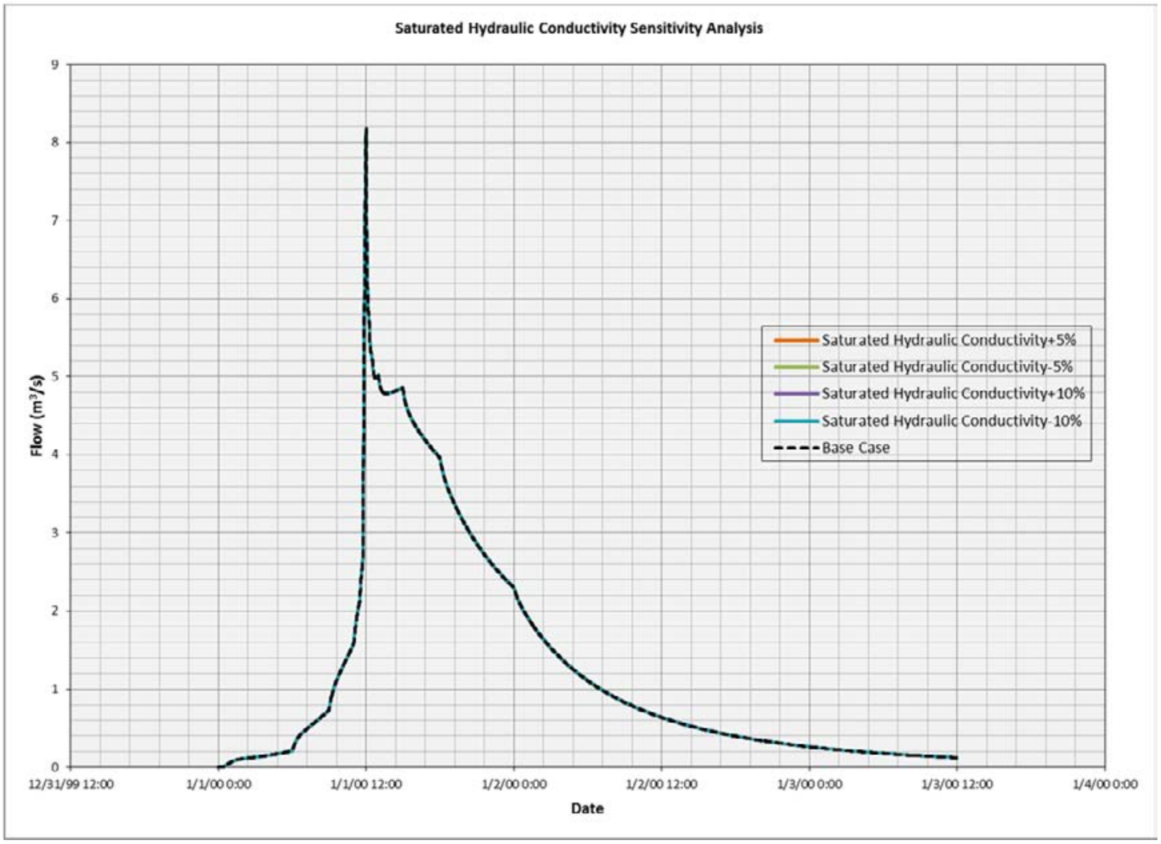


TABLE 8-1 VARIATIONS IN PEAK FLOWS AS A RESULT OF ADJUSTING HYDROLOGIC AND HYDRAULIC PARAMETERS

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

8.2 Hydraulic Model Sensitivity

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

It can be seen that generally, the variations of Manning's n roughness and peak flow rate yield sensible and uniform variations in water levels. The results indicate that the hydraulic model is more sensitive to changes in the peak flow rates. Decreasing the peak flow rates by 10% decreased peak water level at the upstream of Kings Bridge Road by 1.64%. The Manning's n values impacting the flows by 0.62% for a parameter change of 10%.

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

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

FIGURE 8-8 PEAK FLOW RATE SENSITIVITY ANALYSIS

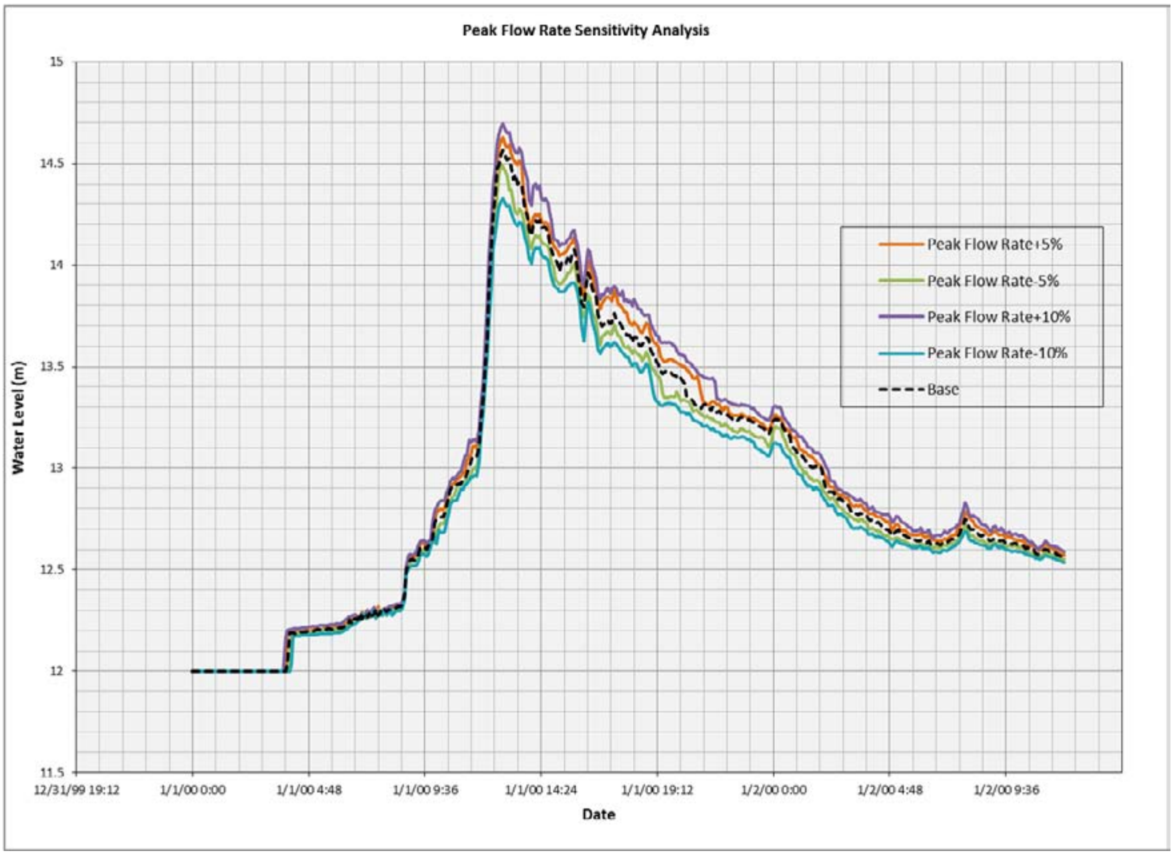
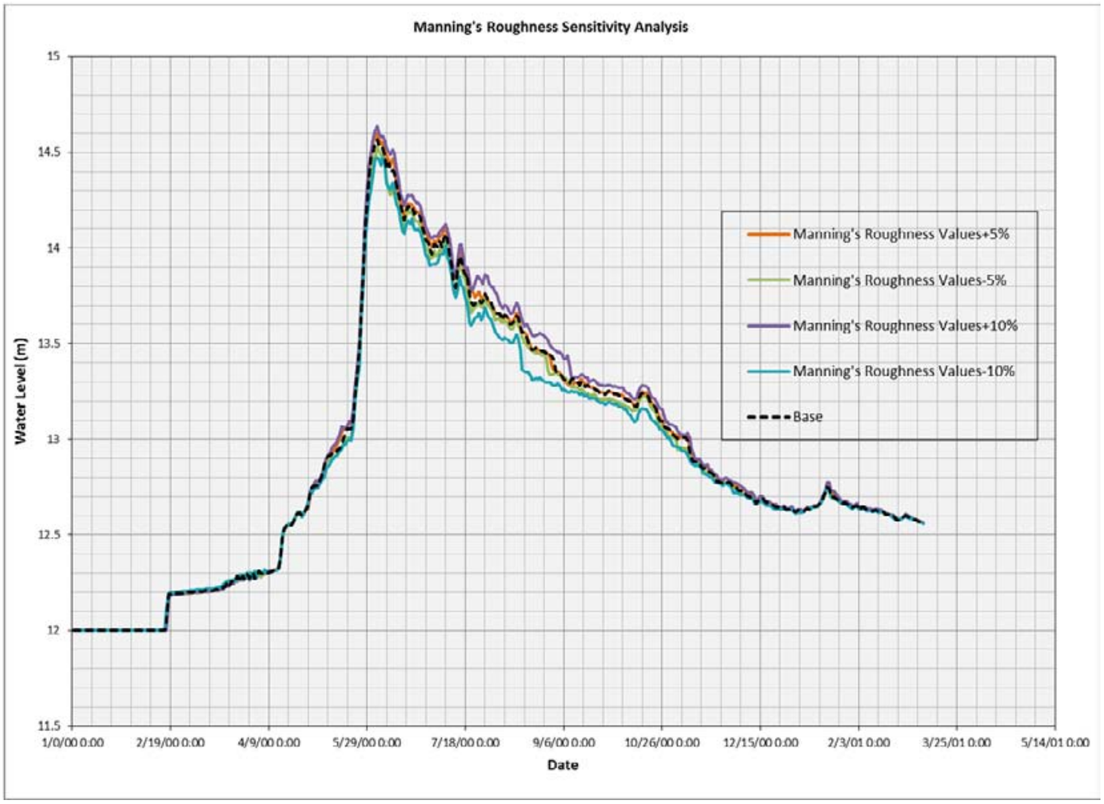


FIGURE 8-9: MANNING'S ROUGHNESS VALUE SENSITIVITY ANALYSIS



CHAPTER 9 FLOODPLAIN AND FLOOD HAZARD MAPPING

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

The existing conditions mapping is based on the following:

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

The future conditions mapping is based on the following:

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

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

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

CHAPTER 10 PRELIMINARY DESIGN

10.1 Flood Control

Locations along Rennies River with known flooding problems include:

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

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

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

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

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

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

Location 3 – Outlet of Long Pond

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Location 2 – Upstream of Capasian Road

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

Location 4 – Clinch Crescent East to Clinch Crescent West

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

Location 5 – Wicklow Street to Thorburn Road

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

Location 6 – Upstream from Avalon Mall Culverts

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

Location 7 – O'Leary Avenue Bridge

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

Location 8 – Downstream of Mews Place Culvert

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

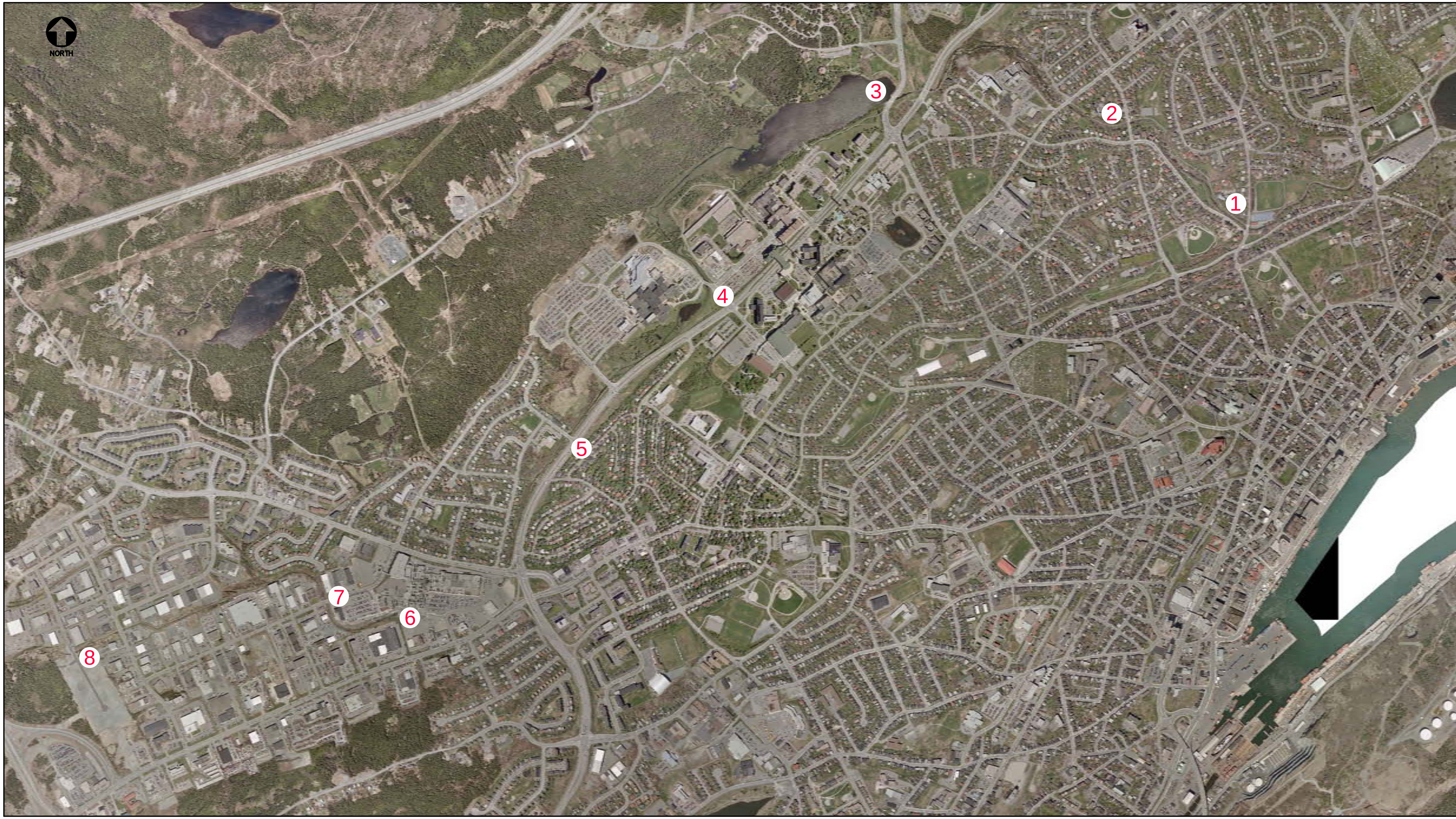
Ken Brook

During the 1:100 AEP event under future conditions, there is localized flooding at private culverts located on Ken Brook; however, Kenmount Road does not flood. In most cases, the hydraulic opening required to upgrade these culverts would result in the raising of private parking lots. Even then, there are some instances where cover for the culverts would not be adequate.

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

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

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



PLAN

SCALE: 1:15000

0m 500m 1000m

DRAWN: J. LARACY	SCALE: 1:15000	CITY OF ST. JOHN'S RENNIES RIVER CATCHMENT STORMWATER MANAGEMENT PLAN
PROJECT NO: 123097.00	DATE: APR 15, 2014	
DRAWING NO: **FIGURE 10-1**		FLOOD CONTROL IMPROVEMENTS LOCATION PLAN

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

By: JLARACY

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PLAN

SCALE: 1:500

0m 10m 20m 30m

LEGEND:

PROPOSED WEIR

DRAWN: J. LARACY	SCALE: 1:500	CITY OF ST. JOHN'S RENNIES RIVER CATCHMENT STORMWATER MANAGEMENT PLAN
PROJECT NO: 123097.00	DATE: APR 15, 2014	
DRAWING NO: **FIGURE 10-2**		LOCATION 3 LONG POND

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

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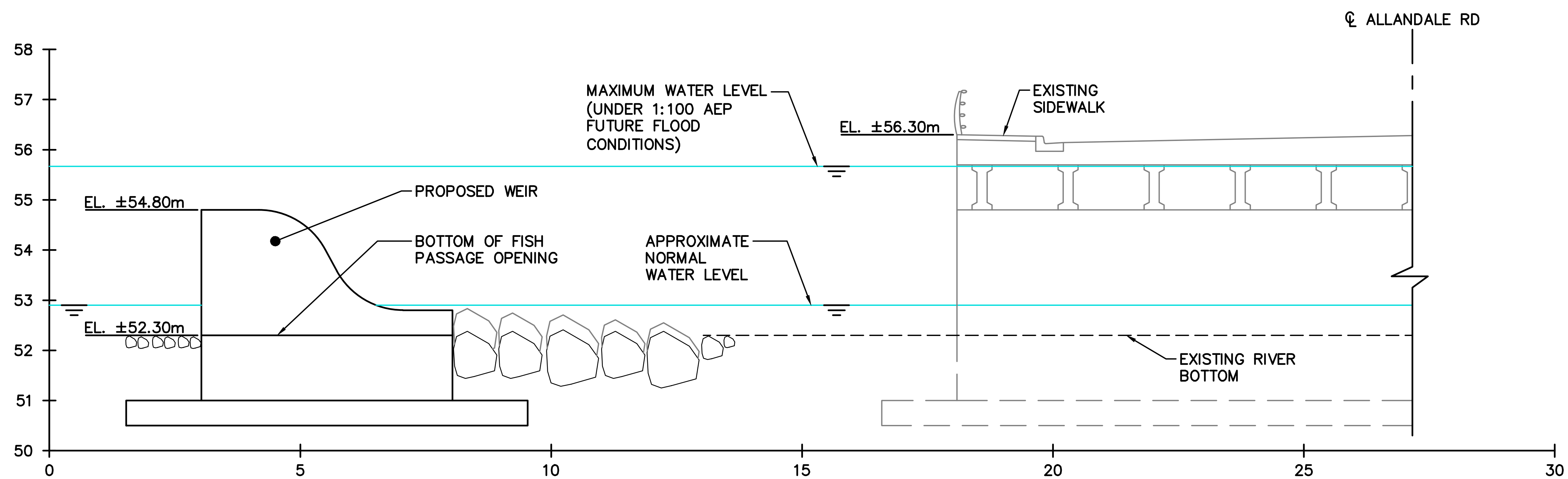

SECTION A - A

SCALE: 1:100

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

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PROJECT NO: 123097.00	DATE: APR 15, 2014	
DRAWING NO: **FIGURE 10-3**		LOCATION 3 LONG POND

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PLAN

SCALE: 1:2000

0m 50m 100m

LEGEND:

PROPOSED SEGMENTAL CONCRETE BLOCK WALL

PROPOSED CAST-IN-PLACE CONCRETE WALL

PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: 1: 2000	CITY OF ST. JOHN'S RENNIES RIVER CATCHMENT STORMWATER MANAGEMENT PLAN
PROJECT NO: 123097.00	DATE: APR 15, 2014	
DRAWING NO: FIGURE 10-4		LOCATION 1 - OPTION A PORTUGAL COVE ROAD

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187 KENMOUNT ROAD
ST. JOHN'S, NL, A1B 3P9
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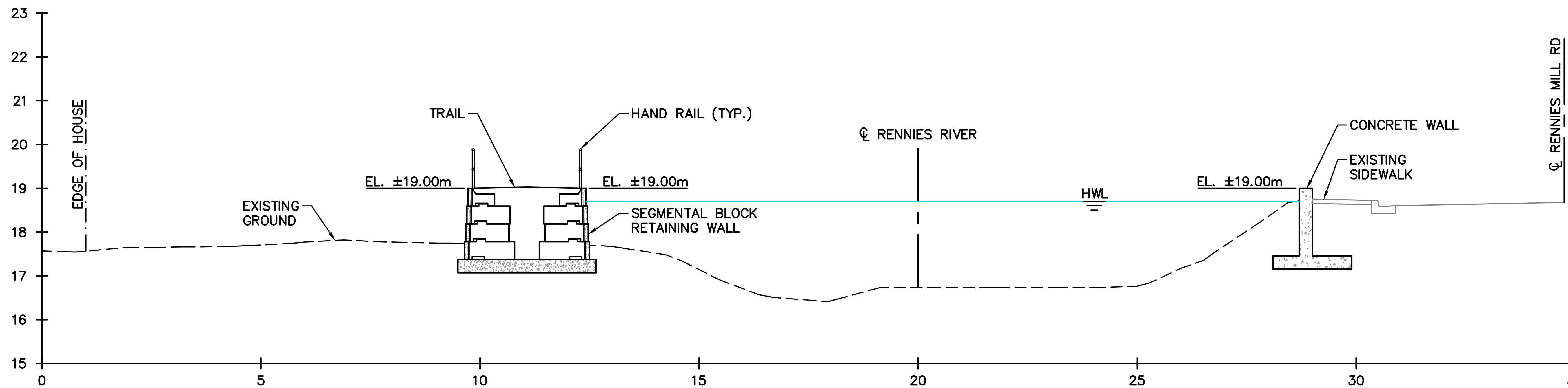

SECTION A - A

SCALE: 1:100

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

DRAWN:	SCALE:
J. LARACY	1:100
PROJECT NO:	DATE:
123097.00	APR 15, 2014
DRAWING NO:	
FIGURE 10-5	

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

LOCATION 1 - OPTION A
PORTUGAL COVE ROAD

CBCL LIMITED
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187 KENMOUNT ROAD
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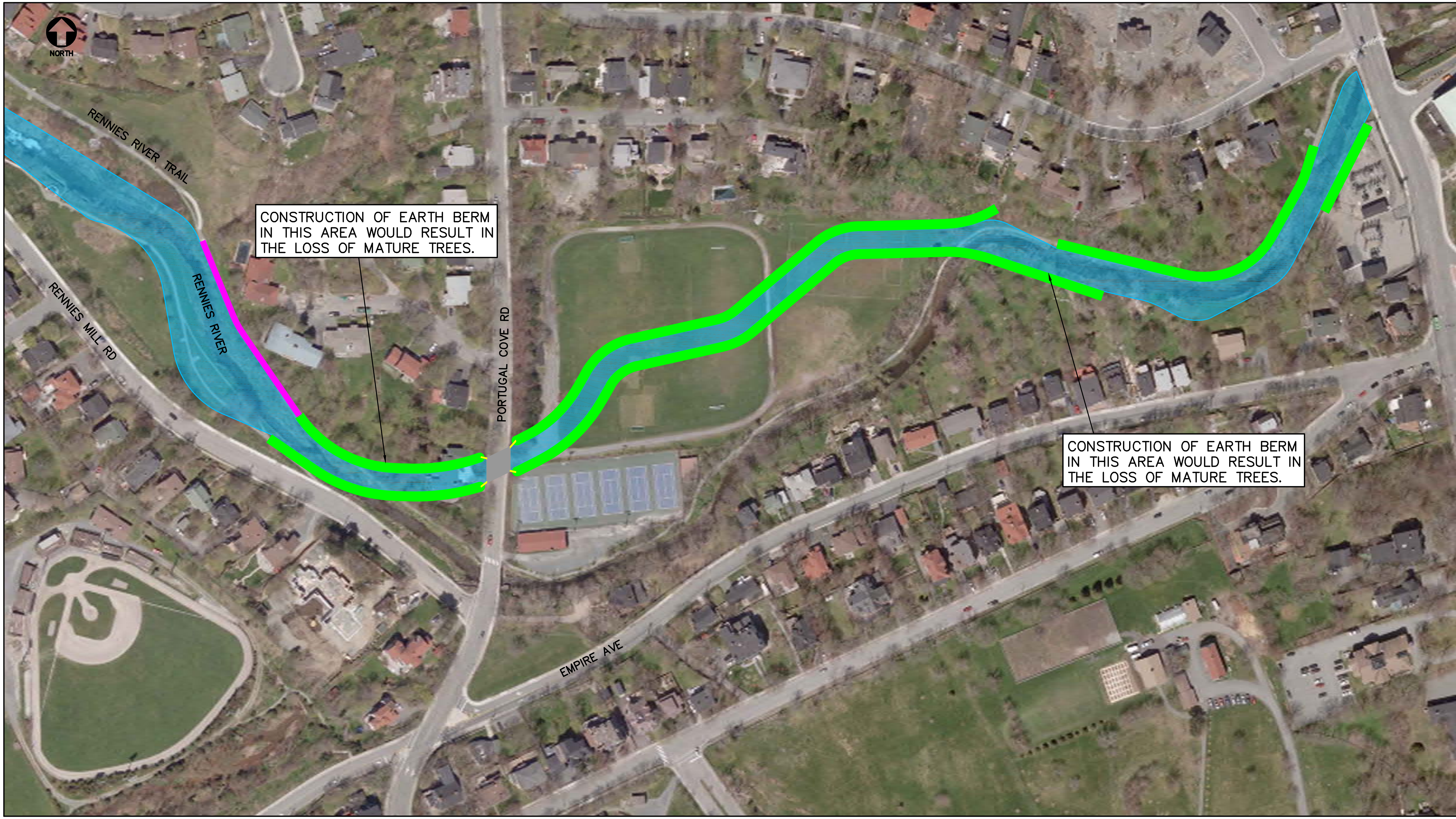

PLAN

SCALE: 1:2000

0m 50m 100m

LEGEND:

PROPOSED SEGMENTAL CONCRETE BLOCK WALL

PROPOSED CAST-IN-PLACE CONCRETE WALL

PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: 1:2000	CITY OF ST. JOHN'S RENNIES RIVER CATCHMENT STORMWATER MANAGEMENT PLAN
PROJECT NO: 123097.00	DATE: APR 15, 2014	
DRAWING NO: **FIGURE 10-6**		LOCATION 1 - OPTION B PORTUGAL COVE ROAD

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

By: JLARACY
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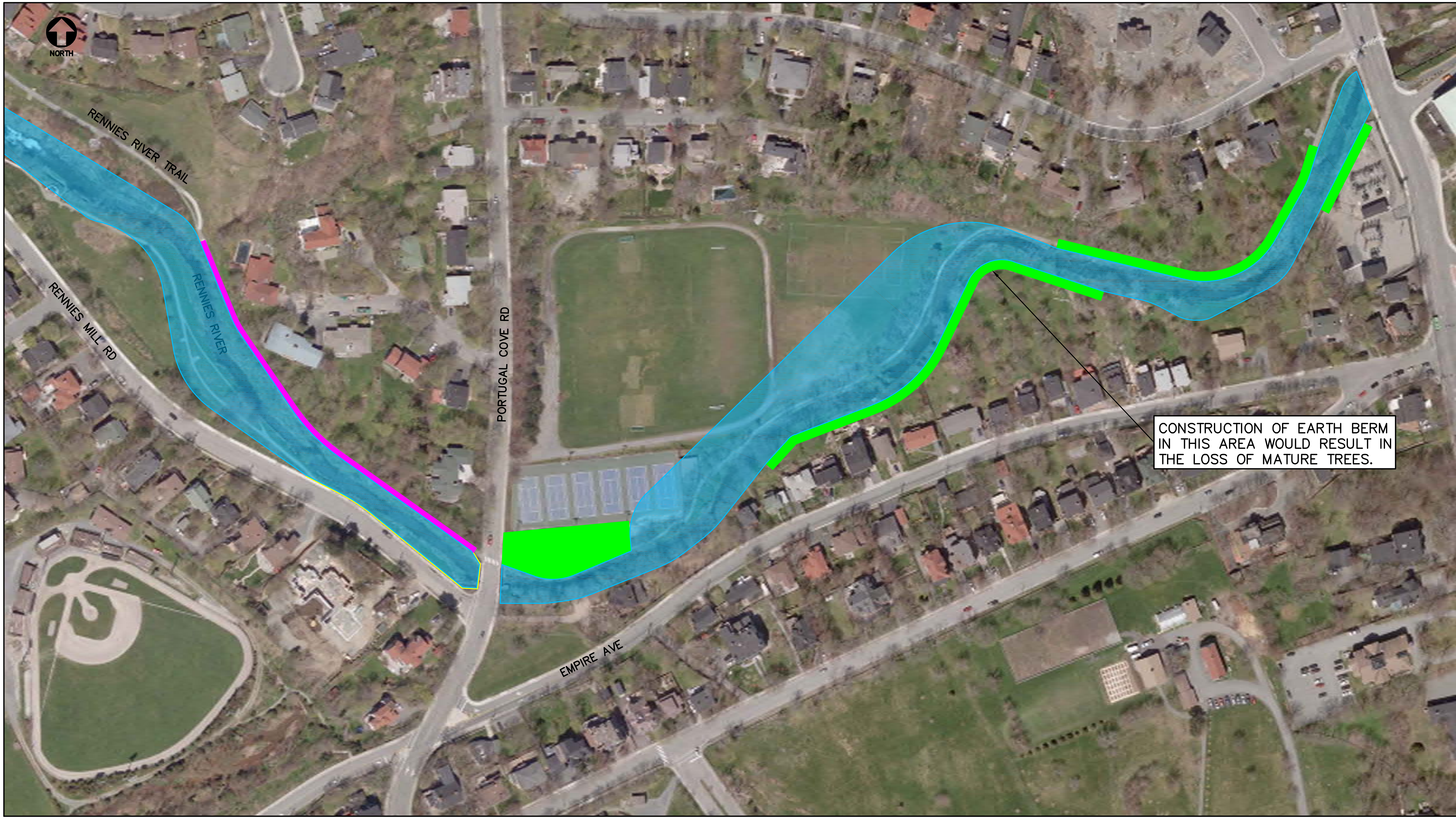

PLAN

SCALE: 1:2000

0m 50m 100m

LEGEND:

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

DRAWN: J. LARACY	SCALE: 1:2000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO: **FIGURE 10-7**	

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

LOCATION 1 - OPTION C
PORTUGAL COVE ROAD

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

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

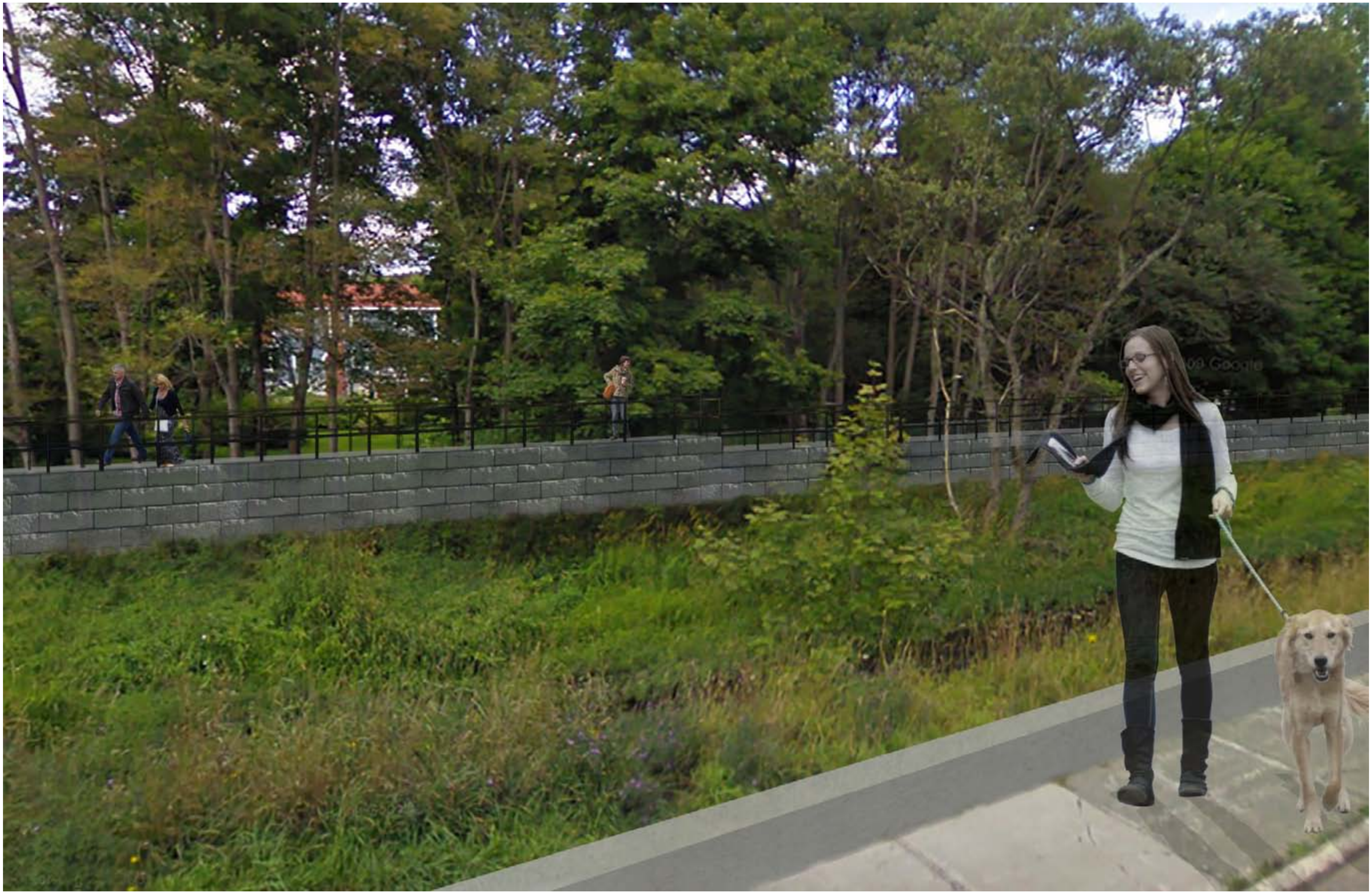
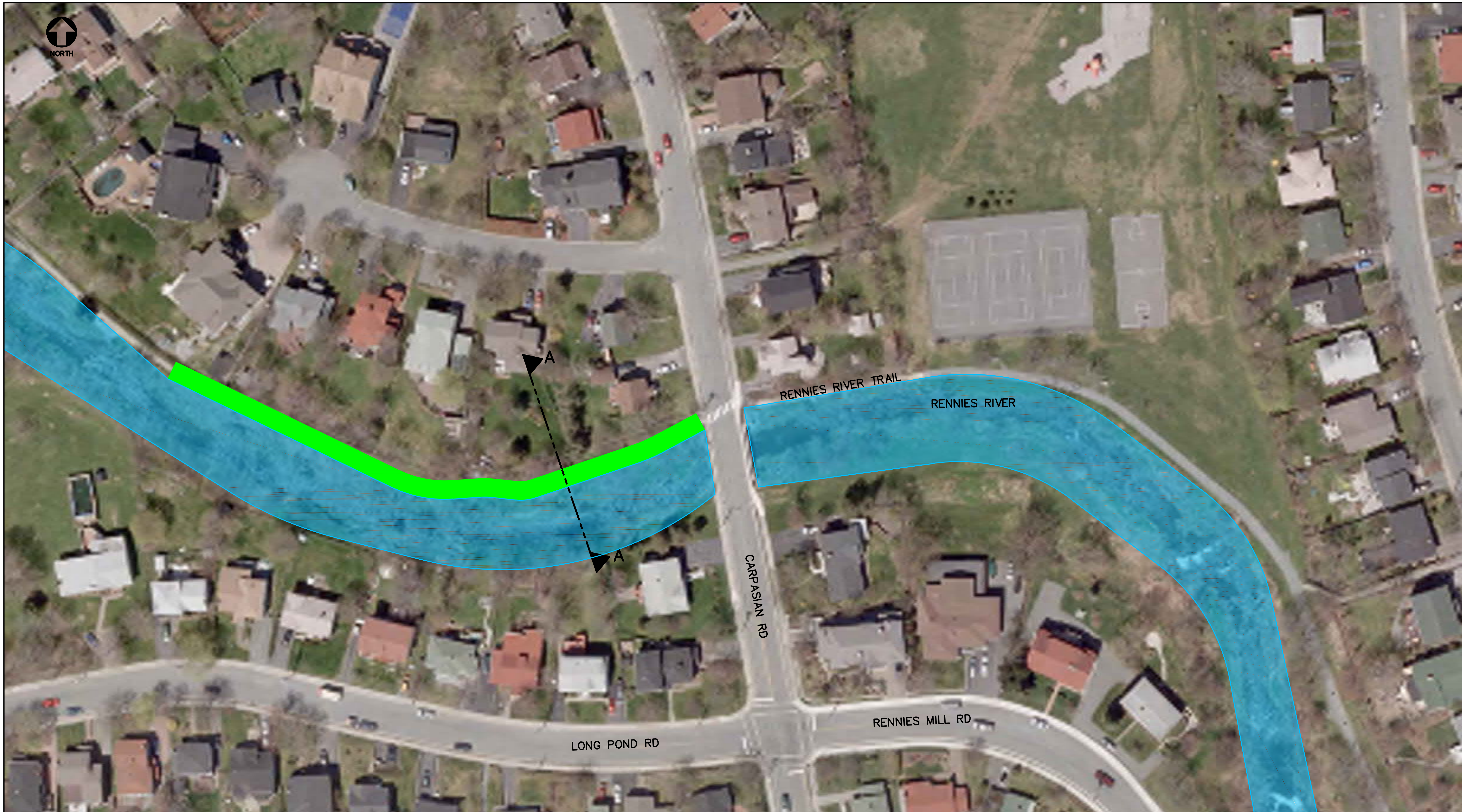


FIGURE 10-10 UPSTREAM OF PORTUGAL COVE ROAD BRIDGE – TRAIL VIEW – BEFORE IMPROVEMENTS

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



PLAN

SCALE: 1:1000

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

LEGEND:

PROPOSED EARTH BERM

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

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

LOCATION 2
UPSTREAM OF CARPASIAN RD

CBCL LIMITED
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187 KENMOUNT ROAD
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Fax: (709) 364-8627

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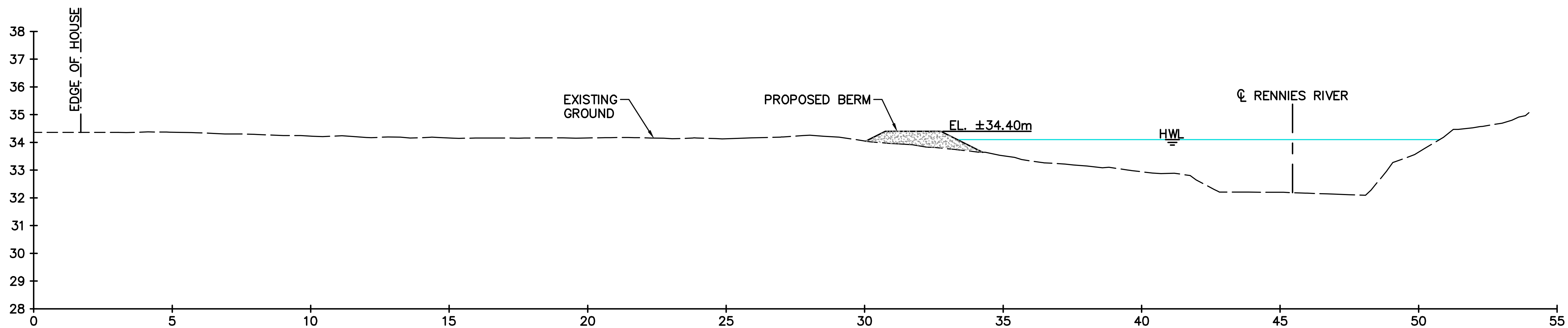


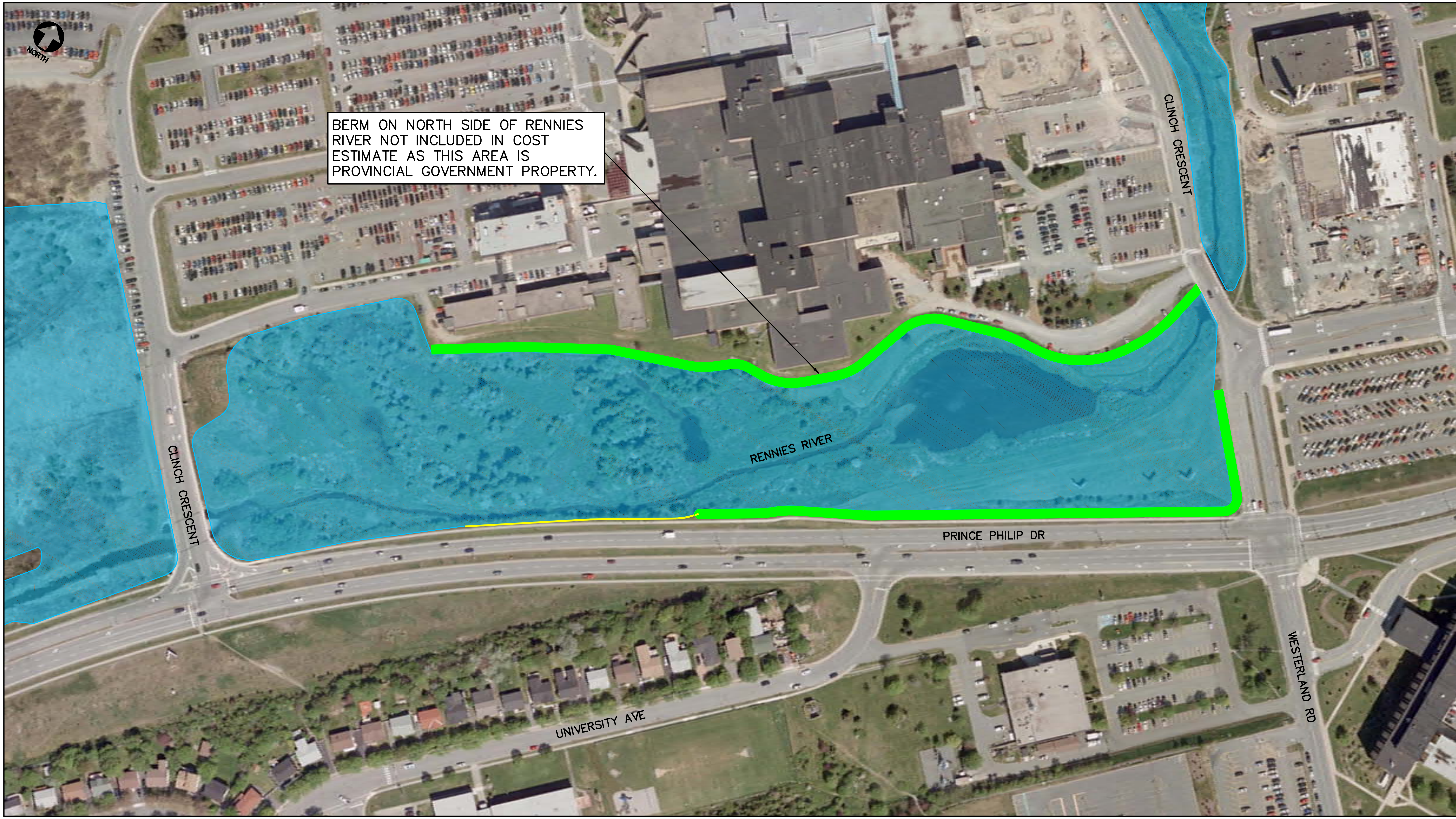
SECTION A - A

SCALE: 1:150

0m 5m 10m

DRAWN:	SCALE:	
J. LARACY	1:150	CITY OF ST. JOHN'S RENNIES RIVER CATCHMENT STORMWATER MANAGEMENT PLAN
PROJECT NO: 123097.00	DATE: APR 15, 2014	
DRAWING NO: **FIGURE 10-13**		LOCATION 2 UPSTREAM OF CARPASIAN RD

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



PLAN

SCALE: 1:2000

0m 50m 100m

LEGEND:

PROPOSED CAST-IN-PLACE CONCRETE WALL

PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: 1:2000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO: FIGURE 10-14	

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

LOCATION 4
CLINCH CRESCENT

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

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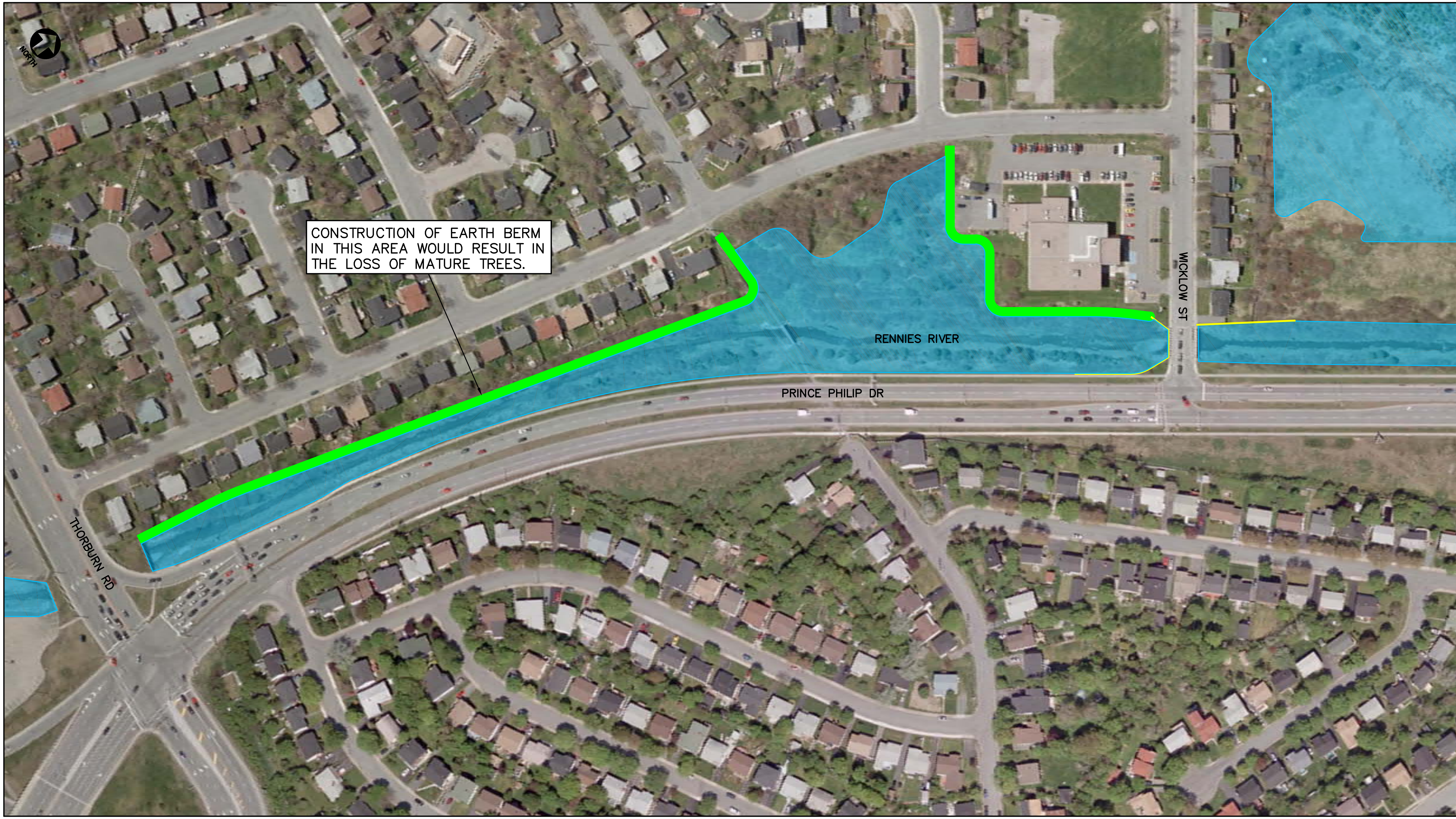

PLAN

SCALE: 1:2000

0m 50m 100m

LEGEND:

PROPOSED CAST-IN-PLACE CONCRETE WALL

PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: 1:2000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO: **FIGURE 10-15**	

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

LOCATION 5
WICKLOW STREET

CBCL LIMITED
Consulting Engineers
187 KENMOUNT ROAD
ST. JOHN'S, NL, A1B 3P9
Phone: (709) 364-8623
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11 X 17

PLAN

SCALE: 1:1000

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

LEGEND:

PROPOSED CAST-IN-PLACE CONCRETE WALL

DRAWN: J. LARACY	SCALE: 1:1000	CITY OF ST. JOHN'S RENNIES RIVER CATCHMENT STORMWATER MANAGEMENT PLAN	CBCL LIMITED Consulting Engineers
PROJECT NO: 123097.00	DATE: APR 15, 2014		
DRAWING NO: FIGURE 10-16		LOCATION 6 AVALON MALL CULVERT	

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

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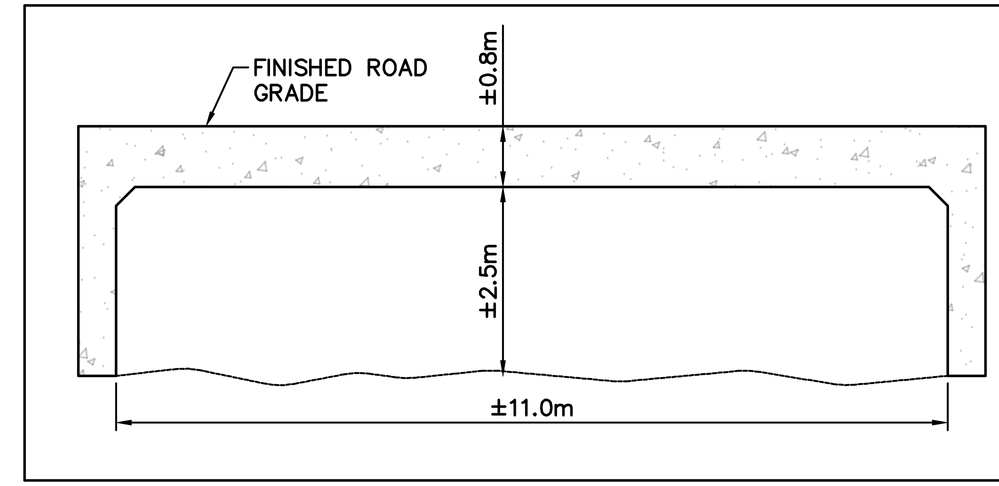
11 X 17

PLAN

SCALE: 1:1000

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



SECTION - NEW BRIDGE

SCALE: 1:100

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

LEGEND:

PROPOSED CAST-IN-PLACE CONCRETE WALL

PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: AS SHOWN
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO: **FIGURE 10-17**	

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

LOCATION 7
O'LEARY AVENUE BRIDGE

CBCL LIMITED
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187 KENMOUNT ROAD
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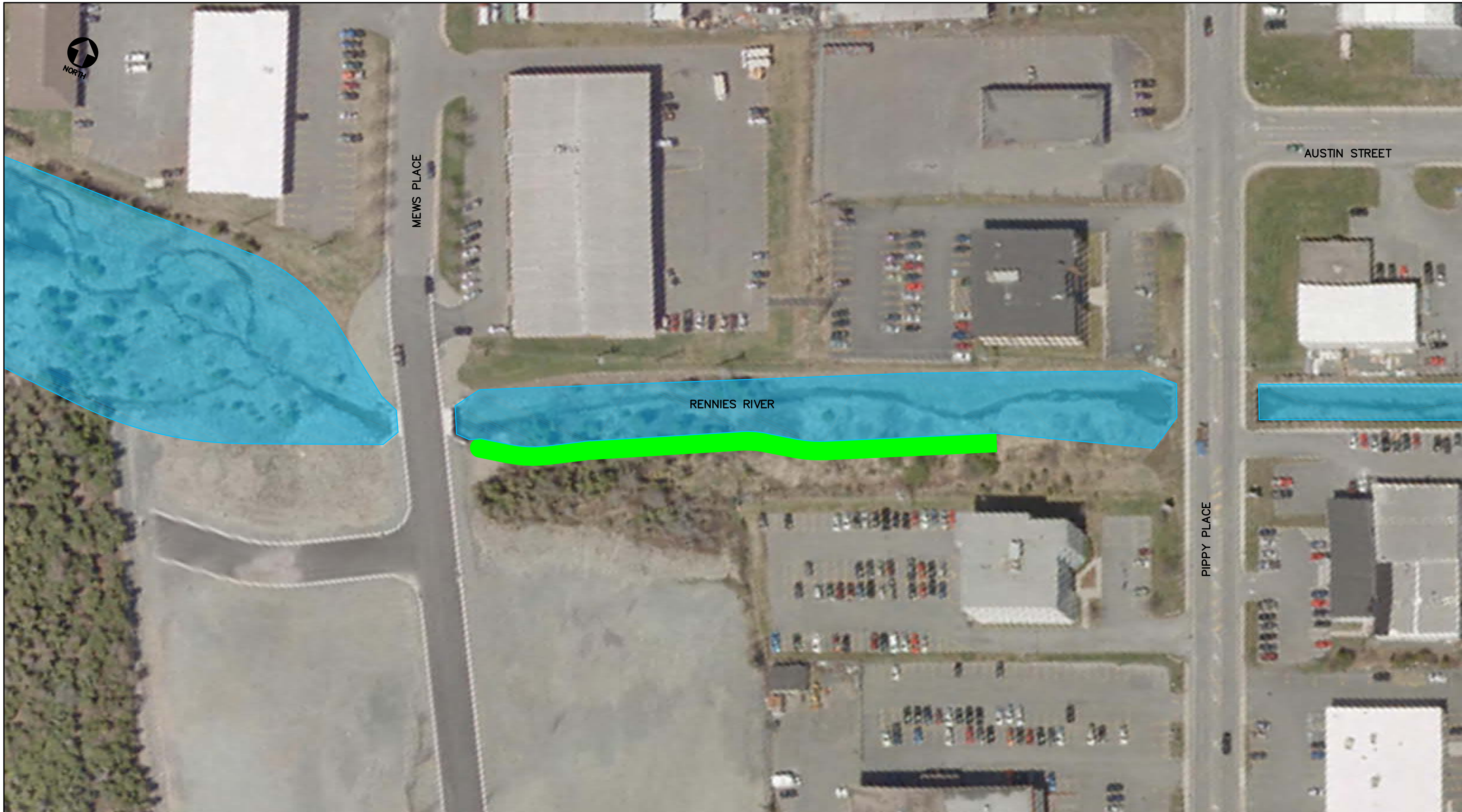
Fax: (709) 364-8627

11 X 17



PLAN

SCALE: 1:1000

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

LEGEND:

PROPOSED EARTH BERM

DRAWN: J. LARACY	SCALE: 1:1000	CITY OF ST. JOHN'S RENNIES RIVER CATCHMENT STORMWATER MANAGEMENT PLAN
PROJECT NO: 123097.00	DATE: APR 15, 2014	
DRAWING NO: **FIGURE 10-18**		LOCATION 8 MEWS PLACE CULVERT

CBCL LIMITED
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ST. JOHN'S, NL, A1B 3P9
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By: JLARACY

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11 X 17

TABLE 10-1 FLOOD PROTECTION IMPROVEMENT OPTIONS

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

Notes:

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

10.2 Erosion Control

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

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

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

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

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

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

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

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

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

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

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

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

PLAN

SCALE: 1:2000

0m 50m 100m

DRAWN: J. LARACY	SCALE: 1:2000
PROJECT NO: 123097.00	DATE: APR 15, 2014
DRAWING NO: **FIGURE 10-19**	

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

EROSION CONTROL IMPROVEMENTS
LOCATION 1

CBCL LIMITED
Consulting Engineers
187 KENMOUNT ROAD
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Phone: (709) 364-8623
Fax: (709) 364-8627

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11 X 17

PLAN

SCALE: 1:3000

0m 50m 100m 150m

DRAWN: J. LARACY	SCALE: 1: 3000	CITY OF ST. JOHN'S RENNIES RIVER CATCHMENT STORMWATER MANAGEMENT PLAN
PROJECT NO: 123097.00	DATE: APR 15, 2014	
DRAWING NO: **FIGURE 10-20**		EROSION CONTROL IMPROVEMENTS LOCATIONS 2-3

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

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11 X 17

PLAN

SCALE: 1:3000

0m 50m 100m 150m

DRAWN: J. LARACY	SCALE: 1: 3000	CITY OF ST. JOHN'S RENNIES RIVER CATCHMENT STORMWATER MANAGEMENT PLAN
PROJECT NO: 123097.00	DATE: APR 15, 2014	
DRAWING NO: **FIGURE 10-21**		EROSION CONTROL IMPROVEMENTS LOCATION 4

CBCL LIMITED
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187 KENMOUNT ROAD
ST. JOHN'S, NL, A1B 3P9
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By: JLARACY Apr 15, 2014 – 8:42am L:\Jobs\2012\123097.00 – City of St. John's – Rennies River Stormwater Management Plan\Drawings\Civil\Deliverables\123097-Erosion-Location Plan.dwg

11 X 17

CHAPTER 11 REGULATORY REQUIREMENTS

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

11.1 Department of Environment and Conservation

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

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

11.2 Department of Fisheries and Oceans

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

CHAPTER 12 CONCLUSIONS AND RECOMMENDATIONS

12.1 Conclusions

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

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

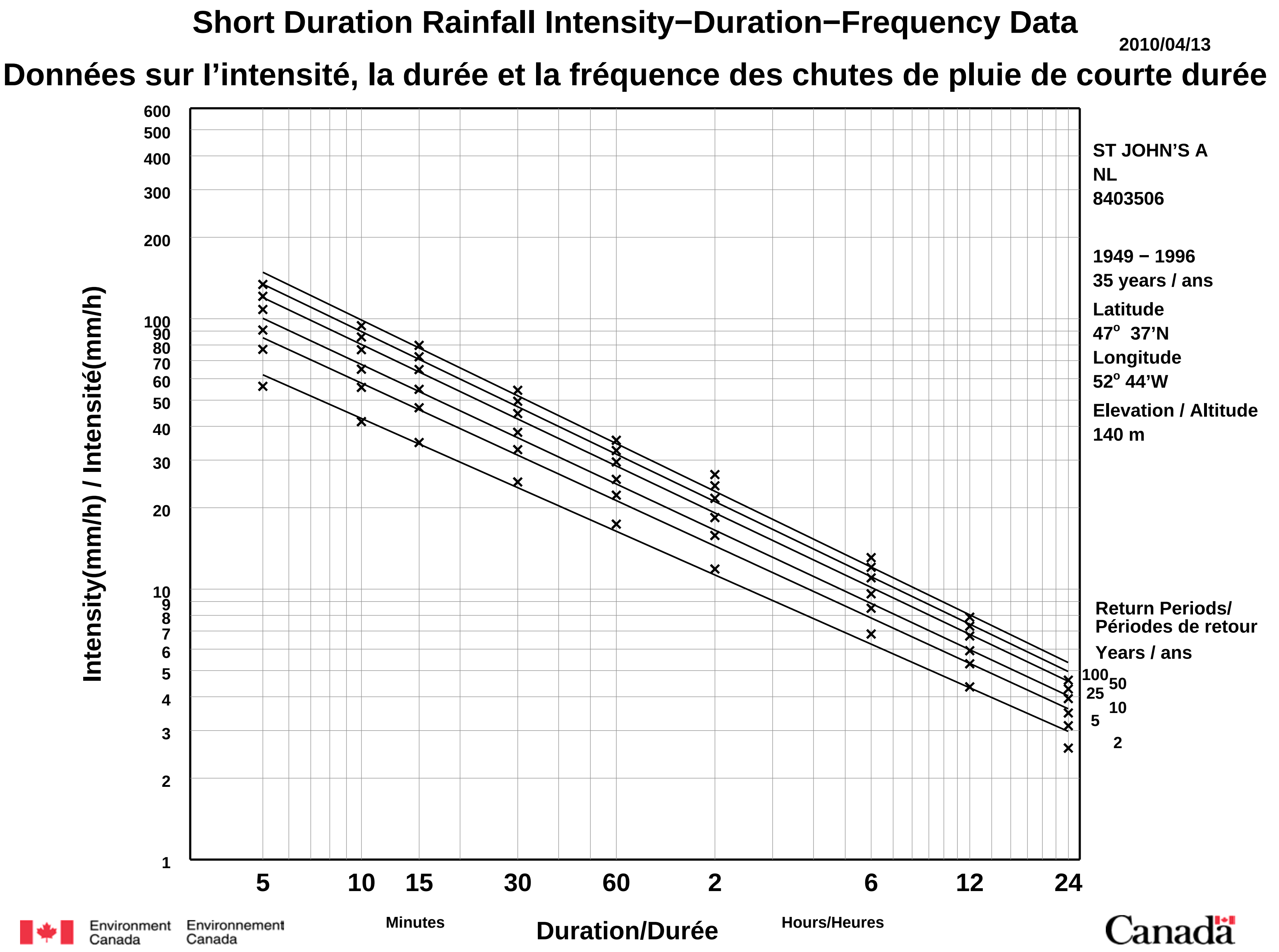
2. Erosion control improvements can be accomplished using a cellular confinement system. It is estimated that approximately 4000 m^2 of the river banks need to be rehabilitated. Based on using a cellular confinement system, the cost opinion to do this work is $567,000.
3. DOEC requires that the Application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

12.2 Recommendations

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

APPENDIX A

IDF Curves for St. John’s Airport Gauge



Environment Canada Environnement Canada

Canada

idf_v2-00_2010_04_13_840_NL_8403506_ST_JOHN_S_A
Environment Canada/Environnement Canada

Short Duration Rainfall Intensity-Duration-Frequency Data
Données sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée

Gumbel - Method of moments/Méthode des moments

2010/04/13

==

ST JOHN'S A NL 8403506

Latitude: 47 37'N Longitude: 52 44'W Elevation/Altitude: 140 m

Years/Années : 1949 - 1996 # Years/Années : 35

==

****

Table 1 : Annual Maximum (mm)/Maximum annuel (mm)

****

Year Année	5 min	10 min	15 min	30 min	1 h	2 h	6 h	12 h	24 h
1949	8.9	8.9	10.2	17.5	28.2	52.6	61.7	62.0	63.5
1961	3.0	4.3	5.3	6.9	8.6	13.5	25.7	35.6	38.6
1962	2.8	4.6	4.6	8.1	13.0	20.6	33.8	54.9	59.7
1963	10.2	11.2	11.7	13.7	18.5	23.6	40.9	52.3	57.9
1964	4.3	6.9	7.9	11.2	19.3	28.2	54.9	72.6	77.5
1965	5.3	7.4	9.9	13.0	17.8	19.6	32.3	51.8	59.7
1966	8.4	13.2	17.0	25.4	29.7	43.7	48.5	64.5	85.3
1967	2.3	3.8	5.3	9.9	10.9	16.3	29.5	44.4	58.4
1968	6.3	12.7	13.7	14.7	17.5	22.4	41.9	55.1	61.7
1969	5.6	7.1	8.4	8.6	11.7	19.0	30.7	34.5	48.3
1970	5.6	7.1	10.7	15.2	16.3	19.6	42.4	62.5	87.4
1971	6.3	10.4	14.5	16.0	19.0	22.1	34.3	41.1	77.7
1972	4.8	5.3	6.6	10.9	15.0	20.6	47.8	72.6	89.2
1973	5.3	6.9	7.9	10.4	16.5	30.0	49.5	65.8	67.1
1974	3.6	5.6	6.3	9.9	16.3	22.4	42.4	53.3	72.9
1975	8.1	10.4	12.2	17.8	19.0	19.6	46.5	71.9	82.3
1976	3.6	4.8	6.1	8.4	12.7	19.0	33.8	42.2	53.6
1977	3.8	5.6	7.6	11.7	17.5	23.4	38.6	40.4	41.4
1978	4.0	5.9	7.4	7.6	12.9	13.1	27.1	37.6	43.0
1979	3.2	4.2	5.9	10.2	16.2	18.1	29.3	41.9	49.2
1980	3.2	6.1	7.4	12.2	17.4	23.9	33.6	41.6	69.8
1981	-99.9	-99.9	-99.9	-99.9	15.0	22.4	46.7	72.5	82.6
1982	5.1	9.0	12.9	17.1	24.5	35.9	80.3	82.4	84.0
1983	1.6	3.2	4.8	9.6	19.2	26.5	47.3	52.8	54.7
1984	5.0	9.9	13.0	21.5	27.1	36.6	61.0	74.0	75.3
1985	5.2	7.1	9.8	11.3	14.1	18.5	36.0	54.9	82.9
1986	3.1	4.8	7.2	14.3	23.3	27.9	40.2	58.9	70.6
1987	5.1	7.3	8.6	16.2	23.5	24.2	30.6	36.6	46.8
1988	6.6	10.6	13.2	17.4	23.4	25.9	44.8	45.8	49.0
1989	2.9	4.5	6.2	8.0	10.9	19.7	43.4	51.6	51.6
1990	3.7	5.9	6.5	12.6	19.2	28.5	48.1	68.7	85.2
1991	7.8	11.4	15.9	23.3	28.8	29.5	51.2	52.2	59.7
1993	4.4	7.0	7.6	11.5	20.0	31.3	47.6	49.4	55.3
1994	6.2	9.1	10.3	12.6	12.8	14.9	-99.5	-99.5	67.5
1995	5.2	9.8	14.5	16.6	27.6	46.7	55.9	58.8	61.6

1996	4.8	6.2	7.4	10.2	15.4	27.2	40.2	44.0	48.4
# Yrs. Années	35	35	35	35	36	36	35	35	36
Mean Moyenne	5.0	7.4	9.3	13.2	18.3	25.2	42.8	54.3	64.4
Std. Dev. Écart-type	2.0	2.7	3.4	4.5	5.5	8.9	11.4	12.8	14.7
Skew. Dissymétrie	0.75	0.52	0.64	0.95	0.52	1.42	1.05	0.32	0.11
Kurtosis	3.58	2.56	2.57	3.85	2.77	5.29	5.24	2.40	2.06

*-99.9 Indicates Missing Data/Données manquantes

Warning: annual maximum amount greater than 100-yr return period amount
Avertissement : la quantité maximale annuelle excède la quantité pour une période de retour de 100 ans

Year/Année	Duration/Durée	Data/Données	100-yr/ans
1982	6 h	80.3	78.5

****

Table 2a : Return Period Rainfall Amounts (mm)
Quantité de pluie (mm) par période de retour

****

Duration/Durée	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans	#Years Années
5 min	4.7	6.4	7.6	9.0	10.1	11.2	35
10 min	6.9	9.3	10.8	12.8	14.3	15.7	35
15 min	8.7	11.7	13.7	16.2	18.1	19.9	35
30 min	12.5	16.4	19.0	22.3	24.8	27.2	35
1 h	17.4	22.3	25.5	29.5	32.5	35.5	36
2 h	23.7	31.6	36.8	43.3	48.2	53.1	36
6 h	40.9	51.0	57.6	66.1	72.3	78.5	35
12 h	52.2	63.5	71.0	80.5	87.6	94.5	35
24 h	62.0	75.0	83.6	94.5	102.6	110.6	36

****

Table 2b :

Return Period Rainfall Rates (mm/h) - 95% Confidence limits
Intensité de la pluie (mm/h) par période de retour - Limites de confiance de 95%

****

Duration/Durée	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans	#Years Années
5 min	56.2	77.0	90.8	108.2	121.1	133.9	35
	+/- 7.2	+/- 12.1	+/- 16.3	+/- 22.0	+/- 26.3	+/- 30.6	35
10 min	41.6	55.7	65.0	76.8	85.6	94.2	35
	+/- 4.8	+/- 8.2	+/- 11.0	+/- 14.9	+/- 17.8	+/- 20.7	35
15 min	34.9	46.9	54.8	64.9	72.3	79.7	35
	+/- 4.1	+/- 7.0	+/- 9.4	+/- 12.7	+/- 15.2	+/- 17.7	35
30 min	24.9	32.8	38.0	44.6	49.5	54.4	35
	+/- 2.7	+/- 4.6	+/- 6.2	+/- 8.3	+/- 10.0	+/- 11.6	35
1 h	17.4	22.3	25.5	29.5	32.5	35.5	36
	+/- 1.6	+/- 2.8	+/- 3.7	+/- 5.1	+/- 6.0	+/- 7.0	36
2 h	11.9	15.8	18.4	21.7	24.1	26.5	36
	+/- 1.3	+/- 2.2	+/- 3.0	+/- 4.1	+/- 4.9	+/- 5.7	36
6 h	6.8	8.5	9.6	11.0	12.0	13.1	35

	+/- 0.6	+/- 1.0	+/- 1.3	+/- 1.8	+/- 2.1	+/- 2.5	35
12 h	4.4	5.3	5.9	6.7	7.3	7.9	35
	+/- 0.3	+/- 0.5	+/- 0.7	+/- 1.0	+/- 1.2	+/- 1.4	35
24 h	2.6	3.1	3.5	3.9	4.3	4.6	36
	+/- 0.2	+/- 0.3	+/- 0.4	+/- 0.6	+/- 0.7	+/- 0.8	36

****

Table 3 : Interpolation Equation / Équation d'interpolation: R = A*T^B

R = Interpolated Rainfall rate (mm/h)/Intensité interpolée de la pluie (mm/h)
RR = Rainfall rate (mm/h) / Intensité de la pluie (mm/h)
T = Rainfall duration (h) / Durée de la pluie (h)

****

Statistics/Statistiques	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans
Mean of RR/Moyenne de RR	22.3	29.7	34.6	40.8	45.4	50.0
Std. Dev. /Écart-type (RR)	18.6	25.6	30.1	35.9	40.3	44.5
Std. Error/Erreur-type	2.3	3.3	3.9	4.8	5.4	6.1
Coefficient (A)	16.4	21.3	24.5	28.5	31.6	34.5
Exponent/Exposant (B)	-0.536	-0.558	-0.568	-0.577	-0.583	-0.587
Mean % Error/% erreur moyenne	6.0	6.3	6.6	6.9	7.0	7.2

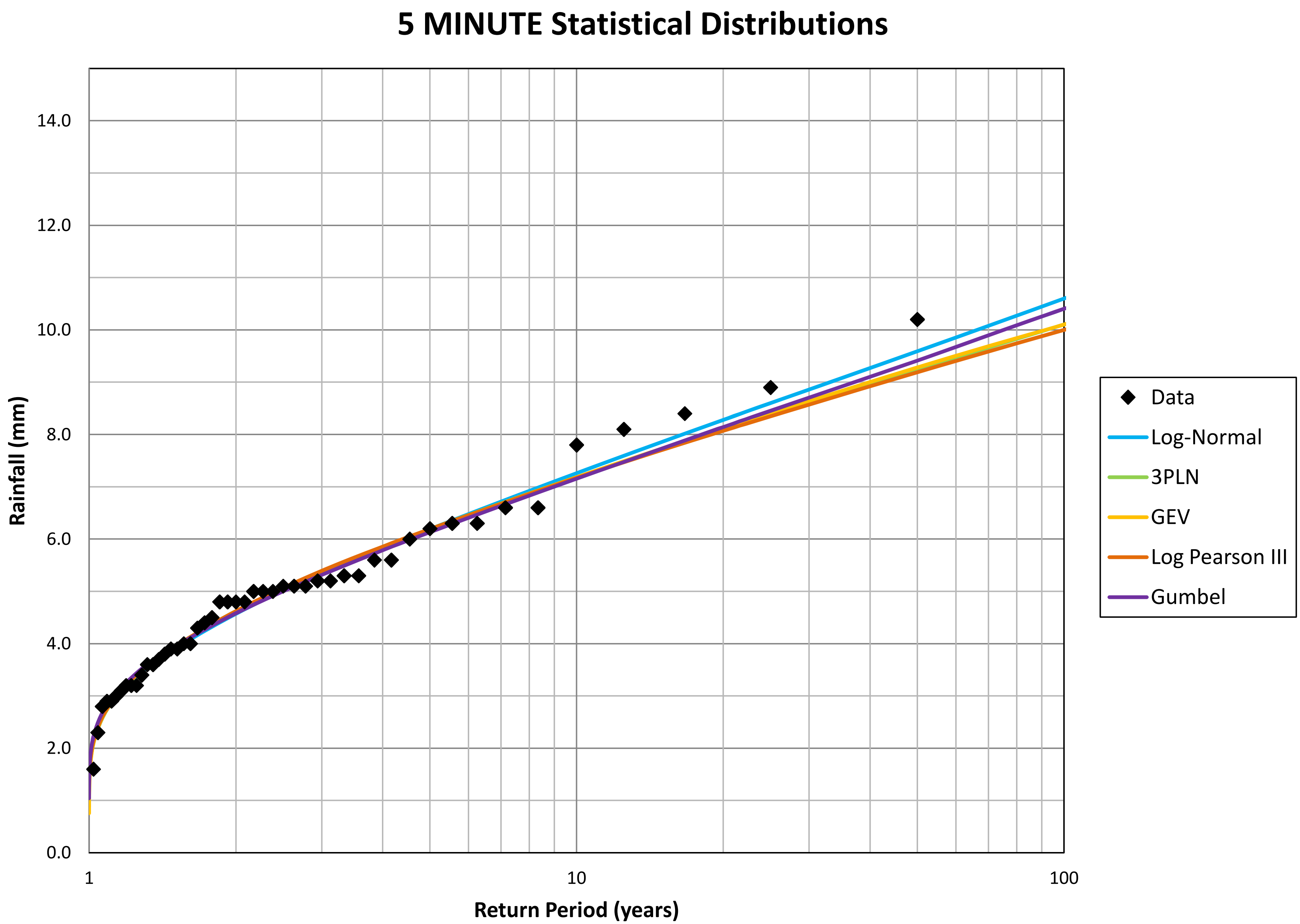
APPENDIX B

Annual Maxima Data for Windsor Lake Gauge

YEAR	Annual Maximum Rainfall (mm)								
	5 MIN	10 MIN	15 MIN	30 MIN	1 HR	2 HR	6 HR	12 HR	24 HR
1949	8.9	8.9	10.2	17.5	28.2	52.6	61.7	62.0	63.5
1961	3.0	4.3	5.3	6.9	8.6	13.5	25.7	35.6	38.6
1962	2.8	4.6	4.6	8.1	13.0	20.6	33.8	54.9	59.7
1963	10.2	11.2	11.7	13.7	18.5	23.6	40.9	52.3	57.9
1964	4.3	6.9	7.9	11.2	19.3	28.2	54.9	72.6	77.5
1965	5.3	7.4	9.9	13.0	17.8	19.6	32.3	51.8	59.7
1966	8.4	13.2	17.0	25.4	29.7	43.7	48.5	64.5	85.3
1967	2.3	3.8	5.3	9.9	10.9	16.3	29.5	44.4	58.4
1968	6.3	12.7	13.7	14.7	17.5	22.4	41.9	55.1	61.7
1969	5.6	7.1	8.4	8.6	11.7	19.0	30.7	34.5	48.3
1970	5.6	7.1	10.7	15.2	16.3	19.6	42.4	62.5	87.4
1971	6.3	10.4	14.5	16.0	19.0	22.1	34.3	41.1	77.7
1972	4.8	5.3	6.6	10.9	15.0	20.6	47.8	72.6	89.2
1973	5.3	6.9	7.9	10.4	16.5	30.0	49.5	65.8	67.1
1974	3.6	5.6	6.3	9.9	16.3	22.4	42.4	53.3	72.9
1975	8.1	10.4	12.2	17.8	19.0	19.6	46.5	71.9	82.3
1976	3.6	4.8	6.1	8.4	12.7	19.0	33.8	42.2	53.6
1977	3.8	5.6	7.6	11.7	17.5	23.4	38.6	40.4	41.4
1978	4.0	5.9	7.4	7.6	12.9	13.1	27.1	37.6	43.0
1979	3.2	4.2	5.9	10.2	16.2	18.1	29.3	41.9	49.2
1980	3.2	6.1	7.4	12.2	17.4	23.9	33.6	41.6	69.8
1981	N/A	N/A	N/A	N/A	15.0	22.4	46.7	72.5	82.6
1982	5.1	9.0	12.9	17.1	24.5	35.9	80.3	82.4	84.0
1983	1.6	3.2	4.8	9.6	19.2	26.5	47.3	52.8	54.7
1984	5.0	9.9	13.0	21.5	27.1	36.6	61.0	74.0	75.3
1985	5.2	7.1	9.8	11.3	14.1	18.5	36.0	54.9	82.9
1986	3.1	4.8	7.2	14.3	23.3	27.9	40.2	58.9	70.6
1987	5.1	7.3	8.6	16.2	23.5	24.2	30.6	36.6	46.8
1988	6.6	10.6	13.2	17.4	23.4	25.9	44.8	45.8	49.0
1989	2.9	4.5	6.2	8.0	10.9	19.7	43.4	51.6	51.6
1990	3.7	5.9	6.5	12.6	19.2	28.5	48.1	68.7	85.2
1991	7.8	11.4	15.9	23.3	28.8	29.5	51.2	52.2	59.7
1993	4.4	7.0	7.6	11.5	20.0	31.3	47.6	49.4	55.3
1994	6.2	9.1	10.3	12.6	12.8	14.9	N/A	N/A	67.5
1995	5.2	9.8	14.5	16.6	27.6	46.7	55.9	58.8	61.6
1996	4.8	6.2	7.4	10.2	15.4	27.2	40.2	44.0	48.4
1999	3.2	5.1	6.6	8.9	15.2	25.3	42.1	63.1	99.6
2000	4	7.3	9.1	13.3	21.9	29.9	43.3	59	70.5
2001	5	9.5	11.9	19.6	33.7	61.9	107.3	147.7	149.6
2002	3.9	7.6	10.3	16.9	21.2	27.2	45.9	47.8	49.6
2003	6	10	13.8	20.3	34	42.3	50	75.9	92.5
2004	3.9	7.5	10.9	17.2	23.8	26.7	59.1	71.6	76.6
2005	5.1	7.1	8.4	13.2	21.4	28.8	65.4	82.3	98.9
2006	4.8	8.5	11.3	18.1	31	36.7	51.9	53.7	58.5
2007	6.6	11.2	15.5	28.3	42	48.3	79.3	104.2	104.9
2008	4.5	7.4	9.8	14.1	17.1	27.1	49.7	57.3	62.4
2009	5	6.8	7.6	11.2	17.1	24.9	46.7	58.2	65
2010	4.8	8.7	12.7	21.2	33.6	57.3	107.8	139.1	180
2011	2.9	4.5	4.9	8.1	13.3	20.3	33.4	38.3	44.2
2012	3.4	5.7	7.3	10.3	13	17.6	32.2	36.7	58.1

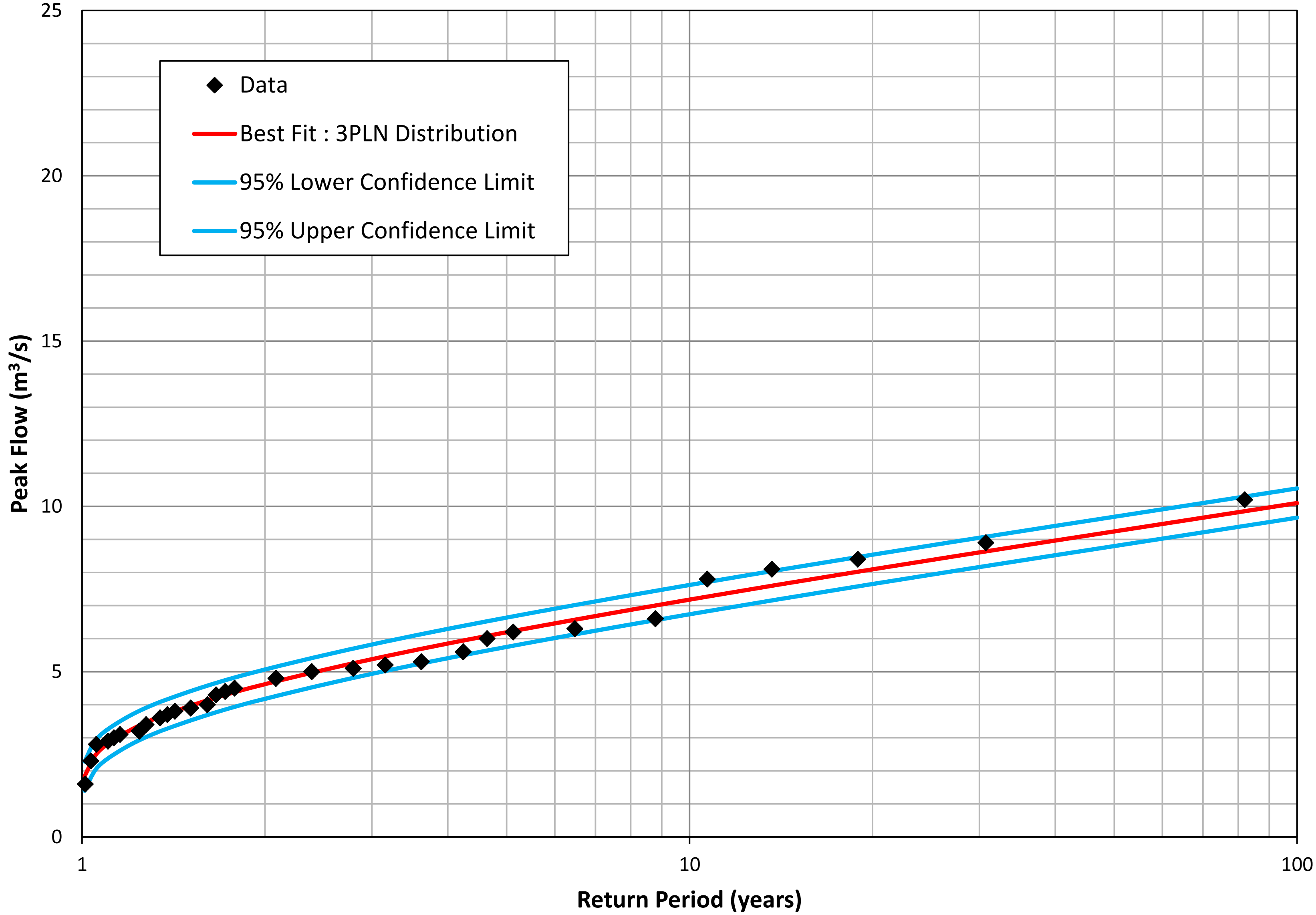
APPENDIX C

Distribution Plots and Screening Tests for Winsor Lake Gauge



5 MINUTE Statistical Distributions
Rainfall (mm)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
1
10
100
Return Period (years)
Data
Log-Normal
3PLN
GEV
Log Pearson III
Gumbel

Statistical Analysis of 5 Min Rainfall Data



--- SPEARMAN TEST FOR INDEPENDENCE ---

0005MIN 5 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER SERIAL CORRELATION COEFF = -.183 D.F.= 43

CORRESPONDS TO STUDENTS T =-1.223

CRITICAL T VALUE AT 5% LEVEL = 1.682 NOT SIGNIFICANT

- - - - 1% - = 2.418 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant serial dependence.

--- SPEARMAN TEST FOR TREND ---

0005MIN 5 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER CORRELATION COEFF = .134 D.F.= 47

CORRESPONDS TO STUDENTS T = .925

CRITICAL T VALUE AT 5% LEVEL = 2.014 NOT SIGNIFICANT

- - - - 1% - = 2.689 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the serial(lag-one) correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant trend.

--- RUN TEST FOR GENERAL RANDOMNESS ---

0005MIN 5 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

THE NUMBER OF RUNS ABOVE AND BELOW THE MEDIAN (RUNAB) = 28

THE NUMBER OF OBSERVATIONS ABOVE THE MEDIAN(N1) = 22

THE NUMBER OF OBSERVATIONS BELOW THE MEDIAN(N2) = 23

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

For this test, Z = 1.361

Critical Z value at the 5% level = 1.960 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the data are random.

At the 5% level of significance, the null hypothesis cannot be rejected. That is, the sample is significantly random.

--- MANN-WHITNEY SPLIT SAMPLE TEST FOR HOMOGENEITY ---

0005MIN 5 Minute Rainfall Data

ANNUAL MAXIMUM FLOW SERIES 1960 TO 2012 DRAINAGE AREA= .0000000

SPLIT BY TIME SPAN, SUBSAMPLE 1 SAMPLE SIZE= 24

SUBSAMPLE 2 SAMPLE SIZE= 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

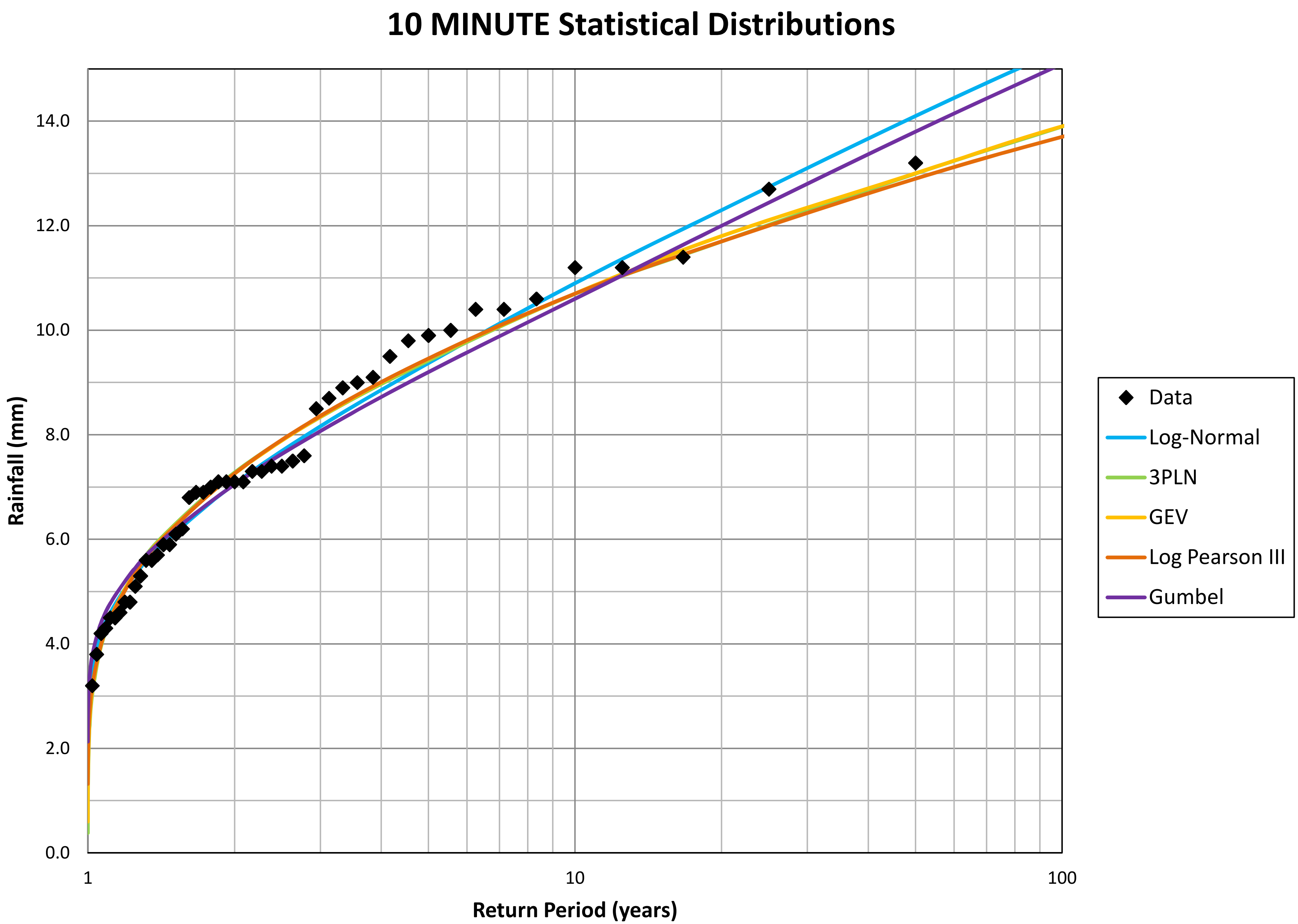
For this test, Z = -.220

CRITICAL Z VALUE AT 5% SIGNIFICANT LEVEL = -1.645 NOT SIGNIFICANT

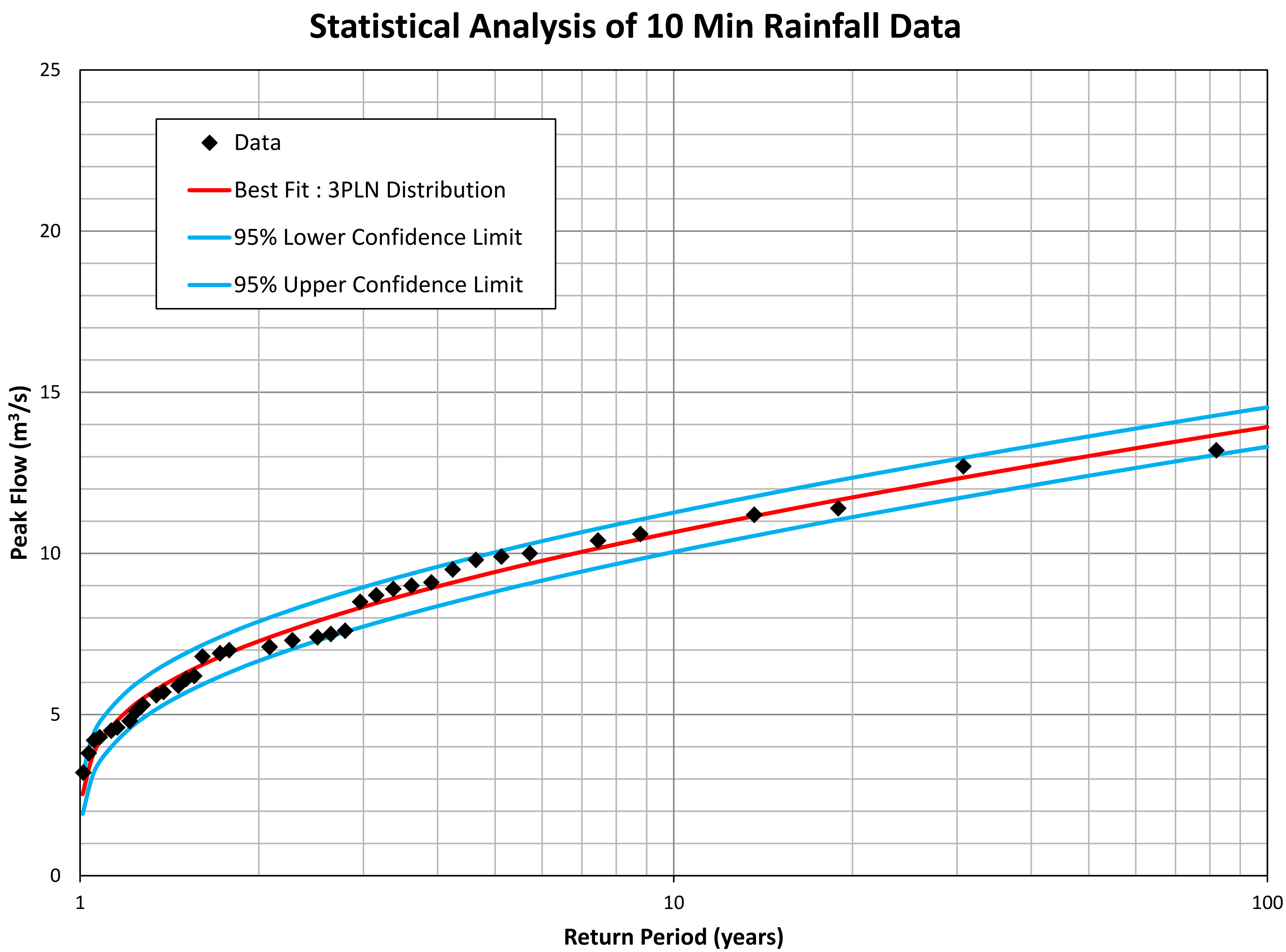
\- - - - 1% - - = -2.326 NOT SIGNIFICANT

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 5% level of significance, there is no significant location difference between the two samples. That is, they appear to be from the same population.



10 MINUTE Statistical Distributions
Rainfall (mm)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
1
10
100
Return Period (years)
Data
Log-Normal
3PLN
GEV
Log Pearson III
Gumbel


Statistical Analysis of 10 Min Rainfall Data
Data
Best Fit : 3PLN Distribution
95% Lower Confidence Limit
95% Upper Confidence Limit
Peak Flow (m³/s)
Return Period (years)
0
5
10
15
20
25
1
10
100

--- SPEARMAN TEST FOR INDEPENDENCE ---

0010MIN 10 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER SERIAL CORRELATION COEFF = -.196 D.F.= 43

CORRESPONDS TO STUDENTS T =-1.310

CRITICAL T VALUE AT 5% LEVEL = 1.682 NOT SIGNIFICANT

- - - - 1% - = 2.418 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant serial dependence.

--- SPEARMAN TEST FOR TREND ---

0010MIN 10 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER CORRELATION COEFF = -.084 D.F.= 47

CORRESPONDS TO STUDENTS T = -.576

CRITICAL T VALUE AT 5% LEVEL =-2.014 NOT SIGNIFICANT

- - - - 1% - =-2.689 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the serial(lag-one) correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant trend.

--- RUN TEST FOR GENERAL RANDOMNESS ---

0010MIN 10 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

THE NUMBER OF RUNS ABOVE AND BELOW THE MEDIAN (RUNAB) = 25

THE NUMBER OF OBSERVATIONS ABOVE THE MEDIAN(N1) = 22

THE NUMBER OF OBSERVATIONS BELOW THE MEDIAN(N2) = 23

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

For this test, Z = .456

Critical Z value at the 5% level = 1.960 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the data are random.

At the 5% level of significance, the null hypothesis cannot be rejected. That is, the sample is significantly random.

--- MANN-WHITNEY SPLIT SAMPLE TEST FOR HOMOGENEITY ---

0010MIN 10 Minute Rainfall Data

ANNUAL MAXIMUM FLOW SERIES 1960 TO 2012 DRAINAGE AREA= .0000000

SPLIT BY TIME SPAN, SUBSAMPLE 1 SAMPLE SIZE= 24

SUBSAMPLE 2 SAMPLE SIZE= 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

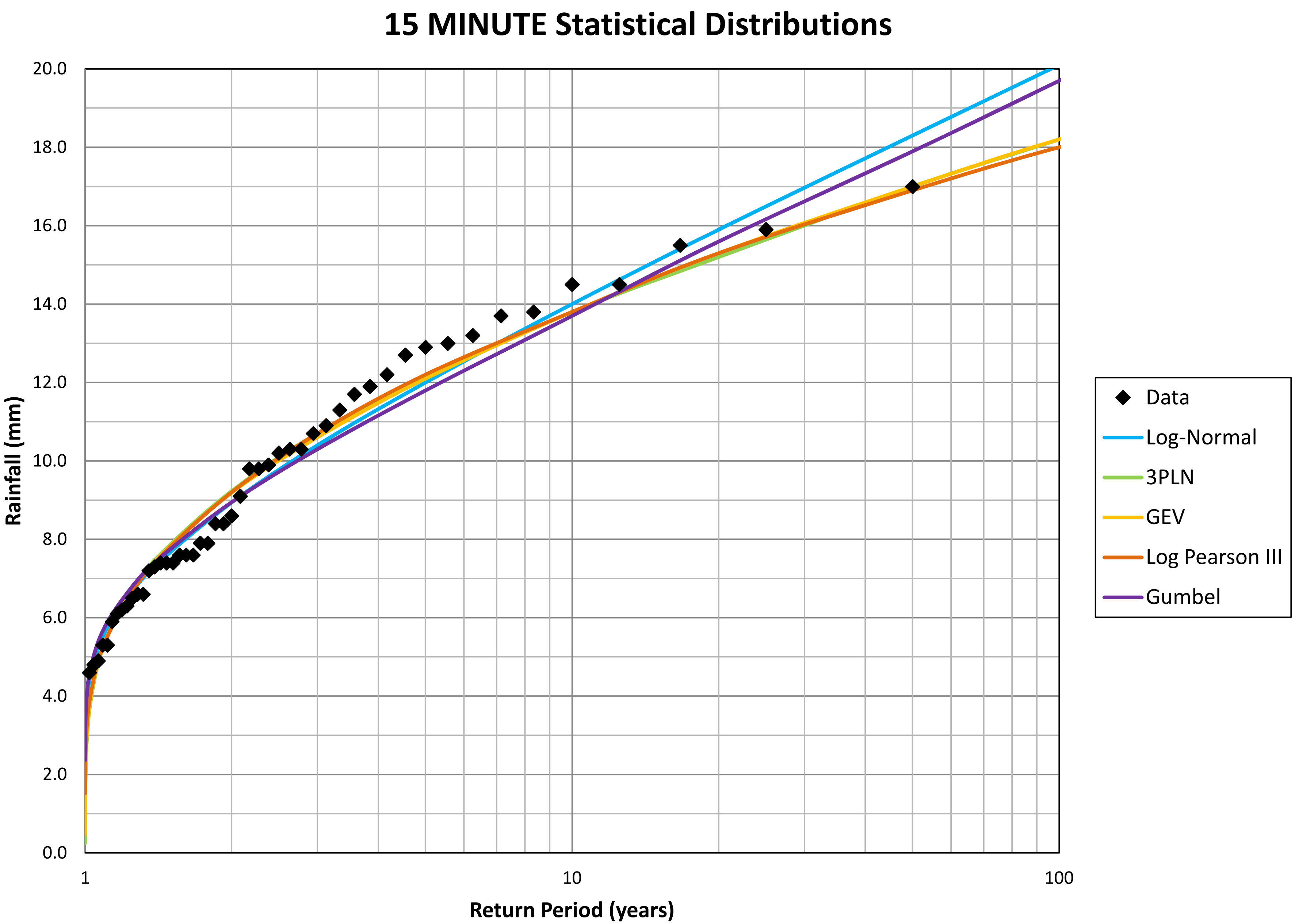
For this test, Z = -.960

CRITICAL Z VALUE AT 5% SIGNIFICANT LEVEL = -1.645 NOT SIGNIFICANT

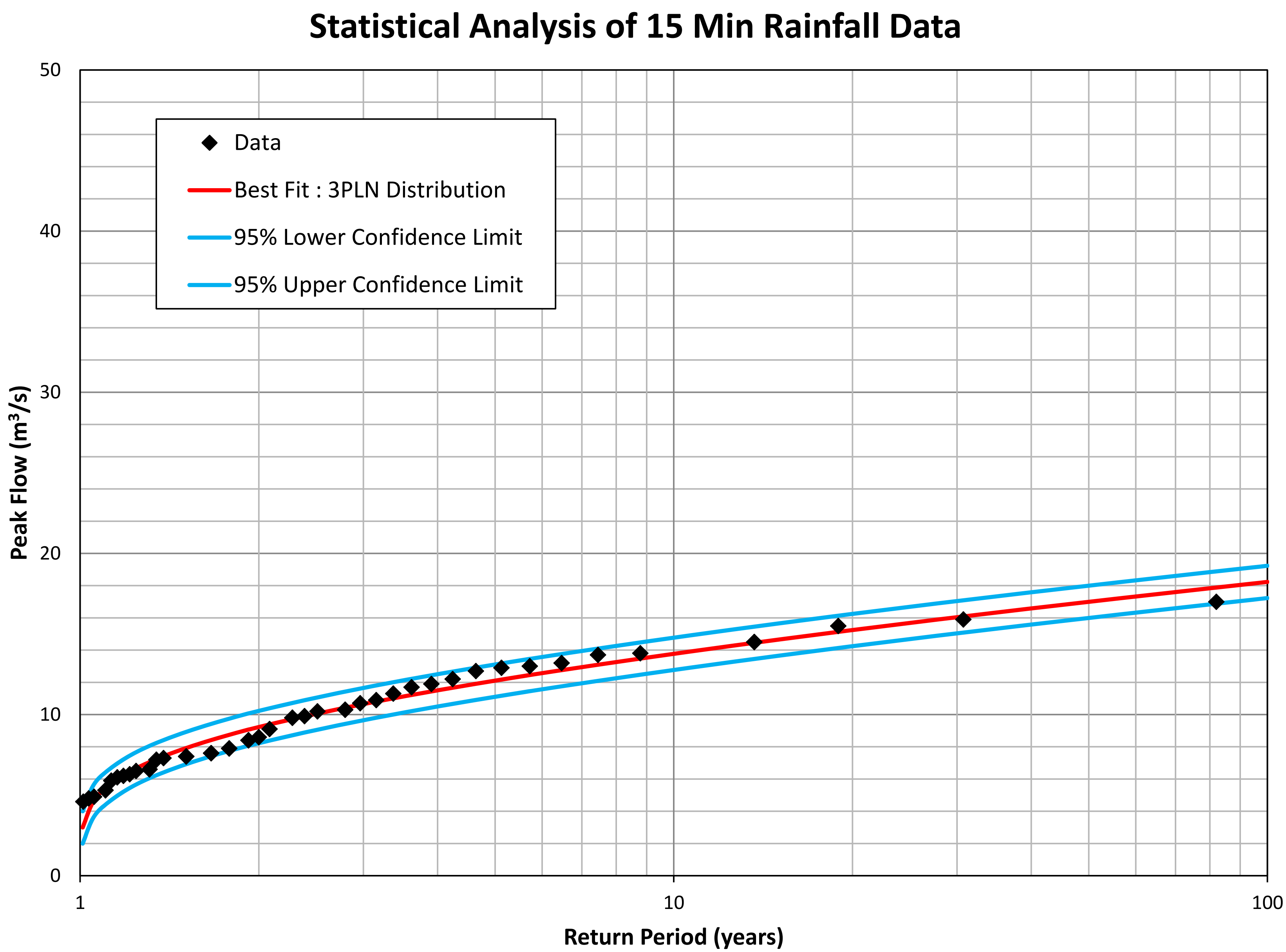
- - - - 1% - - = -2.326 NOT SIGNIFICANT

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 5% level of significance, there is no significant location difference between the two samples. That is, they appear to be from the same population.



15 MINUTE Statistical Distributions
Rainfall (mm)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
1
10
100
Return Period (years)
Data
Log-Normal
3PLN
GEV
Log Pearson III
Gumbel



Statistical Analysis of 15 Min Rainfall Data
Data
Best Fit : 3PLN Distribution
95% Lower Confidence Limit
95% Upper Confidence Limit
Peak Flow (m³/s)
Return Period (years)
0
10
20
30
40
50
1
10
100

--- SPEARMAN TEST FOR INDEPENDENCE ---

0015MIN 15 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER SERIAL CORRELATION COEFF = -.160 D.F.= 43

CORRESPONDS TO STUDENTS T =-1.065

CRITICAL T VALUE AT 5% LEVEL = 1.682 NOT SIGNIFICANT

- - - - 1% - = 2.418 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant serial dependence.

--- SPEARMAN TEST FOR TREND ---

0015MIN 15 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER CORRELATION COEFF = -.125 D.F.= 47

CORRESPONDS TO STUDENTS T = -.866

CRITICAL T VALUE AT 5% LEVEL =-2.014 NOT SIGNIFICANT

- - - - 1% - =-2.689 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the serial(lag-one) correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant trend.

--- RUN TEST FOR GENERAL RANDOMNESS ---

0015MIN 15 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

THE NUMBER OF RUNS ABOVE AND BELOW THE MEDIAN (RUNAB) = 28

THE NUMBER OF OBSERVATIONS ABOVE THE MEDIAN(N1) = 24

THE NUMBER OF OBSERVATIONS BELOW THE MEDIAN(N2) = 24

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

For this test, Z = .875

Critical Z value at the 5% level = 1.960 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the data are random.

At the 5% level of significance, the null hypothesis cannot be rejected. That is, the sample is significantly random.

--- MANN-WHITNEY SPLIT SAMPLE TEST FOR HOMOGENEITY ---

0015MIN 15 Minute Rainfall Data

ANNUAL MAXIMUM FLOW SERIES 1960 TO 2012 DRAINAGE AREA= .0000000

SPLIT BY TIME SPAN, SUBSAMPLE 1 SAMPLE SIZE= 24

SUBSAMPLE 2 SAMPLE SIZE= 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

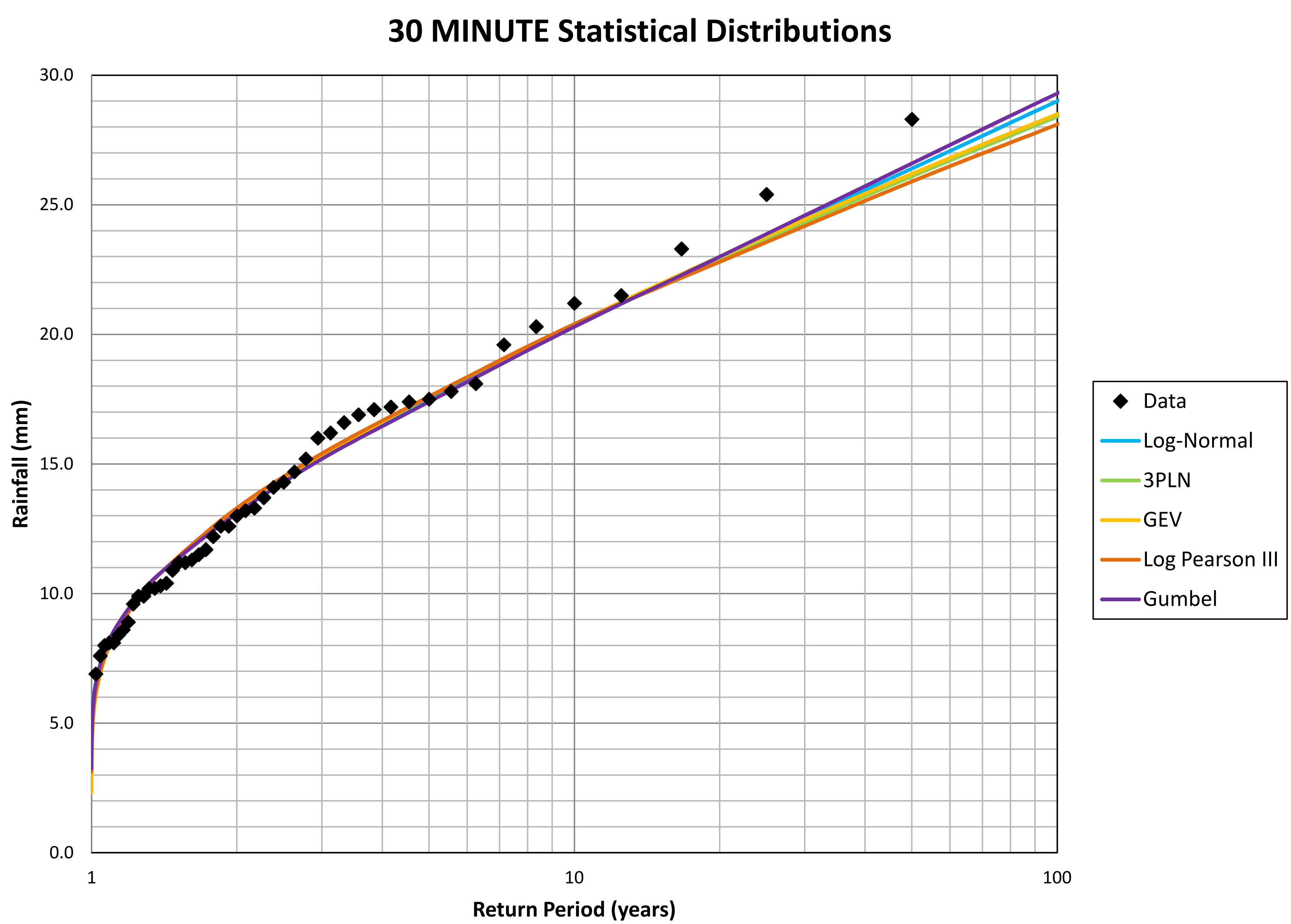
For this test, Z = -1.050

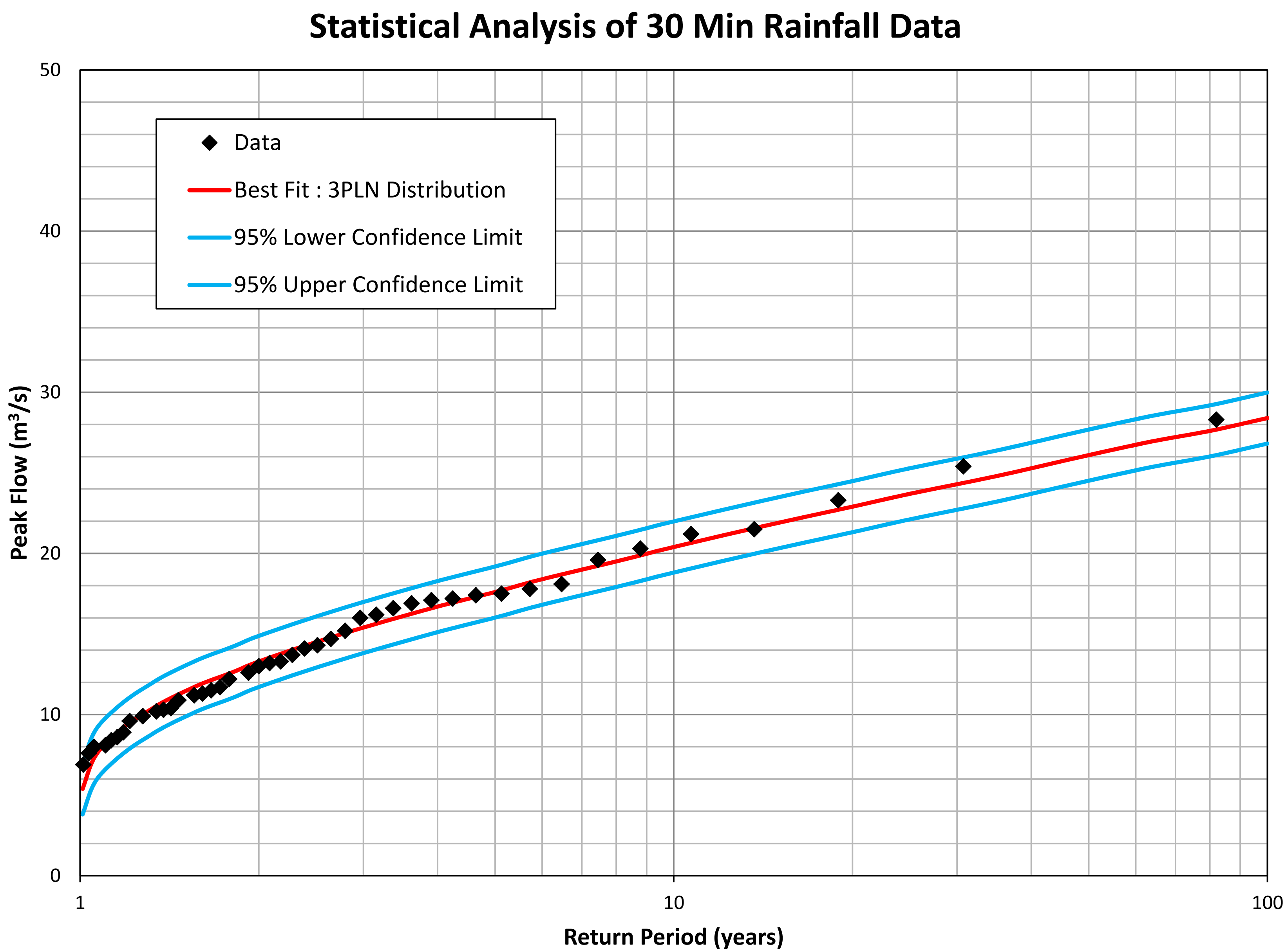
CRITICAL Z VALUE AT 5% SIGNIFICANT LEVEL = -1.645 NOT SIGNIFICANT

- - - - 1% - - = -2.326 NOT SIGNIFICANT

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 5% level of significance, there is no significant location difference between the two samples. That is, they appear to be from the same population.


30 MINUTE Statistical Distributions
Rainfall (mm)
30.0
25.0
20.0
15.0
10.0
5.0
0.0
1
10
100
Return Period (years)
Data
Log-Normal
3PLN
GEV
Log Pearson III
Gumbel


Statistical Analysis of 30 Min Rainfall Data
Data
Best Fit : 3PLN Distribution
95% Lower Confidence Limit
95% Upper Confidence Limit
Peak Flow (m³/s)
Return Period (years)
0
10
20
30
40
50
1
10
100

--- SPEARMAN TEST FOR INDEPENDENCE ---

0030MIN 30 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER SERIAL CORRELATION COEFF = -.062 D.F.= 43

CORRESPONDS TO STUDENTS T = -.406

CRITICAL T VALUE AT 5% LEVEL = 1.682 NOT SIGNIFICANT

- - - - 1% - = 2.418 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant serial dependence.

--- SPEARMAN TEST FOR TREND ---

0030MIN 30 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER CORRELATION COEFF = -.231 D.F.= 47

CORRESPONDS TO STUDENTS T =-1.628

CRITICAL T VALUE AT 5% LEVEL =-2.014 NOT SIGNIFICANT

- - - - 1% - =-2.689 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the serial(lag-one) correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant trend.

--- RUN TEST FOR GENERAL RANDOMNESS ---

0030MIN 30 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

THE NUMBER OF RUNS ABOVE AND BELOW THE MEDIAN (RUNAB) = 26

THE NUMBER OF OBSERVATIONS ABOVE THE MEDIAN(N1) = 24

THE NUMBER OF OBSERVATIONS BELOW THE MEDIAN(N2) = 24

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

For this test, Z = .292

Critical Z value at the 5% level = 1.960 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the data are random.

At the 5% level of significance, the null hypothesis cannot be rejected. That is, the sample is significantly random.

--- MANN-WHITNEY SPLIT SAMPLE TEST FOR HOMOGENEITY ---

0030MIN 30 Minute Rainfall Data

ANNUAL MAXIMUM FLOW SERIES 1960 TO 2012 DRAINAGE AREA= .0000000

SPLIT BY TIME SPAN, SUBSAMPLE 1 SAMPLE SIZE= 24

SUBSAMPLE 2 SAMPLE SIZE= 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

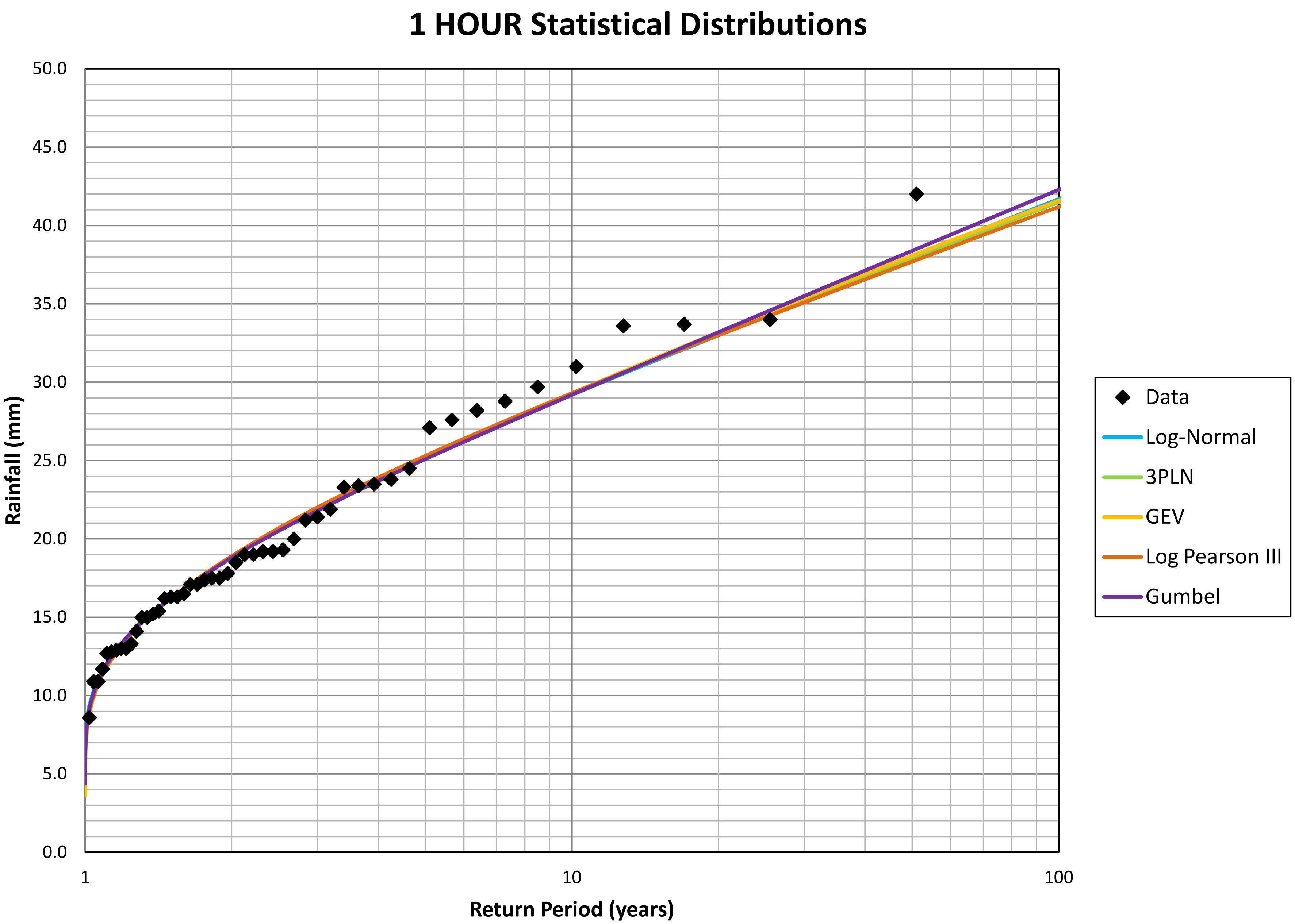
For this test, Z = -1.690

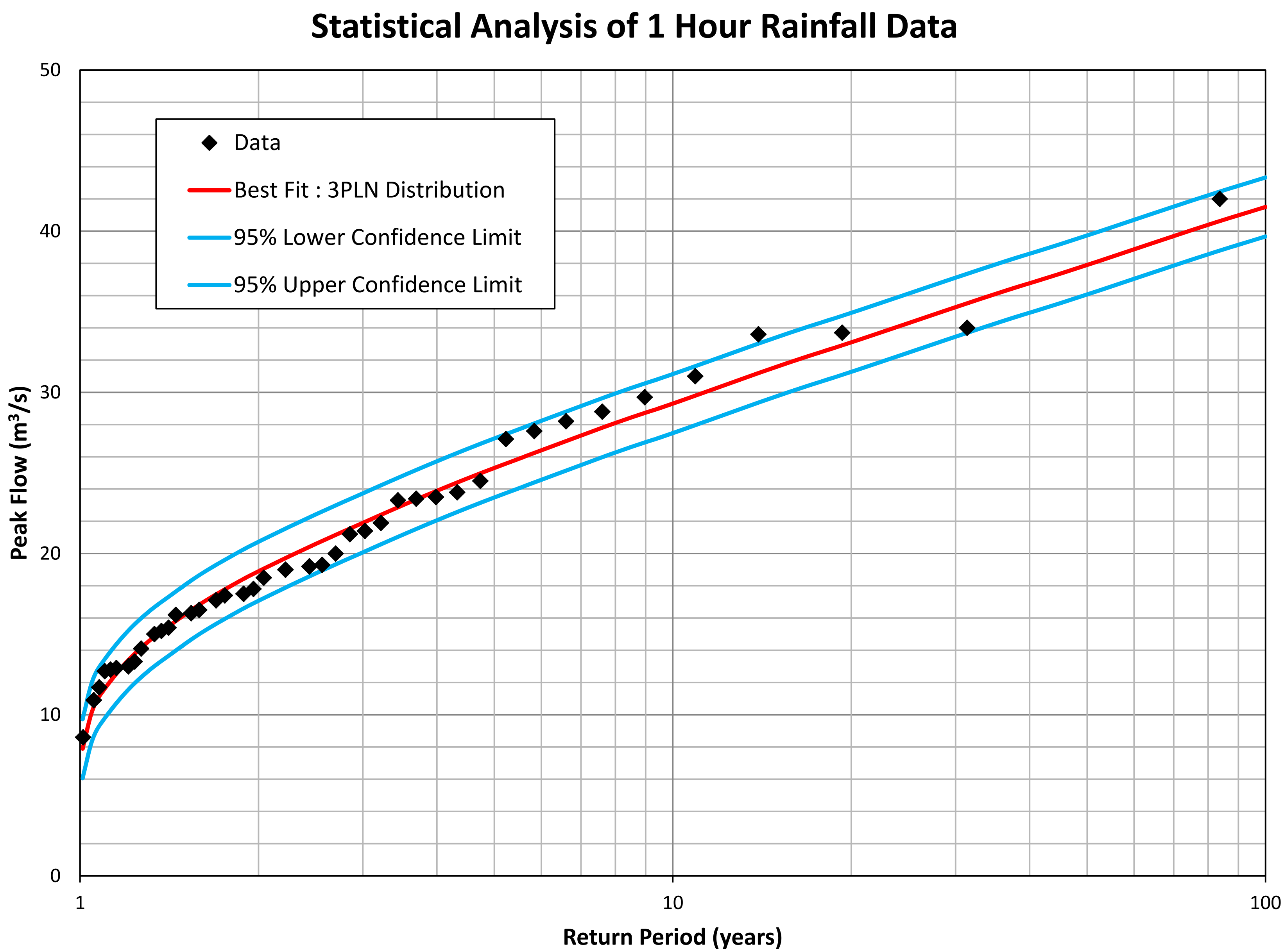
CRITICAL Z VALUE AT 5% SIGNIFICANT LEVEL = -1.645 SIGNIFICANT

- - - - 1% - - = -2.326 NOT SIGNIFICANT

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 5% level of significance, there is a significant difference in location, but not so at the 1% level. That is, the location difference is significant, but not highly so.


1 HOUR Statistical Distributions
Rainfall (mm)
50.0
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
1
10
100
Return Period (years)
Data
Log-Normal
3PLN
GEV
Log Pearson III
Gumbel



Statistical Analysis of 1 Hour Rainfall Data
Data
Best Fit : 3PLN Distribution
95% Lower Confidence Limit
95% Upper Confidence Limit
Peak Flow (m³/s)
Return Period (years)
0
10
20
30
40
50
1
10
100

--- SPEARMAN TEST FOR INDEPENDENCE ---

0060MIN 60 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER SERIAL CORRELATION COEFF = .032 D.F.= 45

CORRESPONDS TO STUDENTS T = .215

CRITICAL T VALUE AT 5% LEVEL = 1.681 NOT SIGNIFICANT

- - - - 1% - = 2.415 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant serial dependence.

--- SPEARMAN TEST FOR TREND ---

0060MIN 60 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER CORRELATION COEFF = -.300 D.F.= 48

CORRESPONDS TO STUDENTS T =-2.177

CRITICAL T VALUE AT 5% LEVEL =-2.013 SIGNIFICANT

- - - - 1% - =-2.686 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the serial(lag-one) correlation is zero.

At the 5% level of significance, the correlation is significantly different from zero, but is not so at the 1% level of significance. That is, the trend is significant but not highly so.

--- RUN TEST FOR GENERAL RANDOMNESS ---

0060MIN 60 Minute Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

THE NUMBER OF RUNS ABOVE AND BELOW THE MEDIAN (RUNAB) = 22

THE NUMBER OF OBSERVATIONS ABOVE THE MEDIAN(N1) = 25

THE NUMBER OF OBSERVATIONS BELOW THE MEDIAN(N2) = 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

For this test, Z = 1.143

Critical Z value at the 5% level = 1.960 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the data are random.

At the 5% level of significance, the null hypothesis cannot be rejected. That is, the sample is significantly random.

--- MANN-WHITNEY SPLIT SAMPLE TEST FOR HOMOGENEITY ---

0060MIN 60 Minute Rainfall Data

ANNUAL MAXIMUM FLOW SERIES 1960 TO 2012 DRAINAGE AREA= .0000000

SPLIT BY TIME SPAN, SUBSAMPLE 1 SAMPLE SIZE= 25

SUBSAMPLE 2 SAMPLE SIZE= 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

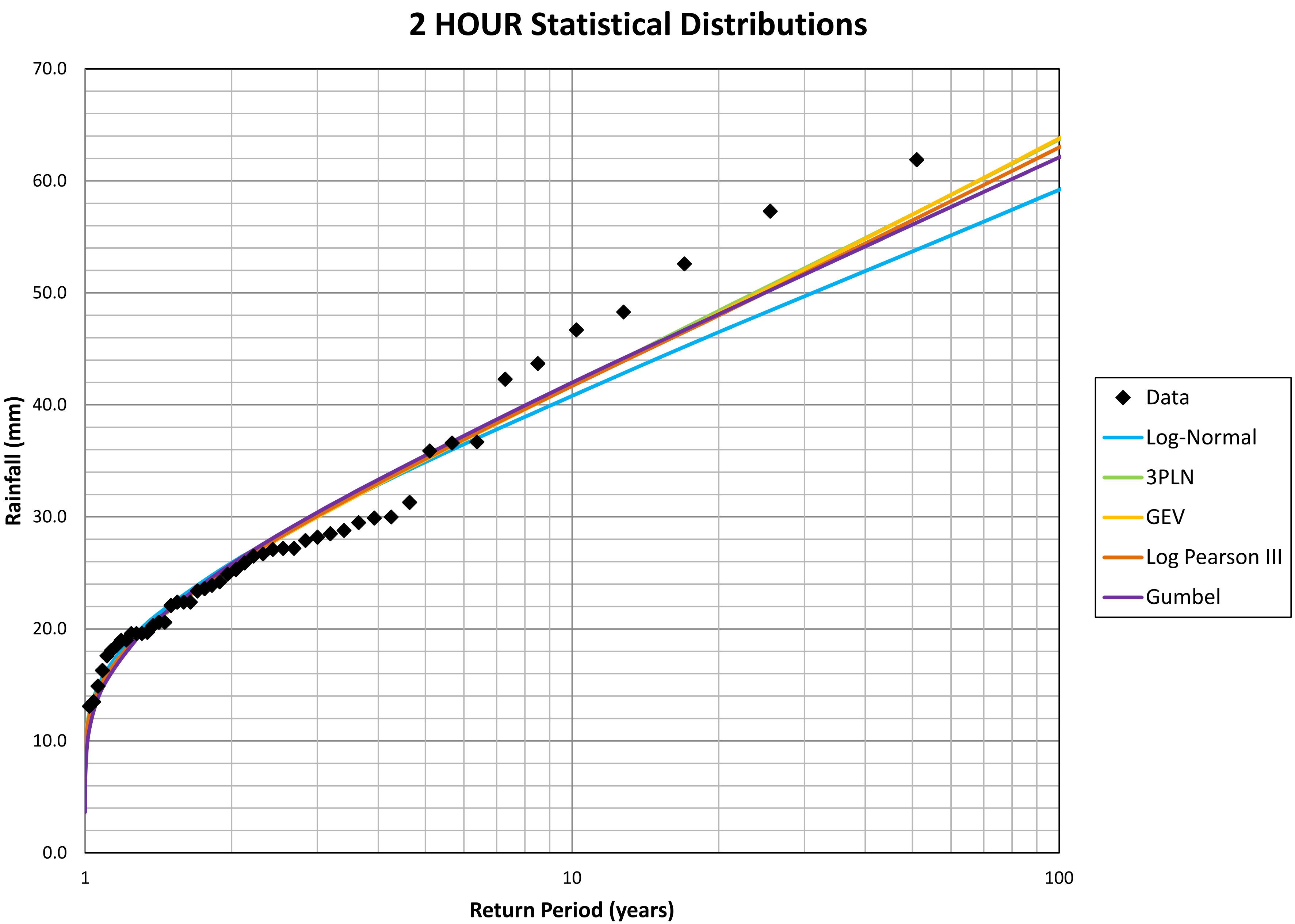
For this test, Z = -2.135

CRITICAL Z VALUE AT 5% SIGNIFICANT LEVEL = -1.645 SIGNIFICANT

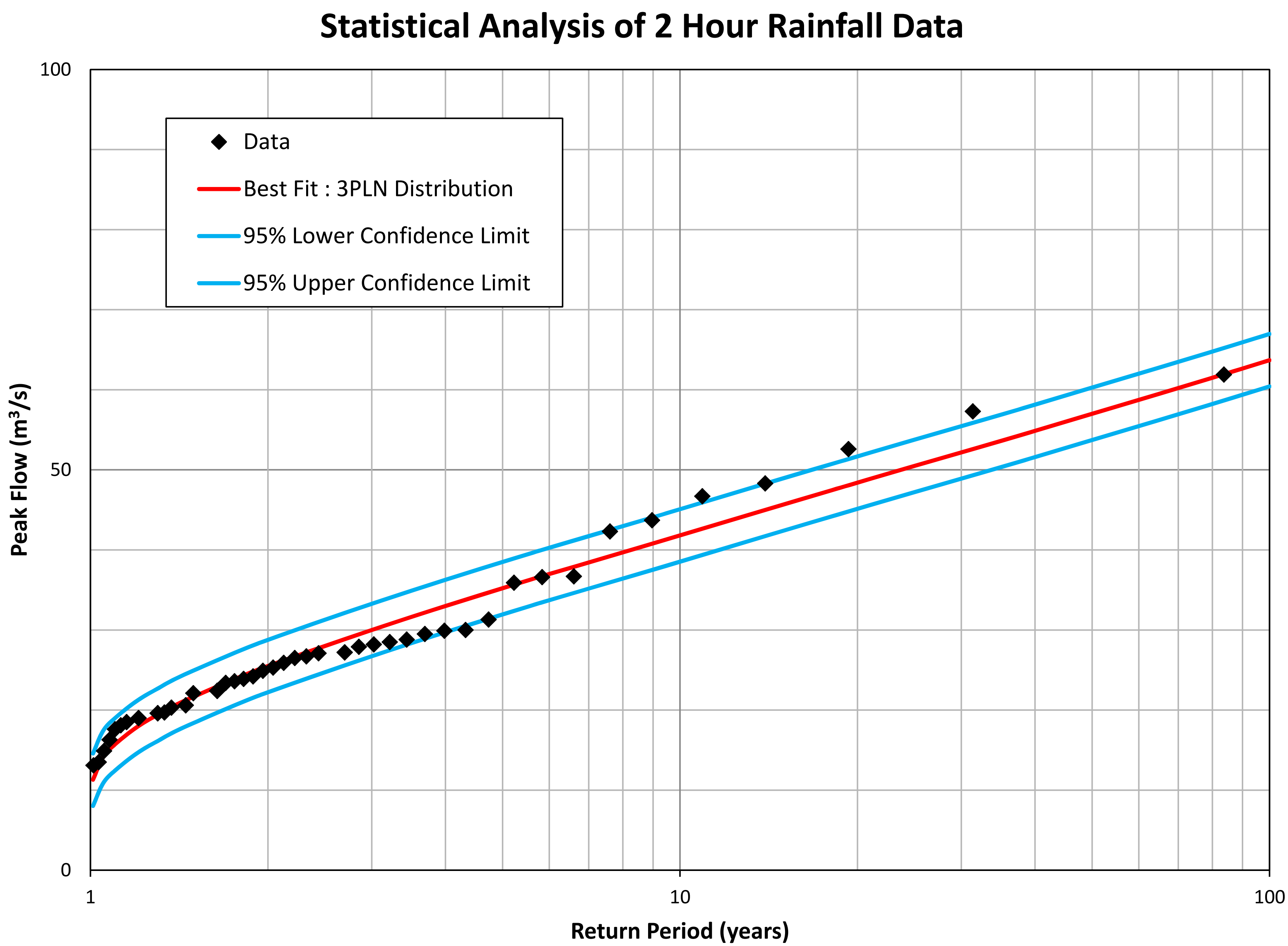
- - - - 1% - - = -2.326 NOT SIGNIFICANT

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 5% level of significance, there is a significant difference in location, but not so at the 1% level. That is, the location difference is significant, but not highly so.



2 HOUR Statistical Distributions
Rainfall (mm)
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
1
10
100
Return Period (years)
Data
Log-Normal
3PLN
GEV
Log Pearson III
Gumbel


Statistical Analysis of 2 Hour Rainfall Data
100
50
0
Peak Flow (m³/s)
Data
Best Fit : 3PLN Distribution
95% Lower Confidence Limit
95% Upper Confidence Limit
1
10
100
Return Period (years)

--- SPEARMAN TEST FOR INDEPENDENCE ---

002HOUR 2 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER SERIAL CORRELATION COEFF = .055 D.F.= 45

CORRESPONDS TO STUDENTS T = .372

CRITICAL T VALUE AT 5% LEVEL = 1.681 NOT SIGNIFICANT

- - - - 1% - = 2.415 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant serial dependence.

--- SPEARMAN TEST FOR TREND ---

002HOUR 2 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER CORRELATION COEFF = -.352 D.F.= 48

CORRESPONDS TO STUDENTS T =-2.604

CRITICAL T VALUE AT 5% LEVEL =-2.013 SIGNIFICANT

- - - - 1% - =-2.686 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the serial(lag-one) correlation is zero.

At the 5% level of significance, the correlation is significantly different from zero, but is not so at the 1% level of significance. That is, the trend is significant but not highly so.

--- RUN TEST FOR GENERAL RANDOMNESS ---

002HOUR 2 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

THE NUMBER OF RUNS ABOVE AND BELOW THE MEDIAN (RUNAB) = 20

THE NUMBER OF OBSERVATIONS ABOVE THE MEDIAN(N1) = 25

THE NUMBER OF OBSERVATIONS BELOW THE MEDIAN(N2) = 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

For this test, Z = 1.715

Critical Z value at the 5% level = 1.960 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the data are random.

At the 5% level of significance, the null hypothesis cannot be rejected. That is, the sample is significantly random.

--- MANN-WHITNEY SPLIT SAMPLE TEST FOR HOMOGENEITY ---

002HOUR 2 Hour Rainfall Data

ANNUAL MAXIMUM FLOW SERIES 1960 TO 2012 DRAINAGE AREA= .0000000

SPLIT BY TIME SPAN, SUBSAMPLE 1 SAMPLE SIZE= 25

SUBSAMPLE 2 SAMPLE SIZE= 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

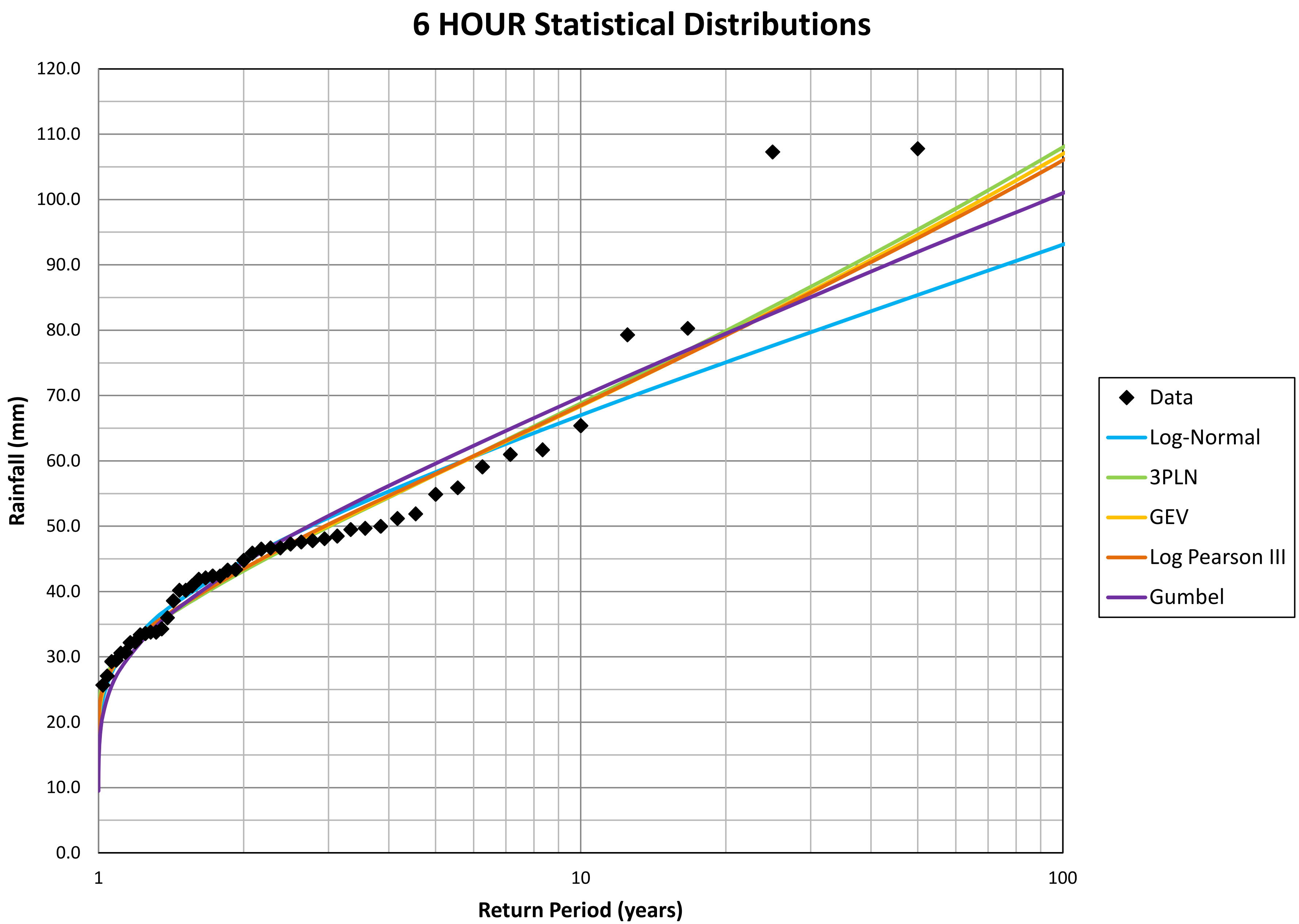
For this test, Z = -2.455

CRITICAL Z VALUE AT 5% SIGNIFICANT LEVEL = -1.645 SIGNIFICANT

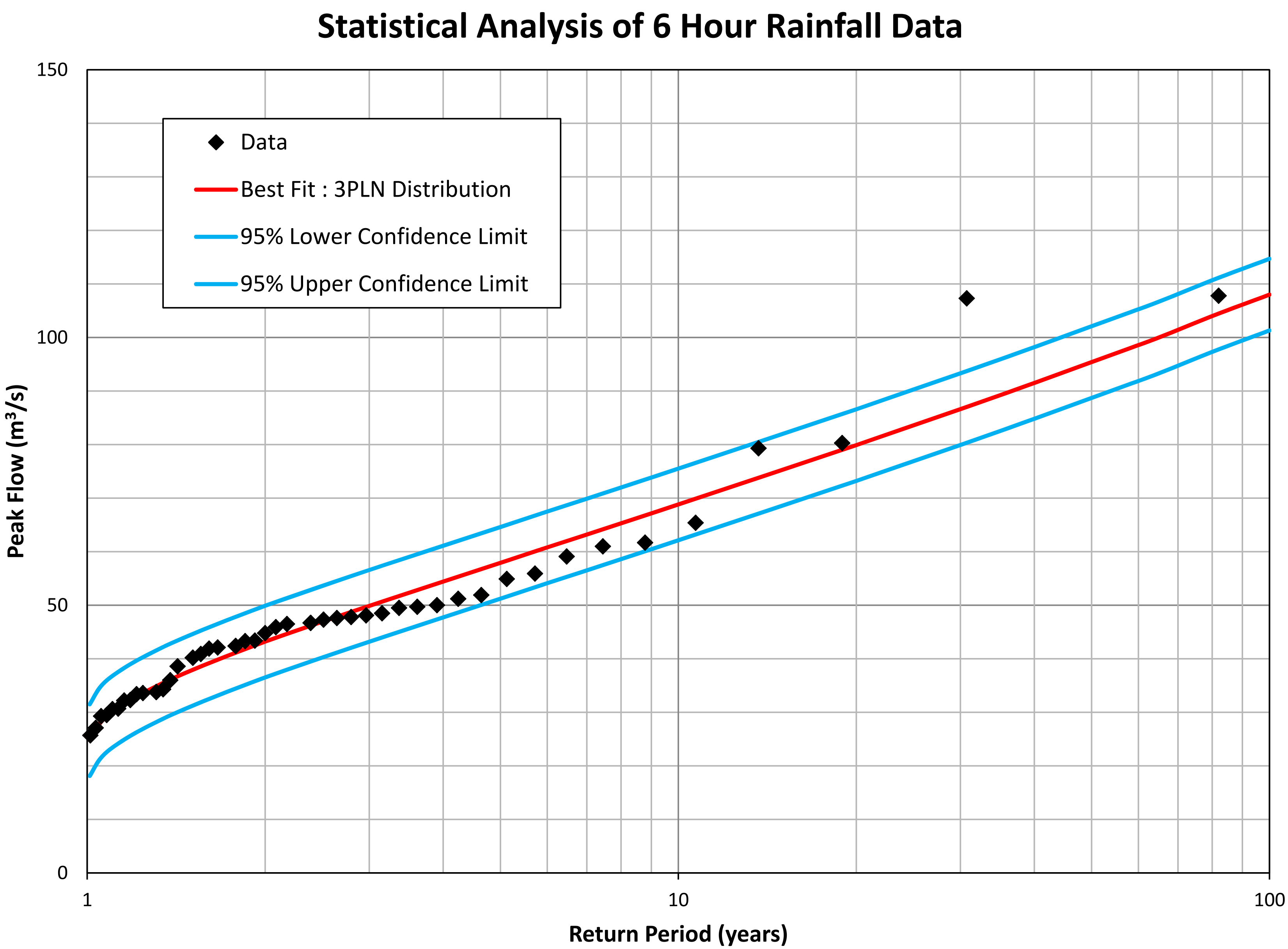
\- - - - 1% - - = -2.326 SIGNIFICANT

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 1% level of significance, the hypothesis of no location difference between the samples is rejected.



6 HOUR Statistical Distributions
Rainfall (mm)
120.0
110.0
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
1
10
100
Return Period (years)
Data
Log-Normal
3PLN
GEV
Log Pearson III
Gumbel



Statistical Analysis of 6 Hour Rainfall Data
Data
Best Fit : 3PLN Distribution
95% Lower Confidence Limit
95% Upper Confidence Limit
Peak Flow (m³/s)
150
100
50
0
1
10
100
Return Period (years)

--- SPEARMAN TEST FOR INDEPENDENCE ---

006HOUR 6 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER SERIAL CORRELATION COEFF = .258 D.F.= 43

CORRESPONDS TO STUDENTS T = 1.747

CRITICAL T VALUE AT 5% LEVEL = 1.682 SIGNIFICANT

- - - - 1% - = 2.418 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is significantly different from zero, but is not so at the 1% level of significance.

That is, the dependence is significant, but not highly so.

--- SPEARMAN TEST FOR TREND ---

006HOUR 6 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER CORRELATION COEFF = -.352 D.F.= 47

CORRESPONDS TO STUDENTS T =-2.575

CRITICAL T VALUE AT 5% LEVEL =-2.014 SIGNIFICANT

- - - - 1% - =-2.689 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the serial(lag-one) correlation is zero.

At the 5% level of significance, the correlation is significantly different from zero, but is not so at the 1% level of significance. That is, the trend is significant but not highly so.

--- RUN TEST FOR GENERAL RANDOMNESS ---

006HOUR 6 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

THE NUMBER OF RUNS ABOVE AND BELOW THE MEDIAN (RUNAB) = 16

THE NUMBER OF OBSERVATIONS ABOVE THE MEDIAN(N1) = 24

THE NUMBER OF OBSERVATIONS BELOW THE MEDIAN(N2) = 24

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

For this test, Z = 2.626

Critical Z value at the 5% level = 1.960 SIGNIFICANT

Critical Z value at the 1% level = 2.575 SIGNIFICANT

Interpretation: The null hypothesis is that the data are random.

At the 1% level of significance, the null hypothesis can be rejected. That is, the sample is not significantly random.

--- MANN-WHITNEY SPLIT SAMPLE TEST FOR HOMOGENEITY ---

006HOUR 6 Hour Rainfall Data

ANNUAL MAXIMUM FLOW SERIES 1960 TO 2012 DRAINAGE AREA= .0000000

SPLIT BY TIME SPAN, SUBSAMPLE 1 SAMPLE SIZE= 24

SUBSAMPLE 2 SAMPLE SIZE= 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

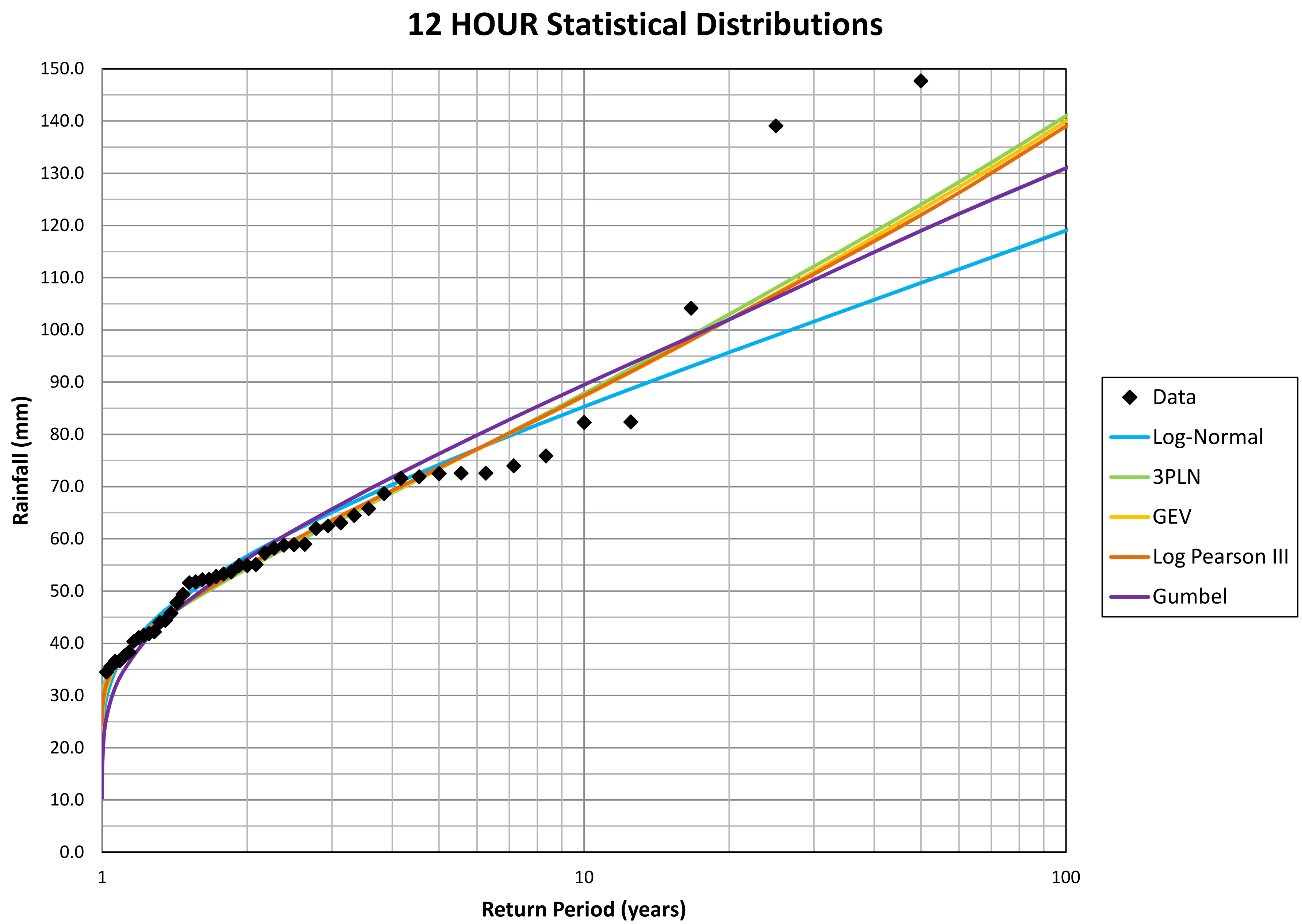
For this test, Z = -2.350

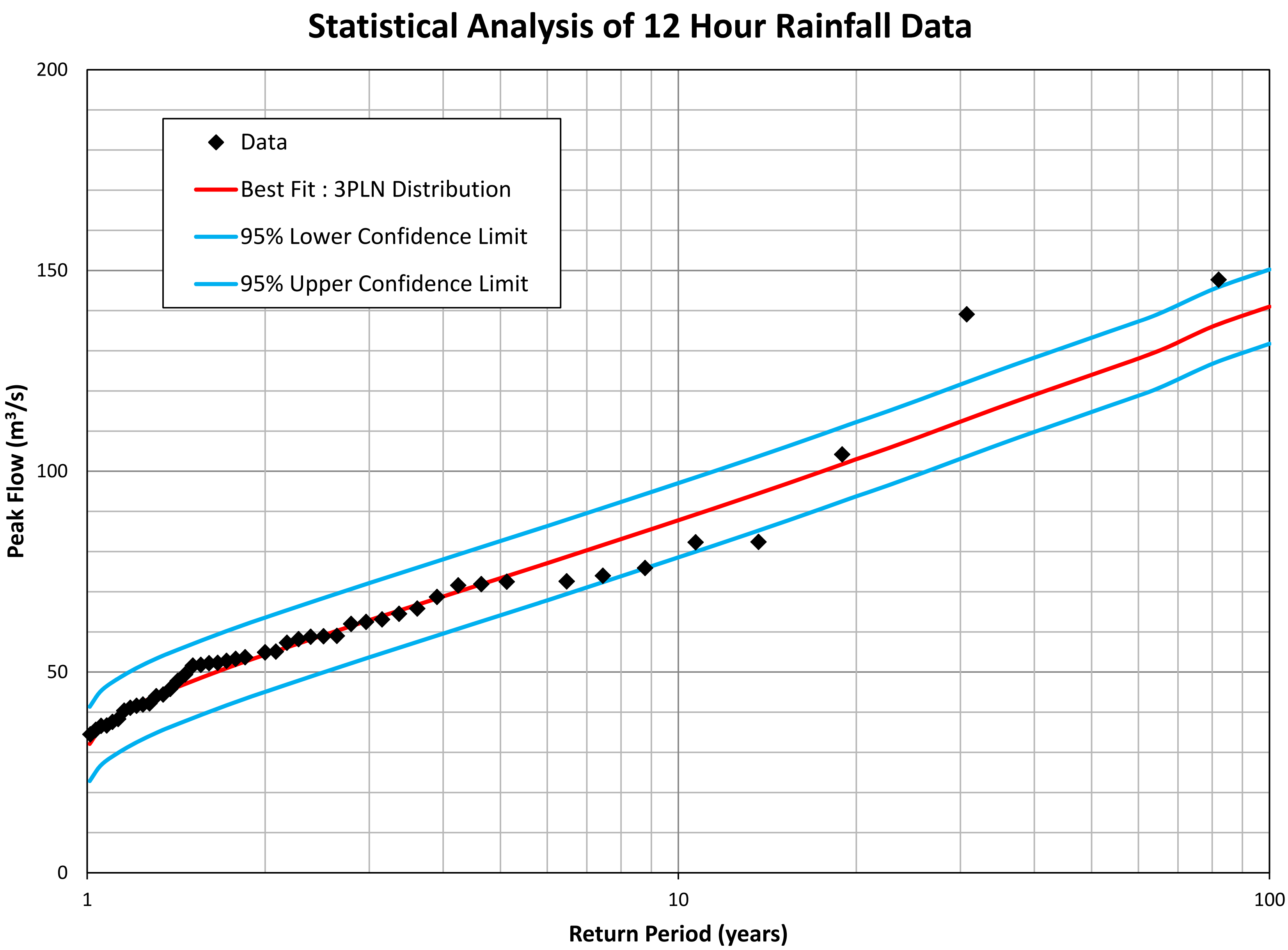
CRITICAL Z VALUE AT 5% SIGNIFICANT LEVEL = -1.645 SIGNIFICANT

- - - - 1% - - = -2.326 SIGNIFICANT

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 1% level of significance, the hypothesis of no location difference between the samples is rejected.


12 HOUR Statistical Distributions
Rainfall (mm)
150.0
140.0
130.0
120.0
110.0
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
1
10
100
Return Period (years)
Data
Log-Normal
3PLN
GEV
Log Pearson III
Gumbel



Statistical Analysis of 12 Hour Rainfall Data
Data
Best Fit : 3PLN Distribution
95% Lower Confidence Limit
95% Upper Confidence Limit
Peak Flow (m³/s)
Return Period (years)
0
50
100
150
200
1
10
100

--- SPEARMAN TEST FOR INDEPENDENCE ---

012HOUR 12 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER SERIAL CORRELATION COEFF = .002 D.F.= 43

CORRESPONDS TO STUDENTS T = .014

CRITICAL T VALUE AT 5% LEVEL = 1.682 NOT SIGNIFICANT

- - - - 1% - = 2.418 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant serial dependence.

--- SPEARMAN TEST FOR TREND ---

012HOUR 12 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER CORRELATION COEFF = -.174 D.F.= 47

CORRESPONDS TO STUDENTS T =-1.212

CRITICAL T VALUE AT 5% LEVEL =-2.014 NOT SIGNIFICANT

- - - - 1% - =-2.689 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the serial(lag-one) correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant trend.

--- RUN TEST FOR GENERAL RANDOMNESS ---

012HOUR 12 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

THE NUMBER OF RUNS ABOVE AND BELOW THE MEDIAN (RUNAB) = 28

THE NUMBER OF OBSERVATIONS ABOVE THE MEDIAN(N1) = 23

THE NUMBER OF OBSERVATIONS BELOW THE MEDIAN(N2) = 24

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

For this test, Z = 1.036

Critical Z value at the 5% level = 1.960 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the data are random.

At the 5% level of significance, the null hypothesis cannot be rejected. That is, the sample is significantly random.

--- MANN-WHITNEY SPLIT SAMPLE TEST FOR HOMOGENEITY ---

012HOUR 12 Hour Rainfall Data

ANNUAL MAXIMUM FLOW SERIES 1960 TO 2012 DRAINAGE AREA= .0000000

SPLIT BY TIME SPAN, SUBSAMPLE 1 SAMPLE SIZE= 24

SUBSAMPLE 2 SAMPLE SIZE= 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

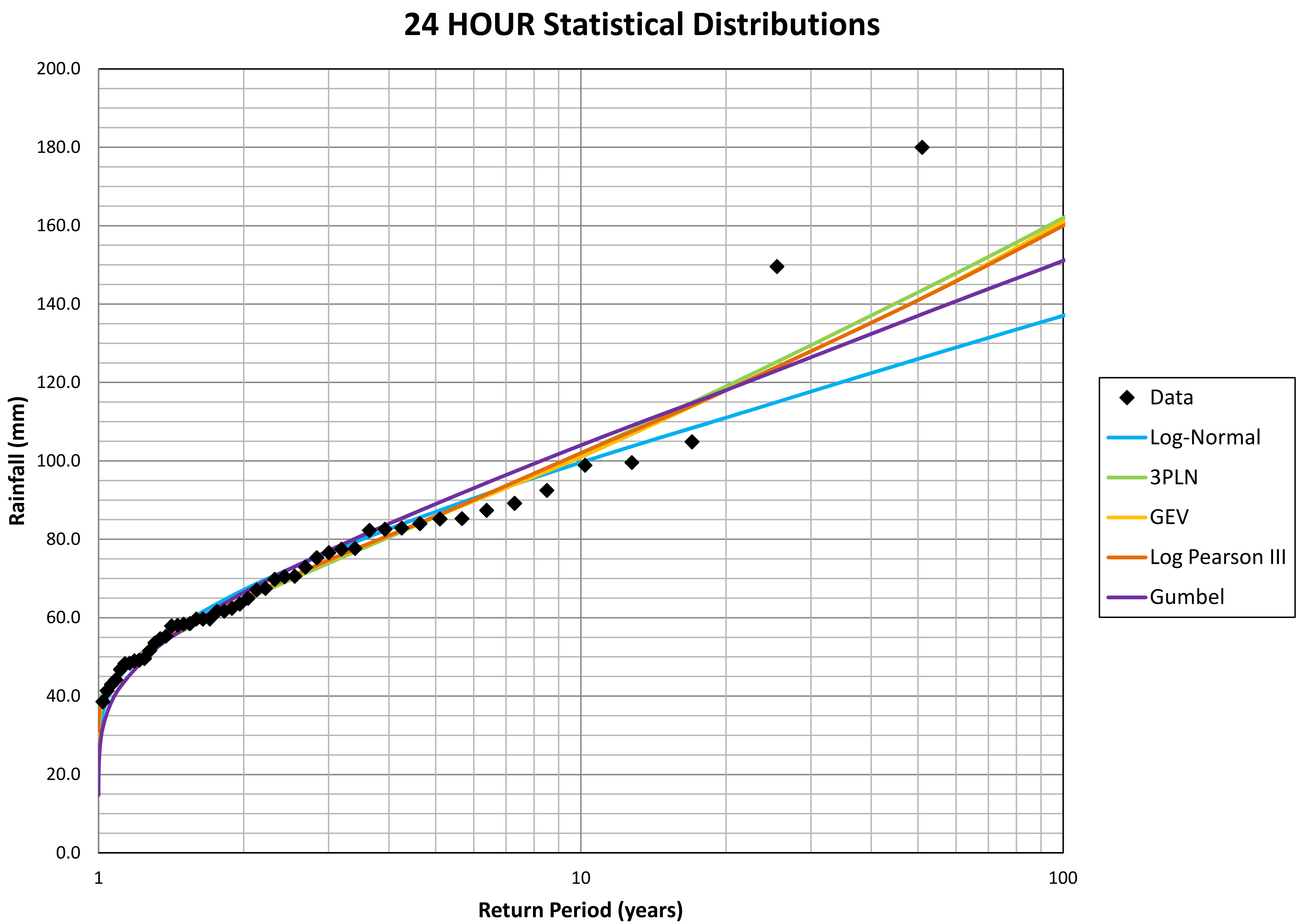
For this test, Z = -1.170

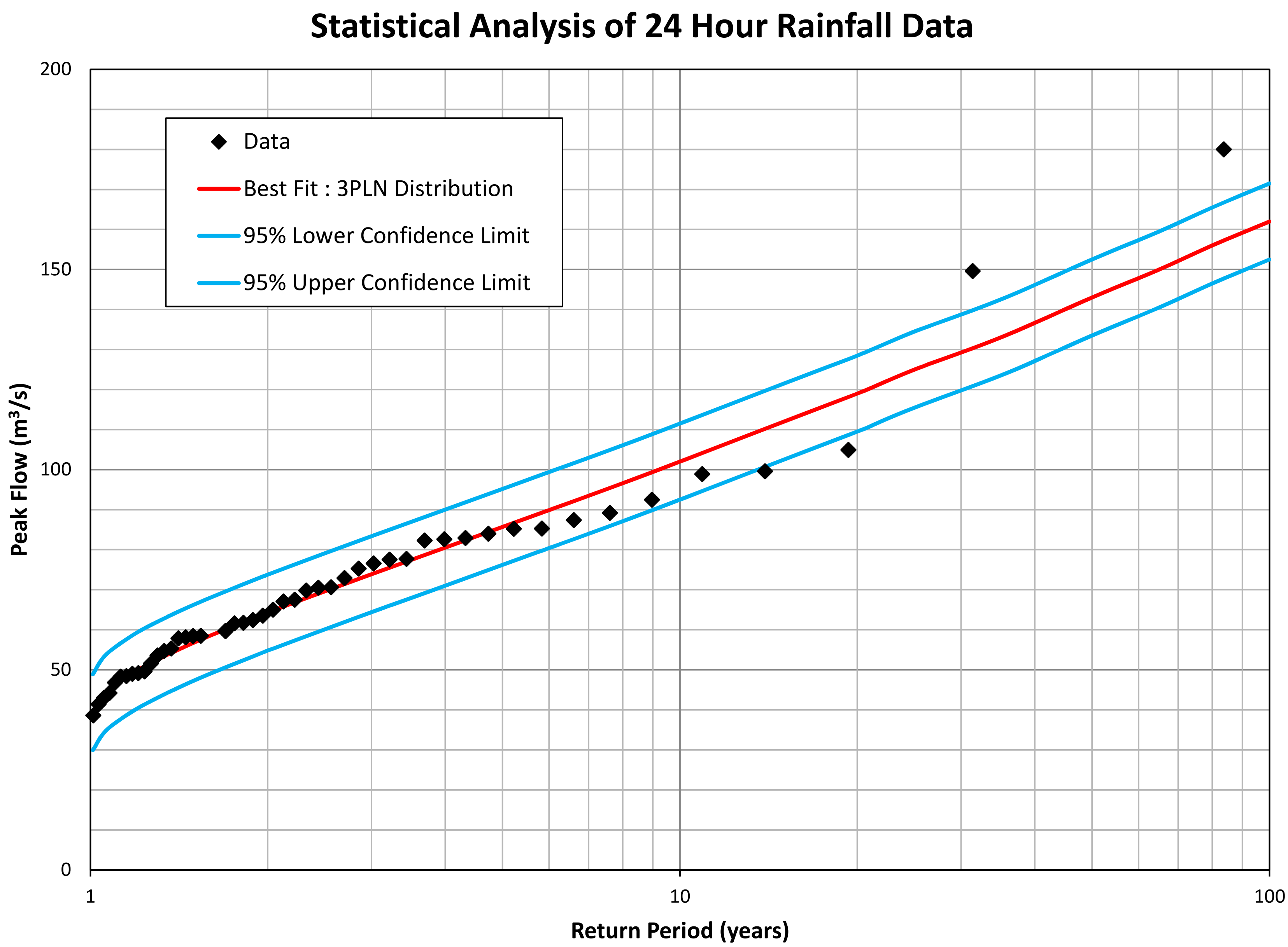
CRITICAL Z VALUE AT 5% SIGNIFICANT LEVEL = -1.645 NOT SIGNIFICANT

\- - - - 1% - - = -2.326 NOT SIGNIFICANT

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 5% level of significance, there is no significant location difference between the two samples. That is, they appear to be from the same population.


24 HOUR Statistical Distributions
Rainfall (mm)
200.0
180.0
160.0
140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
1
10
100
Return Period (years)
Data
Log-Normal
3PLN
GEV
Log Pearson III
Gumbel


Statistical Analysis of 24 Hour Rainfall Data
Data
Best Fit : 3PLN Distribution
95% Lower Confidence Limit
95% Upper Confidence Limit
Peak Flow (m³/s)
Return Period (years)
200
150
100
50
0
1
10
100

--- SPEARMAN TEST FOR INDEPENDENCE ---

024HOUR 24 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER SERIAL CORRELATION COEFF = .055 D.F.= 45

CORRESPONDS TO STUDENTS T = .369

CRITICAL T VALUE AT 5% LEVEL = 1.681 NOT SIGNIFICANT

- - - - 1% - = 2.415 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant serial dependence.

--- SPEARMAN TEST FOR TREND ---

024HOUR 24 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

SPEARMAN RANK ORDER CORRELATION COEFF = -.149 D.F.= 48

CORRESPONDS TO STUDENTS T =-1.041

CRITICAL T VALUE AT 5% LEVEL =-2.013 NOT SIGNIFICANT

- - - - 1% - =-2.686 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the serial(lag-one) correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant trend.

--- RUN TEST FOR GENERAL RANDOMNESS ---

024HOUR 24 Hour Rainfall Data

ANNUAL MAXIMUM DAILY FLOW SERIES 1960 TO 2012 DRAINAGE AREA = .0000000

THE NUMBER OF RUNS ABOVE AND BELOW THE MEDIAN (RUNAB) = 23

THE NUMBER OF OBSERVATIONS ABOVE THE MEDIAN(N1) = 25

THE NUMBER OF OBSERVATIONS BELOW THE MEDIAN(N2) = 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

For this test, Z = .857

Critical Z value at the 5% level = 1.960 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the data are random.

At the 5% level of significance, the null hypothesis cannot be rejected. That is, the sample is significantly random.

--- MANN-WHITNEY SPLIT SAMPLE TEST FOR HOMOGENEITY ---

024HOUR 24 Hour Rainfall Data

ANNUAL MAXIMUM FLOW SERIES 1960 TO 2012 DRAINAGE AREA= .0000000

SPLIT BY TIME SPAN, SUBSAMPLE 1 SAMPLE SIZE= 25

SUBSAMPLE 2 SAMPLE SIZE= 25

(NOTE: Z IS THE STANDARD NORMAL VARIATE.)

For this test, Z = -.689

CRITICAL Z VALUE AT 5% SIGNIFICANT LEVEL = -1.645 NOT SIGNIFICANT

- - - - 1% - - = -2.326 NOT SIGNIFICANT

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 5% level of significance, there is no significant location difference between the two samples. That is, they appear to be from the same population.

APPENDIX D

Design Hyetographs based on Updated IDFs

% Time	1:2 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	0.90	0.79	1.01	1.29	1.17	1.42	1.73	1.51	1.95	2.94	2.48	3.39	0.55	0.46	0.65	4.34	3.69	4.98
16.67%	2.28	2.00	2.55	3.19	2.88	3.50	4.34	3.79	4.90	7.35	6.20	8.48	1.11	0.92	1.29	10.91	9.29	12.52
25.00%	4.00	3.52	4.48	5.70	5.14	6.25	7.68	6.69	8.67	12.98	10.96	14.99	3.82	3.17	4.47	19.34	16.46	22.19
33.33%	6.28	5.53	7.03	8.93	8.07	9.80	12.07	10.51	13.62	20.42	17.25	23.59	9.75	8.08	11.42	30.40	25.87	34.88
41.67%	8.98	7.91	10.06	12.77	11.54	14.01	17.25	15.04	19.47	29.24	24.69	33.77	20.61	17.08	24.12	43.50	37.02	49.92
50.00%	11.10	9.78	12.43	15.75	14.23	17.28	21.28	18.54	24.01	36.05	30.44	41.63	34.19	28.33	40.01	53.63	45.64	61.55
58.33%	11.81	10.40	13.22	16.79	15.16	18.41	22.64	19.73	25.55	38.35	32.39	44.29	43.99	36.46	51.49	57.04	48.54	65.46
66.67%	12.36	10.89	13.83	17.52	15.82	19.22	23.65	20.61	26.69	40.07	33.83	46.27	49.42	40.96	57.84	59.67	50.78	68.48
75.00%	12.79	11.27	14.32	18.17	16.41	19.93	24.49	21.34	27.64	41.49	35.03	47.91	52.14	43.21	61.02	61.77	52.56	70.89
83.33%	13.06	11.51	14.62	18.60	16.80	20.40	25.06	21.83	28.28	42.47	35.86	49.04	53.19	44.08	62.25	63.23	53.81	72.56
91.67%	13.22	11.65	14.80	18.77	16.95	20.59	25.34	22.08	28.59	42.96	36.27	49.61	53.75	44.54	62.90	63.91	54.39	73.35
100.00%	13.30	11.72	14.89	18.90	17.07	20.73	25.50	22.22	28.78	43.20	36.48	49.89	54.30	45.00	63.55	64.30	54.72	73.79

% Time	1:5 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	1.19	1.09	1.30	1.73	1.61	1.86	2.39	2.17	2.61	3.94	3.48	4.39	0.75	0.65	0.84	5.78	5.14	6.42
16.67%	3.01	2.74	3.28	4.27	3.97	4.58	6.00	5.44	6.55	9.85	8.71	10.98	1.50	1.31	1.68	14.54	12.93	16.15
25.00%	5.30	4.82	5.77	7.62	7.07	8.18	10.60	9.62	11.59	17.40	15.39	19.41	5.17	4.51	5.82	25.77	22.92	28.63
33.33%	8.31	7.56	9.06	11.96	11.09	12.82	16.66	15.11	18.21	27.37	24.21	30.54	13.18	11.52	14.85	40.51	36.02	45.00
41.67%	11.89	10.82	12.96	17.10	15.86	18.33	23.82	21.60	26.03	39.19	34.66	43.72	27.86	24.35	31.38	57.98	51.55	64.40
50.00%	14.69	13.37	16.02	21.08	19.56	22.61	29.37	26.64	32.10	48.32	42.73	53.90	46.21	40.39	52.04	71.48	63.56	79.40
58.33%	15.63	14.22	17.04	22.47	20.84	24.10	31.26	28.35	34.17	51.40	45.46	57.34	59.47	51.98	66.96	76.03	67.60	84.45
66.67%	16.35	14.88	17.83	23.45	21.75	25.15	32.65	29.61	35.68	53.70	47.49	59.90	66.81	58.39	75.22	79.53	70.72	88.34
75.00%	16.93	15.40	18.45	24.32	22.56	26.08	33.81	30.67	36.96	55.60	49.18	62.03	70.48	61.60	79.36	82.32	73.20	91.44
83.33%	17.29	15.73	18.85	24.90	23.09	26.70	34.59	31.37	37.81	56.92	50.34	63.49	71.90	62.85	80.96	84.27	74.94	93.61
91.67%	17.50	15.92	19.07	25.13	23.31	26.94	34.98	31.72	38.23	57.57	50.92	64.23	72.65	63.50	81.81	85.18	75.74	94.62
100.00%	17.60	16.01	19.19	25.30	23.47	27.13	35.20	31.92	38.48	57.90	51.21	64.59	73.40	64.15	82.65	85.70	76.21	95.19

% Time	1:10 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	1.38	1.28	1.49	2.01	1.88	2.13	2.84	2.61	3.06	4.68	4.23	5.14	0.89	0.80	0.99	6.88	6.24	7.52
16.67%	3.49	3.22	3.76	4.95	4.64	5.26	7.12	6.56	7.68	11.70	10.56	12.84	1.79	1.60	1.98	17.31	15.70	18.92
25.00%	6.14	5.66	6.62	8.83	8.28	9.38	12.59	11.61	13.58	20.67	18.66	22.68	6.18	5.53	6.83	30.68	27.82	33.53
33.33%	9.63	8.88	10.38	13.85	12.98	14.71	19.78	18.23	21.33	32.53	29.36	35.69	15.77	14.11	17.43	48.22	43.73	52.71
41.67%	13.78	12.71	14.85	19.80	18.56	21.04	28.28	26.07	30.50	46.57	42.04	51.10	33.33	29.82	36.84	69.00	62.58	75.43
50.00%	17.03	15.71	18.35	24.42	22.89	25.94	34.88	32.14	37.61	57.41	51.83	63.00	55.28	49.46	61.10	85.08	77.16	93.00
58.33%	18.11	16.71	19.52	26.02	24.40	27.65	37.12	34.21	40.03	61.08	55.14	67.02	71.13	63.64	78.63	90.49	82.06	98.91
66.67%	18.96	17.48	20.43	27.16	25.46	28.86	38.77	35.73	41.80	63.81	57.60	70.01	79.91	71.50	88.33	94.66	85.85	103.47
75.00%	19.62	18.09	21.14	28.16	26.40	29.92	40.15	37.01	43.30	66.07	59.64	72.50	84.30	75.42	93.18	97.98	88.86	107.10
83.33%	20.04	18.48	21.60	28.83	27.03	30.63	41.07	37.86	44.29	67.63	61.05	74.21	86.01	76.95	95.07	100.30	90.96	109.64
91.67%	20.28	18.70	21.86	29.10	27.28	30.92	41.54	38.28	44.79	68.41	61.76	75.06	86.91	77.75	96.06	101.38	91.95	110.82
100.00%	20.40	18.81	21.99	29.30	27.47	31.13	41.80	38.52	45.08	68.80	62.11	75.49	87.80	78.55	97.05	102.00	92.51	111.49

% Time	1:20 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	1.55	1.45	1.66	2.27	2.14	2.39	3.28	3.06	3.50	5.44	4.98	5.89	1.05	0.95	1.14	8.02	7.38	8.66
16.67%	3.92	3.65	4.19	5.59	5.28	5.90	8.24	7.69	8.80	13.59	12.45	14.73	2.10	1.91	2.29	20.19	18.58	21.81
25.00%	6.89	6.41	7.37	9.98	9.42	10.53	14.58	13.59	15.57	24.01	22.00	26.02	7.25	6.60	7.90	35.79	32.93	38.65
33.33%	10.81	10.06	11.56	15.64	14.78	16.51	22.90	21.35	24.45	37.78	34.61	40.94	18.50	16.84	20.16	56.25	51.77	60.74
41.67%	15.47	14.40	16.54	22.37	21.13	23.61	32.75	30.53	34.97	54.08	49.55	58.61	39.10	35.59	42.61	80.51	74.08	86.93
50.00%	19.12	17.79	20.44	27.58	26.06	29.11	40.38	37.65	43.12	66.67	61.09	72.26	64.85	59.03	70.67	99.26	91.34	107.18
58.33%	20.33	18.92	21.74	29.40	27.77	31.02	42.98	40.07	45.89	70.93	64.99	76.87	83.45	75.96	90.94	105.57	97.15	113.99
66.67%	21.28	19.81	22.75	30.68	28.99	32.38	44.89	41.85	47.93	74.10	67.90	80.31	93.75	85.33	102.17	110.44	101.62	119.25
75.00%	22.02	20.50	23.55	31.82	30.06	33.57	46.49	43.35	49.64	76.73	70.30	83.16	98.90	90.02	107.78	114.31	105.19	123.43
83.33%	22.49	20.94	24.05	32.57	30.77	34.37	47.56	44.34	50.78	78.54	71.96	85.12	100.90	91.84	109.96	117.02	107.68	126.35
91.67%	22.76	21.19	24.34	32.87	31.06	34.69	48.09	44.84	51.35	79.45	72.79	86.10	101.95	92.80	111.10	118.28	108.84	127.72
100.00%	22.90	21.31	24.49	33.10	31.27	34.93	48.40	45.12	51.68	79.90	73.21	86.59	103.00	93.75	112.25	119.00	109.51	128.49

% Time	1:25 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	1.61	1.50	1.72	2.35	2.22	2.47	3.43	3.20	3.65	5.69	5.23	6.14	1.10	1.01	1.19	8.43	7.79	9.07
16.67%	4.05	3.78	4.33	5.79	5.49	6.10	8.60	8.04	9.16	14.22	13.08	15.36	2.20	2.01	2.39	21.21	19.60	22.82
25.00%	7.13	6.65	7.61	10.34	9.79	10.89	15.21	14.23	16.20	25.12	23.11	27.13	7.60	6.95	8.25	37.59	34.74	40.45
33.33%	11.19	10.44	11.93	16.21	15.35	17.08	23.90	22.35	25.45	39.53	36.36	42.69	19.40	17.74	21.06	59.09	54.60	63.58
41.67%	16.01	14.94	17.08	23.18	21.94	24.42	34.17	31.95	36.39	56.59	52.06	61.12	41.00	37.49	44.51	84.56	78.14	90.99
50.00%	19.78	18.46	21.11	28.58	27.06	30.11	42.14	39.40	44.87	69.76	64.18	75.35	68.00	62.18	73.82	104.26	96.34	112.18
58.33%	21.04	19.64	22.45	30.46	28.84	32.09	44.84	41.94	47.75	74.22	68.28	80.16	87.50	80.01	94.99	110.89	102.47	119.31
66.67%	22.02	20.55	23.50	31.79	30.10	33.49	46.84	43.80	49.87	77.53	71.33	83.74	98.30	89.88	106.72	116.00	107.19	124.81
75.00%	22.79	21.27	24.32	32.97	31.21	34.73	48.51	45.36	51.65	80.28	73.86	86.71	103.70	94.82	112.58	120.08	110.96	129.20
83.33%	23.28	21.72	24.84	33.75	31.95	35.55	49.62	46.41	52.84	82.18	75.60	88.76	105.80	96.74	114.86	122.92	113.58	132.25
91.67%	23.56	21.98	25.14	34.07	32.25	35.88	50.18	46.93	53.44	83.13	76.47	89.78	106.90	97.75	116.05	124.24	114.81	133.68
100.00%	23.70	22.11	25.29	34.30	32.47	36.13	50.50	47.22	53.78	83.60	76.91	90.29	108.00	98.75	117.25	125.00	115.51	134.49

% Time	1:50 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	1.77	1.66	1.88	2.60	2.47	2.72	3.87	3.64	4.09	6.49	6.03	6.95	1.26	1.17	1.36	9.64	9.00	10.28
16.67%	4.47	4.19	4.74	6.40	6.09	6.71	9.71	9.15	10.27	16.22	15.09	17.36	2.53	2.34	2.71	24.27	22.66	25.88
25.00%	7.85	7.38	8.33	11.42	10.87	11.97	17.17	16.19	18.16	28.66	26.65	30.67	8.73	8.08	9.38	43.01	40.15	45.86
33.33%	12.32	11.57	13.07	17.91	17.05	18.78	26.97	25.42	28.52	45.10	41.94	48.27	22.27	20.61	23.94	67.60	63.11	72.09
41.67%	17.63	16.56	18.70	25.61	24.38	26.85	38.57	36.35	40.79	64.57	60.04	69.10	47.07	43.56	50.58	96.74	90.32	103.16
50.00%	21.79	20.46	23.11	31.58	30.06	33.11	47.56	44.83	50.29	79.61	74.02	85.19	78.07	72.25	83.90	119.28	111.36	127.19
58.33%	23.17	21.77	24.58	33.66	32.03	35.29	50.62	47.71	53.53	84.69	78.75	90.63	100.46	92.97	107.96	126.86	118.44	135.28
66.67%	24.25	22.78	25.73	35.13	33.43	36.83	52.86	49.83	55.90	88.48	82.27	94.68	112.86	104.45	121.28	132.71	123.90	141.52
75.00%	25.10	23.57	26.62	36.43	34.67	38.19	54.75	51.61	57.90	91.61	85.19	98.04	119.06	110.18	127.94	137.37	128.25	146.49
83.33%	25.64	24.08	27.20	37.29	35.49	39.10	56.01	52.79	59.23	93.78	87.20	100.36	121.47	112.41	130.53	140.62	131.28	149.95
91.67%	25.95	24.37	27.52	37.64	35.82	39.46	56.64	53.39	59.89	94.86	88.21	101.51	122.74	113.58	131.89	142.13	132.70	151.57
100.00%	26.10	24.51	27.69	37.90	36.07	39.73	57.00	53.72	60.28	95.40	88.71	102.09	124.00	114.75	133.25	143.00	133.51	152.49

% Time	1:100 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	1.93	1.82	2.03	2.84	2.72	2.97	4.32	4.10	4.54	7.35	6.89	7.80	1.44	1.34	1.53	10.92	10.28	11.56
16.67%	4.86	4.59	5.13	7.01	6.70	7.32	10.85	10.29	11.41	18.37	17.23	19.51	2.87	2.68	3.06	27.49	25.88	29.10
25.00%	8.55	8.07	9.02	12.51	11.96	13.06	19.19	18.20	20.18	32.45	30.44	34.46	9.92	9.27	10.57	48.72	45.87	51.58
33.33%	13.40	12.66	14.15	19.61	18.75	20.48	30.14	28.59	31.69	51.06	47.90	54.22	25.33	23.67	26.99	76.58	72.09	81.07
41.67%	19.18	18.11	20.26	28.05	26.81	29.28	43.10	40.89	45.32	73.10	68.57	77.63	53.53	50.02	57.04	109.60	103.17	116.02
50.00%	23.71	22.38	25.03	34.58	33.06	36.11	53.15	50.42	55.88	90.12	84.54	95.71	88.78	82.96	94.60	135.12	127.20	143.04
58.33%	25.22	23.81	26.62	36.86	35.23	38.48	56.57	53.66	59.47	95.88	89.94	101.82	114.24	106.74	121.73	143.71	135.29	152.14
66.67%	26.39	24.92	27.86	38.47	36.77	40.16	59.08	56.04	62.12	100.16	93.96	106.37	128.34	119.92	136.75	150.34	141.53	159.15
75.00%	27.31	25.79	28.84	39.89	38.13	41.65	61.19	58.04	64.33	103.71	97.29	110.14	135.39	126.51	144.27	155.62	146.50	164.74
83.33%	27.90	26.34	29.46	40.84	39.04	42.64	62.59	59.38	65.81	106.16	99.59	112.74	138.13	129.07	147.19	159.30	149.96	168.64
91.67%	28.23	26.66	29.81	41.22	39.40	43.03	63.30	60.04	66.55	107.39	100.73	114.04	139.56	130.41	148.72	161.02	151.58	170.45
100.00%	28.40	26.81	29.99	41.50	39.67	43.33	63.70	60.42	66.98	108.00	101.31	114.69	141.00	131.75	150.25	162.00	152.51	171.49

APPENDIX E

Dr. Joel Finnis' Report

Projection of Precipitation Intensity-Duration-Frequency Data for the Mid-21st Century in St. John's, NL

Prepared by Dr. Joel Finnis
for CBCL Limited
as a contribution to a report to the City of St. John's
Final Report, Sept. 4, 2013

1.0 Introduction

As awareness of climate change continues to increase, a growing number of stakeholders have become interested in accessing climate projections at scales suitable for adaptation planning. Unfortunately, the desired information is rarely available in a usable form, and considerable work is often necessary before projections can be put in action. The core tool used in climate change research is the general circulation model (GCM). Given limited inputs such as variations in solar output or atmospheric composition, a GCM will provide a long-term simulation of the state of the atmosphere, ocean, land surface, and snow/ice across the entire planet. By using long simulations (typically longer than 100 years), and allowing different atmospheric compositions to be used, GCMs can be used as a virtual laboratory in order to test various climate-related hypotheses; the best known of these is the hypothesis that consumption of fossil fuels can influence global climate. This is a very different modeling approach than that used in short-term weather forecasts; here, the emphasis is not on predicting the evolution of specific storms, but instead on examining the long-term impact of multiple weather systems. At present, GCMs provide the best available means of assessing large-scale impacts of climate change, such as global mean temperatures or Arctic sea ice extent.

The direct application of GCMs in regional- or local-scale analyses is limited by the low spatial (100-1000km) and temporal (usually 24 hours) resolution of the GCM output. At these scales a location like Newfoundland appears as only a handful of data points, in which it is impossible to distinguish between the distinct climates of St. John's, St. Anthony, Corner Brook, or Gander. The limited resolution is necessary because of the physical complexity and global scope required in a reliable GCM; in order to complete the computations in a reasonable amount of time, resolution must remain low. However, there exist strong relationships between the large-scale phenomena GCMs simulate well and the small-scale phenomena of concern on regional scales. The process of extracting small-scale information from low-resolution data (whether GCM output or observations) is referred to as climate downscaling. Commonly used to assess climate on scales necessary for practical applications, downscaling can be performed with either statistical methods (statistical downscaling) or the use of regional climate models (RCMs) run with a) a limited domain and b) much higher resolution (~10-50 km). Referred to as dynamical downscaling, the RCM approach uses GCM output to provide boundary forcing for the RCM, making the RCM output a physically-constrained, high resolution extension of the original low-resolution GCM data.

2.0 Climate Projection Data

The IDF projections for the mid-21st century presented here were derived from RCM simulations prepared for the North American Regional Climate Change Assessment Project (NARCCAP; Mearns et al. 2012). NARCCAP used multiple RCM/GCM combinations to generate a multi-model ensemble of projections; each combination consists of a paired 20th century (1968-2000) simulation and 21st century (2038-2070) projection. Currently, seven paired ensemble members suitable for analysis in Newfoundland are available; all have been used in the current study. NARCCAP data is saved at 50km spatial resolution and 3 hour time intervals.

3.0 Methodology

3.1 Areal Reduction Factor Approach

Although raw RCM output offers an improvement over raw GCM output, additional analysis is still necessary before the projections can be put into practice. Precipitation statistics calculated from climate model output, whether RCMs or GCMs, do not typically match station observations well. One of the primary reasons for this is that the models calculate precipitation averaged for areas (grid cells; in this case, 50km x 50km), while stations measure precipitation falling over a single point (Emori et al. 2005). Extreme precipitation events typically affect an area much smaller than an RCM grid cell; in a model, the precipitation produced by these small, intense events will be distributed evenly across the grid cell, reducing their maximum intensity. Consequently, extreme events simulated by models are considerably lower than those observed at stations. A variety of methods have been proposed for translating between station data and model output; the current study uses the areal reduction factor (ARF) method proposed by Allen & DeGaetano (2005). In this approach, an ARF is calculated as:

$$ARF(T,d) = \frac{x_c^{(g)}(T,d)}{x_c^{(s)}(T,d)} \qquad (1)$$

where $x_c(T,d)$ is the precipitation amount for the return period T and duration d, in the 20th century (subscript c). Superscript s indicates values observed at a station, and g indicates values output for the model grid cell (here, the grid cell closest to the station). In the current implementation, ARFs were calculated for each requested return period (2, 5, 10, 20, 25, 50 and 100 year return periods) at event durations of 6, 12, and 24 hours. As multiples of the base 3 hourly data output by the NARCCAP RCMs, these are the event durations that can be estimated without additional extrapolation from the model output. Assuming ARFs remain constant under a changing climate, future (subscript f) station values can then be estimated as:

$$x_f^{(s)}(T,d) = \frac{x_f^{(g)}(T,d)}{ARF(T,d)} \qquad (2)$$

Following the methodology used to update the observational IDF curve, NARCCAP return period events were estimated for 6, 12, and 24-hour duration by fitting a three parameter lognormal distribution to an annual precipitation maxima timeseries at the model grid cell closest to the St. John's airport. Distributions were fit to both 20th century NARCCAP simulations and 21st century projections; the former were compared to a distribution fit to observed station values in order to calculate ARFs. These ARFs were then applied to the 21st century distributions to estimate future return period events.

3.2 Extrapolation of Short Duration Return Periods

Additional analysis was required to estimate values for durations shorter than three hours. Following the official IDF curves produced by Environment Canada, this was done by assuming a linear fit between the log of event intensity and log of event duration for a given return period (a log/log linear fit), and extrapolating to short durations. In order to improve the fit, additional data points were first derived by applying the ARF method described above to 3, 9, 15, 18, and 21 hour event durations. This provided a total of eight intensity vs. duration data points for each desired return period. The log/log linear relationship was then fit to these eight values, and intensities for the desired short duration events (5 minutes, 10 minutes, 15 minutes, 30 minutes, 1 hour, and 2 hours) were extrapolated. It is important to note that uncertainty in extrapolated intensities increases sharply as the duration decreases, and results for durations shorter than an hour must be interpreted with caution.

3.3. Monte Carlo Approach & Confidence Bounds

Extreme precipitation calculations are typically sensitive to outliers in a data set; that is, one or two events can dramatically shift return period estimates. Ideally, return period estimates would be based on extremely long precipitation time series; unfortunately, these are rarely available either in observations or RCM output. In the absence of these long time series, it is helpful to assess the robustness of the results by performing repeated calculations using a sub-sample of the full data set (referred to as a ‘Monte Carlo’ approach). By providing a range of results, this approach can be used to estimate confidence bounds on the results. A Monte Carlo approach has been used in the current study. Random samples of twenty-five yearly precipitation maxima (of the available thirty-three years) were taken from the RCM 20th and 21st century simulations. The procedures described above were then applied to obtain return period intensities for all requested durations (5 minutes, 10 minutes, 15 minutes, 30 minutes, 1 hour, 2 hours, 6 hours, 12 hours, and 24 hours). This sub-sampling approach was repeated 100,000 times for each of the seven NARCCAP model combinations, for a total of 700,000 estimates of each requested IDF data point. The mean, 5th percentile, and 95th percentile of the resulting 700,000 estimates is reported in the following table as the mean, minimum, and maximum projection respectively.

4.0 Results

Results of are provided in the following tables, respectively giving the mean (Table 1), 95th percentile (Table 2), and 5th percentile (Table 3) of the 700,000 Monte Carlo tests. Requested intensities are given in millimeters of precipitation for requested return periods and event durations.

Table 1: Mean of the Monte Carlo estimates of future (2038-2070) return period events for various durations.

Return Period	5 mins	10 mins	15 mins	30 mins	1 hour	2 hour	6 hour	12 hour	24 hour
2 year	9.7	12.5	14.4	18.5	23.8	30.5	48.5	59.9	71.4
5 year	11.9	15.5	18.1	23.5	30.7	40.0	64.5	82.1	98.3
10 year	13.4	17.5	20.5	26.9	35.3	46.3	75.1	96.9	116.3
20 year	14.9	19.5	22.9	30.1	39.7	52.3	85.2	111.1	133.5
25 year	15.3	20.2	23.7	31.2	41.1	54.2	88.4	115.6	139.0
50 year	16.8	22.1	26.0	34.3	45.4	60.1	98.3	129.5	155.9
100 year	18.2	24.1	28.3	37.5	49.7	65.9	108.1	143.4	172.7

Table 2: The 95th percentile of the Monte Carlo estimates of future (2038-2070) return period events for various durations.

	Duration								
Return Period	**5 mins**	**10 mins**	**15 mins**	**30 mins**	**1 hour**	**2 hour**	**6 hour**	**12 hour**	**24 hour**
2 year	12.0	14.7	16.7	20.8	26.4	34.1	54.4	70.3	82.3
5 year	15.0	19.2	22.1	28.5	36.7	47.4	74.2	97.5	114.5
10 year	17.8	22.7	26.3	33.8	43.6	56.5	87.5	115.7	136.3
20 year	20.6	26.3	30.4	39.1	50.4	65.2	100.4	133.2	157.1
25 year	21.5	27.4	31.7	40.7	52.5	67.9	104.5	138.8	163.8
50 year	24.3	31.0	35.8	45.9	59.2	76.5	117.0	156.0	185.2
100 year	27.1	34.5	39.9	51.1	65.7	85.0	129.5	173.1	207.0

Table 3: The 5th percentile of the Monte Carlo estimates of future (2038-2070) return period events for various durations.

	Duration								
Return Period	**5 mins**	**10 mins**	**15 mins**	**30 mins**	**1 hour**	**2 hour**	**6 hour**	**12 hour**	**24 hour**
2 year	7.3	9.7	11.5	15.3	20.3	27.0	44.2	53.2	59.4
5 year	9.1	12.2	14.5	19.4	26.0	34.6	57.3	70.9	81.8
10 year	9.5	13.0	15.7	21.4	29.1	39.1	65.5	81.2	95.2
20 year	10.0	13.8	16.7	23.0	31.7	43.3	73.3	90.5	107.3
25 year	10.1	14.1	17.1	23.6	32.7	44.7	75.7	93.5	111.1
50 year	10.6	14.8	18.1	25.2	35.2	48.7	83.2	102.5	122.7
100 year	11.1	15.7	19.2	26.9	37.9	52.9	90.7	111.2	134.1

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APPENDIX F

Design Hyetographs based on Climate Change Projections

% Time	1:2 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	1.26	1.04	1.41	1.63	1.39	1.81	2.07	1.83	2.31	3.30	3.01	3.70	0.61	0.54	0.72	4.81	4.01	5.55
16.67%	3.17	2.62	3.56	4.01	3.43	4.46	5.20	4.60	5.81	8.25	7.52	9.25	1.22	1.08	1.43	12.11	10.08	13.97
25.00%	5.57	4.60	6.25	7.16	6.13	7.96	9.19	8.14	10.27	14.58	13.29	16.34	4.22	3.75	4.95	21.46	17.87	24.76
33.33%	8.74	7.22	9.81	11.23	9.61	12.49	14.44	12.78	16.14	22.94	20.91	25.71	10.76	9.56	12.63	33.73	28.09	38.91
41.67%	12.50	10.33	14.04	16.06	13.74	17.86	20.65	18.27	23.07	32.84	29.94	36.81	22.74	20.20	26.69	48.28	40.19	55.68
50.00%	15.45	12.77	17.35	19.80	16.94	22.02	25.46	22.53	28.45	40.49	36.91	45.38	37.71	33.51	44.26	59.52	49.55	68.65
58.33%	16.43	13.58	18.46	21.10	18.06	23.47	27.10	23.98	30.28	43.07	39.27	48.28	48.53	43.12	56.96	63.30	52.70	73.02
66.67%	17.20	14.21	19.32	22.02	18.84	24.49	28.30	25.05	31.63	45.00	41.03	50.44	54.52	48.44	63.99	66.22	55.14	76.39
75.00%	17.80	14.71	19.99	22.84	19.54	25.40	29.31	25.94	32.76	46.59	42.48	52.22	57.51	51.10	67.50	68.55	57.07	79.07
83.33%	18.18	15.02	20.42	23.38	20.00	26.00	29.99	26.54	33.51	47.69	43.48	53.46	58.68	52.14	68.87	70.17	58.42	80.94
91.67%	18.40	15.20	20.67	23.60	20.19	26.24	30.32	26.83	33.89	48.24	43.99	54.07	59.29	52.68	69.58	70.93	59.05	81.81
100.00%	18.51	15.29	20.79	23.76	20.33	26.42	30.52	27.00	34.10	48.52	44.24	54.38	59.90	53.22	70.30	71.36	59.41	82.31

% Time	1:5 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	1.60	1.32	1.93	2.10	1.78	2.51	2.71	2.34	3.22	4.39	3.89	5.05	0.84	0.72	0.99	6.63	5.51	7.72
16.67%	4.03	3.33	4.87	5.18	4.39	6.20	6.81	5.89	8.08	10.97	9.74	12.62	1.67	1.45	1.99	16.68	13.88	19.44
25.00%	7.08	5.85	8.56	9.24	7.83	11.06	12.05	10.41	14.29	19.39	17.20	22.30	5.78	4.99	6.86	29.57	24.60	34.45
33.33%	11.11	9.18	13.43	14.50	12.28	17.34	18.93	16.36	22.45	30.50	27.07	35.09	14.74	12.74	17.51	46.47	38.67	54.14
41.67%	15.90	13.13	19.23	20.73	17.56	24.80	27.07	23.39	32.10	43.67	38.75	50.23	31.16	26.93	37.00	66.51	55.34	77.48
50.00%	19.65	16.23	23.76	25.56	21.65	30.58	33.37	28.84	39.58	53.84	47.78	61.92	51.68	44.67	61.37	82.00	68.22	95.53
58.33%	20.90	17.26	25.27	27.24	23.08	32.59	35.52	30.69	42.12	57.28	50.83	65.88	66.50	57.48	78.97	87.21	72.56	101.60
66.67%	21.87	18.07	26.45	28.43	24.09	34.01	37.10	32.06	43.99	59.84	53.10	68.82	74.71	64.57	88.72	91.23	75.91	106.29
75.00%	22.63	18.70	27.37	29.48	24.98	35.27	38.42	33.20	45.56	61.96	54.98	71.26	78.82	68.12	93.59	94.43	78.57	110.02
83.33%	23.12	19.10	27.96	30.18	25.57	36.11	39.30	33.96	46.61	63.42	56.28	72.95	80.41	69.50	95.49	96.67	80.43	112.62
91.67%	23.39	19.33	28.30	30.46	25.81	36.44	39.75	34.35	47.13	64.15	56.93	73.79	81.25	70.22	96.48	97.71	81.30	113.84
100.00%	23.53	19.44	28.47	30.67	25.99	36.70	40.00	34.56	47.43	64.52	57.25	74.21	82.08	70.95	97.47	98.31	81.79	114.53

% Time	1:10 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	1.82	1.45	2.30	2.41	1.99	2.99	3.14	2.65	3.83	5.11	4.46	5.95	0.99	0.83	1.18	7.84	6.42	9.19
16.67%	4.60	3.66	5.79	5.96	4.91	7.37	7.88	6.66	9.63	12.77	11.14	14.89	1.97	1.66	2.36	19.73	16.16	23.13
25.00%	8.09	6.43	10.18	10.63	8.76	13.15	13.94	11.78	17.03	22.56	19.68	26.30	6.82	5.72	8.14	34.97	28.65	40.99
33.33%	12.69	10.09	15.97	16.66	13.74	20.62	21.89	18.51	26.75	35.50	30.97	41.38	17.40	14.59	20.77	54.97	45.03	64.43
41.67%	18.16	14.44	22.85	23.83	19.64	29.49	31.31	26.47	38.25	50.83	44.34	59.25	36.78	30.84	43.90	78.67	64.44	92.20
50.00%	22.44	17.85	28.24	29.38	24.22	36.37	38.61	32.64	47.17	62.66	54.66	73.04	61.00	51.15	72.82	96.99	79.45	113.68
58.33%	23.87	18.98	30.04	31.31	25.81	38.76	41.09	34.73	50.20	66.66	58.15	77.70	78.49	65.82	93.70	103.16	84.50	120.91
66.67%	24.98	19.87	31.44	32.68	26.94	40.45	42.91	36.28	52.43	69.64	60.75	81.18	88.18	73.95	105.26	107.92	88.39	126.48
75.00%	25.85	20.56	32.53	33.89	27.94	41.95	44.45	37.57	54.31	72.11	62.91	84.06	93.02	78.01	111.05	111.71	91.50	130.92
83.33%	26.41	21.00	33.23	34.69	28.60	42.94	45.47	38.43	55.55	73.81	64.39	86.04	94.90	79.59	113.30	114.35	93.66	134.02
91.67%	26.73	21.25	33.63	35.02	28.87	43.34	45.98	38.87	56.18	74.67	65.14	87.03	95.89	80.42	114.47	115.58	94.67	135.46
100.00%	26.89	21.38	33.83	35.26	29.06	43.64	46.27	39.11	56.53	75.09	65.51	87.53	96.88	81.25	115.65	116.29	95.25	136.29

% Time	1:20 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	2.04	1.56	2.65	2.72	2.17	3.45	3.55	2.94	4.42	5.80	4.99	6.83	1.13	0.92	1.36	9.00	7.24	10.60
16.67%	5.15	3.94	6.68	6.70	5.36	8.51	8.91	7.37	11.10	14.49	12.46	17.08	2.26	1.84	2.71	22.66	18.21	26.67
25.00%	9.06	6.93	11.75	11.95	9.56	15.18	15.75	13.04	19.64	25.60	22.02	30.18	7.82	6.37	9.38	40.16	32.28	47.26
33.33%	14.22	10.87	18.43	18.75	15.00	23.81	24.74	20.48	30.85	40.29	34.65	47.49	19.96	16.26	23.93	63.13	50.73	74.29
41.67%	20.35	15.56	26.38	26.81	21.45	34.04	35.38	29.29	44.11	57.68	49.60	67.99	42.18	34.36	50.58	90.34	72.61	106.31
50.00%	25.15	19.22	32.60	33.06	26.45	41.98	43.63	36.12	54.39	71.10	61.15	83.82	69.96	56.99	83.88	111.38	89.52	131.08
58.33%	26.75	20.45	34.68	35.23	28.18	44.74	46.44	38.44	57.89	75.64	65.06	89.17	90.02	73.34	107.94	118.46	95.21	139.41
66.67%	27.99	21.40	36.29	36.77	29.42	46.70	48.50	40.14	60.46	79.03	67.97	93.16	101.13	82.39	121.26	123.92	99.60	145.84
75.00%	28.97	22.14	37.56	38.13	30.50	48.42	50.23	41.58	62.62	81.83	70.38	96.46	106.68	86.92	127.92	128.27	103.10	150.96
83.33%	29.59	22.62	38.36	39.03	31.23	49.57	51.38	42.53	64.06	83.76	72.04	98.74	108.84	88.68	130.52	131.31	105.53	154.53
91.67%	29.95	22.89	38.82	39.40	31.52	50.03	51.96	43.01	64.78	84.73	72.87	99.88	109.97	89.60	131.87	132.73	106.67	156.20
100.00%	30.12	23.03	39.05	39.67	31.74	50.38	52.29	43.29	65.19	85.21	73.28	100.45	111.11	90.52	133.23	133.54	107.32	157.15

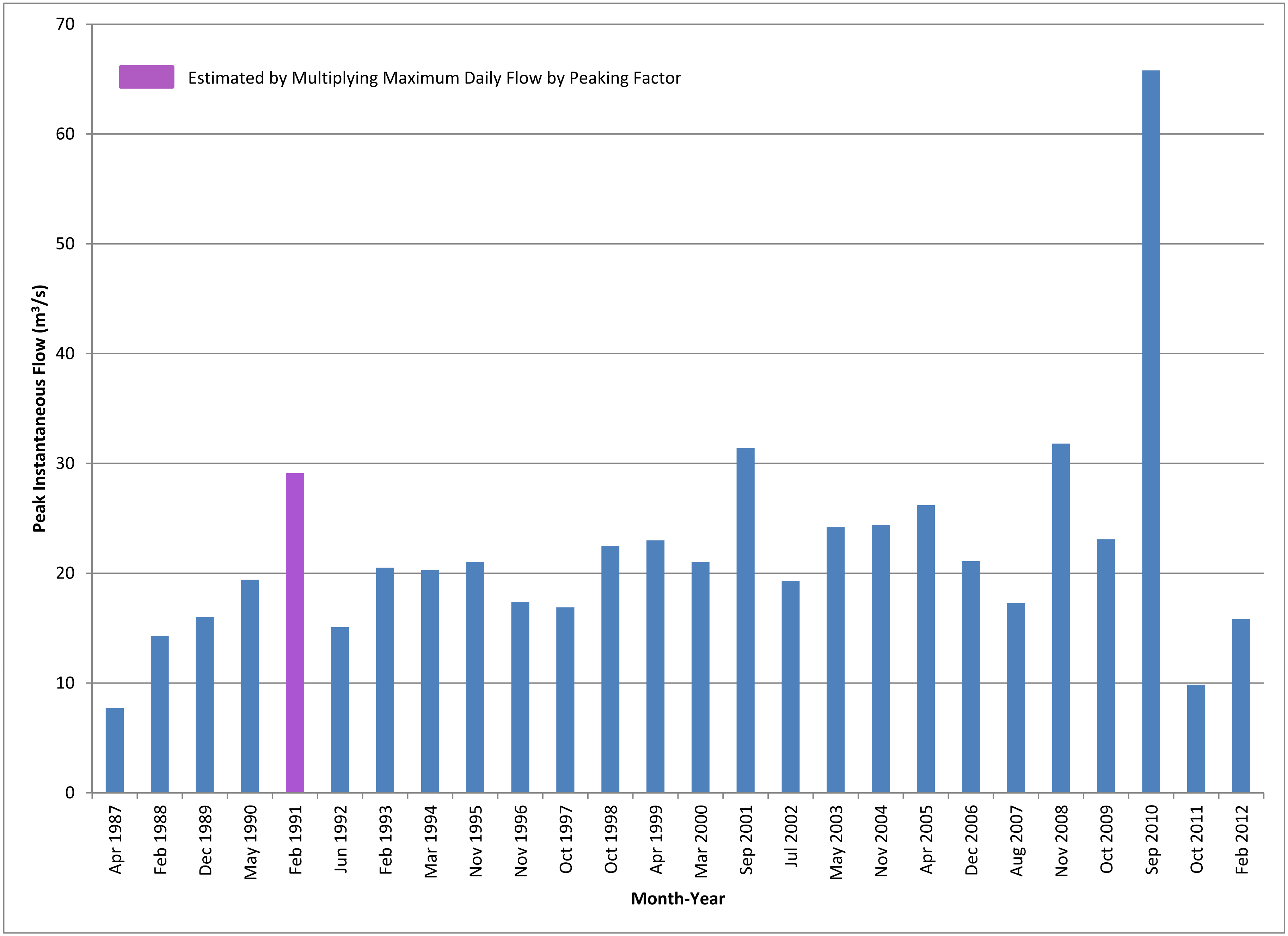
% Time	1:25 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	2.11	1.60	2.76	2.81	2.24	3.60	3.68	3.03	4.61	6.01	5.15	7.11	1.18	0.95	1.41	9.37	7.49	11.04
16.67%	5.33	4.04	6.96	6.94	5.52	8.87	9.23	7.61	11.57	15.03	12.88	17.76	2.35	1.90	2.83	23.59	18.86	27.79
25.00%	9.38	7.11	12.25	12.38	9.85	15.82	16.33	13.46	20.47	26.56	22.75	31.38	8.13	6.58	9.77	41.80	33.42	49.26
33.33%	14.71	11.16	19.21	19.41	15.44	24.81	25.65	21.14	32.15	41.80	35.80	49.39	20.76	16.79	24.93	65.70	52.53	77.42
41.67%	21.05	15.97	27.49	27.76	22.08	35.47	36.68	30.23	45.98	59.84	51.25	70.70	43.88	35.48	52.69	94.03	75.17	110.80
50.00%	26.01	19.73	33.97	34.23	27.23	43.74	45.22	37.28	56.69	73.77	63.19	87.17	72.78	58.85	87.38	115.93	92.68	136.61
58.33%	27.67	20.99	36.14	36.48	29.02	46.62	48.13	39.68	60.34	78.48	67.22	92.73	93.65	75.72	112.44	123.30	98.58	145.29
66.67%	28.95	21.96	37.82	38.07	30.29	48.65	50.27	41.44	63.02	81.99	70.23	96.88	105.21	85.07	126.32	128.99	103.12	151.99
75.00%	29.96	22.73	39.14	39.48	31.41	50.45	52.06	42.92	65.27	84.90	72.72	100.31	110.99	89.74	133.26	133.52	106.74	157.33
83.33%	30.61	23.22	39.98	40.42	32.15	51.65	53.26	43.90	66.77	86.90	74.43	102.68	113.23	91.56	135.96	136.67	109.27	161.05
91.67%	30.97	23.50	40.46	40.79	32.45	52.13	53.86	44.40	67.52	87.90	75.29	103.87	114.41	92.51	137.37	138.15	110.45	162.79
100.00%	31.16	23.64	40.70	41.07	32.68	52.49	54.20	44.68	67.95	88.40	75.72	104.46	115.59	93.46	138.79	138.99	111.12	163.78

% Time	1:50 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	2.33	1.71	3.12	3.11	2.41	4.06	4.07	3.30	5.19	6.68	5.66	7.96	1.32	1.04	1.59	10.51	8.27	12.49
16.67%	5.87	4.31	7.86	7.66	5.95	10.00	10.23	8.30	13.03	16.71	14.15	19.90	2.64	2.09	3.18	26.46	20.81	31.44
25.00%	10.32	7.59	13.82	13.67	10.61	17.84	18.09	14.68	23.05	29.52	25.01	35.16	9.12	7.21	10.98	46.90	36.89	55.71
33.33%	16.19	11.90	21.68	21.44	16.64	27.98	28.42	23.06	36.20	46.46	39.35	55.33	23.27	18.41	28.03	73.71	57.98	87.57
41.67%	23.18	17.04	31.04	30.66	23.79	40.01	40.64	32.97	51.77	66.51	56.34	79.21	49.18	38.90	59.24	105.48	82.98	125.32
50.00%	28.64	21.05	38.35	37.80	29.34	49.34	50.11	40.66	63.83	82.00	69.45	97.65	81.56	64.52	98.25	130.05	102.31	154.51
58.33%	30.47	22.39	40.79	40.29	31.27	52.58	53.33	43.27	67.94	87.24	73.89	103.89	104.95	83.02	126.42	138.32	108.81	164.34
66.67%	31.88	23.43	42.69	42.05	32.64	54.88	55.70	45.19	70.95	91.14	77.19	108.53	117.90	93.27	142.03	144.70	113.83	171.91
75.00%	33.00	24.25	44.18	43.60	33.84	56.91	57.69	46.81	73.49	94.37	79.93	112.38	124.38	98.39	149.83	149.78	117.83	177.95
83.33%	33.70	24.77	45.13	44.64	34.65	58.26	59.02	47.88	75.18	96.59	81.81	115.03	126.90	100.39	152.86	153.32	120.62	182.16
91.67%	34.11	25.07	45.67	45.05	34.97	58.80	59.68	48.42	76.02	97.71	82.76	116.36	128.22	101.43	154.45	154.98	121.92	184.12
100.00%	34.31	25.22	45.94	45.36	35.21	59.21	60.06	48.73	76.51	98.27	83.23	117.02	129.54	102.47	156.04	155.92	122.66	185.25

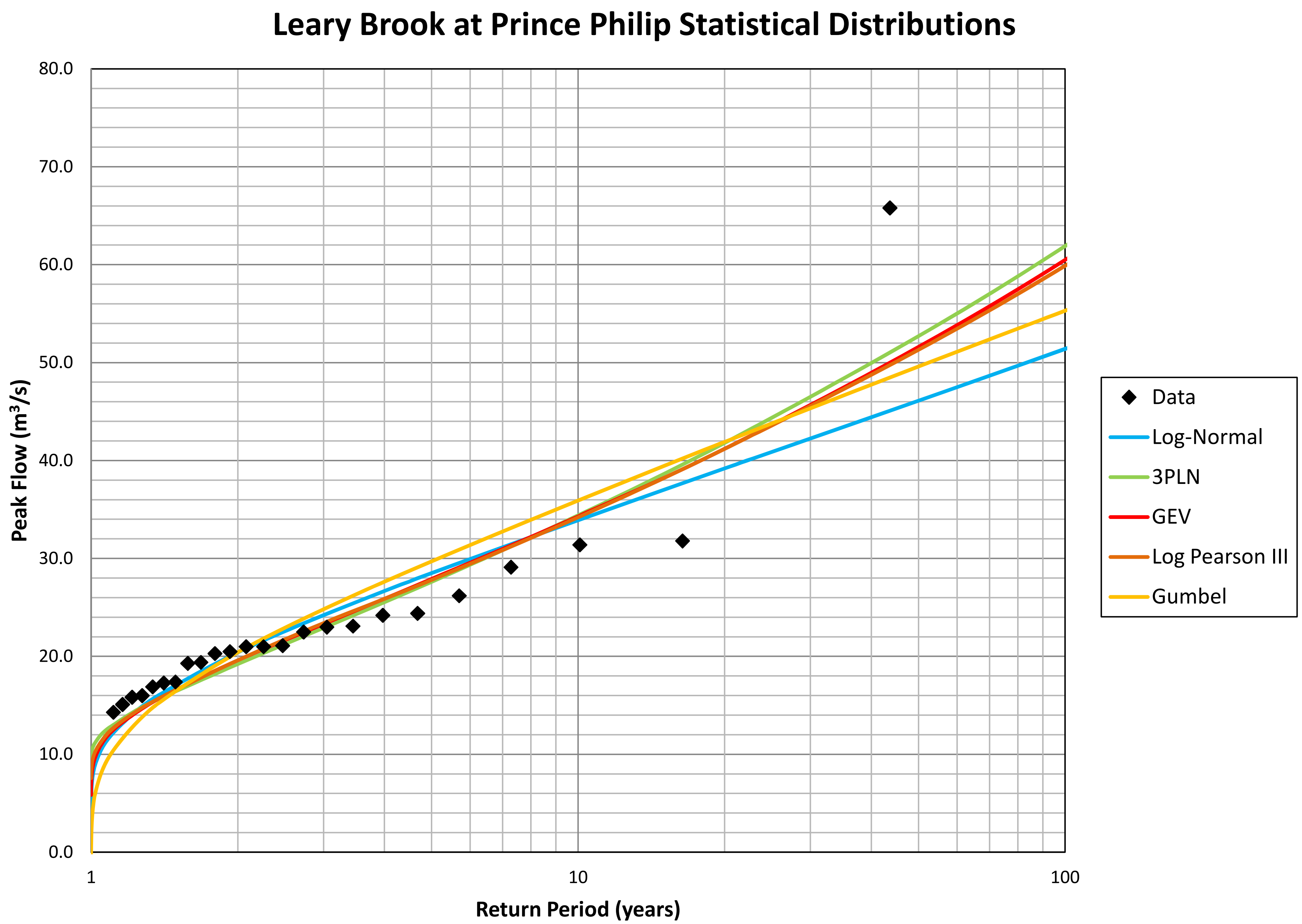
% Time	1:100 AEP Cumulative Rainfall (mm)																	
	0.5 Hour	L 95%	U95%	1 Hour	L 95%	U95%	2 Hour	L 95%	U95%	6 Hour	L 95%	U95%	12 Hour	L 95%	U95%	24 Hour	L 95%	U95%
0.00%	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8.33%	2.54	1.83	3.47	3.40	2.60	4.50	4.47	3.59	5.76	7.35	6.17	8.81	1.46	1.13	1.76	11.65	9.04	13.96
16.67%	6.41	4.61	8.74	8.39	6.40	11.11	11.23	9.01	14.48	18.38	15.42	22.03	2.92	2.27	3.53	29.31	22.76	35.13
25.00%	11.28	8.11	15.37	14.97	11.42	19.81	19.86	15.94	25.60	32.48	27.25	38.92	10.09	7.83	12.18	51.95	40.33	62.26
33.33%	17.69	12.72	24.10	23.47	17.91	31.07	31.19	25.04	40.21	51.10	42.88	61.24	25.76	19.98	31.10	81.65	63.39	97.87
41.67%	25.32	18.20	34.50	33.57	25.61	44.43	44.60	35.80	57.50	73.16	61.39	87.68	54.44	42.23	65.72	116.85	90.72	140.05
50.00%	31.29	22.49	42.63	41.39	31.58	54.78	55.00	44.15	70.91	90.20	75.68	108.09	90.30	70.03	108.99	144.06	111.85	172.68
58.33%	33.28	23.92	45.35	44.11	33.65	58.38	58.53	46.98	75.46	95.96	80.51	115.00	116.19	90.12	140.25	153.22	118.97	183.66
66.67%	34.83	25.04	47.46	46.04	35.12	60.94	61.13	49.07	78.82	100.25	84.11	120.14	130.53	101.24	157.56	160.29	124.45	192.12
75.00%	36.05	25.91	49.11	47.74	36.42	63.19	63.32	50.82	81.63	103.80	87.09	124.40	137.70	106.80	166.22	165.91	128.82	198.87
83.33%	36.82	26.47	50.17	48.88	37.29	64.69	64.77	51.99	83.51	106.25	89.15	127.33	140.49	108.97	169.58	169.84	131.87	203.57
91.67%	37.26	26.79	50.77	49.33	37.63	65.29	65.50	52.57	84.45	107.48	90.18	128.80	141.95	110.10	171.35	171.67	133.29	205.77
100.00%	37.49	26.94	51.07	49.67	37.89	65.74	65.92	52.91	84.98	108.09	90.69	129.54	143.41	111.23	173.11	172.72	134.10	207.02

APPENDIX G

Single Station Frequency Analysis Results



Estimated by Multiplying Maximum Daily Flow by Peaking Factor
Peak Instantaneous Flow (m³/s)
Month-Year
0
10
20
30
40
50
60
70
Apr 1987
Feb 1988
Dec 1989
May 1990
Feb 1991
Jun 1992
Feb 1993
Mar 1994
Nov 1995
Nov 1996
Oct 1997
Oct 1998
Apr 1999
Mar 2000
Sep 2001
Jul 2002
May 2003
Nov 2004
Apr 2005
Dec 2006
Aug 2007
Nov 2008
Oct 2009
Sep 2010
Oct 2011
Feb 2012



Leary Brook at Prince Philip Statistical Distributions
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Peak Flow (m³/s)
1
10
100
Return Period (years)
Data
Log-Normal
3PLN
GEV
Log Pearson III
Gumbel

--- RUN TEST FOR GENERAL RANDOMNESS ---

02ZM020 Leary Brook at Prince Philip

ANNUAL MAXIMUM DAILY FLOW SERIES 1987 TO 2012 DRAINAGE AREA = 17.80000

THE NUMBER OF RUNS ABOVE AND BELOW THE MEDIAN (RUNAB) = 11

THE NUMBER OF OBSERVATIONS ABOVE THE MEDIAN(N1) = 13

THE NUMBER OF OBSERVATIONS BELOW THE MEDIAN(N2) = 13

Range at 5% level of significance: 9. to 19. NOT SIGNIFICANT

Interpretation: The null hypothesis is that the data are random.

At the 5% level of significance, the null hypothesis cannot be rejected. That is, the sample is significantly random.

--- SPEARMAN TEST FOR INDEPENDENCE ---

02ZM020 Leary Brook at Prince Philip

ANNUAL MAXIMUM DAILY FLOW SERIES 1987 TO 2012 DRAINAGE AREA = 17.80000

SPEARMAN RANK ORDER SERIAL CORRELATION COEFF = .155 D.F.= 23

CORRESPONDS TO STUDENTS T = .753

CRITICAL T VALUE AT 5% LEVEL = 1.714 NOT SIGNIFICANT

- - - - 1% - = 2.500 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant serial dependence.

--- SPEARMAN TEST FOR TREND ---

02ZM020 Leary Brook at Prince Philip

ANNUAL MAXIMUM DAILY FLOW SERIES 1987 TO 2012 DRAINAGE AREA = 17.80000

SPEARMAN RANK ORDER CORRELATION COEFF = -.362 D.F.= 24

CORRESPONDS TO STUDENTS T =-1.905

CRITICAL T VALUE AT 5% LEVEL =-2.064 NOT SIGNIFICANT

- - - - 1% - =-2.797 NOT SIGNIFICANT

Interpretation: The null hypothesis is that the serial (lag-one) correlation is zero.

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant trend.

--- MANN-WHITNEY SPLIT SAMPLE TEST FOR HOMOGENEITY ---

02ZM020 Leary Brook at Prince Philip

ANNUAL MAXIMUM FLOW SERIES 1987 TO 2012 DRAINAGE AREA= 17.80000

SPLIT BY TIME SPAN, SUBSAMPLE 1 SAMPLE SIZE= 13

SUBSAMPLE 2 SAMPLE SIZE= 13

MANN-WHITNEY U = 47.5

CRITICAL U VALUE AT 5% SIGNIFICANT LEVEL = 51.0 SIGNIFICANT

- - - - - 1% - - = 39.0 NOT SIGNIFICANT

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 5% level of significance, there is a significant difference in location, but not so at the 1% level. That is, the location difference is significant, but not highly so.

APPENDIX H

Hydraulic Structure Data Sheets

Culvert Data Sheet

Date of Survey: Dec 14 2012

Culvert No.: 1
River: Rennies
Location: Great Eastern Ave.

Shape: Box
Size: 5950 X 1920 mm
Length: 26.3 m
Material: CMP

Road CL Elevation: 174.00 m

Headwall
Width ______ m
Height ______ m
Length ______ m
Material ______

Wingwall
Width 0.41 m
Height 2.7 m
Length 15 m
Angle 125 Deg (approx)
Material Concrete

Comments:

Condition: Box culvert shows no sign of deterioration. Inlet/Outlet free of sediment buildup. Rip rap protection appears to be small.

Date of Construction: 2008
Other:

Provide elevations and photos as follows:

Upstream
Invert Elevation: 170.43 m
Edge of Asphalt Elevation: 174.00 m

Downstream
Invert Elevation: 169.84 m
Edge of Asphalt Elevation: 174.01 m

Culvert Data Sheet

Date of Survey: Dec 14 2012

Culvert No.: 2
River: Rennies
Location: Next to Tim Hortons

Shape: Circular
Size: 1200 mm
Length: 5.81 m
Material: CMP

Road CL Elevation: no road m

Headwall
Width ______ m
Height ______ m
Length ______ m
Material ______

Wingwall
Width ______ m
Height ______ m
Length ______ m
Angle ______ Deg (approx)
Material ______

Comments:

Condition: Culvert bottom has sections that is completely rusted through. Inlet has sediment buildup and trees obstructing flow.

Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream
Invert Elevation: 154.79 m
Edge of Asphalt Elevation: m

Downstream
Invert Elevation: 154.74 m
Edge of Asphalt Elevation: m

Culvert Data Sheet

Date of Survey: April 11 2013

Culvert No.: 3
River: Rennies
Location: NL Power Access

Shape: Circular
Size: 1200 mm
Length: 10 m
Material: CMP

Road CL Elevation: 154.55 m

Headwall
Width ____ m
Height ____ m
Length ____ m
Material ____

Wingwall
Width ____ m
Height ____ m
Length ____ m
Angle ____ Deg (approx)
Material ____

Comments:

Condition: Culvert showing rust on bottom half, still structurally intact. No obstructions at Inlet/Outlet.

Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream
Invert Elevation: 152.06 m
Edge of Asphalt Elevation: 154.36 m

Downstream
Invert Elevation: 152.30 m
Edge of Asphalt Elevation: 152.73 m

Culvert Data Sheet

Date of Survey: Dec 14 2012

Culvert No.: 4
River: Rennies
Location: Lady Smith Dr.

Shape: Box
Size: 5950 X 2180 mm
Length: 26.97 m
Material: Concrete

Road CL Elevation: 152.72 m

Headwall
Width _____ m
Height _____ m
Length _____ m
Material _____

Wingwall
Width 0.41 m
Height 3.34 m
Length 14.7 m
Angle 125 Deg (approx)
Material Concrete

Comments:

Condition: Box culvert shows no sign of deterioration. Inlet/Outlet free of sediment buildup. Rip rap protection appears to be small.

Date of Construction: 2008
Other:

Provide elevations and photos as follows:

Upstream
Invert Elevation: 148.83 m
Edge of Asphalt Elevation: 152.79 m

Downstream
Invert Elevation: 148.01 m
Edge of Asphalt Elevation: 152.66 m

Culvert Data Sheet

Date of Survey: April 11 2012

Culvert No.: 5
River: Rennies
Location: Wing n' it

Shape: Low Profile Arch
Size: 7310x2090 mm
Length: 13.23 m
Material: CMP

Road CL Elevation: 148.01 m

Headwall
Width ______ m
Height ______ m
Length ______ m
Material ______

Wingwall
Width 0.31 m
Height 2.72 m
Length 14.18 m
Angle Various Deg (approx)
Material Concrete

Comments:

Condition: Culvert shows no sign of deterioration. Inlet as some rip rap buildup. Outlet free of material buildup. Rip rap protection appears to be small.

Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream

Invert Elevation:	145.43 m
Edge of Asphalt Elevation:	148.03 m

Downstream

Invert Elevation:	145.29 m
Edge of Asphalt Elevation:	147.99 m

Bridge Data Sheet

Date of Survey: April 11 2012

Bridge No.:	1	Span:	3.4 m
River:	Rennies	Height (underside of bridge to river):	1.35 m
Location:	Private Driveway	Length (parrallel to river):	3.42 m
Upstream Invert Ele:	m	No. of Piers:	
Underside of Deck Ele:	142.72 m	Width of Pier:	mm
Top of Deck Ele:	143.08 m		

Comments:

Condition: Main structure of bridge

Date of Construction: Unknown

Other:

Provide Photos:

Upstream

Downstream

Culvert Data Sheet

Date of Survey: April 11 2012

Culvert No.: 6
River: Rennies
Location: Keith Gordan Car Sales

Shape: Circular
Size: 1500 mm
Length: 38 m
Material: Steel

Road CL Elevation: 143.13 m

Headwall
Width ______ m
Height ______ m
Length ______ m
Material ______

Wingwall
Width ______ m
Height ______ m
Length ______ m
Angle ______ Deg (approx)
Material ______

Comments:

Condition: Culvert bottom upstream completely rusted away. No obstructions in immediate opening of culvert.
Culvert bottom downstream rusted away in sections. Culvert bottom downstream rusted away in sections.
Free of obstructions.
Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream

Invert Elevation:	141.30 m
Edge of Asphalt Elevation:	143.13 m

Downstream

Invert Elevation:	140.43 m
Edge of Asphalt Elevation:	142.23 m

Culvert Data Sheet

Date of Survey: April 11 2012

Culvert No.: 7
River: Rennies
Location: Discount rentals

Shape: Circular
Size: 1500 mm
Length: m
Material: CMP

Road CL Elevation: 142.07 m

Headwall
Width m
Height m
Length m
Material

Wingwall
Width m
Height m
Length m
Angle Deg (approx)
Material

Comments:

Condition: Culvert showing signs of deterioration in its entirety, bottom completely rusted away. Retaining portion of bridge deteriorated allowing fill to fall into river. Inlet has some material buildup. Downstream headwall completely gave way resulting in major obstruction at culvert outlet.
Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream
Invert Elevation: 139.43 m
Edge of Asphalt Elevation: 142.07 m

Downstream
Invert Elevation: m
Edge of Asphalt Elevation: m

Culvert Data Sheet

Date of Survey: April 11 2012

Culvert No.: 8
River: Rennies
Location: Kelsey Drive

Shape: Arch
Size: 6000x2230 mm
Length: 42 m
Material: CMP

Road CL Elevation: 132.05 m

Headwall
Width ____ m
Height ____ m
Length ____ m
Material ____

Wingwall
Width 0.46 m
Height 2.74 m
Length 14.05 m
Angle 125 Deg (approx)
Material Concrete

Comments:

Condition: Culvert inlet has some buildup of rock/sediments.Culvert outlet more significant buildup of sediment/rock/ plant life. Culvert showing no signs of deterioration.

Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream

Invert Elevation:	135.43 m
Edge of Asphalt Elevation:	138.01 m

Downstream

Invert Elevation:	136.91 m
Edge of Asphalt Elevation:	138.09 m

Bridge Data Sheet

Date of Survey: April 11 2012

Bridge No.:	2	Span:	3.5 m
River:	Rennies	Height (underside of bridge to river):	2.2 m
Location:	Private Driveway	Length (parrallel to river):	7.5 m

Upstream Invert Ele:	m	No. of Piers:	
Underside of Deck Ele:	135.56 m	Width of Pier:	mm
Top of Deck Ele:	136.27 m		

Comments:

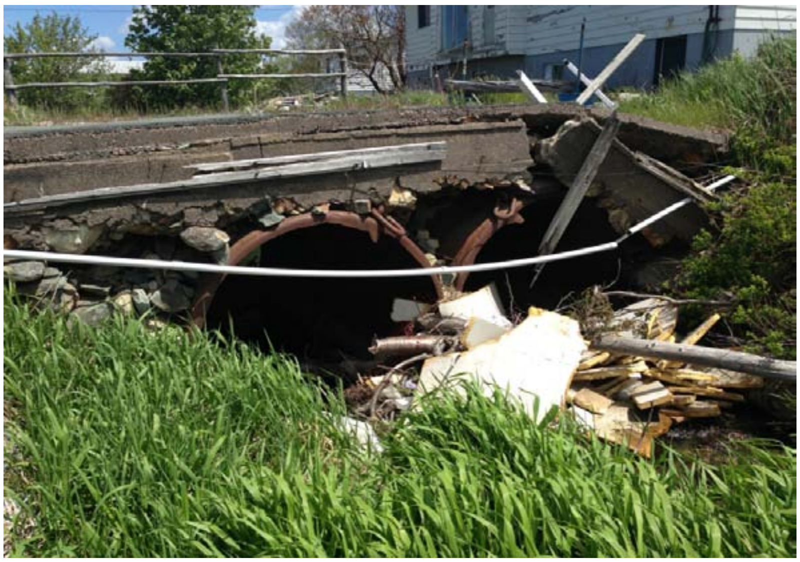
Condition: Bridge/Culverts severely deteriorated. Major building up garbage/sediments/wood at inlet.

Date of Construction: Unknown

Other:

Provide Photos:

Upstream



Downstream

Culvert Data Sheet

Date of Survey: April 11 2012

Culvert No.: 9
River: Rennies
Location: Personal Yard

Shape: Circular
Size: 2050 mm
Length: 41.77 m
Material: CMP

Road CL Elevation: m

Headwall
Width m
Height m
Length m
Material

Wingwall
Width m
Height m
Length m
Angle Deg (approx)
Material

Comments:

Condition: Culvert inlet has be deformed over time and surface rust has started on bottom half through entire length.
Culvart outlet is free of any buildup of materials.

Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream
Invert Elevation: 132.92 m
Edge of Asphalt Elevation: m

Downstream
Invert Elevation: 132.11 m
Edge of Asphalt Elevation: m

Culvert Data Sheet

Date of Survey: April 19 2012

Culvert No.: 10
River: Rennies
Location: New Gushue Ramp 1

Shape: Box
Size: 3220x3650 mm
Length: 37.09 m
Material: Concrete

Road CL Elevation: Road not complete m

Headwall
Width 0.332 m
Height 4.12 m
Length 18.02 m
Material Concrete Panels

Wingwall
Width m
Height m
Length m
Angle Deg (approx)
Material

Comments:

Condition: Structure is in great shape. No obstructions or sediment buildup to date.

Date of Construction: 2012
Other:

Provide elevations and photos as follows:

Upstream
Invert Elevation: 124.27 m
Edge of Asphalt Elevation: m

Downstream
Invert Elevation: 123.64 m
Edge of Asphalt Elevation: m

Culvert Data Sheet

Date of Survey:	April 19 2012		
Culvert No.:	11		
River:	Rennies		
Location:	New Gushue Ramp 2		
Shape:	Box		
Size:	3730x3320	mm	
Length:	25.15	m	
Material:	Concrete		
Road CL Elevation:	Road not complete	m	

Headwall		
Width	0.38	m
Height	3.41	m
Length	18.16	m
Material	Concrete Panels	

Wingwall		
Width		m
Height		m
Length		m
Angle		Deg (approx)
Material		

Comments:

Condition: Structure is in great shape. No obstructions or sediment buildup to date.

Date of Construction: 2012
Other:

Provide elevations and photos as follows:

Upstream

Invert Elevation:	116.55	m
Edge of Asphalt Elevation:		m

Downstream

Invert Elevation:	115.12	m
Edge of Asphalt Elevation:		m

Culvert Data Sheet

Date of Survey:	May 1 2013				
Culvert No.:	12		**Headwall**		
River:	Rennies		Width		m
Location:	North on Kelsey (Yellow Marsh)		Height		m
			Length		m
			Material		
Shape:	Box				
Size:	4000x1900	mm	**Wingwall**		
Length:	64.07	m	Width	0.45	m
Material:	Concrete		Height	3.45-4.55	m
			Length	13.57	m
			Angle	125	Deg (approx)
Road CL Elevation:	127.64	m	Material	Concrete	

Comments:

Condition: Structure is in great shape. No obstructions or sediment buildup to date.

Date of Construction: 2008

Other:

Provide elevations and photos as follows:

Upstream

Invert Elevation: 122.47 m

Edge of Asphalt Elevation: 127.81 m

Downstream

Invert Elevation: 117.55 m

Edge of Asphalt Elevation: 127.47 m

Culvert Data Sheet

Date of Survey: May 9 2013

Culvert No.: 17
River: Rennies
Location: Mews Place

Shape: Box
Size: 6081x1790 mm
Length: 15.84 m
Material: Concrete

Road CL Elevation: 102.90 m

Headwall
Width ______ m
Height ______ m
Length ______ m
Material ______

Wingwall
Width 0.478 m
Height 2.78 m
Length 15.53 m
Angle 126 Deg (approx)
Material Concrete

Comments:

Condition: Structure is in great shape. No obstructions with minimal sediment buildup to date.

Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream

Invert Elevation:	99.88 m
Edge of Asphalt Elevation:	102.87 m

Downstream

Invert Elevation:	99.66 m
Edge of Asphalt Elevation:	102.93 m

Culvert Data Sheet

Date of Survey:	May 9 2013				
Culvert No.:	18		**Headwall**		
River:	Rennies		Width	0.31	m
Location:	Pippy Place		Height	2.62	m
			Length	10.03	m
			Material	Concrete	
Shape:	Box				
Size:	8650x1900	mm	**Wingwall**		
Length:	15.82	m	Width	0.31	m
Material:	Concrete		Height	2.92	m
			Length	13.98	m
			Angle	139	Deg (approx)
Road CL Elevation:	148.01	m	Material	Concrete	

Comments:

Condition: Concrete structure is showing no signs of deterioration. Seditment buildup minimal at inlet/outlet.

Date of Construction: Unknown

Other:

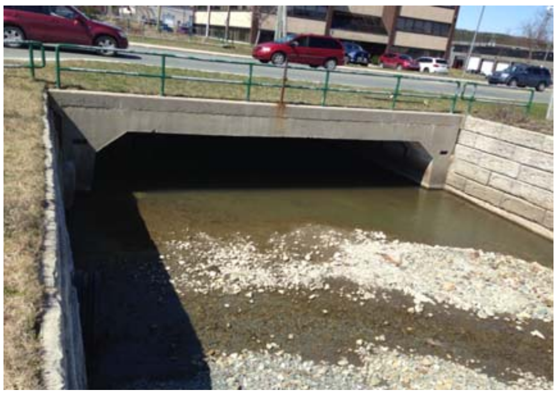
Provide elevations and photos as follows:

Upstream

Invert Elevation:	97.12 m
Edge of Asphalt Elevation:	99.94 m

Downstream

Invert Elevation:	97.18 m
Edge of Asphalt Elevation:	99.93 m



Bridge Data Sheet

Date of Survey: May 7 2013

Bridge No.:	3	Span:	8.94 m
River:	Rennies	Height (underside of bridge to river):	1.45 m
Location:	O'Leary Ave	Length (parrallel to river):	17.66 m

Upstream Invert Ele:	80.10 m	No. of Piers:	0
Underside of Deck Ele:	81.55 m	Width of Pier:	N/A mm
Top of Deck Ele:	81.95 m		

Comments:

Condition: Significant sediment/rock building at inlet/outlet.

Date of Construction: Unknown

Other:

Provide Photos:

Upstream

Downstream

Bridge Data Sheet

Date of Survey: May 7 2013

Bridge No.:	4	Span:	15.25 m
River:	Rennies	Height (underside of bridge to river):	1.05 m
Location:	Wicklow St	Length (parrallel to river):	15.46 m
Upstream Invert Ele:	58.64 m	No. of Piers:	0
Underside of Deck Ele:	59.69 m	Width of Pier:	N/A mm
Top of Deck Ele:	60.89 m		

Comments:

Condition: Significant vegetation upstream.

Date of Construction: 2005

Other:

Provide Photos:

Upstream

Downstream

Bridge Data Sheet

Date of Survey: May 7 2013

Bridge No.:	5	Span:	13.62 m
River:	Rennies	Height (underside of bridge to river):	1.38 m
Location:	Clinch Cres (W)	Length (parrallel to river):	20.2 m
Upstream Invert Ele:	57.29 m	No. of Piers:	0
Underside of Deck Ele:	58.65 m	Width of Pier:	N/A mm
Top of Deck Ele:	59.79 m		

Comments:

Condition: Significant vegetation upstream.

Date of Construction: Unknown

Other:

Provide Photos:

Upstream

Downstream

Bridge Data Sheet

Date of Survey: May 7 2013

Bridge No.:	6	Span:	23.23 m
River:	Rennies	Height (underside of bridge to river):	2.58 m
Location:	Clinch Cres (E)	Length (parrallel to river):	17.41 m
Upstream Invert Ele:	54.17 m	No. of Piers:	4
Underside of Deck Ele:	56.47 m	Width of Pier:	600 mm
Top of Deck Ele:	57.37 m		

Comments:

Condition: Significant vegetation upstream.

Date of Construction: Unknown

Other:

Provide Photos:

Upstream

Downstream

Culvert Data Sheet

Date of Survey: May 1 2013

Culvert No.: 12
River: Rennies
Location: North on Kelsey Dr.

Shape: Box
Size: 4000x1900 mm
Length: 64.07 m
Material: Concrete

Road CL Elevation: 127.64 m

Headwall
Width ______ m
Height ______ m
Length ______ m
Material ______

Wingwall
Width 0.45 m
Height 3.45-4.55 m
Length 13.57 m
Angle 125 Deg (approx)
Material Concrete

Comments:

Condition: Structure is in great shape. No obstructions or sediment buildup to date.

Date of Construction: 2008
Other:

Provide elevations and photos as follows:

Upstream
Invert Elevation: 122.47 m
Edge of Asphalt Elevation: 127.81 m

Downstream
Invert Elevation: 117.55 m
Edge of Asphalt Elevation: 127.47 m

Culvert Data Sheet

Date of Survey: May 9 2013

Culvert No.:	14		**Headwall**	
River:	Rennies		Width	m
Location:	Gushue Crossing (S)		Height	m
			Length	m
			Material	
Shape:	Circular			
Size:	2080	mm	**Wingwall**	
Length:	53.67	m	Width	m
Material:	CMP		Height	m
			Length	m
			Angle	Deg (approx)
Road CL Elevation:	Divided Highway	m	Material	

Comments:

Condition: Culvert is showing no signs of damage or obstructions with minimal surface rust.

Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream

Invert Elevation:	110.43 m
Edge of Asphalt Elevation:	113.17 m

Downstream

Invert Elevation:	108.11 m
Edge of Asphalt Elevation:	112.58 m

Culvert Data Sheet

Date of Survey: May 9 2013

Culvert No.: 15
River: Rennies
Location: Gushue Crossing (N)

Shape: Circular
Size: 2070
Length: 53.76 m
Material: CMP

Road CL Elevation: Divided Highway m

Headwall
Width m
Height m
Length m
Material

Wingwall
Width m
Height m
Length m
Angle Deg (approx)
Material

Comments:

Condition: Culvert is showing no signs of damage or obstructions with minimal surface rust.

Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream
Invert Elevation: 108.85 m
Edge of Asphalt Elevation: 113.50 m

Downstream
Invert Elevation: 107.32 m
Edge of Asphalt Elevation: 112.75 m

Culvert Data Sheet

Date of Survey: May 9 2013

Culvert No.: 16
River: Rennies
Location: NL Power Yard

Shape: Circular
Size: 1050 mm
Length: 9.75 m
Material: CMP

Road CL Elevation: 108.22 m

Headwall
Width ____ m
Height ____ m
Length ____ m
Material ____

Wingwall
Width 0.23 m
Height 2.72 m
Length 14.18 m
Angle Various Deg (approx)
Material Concrete

Comments:

Condition: Significant amount of vegetation at inlet/outlet. Culvert appears to be structurally intact.

Date of Construction: Unknown
Other:

Provide elevations and photos as follows:

Upstream
Invert Elevation: 105.98 m
Edge of Asphalt Elevation: 108.24 m


Downstream
Invert Elevation: 106.10 m
Edge of Asphalt Elevation: 108.21 m


Bridge Data Sheet

Date of Survey: May 7 2013

Bridge No.:	7	Span:	22.21 m
River:	Rennies	Height (underside of bridge to river):	1.67 m
Location:	Allandale Road	Length (parrallel to river):	23.72 m
Upstream Invert Ele:	52.62 m	No. of Piers:	0
Underside of Deck Ele:	54.64 m	Width of Pier:	N/A mm
Top of Deck Ele:	56.32 m		

Comments:

Condition: No obstructions, some vegetation growth downstream.

Date of Construction: Unknown

Other:

Provide Photos:

Upstream

Downstream

Bridge Data Sheet

Date of Survey: May 7 2013

Bridge No.:	8	Span:	7.47 m
River:	Rennies	Height (underside of bridge to river):	2.49 m
Location:	Prince Philip Drive	Length (parrallel to river):	27.09 m
Upstream Invert Ele:	51.68 m	No. of Piers:	1
Underside of Deck Ele:	54.17 m	Width of Pier:	300 mm
Top of Deck Ele:	54.78 m		

Comments:

Condition: Upstream (left side) inlet is completely blocked off with gabion basket. Right side has minimal obstructions.

Date of Construction: Unknown

Other:

Provide Photos:

Upstream

Downstream

Bridge Data Sheet

Date of Survey: May 7 2013

Bridge No.:	9	Span:	15.4 m
River:	Rennies	Height (underside of bridge to river):	2.91 m
Location:	Elizabeth Ave	Length (parrallel to river):	15.3 m
Upstream Invert Ele:	34.29 m	No. of Piers:	0
Underside of Deck Ele:	37.20 m	Width of Pier:	N/A mm
Top of Deck Ele:	37.81 m		

Comments:

Condition: Structure is in good shape with minimal obstructions to water flow.

Date of Construction: Unknown

Other:

Provide Photos:

Upstream

Downstream

Bridge Data Sheet

Date of Survey: May 7 2013

Bridge No.:	10	Span:	9.9 m
River:	Rennies	Height (underside of bridge to river):	2.74 m
Location:	Carpasian Road	Length (parrallel to river):	10.9 m
Upstream Invert Ele:	32.06 m	No. of Piers:	0
Underside of Deck Ele:	34.49 m	Width of Pier:	N/A mm
Top of Deck Ele:	34.80 m		

Comments:

Condition: Structure is in good shape with minimal obstructions to water flow.

Date of Construction: Unknown

Other:

Provide Photos:

Upstream

Downstream

Bridge Data Sheet

Date of Survey: May 6 2013

Bridge No.:	11	Span:	12.8 m
River:	Rennies	Height (underside of bridge to river):	1.75 m
Location:	Portugal Cove Road	Length (parrallel to river):	12.1 m
Upstream Invert Ele:	16.02 m	No. of Piers:	0
Underside of Deck Ele:	17.77 m	Width of Pier:	N/A mm
Top of Deck Ele:	19.19 m		

Comments:

Condition:

Date of Construction: Unknown

Other:

Provide Photos:

Upstream

Downstream

Bridge Data Sheet

Date of Survey: May 6 2013

Bridge No.:	12	Span:	12.2 m
River:	Rennies	Height (underside of bridge to river):	2.8 m
Location:	Kings Bridge Road	Length (parrallel to river):	17.3 m
Upstream Invert Ele:	13.05 m	No. of Piers:	0
Underside of Deck Ele:	15.14 m	Width of Pier:	N/A mm
Top of Deck Ele:	15.85 m		

Comments:

Condition: Vegetation growth upstream with sediment building downstream.

Date of Construction: Unknown

Other:

Provide Photos:

Upstream

Downstream

Bridge Data Sheet

Date of Survey: May 6 2013

Bridge No.: 13
River: Rennies
Location: Carnell Drive

Span: 8.2 m
Height (underside of bridge to river): 1.4 m
Length (parrallel to river): 9.7 m

Upstream Invert Ele: m
Underside of Deck Ele: m
Top of Deck Ele: 12.77 m
12.94

No. of Piers: 0
Width of Pier: N/A mm

Comments:

Condition: Under Construction - 2013

Date of Construction: Under Construction - 2013
Other:

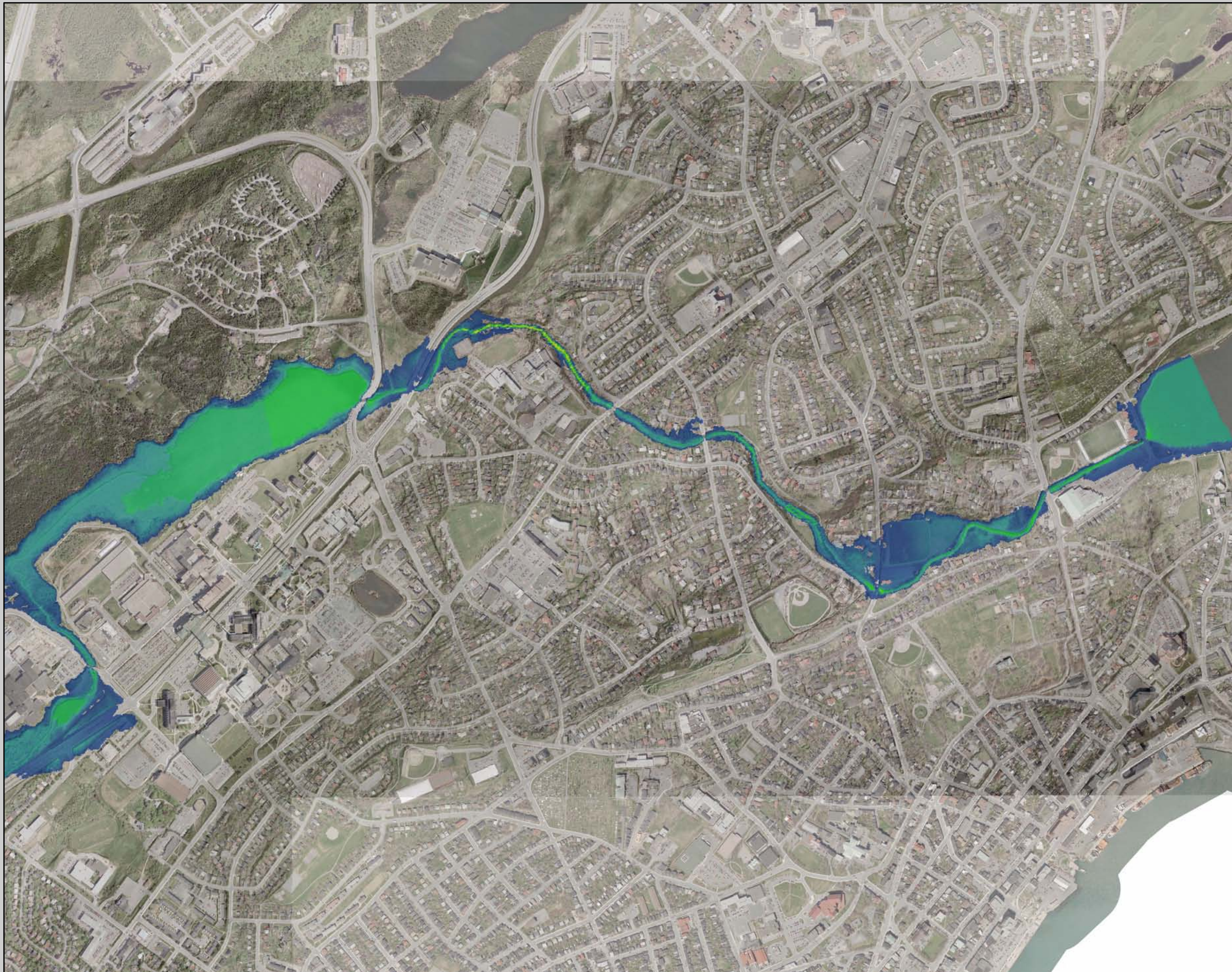
Provide Photos:

Upstream

Downstream

APPENDIX I

Calibrated Floodplain Mapping



ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Model Calibration
Igor

Rennies River Catchment
Stormwater Management Study

Water Depth (m)
High : 9
Low : 0

Map 1

CBCL
CBCL LIMITED
Consulting Engineers

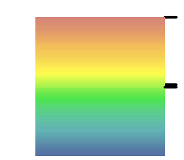
Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

N
W
E
S

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Model Calibration
Igor

Rennies River Catchment
Stormwater Management Study



Map 2

CBCL LIMITED
Consulting Engineers

Coordinate System:
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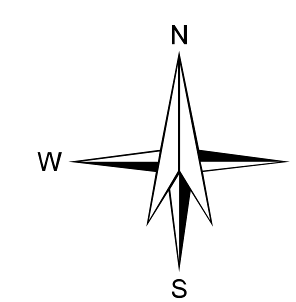
Units:Metre
Scale:1:10,000



ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Model Calibration
Igor

Rennies River Catchment
Stormwater Management Study

Water Depth (m)

High : 9

Low : 0

Map 3

CBCL LIMITED
Consulting Engineers

Coordinate System:
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Units:Metre
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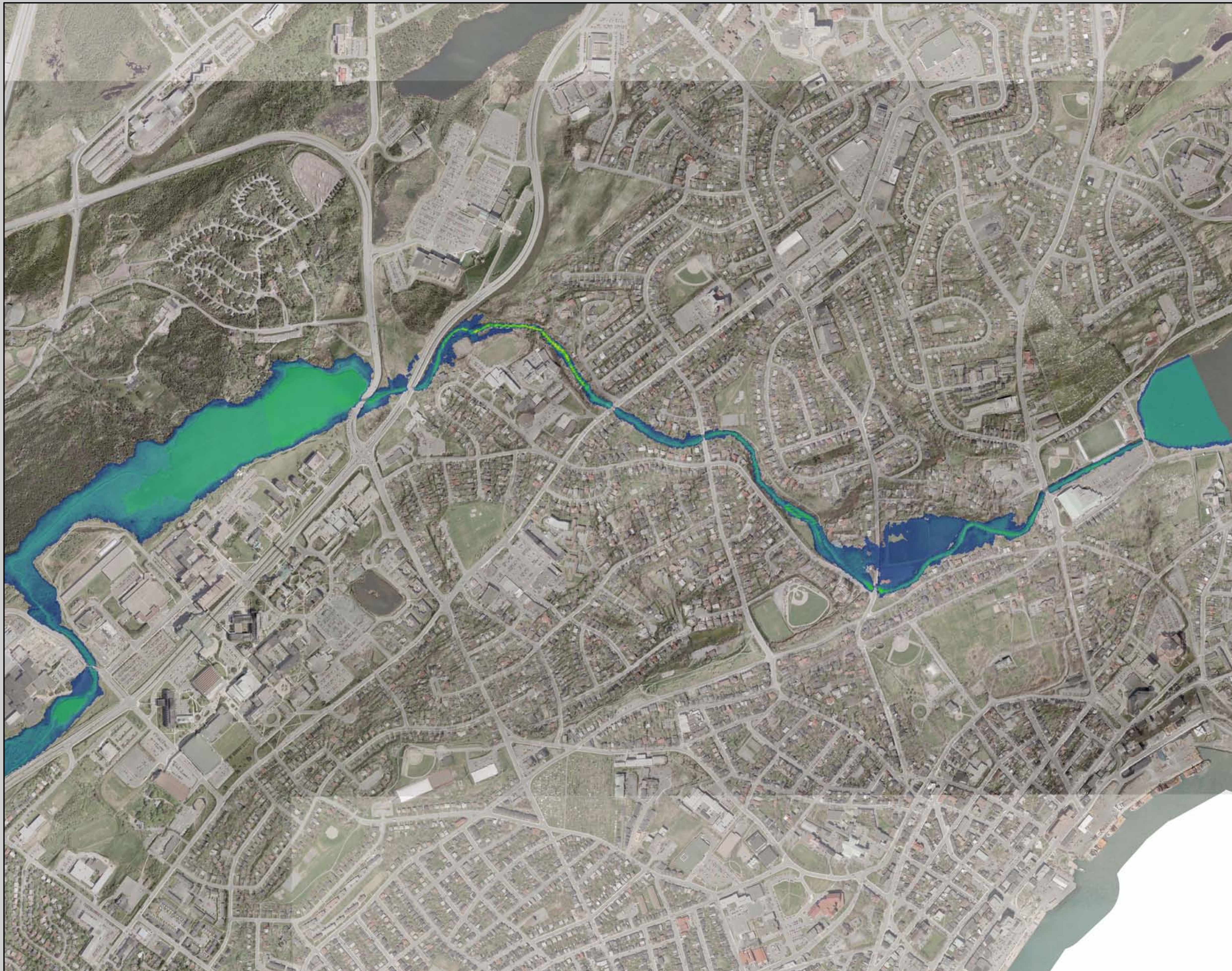
APPENDIX J

Floodplain Mapping



ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Existing Conditions
1:20 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)

High : 9

Low : 0

Map 1

CBCL LIMITED
Consulting Engineers

Coordinate System:
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Units:Metre
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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Existing Conditions
1:20 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)
High : 9
Low : 0

Map 2

CBCL
CBCL LIMITED
Consulting Engineers

Coordinate System:
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Units:Metre
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Existing Conditions
1:20 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)

High : 9

Low : 0

Map 3

CBCL
CBCL LIMITED
Consulting Engineers

Coordinate System:
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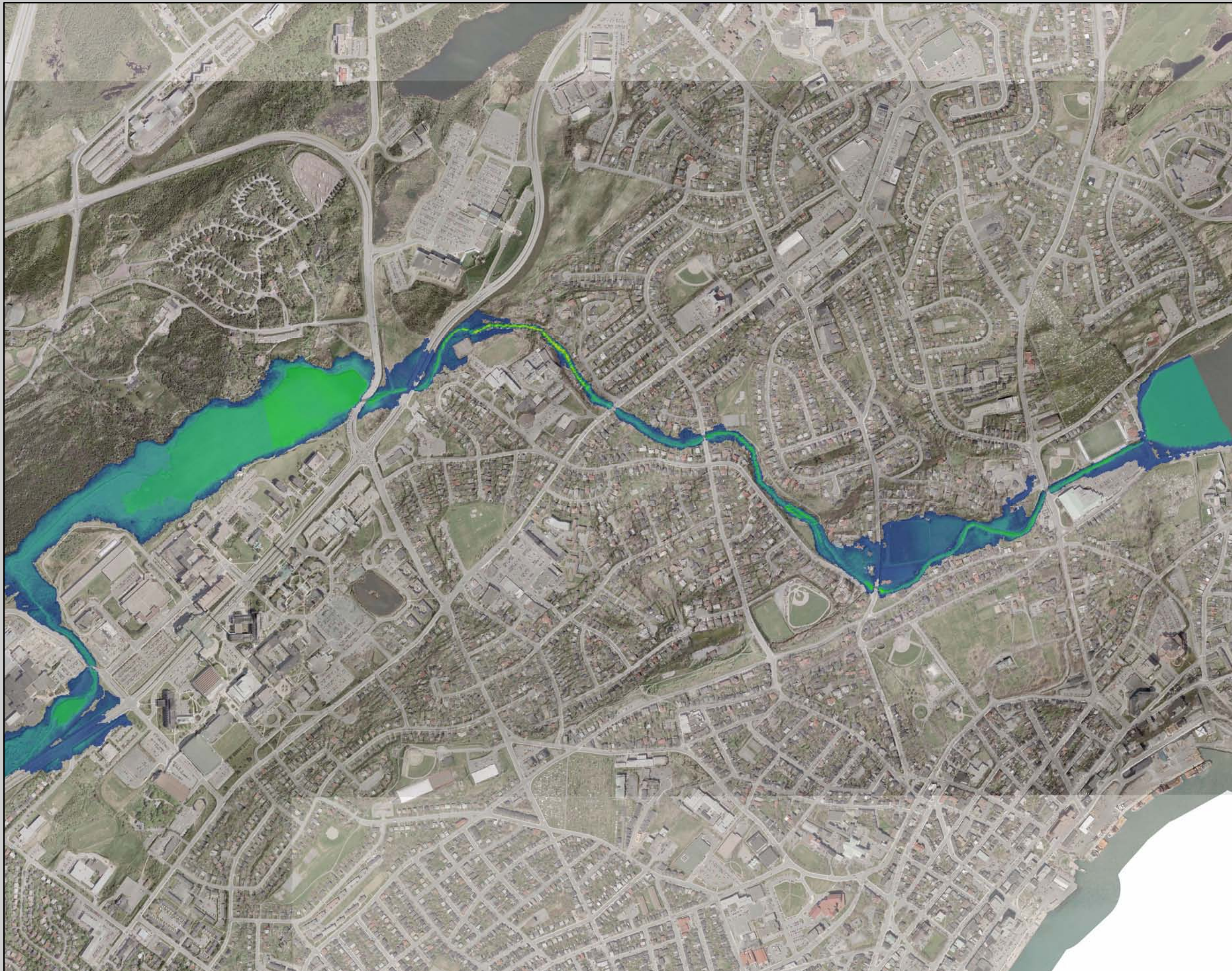

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Existing Conditions
1:100 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)

High : 9

Low : 0

Map 1

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Existing Conditions
1:100 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)

High : 9

Low : 0

Map 2

CBCL LIMITED
Consulting Engineers

Coordinate System:
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Units:Metre
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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Existing Conditions
1:100 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)

High : 9

Low : 0

Map 3

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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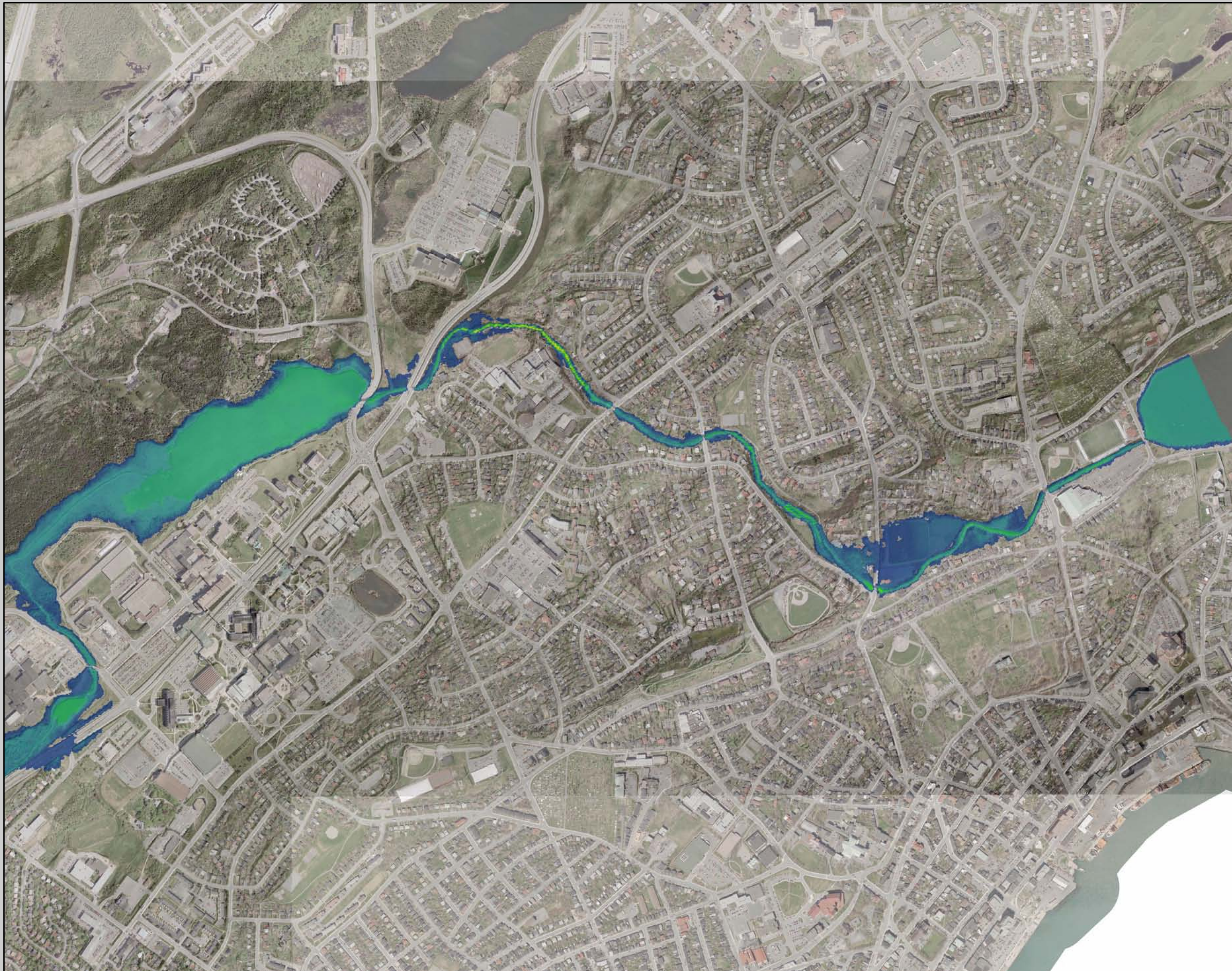

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:20 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)

High : 9

Low : 0

Map 1

CBCL LIMITED
Consulting Engineers

Coordinate System: NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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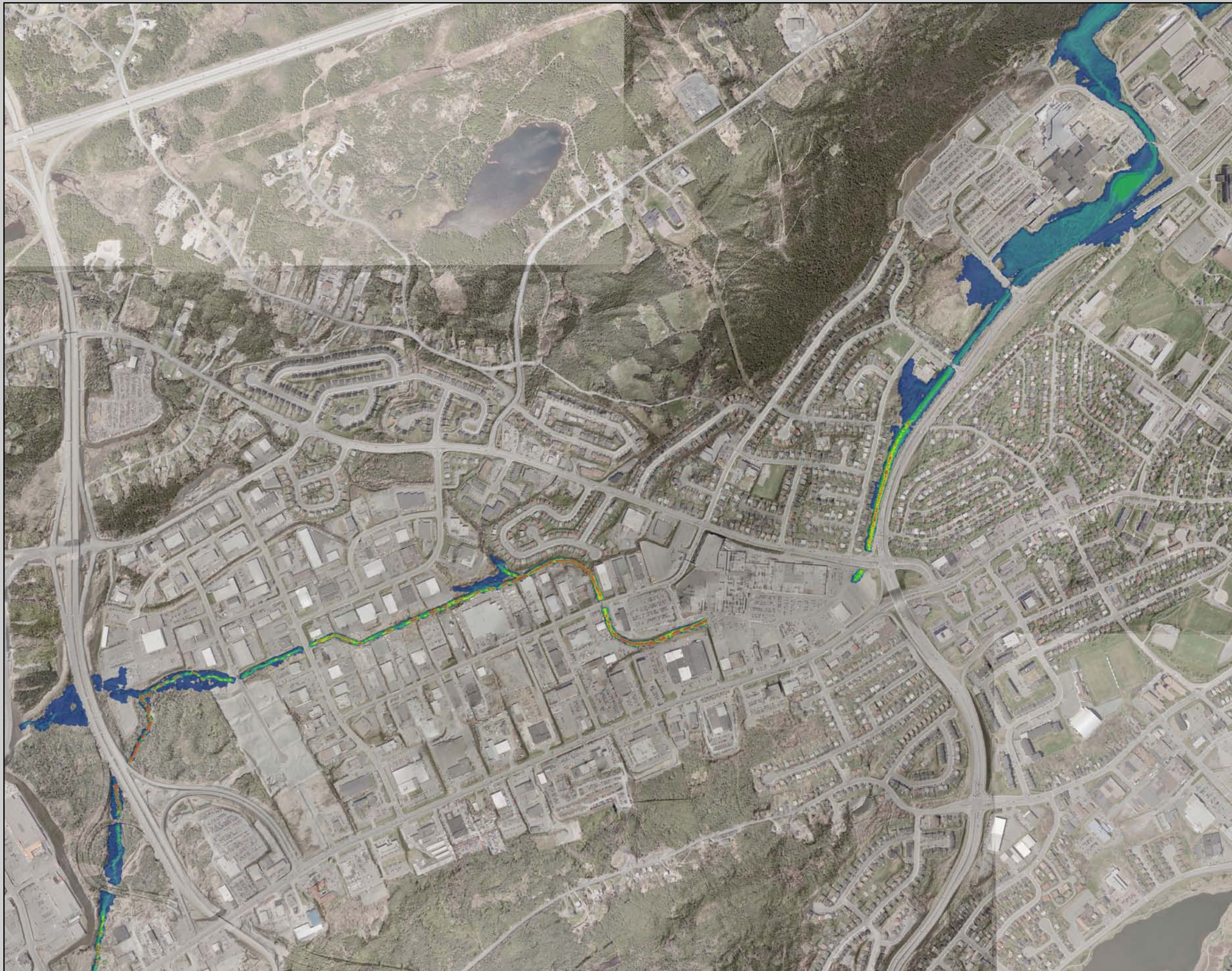

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:20 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)
High : 9
Low : 0

Map 2

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:20 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)

High : 9

Low : 0

Map 3

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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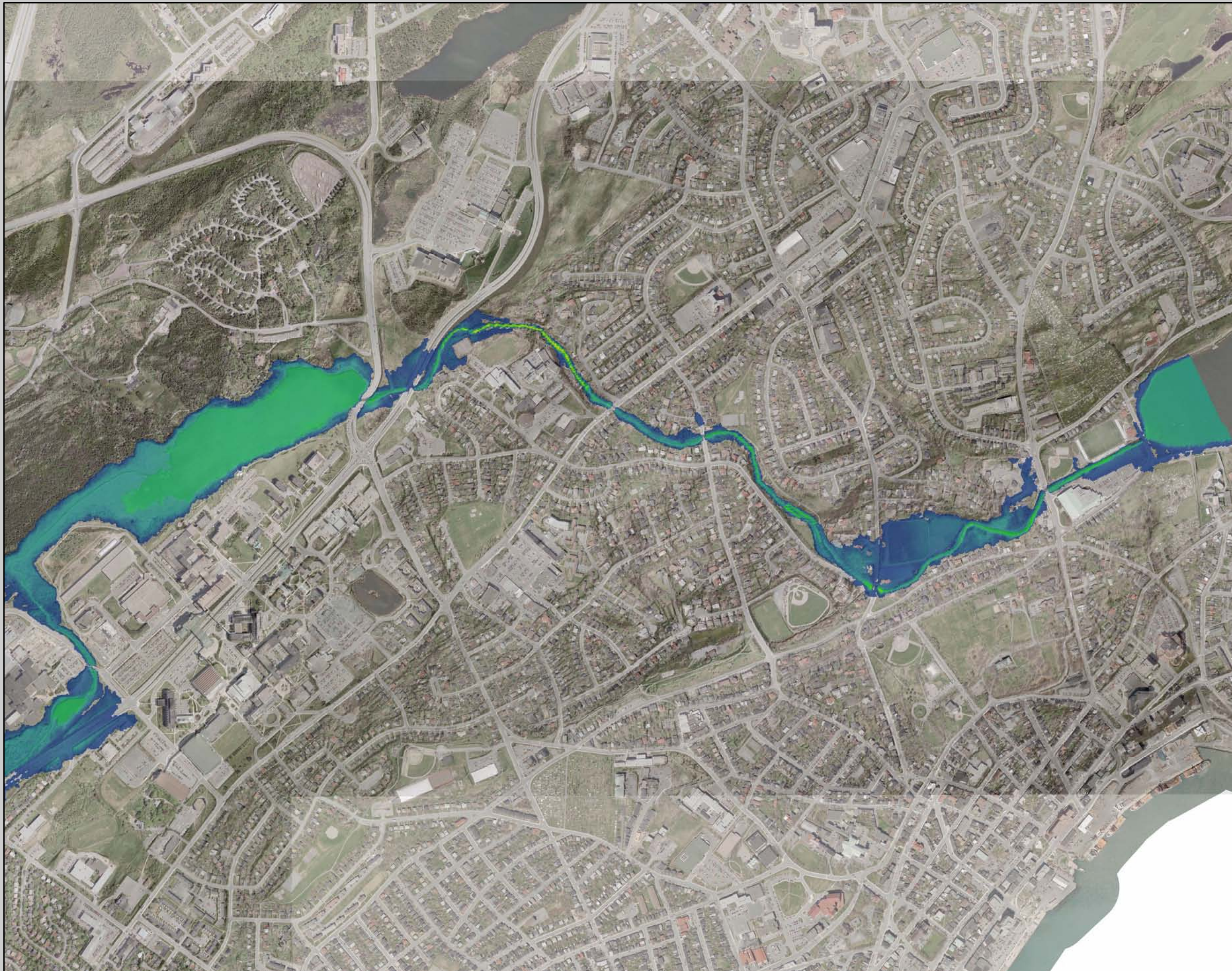

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:100 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)

High : 9

Low : 0

Map 1

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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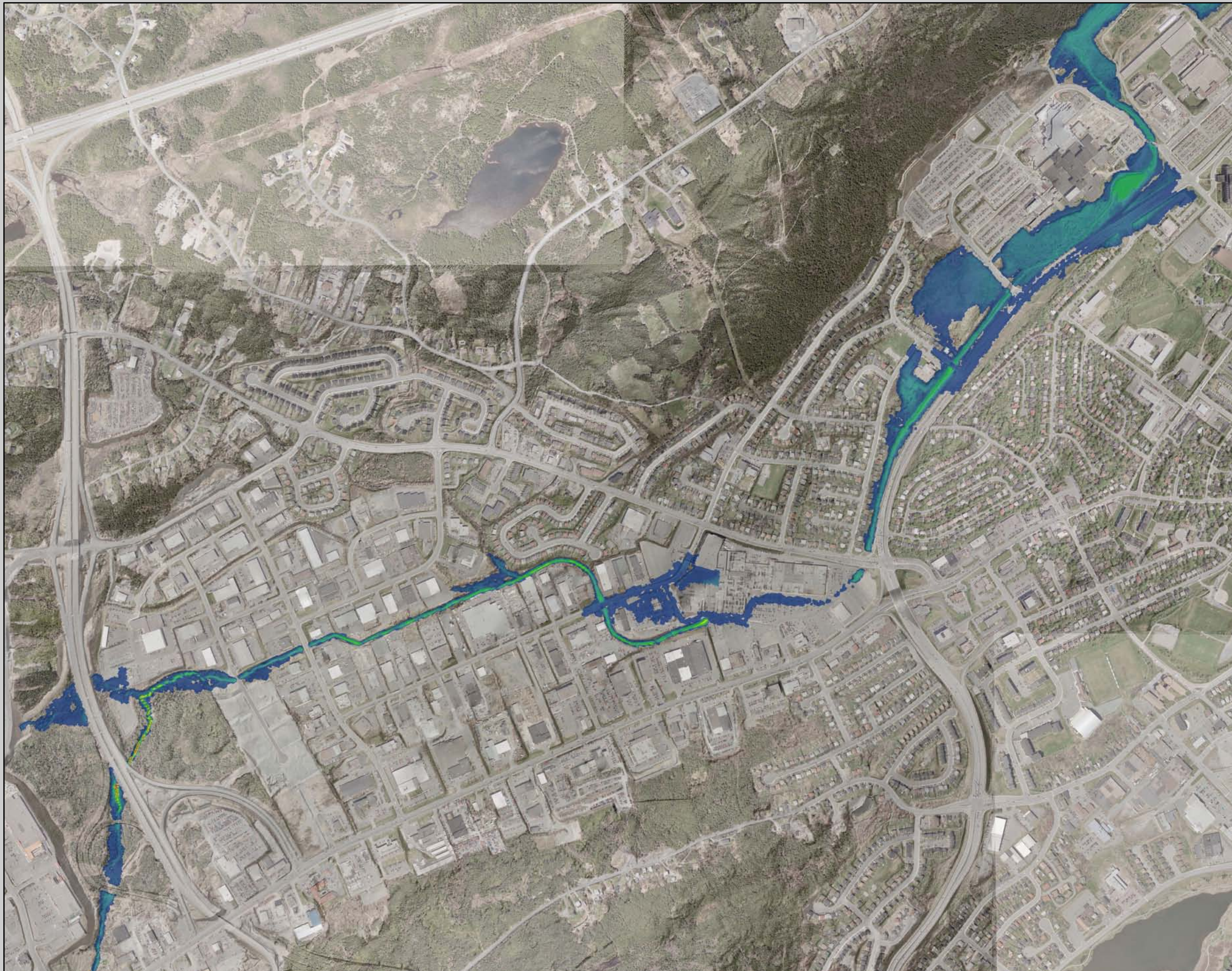

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:100 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)
High : 9
Low : 0

Map 2

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:100 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)
High : 9
Low : 0

Map 3

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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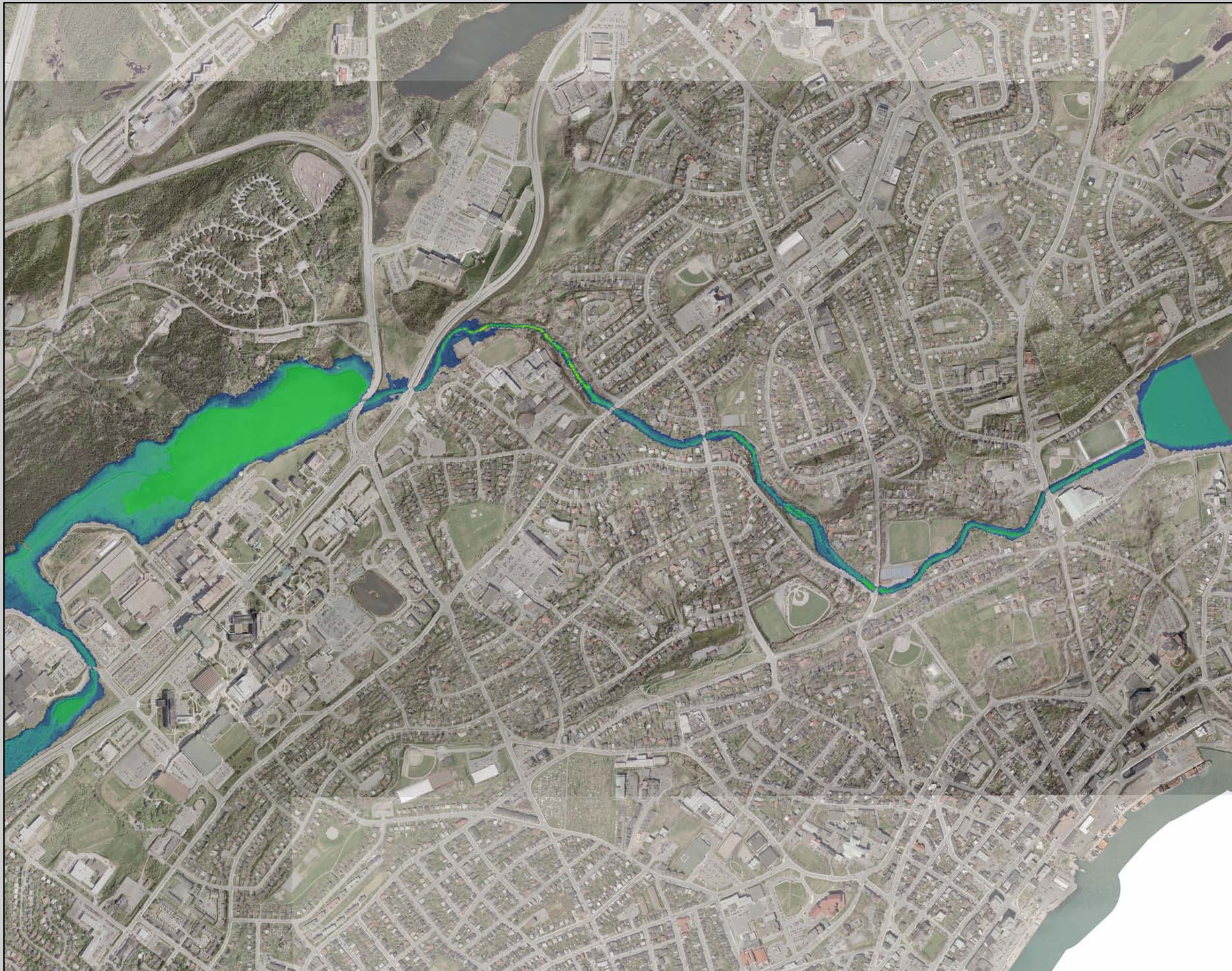
APPENDIX K

Floodplain Mapping with Improvements



ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Flood Control Measures
1:100 AEP Floodplain

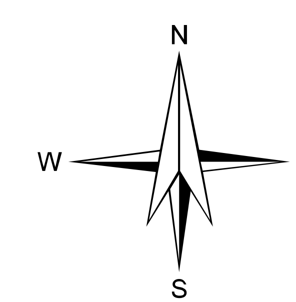
Rennies River Catchment
Stormwater Management Study

Water Depth (m)
High : 9
Low : 0

Map 1

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000



ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Flood Control Measures
1:100 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)

High : 9

Low : 0

Map 2

CBCL
CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Flood Control Measures
1:100 AEP Floodplain

Rennies River Catchment
Stormwater Management Study

Water Depth (m)
High : 9
Low : 0

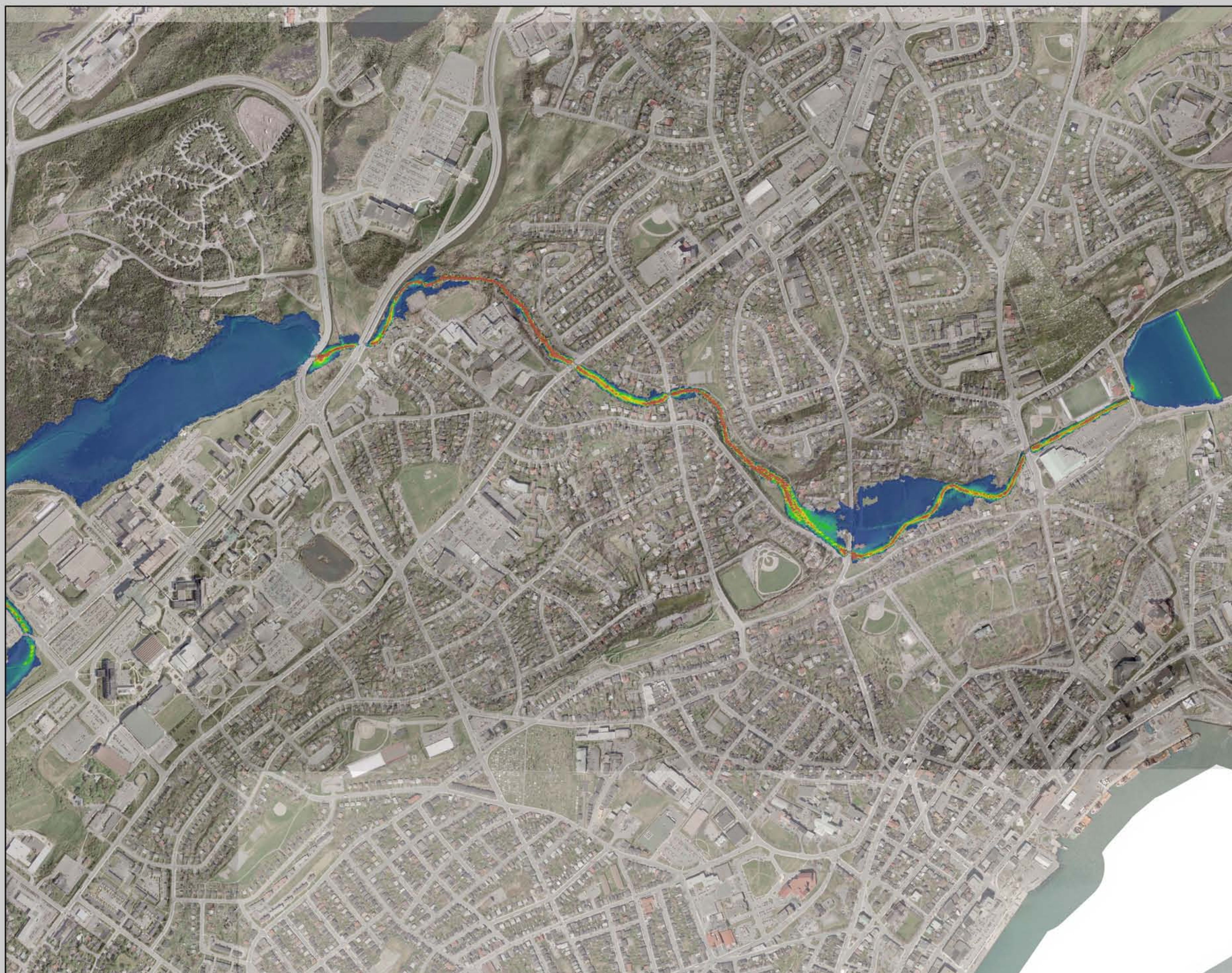
Map 3

CBCL
CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

APPENDIX L

Flood Hazard Mapping



ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

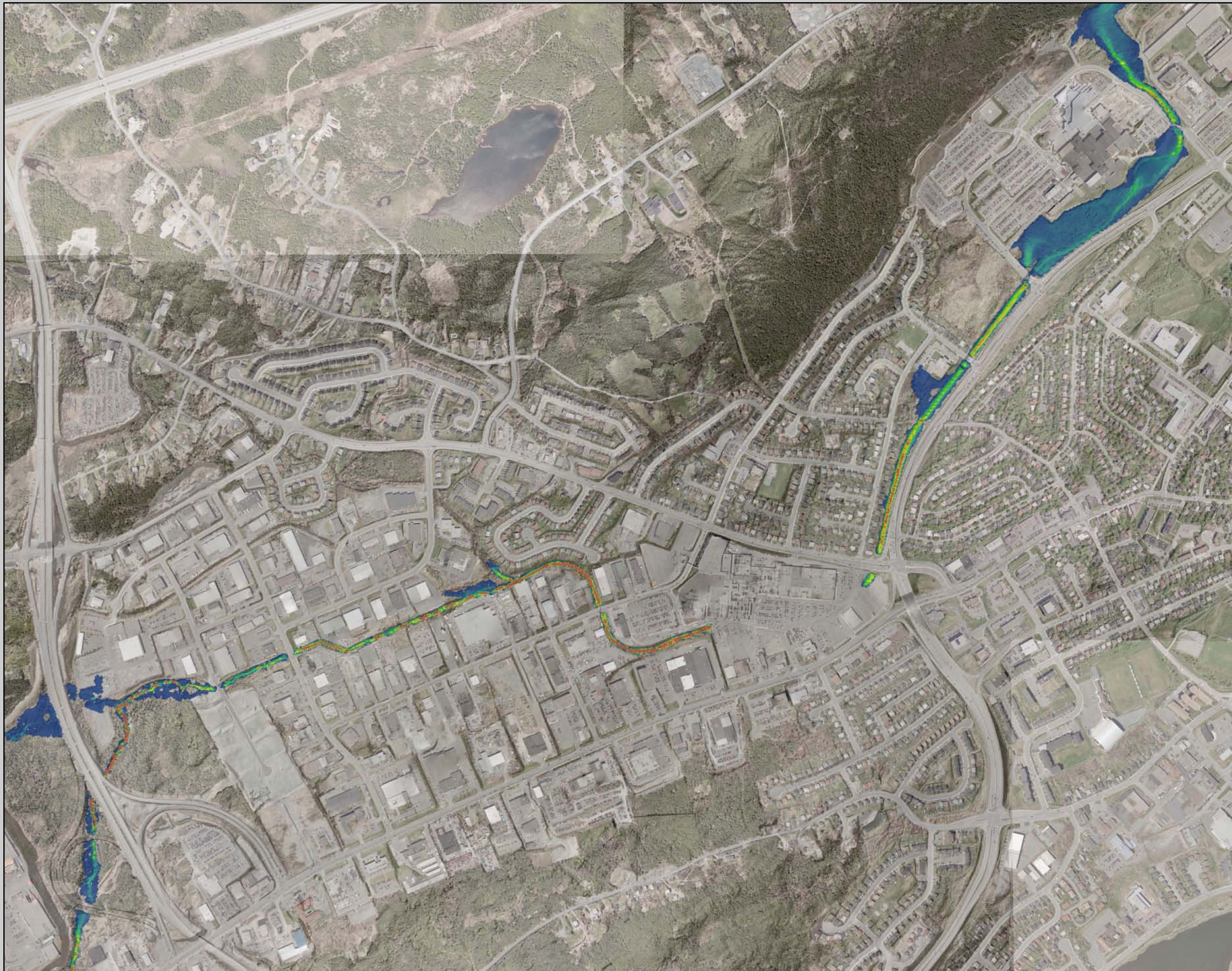
Existing Conditions
1:20 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Map 1

CBCL
CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000



ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Existing Conditions
1:20 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Hazard (VxD)
High : 6
Low : 0

Map 2

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Existing Conditions
1:20 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Hazard (VxD)
High : 6
Low : 0

Map 3

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
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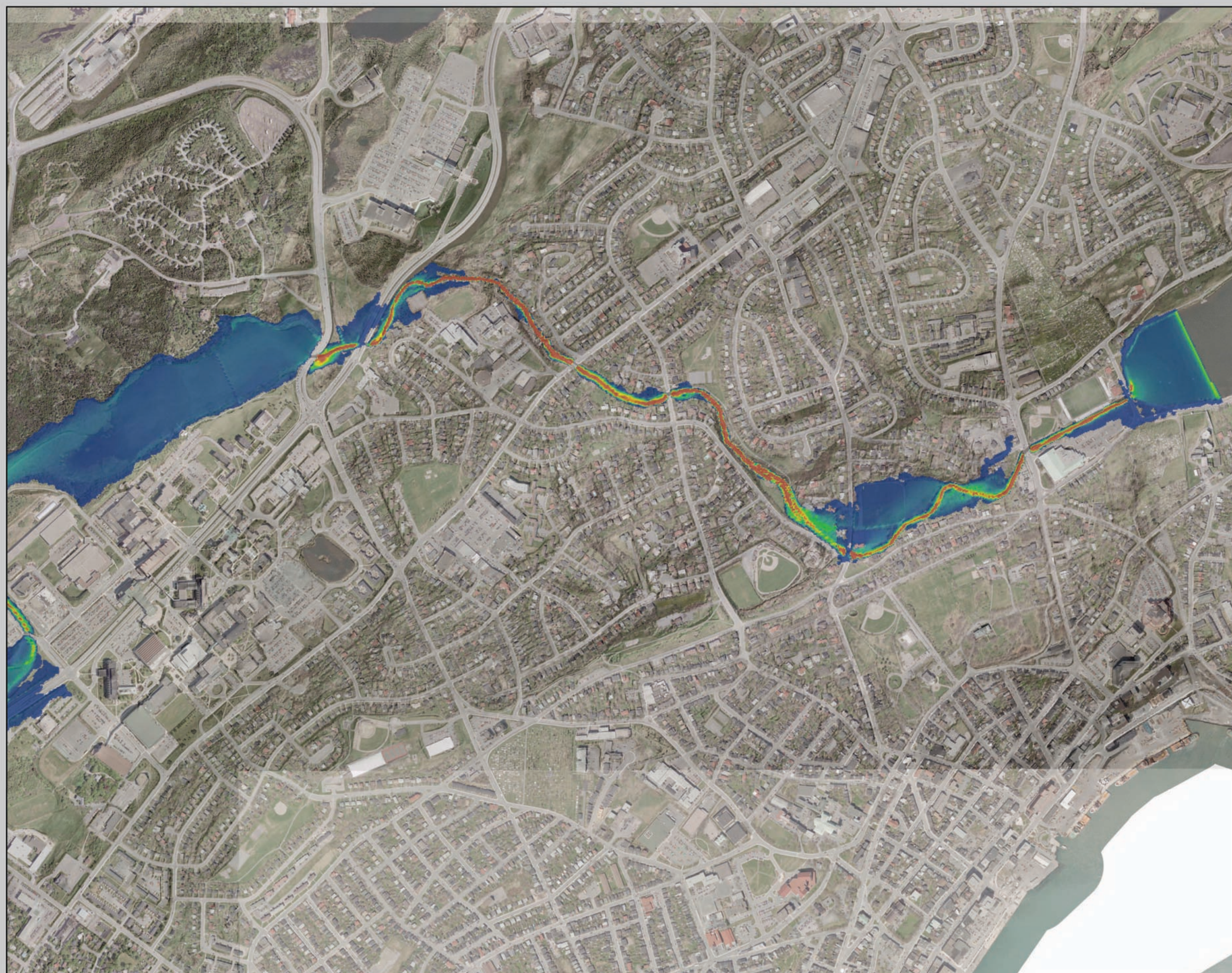

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Existing Conditions
1:100 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Hazard (VxD)

High : 6

Low : 0

Map 1

CBCL LIMITED
Consulting Engineers

Coordinate System:
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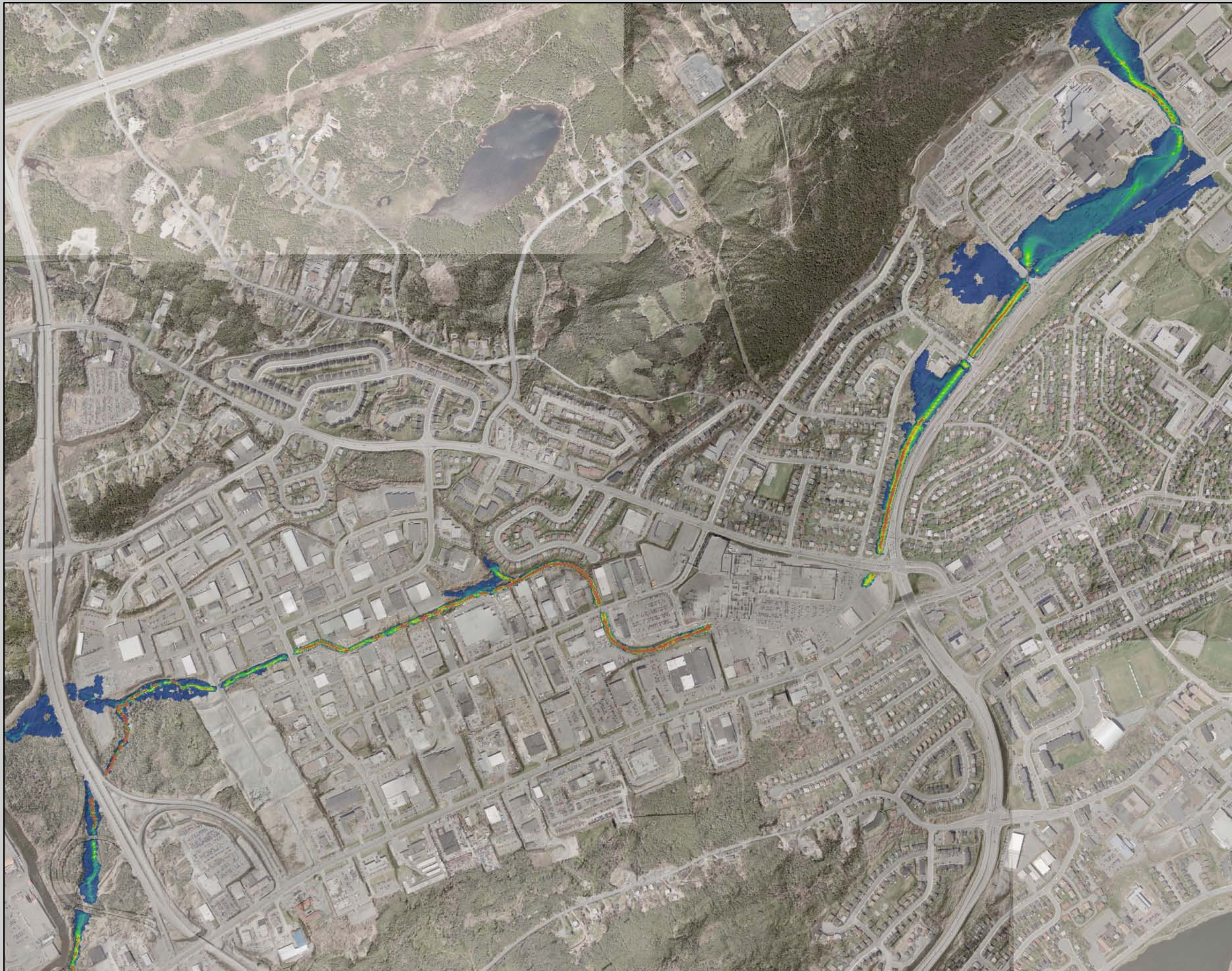

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Existing Conditions
1:100 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Hazard (VxD)
High : 6
Low : 0

Map 2

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Existing Conditions
1:100 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Hazard (VxD)
High : 6
Low : 0

Map 3

CBCL LIMITED
Consulting Engineers

Coordinate System:
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Units:Metre
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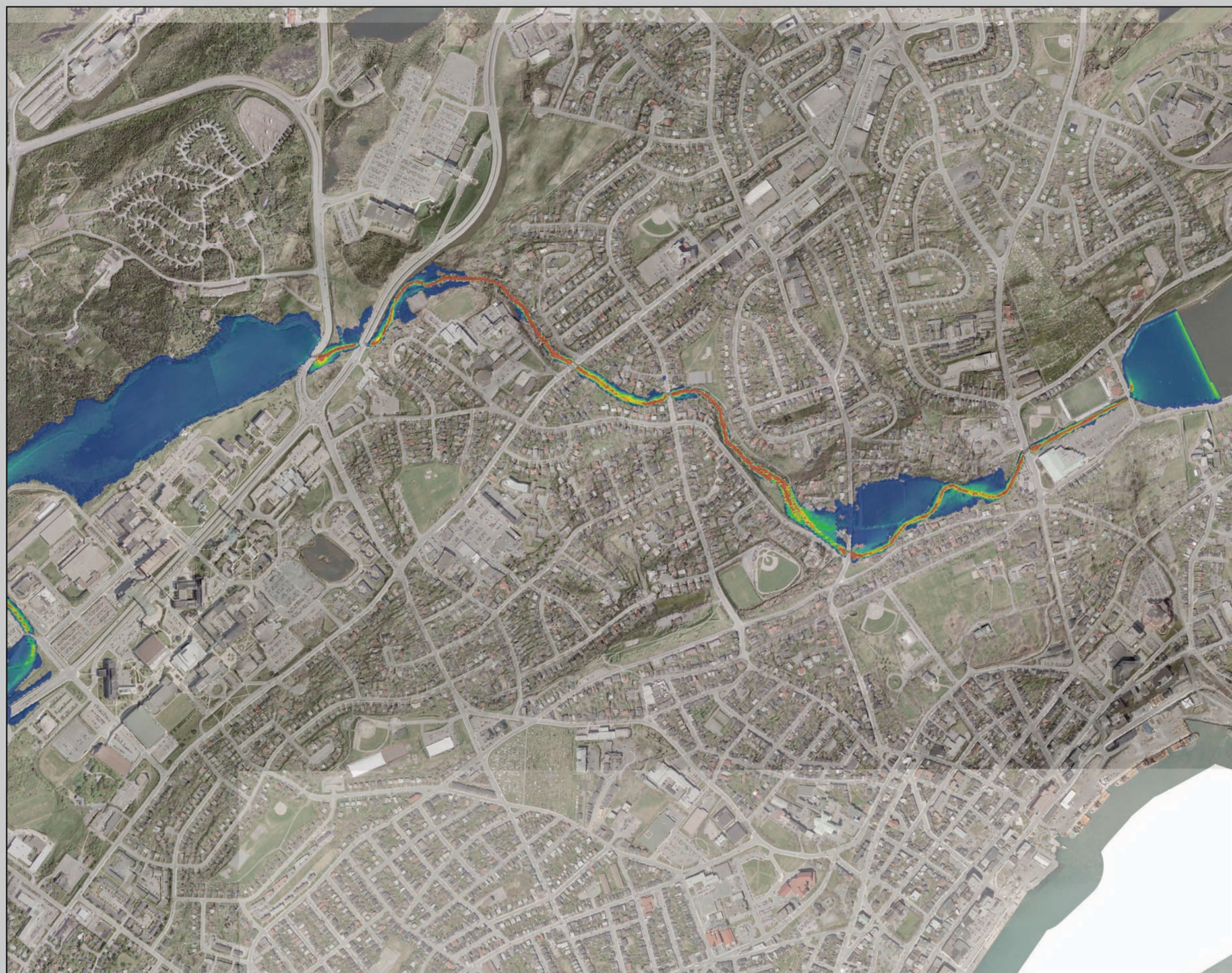

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:20 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Hazard (VxD)

High : 6

Low : 0

Map 1

CBCL LIMITED
Consulting Engineers

Coordinate System:
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Units:Metre
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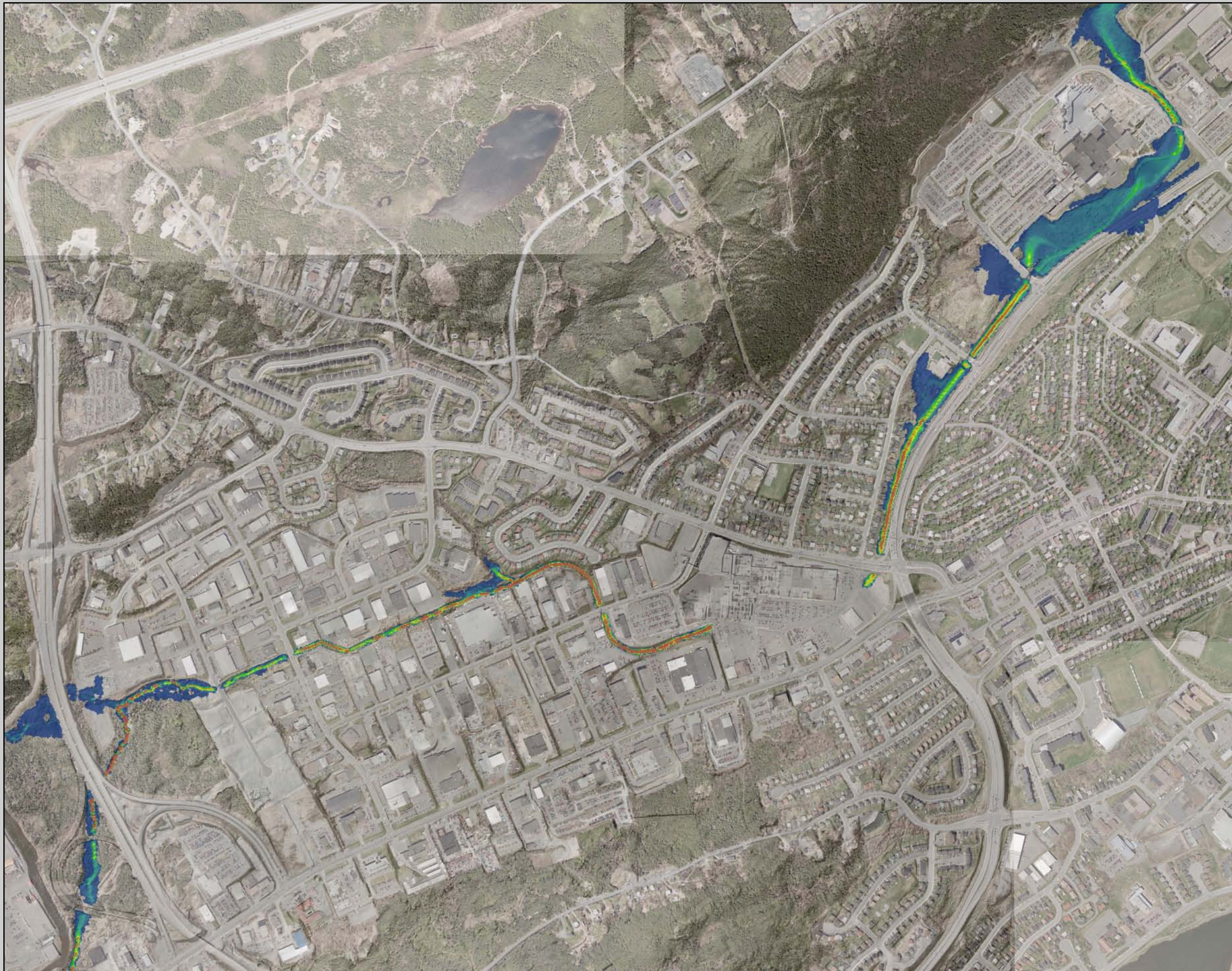

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:20 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Hazard (VxD)

High : 6

Low : 0

Map 2

CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:20 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Hazard (VxD)
High : 6
Low : 0

Map 3

CBCL
CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
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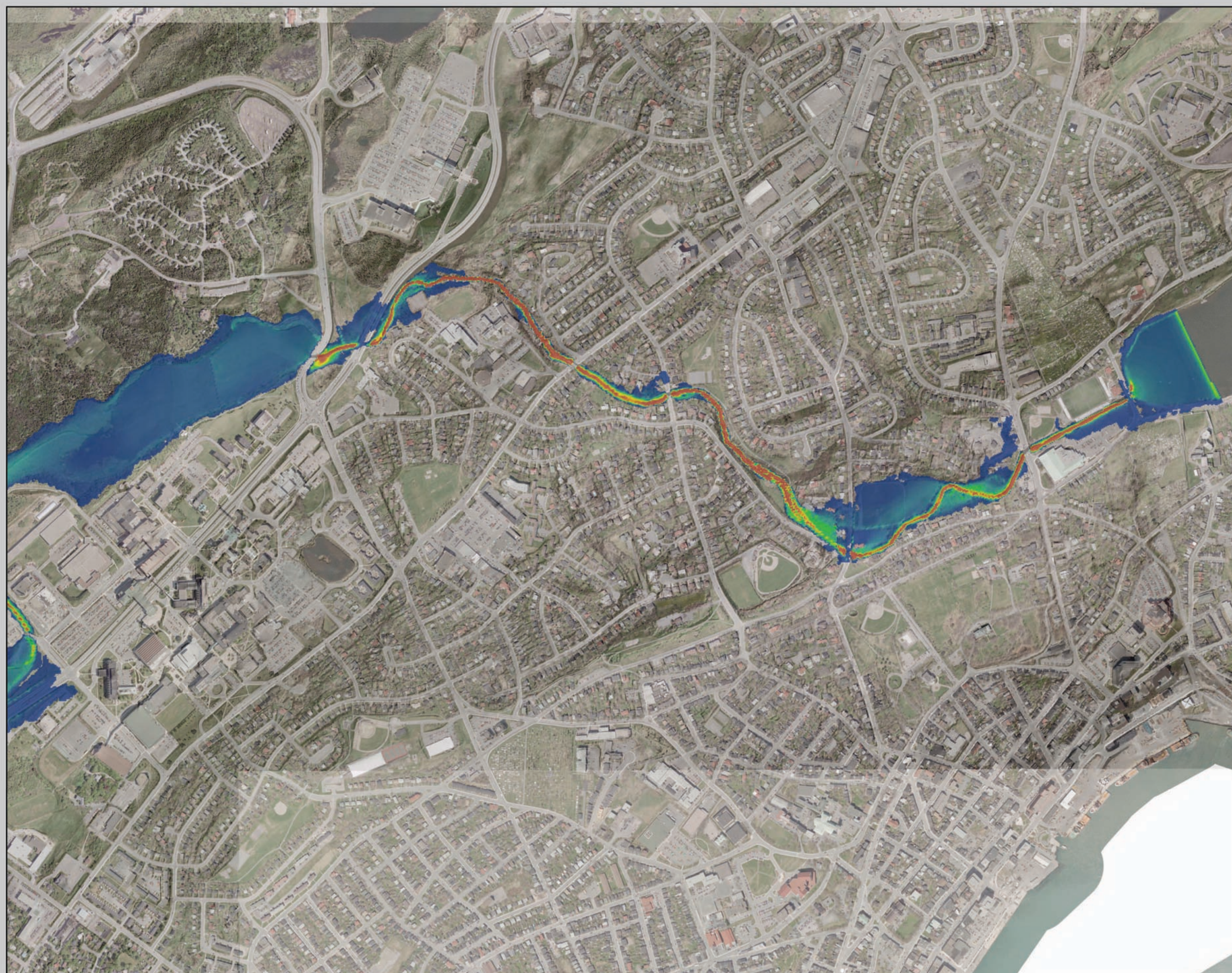
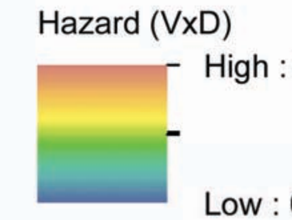


ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:100 AEP Hazard

Rennies River Catchment Stormwater Management Study



Map 1

CBCL LIMITED
Consulting Engineers

Coordinate System:
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Units:Metre
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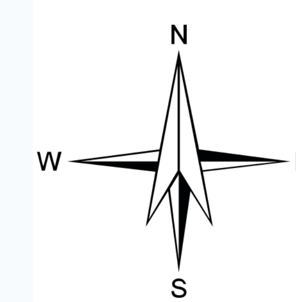

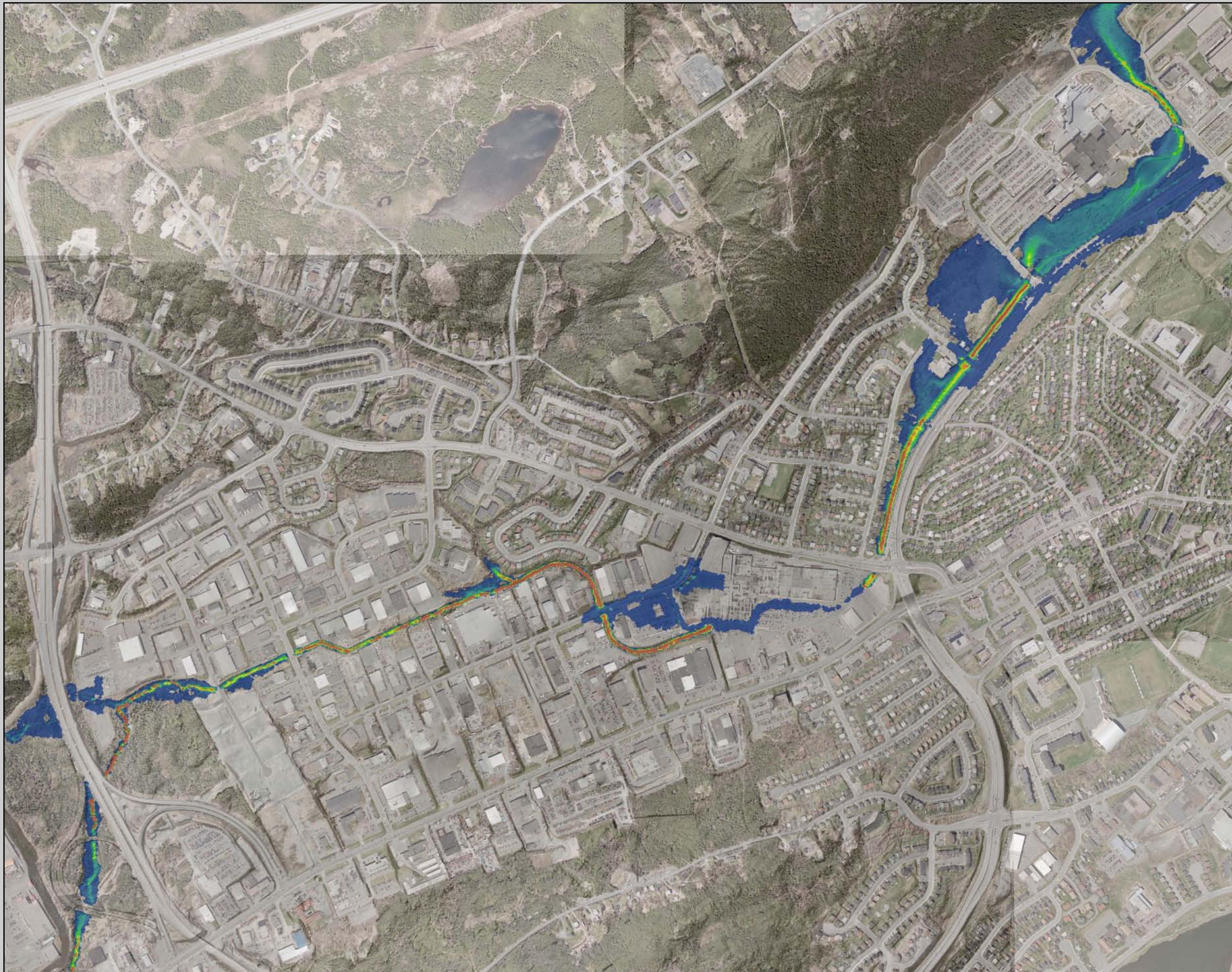

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:100 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Hazard (VxD)
High : 6
Low : 0

Map 2

CBCL LIMITED
Consulting Engineers

Coordinate System:
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Units:Metre
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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Future Conditions
1:100 AEP Hazard

Rennies River Catchment
Stormwater Management Study

Hazard (VxD)

High : 6

Low : 0

Map 3

CBCL LIMITED
Consulting Engineers

Coordinate System:
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Units:Metre
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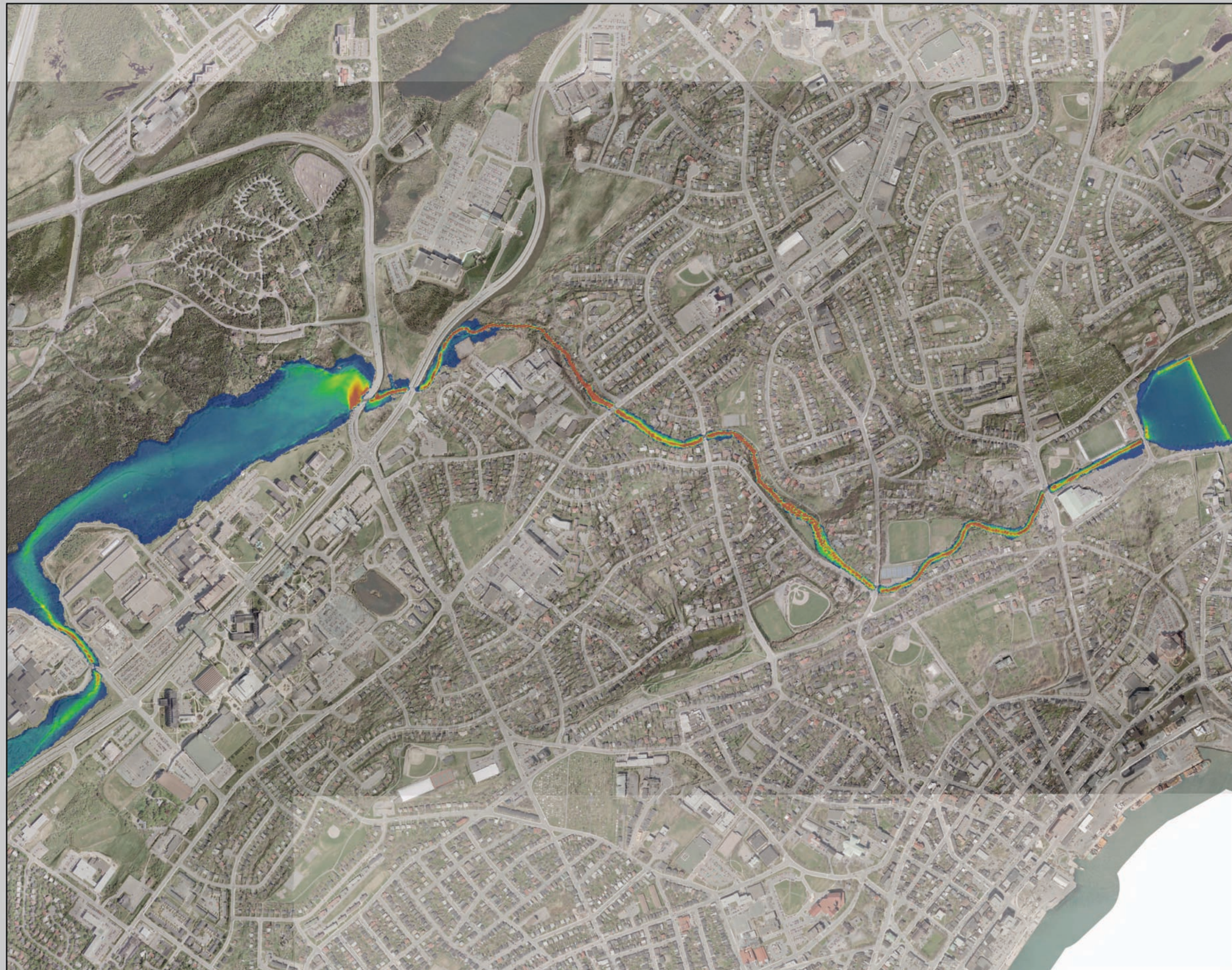

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Flood Control Measures Hazard

Rennies River Catchment Stormwater Management Study

Hazard (VxD)
High : 6
Low : 0

Map 1

CBCL LIMITED
Consulting Engineers

Coordinate System: NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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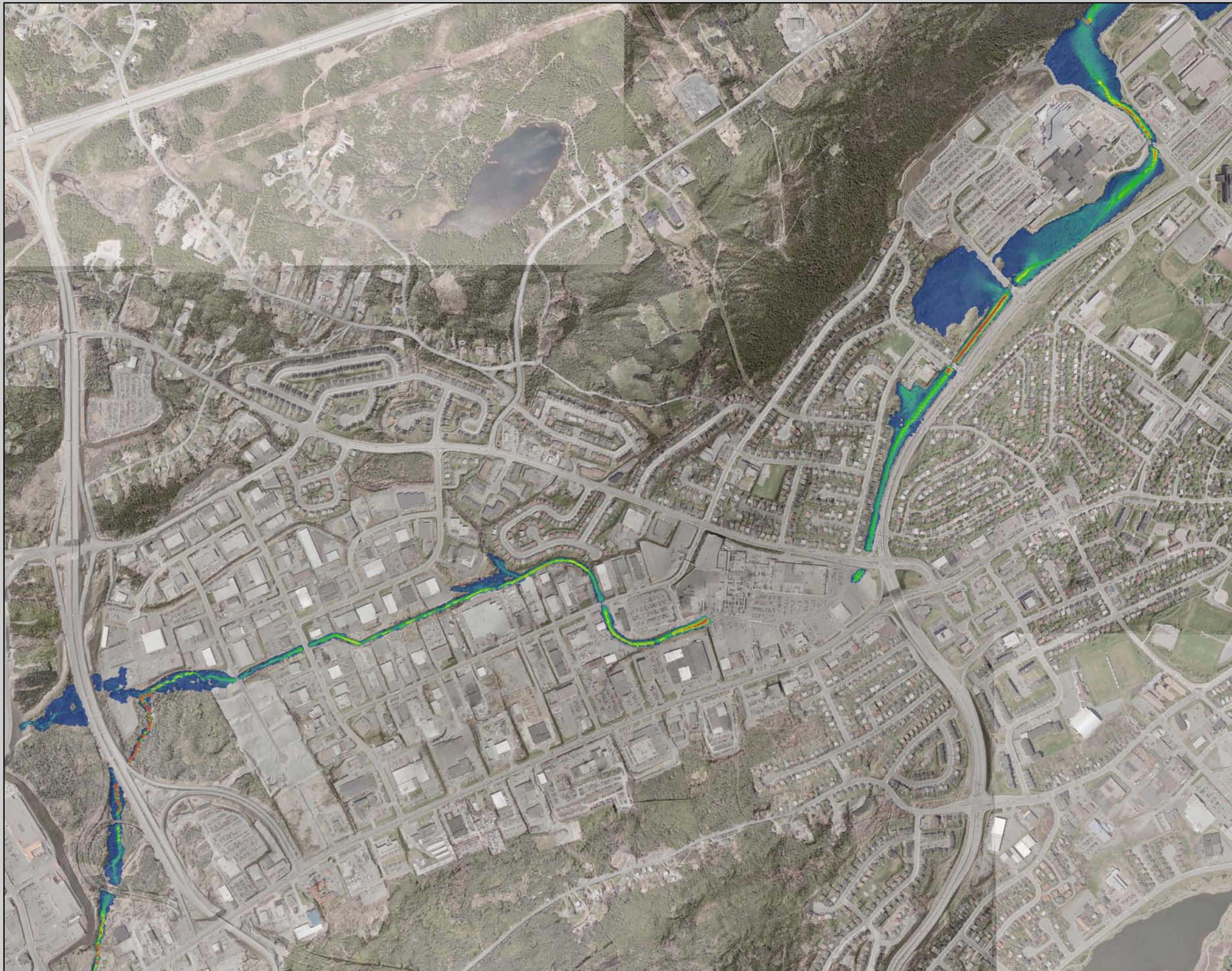

ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Flood Control Measures Hazard

Rennies River Catchment Stormwater Management Study

Hazard (VxD)
High : 6
Low : 0

Map 2

CBCL LIMITED
Consulting Engineers

Coordinate System: NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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ST. JOHN'S
NEWFOUNDLAND AND LABRADOR, CANADA

Flood Control Measures Hazard

Rennies River Catchment Stormwater Management Study

Hazard (VxD)
High : 6
Low : 0

Map 3

CBCL
CBCL LIMITED
Consulting Engineers

Coordinate System:
NAD_1983_MTM_1
Units:Metre
Scale:1:10,000

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APPENDIX M

Opinions of Probable Costs

Preliminary Cost Estimate

Location 1 - Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road Bridge
Option A
- **700m earth berm**
- **340m segmental concrete block wall**
- **130m cast-in-place concrete wall**

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
153	MOBILIZATION/DEMOBILIZATION	LS	1	$35,000.00	$35,000
311	CLEARING AND GRUBBING	HA	0.50	$20,000.00	$10,000
321	STREET EXCAVATION				
	Concrete removal (curb and sidewalk)	SM	150	$10.00	$1,500
	Asphalt removal	SM	100	$5.00	$500
322	BORROW				
	Gravel Borrow	CM	3300	$8.00	$26,400
323	GRAVEL FOR STREETS				
	Granular "A"	TONNE	250	$22.00	$5,500
	Quarter minus	TONNE	50	$20.00	$1,000
330	CONCRETE CURB, GUTTER AND/OR SIDEWALK				
	Curb and gutter	LM	100	$90.00	$9,000
	Sidewalk	LM	100	$140.00	$14,000
352	FULL DEPTH ASPHALT PATCH				
	Surface and base courses (80mm)	SM	100	$30.00	$3,000
402	EXCAVATION FOR FOUNDATIONS				
	OM	CM	1100	$20.00	$22,000
404	CONCRETE STRUCTURES				
	Cast-in-place Walls	CM	120	$1,200.00	$144,000
511	TOPSOILING, SODDING AND/OR HYDROSEEDING				
	100mm Topsoil and Hydroseeding	SM	3100	$7.50	$23,250

Preliminary Cost Estimate

Location 1 - Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road Bridge

Option A

- 700m earth berm

- 340m segmental concrete block wall

- 130m cast-in-place concrete wall

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
OTHER	**SEGMENTAL CONCRETE BLOCK WALLS**				
	Concrete Block	**SM**	**900**	**$450.00**	**$405,000**
	PVC Liner	**SM**	**1900**	**$14.00**	**$26,600**
	Handrail	**LM**	**500**	**$50.00**	**$25,000**

Subtotal =	$751,750
Contingency (20%) =	$150,350
Engineering (15%) =	$135,315
Subtotal =	$1,037,415
HST (13%) =	$134,864
TOTAL =	$1,172,279
BUDGET =	$1,173,000

This opinion of probable costs is presented on the basis of experience, qualifications and best judgement. It has been prepared in accordance with acceptable principles and practices. Sudden market trend changes, non-competitive bidding situations, unforeseen labour and material adjustments and the like are beyond the control of CBCL Limited. We cannot warrant or guarantee that actual costs will not vary significantly from the opinion provided.

Preliminary Cost Estimate

Location 1 - Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road Bridge
Option B
- 300m earth berm
- 420m new channel (including earth berms for channel)
- 110m segmental concrete block wall
- Remove and replace bridge
- Remove house at 1 Portugal Cove Road

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
153	MOBILIZATION/DEMOBILIZATION	LS	1	$100,000.00	$100,000
311	CLEARING AND GRUBBING	HA	1.50	$20,000.00	$30,000
321	STREET EXCAVATION				
	OM	CM	4000	$20.00	$80,000
322	BORROW				
	Gravel Borrow	CM	5500	$8.00	$44,000
323	GRAVEL FOR STREETS				
	Granular "A"	TONNE	210	$22.00	$4,620
	Granular "B"	TONNE	200	$20.00	$4,000
	Quarter minus	TONNE	50	$20.00	$1,000
330	CONCRETE CURB, GUTTER AND SIDEWALK				
	Curb and Gutter	LM	80	$90.00	$7,200
	Sidewalk	LM	80	$140.00	$11,200
351	HOT MIX ASPHALT CONCRETE				
	Surface course	TONNE	40	$165.00	$6,600
	Base course	TONNE	44	$165.00	$7,260
402	EXCAVATION FOR FOUNDATION				
	OM	CM	2100	$20.00	$42,000
	Removal of Existing Bridge	LS	1	$75,000.00	$75,000
404	CONCRETE STRUCTURES				
	Cast-in-Place Wing Walls (new bridge)	CM	50	$1,200.00	$60,000
	Cast-in-Place Footings (new bridge)	CM	35	$1,200.00	$42,000
	Pre-cast Box Culvert Sections	LM	16	$18,500.00	$296,000
415	ALUMINUM BRIDGE RAILING				
	Bridge Railing	LS	1	$5,000.00	$5,000
511	TOPSOILING, SODDING AND/OR HYDROSEEDING				
	100mm Topsoil and Hydroseeding	SM	5400	$7.50	$40,500
OTHER	SEGMENTAL CONCRETE BLOCK WALLS				
	Concrete Block	SM	350	$450.00	$157,500
	PVC Liner	SM	700	$14.00	$9,800
	Handrail	LM	220	$50.00	$11,000

Preliminary Cost Estimate

Location 1 - Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road Bridge
Option B

- **300m earth berm**
- **420m new channel (including earth berms for channel)**
- **110m segmental concrete block wall**
- **Remove and replace bridge**
- **Remove house at 1 Portugal Cove Road**

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
OTHER	CELLULAR CONFINEMENT SYSTEM				
	Cellular Confinement System (erosion protection for new channel)	SM	6800	$75.00	$510,000
OTHER	REMOVAL OF EXISTING HOUSE				
	Removal of Existing House (1 Portugal Cove Road)	LS	1	$50,000.00	$50,000
	Property Acquisition (1 Portugal Cove Road)	LS	1	$900,000.00	$900,000

Subtotal =	$2,494,680
Contingency (20%) =	$498,936
Engineering (15%) =	$449,042
Subtotal =	$3,442,658
HST (13%) =	$447,546
TOTAL =	$3,890,204
BUDGET =	$3,891,000

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Preliminary Cost Estimate

Location 1 - Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road Bridge

Option C

- 460m earth berm
- 230m segmental concrete block wall
- 130m cast-in-place concrete wall
- Remove and replace Riverdale Tennis Club building
- Raise parking lot and building pad

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
153	MOBILIZATION/DEMOBILIZATION	LS	1	$50,000.00	$50,000
311	CLEARING AND GRUBBING	HA	0.50	$20,000.00	$10,000
321	STREET EXCAVATION				
	Concrete removal (curb and sidewalk)	SM	150	$10.00	$1,500
	Asphalt removal	SM	150	$5.00	$750
322	BORROW				
	Gravel Borrow	CM	3700	$8.00	$29,600
323	GRAVEL FOR STREETS				
	Granular "A"	TONNE	250	$22.00	$5,500
	Quarter minus	TONNE	50	$20.00	$1,000
330	CONCRETE CURB, GUTTER AND/OR SIDEWALK				
	Curb and gutter	LM	100	$90.00	$9,000
	Sidewalk	LM	100	$140.00	$14,000
352	FULL DEPTH ASPHALT PATCH				
	Surface and base courses (80mm)	SM	100	$30.00	$3,000
	Surface course (50mm)	SM	50	$30.00	$1,500
402	EXCAVATION FOR FOUNDATIONS				
	OM	CM	850	$20.00	$17,000
404	CONCRETE STRUCTURES				
	Cast-in-place Walls	CM	120	$1,200.00	$144,000
511	TOPSOILING, SODDING AND/OR HYDROSEEDING				
	100mm Topsoil and Hydroseeding	SM	1500	$7.50	$11,250
OTHER	SEGMENTAL CONCRETE BLOCK WALLS				
	Concrete Block	SM	200	$450.00	$90,000
	PVC Liner	SM	750	$14.00	$10,500
	Guiderail	LM	460	$120.00	$55,200

Preliminary Cost Estimate

Location 1 - Kings Bridge Road to Portugal Cove Road & Upstream of Portugal Cove Road Bridge
Option C

- **460m earth berm**
- **230m segmental concrete block wall**
- **130m cast-in-place concrete wall**
- **Remove and replace Riverdale Tennis Club building**
- **Raise parking lot and building pad**

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
OTHER	DEMOLITION				
	Building removal	LS	1	$30,000.00	$30,000
OTHER	BUILDINGS				
	New building	SM	1	$400,000.00	$400,000

Subtotal =	$883,800
Contingency (20%) =	$176,760
Engineering (15%) =	$159,084
Subtotal =	$1,219,644
HST (13%) =	$158,554
TOTAL =	$1,378,198
BUDGET =	$1,379,000

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Preliminary Cost Estimate

Location 2 - Upstream of Carpasian Road Bridge
- 150m earth berm at left bank

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
153	MOBILIZATION/DEMOBILIZATION	LS	1	$2,000.00	$2,000
311	CLEARING AND GRUBBING	HA	0.50	$20,000.00	$10,000
322	BORROW				
	Gravel Borrow	CM	250	$8.00	$2,000
323	GRAVEL FOR STREETS				
	Quarter minus	TONNE	35	$20.00	$700
511	TOPSOILING, SODDING AND/OR HYDROSEEDING				
	100mm Topsoil and Hydroseeding	SM	300	$7.50	$2,250

Subtotal =	$16,950
Contingency (20%) =	$3,390
Engineering (15%) =	$3,051
Subtotal =	$23,391
HST (13%) =	$3,041
TOTAL =	$26,432
BUDGET =	$27,000

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Preliminary Cost Estimate

Location 3 - Outlet of Long Pond

- 25m concrete weir and fish passage at outlet

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
153	MOBILIZATION/DEMOBILIZATION	LS	1	$50,000.00	$50,000
402	EXCAVATION FOR FOUNDATION				
	OM	CM	200	$50.00	$10,000
	Unwatering	LS	1	$75,000.00	$75,000
404	CONCRETE STRUCTURES				
	Cast-in-Place Weir and Fish passage	CM	600	$1,800.00	$1,080,000
	Hardware including walkway, handrail, gates, etc.	LS	1	$50,000.00	$50,000
417	HYDRAULIC RIP-RAP				
	Class II	CM	50	$75.00	$3,750

Subtotal =	$1,268,750
Contingency (20%) =	$253,750
Engineering (15%) =	$228,375
Subtotal =	$1,750,875
HST (13%) =	$227,614
TOTAL =	$1,978,489
BUDGET =	$1,979,000

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Preliminary Cost Estimate

Location 4 - Clinch Crescent East to Clinch Crescent West
- 360m earth berm at right bank
- 120m cast-in-place concrete wall at right bank

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
153	MOBILIZATION/DEMOBILIZATION	LS	1	$10,000.00	$10,000
311	CLEARING AND GRUBBING	HA	0.50	$10,000.00	$5,000
321	STREET EXCAVATION				
	Concrete removal (sidewalk)	SM	180	$10.00	$1,800
322	BORROW				
	Gravel Borrow	CM	1400	$8.00	$11,200
330	CONCRETE CURB, GUTTER AND/OR SIDEWALK				
	Sidewalk	LM	120	$140.00	$16,800
402	EXCAVATION FOR FOUNDATION				
	OM	CM	600	$20.00	$12,000
404	CONCRETE STRUCTURES				
	Cast-in-Place Walls	CM	120	$1,200.00	$144,000
511	TOPSOILING, SODDING AND/OR HYDROSEEDING				
	100mm Topsoil and Hydroseeding	SM	2400	$7.50	$18,000

Subtotal =	$218,800
Contingency (20%) =	$43,760
Engineering (15%) =	$39,384
Subtotal =	$301,944
HST (13%) =	$39,253
TOTAL =	$341,197
BUDGET =	$342,000

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Preliminary Cost Estimate

Location 5 - Wicklow Street to Thorburn Road Bridge
- 580m earth berm
- 120m cast-in-place concrete wall

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
153	MOBILIZATION/DEMOBILIZATION	LS	1	$15,000.00	$15,000
311	CLEARING AND GRUBBING	HA	1.00	$10,000.00	$10,000
321	STREET EXCAVATION				
	Concrete removal (sidewalk)	SM	60	$10.00	$600
322	BORROW				
	Gravel Borrow	CM	2100	$8.00	$16,800
330	CONCRETE CURB, GUTTER AND/OR SIDEWALK				
	Sidewalk	SM	40	$140.00	$5,600
402	EXCAVATION FOR FOUNDATION				
	OM	CM	200	$20.00	$4,000
404	CONCRETE STRUCTURES				
	Cast-in-Place Walls	CM	90	$1,200.00	$108,000
511	TOPSOILING, SODDING AND/OR HYDROSEEDING				
	100mm Topsoil and Hydroseeding	SM	3800	$7.50	$28,500

Subtotal =	$188,500
Contingency (20%) =	$37,700
Engineering (15%) =	$33,930
Subtotal =	$260,130
HST (13%) =	$33,817
TOTAL =	$293,947
BUDGET =	$294,000

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Preliminary Cost Estimate

Location 7 - O'Leary Avenue Bridge
- Remove and replace bridge
- 70m earth berm at left bank

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
153	MOBILIZATION/DEMOBILIZATION	LS	1	$50,000.00	$50,000
322	BORROW				
	Gravel Borrow	CM	400	$8.00	$3,200
323	GRAVEL FOR STREETS				
	Granular "A"	TONNE	50	$22.00	$1,100
	Granular "B"	TONNE	100	$20.00	$2,000
330	CONCRETE CURB, GUTTER AND SIDEWALK				
	Curb and Gutter	LM	40	$90.00	$3,600
	Sidewalk	LM	40	$140.00	$5,600
351	HOT MIX ASPHALT CONCRETE				
	Surface course	TONNE	20	$165.00	$3,300
	Base course	TONNE	22	$165.00	$3,630
402	EXCAVATION FOR FOUNDATION				
	OM	CM	150	$20.00	$3,000
	Unwatering	LS	1	$50,000.00	$50,000
	Removal of Existing Bridge	LS	1	$75,000.00	$75,000
404	CONCRETE STRUCTURES				
	Cast-in-Place Retaining Walls	CM	50	$1,200.00	$60,000
	Cast-in-Place Footings	CM	35	$1,200.00	$42,000
	Pre-cast Box Culvert Sections	LM	16	$14,500.00	$232,000
415	ALUMINUM BRIDGE RAILING				
	Bridge Railing	LS	1	$5,000.00	$5,000

Preliminary Cost Estimate

Location 7 - O'Leary Avenue Bridge

- Remove and replace bridge

- 70m earth berm at left bank

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
511	**TOPSOILING, SODDING AND/OR HYDROSEEDING**				
	100mm Topsoil and Hydroseeding	**SM**	**450**	**$7.50**	**$3,375**

Subtotal =	$542,805
Contingency (20%) =	$108,561
Engineering (15%) =	$97,705
Subtotal =	$749,071
HST (13%) =	$97,379
TOTAL =	$846,450
BUDGET =	$847,000

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Preliminary Cost Estimate

Location 8 - Downstream of Mews Place Culvert
- 140m of earth berm at right bank

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
153	**MOBILIZATION/DEMOBILIZATION**	**LS**	**1**	**$2,000.00**	**$2,000**
311	**CLEARING AND GRUBBING**	**HA**	**0.2**	**$20,000.00**	**$4,000**
322	**BORROW**				
	Gravel Borrow	**CM**	**1100**	**$8.00**	**$8,800**
511	**TOPSOILING, SODDING AND/OR HYDROSEEDING**				
	100mm Topsoil and Hydroseeding	**SM**	**1200**	**$7.50**	**$9,000**

Subtotal =	$23,800
Contingency (20%) =	$4,760
Engineering (15%) =	$4,284
Subtotal =	$32,844
HST (13%) =	$4,270
TOTAL =	$37,114
BUDGET =	$38,000

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Preliminary Cost Estimate

Erosion Control System

SECTION	DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL
153	**MOBILIZATION/DEMOBILIZATION**	**LS**	**1**	**$15,000.00**	**$15,000**
311	**CLEARING AND GRUBBING**	**HA**	**0.4**	**$20,000.00**	**$8,000**
321	**STREET EXCAVATION**				
	USM	**CM**	**500**	**$20.00**	**$10,000**
511	**TOPSOILING, SODDING AND/OR HYDROSEEDING**				
	100mm Topsoil and Hydroseeding	**SM**	**4000**	**$7.50**	**$30,000**
OTHER	**CELLULAR CONFINEMENT SYSTEM**				
	Cellular confinement system	**SM**	**4000**	**$75.00**	**$300,000**

Subtotal =	$363,000
Contingency (20%) =	$72,600
Engineering (15%) =	$65,340
Subtotal =	$500,940
HST (13%) =	$65,122
TOTAL =	$566,062
BUDGET =	$567,000

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APPENDIX N

Cellular Confinement System Product Literature

GEOWEB® CHANNEL PROTECTION

SOIL STABILIZATION

The **GEOWEB® Channel Protection System** stabilizes and protects channels exposed to erosive conditions of all types and can be designed with appropriate infill types to withstand even the highest velocities.

CHANNEL OPTIONS:

Vegetated Protection:
Replaces costly, higher-maintenance rip-rap with lower-maintenance, less expensive, stabilized vegetation. Effective in low-flow channels and when low-to-high intermittent flows occur.

With a TRM, the vegetated GEOWEB® system can withstand velocities as high as 30 ft/sec (9m/sec). Ideal for drainage ditches, swales and stormwater channels.

Aggregate Protection:
Aggregate confined in the GEOWEB® system is far more stable than when unconfined. As a result, rather than using large, difficult to handle rip-rap, smaller and less expensive infill can be used in low-to-challenging flow conditions.

Concrete Hard-Armor Protection:
Concrete-filled GEOWEB® structures are ideal for channels exposed to severe hydraulic stresses. Concrete is poured in the structure onsite, creating an easy-to-install, flexible yet hard-armored system that is less costly than pre-formed concrete systems.

Multi-Layered Protection:
GEOWEB® multi-layered, vegetated channels create natural living retaining walls that can withstand high flows for short durations. They tolerate differential settlement while maintaining their structural integrity, and are quicker and easier to install than typical block systems.

GEOWEB® Key Applications

- Swales & Drainage Ditches
- Storm Water Diversion or Containment
- Process Water Channels or Containment
- Spillways/Downchutes/Drop Structures
- Culvert Outfalls
- Intermittent or Continuous/ Low- to High-Flow Channels

GEORUNNER® SURFACE FLOW PROTECTION

SOIL STABILIZATION

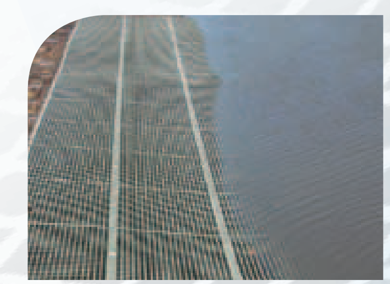
GEORUNNER® Flow Protection Mats are a low-cost solution for protecting embankments from scour and the erosive effects caused by water flow.

PROTECTS HIGH IMPACT AREAS

The series of lightweight, durable mats protects surfaces from intermittent and concentrated surface flows, water fluctuations and light wave action. They offer resistance to shear stresses and protect more efficiently than typical vegetation or rip-rap systems.

GEORUNNER® ADVANTAGES

- Effective in areas where erosion control blankets and turf reinforcement mats alone are not sufficient.
- Open mesh design promotes dense grass growth, increases system stability, reduces visibility and blends naturally with its environment.
- Mats are fully secured unit-to-unit, creating a fully integrated, flush surface, versus shingling found in other products.
- Anchored with industry-standard components to resist pull-out caused from saturated soils. A pneumatic driver allows quick driving of anchors, reduces worker fatigue.
- When anchored, the flexible system allows full contact with ground over landscape contours.
- Fully anchored system can be driven on by mowing or other lawn maintenance equipment.



GEORUNNER® Key Applications

- Culvert Outfalls
- Stormwater Channels
- Containment Ponds
- Swales & Drainage Ditches
- Shoreline Embankments
- Spillways, Down Chutes & Drop Structures
- Parking Lot Point Discharges