

FALMOUTH STORMWATER MANAGEMENT PLAN for the Municipality of the District of West Hants

Final Report



CBCL LIMITED

Consulting Engineers

Prepared for



WEST HANTS
NOVA SCOTIA

Final Report		2016/05/26	
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<i>Issue or Revision</i>	<i>Reviewed By:</i>	<i>Date</i>	<i>Issued By:</i>
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CHAPTER 1 **INTRODUCTION**

1.1 Background

The community of Falmouth is centrally located in Nova Scotia, and has geographic area of 5.22 km². Prominent to the geography of Falmouth is the Avon River. The community lies completely within the Avon River watershed. The river originates in the South Mountain and discharges into the Minas Basin in the Bay of Fundy. The lower portion of the river is tidal.

The population of the community of Falmouth has increased slightly since 2006. The total population of the community (according to the 2011 Census data) was 1,213 up 3% from the 2006 Census total of 1,179. Sufficient land is available within the community to accommodate future growth.

The development of the land in the community of Falmouth is being affected by a lack of stormwater drainage capacity. Stormwater drainage flooding and overflows are typically the result of peak stormwater flows exceeding the capacity of the drainage system. However, they can also occur as a result of high water levels in the Avon River. The outlet of the minor and major drainage systems drains into the Avon River upstream of the aboiteau. The closure of the aboiteau gate with every high tide and storm surge event prevents any drainage from the Avon River, which gradually backs up into the Falmouth area. This in turn, prevents the Falmouth stormwater system from draining as well, leading to widespread flooding which is very slow to recede. This has been an issue made worse since the construction of the causeway and aboiteau, and will only become worse with climate change, as increased rainfall amounts will lead to increased drainage which will face higher tide levels.

In 2015, the Municipality of West Hants retained CBCL to identify the causes of the flooding and propose solutions to alleviate the risks.

1.2 Purpose of Study

Risks to public safety and flooding damage to municipal infrastructure has resulted from insufficient drainage capacity in the minor and major stormwater system in the Falmouth area. In order to address issues of this kind, and to facilitate future development of the area, a stormwater management study is presented here. This study assessed the capacity of the present stormwater infrastructure capacity for conveying expected peak flow resulting from climate change. Pre-development and existing conditions flows were calculated throughout the community boundary and presented in a series of maps.

Necessary pipe sizes to carry those flows were also presented in the report, for a range of return periods. This allows a comparison between a given system's capacity and its extreme flows. Pre-development flows are presented to consider the potential value of implementing stormwater Best Management Practices to restore the pre-development hydrology of the area.

1.3 Study Approach

The main scope of this project included the following steps:

- Meet with Municipality of the District of West Hants representative to start the project and review information requirements and project details;
- Obtain and review all available background material concerning the project, including previous reports, dyke works, air photos, flow information, weather data, etc.;
- Delineate the existing development footprint and agree on extents of the Study Area;
- Obtain as much flooding extents information (including anecdotal) as possible to delineate the extents of previous flood events. This was used as a calibration data during this study;
- Obtain available information on the stormwater drainage system, including GIS and record drawings if available;
- Complete cross-sections survey at the main structures;
- Install a tide gauge in the study area to assess the tidal level;
- Update the delineation of the watersheds tributary to the study area using the Lidar and other available topographic data;
- Assemble computer models of the hydraulic system, including a Two-Dimensional dynamic model (EPA SWMM5 with the PCSWMM-2D platform);
- Conduct a Radar-Rainfall analysis to obtain precise rainfall information during calibration events;
- Calibrate the models on measured lake level at the study area location;
- Conduct comprehensive modelling to resolve the tidal peak water level characteristics of the site, as well as a tidal climate change impact analysis;
- Perform peak water level calculations throughout the stormwater system with the models, taking into account both tidal influence and extreme rainfall;
- Identify existing risks of flooding, for the 1 in 2 year, 1 in 5 year, 1 in 10 year and the 1 in 100 year events, by delineating and mapping floodlines throughout the study area;
- Assess culverts size to convey the 1 in 10 and 1 in 100 year extreme flows;
- Conduct a comprehensive review of Stormwater Best Management Practices that are applicable to the area; and
- Identify and evaluate temporary and permanent flood control options with the model.

2.1 Data Collection**2.1.1 Existing Data Collection**

The following existing data was obtained and reviewed for the community of Falmouth Stormwater Management Study:

- Provincial topographic data (5m contours) from the Government of Nova Scotia;
- Soil and geology mapping from Agriculture and Agri-Food Canada;
- Intensity-Duration-Frequency (IDF) curves for Kentville climate station from Environment Canada;
- Historical radar data for the community of Falmouth from Environment Canada;
- Historical tide data and tide predictions for the Avon River from Fisheries and Oceans Canada;
- Historical lake water level data from the Nova Scotia Department of Agriculture;
- Published reports on climate change by Environment Canada and Fisheries and Oceans Canada;
- Anecdotal information on historical flood events in the community of Falmouth; and
- Anecdotal information on the existing municipal infrastructure in the community of Falmouth.

2.1.2 Field Data Collection

A survey was completed to gather locations, inverts, and dimensions of the culverts with the community of Falmouth. Photos of each drainage structure were provided with the field survey. Survey data was combined with the existing information to allow the calculation of the capacity of each main culvert in the drainage system. Culvert capacities were calculated based on the Federal Highways Administration Hydraulic Design Series Number 5, "hydraulic design of highway culverts", which is the most widely accepted reference manual for culvert capacity calculations. Each culvert size and capacity is presented in the database in **Table 2-1**. Structure locations are shown in **Figure 2-1**.

2.2 Data Analysis**2.2.1 Watershed Delineation**

Watersheds were delineated using a combination of the Lidar data and the 5 m contours for the area of Falmouth. The major and minor watershed delineations are presented in **Figure 2-2**.

Watershed characteristics were estimated based on the Lidar data, aerial photography, land use mapping and soil mapping using GIS techniques. Imperviousness and roughness coefficients were estimated for each land use and applied to the watersheds using area-weighted averages. The capillary suction head and saturated hydraulic conductivity of the soil were estimated for each soil class from the soil mapping provided by Agriculture and Agri-Foods Canada and then applied to the watersheds using area-weighted averages. The estimated watershed characteristics for each watershed are presented in **Table 2-2**.

Table 2-1: Culvert Survey Data

Name	Size (mm)	Capacity (m ³ /s)	Name	Size (mm)	Capacity (m ³ /s)
Culvert #1	1200Ø	1.78	Culvert #27	620Ø	0.33
Culvert #2	1200Ø	2.10	Culvert #28	620Ø	0.31
Culvert #3	1200Ø	1.80	Culvert #29	780Ø	0.72
Culvert #4	300Ø	0.06	Culvert #30	620Ø	0.34
Culvert #5	800Ø	0.76	Culvert #31	460Ø	0.16
Culvert #6	620Ø	0.38	Culvert #32	620Ø	0.34
Culvert #7	900Ø	1.02	Culvert #33	900Ø	1.02
Culvert #8	900Ø	1.02	Culvert #34	900Ø	1.03
Culvert #9	900Ø	1.04	Culvert #35	620Ø	0.33
Culvert #10	1200Ø	1.78	Culvert #36	460Ø	0.15
Culvert #11	750Ø	0.65	Culvert #37	620Ø	0.34
Culvert #12	750Ø	0.66	Culvert #38	620Ø	0.4
Culvert #13	750Ø	0.64	Culvert #39	2x780Ø	1.44
Culvert #14	900Ø	1.03	Culvert #40	710Ø	0.35
Culvert #15	1090Ø	1.68	Culvert #41	460Ø	0.1
Culvert #16	460Ø	0.16	Culvert #42	Box: 620x620	0.43
Culvert #17	620Ø	0.34	Culvert #43	3x900Ø	2.58
Culvert #18	620Ø	0.27	Culvert #44	Box: 2x1350x1350	5.2
Culvert #19	1250Ø	1.00	Culvert #45	900Ø	1.04
Culvert #20	460Ø	0.16	Culvert #46	900Ø	0.88
Culvert #21	620Ø	0.40	Culvert #47	620Ø	0.34
Culvert #22	620Ø	0.21	Culvert #48	1100Ø	1.4
Culvert #23	900Ø	0.86	Culvert #49	5x230Ø	0.16
Culvert #24	460Ø	0.16	Culvert #50	2x620Ø	0.8
Culvert #25	460Ø	0.16	Culvert #51	620Ø	0.34
Culvert #26	620Ø	0.34	Aboiteau#1	4877x6096	108
			Aboiteau#2	4877x6096	108

Table 2-2: Watershed Characteristics

Subwatershed	Area (ha)	Maximum Overland Flow Length(m)	Slope (%)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)
S1	10.43	1621.93	5.79	35.04	0.01	0.31	210.06	1.02
S10	3.05	726.37	7.06	12.61	0.01	0.30	210.06	1.02
S11	11.19	1389.93	3.46	80.76	0.01	0.30	210.06	1.02
S12	0.50	219.67	7.54	100.00	0.01	0.00	210.06	1.02
S13	14.25	1594.27	3.86	62.50	0.01	0.27	210.06	1.02
S14	2.70	990.10	4.11	32.70	0.01	0.30	204.70	0.99
S15	2.04	734.55	1.51	2.16	0.01	0.30	164.80	0.80
S16	7.11	1097.05	6.28	16.50	0.01	0.30	198.58	0.96
S17	2.74	623.90	6.12	100.00	0.01	0.00	208.82	1.01
S18	5.29	877.98	2.92	100.00	0.01	0.00	80.55	0.39
S19	2.23	580.49	7.39	15.99	0.01	0.30	210.06	1.02
S2	56.07	3881.19	4.03	4.09	0.01	0.22	210.28	1.02
S20	4.31	933.97	8.93	43.22	0.01	0.30	108.63	0.52
S21	16.13	1624.70	5.97	24.14	0.01	0.18	210.06	1.02
S22	2.18	831.34	7.93	76.05	0.01	0.30	210.06	1.02
S23	3.66	872.64	5.95	27.32	0.01	0.24	210.06	1.02
S24	5.16	1422.12	7.44	61.88	0.01	0.24	234.77	1.02
S25	20.58	2029.24	8.45	11.66	0.01	0.29	241.18	1.02
S26	11.63	3416.09	6.22	57.11	0.01	0.23	210.10	1.02
S27	2.36	652.00	4.11	17.60	0.01	0.30	109.98	10.92
S28	8.40	1013.49	4.77	33.53	0.01	0.30	109.98	10.92
S29	37.87	1822.85	3.87	47.42	0.01	0.27	184.63	3.53
S3	23.14	3564.96	3.29	8.26	0.01	0.30	207.07	0.94
S30	3.01	500.90	4.99	48.21	0.01	0.30	109.98	10.92
S31	5.74	694.87	3.50	21.53	0.01	0.30	163.92	5.58
S32	11.37	1111.11	6.48	18.99	0.01	0.30	210.06	1.02
S33	1.05	390.76	6.41	100.00	0.01	0.00	210.06	1.02
S34	0.45	362.72	3.48	99.96	0.01	0.30	210.06	1.02
S35	0.82	391.97	10.73	54.43	0.01	0.30	210.06	1.02
S36	2.31	481.35	5.63	82.25	0.01	0.30	210.06	1.02
S37	0.21	201.76	4.14	4.37	0.01	0.30	210.06	1.02
S38	3.18	1046.98	8.30	100.00	0.01	0.00	210.06	1.02
S39	10.53	1305.39	3.50	25.20	0.01	0.25	210.06	1.02
S4	6.51	1500.54	3.51	4.02	0.01	0.30	194.12	0.80
S40	0.82	420.71	6.77	100.00	0.01	0.00	210.06	1.02
S41	0.78	469.81	10.04	100.00	0.01	0.00	210.06	1.02
S42	6.76	1262.07	4.39	29.00	0.01	0.30	210.06	1.02

Subwatershed	Area (ha)	Maximum Overland Flow Length(m)	Slope (%)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)
S43	0.65	330.42	6.34	100.00	0.01	0.00	210.06	1.02
S44	0.31	310.78	6.68	100.00	0.01	0.00	210.06	1.02
S45	0.19	175.37	6.80	100.00	0.01	0.00	210.06	1.02
S46	2.50	492.36	8.46	0.23	0.01	0.30	199.64	0.90
S47	1.86	380.21	7.98	77.03	0.01	0.30	210.06	1.02
S48	5.90	1203.72	7.98	77.03	0.01	0.30	210.06	1.02
S49	0.19	52.57	4.71	70.16	0.01	0.26	210.06	1.02
S5	13.71	1578.35	4.79	98.26	0.01	0.30	185.03	0.90
S50	1.79	488.01	4.71	70.16	0.01	0.26	210.06	1.02
S51	11.52	771.68	2.76	49.85	0.01	0.32	210.06	1.02
S52	24.25	1624.81	2.76	49.85	0.01	0.32	210.06	1.02
S53	0.45	28.87	3.67	17.13	0.01	0.28	160.47	0.78
S54	59.82	3835.32	3.67	17.13	0.01	0.28	160.47	0.78
S55	1.44	194.69	11.23	24.53	0.01	0.24	210.06	1.02
S56	10.17	1377.84	11.23	24.53	0.01	0.24	210.06	1.02
S57	0.56	105.30	6.19	65.59	0.01	0.17	210.06	1.02
S58	5.23	980.41	6.19	65.59	0.01	0.17	210.06	1.02
S59	52.65	2584.90	6.64	45.36	0.01	0.28	174.60	3.86
S6	26.98	2633.84	6.65	10.46	0.01	0.24	229.51	1.02
S60	8.08	559.20	6.03	45.44	0.01	0.30	210.06	1.02
S61	3.38	233.98	6.03	45.44	0.01	0.30	210.06	1.02
S62	3.67	444.32	7.01	94.00	0.01	0.24	224.03	1.02
S63	3.49	1466.39	7.94	7.35	0.01	0.30	213.84	1.02
S64	1.47	324.80	9.28	9.86	0.01	0.30	256.73	1.02
S65	4.55	664.84	7.52	31.50	0.01	0.30	230.29	1.02
S66	0.40	406.45	4.90	0.00	0.00	0.27	210.06	1.02
S7	4.91	792.57	7.59	45.13	0.01	0.15	210.06	1.02
S8	2.92	524.91	9.44	19.90	0.01	0.20	210.06	1.02
S9	4.65	1100.29	4.86	90.39	0.01	0.15	210.06	1.02

2.3 Rainfall Analysis

Multiple flood events have occurred within the area of Falmouth over the past decade that have resulted in significant damage to private and public properties. The September 2012 rainfall event was selected for hydraulic model calibration since it was one of the largest rainfall events that had occurred in recent years with a total rainfall amount of 68 mm recorded at the Kentville climate station.

2.3.1 Radar Rainfall Analysis

Rainfall data is often one of the largest uncertainties during the calibration process. This uncertainty is amplified when the watershed is large and the available rain gauges are very sparse. The rainfall intensity and total rainfall volume can vary significantly according to the location within the watershed. As the watershed size becomes larger, using a single point to estimate the rainfall across the entire watershed becomes less and less representative. The other uncertainty with using an isolated rainfall gauge is that the peak flow can be significantly affected by how the storm travels over the watershed.

Radar Data can be used to help reduce the uncertainty of using sparse rain gauge data by helping to determine the distribution of rainfall over a defined study area. Environment Canada radar data is



provided in 1 km² grid format. Each 1 km² within the grid has a unique, 10-minute time step, rainfall intensity time series. Using this data can help capture the spatial variation of a storm event and provide detailed rainfall data over the entire study area. **Figure 2-3** shows an example of a snapshot of the radar data grid superimposed on the study area watersheds, which clearly demonstrates the spatial variation of the rainfall intensity distribution across the various watersheds in the study area.

Figure 2-3: Spatial Variation of Rainfall Intensity Across Study Area Watersheds

Radar data must be calibrated before it is used for modelling. Radar data is very useful for determining the spatial distribution of a storm event. However, areal measurements and ground measurements (from point rain gauges) are often different and can be in error of a factor of 2 or more. This error is due to the vertical and horizontal air motions, the measurement of radar reflectivity factor, evaporation and advection of the precipitation prior to reaching the ground and variations in drop-size distribution. One of the most common calibration methods is to use rain gauge data to “ground-truth” the radar data. Calibration results will improve based on the amount of rain gauges within the study area and the spatial distribution of the rain gauges. Calibration results will also improve based on how close the rain gauges are to the study area itself.

For the September 10th 2012 storm event, there are no Environment Canada operated rainfall gauges within the study area. However, five Environment Canada climate stations within relatively close proximity to the study area were used to calibrate the data including Kentville, Kejimikujik, Greenwood, Parsborro, and Debert. These stations and their proximity to the study area are presented on Figure 2-4. The closest Environment Canada operated climate station is located in Kentville, approximately 25 km from Falmouth. The Kentville rain gauge only reported 68 mm of rain from September 9th to September 10th and the community of Kentville did not report any major flooding issues for this event.

Therefore an investigation into nearby private rain gauges was completed in order to determine whether there were higher rainfall intensities recorded near Falmouth.



Figure 2-4: Proximity of Rain Gauges Used for Calibration to Falmouth and Study Area

One private rain gauging station (INOVASCO40) is located directly in the community of Falmouth. INOVASCO40 recorded 120 mm of rain for the September 9th and 10th storm event. Private rain gauges can be an excellent source to supplement Environment Canada rainfall data, however, private rain gauges do not typically go through any type of quality assurance or quality control checks. Because there is no QA/QC for private gauges, the rainfall amount recorded at INOVASCO40 and Kentville Station was compared to other private rain gauges within close proximity. The trend shows that there are significantly higher rainfall volumes recorded near Falmouth when compared to the Kentville climate station. In addition to this trend, rainfall volumes recorded at private rainfall gauges near the Environment Canada Kentville climate station are generally consistent. The results of this evaluation are shown on **Figure 2-5**. Based on this evaluation, rainfall data from INOVASCO40 was used for calibration and is shown in **Figure 2-5**.

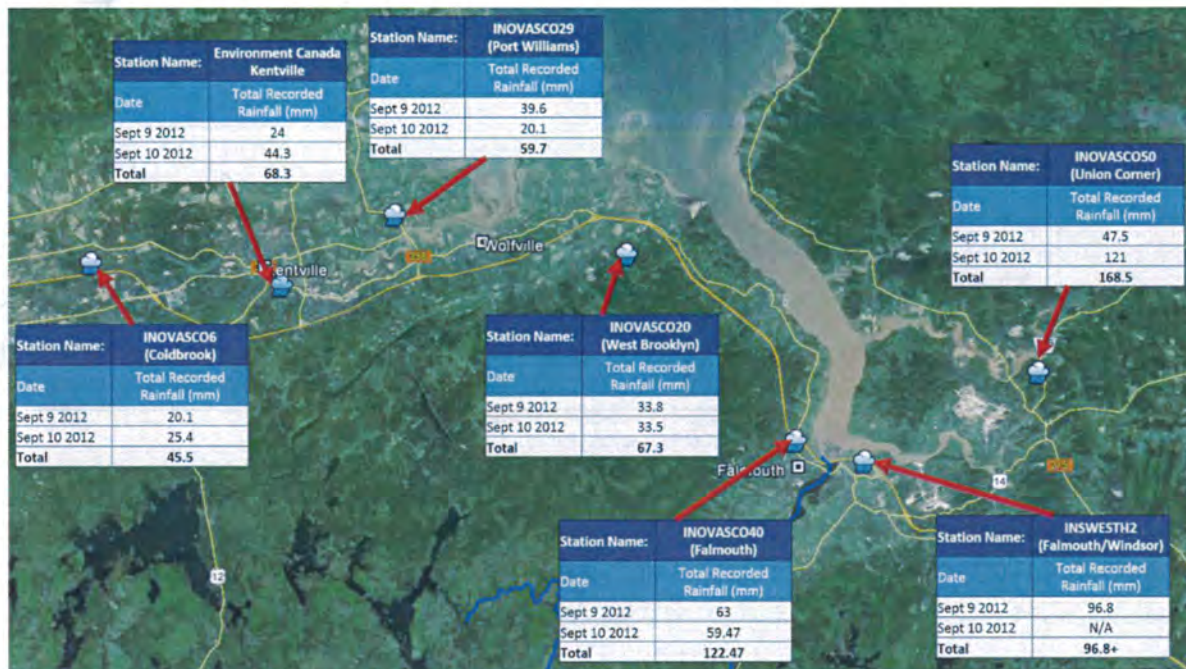


Figure 2-5: Evaluation of Private Rain Gauges to Determine Temporal Distribution of Rainfall

Calibration and the radar analysis was completed using PCSWMM's Radar Acquisition and Processing (RAP) Toolkit. PCSWMM's RAP toolkit is an innovative tool that, in Canada, is currently limited to the province of Nova Scotia. The calibration method chosen was the average method, which compares the average rainfall of each 1 km² radar grid element to the average rainfall measured by nearby rain gauges over a specified duration of time. The average measured rainfalls of the nearby gauges are interpolated between the gauges based on the relative location of each 1 km² grid element. Based on this comparison, a calibration factor for each 1 km² grid element is calculated and then applied to the radar measured rainfall. These calibrated rainfall intensities (for each 1 km² grid element) are then averaged across each watershed in the study area, providing each watershed with a unique and representative rainfall intensity distribution that could then be used for a more representative hydrologic model calibration. The results indicate that significantly more rainfall occurs near the community of Falmouth and southwest of the community. **Figure 2-6** summarizes the results of the radar analysis by showing the spatial distribution of the total rainfall volume for each watershed in the study area. The total rainfall amount for each watershed varies significantly, ranging from approximately 65 mm to a maximum of 125 mm, which is almost double. This clearly demonstrates how valuable radar data can be when compared to only using point rain gauges.

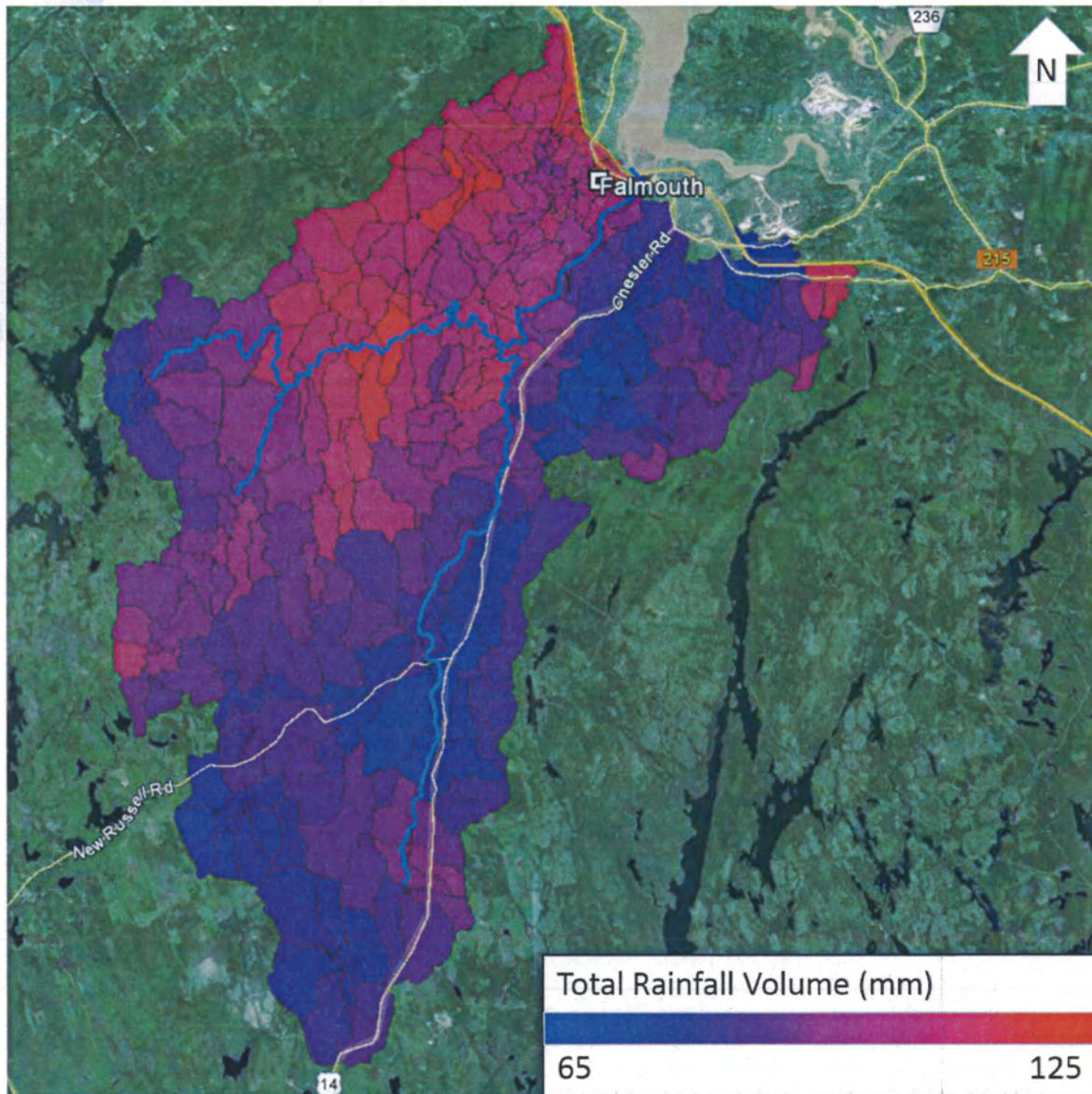


Figure 2-6: Spatial Distribution of the Total Rainfall Volume

2.3.2 Impact of Climate Change on Rainfall

The impact of climate change on extreme rainfall amounts was analysed for this study to model future climate change conditions for the year 2100. Environment Canada (EC) has used the results from several Global Climate Change models to estimate changes in the 24 hour rainfall amount for various return periods at several major climate stations in the region located at Greenwood, Shearwater and Sydney. These results are outlined in the 2009 EC report "Climate Change Scenarios for Atlantic Canada Utilizing a Statistical Downscaling Model". According to the report, rainfall intensity in the Halifax area may increase by 30% within the next 100 years. Due to the proximity of the community of Falmouth to the Halifax area we can use these results to estimate the potential effect of climate change in the study area. Therefore, climate change scenarios for future condition assessment were based on a 29% increase to the calculated design storms and to the projected extreme flows.

Table 2-3: Results for Return Periods for Extreme Precipitation Amounts

Return Period Projections for Extreme 24-hour Precipitation Amounts (mm)												
Return Period	10 Years				50 Years				100 Years			
	Hist	2020s	2050s	2080s	Hist	2020s	2050s	2080s	Hist	2020s	2050s	2080s
Nova Scotia												
Greenwood CGCM2	78.7	122.3	105.1	99.9	100.8	166.6	140	129.7	110.2	185.3	154.7	142.3
Greenwood HadCM3	78.7	94.2	79.9	86.3	100.8	127.9	101.3	112.8	110.2	142.1	110.3	124
Shearwater CGCM2	102.1	148.9	130.1	132.8	135.1	202.6	165.8	175.6	149.1	225.4	180.8	193.7
Shearwater HadCM3	102.1	120.9	103.7	109.8	135.1	157	131	141.5	149.1	172.2	142.6	154.9
Sydney CGCM2	N/A	135.1	137.8	129.2	N/A	186.5	180.6	165.3	N/A	208.2	198.6	180.5
Sydney HadCM3	N/A	109.4	108	113.8	N/A	137.9	140.1	143.5	N/A	150	153.7	156.1
New Brunswick												
Fredericton CGCM2	86.2	93.2	96.7	111.1	113.2	121.5	123.8	146.7	124.3	133.5	135.2	161.8
Fredericton HadCM3	86.2	89.1	94.5	93.5	113.2	113	118.2	119.3	124.3	123.1	128.3	130.1
Moncton CGCM2	86.8	93.3	103.1	132.8	113.7	122.6	135.1	175.6	125.1	134.9	148.7	193.7
Moncton HadCM3	86.8	89.9	104.3	103.5	113.7	112.9	136.3	141.5	125.1	122.6	149.9	157.5
Saint John CGCM2	100	119.7	107.4	118.8	143.4	153	133.2	149.8	157.1	167.1	144.1	162.9
Saint John HadCM3	100	117.5	128.6	124.8	143.4	148.3	168.6	159.6	157.1	161.4	185.5	174.3
Prince Edward Island												
Charlottetown CGCM2	74	120.7	107.7	99.8	94.2	168.4	144.8	128.6	102.7	188.6	160.6	140.7
Charlottetown HadCM3	74	96.4	93.7	79.1	94.2	110.3	121.1	97.2	102.7	120.4	132.7	104.8

This increase by 30% within the next 100 years is comparable to estimates of change obtained with other methods, such as the Clausius-Clapeyron equation and the Western University IDF Climate Change tool, both described below.

According to S. Westra, a variety of studies use of the Clausius-Clapeyron equation to estimate changes in rainfall caused by climate change. Using this equation, a rainfall increase rate of up to 14% per each °C of temperature increase has been found. However, this rate may vary as a function of location and seasonal temperature. One approach for including the impact of climate change in IDFs is to assume that extreme rainfall intensity increases linearly with changes in precipitation rate. Projected changes in temperature for Falmouth are on average 2°C for the 2050s and 3°C for the 2080s. Figure 2-7 shows projected increases in temperature for Falmouth from all available GCMs, with the ensemble average shown in black (graph constructed with the University of PEI Climate Change tool). This means that using the Clausius-Clapeyron equation, precipitation rate is expected to increase by approximately 30% by the 2050s.

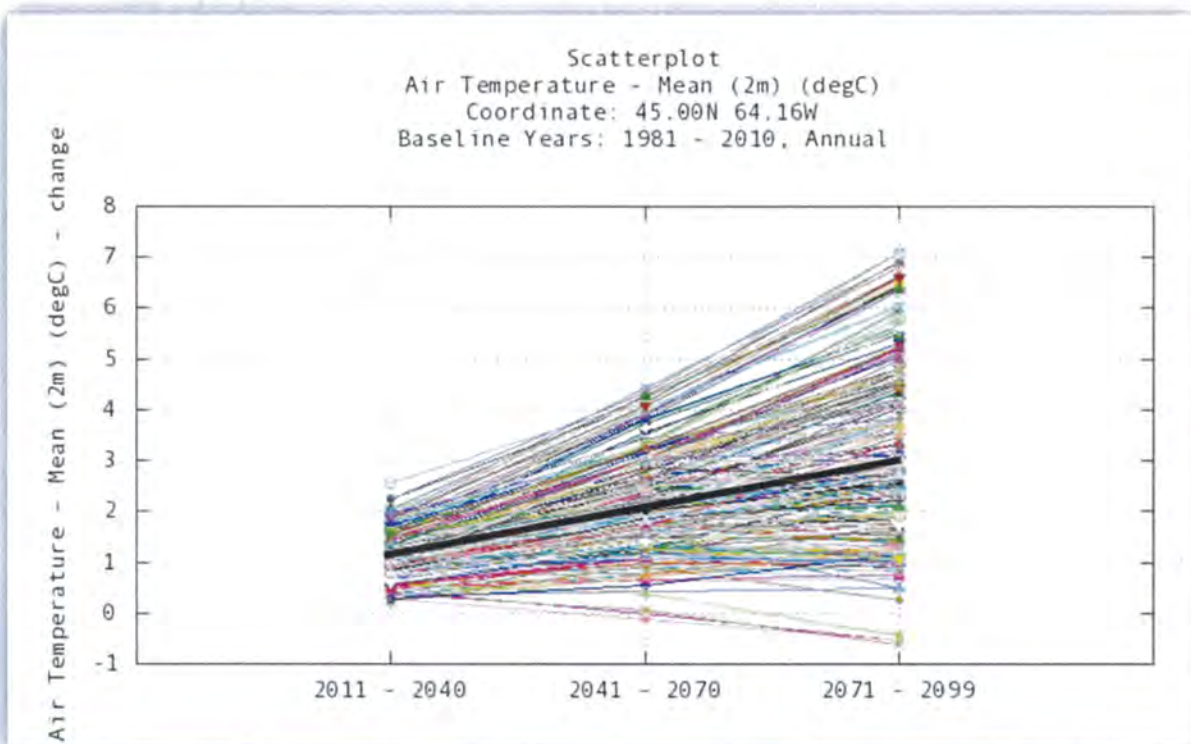


Figure 2-7: IDF Curves Resulting from the University of PEI Climate Change Tool

In 2015, the University of Western Ontario released a “Computerized IDF_CC Tool for the Development of Intensity-Duration-Frequency Curves under a Changing Climate”. This tool downscales global climate models (GCM) that are based on the findings of the Fifth Assessment Report (AR5) of the Intergovernmental Panel on Climate Change (IPCC 2013). The tool relates historical maximum daily data from GCM to historical sub daily extremes at a particular station; and uses this relationship to estimate future sub-daily extremes based on the future scenarios calculated with the GCM. The nearest climate station to Falmouth which has sufficient data and is available in the Western University IDF tool is

located in Kentville. Analysis using the tool shows a 20% and 25% increase in the 100-year storm total precipitation for the 2050s and 2080s respectively in Kentville (using RCP 8.5).

Therefore, the above comparison to the Clausius-Clapeyron and Western University IDF tool confirms that the EC estimate of 30% increase in rainfall intensity over the 100 years is adequate for the present analysis in Falmouth. Hence, climate change scenarios for future condition assessment were based on a 29% increase to the calculated design storms and to the projected extreme flows.

2.3.3 IDF Curve Data

Intensity Duration Frequency (IDF) curves for the Kentville CDA station are available, and are based on 32 years of rainfall data (1960-1995). Since floodlines need to represent very specific events, it is necessary to use very specific rainfall data. The IDF curves are the result of statistical analyses by Environment Canada to estimate the rainfall amount that is likely to fall on average every 2, 5, 10, or 100 years.

A Chicago distribution was then used to produce 24 hour design rainfall events using the IDF curves for the Kentville CDA station. The resultant 1 in 2,5,10, and 100 year design storms that were used for the hydraulic model are presented in **Figure 2-7** along with the associated IDF curves.

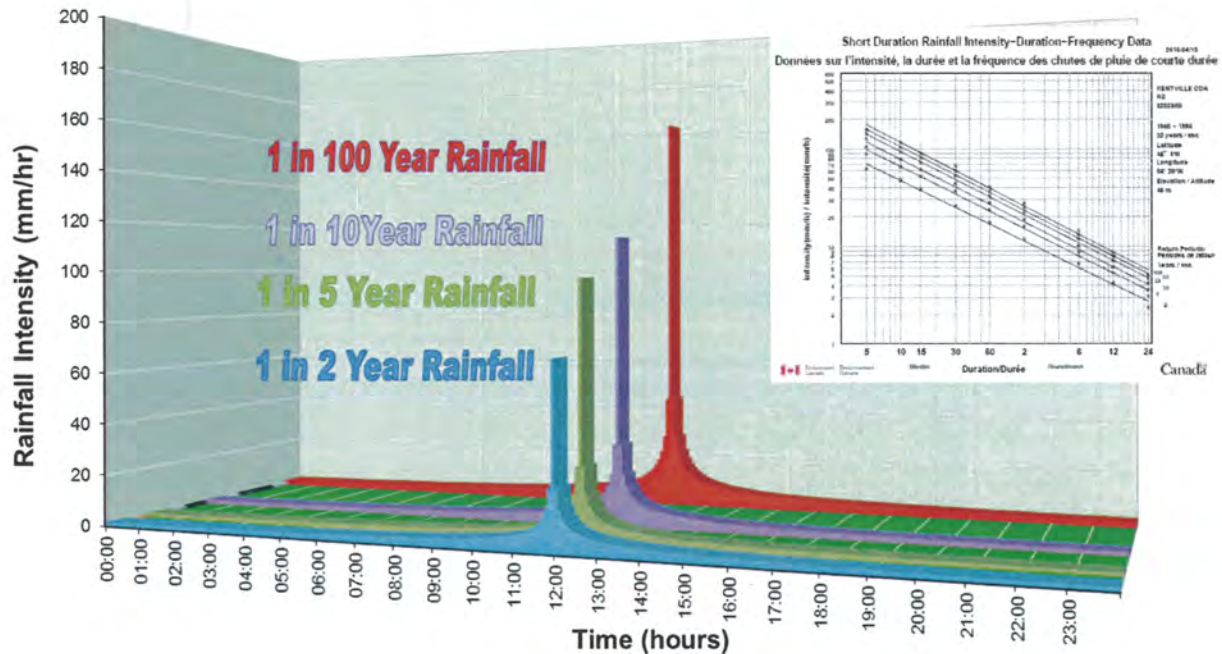


Figure 2-8: IDF Curve for the Kentville CDA Station

2.3.4 Tidal Data

The Avon River discharges into the Minas Basin, an inlet to the Bay of Fundy and sub-basin of the Fundy Basin. Tidal cycles in the Minas Basin have an effect on the degree and nature of flooding along the Avon River.

The processes involved in extreme tidal levels are difficult to estimate without a model or water level measurements. More accurate levels could be determined by creating a computer model of the Minas Basin, including the tidal inlet to the project site. This is an extensive undertaking, and would still produce results with a high degree of uncertainty. The extreme tidal levels were therefore estimated by analysing the individual components based on available information from various sources on the tides in the area.

Storm Surges

Storm surges are caused by meteorological effects on sea level, such as wind set up due to shoreward wind stresses and low atmospheric pressure, and can be defined as the difference between the observed water level during a storm and the predicted astronomical tide. In the 2011 Scenarios and Guidance for Adaptation to Climate Change and Sea Level Rise – NS and PEI Municipalities, Richard Daigle presents estimates of extreme storm surges for return periods of 10, 25 50 and 100 years for Hantsport with an uncertainty of 0.2m. The 2 year storm values was interpolated from the upper bound estimates presented by Daigle using a log-normal regression. **Table 2-4** shows the values derived from Daigle's report.

Table 2-4: Estimated Extreme Storm Surges for Sydney (Daigle, 2011)

Return Period	Storm Surge (m)	Upper Bound Residual
2	0.65	0.85
10	0.85	1.05
25	0.96	1.16
50	1.04	1.24
100	1.13	1.33

Sea Level Rise

Sea level rise in the Maritimes has been occurring since the end of the last ice age, about 10,000 years ago, when Prince Edward Island was still linked to the mainland of Nova Scotia and New Brunswick. The sea level rise at Charlottetown and Halifax has been approximately 0.3 m in the last century, which included an approximate 0.2 m of crustal subsidence. The rate of global mean sea level is accelerating in the 21st century due to global warming impacts, notably the melting of polar ice caps. The future rate of global mean sea level rise will likely be well above 2007 projections (which were up to 0.59m/century) from the Intergovernmental Panel on Climate Change. The future rate of sea level rise will likely be greater than 1m/century.

Richard and Daigle estimated values of Seal Level Rise in their 2011 report for the years 2025, 2055, 2085 and 2100. **Table 2-5** shows the values estimated by their report and the values interpolated by CBCL for the year 2015.

Table 2-5: Sea Level Rise from Richard and Daigle

	2000	2015	2025	2055	2085	2100
Estimated	0	0.096	0.16	0.45	0.86	1.1
Incertitude	0	0.018	0.03	0.15	0.36	0.48
Upper Bound	0	0.114	0.19	0.6	1.22	1.58

Extreme Tidal Level Estimations

The local astronomical tides at Avon River are semi-diurnal in character. **Table 2-6** shows the tidal water levels reported in the 2015 Canadian Tide Tables for Windsor relative to Chart Datum (CD) and the Canadian Geodesic Vertical Datum 28 (CGVD28).

Time series for extreme total water levels for present climate conditions were estimated by adding up the derived Higher High Water Large Tide (HHWLT) and storm surge time series. The estimated sea level rise was added to the resulting time series in order to estimate extreme water levels within 100 years. The resultant time series were used as boundary conditions in the PCSWMM model to estimate the extreme water levels around the Falmouth area.

Table 2-6: Tides and Extreme Still Water Levels

		2000	2015	2025	2055	2085	2100
Extreme Water Level (m CGD)	1 in 2 Year	8.95	9.05	9.11	9.40	9.81	10.05
	1 in 100 Year	9.43	9.53	9.59	9.88	10.29	10.53
	1 in 2 Year (Upper Bound)	9.15	9.26	9.34	9.75	10.37	10.73
	1 in 100 Year (Upper Bound)	9.63	9.74	9.82	10.23	10.85	11.21

To put this into perspective, the NSDA dykes are at elevation 8.5m approximately, and the Highway 101 is at elevation 10.0m approximately.

HYDROLOGIC AND HYDRAULIC MODEL DEVELOPMENT AND ANALYSIS

3.1 Modelling Approach

Hydrologic and hydraulic modelling was carried out for this study to quantify and analyse the flood risks in the community. Flood simulations were performed using the hydrologic and hydraulic modelling program PCSWMM2D.

PCSWMM2D integrates Version 5 of the Storm Water Management Model (SWMM) with a GIS engine and is capable of performing 2D hydrodynamic simulations. SWMM is a hydrologic and hydraulic model produced by the United States Environmental Protection Agency to study semi urban drainage systems. It conducts unsteady flow calculations to simulate water backup, pooling and culvert hydraulics by dynamically solving the continuity and momentum equations with a finite difference scheme.

PCSWMM2D was used for this study to estimate watershed flows and to simulate flooding in the community of Falmouth.

3.2 Hydrologic and Hydraulic Model Analysis

A hydrologic model was developed using PCSWMM to estimate the flows from each watershed. The model was developed by inputting the estimated watershed characteristics (**Table 2-2**). The flows estimated by the hydrologic model were then used as inputs for the hydraulic model.

An integrated 1D and 2D hydrodynamic model of the watercourses that pass through the community of Falmouth was developed using Lidar mapping and field measurements to estimate water levels along the floodplain. The 1D model components included the bridges and culverts. The floodplain was then modelled by assembling a 2D mesh based on the Lidar surface, with a mesh resolution that varied between 2.5 m to 10 m. Break lines were included in the mesh at roadways to ensure that locations with sharp changes in elevation were modelled with sufficient detail.

3.3 Model Calibration

Calibration of the hydrologic model was undertaken through a comparison of the simulated lake level and the observed water level at the Lake Pesaquid. There are two gates normally opened and closed with manual input in order to maintain Lake Pesaquid at a fairly constant level, by allowing excess freshwater to flow downstream while preventing rising seawater from flowing upstream. The gates can be opened to one quarter, one half or three quarters of their maximum extent. Finer control is also possible by the ability to open the gates by only one eighth of the maximum. The lake level is typically kept at approximately 2.7 m (based on geodetic elevation) in order to meet the needs of local recreational users of the lake.

A rainfall-runoff event from September 9-10, 2012 was selected for calibration. This event was selected as it represents the largest rainfall event that occurred recently. The one-minute interval lake level data was obtained from the Nova Scotia Department of Agriculture. The exact operation of the gate during the event was also shared with CBCL and we were able to enter the exact gate operation during the event in the model. Additional water level measurements were also conducted in Lake Pesaquid to confirm the measurements and supplement them where they may not be reliable, i.e. during high flows through the gate. Those measurements are presented in **Figure 3-1a**.

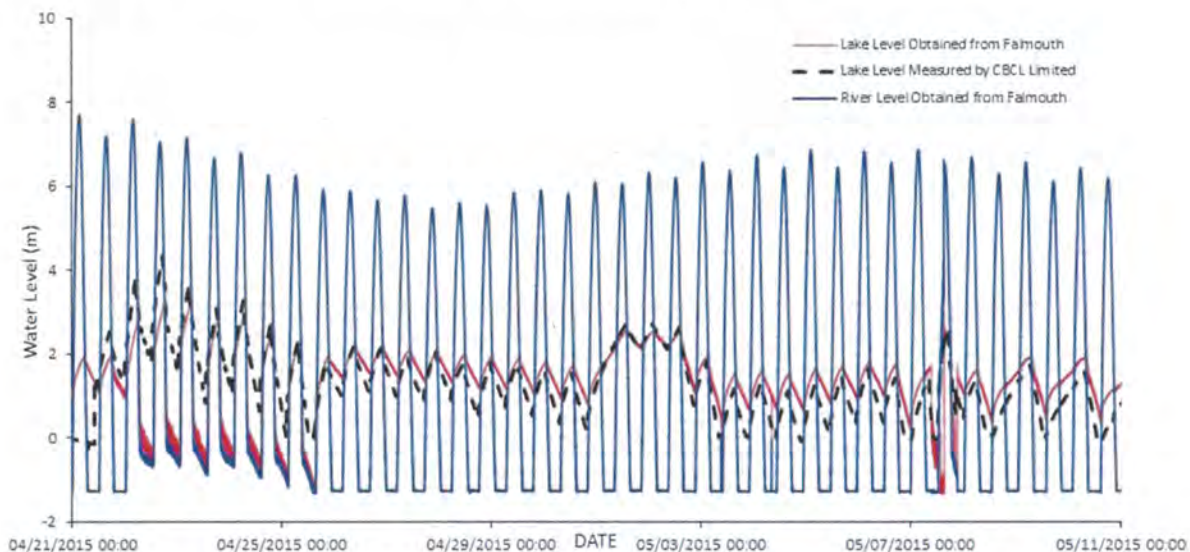


Figure 3-1a: Lake and River (Downstream of Gate) Level Measurements from NSDA and CBCL

Model parameters were adjusted until a reasonable calibration was achieved. This is a delicate task since the upstream watershed is very large, and the Pesaquid Lake captures all the flows, including groundwater, that drain towards the ocean. All the surface and groundwater processes involved can not possibly be included in any model, although we have made our best attempts at this using our hydrologic and hydraulic model. Illustrated in **Figure 3-1b** is the result from the model calibration. As shown, the PCSWMM model provides a reasonably accurate estimation of water levels.

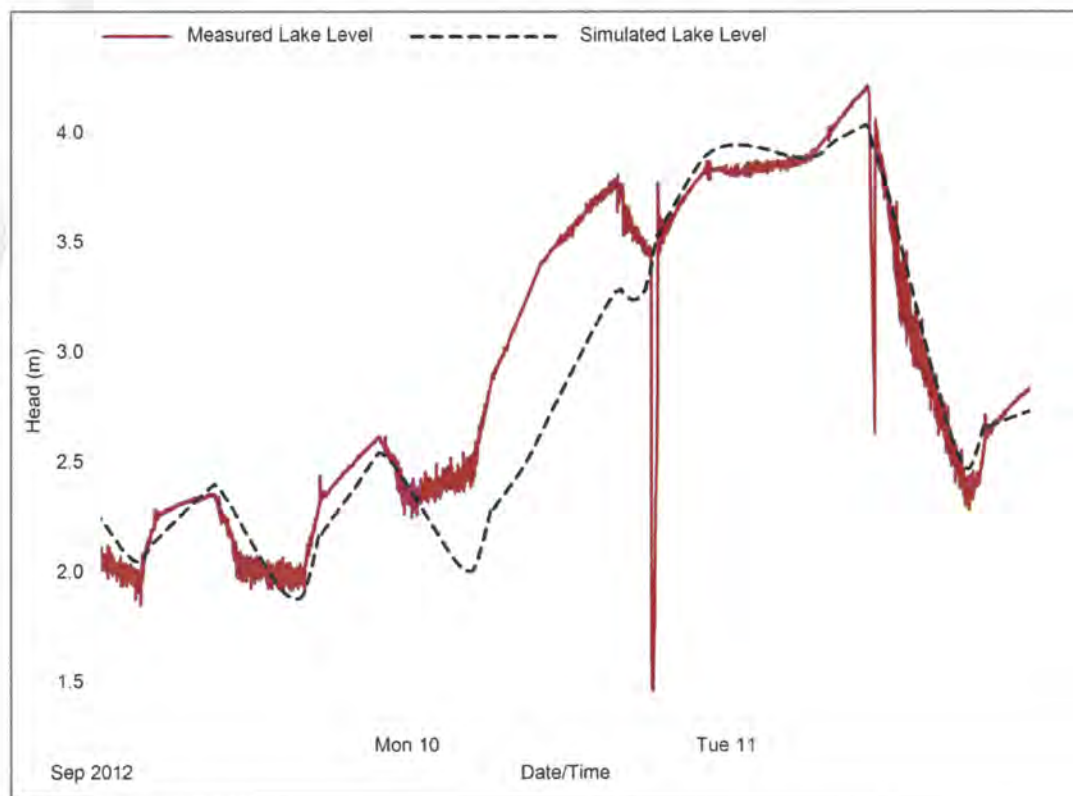


Figure 3-1b: Lake Level Calibration for September 2012 Event.

No historical water level or flow data was available within the community of Falmouth. The 2D model was therefore calibrated based on anecdotal flooding information gathered from meetings with community Staff, by simulating the calibration flood event and comparing the results to what was reported.

Floodlines developed for the September 2012 event are presented in **Figure 3-2**, comparing the estimated flood extents with the identified flood areas. The floodlines were produced by using GIS tools to interpolate the model results to the resolution of the Lidar data, which is at a 1 m by 1 m horizontal square grid. The model and resulting floodplain mapping includes the local flooding issues in these areas likely caused by local drainage systems.

3.4 Extreme Watershed Flow Estimation

Rainfall hyetographs for future climate change conditions were developed for the 1 in 2, 5, 10, and 100 year design storms by scaling the hyetographs to 29% predicted increases in 24-hour rainfall amounts for the year 2100.

The hydrologic model calculated extreme runoff flows from each watershed for the 1 in 2, 5, 10, and 100 year storm event including the impacts of climate change, which were then used as input for the hydraulic model for the flood protection design. Peak watershed flows estimated by the hydrologic model for each storm event are presented in **Table 3-1**.

Table 3-1: Estimated Peak Flows and Runoff Coefficients

Subwatershed	1 in 2 Year Storm (2100)		1 in 5 Year Storm (2100)		1 in 10 Year Storm (2100)		1 in 100 Year Storm (2100)	
	Peak Runoff (m ³ /s)	Runoff Coefficient	Peak Runoff (m ³ /s)	Runoff Coefficient	Peak Runoff (m ³ /s)	Runoff Coefficient	Peak Runoff (m ³ /s)	Runoff Coefficient
S1	0.71	0.70	1.06	0.75	1.31	0.77	2.12	0.82
S10	0.17	0.64	0.25	0.70	0.31	0.73	0.49	0.79
S11	1.09	0.91	1.61	0.93	1.98	0.93	3.23	0.95
S12	0.11	1.00	0.16	1.00	0.19	1.00	0.29	1.00
S13	1.21	0.83	1.79	0.86	2.21	0.88	3.60	0.90
S14	0.20	0.71	0.30	0.76	0.37	0.78	0.60	0.83
S15	0.04	0.58	0.07	0.65	0.08	0.68	0.14	0.75
S16	0.39	0.65	0.58	0.70	0.71	0.73	1.12	0.79
S17	0.43	0.99	0.64	0.99	0.78	0.99	1.24	1.00
S18	0.63	0.98	0.93	0.99	1.14	0.99	1.85	0.99
S19	0.15	0.67	0.22	0.72	0.27	0.75	0.43	0.81
S2	0.89	0.42	1.37	0.49	1.72	0.53	2.89	0.62
S20	0.45	0.86	0.66	0.88	0.81	0.89	1.27	0.92
S21	1.01	0.68	1.51	0.73	1.85	0.76	2.99	0.81
S22	0.30	0.91	0.45	0.92	0.55	0.93	0.88	0.95
S23	0.29	0.70	0.43	0.76	0.52	0.78	0.83	0.83
S24	0.53	0.84	0.79	0.87	0.97	0.88	1.56	0.91
S25	0.79	0.57	1.18	0.63	1.46	0.67	2.36	0.74
S26	0.74	0.79	1.11	0.82	1.37	0.84	2.26	0.87
S27	0.11	0.20	0.16	0.23	0.20	0.26	0.32	0.32
S28	0.55	0.35	0.82	0.38	1.01	0.40	1.65	0.45
S29	2.45	0.57	3.70	0.62	4.59	0.65	7.67	0.71
S3	0.47	0.41	0.72	0.48	0.89	0.52	1.48	0.60
S30	0.32	0.51	0.47	0.53	0.58	0.55	0.93	0.59
S31	0.31	0.31	0.47	0.37	0.58	0.39	0.96	0.48
S32	0.67	0.65	0.99	0.71	1.21	0.73	1.94	0.79
S33	0.20	1.00	0.29	1.00	0.35	1.00	0.55	1.00
S34	0.08	0.99	0.11	1.00	0.14	1.00	0.22	1.00
S35	0.13	0.83	0.18	0.86	0.22	0.88	0.35	0.91
S36	0.38	0.93	0.56	0.95	0.68	0.95	1.07	0.96
S37	0.01	0.64	0.02	0.70	0.02	0.74	0.04	0.80
S38	0.44	0.99	0.65	0.99	0.79	0.99	1.28	0.99
S39	0.62	0.66	0.93	0.71	1.14	0.74	1.85	0.80
S4	0.15	0.55	0.22	0.62	0.27	0.65	0.45	0.72
S40	0.15	1.00	0.22	1.00	0.27	1.00	0.42	1.00
S41	0.15	1.00	0.21	1.00	0.26	1.00	0.41	1.00
S42	0.44	0.68	0.66	0.73	0.81	0.76	1.30	0.81

Subwatershed	1 in 2 Year Storm (2100)		1 in 5 Year Storm (2100)		1 in 10 Year Storm (2100)		1 in 100 Year Storm (2100)	
	Peak Runoff (m ³ /s)	Runoff Coefficient	Peak Runoff (m ³ /s)	Runoff Coefficient	Peak Runoff (m ³ /s)	Runoff Coefficient	Peak Runoff (m ³ /s)	Runoff Coefficient
S43	0.13	1.00	0.18	1.00	0.22	1.00	0.35	1.00
S44	0.06	1.00	0.09	1.00	0.11	1.00	0.17	1.00
S45	0.04	1.00	0.06	1.00	0.07	1.00	0.11	1.00
S46	0.10	0.63	0.15	0.70	0.18	0.73	0.30	0.79
S47	0.34	0.92	0.49	0.93	0.59	0.94	0.92	0.96
S48	0.72	0.90	1.06	0.92	1.30	0.93	2.10	0.95
S49	0.04	0.90	0.06	0.92	0.08	0.93	0.11	0.95
S5	1.42	0.98	2.10	0.98	2.58	0.98	4.21	0.98
S50	0.27	0.89	0.39	0.91	0.48	0.92	0.76	0.94
S51	1.10	0.79	1.62	0.82	1.99	0.84	3.20	0.88
S52	1.66	0.76	2.49	0.79	3.07	0.81	5.03	0.85
S53	0.07	0.77	0.11	0.82	0.13	0.84	0.20	0.88
S54	1.81	0.52	2.73	0.58	3.38	0.61	5.58	0.68
S55	0.18	0.73	0.26	0.78	0.32	0.80	0.50	0.85
S56	0.73	0.69	1.08	0.74	1.32	0.77	2.10	0.82
S57	0.12	0.88	0.18	0.90	0.21	0.91	0.32	0.94
S58	0.64	0.87	0.94	0.89	1.15	0.90	1.86	0.93
S59	3.18	0.53	4.80	0.59	5.95	0.61	9.95	0.68
S6	0.87	0.54	1.32	0.61	1.63	0.64	2.68	0.72
S60	0.95	0.79	1.39	0.83	1.69	0.85	2.66	0.88
S61	0.50	0.80	0.72	0.84	0.86	0.86	1.34	0.89
S62	0.68	0.97	0.99	0.98	1.20	0.98	1.89	0.99
S63	0.12	0.57	0.18	0.64	0.22	0.67	0.35	0.74
S64	0.10	0.65	0.15	0.71	0.18	0.74	0.30	0.80
S65	0.43	0.73	0.63	0.77	0.76	0.80	1.20	0.84
S66	0.02	0.61	0.02	0.68	0.03	0.71	0.05	0.78
S7	0.58	0.79	0.86	0.83	1.04	0.85	1.66	0.89
S8	0.26	0.70	0.38	0.75	0.46	0.78	0.73	0.83
S9	0.57	0.95	0.85	0.96	1.04	0.97	1.69	0.97

3.5 Hydraulic Assessment of Drainage System

Flood estimates for the community of Falmouth were challenging to accurately estimate due to the wide range of land use types which varies from rural residential to commercial, but also due to the types of land drainages employed. Land drainage is provided primarily via open drainage channels with a limited discharge capacity due to several factors, including the local topography, but also the large number of culvert crossings. Detailed hydraulic assessments of the drainage network were undertaken using the hydraulic model.

Culverts are designed to convey the flow of water through manmade obstructions to natural flow, such as roadways or railway embankments. Typically, a culvert is designed to convey the peak flow from a specified design storm. The existing culverts' crossing were assessed to determine if the structures could meet the hydraulic requirements for the 1 in 2, 5, 10, and 100 design storms with climate change conditions. Analysis results were included in the floodplain maps.

The culverts were also sized to convey the 1 in 10 and 100 year storms without pipe surcharging, using Manning's equation and with an assumed slope of 0.5%. Inlet control was primary assumed, meaning that sizing decisions are based on the point of water inflow to the culvert. The sizing method used for this study is a simple method appropriate for planning purposes. This design approach results in a capacity determination such that, if a pipe has less than the calculated capacity based on peak flow, flooding may occur and therefore the pipe size is considered undersized. The estimated replacement sizes are based solely on hydrologic capacity and do not include site-specific design considerations that may optimize culverts.

Table 3-2 shows the required cross-section compared with the actual cross-section of the culverts currently in place, to determine the adequacy of the current culverts.

Table 3-2: Recommended Culvert Size for 1 in 10 Year and 1 in 100 Year Design Storms

Culvert Name	Existing Culvert Size (mm)	Culvert Capacity (m ³ /s)	1 in 10 Years Rain + Climate Change		1 in 100 Years Rain + Climate Change	
			Flow (m ³ /s)	Culvert Size (mm)	Flow (m ³ /s)	Culvert Size (mm)
Culvert #1	1200Ø	1.78	1.20	1050Ø	1.89	1200Ø
Culvert #2	1200Ø	2.10	0.34	750Ø	0.67	900Ø
Culvert #3	1200Ø	1.80	0.25	600Ø	0.26	600Ø
Culvert #4	300Ø	0.06	0.26	600Ø	0.42	750Ø
Culvert #5	800Ø	0.76	0.14	450Ø	0.40	750Ø
Culvert #6	620Ø	0.38	0.78	900Ø	1.20	1050Ø
Culvert #7	900Ø	1.02	0.83	900Ø	1.12	1050Ø
Culvert #8	900Ø	1.02	1.40	1050Ø	2.30	1350Ø
Culvert #9	900Ø	1.04	1.39	1050Ø	2.50	1350Ø
Culvert #10	1200Ø	1.78	1.39	1050Ø	2.50	1350Ø
Culvert #11	750Ø	0.65	0.02	300Ø	0.03	300Ø
Culvert #12	750Ø	0.66	0.64	900Ø	0.80	900Ø
Culvert #13	750Ø	0.64	0.57	750Ø	0.87	900Ø
Culvert #14	900Ø	1.03	1.07	1050Ø	1.76	1200Ø
Culvert #15	1090Ø	1.68	0.12	450Ø	0.15	450Ø
Culvert #16	460Ø	0.16	0.07	375Ø	0.11	450Ø
Culvert #17	620Ø	0.34	0.11	450Ø	0.17	450Ø
Culvert #18	620Ø	0.27	0.45	750Ø	0.71	900Ø
Culvert #19	1250Ø	1.00	0.31	600Ø	0.49	750Ø
Culvert #20	460Ø	0.16	0.62	900Ø	0.90	900Ø

Culvert Name	Existing Culvert Size (mm)	Culvert Capacity (m3/s)	1 in 10 Years Rain + Climate Change		1 in 100 Years Rain + Climate Change	
			Flow (m3/s)	Culvert Size (mm)	Flow (m3/s)	Culvert Size(mm)
Culvert #21	620Ø	0.40	0.57	750Ø	0.93	900Ø
Culvert #22	620Ø	0.21	0.58	750Ø	0.96	1050Ø
Culvert #23	900Ø	0.86	1.20	1050Ø	1.92	1350Ø
Culvert #24	460Ø	0.16	0.25	600Ø	0.40	750Ø
Culvert #25	460Ø	0.16	0.14	450Ø	0.22	525Ø
Culvert #26	620Ø	0.34	0.07	375Ø	0.11	450Ø
Culvert #27	620Ø	0.33	1.13	1050Ø	1.83	1200Ø
Culvert #28	620Ø	0.31	0.34	750Ø	0.54	750Ø
Culvert #29	780Ø	0.72	4.10	1800Ø	6.43	2100Ø
Culvert #30	620Ø	0.34	1.11	1050Ø	1.71	1200Ø
Culvert #31	460Ø	0.16	0.19	525Ø	0.32	600Ø
Culvert #32	620Ø	0.34	1.05	1050Ø	1.63	1200Ø
Culvert #33	900Ø	1.02	2.16	1350Ø	3.33	1500Ø
Culvert #34	900Ø	1.03	1.81	1200Ø	2.92	1500Ø
Culvert #35	620Ø	0.33	0.80	900Ø	1.29	1050Ø
Culvert #36	460Ø	0.15	0.02	300Ø	0.04	300Ø
Culvert #37	620Ø	0.34	0.22	525Ø	0.34	750Ø
Culvert #38	620Ø	0.40	0.20	525Ø	0.31	600Ø
Culvert #39	2x780Ø	1.44	1.04	1050Ø	1.68	1200Ø
Culvert #40	710Ø	0.35	0.82	900Ø	1.04	1050Ø
Culvert #41	460Ø	0.10	1.89	1200Ø	3.00	1500Ø
Culvert #42	Box: 620x620	0.43	1.38	1050Ø	1.89	1200Ø
Culvert #43	3x900Ø	2.58	3.66	1800Ø	4.90	1800Ø
Culvert #44	Box: 2x1350x1350	5.20	3.75	1800Ø	4.94	1800Ø
Culvert #45	900Ø	1.04	1.16	1050Ø	1.89	1200Ø
Culvert #46	900Ø	0.88	1.01	1050Ø	1.69	1200Ø
Culvert #47	620Ø	0.34	0.79	900Ø	1.15	1050Ø
Culvert #48	1100Ø	1.40	0.78	900Ø	1.14	1050Ø
Culvert #49	5x230Ø	0.16	0.01	300Ø	0.01	300Ø
Culvert #50	2x620Ø	0.80	0.47	750Ø	0.77	900Ø
Culvert #51	620Ø	0.34	0.67	900Ø	0.94	900Ø

3.6 Avon River Causeway Gate

Since this structure was included in the model, and the model was carefully calibrated to produce a representative picture of the local flows and water levels, it was analysed for its impact on water levels. Questions from the Municipality were also asked during the Start-Up meeting, relating to its relative influence on flooding risks.

The main finding is that Pesaquid Lake levels do not currently contribute to flooding risks within the developed area of Falmouth. This is demonstrated in Figure 4-4 a (Next Chapter), which shows that only small undeveloped areas by the Lake are more at risk of flooding from extreme lake levels than from extreme rainfall events.

Modifying the size of the causeway structure did make a notable difference in the peak water levels in the Lake, as water can be released faster during low tide. Similarly, the approach to operating the gate was studied with the model, and it was found that peak water levels can be reduced if the water level in the Lake is allowed to be lower for a temporary duration.

In summary, the model showed that the causeway gate does not impact flooding to sensitive areas in Falmouth. As peak river flows increase and sea levels rise in the future, however, the risk of high lake levels will increase.

3.7 Best Management Practice Analysis

Best Management Practices (BMP) increase infiltration and storage of water which are increased due to development, decreasing runoff to more closely approximate the natural, pre-development hydrology of a site. BMP benefit communities by dispersing stormwater runoff on-site, rather than concentrating and transferring it downstream. At both the parcel and watershed scale, these methods reduce both the volume of run-off and the contamination of runoff to receiving waters from non-point source pollution. In order to study the capacity of runoff reduction methods for mitigating impacts of climate change and population growth BMP were simulated in the PCSWMM model by minimizing the percentage of imperviousness in the catchment (thereby increasing perviousness and infiltration). Peak flows were compared for scenarios with, and without, the application of BMP. The difference between the number of undersized culverts with and without BMP was used as an indicator of the potential efficacy of BMP. BMP is not one single prescription, but rather a combination of techniques deployed according to site conditions and the judgment of the designer. Floodplain maps with the application of BMP for 1 in 2, 5, 10, and 100 year design events with climate change are delineated and are presented in **Figure 3-3**.

FLOODPLAIN MAPPING AND & FLOOD MITIGATION ANALYSIS

4.1 Floodplain Mapping

Once compiled and calibrated, the model was used to assess the 1 in 2, 5, 10, and 100 year flood scenarios with climate change for the community of Falmouth and the Avon River. Since the rainfall and tides have their own individual influences on the river's extreme water levels, for the 1 in 100 year flood event, the dominating influence of both the rainfall and tides was modeled. Scenarios consisting of combinations of rainfall, tides, storm surges, and sea level rise conditions were simulated for floodlines. Since extreme rainfall events often bring storm surges, the 1 in 2 year storm surge occurring during the HHWLT was selected as representative of the sea level conditions during the 1 in 100 year rainfall events, and vice versa. Thus, each floodline delineation consists of the maximum of the extreme rainfall flood event and extreme sea level flood event. The resulting floodline therefore does not represent the flood extents for a single event that can occur, but rather the combined flood extents for two different types of events, each having the same return period.

Climate change will likely bring more precipitation to Atlantic Canada. In this study, future (Year 2100) flood risks were assessed. Assessing future flood risks allows for better management of surface and storm water resources through land use planning and infrastructure design specifications. Floodlines were delineated based on the selected event combinations. The detailed information of tides and extreme still water levels can be found in **Table 2-6**. The 1 in 2 year storm surge value was interpolated from the upper bound estimates presented by Richards & Daigle using a log- normal distribution. Floodplains for these extreme events were presented in **Figure 4-1, 4-2, 4-3, and 4-4 (a and b)**.

4.2 Analysis of Flooded Areas

The flood discharge is influenced by physical characteristics of the watershed like watershed area, slope, flow length, the percentage of forest area and the percentage of wetlands and lakes. Flooding may be also triggered when a large volume of stormwater is transported by drainage systems. Often, in the event of a flood, culverts do not do their job, which can result in significant damages to the road and in the interruption of highway traffic flows. In addition to an increased risk for the road network users and road infrastructures, an under-dimensioned culvert can decrease the stream channel stability.

Consequently, the objects in flooding areas, such as people, estates and assets, as well as public infrastructure, may be damaged.

The floodline delineations shown in **Figures 4-1 to 4-4 (a and b)** demonstrate that there are widespread flooding risks estimated in the community of Falmouth. The flooding in Falmouth is mainly caused by the insufficient drainage capacity and the flat landscapes within the community. It is also caused by the complex interaction between rainfall, Avon river flows, tides and storm surges. Figure 4-4b for example shows the widespread extent of the flooding if the dykes are not raised to prevent overtopping.

Elderkin Creek in northwestern Falmouth is crossed by Highway #1 and Highway #101. Its downstream receiving water body is the Avon River. The flooding along the Elderkin Creek is caused by several sources acting in combination. Flood risk of this tributary is influenced by rainfall, the tidal level of the Avon River, and the drainage systems. Hydraulically, the existing culverts along the creek cause appreciable backwater during the extreme weather conditions. Although the existing drainage system may cope with some rainfall, floods may still occur when rainfall or the tidal level is higher than the threshold.

Flood risks were found to be higher along Highway #1, Falmouth Back Road, Gabriel Road, and Palmetter Drive. Flooding in these locations can cause damage to private residences. The flooding occurring in these areas during 1 in 100 year storm event is only very marginally due to the undersized drainage capacity.

Flood risks were also noticed along Falmouth Dyke Road, especially in the area adjacent to the pond which is near to Shetland Road. The pond is connected to the Elderkin Creek by an outlet pipe with the diameter of 0.7 m. When the creek floods in severe weather, the 0.7 m outlet pipe backs up causing overflow from the pond. The identified flood risks around this area also indicated that the pond's outlet pipe was undersized.

4.3 Flood Mitigation Options

The analysis carried out in the previous sections has identified flooding risks from the hydrologic and hydraulic assessment, with a significant increase as climate change considerations are factored in. This section will seek to evaluate the potential benefits of various flood mitigation measures.

Typical solutions to flooding problems include:

1. Increased conveyance capacity for the drainage system to convey peak flows at a lower water level.
2. Decreasing the peak runoff flows in the drainage system by adding additional storage to the system, or by increasing the amount of rainfall infiltration (Best Management Practices).

Each of these potential solutions is investigated in this section.

4.3.1 Increasing the Conveyance Capacity

Flooding can be caused by a multitude of factors, including not only high flows and insufficient culvert capacities, but also high surface roughness, low channel roughness, low channel slope or insufficient

room in the floodplain. Thus, the first step in determining the factors that contribute to flooding is typically to assess whether the culvert structures have an impact on the overall flood levels and whether or not upgrading these structures will address the flood risks. The model was therefore used to evaluate the impact of removing the culvert restrictions on flooding risks. This was achieved by upgrading their dimensions to an opening size large enough to convey the upstream flows without surcharge.

The model results, presented in **Figure 4-5**, indicate that the floodlines are not estimated to be significantly reduced under unrestricted flow conditions compared to existing conditions. This is important, because it indicates that the culverts themselves are not the primary cause of flooding, even though they are indeed the source of some of the localised flooding. Thus, upsizing the culverts would locally reduce some of the accumulation of water, but would not eliminate the flooding within the adjacent upstream and downstream private properties.

Flooding can therefore be explained in this case simply by either a lack of capacity in the floodplain, or by the fact that properties exist within the natural floodplain. This points to the fact that the flood extents of most areas are natural phenomenon that have likely been occurring since before the land was developed. Furthermore, the existing development areas that drain to the major watercourses contribute to increased peak flows and runoff volumes to these watercourses compared to the natural conditions before tree clearing and construction. In particular, some of the smaller watersheds have a larger percentage of developed area, which results in the development having a higher impact on the river flows.

It is also noted that the development within the floodplain area has likely slightly decreased the roughness of the floodplain, resulting in slightly faster drainage of floodplain flows to the Avon River. Thus, the flood extents are likely slightly impacted by both the increased runoff from development and the faster drainage of flows in areas where the floodplain is more developed.

4.3.2 Reducing Flooding Risks by Adding Storage

Reducing flows can be an effective approach to flood reduction if the flows and runoff volumes during storm events are not too large. However, in this case, the major watershed sizes are indeed relatively large and will therefore require a significant amount of excavation and land area to make an impact on the flows. To assess the feasibility of this option, an additional detention pond with a volume of 21,000 m³ was added besides the existing ponds near Shetland Road to reduce the peak flows along the Elderkin Creek watercourses, as shown on **Figures 4-6 and 4-7**. With the detention storage, the flooding area is reduced from 38,000 m² to 20,500 m², which leads to approximately 46% reduction in flooding area with the total cost of \$1.05M if a value of \$50/m³ is used. The overall performance of detention ponds in attenuating peak flows strongly depends on the site location and site topography. The proposed detention ponds were located in order decrease the flood depth on nearby areas, which they did, but this had little effect on flooding in the most affected area, by Shetland road, and had almost no impact on the number of flooded buildings. It can be concluded that due to the large volumes of storage necessary, the available flood storage would not be sufficient to provide adequate flood protection.

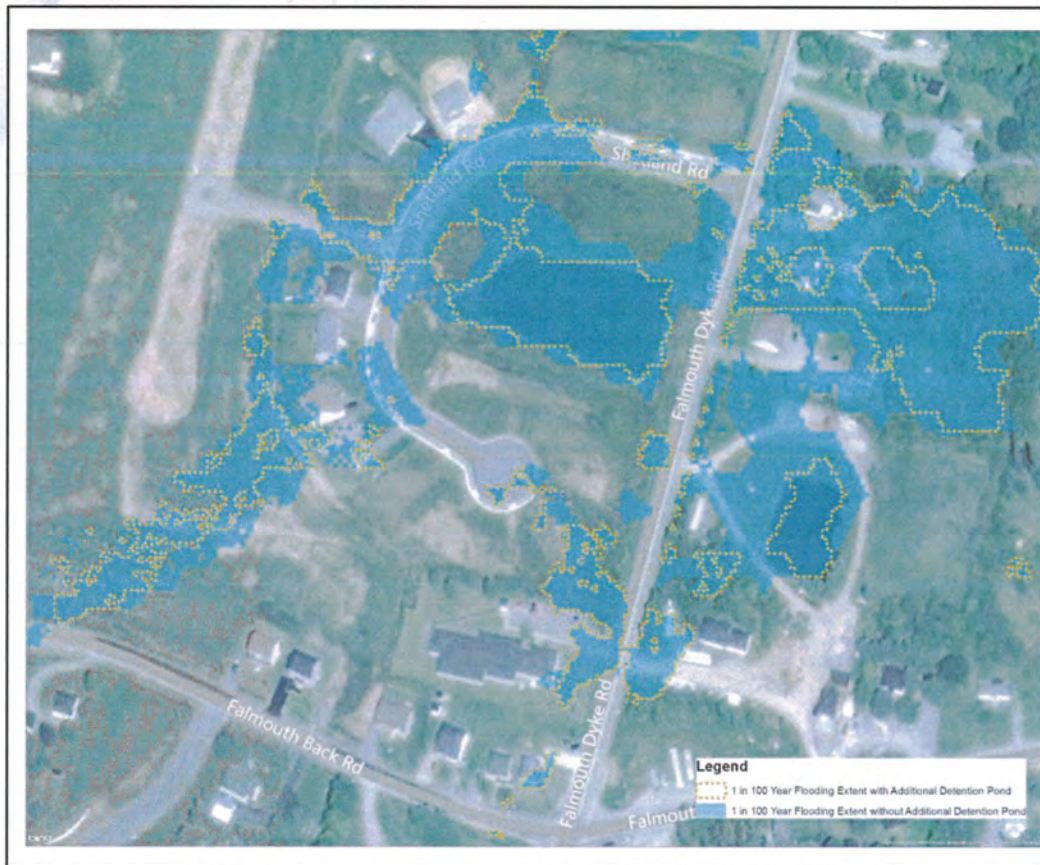


Figure 4-6: 1 in 100 Year Flooding Extent with Climate Change with or without Additional Detention Pond installed

CONCLUSIONS & RECOMMENDATIONS

One the most efficient ways to deal with flooding risks is to manage the issue of high runoff at its source. Flooding is only amplified when runoff is allowed to increase, and infiltration of water into the ground is decreased. Most often, development allows this to happen and therefore increases flooding risks. The more water is encouraged to infiltrate in the ground, the more the high water levels are controlled, and the more the overall river health is protected. Stormwater Management has the following benefits for the river, the riverine residents, the overall watershed community and the residents of the Falmouth area:

- Decreases flooding risks and entailed risks to infrastructure, land value, liability and public safety;
- Decreases peak flows, resulting in smaller infrastructure costs;
- Aquifer recharge, reducing the strain on water supply sources;
- Reduces pollution to drinking water supplies, recreational waters and wetlands, saving future expenditures for restoration of valuable water resources;
- Protects water quality and increases low flows in the river, enhancing fish habitat in this uniquely valuable river system;
- Reduces energy costs by constructing new green roofs or retrofitting existing roofs; and
- Through the above results, improves the quality of life and increases property value.

A more concentrated (cluster) subdivision design, with less impervious area and smaller infrastructure (stormwater drainage and other utilities), also means significant cost savings to developers (who will therefore show less resistance in implementing this type of design) and reduces maintenance costs.

These aspects show that even if the original target for stormwater management is the reduction of flooding risks, there are a host of other associated benefits to the overall community, which all contribute to more sustainable development. This makes stormwater management through Low Impact Development (LID) and Best Management Practices (BMPs) an important recommendation for implementation at the planning and by-law levels in the community.

Stormwater management is no longer an innovative or rare approach. It is now very well understood and implemented in a majority of communities across North America. There is a multitude of very-well researched documents describing comprehensively how-to approaches to Low Impact Development (LID) from the planning level to the lot development stage, including retrofitting existing developments. This section of the report gives a summary of typical approaches in various settings from areas with

proven records of successfully implementing LID and BMPs. General recommendations on typical targets for results are also included at the end of the chapter.

5.1 Planning for Low Impact Development

The LID approach provides opportunities to build homes and businesses, while conserving natural areas and drainage patterns. LID is accomplished as a two-step process;

- Thoughtful site planning; and
- Incorporation of "natural" stormwater best management practices (BMPs).

Thoughtful site planning begins with the identification of critical site features such as wetlands, habitat areas, and /or drinking water protection areas that should be set aside as protected open space. Natural features, such as vegetated buffers and view sheds, will also play an integral role in any LID planning exercise. After the critical open space areas are identified and set aside, sustainable development areas are then identified as "building envelopes". General goals include the following:

- Concentrate Development and Mix Uses: The LID site planning process sets aside key natural features and focuses development into clustered patterns on the remaining land. The LID planning process results in housing that makes more efficient use of land and conserves critical natural features such as wetlands, vegetated buffers, and drinking water protection areas; and
- Protect Land and Ecosystems: The reduction of impervious surfaces reduces the amount of surface runoff and through the infiltration of stormwater recharges the groundwater system, thereby restoring the natural hydrologic cycle. This preserves groundwater supplies and base flow to streams and wetlands.

The Credit Valley Conservation a community conservation area in Ontario, has published a comprehensive guide to implementing sustainable development through Low Impact Development and Stormwater Best Management Practices ("Low Impact Development Stormwater Management Planning and Design Guide", Version 1.0 – 2011). An extract is presented below, which tabulates the summary of stormwater management and land use planning steps in order to achieve Low Impact Development. This guide presents a very well organised approach, without being overly prescriptive on individual approaches to achieving Low Impact Development. This document could serve as an example for creating a similar document in the community of Falmouth.

It is recommended that at the planning level, the system described above be adopted, which includes the development of:

- Watershed Master Plans, a Growth Plan and an Official Plan;
- Municipal / Subwatershed Secondary and Block Plans;
- Neighbourhood Draft Plan of Subdivision / Functional Servicing Plan, followed by the Registered Plan;
- Site Plan with site specific opportunities, while staying consistent with the direction of the higher level plans; and
- Site Permits and approvals.

Stormwater management planning is closely linked with resource, land use and Municipal planning. This is essential since it establishes the linkage and inter-dependence of community planning with stormwater management planning at all levels. At the higher watershed level, it typically requires the cooperation of community and Municipal levels to incorporate into their official planning documents the approaches recommended in the watershed plan. Full descriptions of each type of plan are given in the “Infraguide: Stormwater Management Planning”, NRC, 2005 document.

Table 5-1: Summary of Stormwater Management Planning at Key Scales and Land Use Planning Stages

Scale	Planning Stage	Description
Watershed plans	Master Plans Growth Plan Official Plan	Major themes and objectives for the municipality's future growth are established, and challenges and opportunities for growth are identified, such as municipal policy direction for innovative SWM approaches and other climate change initiatives.
Community/ Subwatershed	Secondary Plan	Major elements of the natural heritage system are identified including terrestrial, aquatic and water resources (hydrology, hydrogeology, fluvial geomorphology, etc.). Stormwater management objectives for surface and groundwater resources. Future drainage boundaries, locations of stormwater management facilities and watercourse realignments are established.
	Block Plan	The location of lots, roads, parks and open space blocks, natural heritage features and buffers, and stormwater management facilities are defined. A full range of opportunities to achieve stormwater management objectives are identified, establishing a template for the more detailed resolution of the design of stormwater management facilities at subsequent stages in the planning and design process.
Neighbourhood	Draft Plan of Subdivision/ Functional Servicing Plan	Conceptual design is carried out for stormwater management facilities. Consideration must be given to how stormwater management objectives can be achieved and how these objectives influence the location and configuration of each of the components listed above
	Registered Plan	Detailed design is carried out for stormwater management facilities.
Site	Site Plan	Site-specific opportunities are identified to integrate stormwater management facilities into all of the components of a development including landscaped areas, parking lots, roof tops and subsurface infrastructure. Solutions must be considered in the context of the overall stormwater management strategy for the block or secondary plan area to ensure that functional requirements are achieved
Site	CA Permits and other approvals	Detailed design of SWM for the site

5.2 Best Management Practices in Forested Areas

Sources of pollution associated with forestry activities include removal of streamside vegetation, road construction and use, timber harvesting, and mechanical preparation for the planting of trees. Of these, road construction and road use are the primary sources of pollution on forested lands, contributing up to 90 percent of the total sediment from forestry operations.

Harvesting trees in the area beside a stream can affect water quality by reducing the stream bank shading that regulates water temperature and by removing vegetation that stabilizes the stream banks. These changes can harm aquatic life by limiting sources of food, shade and shelter.

5.2.1 Recommendation to Use Nova Scotia's Code of Forest Practice

Nova Scotia has recently published a very good document in this specific area, which describes an approach to implementing sustainable forest management techniques. The document was produced for Crown Land, but it is applicable to any forested area in the province. The document indeed recommends that it be used for forest management in the province wherever possible.

The reference for this document is the following:

Nova Scotia's Code of Forest Practice

A Framework for the Implementation of Sustainable Forest Management
Guidelines for Crown Land

Nova Scotia Natural Resources

August 2012

It is recommended that this document be made a requirement for all forestry activities in the community of Falmouth.

General recommendations for forestry Best Management Practices to include in a preharvest plan are listed in the following section, extracted from the USEPA.

5.2.2 Typical Forestry Best Management Practices

Preharvest Planning: Opportunities to Prevent Pollution

To limit water quality impacts caused by forestry, public and private forest managers have developed and followed site-specific forest management plans.

Following properly designed preharvest plans can result in logging activities that are both profitable and highly protective of water quality. Such plans address the full range of forestry activities that can cause pollution. They clearly identify the area to be harvested; locate special areas of protection, such as wetlands and streamside vegetation; plan for the proper timing of forestry activities; describe management measures for road layout, design, construction, and maintenance, as well as for harvesting methods and forest regeneration.

Public meetings should be held to provide residents with an opportunity to review and comment on the development of forest management plans.

Factors Considered in the Preharvest Plan

Surveying the Site

Preactivity surveys can help identify areas that might need special protection or management during forestry operations. Sensitive landscapes usually have steep slopes, a greater potential for landslides, sensitive rock formations, high precipitation levels, snowpack, or special ecological functions such as

those provided by streamside vegetation. Forestry activities occurring in these areas have a high potential of affecting water quality.

Timing

Because most forestry activities disturb soil and contribute to erosion and runoff, timing operations carefully can significantly reduce their impact on water quality and aquatic life. Rainy seasons and fish migration and spawning seasons, for example, should be avoided when conducting forestry activities.

Establishing Streamside Management Areas (SMAs)

Plans often restrict forestry activities in vegetated areas near streams (also known as buffer strips or riparian zones), thereby establishing special SMAs. The vegetation in an SMA is highly beneficial to water quality and aquatic habitat. Vegetation in the SMA stabilizes streambanks, reduces runoff and nutrient levels in runoff, and traps sediment generated from upslope activities before it reaches surface waters. SMA vegetation moderates water temperature by shading surface water and provides habitat for aquatic life. For example, large trees provide shade while alive and provide aquatic habitat after they die and fall into the stream as large woody debris.

Managing Road Construction, Layout, Use, and Maintenance

Good road location and design can greatly reduce the transport of sediment to water bodies. Whenever possible, road systems should be designed to minimize road length, road width, and the number of places where water bodies are crossed. Roads should also follow the natural contours of the land and be located away from steep gradients, landslide-prone areas, and areas with poor drainage. Proper road maintenance and closure of unneeded roads can help reduce pollution impacts from erosion over the long term.

Managing Timber Harvesting

Most detrimental effects of harvesting are related to the access and movement of vehicles and machinery, and the dragging and loading of trees or logs. These effects include soil disturbance, soil compaction, and direct disturbance of stream channels. Poor harvesting and transport techniques can increase sediment production by 10 to 20 times and disturb as much as 40 percent of the soil surface. In contrast, careful logging disturbs as little as 8 percent of the soil surface.

Careful selection of equipment and methods for transporting logs from the harvest area to areas where logs are gathered can significantly reduce the amount of soil disturbed and delivered to water bodies. Stream channels should be protected from logging debris at all times during harvesting operations.

Managing Replanting

Forests can be regenerated from either seed or seedlings. Seeding usually requires that the soil surface be prepared before planting. Seedlings can be directly planted with machines after minimal soil preparation. In either case, the use of heavy machinery, if not performed carefully, can result in significant soil disturbance.

5.3 Recommendations for Stormwater Management in Urban Areas

This section will focus on the stormwater management practices that are most suited to urban areas, which cover a large part of the study area. The following descriptions reference the USEPA “National Menu of Stormwater Best Management Practices”, which is very comprehensive and constantly updated (it contains close to 150 fact sheets on individual approaches, from education to implementation). Typical BMPS at the site level are shown below, distinguished by whether they apply best to new developments or as retrofits to existing developments.

5.3.1 Recommendations Based on Density of Development and New Vs Retrofit Application

Table 4.1 below shows the various approaches that are recommended depending on whether a site is located within a high density or low density development, as well as whether the project involves a new development or a retrofit application.

Table 5-2: Recommendations Based on Density of Development and New Vs Retrofit Application

Recommendations for Stormwater Management in Low-Density Urban Areas	
New Development	Retrofit Applications
Grassed swales;	Curb and gutter elimination;
Infiltration trenches;	Permeable pavement;
Permeable pavement;	Sand and organic filters;
Riparian buffers;	Soil amendments;
Sand and organic filters;	Vegetated filter strips; and
Soil amendments; and	Rain barrels and cisterns.
Vegetated filter strips.	
Recommendations for Stormwater Management in High-Density Urban Areas	
New Developments	Retrofit Applications
Bioretention cells;	Inlet protection devices;
Green parking design;	Permeable pavement;
Infiltration trenches;	Permeable pavers;
Inlet protection devices;	Rain barrels and cisterns;
Permeable pavement;	Sand and organic filters;
Permeable pavers;	Soil amendments;
Rain barrels and cisterns;	Stormwater planters; and
Sand and organic filters;	Tree box filter.
Soil amendments;	
Stormwater planters;	
Tree box filters;	
Vegetated filter strips; and	
Vegetated roofs.	

Each of these measures is described in detail below.

5.3.2 Description of Each Recommended Site Stormwater BMP Measure

Grassed Swales

Grassed swales are shallow grass-covered hydraulic conveyance channels that help to slow runoff and facilitate infiltration. The suitability of grassed swales depends on land use, soil type, imperviousness of the contributing watershed, and dimensions and slope of the grassed swale system. In general, grassed swales can be used to manage runoff from drainage areas that are less than 4 hectares (10 acres) in size, with slopes no greater than 5 percent. Use of natural, low-lying areas is encouraged and natural drainage courses should be preserved and utilized.



Infiltration Trenches

Infiltration trenches are rock-filled ditches with no outlets. These trenches collect runoff during a storm event and release it into the soil by infiltration (the process through which stormwater runoff penetrates into soil from the ground surface). Infiltration trenches may be used in conjunction with another stormwater management device, such as a grassed swale, to provide both water quality control and peak flow attenuation. Runoff that contains high levels of sediments or hydrocarbons (for example, oil and grease) that may clog the trench are often pretreated with other techniques such as water quality inlets (a series of chambers that promote sedimentation of coarse materials and separation of free oil from storm water), inlet protection devices, grassed swales, and vegetated filter strips.

Permeable Pavement

As an alternative to asphalt or concrete surfaces, permeable pavement allows stormwater to drain through the porous surface to a stone reservoir underneath. The reservoir temporarily stores surface runoff and allows it to infiltrate into the subsoil. The appearance of the alternative surface is often similar to asphalt or concrete, but it is manufactured without fine materials and instead incorporates void spaces that allow for storage and infiltration. Underdrains may also be used below the stone reservoir if soil conditions are not conducive to complete infiltration of runoff.



Riparian Buffers

A riparian, or forested, buffer is an area along a shoreline, wetland, or stream where development is restricted or prohibited. The primary function of aquatic buffers is to physically protect and separate a stream, lake, or wetland from future disturbance or encroachment. If properly designed, a buffer can provide stormwater management and can act as a right-of-way or floodplain during floods, sustaining the integrity of stream ecosystems and habitats.

Sand and Organic Filters

Sand and organic filters direct stormwater runoff through a sand bed to remove floatables, particulate metals, and pollutants. Sand and organic filters provide water quality treatment, reducing sediment, biochemical oxygen demand (BOD), and fecal coliform bacteria, although dissolved metal and nutrient removal through sand filters is often low. Sand and organic filters are typically used as a component of a

treatment train to remove pollution from stormwater before discharge to receiving waters, to groundwater, or for collection and reuse. Variations on the traditional surface sand filter (such as the underground sand filter, perimeter sand filter, organic media filter, and multi-chamber treatment train) can be made to fit sand filters into more challenging design sites or to improve pollutant removal.

Soil Amendments

Soil amendments increase the soil's infiltration capacity and help reduce runoff from the site. They have the added benefit of changing physical, chemical, and biological characteristics so that the soils become more effective at maintaining water quality. Soil amendments, which include both soil conditioners and fertilizers, make the soil more suitable for the growth of plants and increase water retention capabilities. The use of soil amendments is conditional on their compatibility with existing vegetation, particularly native plants.



Vegetated Filter Strips

Filter strips are bands of dense vegetation planted downstream of a runoff source. The use of natural or engineered filter strips is limited to gently sloping areas where vegetative cover can be established and channelized flow is not likely to develop. Filter strips are well suited for treating runoff from roads and highways, roof downspouts, very small parking lots, and other small or linear impervious surfaces. They are also ideal components for the fringe of a stream buffer, or as pretreatment for a structural practice.

Curb and Gutter Elimination

Curbs and gutters transport flow as quickly as possible to a stormwater drain without allowing for infiltration or pollutant removal. Eliminating curbs and gutters can increase sheet flow and reduce runoff volumes. Sheet flow, the form runoff takes when it is uniformly dispersed across a surface, can be established and maintained in an area that does not naturally concentrate flow, such as parking lots. Maintaining sheet flow by eliminating curbs and gutters and directing runoff into vegetated swales or bioretention basins helps to prevent erosion and more closely replicate predevelopment hydraulic conditions. A level spreader, which is an outlet designed to convert concentrated runoff to sheet flow and disperse it uniformly across a slope, may also be incorporated to prevent erosion.

Bioretention Cells / Rain Gardens

A bioretention cell or rain garden is a depressed area with porous backfill (material used to refill an excavation) under a vegetated surface. These areas often have an underdrain to encourage filtration and infiltration, especially in clayey soils. Bioretention cells provide groundwater recharge, pollutant removal, and runoff detention. Bioretention cells are an effective solution in parking lots or urban areas where green space is limited.



Green Parking Design

Green parking refers to several techniques that, applied together, reduce the contribution of parking lots to total impervious cover. Green parking lot techniques include: setting maximums for the number of parking lots created; minimizing the dimensions of parking lot spaces; utilizing alternative / porous pavers in overflow parking areas; using bioretention areas to treat stormwater; encouraging shared parking; and providing economic incentives for structured parking.



Rain Barrels and Cisterns

Rain barrels and cisterns harvest rainwater for reuse. Rain barrels are placed outside a building at roof downspouts to store rooftop runoff for later reuse in lawn and garden watering. Cisterns store rainwater in significantly larger volumes in manufactured tanks or underground storage areas. Rainwater collected in cisterns may also be used in non-potable water applications such as toilet flushing. Both cisterns and rain barrels can be implemented without the use of pumping devices by relying on gravity flow instead. Rain barrels and cisterns are low-cost water conservation devices that reduce runoff volume and, for very small storm events, delay and reduce the peak runoff flow rates. Both rain barrels and cisterns can provide a source of chemically untreated “soft water” for gardens and compost, free of most sediment and dissolved salts.



Stormwater Planters

Stormwater planters are small landscaped stormwater treatment devices that can be placed above or below ground and can be designed as infiltration or filtering practices. Stormwater planters use soil infiltration and biogeochemical processes to decrease stormwater quantity and improve water quality, similar to rain gardens and green roofs but smaller in size—stormwater planters are typically a few square feet of surface area compared to hundreds or thousands of square feet for rain gardens and green roofs. Types of stormwater planters include contained planters, infiltration planters, and flow-through planters.



Tree Box Filters

Tree box filters are in-ground containers used to control runoff water quality and provide some detention capacity. Often premanufactured, tree box filters contain street trees, vegetation, and soil that help filter runoff before it enters a catch basin or is released from the site. Tree box filters can help meet a variety of stormwater management goals, satisfy regulatory requirements for new development, protect and restore streams, control combined sewer overflows (CSOs), retrofit existing urban areas, and protect reservoir watersheds. The compact size of tree box filters allows volume and water quality control to be



tailored to specific site characteristics. Tree box filters provide the added value of aesthetics while making efficient use of available land for stormwater management. Typical landscape plants (for example, shrubs, ornamental grasses, trees and flowers) are an integral part of the bioretention system. Ideally, plants should be selected that can withstand alternating inundation and drought conditions and that do not have invasive root systems, which may reduce the soil's filtering capacity.

Vegetated Roofs

Green roofs consist of an impermeable roof membrane overlaid with a lightweight planting mix with a high infiltration rate and vegetated with plants tolerant of heat, drought, and periodic inundations. In addition to reducing runoff volume and frequency and improving runoff water quality, a green roof can reduce the effects of atmospheric pollution, reduce energy costs, and create an attractive environment. They have reduced replacement and maintenance costs and longer life cycles compared to traditional roofs.



5.4 Summary of Recommendations and Proposed Targets

One of the most efficient ways to deal with flooding risks is to manage the issue of high runoff at its source. Municipal Plan policies could include statements such as "expansion of the built environment or forestry operations shall be managed according to Best Management Practices and the BMP documents referenced". At the implementation (regulatory) level, specific BMPs could be incorporated into language in the municipal regulations – such as requirements for a certain width of stream buffer or how the buffer would be calculated for example. Several examples are presented here as a general guide. In order not to be too prescriptive, the documents referenced focus more on the objectives rather than on the detailed approach to reaching them. Attaining the objectives will be made through a combination of various measures, both at the planning and site level. Since every site and every development is different, it is recommended that the policies and By-laws produced be generic enough to allow flexibility of method employed, while focusing on the ultimate objective of controlling runoff volumes and runoff water quality.

In this spirit, some good reference documents are presented, and a range of best management practices are described. Some standard objectives are proposed, which are not expected to be excessively stringent or expensive to implement. Costs can be best controlled with a clear watershed master plan and efficient site plans that blends stormwater management practices smoothly into the local site characteristics.

5.4.1 Planning Level

The Credit Valley Conservation, a community conservation area in Ontario, has published a comprehensive guide to implementing sustainable development through Low Impact Development and Stormwater Best Management Practices ("Low Impact Development Stormwater Management Planning and Design Guide", Version 1.0 – 2011). It is recommended that at the planning level, the main steps in the document mentioned above be followed, which refers to the development of:

- Watershed Master Plans, a Growth Plan and an Official Plan;

- Community / Subwatershed Secondary and Block Plans;
- Neighbourhood Draft Plan of Subdivision / Functional Servicing Plan, followed by the Registered Plan;
- Site Plan with site specific opportunities, while staying consistent with the direction of the higher level plans; and
- Site Permits and approvals.

5.4.2 Forested Areas

Nova Scotia has recently published a very good document in this specific area, which describes an approach to implementing sustainable forest management techniques. The document was produced for Crown Land, but it is applicable to any forested area in the province. The document indeed recommends that it be used for forest management in the province wherever possible.

The reference for this document is the following:

Nova Scotia's Code of Forest Practice
A Framework for the Implementation of Sustainable Forest Management
Guidelines for Crown Land
Nova Scotia Natural Resources
August 2012

It is recommended that forestry operations in the community of Falmouth shall be managed according to Forestry Best Management Practices as described in this document.

5.4.3 Low Density and High Density Urban Areas – New Developments

It is recommended that by-laws be implemented which require no increase through lot development of:

- Total Suspended Solids in surface runoff;
- Phosphorous in surface runoff;
- Nitrates and ammonia in surface runoff;
- Peak flows; and
- Total Runoff Volume

This is especially relevant for watersheds in Falmouth that drain to Lake Pesaquid. Areas on the northern end that drain to the Avon River, downstream of Highway 101, without impacting other developed areas may not need as stringent regulations for water quality. The most important control is the total runoff volume control, which inherently reduces downstream flows and restores pre-development hydrology. Most volume control systems (infiltration) will allow nutrients to be treated in the gravel and surrounding soil media naturally, without the need to additional measures.

In terms of cost, infiltration systems will be economically advantageous if accounted for initially during the planning phase. Overall costs will be lower to developers since additional land is not required to store stormwater, and in some cases, a stormwater piped system is not needed at all, as surface water can be managed on the surface. Care must still be taken to ensure overland flow during extreme events is safely drained in suitably sized systems (e.g. ditches and culverts). However, since those systems will

be draining smaller flows than without infiltration systems, they can be smaller and therefore less costly as well.

To help achieve these targets, it is recommended that the practices listed above and any others the community feels are relevant be encouraged through education and incentives.

5.4.4 Low Density and High Density Urban Areas – Retrofit Applications

For development of previously developed sites, it is probably quite challenging to re-establish pre-development level aspects. For these areas, it is therefore recommended that by-laws be implemented which require a 25% reduction through on-lot development of:

- Total Suspended Solids in surface runoff;
- Phosphorous in surface runoff (not applicable if only hard surface);
- Nitrates and ammonia in surface runoff (not applicable if only hard surface);
- Peak flows; and
- Total Runoff Volume.

These would apply to the surface area being re-developed and not the entire site. For areas that already include stormwater management features that work very efficiently, the percentage reduction requirement can be decreased if the developer can show that this would bring runoff volume to a lower level than the runoff volume of the original natural treed site.

Similarly to new development, areas that do not drain to Lake Pesaquid or to other developed areas may not need as stringent water quality regulations. The most important control is the total runoff volume control, which inherently reduces downstream flows and restores pre-development hydrology. Most volume control systems (infiltration) will allow nutrients to be treated in the gravel and surrounding soil media naturally, without the need to additional measures.

To help achieve these targets, it is recommended that the practices listed above and any others the community feels are relevant be encouraged through education and incentives.