



DILLON
CONSULTING

MUNICIPALITY OF THE DISTRICT OF WEST HANTS
**Three Mile Plains Flood Risk
Assessment Study**



May 17, 2019



Hantsport Municipal Public Works Office
20 Main Street
Hantsport, NS
B0N 2T0

***Municipality of the District of West Hants
Three Mile Plains Flood Risk Assessment Study***

Attention: Mr. Brad Carrigan, Director of Public Works

Dear Mr. Carrigan,

Dillon Consulting Limited (Dillon) is pleased to submit the following final report describing the findings of the flood risk assessment completed for the community of Three Mile Plains in the Municipality of the District of West Hants.

The attached report provides the methodology and hydrologic and hydraulic simulation results for the drainage network in Three Mile Plains. The report assesses the current drainage network under existing and future development conditions. The effects of climate change have also been investigated as part of the study. Preliminary recommendations have been developed for areas of concern with respect to the need for stormwater conveyance improvements. Acknowledging that many areas of concern are related to provincial infrastructure, recommendations also include improvements that can be implemented on municipal property such as low impact design (LID) elements.

Sincerely,

DILLON CONSULTING LIMITED

A handwritten signature in blue ink that reads "S. Devereaux".

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Executive Summary

The Municipality of West Hants (the Municipality) retained Dillon Consulting Limited (Dillon) to undertake a Flood Risk Assessment Study within the Community of Three Mile Plains (Community). Three Mile Plains is a predominantly rural community near the Town of Windsor at Exit 5 on Highway 101 in West Hants, Nova Scotia.

The purpose of this study was to assess the performance of the existing drainage system under existing and projected future development conditions. Based on these assessments, a set of conceptual drainage improvement options were developed with the following objectives:

- Protect citizens from flooding dangers (loss of life, loss of property, etc.);
- Assess areas of planned development for flood risk;
- Mitigate damage to public infrastructure;
- Provide recommendations to improve drainage conditions under existing and future development conditions; and
- Use low impact development (LID) and best management practices (BMP) stormwater management techniques to improve groundwater and surface runoff quality, and in turn, improve the quality of aquatic habitat in the area.

Drainage infrastructure within the Community consists of a combination of ditches, culverts and bridge crossings, natural channels, and numerous overland drainage pathways. Three Mile Plains is rural in nature and does not have a formal storm sewer system. Typical roadway cross sections consist of a paved asphalt surface with a roadside ditch to one side.

Hydrologic/hydraulic modelling was undertaken to simulate the performance of current drainage network. The numerical simulation was completed using the latest version (2017 Professional) of the Computational Hydraulic Institutes (CHI) PCSWMM modelling software. The PCSWMM model assessed the drainage network in Three Mile Plains for six 24-hour rainfall events with return periods ranging from 1:2 years to 1:100 years. These rainfall events were developed using historical rainfall data as well as future climate change adjustments assuming a moderate climate change severity.

The future land use model was developed based on the discussion with the Municipality in February 2019, acknowledging that the majority of development would be focused around Garland's Crossing and Denise Drive. Moderate development conditions were modeled for the 25-year build out which increased the developed land by 72.7 hectares (ha) equating to approximately 102 new private dwellings. Complete development conditions were based on a 50 year planning horizon which increased developed land by 151.1 ha (or 212 new private dwellings). The increase in development for moderate and complete development conditions were calculated using the historical rate within the Community from Statistics Canada Census Data.

A series of model simulations for both existing and future conditions were completed to assess the performance of the existing drainage works. The simulations included existing conditions with historical climate and climate change projections, moderate development conditions with climate change projections, and complete development conditions with climate change projections. Under existing conditions, these simulations identified three Lebreau Creek Brook crossings that are potentially vulnerable to flooding, including:

- Old Shanks Road Bridge;
- Mountain Road Bridge; and
- Panuke Road Bridge.

In addition to these crossings, areas potentially susceptible to ditches overtopping were also identified along the following roads:

- Alexander Drive;
- Denise Drive;
- Windsor Back Road;
- Highway 1; and
- Three Mile Plains Cross Road.

A set of conceptual drainage improvement options were prepared for the flood vulnerable areas. Recommendations also include regularly scheduled inspection of all crossings of the Lebreau Creek Brook including maintenance and removal of debris where necessary, with particular attention paid to the bridge at Old Shanks Road, the Mountain Road Bridge, and the Panuke Road Bridge.

It was noted in discussions with the Municipality that the majority of drainage infrastructure within the Community is provincially-owned by Transportation and Infrastructure Renewal (TIR). Acknowledging this challenge, a set of potential implementation strategies were outlined to guide the Municipality in initiating a response to the drainage issues identified in areas within the Community. Recommendations have also been provided to improve lot-level drainage; particularly in new developments. These implementation strategies as well as lot-level drainage recommendations are summarized below.

- Conduct engagement sessions with TIR and impacted stakeholders to discuss the findings of this study;
- Evaluation of federal and provincial funding programs to subsidize the cost of implementation;
- Collaboration with the Ecology Action Centre (EAC) for potential opportunities to partner on implementing drainage solutions; and,
- Introduction of prescriptive stormwater management for new developments with incentives for the use of BMPs and LIDs.

1.0 Introduction

The Municipality of the District of West Hants (Municipality) commissioned Dillon Consulting Limited (Dillon) to undertake a Flood Risk Assessment Study within the Community of Three Mile Plains (Community). This report describes the methodology and findings of the existing and projected future development conditions assessments.

The objectives of this study are as follows:

- Protect citizens from flooding dangers (loss of life, loss of property, etc.);
- Assess areas of planned development for flood risk;
- Mitigate damage to public infrastructure;
- Provide recommendations to improve drainage conditions under existing and future development conditions; and
- Use of low impact development (LID) and best management practices (BMP) stormwater management techniques to improve groundwater and surface runoff quality, and in turn improve the quality of aquatic habitat in the area.

2.0 Background

The Community of Three Mile Plains is a predominantly rural community located near the Town of Windsor in the vicinity of Exit 5 on Highway 101 in West Hants, Nova Scotia. The general location of Three Mile Plains and municipal boundaries are presented in **Figure 2-1**. While there is no history of significant flooding within the community of Three Mile Plains, several nearby communities do have a recent history of flooding.

The primary watercourse through the Community is Lebreau Creek Brook, which flows westerly from the community of Newport Station to its outlet at the Avon River in the community of Martock. Three Mile Plains does not have a formal storm sewer system. Drainage Infrastructure within the Community consists of natural channels, overland flow, roadside ditching, and culvert conveyance. A typical roadway cross section consists of a paved asphalt surface (crowned) with roadside ditching. A photograph of a representative roadway cross section is presented in **Figure 2-2**.

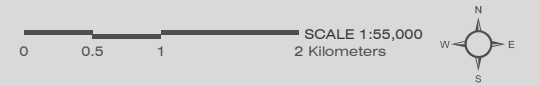


THREE MILE PLAINS
FLOOD RISK ASSESSMENT STUDY

COMMUNITY AND WATERSHED BOUNDARIES

FIGURE 2-1

- - - THREE MILE PLAINS COMMUNITY BOUNDARY
- _ _ _ LEBREAU CREEK BROOK WATERSHED
- - - TOWN OF WINDSOR BOUNDARY



MAP DRAWING INFORMATION:
DATA PROVIDED BY WEST HANTS AND ESRI

MAP CREATED BY: JGC
MAP CHECKED BY: JAM
MAP PROJECTION: NAD 1983 UTM Zone 20N

FILE LOCATION: \\DILLON.CA\DILLON_DFS\LONDON\LONDON CAD\GIS\VISUAL COMMUNICATIONS DIMXD TEMPLATES\BEIGE - 11X17 LANDSCAPE - LEGEND RIGHT.MXD



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DATE: 04/16/19

To address flood risk within the Community and to identify mitigation options to improve drainage, an assessment of existing stormwater drainage system has been undertaken. This assessment identifies drainage routes within the study area and identifies hydraulic constrictions (i.e. bottlenecks) where overtopping of ditch/channel banks occur. Numerical simulation of design rainfall events has been undertaken to facilitate this assessment. It should be noted that the majority of development within the community has occurred, and is proposed to occur, north of Lebreau Creek Brook; therefore, this study has placed greater emphasis on these areas.



Figure 2-2: Typical roadway cross section with ditching in Three Mile Plains

3.0 Existing Drainage Network

The Three Mile Plains area of the Lebreau Creek Brook watershed comprises primarily the areas of Highway 1 between Green Street and Hatfield Lane, Three Mile Plains Cross Road, Windsor Back Road, Mountain Road, Panuke Road, and the surrounding area, as bounded in red on **Figure 3-1**. Drainage is conveyed primarily via overland flow, roadside ditching, and natural channels. These conveyance pathways are directed to Lebreau Creek Brook and ultimately discharge into the Avon River (Pesaquid Lake). To better understand these systems, several site visits took place between fall 2018 and spring 2019. These site visits are described below.

3.1 Site Reconnaissance and Survey

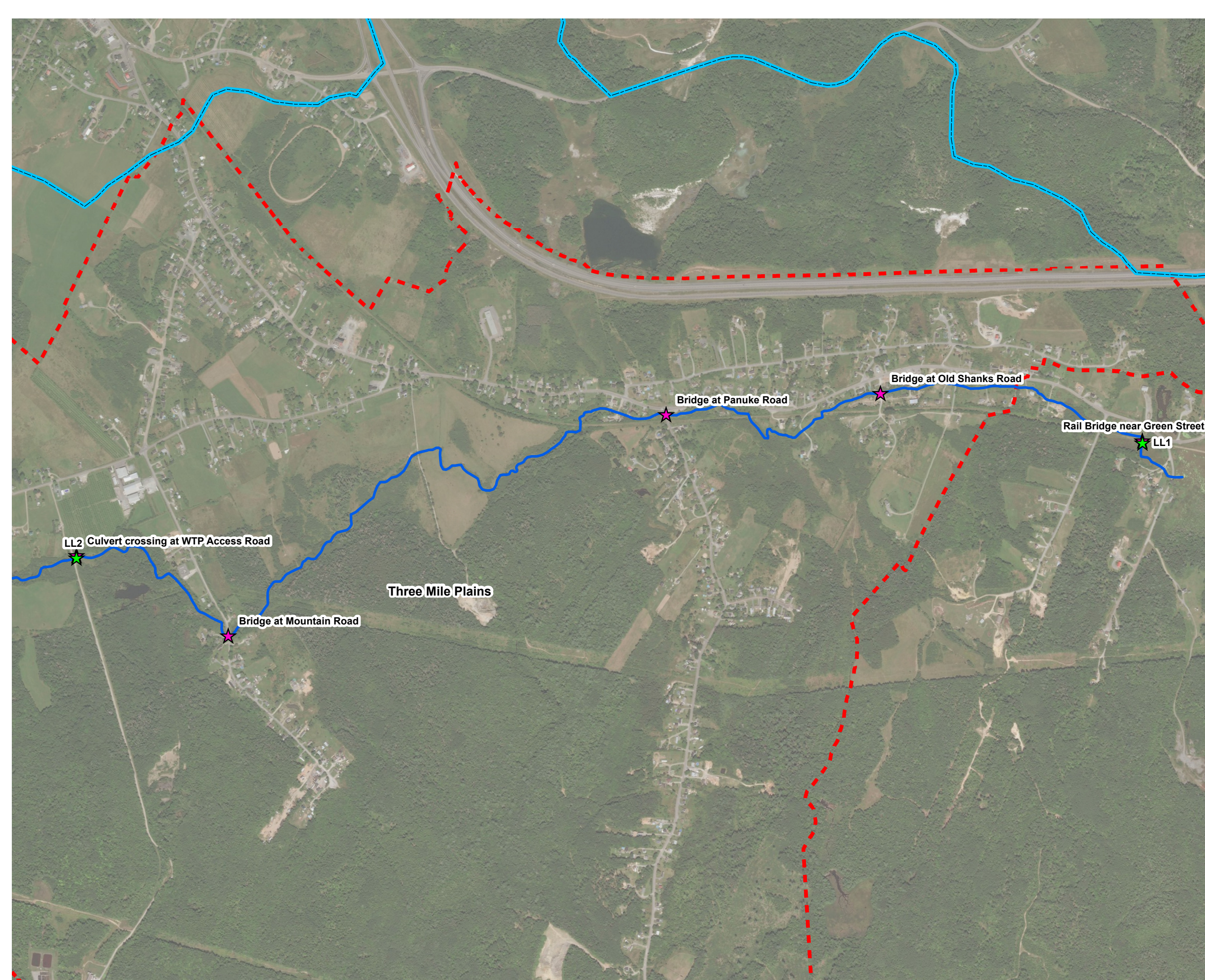
To understand the existing level of flood risk within the Community, a series of site visits were completed in fall 2018 and spring 2019. These visits included topographical survey, visual inspection of drainage infrastructure, and water level monitoring. These observations were also used to support the

development of the hydrologic/hydraulic model (described in subsequent sections of this report). A summary of each site visit is provided in the sections below.

3.1.1 October 12 and 15, 2018






The first site visit took place on October 12 to install two water level loggers and a barometric pressure logger to monitor water levels in Lebreau Creek Brook. Due to elevated water levels, only one water level logger and the barometric pressure logger were installed on October 12. The second water level logger was subsequently installed on October 15 when water levels subsided.

The purpose of the level logger deployments was to record a period of water level fluctuations in the Lebreau Creek Brook watershed to assist in hydrologic/hydraulic model calibration. Water level logger 1 (LL1) was installed along Lebreau Creek Brook upstream of the rail bridge crossing off Pleasant Street (see **Figure 3-2**). The pressure logger was installed nearby (in the dry) to record variation in atmospheric pressure. Water level logger 2 (LL2) was installed along Lebreau Creek Brook upstream of the Water Treatment Plant (WTP) access road crossing (see **Figure 3-3**).



THREE MILE PLAINS
FLOOD RISK ASSESSMENT STUDY

**WATER LEVEL LOGGER
AND SURVEY LOCATIONS**
FIGURE 3-1

-  THREE MILE PLAINS COMMUNITY BOUNDARY
-  LEBREAU CREEK BROOK WATERSHED
-  LEBREAU CREEK BROOK
-  LEVEL LOGGER INSTALLATION LOCATION
-  SURVEY LOCATION



MAP DRAWING INFORMATION:
DATA PROVIDED BY WEST HANTS, CANVEC, GEONOVA, AND ESRI

MAP CREATED BY: HML
MAP CHECKED BY: JAM
MAP PROJECTION: NAD 1983 UTM Zone 20N

FILE LOCATION: \\DILLON.CA\DILLON_DFS\LONDON\LONDON CAD\GIS\VISUAL COMMUNICATIONS D\IMXD TEMPLATES\BEIGE - 11X17 LANDSCAPE - LEGEND RIGHT.MXD



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Figure 3-2: Surveying the water level logger



Figure 3-3: Water Level Logger 2 (LL2) installation

3.1.2 November 22, 2018

The site visit undertaken on November 22 involved removal and data offloading of the water level loggers and barologger. A survey of select drainage features along the Lebreau Creek Brook was also undertaken at this time.

This survey focused on major bridges and culvert crossings on the Lebreau Creek Brook to investigate structural condition as well as obtain estimates of channel width, channel depth, and structure size. The invert and roadway elevations at each crossing were also measured. The locations of these crossings are shown in **Figure 3-1**. Based on this survey it was noted that several crossings along Lebreau Creek Brook have significant buildup of debris including leaves and branches (see examples in **Figure 3-4** and **Figure 3-5**). These blockages would contribute to elevated water levels which could be consequential during high flow conditions. Regular cleaning and maintenance would reduce this risk and is recommended.

Significant snow cover was present during the November 22 site visit. This made identification and survey of minor/major drainage works (ditches/culverts) within the community difficult; however, a representative ditch cross-section survey was conducted. A subsequent site visit was completed in April 2019 after the snow melt to identify and document these drainage works.



Figure 3-4: WTP access road culvert (1 of 4) debris buildup – looking upstream from downstream end (October 15, 2018)



Figure 3-5: Panuke Road Bridge debris buildup – looking downstream from upstream end (November 22, 2018)

3.1.3

April 12, 2019

The site visit on April 12, 2019, involved confirmation of drainage pathways as well as measurement of cross culvert sizes and ditch sizes within the Three Mile Plains community. As previously discussed, snow cover made it difficult to identify drainage pathways and crossing structures during the site visit conducted in November 2018; therefore, it was necessary to complete this site visit following snow melt.

This site visit focused on identifying these drainage pathways including roadside ditches and smaller natural channels that convey runoff from the Three Mile Plains area to Lebreau Creek Brook. Major culvert crossings along Highway 1 and Windsor Back Road were also measured for input to the model. A total of six culvert crossings were identified along these two roads. Figures 3-6 and 3-7 show the box culvert crossings on Highway 1 near the railway crossing and Civic #4627, respectively.



Figure 3-6: Downstream Side of Box Culvert Crossing at Civic #4627 on Highway 1

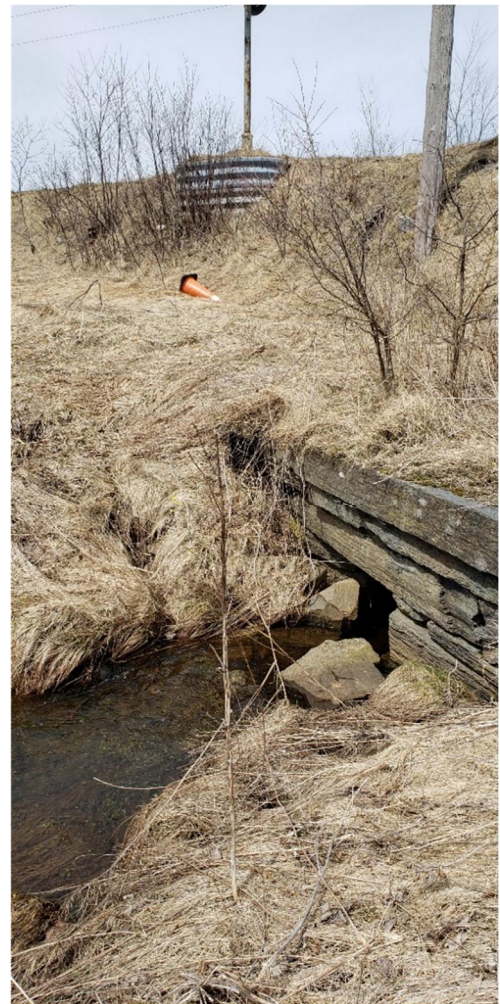


Figure 3-7: Downstream Side of Box Culvert Crossing at Highway 1 Railway Crossing

3.2 Hydrometric Observations

3.2.1 Upper Lebreau Creek Brook (October 12 – November 22, 2018) – LL1

Water level readings were collected at the LL1 monitoring stations between October 12, 2018, and November 22, 2018 (43 days). The water level data was then compensated for changes in barometric pressure.

A summary of water level observations and daily rainfall records from the Environment Canada Kentville climate station (#8202810) are presented in **Figure 3-8**. The largest rainfall event occurred on November 2, 2018, with 40 mm of daily rainfall reported. While more significant rainfall events (total precipitation of 50+ mm) would provide a stronger indication of rainfall-runoff response, this event was the highest observed daily rainfall accumulation during the monitoring period. This rainfall data was used to support subsequent model calibration.

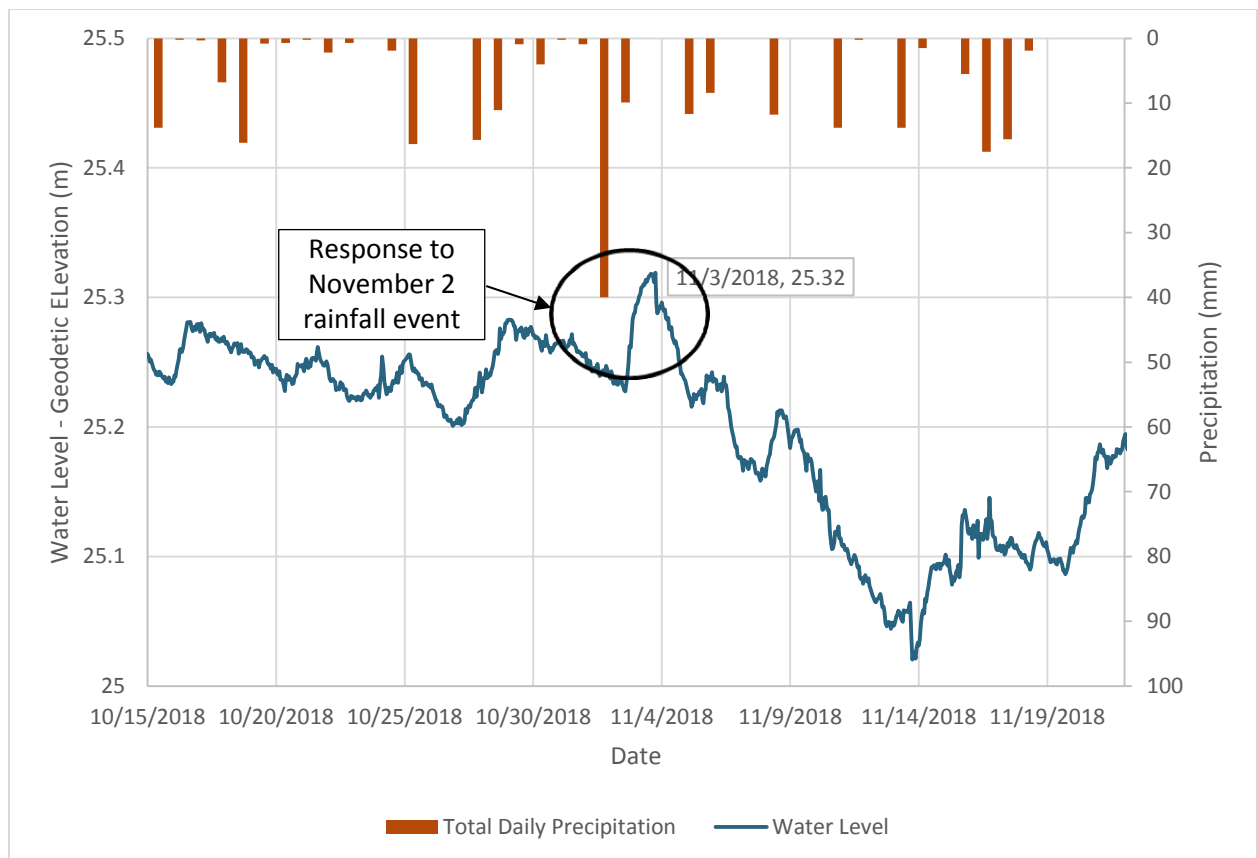


Figure 3-8: LL1 – Upper Lebreau Creek Brook Water Level Observations (Geodetic Elevation)

3.2.2 Lower Lebreau Creek Brook (October 15 – November 22, 2018) – LL2

As noted previously, the lower Lebreau Creek Brook water level logger (LL2) was deployed on October 15 and removed on November 22; an observation period of 39 days. The lower portion of the Lebreau Creek Brook receives runoff from a substantial portion of the Three Mile Plains study area and therefore provides a suitable location for model calibration.

A summary of water level observations and rainfall records from the Environment Canada Kentville climate station (#8202810) are presented in **Figure 3-9**. An increase in water level can be observed in response to the 40 mm rainfall event on November 2, 2018.

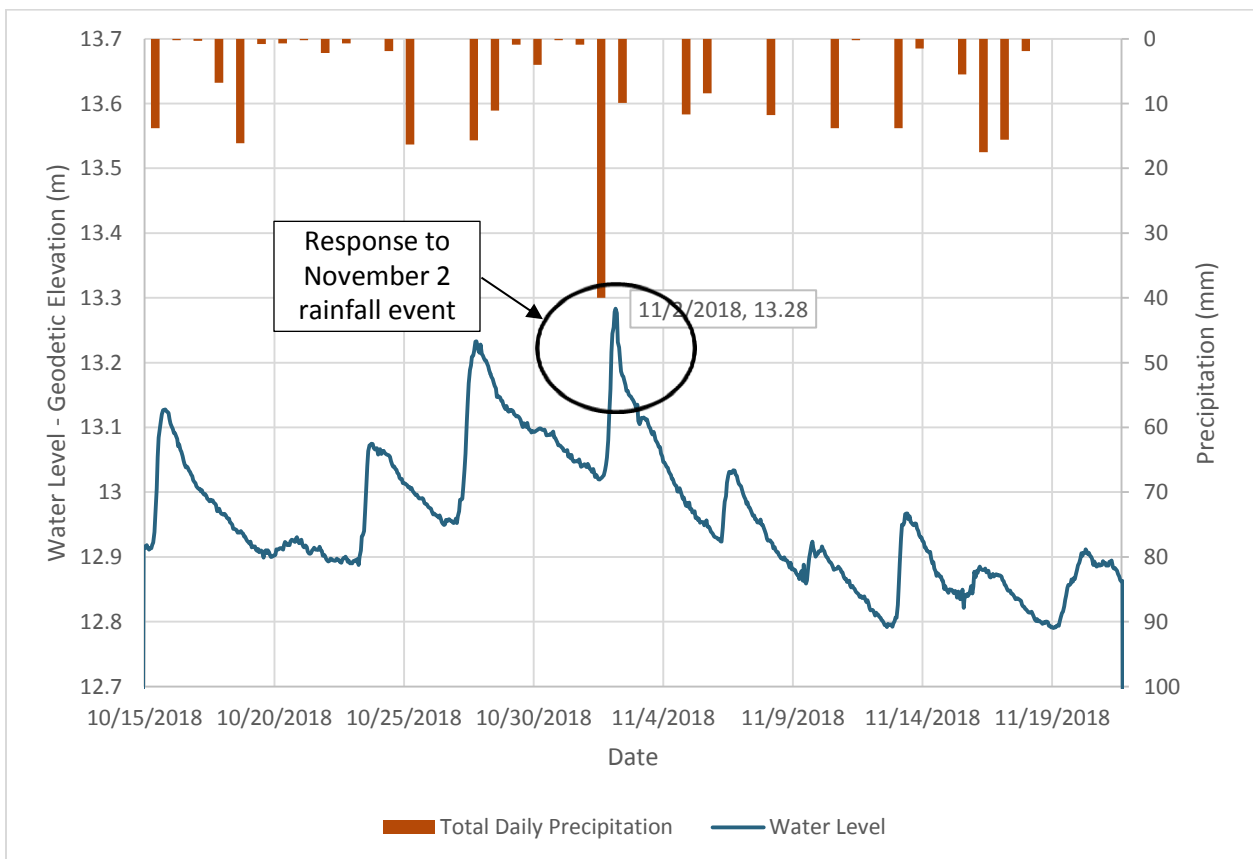


Figure 3-9: LL2 – Lower Lebreau Creek Brook Water Level Observations (Geodetic Elevation)

3.2.3 Flood Frequency Analysis

A review of regional hydrometric gauges along the west coast of Nova Scotia was undertaken to generate a regional flood frequency analysis for the study region. This exercise involved using the HYFRAN statistical software to perform a frequency analysis of six hydrometric stations in Nova Scotia. The stations used for the frequency analysis are summarized in **Table 3-1**, and were chosen due to their proximity to the study area and the period of record. The instantaneous annual peak water level data

for each station was used for this analysis. Only stations with greater than 20 years of data were used to support the regional frequency analysis discussed below.

Table 3-1: Regional Hydrometric Station Summary

Station Name	Station ID	Period of Record	Years of Complete Data	Drainage Area (km ²)
Kelley River at Eight Mile Ford	01DL001	1969 - 2015	46	63.2
Great Village River near Scrabble Hill	01DJ005	1993 - 2017	24	89.0
Sharpe Brook at Lloyds	01DD004	1966 - 1995	30	8.81
Fraser Brook near Archibald	01DH003	1965 - 1991	27	10.1
North Brook at North River	01DD004	1973 - 1995	20	202
Salmon River at Union	01DH005	1977 - 1999	23	287

Based on the frequency analysis of the six regional stations with greater than 20 years of observations, a log-log graph was plotted of drainage area against peak flow for each return period flood events (i.e. 2-year, 5-year, etc.). A trend line was fitted to each return period event to obtain a relationship between peak flow and drainage area for the west coast of Nova Scotia. Using these relationships, the estimated peak flow for each return period flood may be estimated for ungauged watersheds of various sizes within the west coast of Nova Scotia. The log-log graph of the six regional stations is presented in **Figure 3-10**.

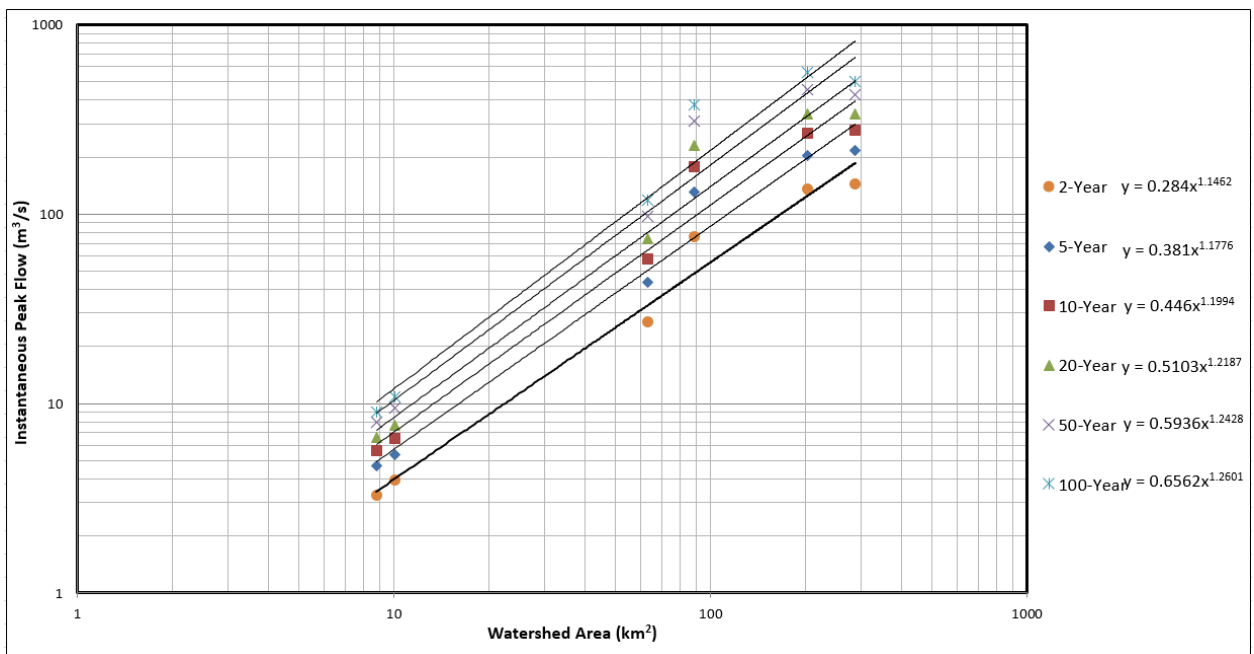


Figure 3-10: Regional Frequency Analysis Theoretical Peak Flow Plot

Table 3-2 presents a summary of the estimated instantaneous peak flow values for Lebreau Creek Brook using regional frequency analysis approach. The total drainage area for the Lebreau Creek Brook is approximately 54.8 km². The estimated flows in **Table 3-2** provide additional flow estimates to base model calibration and validation.

Table 3-2: Regional Flood Frequency Instantaneous Peak Flow Estimates for Lebreau Creek Brook

Return Period (Years)	Regional Flood Frequency Results for Lebreau Creek Brook (m ³ /s)
100	101.9
50	86.0
20	67.1
10	54.3
5	42.5
2	27.9

4.0 Hydrologic and Hydraulic Analysis

Hydrologic/hydraulic modelling was undertaken to simulate the performance of the drainage system within the study area under both existing and projected future conditions. The numerical simulation was completed using the latest version (2017 Professional) of the Computational Hydraulic Institutes (CHI) PCSWMM modelling software. This model was selected based on its integrated hydrologic and hydraulic computational abilities.

The following sections describe the model inputs and set up required to complete the numerical simulation.

4.1 Hydrologic Characteristics

The following sections describe the key hydrological characteristics of the numerical model.

4.1.1 Catchment Parameters

4.1.1.1 Catchment Delineation

Catchment delineation was initially completed using the existing LiDAR data (obtained from GeoNova and West Hants) for the Three Mile Plains study area. The initial coarse sub-catchment delineation was further refined based on the observations made during the site visits and additional high-resolution LiDAR purchased from the Centre of Geographic Sciences (COGS). The basin was sub-divided into 33 sub-catchments as shown in **Figure 4-1**. Each sub-catchment was given a unique identifier for the PCSWMM model.

4.1.1.2

Rainfall-Runoff Parameters

Runoff from the catchment that can directly contribute to flooding occurs when the rate of rainfall far exceeds the capacity of the ground to absorb the water. The magnitude of runoff that may lead to flooding depends on the intensity and duration of the rainfall, the areal extent of the storm, antecedent (pre-storm) ground moisture conditions, and drainage basin characteristics (e.g. topography, slope, soils, vegetation, and land cover type).

The U.S. Soil Conservation Service (SCS) Unit Hydrograph transformation method was adopted for the numerical simulation. Using the curve number, catchment lag is estimated as a function of the average basin slope, flow length, and the SCS Curve Number (CN) value. The digital elevation model derived from existing LiDAR data was used to estimate the average basin slope and flow length for each sub-catchment. The CN value for each sub-catchment was estimated based on the dominant land cover type/surficial geology (OAS, 1984), and published CN values according to Ponce (1989).

Based on provincial surficial geologic mapping, the Three Mile Plains watershed soil has been classified as Hydrologic Soil Group B.

Losses of precipitation due to infiltration, depression storage and interception in the sub-catchments were incorporated into the model through Runoff Curve Number (CN) values. A summary of the runoff parameters assigned to each sub-catchment is provided in **Table A-1** in **Appendix A**.

THREE MILE PLAINS
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SUBCATCHMENT DELINEATIONS
FIGURE 4-1

-  SUBCATCHMENT
-  THREE MILE PLAINS COMMUNITY BOUNDARY
-  CONTOUR 10m



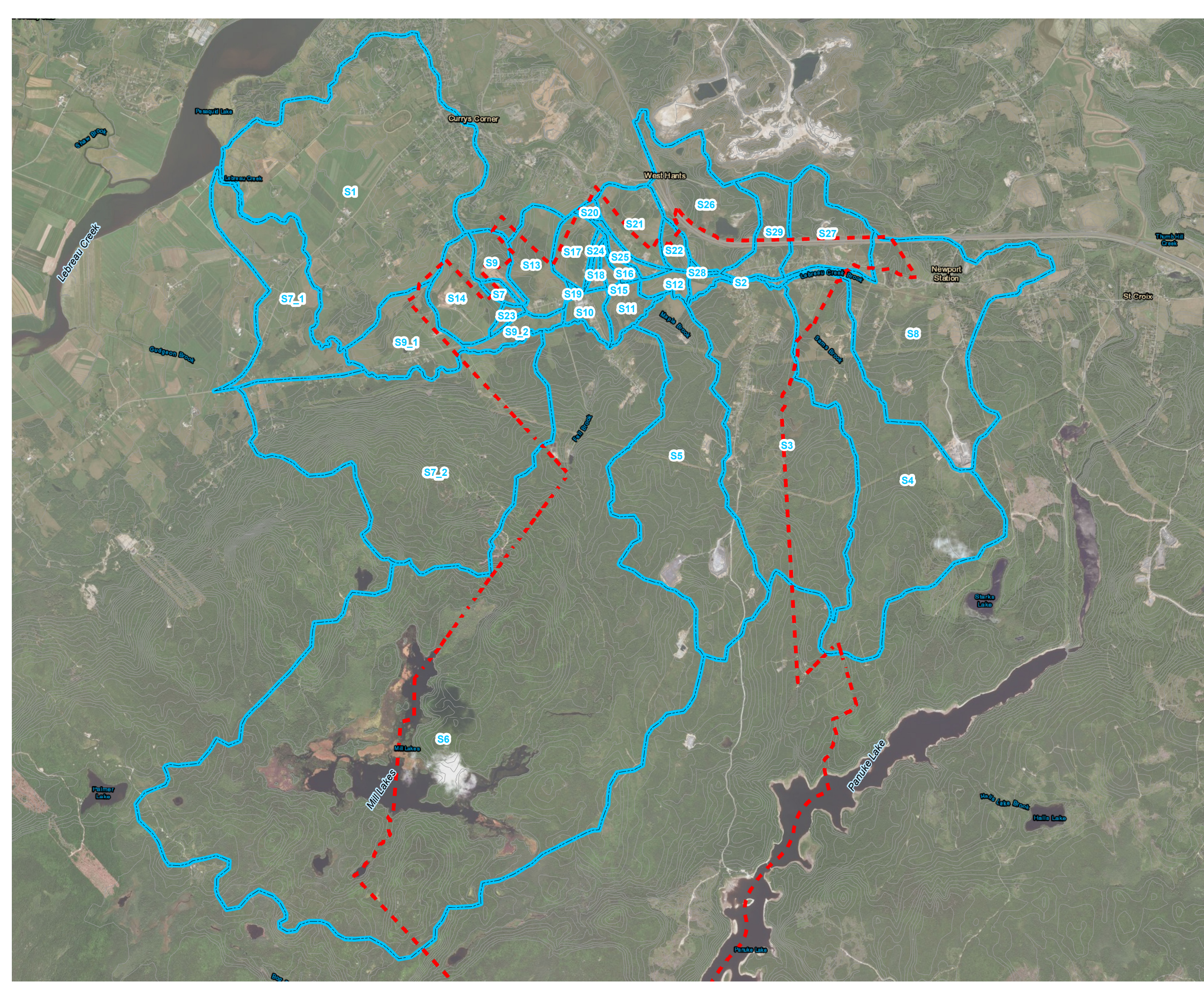
MAP DRAWING INFORMATION:
DATA PROVIDED BY WEST HANTS, GEONOVA, AND ESRI

MAP CREATED BY: JGC
MAP CHECKED BY: JAM
MAP PROJECTION: NAD 1983 UTM Zone 20N

FILE LOCATION: \\DILLON.CAD\DILLON_DFS\LONDON\LONDON CAD\GIS\VISUAL COMMUNICATIONS DIMXD TEMPLATES\BEIGE - 11X17 LANDSCAPE - LEGEND RIGHT.MXD



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4.1.2 Meteorological Inputs

Rainfall data for the Environment Canada’s Kentville Climate Station (#8202800) was used to support this study. The Kentville station is approximately 35 km from Three Mile Plains with about 32 years (1960 – 1995) of continuous climate observations, including precipitation. Historic rainfall intensity-duration-frequency (IDF) statistics for the historic Kentville Environment Canada climate station (#8202800) were used to estimate historical rainfall depths for a range of 24-hour rainfall events. These historical rainfall depths are presented in **Table 4-1**.

Climate change impacts were also considered in this assessment. The Canadian Water Network’s IDF Climate Change Computerized Tool (<https://www.idf-cc-uwo.ca>) has been used to estimate future rainfall depths. A representative concentration pathway of 4.5 (RCP 4.5) has been used for climate change adjustments. RCP 4.5 assumes radiative forcing peaks at 4.5 W/m² by the year 2100 resulting in a moderate climate change severity. A comparison of historical and future rainfall for the Kentville climate station is also presented in **Table 4-1**.

Table 4-1: Summary of Historical and Estimated Future Rainfall Depth for 24-hour Rainfall Events at Kentville, NS

Return Period (Years)	Historical Rainfall Depth (mm)	Estimated Future (2020 – 2070) Climate Change Rainfall Depth (mm) ¹	Percent Difference
2	56.2	64.1	+14%
5	74.2	87.3	+18%
10	86.3	103.9	+20%
25	101.7	126.3	+24%
50	113.3	144.7	+28%
100	124.9	162.7	+30%

¹Assumes Moderate Emissions Scenario – RCP 4.5

It can be seen that for a range of return periods with a storm duration of 24-hours, an increase of roughly 14 – 30% is expected under estimated future conditions.

The estimated future total rainfall depths were distributed using the SCS Type II distribution. The SCS Type II distribution allows for the development of a design hyetograph from regional IDF statistics. The rainfall time series generated includes rainfall accumulation for 6-minute intervals, up to the 24-hour storm duration. An example of this distribution for the 100-year future rainfall event is presented in **Figure 4-2**.

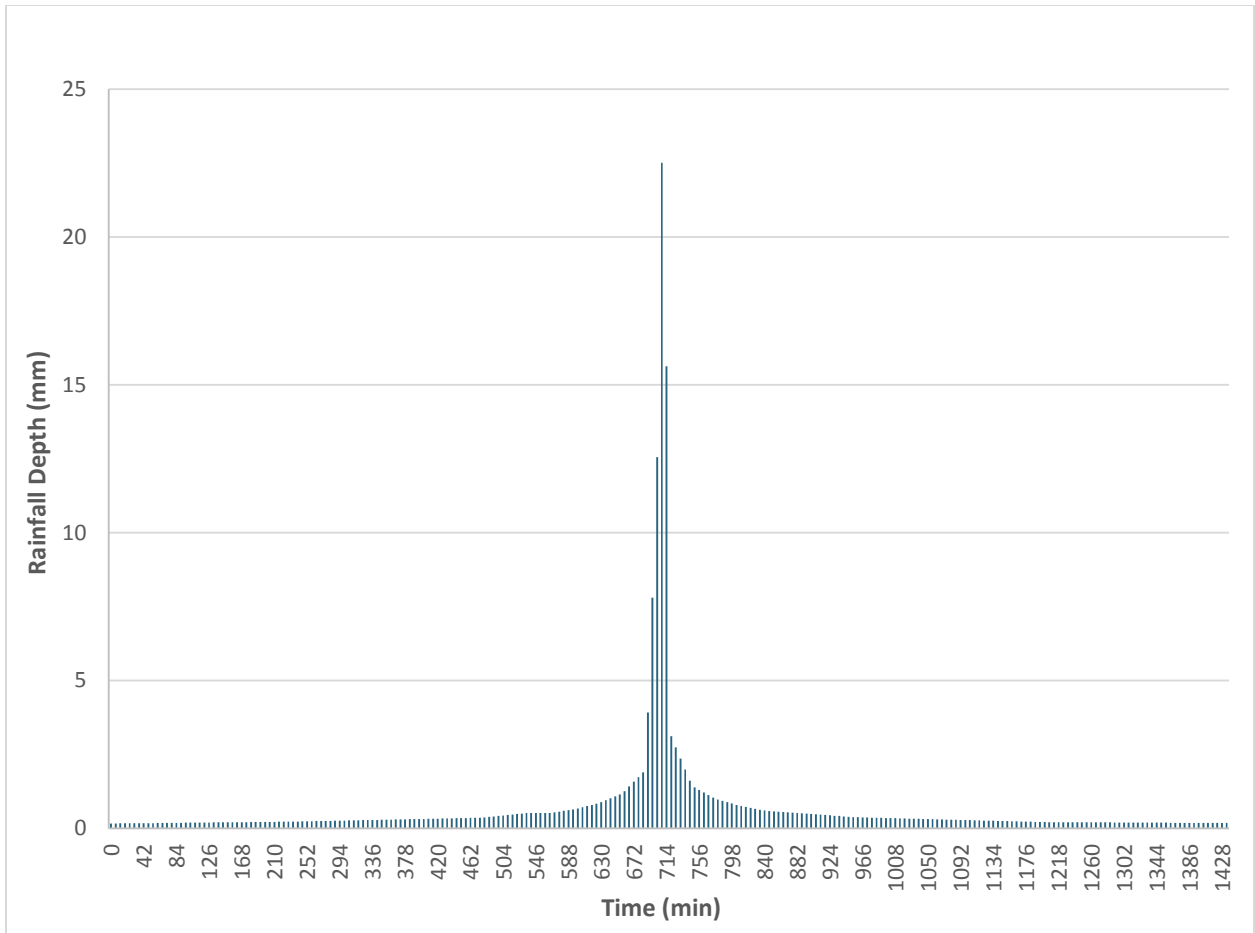


Figure 4-2: 100-Year 24-Hour Rainfall Event with Climate Change Projections distributed using the SCS Type II Distribution (6-minute Interval)

4.2 Hydraulic Model Development

To simulate drainage within the study area under existing conditions, hydrologic/hydraulic modelling was undertaken to simulate the performance of the existing drainage network. The numerical simulation was completed using the latest version (2017 Professional) of the Computational Hydraulic Institute's (CHI) PCSWMM modelling software. This model was selected based on its integrated hydrologic and hydraulic computational abilities. This allows for the dynamic simultaneous simulation of the rainfall-runoff relationship for complex drainage networks. The PCSWMM model was used to assess the existing drainage network in Three Mile Plains with six (6) rainfall events.

The drainage network observed during the site visits in October and November was used to develop the model in PCSWMM for analysis. The drainage network in the Community of Three Mile Plains consists primarily of Lebreau Creek Brook, culverts and open ditches, and tributary watercourses. For the purposes of this analysis the PCSWMM model assumed the drainage network to be operating under

ideal conditions (i.e. culverts have been cleaned). A visual overview of the Community's drainage network is presented in **Figure 4-3**.

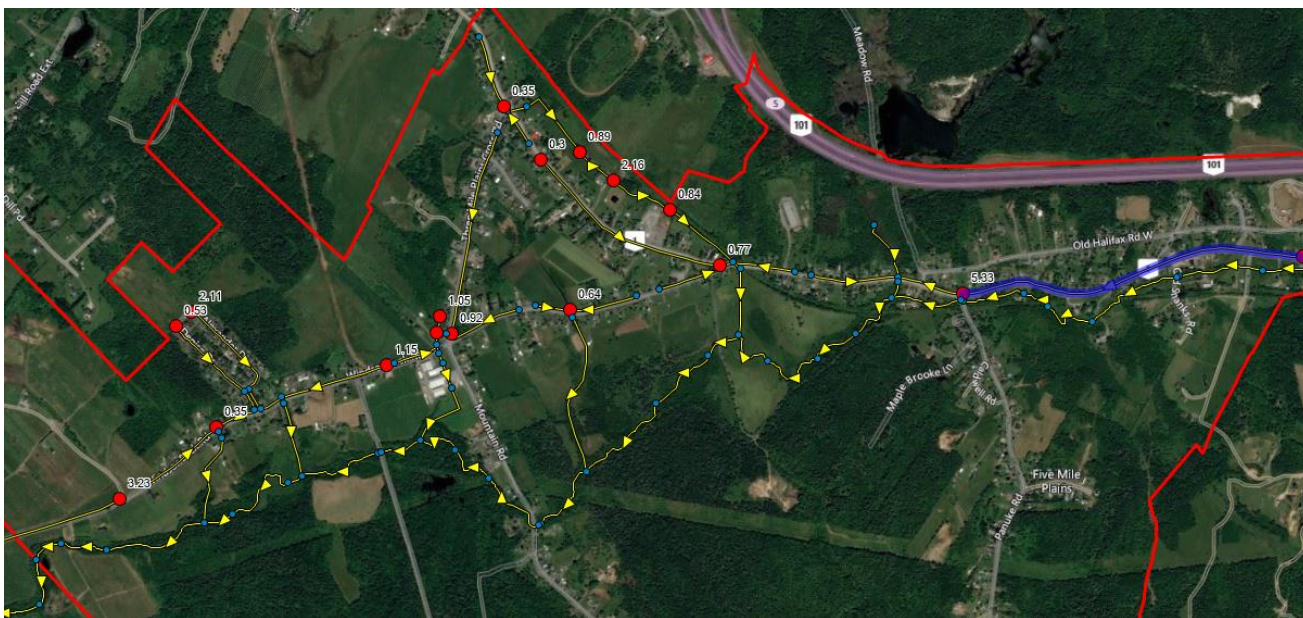


Figure 4-3: Three Mile Plains Drainage Network Overview

A targeted survey of the network was completed during the site visits carried out between October 12 and November 22, 2018 (see **Section 3.1**). These surveys captured the necessary components for model development; however, several assumptions were necessary to complete the model. These assumptions are summarized below:

- Watercourse cross sections that weren't collected by high resolution GPS have been estimated from LiDAR where available; and
- Watercourse and ditch centerlines that weren't collected by high resolution GPS or estimated from LiDAR have been estimated from aerial imagery.

4.3 Model Calibration

The most significant rainfall event during the hydrometric observation period took place on November 2, 2018, with a total of 40 mm of precipitation. Environment Canada provided 15-interval rainfall data at the Kentville Climate Station (#8202810) between August and December 2018. This rainfall event was simulated to support model calibration.

4.3.1 Model Calibration

Following the sensitivity analysis, model calibration was undertaken to match the observed water levels from the November 2, 2018, calibration. Model calibration was completed by making adjustments to hydrologic and hydraulic parameters in the model. These changes are described below:

- CN values of 55 and 69 were assigned to wooded and cultivated areas, respectively (Hydrologic Soil Group B);
- Percent impervious was increased by 10%;
- Pervious overland roughness (Manning's) values of 0.4 and 0.04 were assigned to wooded and cultivated areas, respectively;
- Manning's roughness value of 0.025 was assigned to the Lebreau Creek Brook channel;
- Manning's roughness value of 0.1 was assigned to wooded floodplain areas along Lebreau Creek Brook; and
- Manning's roughness value of 0.03 was assigned to agricultural/short grassed floodplain areas along Lebreau Creek Brook.

In addition to the changes above, the PCSWMM model was updated to account for the observed baseline discharge during the November 2 calibration rainfall event. An additional 10 m³/s was input at the mid-point of the study reach to account for tailwater effects near the Water Treatment Plant access road crossing (LL2). These baseline flows were maintained for all subsequent simulations.

A comparison of simulated and observed peak water levels for the November 2 event is presented in **Table 4-2**. It can be seen that there is close agreement between the observed and simulated values at both logger locations.

Table 4-2: Summary of Hydraulic Calibration Results

Location	Observed Water Level during November 2 Calibration Event (m)	Simulated Water Level during November 2 Calibration Event (m)	Difference (m)
LL1	13.28	13.20	-0.08
LL2	25.32	25.25	-0.07

4.3.2 Hydrologic Sensitivity Analysis

Prior to the calibration exercise, a sensitivity analysis of hydrologic parameters was completed for the PCSWMM model using the calibration rainfall event (November 2, 2018). Model sensitivity was investigated by ranging the following parameters by $\pm 10\%$.

- Flow length;
- Slope;
- Impervious Area;
- Overland roughness coefficient; and
- Runoff Curve Number.

Based on this analysis, the most sensitive parameter was Runoff Curve Number followed by pervious roughness and flow length. The average percent change in runoff for each parameter is summarized in **Table 4-3**. The simulated runoff for each scenario is provided in **Appendix F**.

Table 4-3: Hydrologic Sensitivity Analysis Summary

Parameter	Average Deviation of Simulated Peak Flow – 90% of Base Value	Average Deviation of Simulated Peak Flow – 110% of Base Value
Overland Flow Length	+6.41%	-5.19%
Slope	-2.41%	+3.44%
Percent Impervious	-3.10%	+4.15%
Impervious Surface Roughness	+0.27%	0.00%
Pervious Surface Roughness	+6.38%	-5.16%
Runoff Curve Number (CN) Value	-15.68%	+20.79%

Based on the sensitivity results summary presented in the table above, it can be seen that the deviation for most of the parameters is in the order of $\pm 5\%$, with the exception of runoff CN value. These low variations in runoff can be partially attributed to the steep grades (typically in the order of 6%) in the upper watershed which leads to a rapid runoff response. Due to the high sensitivity of the CN value, it was a key parameter in model calibration.

4.3.3 Downstream Boundary – Sensitivity Analysis

Water level data was not available at the boundary of the study reach; however, boundary condition impacts on the study area were expected to be minor based on the distance from the study area (~5 km) and bed slope of Lebreau Creek Brook (~2%). A sensitivity analysis was undertaken to quantify the impact of the boundary condition on the simulation results. The peak water surface elevation (WSL) for a normal boundary condition was varied by $\pm 25\%$ (± 1.66 m) to assess the impact of simulated WSL at LL1 and LL2. The results of this analysis are summarized in **Table 4-4**.

Table 4-4: Summary of Simulated Peak Water Levels for Existing Conditions

Location	Simulated WSEL (m)		
	Outlet WSEL of 4.97m – -25% of Normal Boundary Condition	Outlet WSEL of 6.63m – Normal Boundary Condition	Outlet WSEL of 8.29m – +25% of Normal Boundary Condition
LL1	26.66	26.66	26.66
LL2	14.29	14.29	14.29

As demonstrated above, the boundary condition does not significantly impact results in the study area; therefore, a normal boundary condition for this assessment.

5.0 Simulation Results

The following sections present the simulated results for the Three Mile Plains drainage network for existing conditions and projected future climate/development conditions. The drainage network was assessed for six 24-hour rainfall events with return periods ranging between 1:2 year and 1:100-year. It should be noted that level of service (LOS) for ditches is defined as the rainfall event during which the banks overtop. LOS for crossings, such as culvert and bridges, is the rainfall event during which overtopping of the roadway occurs.

The maximum ponding depth considered for ditches in the model was 0.3 m. A review of stormwater management guidelines in other municipalities (e.g. cities of Waterloo and Windsor in Ontario) suggest that 0.3 m is a common maximum for acceptable ponding depth in major drainage networks for the 1:100 year return period event. It is noted that any amount of roadway ponding during the 1:5 year event is generally considered unacceptable. Minimizing roadway ponding is an important consideration for the safety of motorists (e.g. reduced potential for hydroplaning) and increases the longevity of the roadway surface.

Figure 5-1 and **5-3** provide a summary of the flood vulnerabilities identified within the study area for existing development with historical climate conditions and complete development with climate change conditions. Given the minor differences in results, maps have not been prepared for existing development and moderate development with climate change results.

A detailed review of the simulation results and flood vulnerable areas is provided in the following sections.

5.1 Existing Performance

5.1.1 Historic Climate Scenario

An assessment of existing drainage conditions was completed for the study area using the PCSWMM model. These simulation results are discussed below for Lebreau Creek Brook and the drainage network within the community (ditching/culverts). These results are summarized in **Figure 5-1**.

5.1.1.1 Lebreau Creek Brook

The model suggests existing hydraulic constraints along the study reach of Lebreau Creek Brook. The Lebreau Creek Brook crossing at Old Shanks Road (private) is expected to overtop during the 1:2 year event, and Mountain Road and Panuke Road are expected to overtop for the 1:25 year event. The crossing at the Water Treatment Plant (WTP) Access Road is not expected to overtop under the 1:100 year event.

Peak water levels at these crossings may have an impact on surrounding residential properties and infrastructure. For instance, the model suggests that there is potential for basement flooding upstream and downstream of the Mountain Road crossing (J33, J96). Similarly, residential properties could be impacted in the areas upstream and downstream of the Panuke Road crossing (J32) and upstream of Old Shanks Road (J24_1, J21_1, and J50_1). These areas are presented in **Figure 5-1**.

The simulated peak water levels in flood vulnerable areas along Lebreau Creek Brook are summarized in **Table 5-1**.

Table 5-1: Summary of Simulated Peak Water Levels at Flood Vulnerable Areas along Lebreau Creek Brook – Existing Development Conditions with Historical Climate

Location	Existing Development Conditions Peak Water Level (m)					
	2 yr	5 yr	10 yr	25 yr	50-yr	100 yr
Hydraulic Structures						
Old Shanks Road Bridge – Upstream (J24_1)	22.94 ¹	23.14 ¹	23.30 ¹	23.42 ¹	23.50 ¹	23.57 ¹
Panuke Road Bridge – Upstream (J741)	20.57	21.12	21.52	21.76	21.85	21.94
Panuke Road Bridge – Downstream (J32)	20.42 ¹	20.69 ¹	20.77 ¹	20.87 ¹	20.93 ¹	20.98 ¹
Mountain Road Bridge – Upstream (J33)	15.81 ¹	16.22 ¹	16.56 ¹	16.80 ¹	16.92 ¹	17.02 ¹
WTP Access Road Crossing – Upstream (J3)	13.36	13.59	13.75	13.95	14.10	14.26
Floodplain Impacts						
Old Shanks Road Area – Upstream (J21_1)	24.08 ¹	24.22 ¹	24.29 ¹	24.33 ¹	24.35 ¹	24.37 ¹
Old Shanks Road Area – Downstream (J50_1)	22.34	22.43	22.49	22.55 ¹	22.60 ¹	22.63 ¹
Panuke Road Area – Upstream (J8_2)	21.11	21.24	21.54	21.77	21.87	21.94
Mountain Road Area – Downstream (J96)	14.59 ¹	14.71 ¹	14.78 ¹	14.86 ¹	14.92 ¹	14.98 ¹

¹ Potential basement flooding issues – assuming 6ft (1.8m) basement

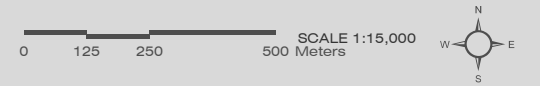
Note: Items in red indicate potential flooding of roadways, driveways, and houses

THREE MILE PLAINS
FLOOD RISK ASSESSMENT STUDY

FLOOD VULNERABLE AREAS - EXISTING DEVELOPMENT WITH HISTORICAL CLIMATE

FIGURE 5-1

- 1:2 YEAR FLOOD VULNERABLE AREAS
- 1:5 YEAR FLOOD VULNERABLE AREAS
- 1:10 YEAR FLOOD VULNERABLE AREAS
- 1:25 YEAR FLOOD VULNERABLE AREAS
- 1:50 YEAR FLOOD VULNERABLE AREAS
- 1:100 YEAR FLOOD VULNERABLE AREAS
- POTENTIAL BASEMENT FLOODING ONLY
- LEBREAU CREEK BROOK CENTERLINE
- DITCH CENTERLINE
- THREE MILE PLAINS COMMUNITY BOUNDARY
- ROADWAY



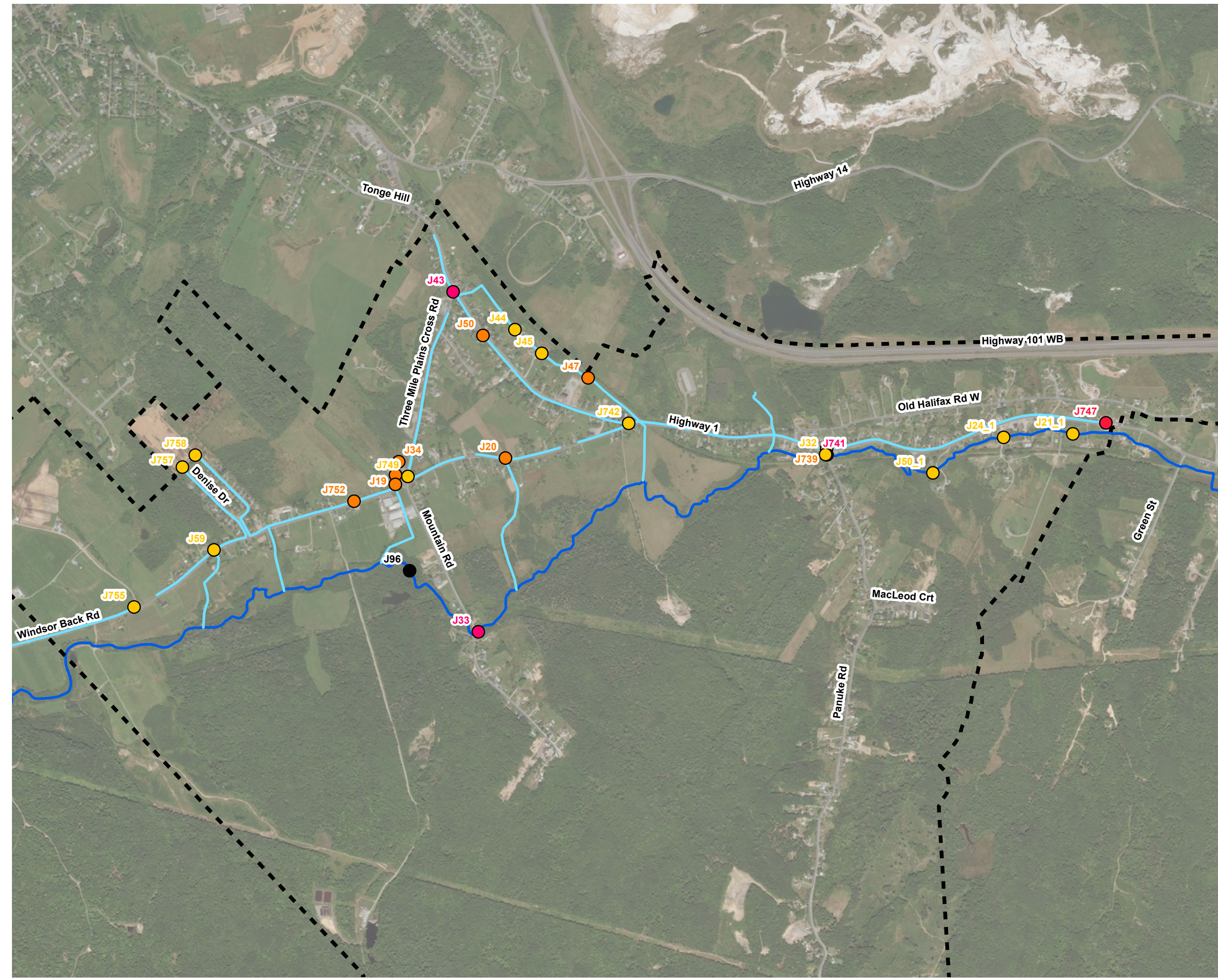
MAP DRAWING INFORMATION:
DATA PROVIDED BY WEST HANTS AND ESRI

MAP CREATED BY: JGC
MAP CHECKED BY: JAM
MAP PROJECTION: NAD 1983 UTM Zone 20N

FILE LOCATION: \\DILLON.CA\DILLON_DFS\LONDON\LONDON CAD\GIS\VISUAL COMMUNICATIONS DIMXD TEMPLATES\BEIGE - 11X17 LANDSCAPE - LEGEND RIGHT.MXD



PROJECT: 18-8544
STATUS: FINAL
DATE: 04/18/19



5.1.1.2

Stormwater Conveyance System

The model also identifies issues with several ditches throughout the Community. At the 1:2 year level, potential flooding is indicated on the Three Mile Plains Cross Road, Highway 1, at the eastern end of Windsor Back Road, and the upstream areas of Denise Drive and Alexander Drive. Impacted areas are expected to increase to include a significant portion of Highway 1 and Windsor Back Road at the 5-year historical rainfall event. The simulated peak water level at these locations is summarized in **Table 5-2**.

Table 5-2: Summary of Simulated Peak Water Levels at Flood Vulnerable Areas in Ditches – Existing Development Conditions with Historical Climate

Location	2-year Peak Water Level (m)	5-year Peak Water Level (m)	10-year Peak Water Level (m)	25-year Peak Water Level (m)	50-year Peak Water Level (m)	100-year Peak Water Level (m)
Highway 1 (J747)	26.73	26.88	27.26	27.26	27.26	27.26
Highway 1 (J739)	22.01	22.41	22.41	22.41	22.41	22.41
Highway 1 (J746)	28.27	28.37	28.44	28.52	28.57	28.64
Highway 1 (J50)	32.24	32.64	32.64	32.64	32.64	32.64
Highway 1 (J742)	26.52	26.52	26.52	26.52	26.52	26.52
Highway 1 (J43)	26.95	27.26	27.45	27.95	27.95	27.95
Highway 1 (J44)	24.10	24.10	24.10	24.10	24.10	24.10
Highway 1 (J45)	23.25	23.25	23.25	23.25	23.25	23.25
Highway 1 (J47)	22.76	23.17	23.17	23.17	23.17	23.17
Three Mile Plains Cross Road (J753)	28.78	28.86	28.91	28.96	29.00	29.04
Three Mile Plains Cross Road (J749)	23.92	23.92	23.92	23.92	23.92	23.92
Three Mile Plains Cross Road (J34)	22.32	22.34	22.34	22.34	22.34	22.34
Three Mile Plains Cross Road (J29)	22.29	22.30	22.30	22.30	22.30	22.30
Windsor Back Road (J20)	25.86	26.59	26.59	26.59	26.59	26.59
Windsor Back Road (J19)	22.24	22.25	22.25	22.25	22.25	22.25
Windsor Back Road (J752)	23.28	23.75	23.75	23.75	23.75	23.75
Windsor Back Road (J59)	17.11	17.11	17.11	17.11	17.11	17.11

Location	2-year Peak Water Level (m)	5-year Peak Water Level (m)	10-year Peak Water Level (m)	25-year Peak Water Level (m)	50-year Peak Water Level (m)	100-year Peak Water Level (m)
Windsor Back Road (J16)	21.83	21.91	21.96	22.02	22.07	22.12
Windsor Back Road (J755)	16.29	16.29	16.29	16.29	16.29	16.29
Denise Drive (J757)	25.59	25.59	25.59	25.59	25.59	25.59
Alexander Drive (J758)	25.04	25.04	25.04	25.04	25.04	25.04

Note: Items in red indicated roadway ponding in excess of 0.3 m

5.1.2 Projected Climate Change Scenario

An assessment of projected future climate impacts was also completed. Projected future rainfall events were simulated in the PCSWMM model to evaluate impacts to drainage infrastructure. Projected future simulation results are discussed below for Lebreau Creek Brook and the stormwater conveyance system within the Community (ditches, culverts).

5.1.2.1 Lebreau Creek Brook

Under existing development conditions with climate change, the model suggests hydraulic restrictions along Lebreau Creek Brook will slightly worsen. The crossing at Old Shanks Road (private) is expected to overtop during the 1:2 year climate change event, and Mountain Road and Panuke Road are expected to overtop for the 1:10 year climate change event. The LOS of the crossings at Mountain Road and Panuke is slightly lower historical climate conditions.

The crossing at the Water Treatment Plant access road is not expected to overtop during the 1:100 year climate change event. As mentioned above, peak water levels at these crossings may have an impact on surrounding infrastructure particularly in areas upstream of Mountain Road to the eastern community boundary.

The simulated peak water levels at flood vulnerable areas along Lebreau Creek Brook are summarized in **Table 5-3**.

Table 5-3: Summary of Simulated Peak Water Levels at Flood Vulnerable Areas along Lebreau Creek Brook – Existing Development Conditions with Projected Future Climate Conditions

Location	Existing Development Conditions Peak Water Level (m)					
	2 yr	5 yr	10 yr	25 yr	50-yr	100 yr
Hydraulic Structures						
Old Shanks Road Bridge – Upstream (J24_1)	23.00 ¹	23.30 ¹	23.41 ¹	23.57 ¹	23.66 ¹	23.76 ¹
Panuke Road Bridge – Upstream (J741)	20.76	21.52	21.74	21.93	22.06	22.20
Panuke Road Bridge – Downstream (J32)	20.53 ¹	20.77 ¹	20.87 ¹	20.97 ¹	21.06 ¹	21.15 ¹
Mountain Road Bridge – Upstream (J33)	15.97 ¹	16.56 ¹	16.78 ¹	17.01 ¹	17.18 ¹	17.36 ¹
WTP Access Road Crossing – Upstream (J3)	13.45	13.75	13.93	14.24	14.51	14.86
Floodplain Impacts						
Old Shanks Road Area – Upstream (J21_1)	24.15 ¹	24.29 ¹	24.32 ¹	24.37 ¹	24.40 ¹	24.43 ¹
Old Shanks Road Area – Downstream (J50_1)	22.37	22.49	22.55 ¹	22.63 ¹	22.70 ¹	22.78 ¹
Panuke Road Area – Upstream (J8_2)	21.16	21.53	21.76	21.95	22.08	22.23 ¹
Mountain Road Area – Downstream (J96)	14.65 ¹	14.78 ¹	14.85 ¹	14.96 ¹	15.07 ¹	15.15 ¹

¹ Potential basement flooding issues – assuming 6ft (1.8m) basement

Note: Items in red indicate potential flooding of roadways, driveways, and houses

5.1.2.2 Stormwater Conveyance System

Similarly to the historical climate conditions results, ditch capacity is a problem throughout the Community with LOS expected to decrease in some areas for climate change conditions. **Table 5-4** provides a summary of peak water levels in areas of concern.

Table 5-4: Summary of Simulated Peak Water Levels at Flood Vulnerable Areas in Ditches – Existing Development Conditions with Climate Change Projections

Location	2-year Peak Water Level (m)	5-year Peak Water Level (m)	10-year Peak Water Level (m)	25-year Peak Water Level (m)	50-year Peak Water Level (m)	100-year Peak Water Level (m)
Highway 1 (J747)	27.26	27.26	27.26	27.26	27.26	27.26
Highway 1 (J739)	22.41	22.41	22.41	22.41	22.41	22.41
Highway 1 (J746)	28.63	28.44	28.51	28.63	29.00	29.00
Highway 1 (J50)	32.64	32.64	32.64	32.64	32.64	32.64
Highway 1 (J742)	26.52	26.52	26.52	26.52	26.52	26.52
Highway 1 (J43)	28.15	27.45	27.69	27.95	27.95	27.95
Highway 1 (J44)	24.10	24.10	24.10	24.10	24.10	24.10
Highway 1 (J45)	23.25	23.25	23.25	23.25	23.25	23.25
Highway 1 (J47)	23.17	23.17	23.17	23.17	23.17	23.17
Three Mile Plains Cross Road (J753)	29.04	28.91	28.96	29.04	29.10	29.42
Three Mile Plains Cross Road (J749)	23.92	23.92	23.92	23.92	23.92	23.92
Three Mile Plains Cross Road (J34)	22.34	22.34	22.34	22.34	22.34	22.34
Three Mile Plains Cross Road (J29)	22.30	22.30	22.30	22.30	22.30	22.30
Windsor Back Road (J20)	26.59	26.59	26.59	26.59	26.59	26.59
Windsor Back Road (J19)	22.25	22.25	22.25	22.25	22.25	22.25
Windsor Back Road (J752)	23.75	23.75	23.75	23.75	23.75	23.75
Windsor Back Road (J59)	17.11	17.11	17.11	17.11	17.11	17.11
Windsor Back Road (J16)	22.11	21.96	22.02	22.11	22.45	22.45
Windsor Back Road (J755)	16.29	16.29	16.29	16.29	16.29	16.29
Denise Drive (J757)	25.59	25.59	25.59	25.59	25.59	25.59
Alexander Drive (J758)	25.04	25.04	25.04	25.04	25.04	25.04

Note: Items in red indicated roadway ponding in excess of 0.3 m

5.2 Future Development Scenarios

The future development condition assessment considers two development scenarios: moderate development and complete development. Based on Statistics Canada Census Data from 2001 to 2016, the number of private dwellings in the Municipality has increased by approximately 12.30%; an average increase of 0.82% (or just slightly over 4 dwellings) per year. This average increase was then applied to the 2011 census number of private dwellings within the Three Mile Plains community to estimate the expected number of additional private dwellings for the proposed future development scenarios. It should be noted that the 2011 census number of private dwellings was 541 or approximately 384.11 ha in area.

In the moderate development model (a 25 year planning horizon), an estimated 102 additional private dwellings would be expected for a total of 643 private dwellings by 2044. Further, an additional 110 new private dwelling would be then expected under the complete development scenario (50 year planning horizon) for a total of 753 private dwelling by 2069. The estimated total increase in developed area for the moderate and complete development scenarios are 72.7 ha and 151.1 ha, respectively.

Based on discussions with the Municipality, it is anticipated that the majority of residential development over the 50 year planning horizon will take place in the Three Mile Plains' "growth centre" area, encompassed within sub-catchments shaded red in **Figure 5-2**.

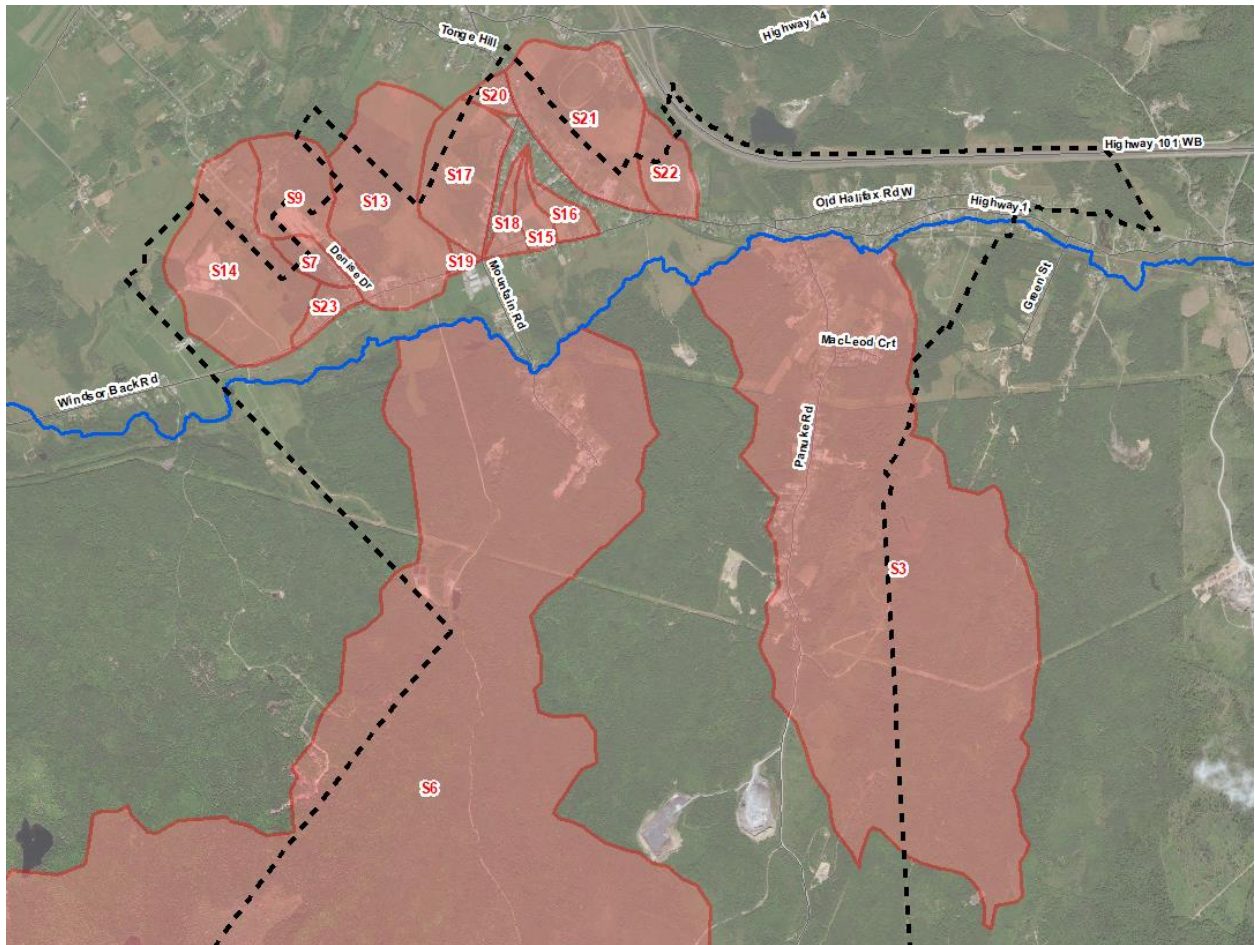


Figure 5-2: Areas of Expected Development within the Three Mile Plains Community

The projected development conditions were included in the PCSWMM future conditions models by increasing the percent impervious of the subcatchments within the anticipated growth areas. A summary of the total change to the catchment parameters is presented **Table 5-5**.

Table 5-5: Summary of Total Changes to Imperviousness for Moderate and Complete Development Scenarios

Catchment ID	Imperviousness (%) – Existing Conditions	Imperviousness (%) – Moderate Development (25 years)	Imperviousness (%) – Complete Development (50 years)
S3	5.5	7.0	8.5
S6	1.1	1.3	1.5
S7	30	32.4	34.9
S9	20	22.4	24.9
S13	5	7.4	9.9
S14	22.8	25.2	27.7
S15	35	49.1	64.2
S16	20	34.1	49.2
S17	15.7	23.9	32.7
S18	30	37.3	45.2
S19	40	47.3	55.2
S20	15	20.8	27.1
S21	20	25.8	32.1
S22	5	8.1	11.4
S23	35	37.4	39.9

5.2.1 Proposed Climate Change Scenario – Moderate Development Conditions

Simulation results for moderate development conditions (25 year planning horizon) with climate change projections are very similar to the results for existing development with climate change. These negligible differences can be attributed to the high grades (~6%) in the study area which limit the impact of moderate changes to impervious area.

5.2.1.1 Lebreau Creek Brook

Peak water levels at flood vulnerable areas along Lebreau Creek Brook for the moderate development condition are summarized in **Table 5-6**. The LOS for the hydraulic crossings and floodplain flood risk are similar to those presented for the future climate, existing development condition scenario (section 5.1.2).

Table 5-6: Summary of Simulated Peak Water Levels at Flood Vulnerable Areas along Lebreau Creek Brook – Existing Development Conditions with Historical Climate

Location	Existing Development Conditions Peak Water Level (m)					
	2 yr	5 yr	10 yr	25 yr	50-yr	100 yr
Hydraulic Structures						
Old Shanks Road Bridge – Upstream (J24_1)	23.00 ¹	23.30 ¹	23.41 ¹	23.57 ¹	23.66 ¹	23.76 ¹
Panuke Road Bridge – Upstream (J741)	20.77	21.52	21.75	21.94	22.07	22.21
Panuke Road Bridge – Downstream (J32)	20.53 ¹	20.78 ¹	20.87 ¹	20.98 ¹	21.07 ¹	21.15 ¹
Mountain Road Bridge – Upstream (J33)	15.98 ¹	16.57 ¹	16.79 ¹	17.02 ¹	17.18 ¹	17.37 ¹
WTP Access Road Crossing – Upstream (J3)	13.46	13.75	13.94	14.25	14.52	14.87
Floodplain Impacts						
Old Shanks Road Area – Upstream (J21_1)	24.15 ¹	24.29 ¹	24.32 ¹	24.37 ¹	24.40 ¹	24.43 ¹
Old Shanks Road Area – Downstream (J50_1)	22.37	22.50	22.55 ¹	22.64 ¹	22.70 ¹	22.78 ¹
Panuke Road Area – Upstream (J8_2)	21.16	21.55	21.77	21.95	22.09	22.24 ¹
Mountain Road Area – Downstream (J96)	14.65 ¹	14.78 ¹	14.86 ¹	14.97 ¹	15.07 ¹	15.15 ¹

¹ Potential basement flooding issues – assuming 6ft (1.8m) basement

Note: Items in red indicate potential flooding of roadways, driveways, and houses

5.2.1.2

Stormwater Conveyance System

Ditch capacity issues are similar to the simulation results for existing development conditions with climate change projections; however, potential flooding is expected for more significant (i.e. total precipitation) rainfall event in some areas. **Table 5-7** provides a summary of peak water levels in areas of concern.

Table 5-7: Summary of Simulated Peak Water Levels at Flood Vulnerable Areas in Ditches – Moderate Development Conditions with Climate Change Projections

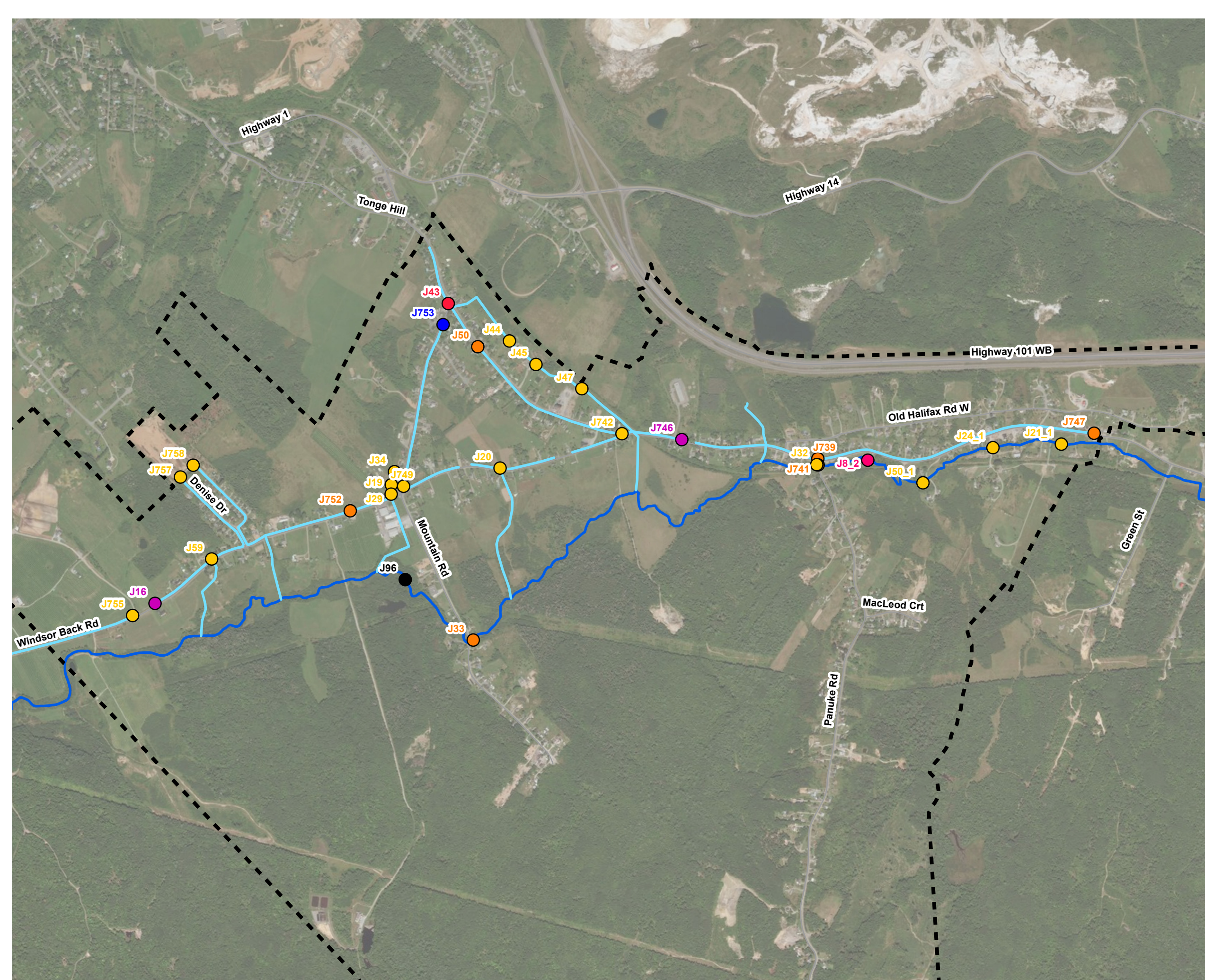
Location	2-year Peak Water Level (m)	5-year Peak Water Level (m)	10-year Peak Water Level (m)	25-year Peak Water Level (m)	50-year Peak Water Level (m)	100-year Peak Water Level (m)
Highway 1 (J747)	26.79	27.26	27.26	27.26	27.26	27.26
Highway 1 (J739)	22.08	22.41	22.41	22.41	22.41	22.41
Highway 1 (J746)	28.33	28.45	28.52	28.65	29.00	29.00
Highway 1 (J50)	32.30	32.64	32.64	32.64	32.64	32.64
Highway 1 (J742)	26.52	26.52	26.52	26.52	26.52	26.52
Highway 1 (J43)	27.16	27.52	27.95	27.95	27.95	27.95

Location	2-year Peak Water Level (m)	5-year Peak Water Level (m)	10-year Peak Water Level (m)	25-year Peak Water Level (m)	50-year Peak Water Level (m)	100-year Peak Water Level (m)
Highway 1 (J44)	24.10	24.10	24.10	24.10	24.10	24.10
Highway 1 (J45)	23.25	23.25	23.25	23.25	23.25	23.25
Highway 1 (J47)	23.17	23.17	23.17	23.17	23.17	23.17
Three Mile Plains Cross Road (J753)	28.82	28.91	28.96	29.04	29.10	29.42
Three Mile Plains Cross Road (J749)	23.92	23.92	23.92	23.92	23.92	23.92
Three Mile Plains Cross Road (J34)	22.34	22.34	22.34	22.34	22.34	22.34
Three Mile Plains Cross Road (J29)	22.30	22.30	22.30	22.30	22.30	22.30
Windsor Back Road (J20)	26.59	26.59	26.59	26.59	26.59	26.59
Windsor Back Road (J19)	22.25	22.25	22.25	22.25	22.25	22.25
Windsor Back Road (J752)	23.38	23.75	23.75	23.75	23.75	23.75
Windsor Back Road (J59)	17.11	17.11	17.11	17.11	17.11	17.11
Windsor Back Road (J16)	21.88	21.97	22.02	22.12	22.45	22.45
Windsor Back Road (J755)	16.29	16.29	16.29	16.29	16.29	16.29
Denise Drive (J757)	25.59	25.59	25.59	25.59	25.59	25.59
Alexander Drive (J758)	25.04	25.04	25.04	25.04	25.04	25.04

Note: Items in red indicated roadway ponding in excess of 0.3 m

5.2.2 Proposed Climate Change Scenario – Complete Development Conditions

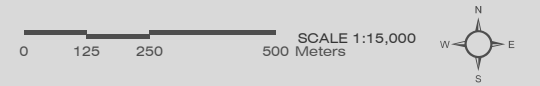
The simulation results for complete development conditions with climate change projections are much the same as the moderate development results. Summaries of peak water levels in flood vulnerable areas are provided in Appendix B. As discussed previously, these results are presented visually in Figure 5-3.



THREE MILE PLAINS
FLOOD RISK ASSESSMENT STUDY

FLOOD VULNERABLE AREAS - COMPLETE DEVELOPMENT WITH CLIMATE CHANGE ADJUSTMENTS
FIGURE 5-3

- 1:2 YEAR FLOOD VULNERABLE AREAS
- 1:5 YEAR FLOOD VULNERABLE AREAS
- 1:10 YEAR FLOOD VULNERABLE AREAS
- 1:25 YEAR FLOOD VULNERABLE AREAS
- 1:50 YEAR FLOOD VULNERABLE AREAS
- 1:100 YEAR FLOOD VULNERABLE AREAS
- POTENTIAL BASEMENT FLOODING ONLY
- LEBREAU CREEK BROOK CENTERLINE
- DITCH CENTERLINE
- THREE MILE PLAINS COMMUNITY BOUNDARY
- ROADWAY



MAP DRAWING INFORMATION:
DATA PROVIDED BY WEST HANTS AND ESRI

MAP CREATED BY: JGC
MAP CHECKED BY: JAM
MAP PROJECTION: NAD 1983 UTM Zone 20N

FILE LOCATION: \\DILLON.CA\DILLON_DFS\LONDON\LONDON CAD\GIS\VISUAL COMMUNICATIONS D\IMXD TEMPLATES\BEIGE - 11X17 LANDSCAPE - LEGEND RIGHT.MXD



PROJECT: 18-8544
STATUS: FINAL
DATE: 04/18/19

6.0 Assessment of Flood Risk Vulnerability

6.1 Lebreau Creek Brook Flood Risk

As presented in **Section 5.0** of this report, there are several locations of potential flood risk along the Lebreau Creek Brook study reach. Based on a review of existing ground elevations, the areas identified below are potentially vulnerable to damage of roads, driveways, basements and/or surrounding private property:

- **Old Shank Road Crossing Area (J50_1, J24_1, J21_1)** – Simulations suggest flood vulnerable properties in this area at the 2-year historical level. The private crossing at Old Shanks Road is expected to overtop during the 2-year historical rainfall event. The potential of basement flooding was also identified as an issue in areas upstream and downstream of the crossing during the 2-year historical event;
- **Panuke Road Crossing Area (J32, J741, J8_2)** – Based on the simulated results, the roadway and adjacent driveways are vulnerable to flooding during the 2-year historical rainfall event with flooding extents expected to increase at the 25-year historical event (10-year climate change event). There is also potential for basement flooding immediately downstream of Panuke Road during the 2-year historical event and upstream of Panuke Road during the 100-year climate change event; and
- **Mountain Road Crossing Area (J96, J33)** – The model indicates that basements could be at risk of flooding in this area during the 2-year historical event. Roadway and driveways are expected to be impacted during the 25-year historical rainfall event and 10-year climate change event.

The drainage issues discussed above are likely related to the undersized roadway crossings at Old Shanks Road, Panuke Road, and Mountain Road which restrict flow. A detailed assessment of the capacity of these structures is recommended to determine the required size and impact of these upgrades on water levels and flows. An assessment of downstream impacts should also be included in this scope of work to avoid significant impact to downstream infrastructure as a result of increased capacity.

6.2 Open Ditch Drainage Flood Risk

As previously discussed, the drainage network in the Three Mile Plains community is comprised of overland flow, open ditching, culverts, and natural channels. The simulation results suggest potential drainage challenges at various locations within the ditching and open channel network under both existing and proposed development conditions. These areas of concern are outlined below:

- **Alexander Drive** – The simulation results indicates potential flooding on Alexander Drive for the 2-year historical rainfall event. These issues are expected at the upstream areas of Alexander Drive;

- **Denise Drive** – Simulations suggest the potential for flooding in the upstream areas of Denise Drive. Based on the model, these drainage issues are expected to begin at the 2-year level for both existing and future climate conditions;
- **Windsor Back Road** – The model indicates a LOS of between 2-year and 5-year for the ditching along Windsor Back Road. Potential flooding would be expected in the vicinity of the intersection with Highway 1 and at the intersection of Three Mile Plains Cross Road during the 2-year historical rainfall event;
- **Highway 1** – Based on the model, potential flooding is indicated in several areas along Highway 1 beginning in the upstream areas near Three Mile Plains Cross Road during the 2-year historical rainfall event. The downstream areas along Highway 1 have an estimated LOS of 25-years under historical climate conditions and 5-years under projected future climate conditions; and
- **Three Mile Plains Cross Road** – Simulations suggest a low LOS along Three Mile Plains Cross Road with drainage issues during the 2-year historical rainfall event.

A review of ditch and culvert capacity as well as maintenance frequency for drainage infrastructure is recommended in these locations. It should be noted that potential flooding in these areas is expected to be somewhat minimized due to the surface grade (~6%). Any significant ponding in these areas would be expected to transition to overland flow, ultimately discharging to Lebreau Creek Brook.

7.0 Proposed Development Stormwater Management Review

7.1 Garland's Crossing

The Garland's Crossing community is bound by Highway 101 to the north and east, the Town of Windsor and Curry's Corner to the west, and Three Mile Plains to the south. Garland's Crossing is not part of the Lebreau Creek watershed – it drains to Tregothic Creek, which flows to the St. Croix River. Garland's Crossing has experienced rapid residential and commercial development in recent years, and is a focused area of growth in the Municipality of the District of West Hants, particularly at the Highway 14/Highway 1 interchange and at Exit 5. The Municipality has expressed concern that increased development within Garland's Crossing may contribute to increased flood hazard in Three Mile Plains.

The most recent major development is a mixed use single family home and mobile home development called "The Crossing" situated between Burgess Crescent, the Town of Windsor, Underwood Road, and Tregothic Creek. The Municipality requires developers to provide a plan to maintain pre-development peak flows after construction. In this case, the developer has proposed two stormwater management ponds that will store runoff from The Crossing development. This design is outlined in the *Stormwater Management System Design for The Crossing* (KVM Consultants, 2013).

As The Crossing is not located within the Lebreau Creek watershed, there is no increase in flood risk to the Three Mile Plains area. This assumes that the as-built conditions of the development site do not change the existing overland drainage patterns to re-direct flows toward Three Mile Plains.

7.2 Stormwater Management in the Municipality of the District of West Hants

Currently, stormwater management for developments in the Municipality relies primarily on maintaining pre-development peak flows associated with the area being developed. Several cities and municipalities throughout Canada have begun setting very prescriptive stormwater management planning and reporting requirements in flood-prone areas. Larger municipalities have established stringent stormwater management planning and reporting requirements for developers, given the history of flooding in the city. An example of this is the City of Saint John in New Brunswick which has published the *Storm Drainage Design Manual* to standardize drainage design and evaluation of pre/post runoff design.

The Municipality may also wish to consider the following when assessing stormwater management plans:

- Setting a standard Intensity-Duration-Frequency dataset for use by developers;
- Requiring stormwater quality analysis in addition to quantity analysis, including suspended solids and nutrient removal for maintenance of receiving water quality;
- Offering incentives for the consideration of Low-Impact-Development stormwater management features such as:
 - Absorbent Landscaping,
 - Bioretention Areas,
 - Bioswales,
 - Green Roofs,
 - Permeable Pavements,
 - Rainwater Harvesting, and
 - Stormwater Capture;
- Third-party review of stormwater management plan submissions for high risk areas; and
- Detailed design plan for major and minor stormwater systems including:
 - Pre-development and post-development overland flow patterns; and
 - Proposed stormwater pipe map with proposed pipe sizing.

A summary of applicable stormwater best management practices are provided in the following section.

7.2.1 Summary of Potential Best Management Practices

7.2.1.1 Outfall Outlet Erosion Assessment

The allowable velocity of a watercourse is an important consideration in outfall design. The allowable velocity is the maximum velocity that will not cause erosion of the streambed and banks; this parameter is related to the streambed material. The piped outlets discharging to Lebreau Creek Brook should be evaluated for outlet velocity and appropriate erosion resistance.

Energy dissipation basins can be constructed at erosion susceptible outfalls as part of detailed design; the design process outlined in Hydraulic Engineering Circular No. 14 (HEC 14) (U.S. Department of Transportation, 2006) is an appropriate resource for the design of energy dissipation.

Enhanced Vegetated Swales (Roadside Ditch Upgrades)

Enhanced vegetated swales, also called grass swales, are low-slope vegetated open channels designed to attenuate, treat, and convey stormwater runoff. Enhanced vegetated swales are often sited next to roadways as an alternative for simple ditching. Swales may be used in conjunction with other stormwater and erosion control measures such as check dams or level spreaders to achieve treatment and attenuation goals (TRCA, nd). Plan and profile views of a typical grass swale layout are shown in **Figure 7-1** and **Figure 7-2**, respectively.

Numerous vegetated roadside swales were observed during site visits completed as part of this project. Vegetated swales allow for conveyance of stormwater during intense rainfall events, and have the potential to contribute to treatment of surface runoff contaminants and reduce runoff velocity through infiltration.

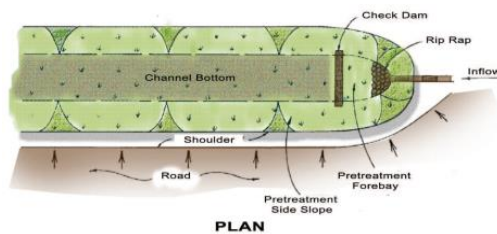


Figure 7-1: Enhanced Grass Swale Layout, Plan View (TRCA, nd)

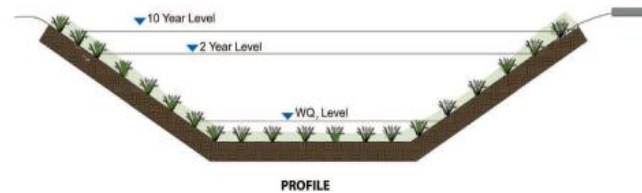


Figure 7-2: Enhanced Grass Swale Layout, Profile View (TRCA, nd)

The infiltration capacity of underlying soils, and depth to groundwater, play an important role in determining whether or not infiltration can reduce peak runoff volumes. Test pitting and infiltration testing of underlying soils prior to construction of a vegetated swale can assist in determining site suitability.

Over time, the permeability of the soil media can decrease significantly due to the accumulation of fines in the overlying top soil and vegetation. A simple test can be used to measure infiltration capacity in a vegetated soil media. A double ringed infiltrometer (**Figure 7-3**) provides a simple and effective means to estimate infiltration rate. This approach can be used in estimating in-situ infiltration rates for new swale/basin construction, or to evaluate infiltration rate of existing vegetated surfaces. For areas found to have poor infiltration capacity, the overlying soils may be removed and replaced with well-draining top soil or specially-engineered soil amendments, and re-vegetated; however, adequate underdrainage should be confirmed prior to construction.



Figure 7-3: Double Ring Infiltrimeter (Wikipedia, 2018)

7.2.1.2 Engineered/Natural Wetlands

Wetland enhancement/creation for stormwater treatment is an effective tool to filter stormwater runoff and can provide high removal efficiencies for stormwater pollutants (i.e., TSS, metals and hydrocarbons) and can be used to filter reduce stormwater runoff peak discharge rates (velocity/flow). A conceptual sketch of wetland creation along a watercourse is depicted below in **Figure 7-4**.

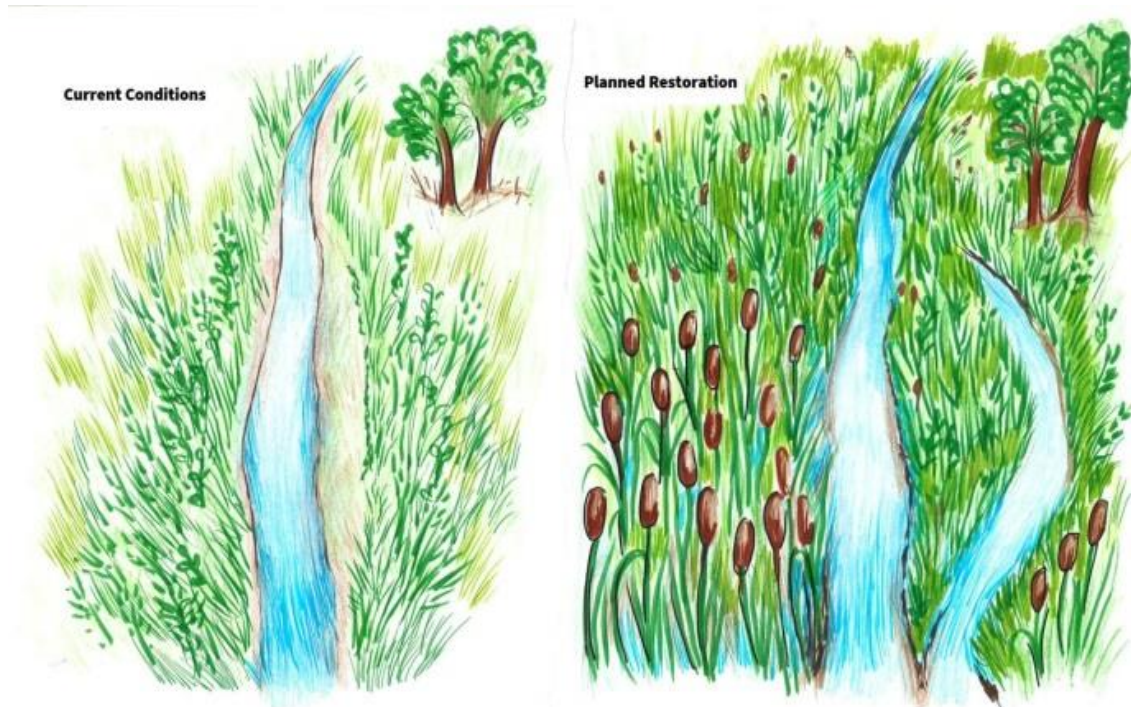


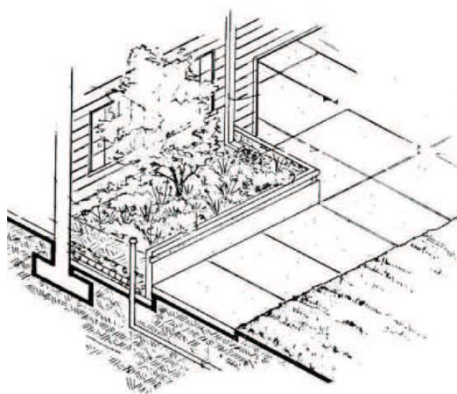
Figure 7-4: Conceptual Sketch of Wetland Upgrades

Engineered wetlands should be inspected twice per year (every 6 months). At time of inspection, undesirable plant species should be pulled out, dead (not dormant) vegetation removed, and pipes, grates, and screens cleaned. Fertilization may also be required to initiate or sustain vegetation growth. Depending on pollutant loading and removal efficiency, gravel within the engineered wetland may also need to be cleaned and/or replaced if it becomes clogged and the wetland is no longer performing effectively (Taylor, Yahner, Jones, Ogden, & Dunn, 1998).

7.2.1.3 Rain Gardens/Bioretention Cells

Bioretention cells, often also referred to as Rain Gardens, are designed to temporarily store, treat, and infiltrate stormwater runoff. Bioretention cells vary significantly in size depending on the application, but are particularly suited to natural depression areas and vegetated strips in paved areas to slow runoff velocities, and also help settle out sediment, support contaminant and pollutant removal, and encourage infiltration into native soils (TRCA, nd). A sample drawing of a bioretention cell is shown in Figure 7-5.

Source: Wisconsin Department of Natural Resources



Source: City of Portland

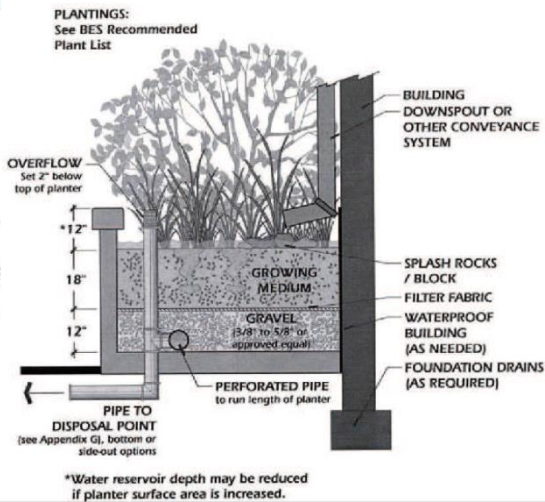


Figure 7-5: Conceptual Sketch of Bioretention Cell Components (TRCA, nd)

7.2.1.4 Permeable Pavements

Permeable Pavements are an alternative to traditional pavement surfaces such as asphalt and concrete. Permeable pavement systems allow stormwater to infiltrate from the paved surface, rather than generating runoff. Permeable pavements help reduce the negative impacts often associated with paved



Figure 7-6: Installed Permeable Pavers in Winter (trca, 2015)

areas, such as increased runoff velocity and water temperatures (TRCA, 2015).

Permeable pavements are available in four general types: permeable interlocking pavers (as shown in **Figure 7-6**), permeable interlocking grids, pervious concrete and pervious asphalts. Permeable pavements are often better suited to new development projects, but may also be appropriate for deep retrofits of existing parking areas on the Base. These systems are best suited to low-to-medium vehicle traffic areas including parking lots, plazas, walkways, and driveways (TRCA, 2015).

As with green roofs, a formal consideration for the installation of permeable pavements should be implemented as part of the planning process for new developments as well as for deep retrofit projects.

8.0 Preliminary Drainage Improvement Options

Based on field observations, and the PCSWMM simulation results, a set of conceptual drainage improvement options were prepared and are described in the subsequent sections. It is noted that these upgrades have been developed using the PCSWMM model assuming complete development conditions and the 100-year projected future climate rainfall depth distributed using the SCS Type II distribution. These upgrades are preliminary in nature based on a major rainfall event; therefore, detailed design is recommended for any improvement options the Municipality wishes to implement. This is particularly true for improvements along Lebreau Creek Brook where peak flows can be significantly impacted by both total precipitation and spring snow melt.

A Community wide overview of the recommended preliminary improvements is presented in **Table 8-1**. The recommendations below are also presented visually in **Figure 8-1**. A prioritization ranking from 1 (higher priority) to 3 (lower priority) has been assigned to each drainage challenge based on the extent of the issue and the potential risk to Community infrastructure.

Table 8-1: Summary of Preliminary Drainage Improvement Options

Drainage Improvement Option	Description	Prioritization Rank
Old Shanks Road Bridge Crossing (Private)	<p>Based on the simulation results, the crossing at Old Shanks Road has an estimated LOS of 2-years for historical climate conditions. The limited capacity of this crossing likely contributes to flooding challenges upstream.</p> <p>Based on preliminary sizing completed using the PCSWMM model, a 3.6m span x 2.4m rise concrete box culvert or equivalent open span bridge is recommended. This provides a headwater depth to structure height (H/D) ratio of 1.03 during the 1:100 year rainfall event with climate change adjustments. During the detailed design process, it is important that the following tasks are completed:</p> <ul style="list-style-type: none"> • An evaluation of downstream impacts is recommended as higher flows could increase flood vulnerability downstream; and • An evaluation of roadway elevation to accommodate the new structure. 	1
Panuke Road Bridge Crossing (NSTIR)	<p>The crossing at Panuke Road has an estimated LOS of 10-years for historical climate conditions and 5-years for projected future climate conditions. Preliminary sizing estimates indicate upgrading to a twin 3.6m span x 1.8m rise concrete box culvert (H/D = 1.03 for the 1:100 year CC rainfall event) or equivalent open span bridge.</p> <p>It is important to evaluate downstream impacts prior to implementing upgrades. An evaluation of roadway elevations should also be completed during the detailed design process.</p>	3
Mountain Road Bridge Crossing (NSTIR)	<p>The model indicates the crossing at Mountain Road has an estimated LOS of 10-years under historical climate conditions and 5-years for projected future climate conditions. Based on preliminary estimates, this structure should be upgraded to a twin 3.6m span x 2.4m rise concrete box culvert crossing (H/D = 0.98 for the 1:100 year CC rainfall event) or equivalent open span bridge.</p> <p>An evaluation of downstream impacts and roadway elevations is also recommended during detailed design.</p>	3

Drainage Improvement Option	Description	Prioritization Rank
Ditch and Channel Upgrades (NSTIR)	<p>Preliminary estimates indicate ditches depths should be increased to a maximum depth of 1.5m in depth. These estimates have assumed a bottom width of 0.5m and 3:1 side slopes (SS). A minimum freeboard of 0.1m should also be considered for the upgraded ditches. The recommended ditches upgrades for flood vulnerable areas are summarized below:</p> <ul style="list-style-type: none"> • Highway 1 (Civic #4600 to Civic #4801) – 1.5m Depth • Highway 1 (Intersection with Three Mile Plains Cross Road to Civic #4600) – 1.2m Depth • Three Mile Plains Cross Road – 1.2m Depth • Denise Drive – 1.5m Depth • Alexander Drive – 1.5m Depth • Windsor Back Road (Civic #1001 to intersection with Highway 1) – 1.2m Depth • Windsor Back Road (Three Mile Plains Cross Road to Denise Drive) – 1.5m Depth • Windsor Back Road (Denise Drive to western Community boundary) – 1.2m Depth <p>It is recommended that enhanced vegetated swales (Section 7.2.1.1) be considered in these areas to increase infiltration of stormwater runoff along the ditches.</p> <p>Upgrades to some of minor natural channels is also recommended. The preliminary estimates below have been made assuming 3:1 Side Slopes:</p> <ul style="list-style-type: none"> • Windsor Back Road (Civic #847) – 1.8m Depth, 1.0m Bottom Width • Highway 1 (between Three Mile Plains Cross Road and Windsor Back Road) – 1.5m-1.8m Depth, 1m-2m Bottom Width <p>Based on the location of these drains, it is expected that they are owned by the Municipality. These drains convey runoff from a relatively large area within the core of the Community, making them a priority to improve drainage. The implementation of BMPs (Section 7.2.1) in conjunction with the upgrades above could significantly improve drainage in these areas.</p>	2

Drainage Improvement Option	Description	Prioritization Rank
Culvert Upgrades	<p>Culvert upgrades are recommended to convey the additional flows from upgraded ditches. Preliminary upgrades for major culverts along Highway 1 and Windsor Back Road are as follows:</p> <ul style="list-style-type: none"> • Highway 1 (Civic #4627) – 2.4m x 1.8m Concrete Box Culvert • Highway 1 (At Railway Crossing) – 2.4m x 1.8m Concrete Box Culvert • Highway 1 (At Windsor Back Road) – Twin 900mm Diameter Concrete Culverts • Highway 1 (At Three Mile Plains Cross Road) – 600m Diameter Concrete Culvert • Three Mile Plains Cross Road (At Windsor Back Road) – 2m x 0.6m Box Culvert • Windsor Back Road (Civic #948) – Twin 1500mm Diameter Concrete Culverts • Windsor Back Road (Civic #847) – Twin 2.4m x 1.8m Concrete Box Culverts • Windsor Back Road (Civic #721) – Twin 2.4m x 1.8m Concrete Box Culverts • Windsor Back Road (Civic #684) – 1200mm Diameter Concrete Culvert • Mountain Road (At Mason Apples) - Twin 2.4m x 1.8m Concrete Box Culverts 	2
Wet Pond/Engineered Wetlands*	<p>The addition of wet ponds or engineered wetlands are an effective means of attenuating flow to limit required upgrades to the existing drainage system. As shown in Figure 8-1, three areas have been proposed for potential implementation of these facilities.</p> <ul style="list-style-type: none"> • Behind Civic #140 on Three Mile Plains Cross Road; • The field adjacent to Civic #953 on Windsor Back Road; and, • Expansion of the existing wetland along the railway adjacent to Cornwallis Veterinarians (Civic #4505 on Highway 1). <p>As previously noted, engineered wetlands have the added advantage of filtering stormwater runoff and provide high removal efficiencies for stormwater pollutants including, TSS, metals and hydrocarbons. The feasibility of including these facilities in the above noted areas should be evaluated based on surrounding topography and infrastructure, land acquisition requirements, and existing conveyance downstream capacity.</p>	3

8.1 Implementation Strategies

It was noted in discussions with the Municipality that the majority of drainage infrastructure within the Community is provincially-owned by Transportation and Infrastructure Renewal (TIR). This presents a challenge for the Municipality in undertaking the recommended drainage upgrades. It is recommended that the Municipality consider the following strategies to initiate a response to the drainage issues identified in this study.

- Conduct engagement sessions with TIR and impacted stakeholders (e.g. Mason Apples, Cornwallis Veterinarians) to discuss the findings of this study. This meeting should outline the estimated level of service of the existing drainage system, flood risk to citizens, and recommended drainage upgrades to prioritize areas throughout the Community. Dillon can provide guidance to the Municipality in preparing for these sessions.
- Completion of site assessments, including notes and photos, following major rainfall events and high flow conditions (e.g. spring freshet) to document impacts of these events on infrastructure within the Community. Funding programs are available to repair damage caused by these disaster events. One such programs is the National Disaster Mitigation Program (NDMP) which, unfortunately, comes to an end in 2020; however, it is expected that the NDMP will lead into future funding programs to address flood risks and costs. Baseline data (condition assessment, photos, etc.) should be collected so that resulting damage can be documented and used to support future applications.
- Funding programs accessible through the federal and provincial government, and other organizations can be used to supplement the cost of the proposed drainage upgrades outlined in **Table 8-1**. These programs seek to fund projects that will build resiliency against the impacts of climate change. A summary of applicable funding programs is provided below. Dillon has experience preparing these types of applications and can provide assistance in the preparation of applications.
 - Federal Gas Tax Fund (GTF): The renewed Federal GTF provides predictable, long-term, stable funding for Canadian municipalities to help them build and revitalize their local public infrastructure while creating jobs and long term prosperity (Infrastructure Canada, 2018).
 - Federation of Canadian Municipalities Funding Programs: The Federation of Canadian Municipalities offers funding programs to support green initiatives and climate protection projects to increase sustainability and enhance quality of life. Currently, their programs include the Green Municipal Fund and Municipalities for Climate Innovation Program (Federation of Canadian Municipalities, 2019).
 - Environmental Damages Fund (EDF): The EDF directs funds received as a result of fines, court orders, and voluntary payments to projects that will benefit the natural environment. Priority projects focus on environmental restoration, environmental

quality improvement, research and development, and education and awareness (Environment and Natural Resources, 2018).

- Clean Water and Wastewater Fund (CWWF): The CWWF program targets projects that will contribute to the rehabilitation of both water treatment and distribution infrastructure and existing wastewater and storm water treatment systems; collection and conveyance infrastructure; and initiatives that improve asset management, system optimization, and planning for future upgrades to water and wastewater systems. Although this program is set to end in 2020, it is expected that funding for this program will continue for the foreseeable future.
- The Ecology Action Centre (EAC) is an environmental interest group in Nova Scotia that seeks to provide environmentally and economically sustainable solutions in the Marine, Coastal and Water, Wilderness, Food, Transportation and Energy sectors. EAC is always looking for opportunities to implement pilot programs to assess innovative and environmentally friendly solutions. The EAC may be interested in collaborating with the Municipality to launch a pilot project for some of the BMPs identified in **Section 7.2** (e.g. rain garden, engineered wetland, etc.). Collaborating with EAC on these pilot programs may be a cost effective opportunity for the Municipality to improve drainage in select areas within the Community.

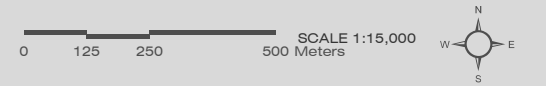
The Municipality can also improve drainage within the Community by making changes at a lot level scale, specifically with respect to new developments. As discussed in **Section 7.2**, prescriptive stormwater management guidelines are recommended to standardize the evaluation of pre/post development runoff (stormwater quantity). These guidelines should also address stormwater quality to achieve improved stormwater runoff quality from new developments. As discussed in **Section 7.2.1**, the use of BMPs and LIDs including detention areas and rain gardens can greatly improve stormwater quality. Incentive programs are a useful tool to increase the implementation of these strategies; examples of these programs are described below.

- Development Incentives: These types of incentives can include reduced permit fees, expedited permit processing, and the allowance for higher density development (United States Environmental Protection Agency, 2002).
- LID Installation Subsidies: The Municipality can offer grants, funding or reimbursements to property owners or developers who implement LID practices or systems (United States Environmental Protection Agency, 2002).
- Awards and Recognition Programs: These types of incentives can include features in newspaper articles and websites, as well as issuing yard signs to recognize the property owner. Recognition programs have the potential to increase property value and generally increase demand for properties (United States Environmental Protection Agency, 2002).

THREE MILE PLAINS
FLOOD RISK ASSESSMENT STUDY

DRAINAGE IMPROVEMENT OPTIONS
FIGURE 8-1

- DITCH IMPROVEMENT - 1.2m DEPTH
- DITCH/CHANNEL IMPROVEMENT - 1.5m DEPTH
- CHANNEL IMPROVEMENT - 1.8m DEPTH
- CULVERTS
- LEBREAU CREEK BROOK CENTERLINE
- DITCHES/NATURAL CHANNELS - UNCHANGED
- THREE MILE PLAINS COMMUNITY BOUNDARY
- POTENTIAL STORAGE AREAS



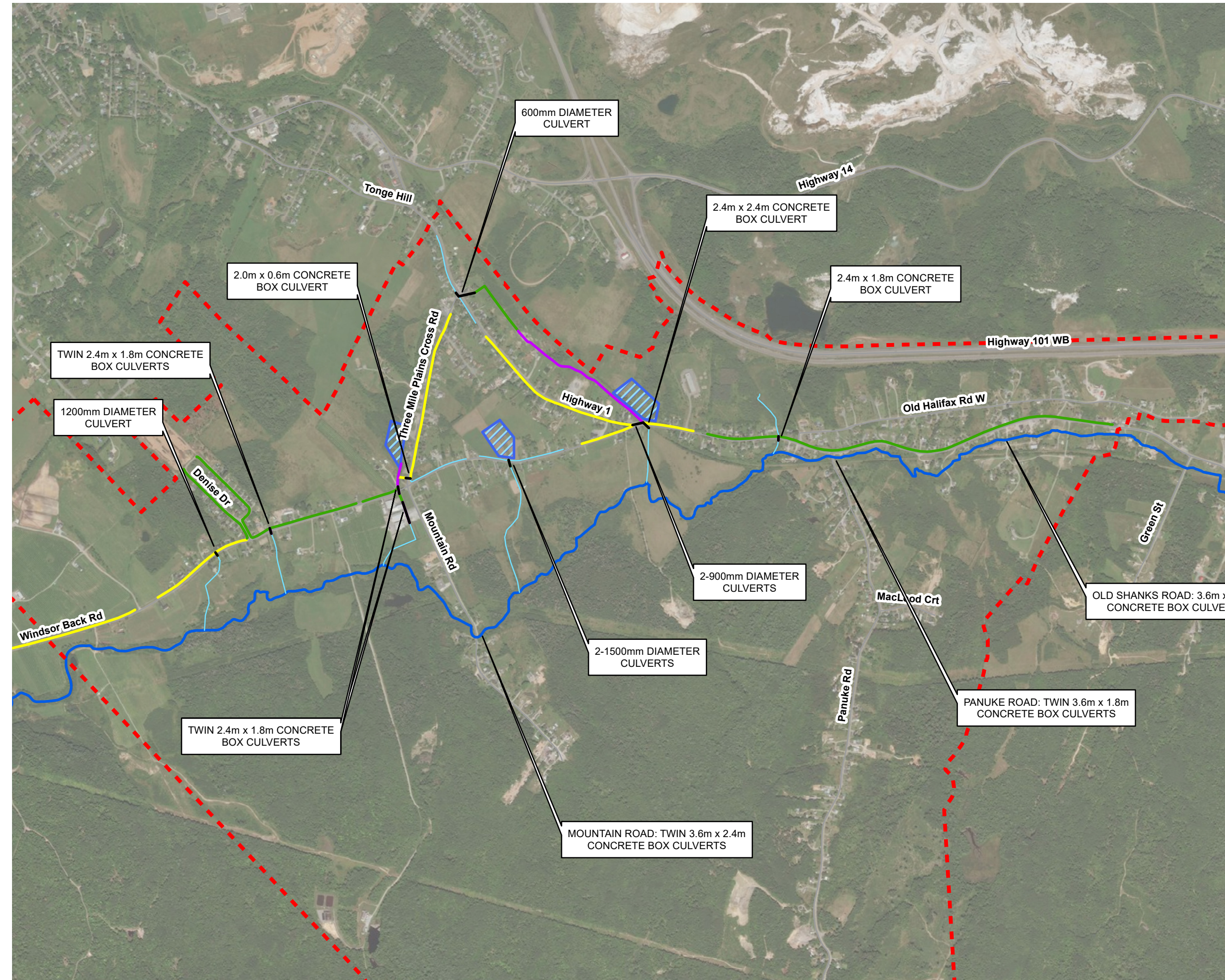
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DATA PROVIDED BY WEST HANTS AND ESRI

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MAP CHECKED BY: JAM
MAP PROJECTION: NAD 1983 UTM Zone 20N

FILE LOCATION: \\DILLON.CA\DILLON_DFS\LONDON\LONDON CAD\GIS\VISUAL COMMUNICATIONS DIMXD TEMPLATES\BEIGE - 11X17 LANDSCAPE - LEGEND RIGHT.MXD



PROJECT: 18-8544
STATUS: FINAL
DATE: 04/16/19



TWIN 2.4m x 1.8m CONCRETE BOX CULVERTS

1200mm DIAMETER CULVERT

2.0m x 0.6m CONCRETE BOX CULVERT

600mm DIAMETER CULVERT

2.4m x 2.4m CONCRETE BOX CULVERT

2.4m x 1.8m CONCRETE BOX CULVERT

Highway 101 WB

Highway 1

Old Halifax Rd W

TWIN 2.4m x 1.8m CONCRETE BOX CULVERTS

2-900mm DIAMETER CULVERTS

OLD SHANKS ROAD: 3.6m x 2.4m CONCRETE BOX CULVERT

2-1500mm DIAMETER CULVERTS

MacLeod Crt

PANUKE ROAD: TWIN 3.6m x 1.8m CONCRETE BOX CULVERTS

MOUNTAIN ROAD: TWIN 3.6m x 2.4m CONCRETE BOX CULVERTS

Panuke Rd

Green St

Tonge Hill

Highway 14

Three Mile Plains Cross Rd

Mountain Rd

Windsor Back Rd

Denise Dr

9.0 Conclusion

This report provides a review of the performance of the existing drainage system in the Community of Three Mile Plains. Drainage infrastructure within the Community consists of a combination of ditches, culverts and bridge crossings, natural channels, and numerous overland drainage routes. Hydrologic and hydraulic modeling was completed to assess the performance of the existing drainage system for a series of development and climate scenarios. The simulated scenarios included existing development with both historical and projected future climate conditions, moderate development (25 year planning horizon) with future climate, and complete development (50 year planning horizon) with future climate conditions. The drainage system was assessed using six different rainfall events with return periods ranging from 1:2 year to 1:100 year. The projected future climate change events were adjusted assuming a moderate greenhouse gas emissions scenario (RCP 4.5). Moderate and complete development conditions included the addition of 102 and 212 new private dwellings, respectively.

Based on the simulations, flood vulnerabilities were identified along Lebreau Creek Brook as well as throughout the local stormwater conveyance system (ditches, culverts). As summarized in **Figure 5-1**, these flood vulnerabilities are located in the following general areas:

Lebreau Creek Brook

- Upstream and downstream of the crossing at Old Shanks Road;
- Upstream and downstream of the crossing at Panuke Road; and
- Upstream and downstream of the crossing at Mountain Road.

Stormwater Conveyance System

- Alexander Drive;
- Denise Drive;
- Windsor Back Road;
- Highway 1; and
- Three Mile Plains Cross Road.

The complete development model (with climate change adjustments) was then updated to include the recommended drainage improvements summarized in **Table 8-1**. The recommended structure sizes are based on providing conveyance of the 100-year return period event with climate change projections. It is noted that these recommended improvements are preliminary and a detailed analysis would be required prior to implementation. Upgrades to crossings along Lebreau Creek Brook should also be subject to an analysis of downstream impacts to limit the potential of flooding and erosion in downstream areas.

It was noted in discussions with the Municipality that the majority of drainage infrastructure within the Community is provincially-owned by Transportation and Infrastructure Renewal (TIR). Acknowledging this challenge, a set of potential implementation strategies have been outlined to guide the Municipality in initiating a response to the drainage issues identified in areas within the Community.

Recommendations have also been provided to improve lot-level drainage; particularly in new developments. These implementation strategies as well as lot-level drainage recommendations are summarized below.

- Conduct engagement sessions with TIR and impacted stakeholders to discuss the findings of this study;
- Evaluation of federal and provincial funding programs to subsidize the cost of implementation;
- Collaboration with the Ecology Action Centre (EAC) for potential opportunities to partner on implementing drainage solutions; and,
- Introduction of prescriptive stormwater management for new developments with incentives for the use of BMPs and LIDs.

Appendix A

PCSWMM Catchment Parameters

Table A-1: PCSWMM Catchment Parameters – Existing Conditions

Name	Area (ha)	Flow Length (m)	Slope (%)	Imperv. (%)	N Imperv	N Perv	Curve Number
S1	572.9533	1792	4.767	33	0.02	0.04	69
S2	13.3171	261	6.74	22.8	0.02	0.4	55
S3	405.5721	2342	13.756	5.5	0.02	0.4	55
S4	397.4433	3267	14.192	2.2	0.02	0.4	55
S5	356.7505	3849	14.83	5.5	0.02	0.4	55
S6	2025.153	7441	10.17	1.1	0.02	0.4	55
S8	320.1134	2148	12.682	11	0.02	0.04	69
S10	20.0671	429	4.97	38	0.02	0.04	69
S11	22.736	668	6.556	27	0.02	0.04	69
S12	13.8383	354	4.648	10	0.02	0.04	69
S9_1	83.4075	1174	5.78	5.5	0.02	0.04	69
S9_2	20.5225	252	4.494	10	0.02	0.04	69
S7_1	155.0983	994	4.26	5.5	0.02	0.04	69
S7_2	539.2051	2096	8.918	1.1	0.02	0.4	55
S7	6.3728	163	4.86	30	0.02	0.04	69
S9	24.7439	650	6.008	20	0.02	0.04	69
S13	57.9022	1090	5.047	5	0.02	0.04	69
S14	59.7854	1259	4.766	22.8	0.02	0.04	69
S15	5.752	522	3.593	35	0.02	0.04	69
S16	6.4031	603	2.933	20	0.02	0.04	69
S17	29.7978	720	4.342	15.7	0.02	0.04	69
S18	5.809	539	3.714	30	0.02	0.04	69
S19	1.8758	191	3.136	40	0.02	0.04	69
S20	3.9276	325	5.66	15	0.02	0.04	69
S21	49.3571	627	4.114	20	0.02	0.04	69
S22	13.3816	580	5.85	5	0.02	0.04	69
S23	5.9259	167	6.689	35	0.02	0.04	69
S24	3.6198	105	4.638	40	0.02	0.04	69
S25	10.8193	161	4.244	40	0.02	0.04	69
S26	122.1008	1650	8.562	5	0.02	0.4	55

Name	Area (ha)	Flow Length (m)	Slope (%)	Imperv. (%)	N Imperv	N Perv	Curve Number
S27	79.2184	1114	13.382	15	0.02	0.4	55
S28	2.0389	137	4.845	40	0.02	0.4	55
S29	49.823	1178	9.496	15	0.02	0.4	55

Appendix B

Additional Simulation Results

Lebreau Creek Brook

Table B-2: Summary of Simulated Peak Water Levels at Flood Vulnerable Areas along Lebreau Creek Brook – Existing Development Conditions with Historical Climate

Location	Existing Development Conditions Peak Water Level (m)					
	2 yr	5 yr	10 yr	25 yr	50-yr	100 yr
Hydraulic Structures						
Old Shanks Road Bridge – Upstream (J24_1)	23.00 ¹	23.30 ¹	23.41 ¹	23.57 ¹	23.66 ¹	23.76 ¹
Panuke Road Bridge – Upstream (J741)	20.77	21.52	21.75	21.94	22.07	22.21
Panuke Road Bridge – Downstream (J32)	20.53 ¹	20.78 ¹	20.87 ¹	20.98 ¹	21.07 ¹	21.15 ¹
Mountain Road Bridge – Upstream (J33)	15.98 ¹	16.57 ¹	16.79 ¹	17.02 ¹	17.18 ¹	17.37 ¹
WTP Access Road Crossing – Upstream (J3)	13.46	13.75	13.94	14.25	14.52	14.87
Floodplain Impacts						
Old Shanks Road Area – Upstream (J21_1)	24.15 ¹	24.29 ¹	24.32 ¹	24.37 ¹	24.40 ¹	24.43 ¹
Old Shanks Road Area – Downstream (J50_1)	22.37	22.50	22.55 ¹	22.64 ¹	22.70 ¹	22.78 ¹
Panuke Road Area – Upstream (J8_2)	21.16	21.55	21.77	21.95	22.09	22.24 ¹
Mountain Road Area – Downstream (J96)	14.65 ¹	14.78 ¹	14.86 ¹	14.97 ¹	15.07 ¹	15.15 ¹

¹ Potential basement flooding issues – assuming 6ft (1.8m) basement

Note: Items in red indicate potential flooding of roadways, driveways, and houses

Stormwater Conveyance System

Table B-3: Summary of Simulated Peak Water Levels at Flood Vulnerable Areas in Ditches – Complete Development Conditions with Climate Change Projections

Location	2-year Peak Water Level (m)	5-year Peak Water Level (m)	10-year Peak Water Level (m)	25-year Peak Water Level (m)	50-year Peak Water Level (m)	100-year Peak Water Level (m)
Highway 1 (J747)	26.79	27.26	27.26	27.26	27.26	27.26
Highway 1 (J739)	22.08	22.41	22.41	22.41	22.41	22.41
Highway 1 (J746)	28.34	28.47	28.54	28.66	29.00	29.00
Highway 1 (J50)	32.30	32.64	32.64	32.64	32.64	32.64
Highway 1 (J742)	26.52	26.52	26.52	26.52	26.52	26.52
Highway 1 (J43)	27.24	27.60	27.95	27.95	27.95	27.95
Highway 1 (J44)	24.10	24.10	24.10	24.10	24.10	24.10
Highway 1 (J45)	23.25	23.25	23.25	23.25	23.25	23.25
Highway 1 (J47)	23.17	23.17	23.17	23.17	23.17	23.17
Three Mile Plains Cross Road (J753)	28.82	28.91	28.96	29.04	29.10	29.42
Three Mile Plains Cross Road (J749)	23.92	23.92	23.92	23.92	23.92	23.92
Three Mile Plains Cross Road (J34)	22.34	22.34	22.34	22.34	22.34	22.34
Three Mile Plains Cross Road (J29)	22.30	22.30	22.30	22.30	22.30	22.30
Windsor Back Road (J20)	26.59	26.59	26.59	26.59	26.59	26.59
Windsor Back Road (J19)	22.25	22.25	22.25	22.25	22.25	22.25
Windsor Back Road (J752)	23.41	23.75	23.75	23.75	23.75	23.75
Windsor Back Road (J59)	17.11	17.11	17.11	17.11	17.11	17.11
Windsor Back Road (J16)	21.88	21.98	22.03	22.13	22.45	22.45
Windsor Back Road (J755)	16.29	16.29	16.29	16.29	16.29	16.29
Denise Drive (J757)	25.59	25.59	25.59	25.59	25.59	25.59
Alexander Drive (J758)	25.04	25.04	25.04	25.04	25.04	25.04

Note: Items in red indicated roadway ponding in excess of 0.3 m

Appendix C

Hydrologic Sensitivity Analysis Results

Table C-1: Summary of Sensitivity Analysis Results – Flow Length

Catchment Name	Peak Runoff (m³/s) - Baseline	Peak Runoff (m³/s) - 90% of Baseline	Peak Runoff (m³/s) - 110% of Baseline	Percent Difference - 90% of Baseline	Percent Difference - 110% of Baseline
S1	2.27	2.37	2.18	4.41%	-3.96%
S2	0.3	0.32	0.29	6.67%	-3.33%
S3	0.35	0.38	0.33	8.57%	-5.71%
S4	0.2	0.22	0.19	10.00%	-5.00%
S5	0.2	0.22	0.19	10.00%	-5.00%
S6	0.34	0.37	0.31	8.82%	-8.82%
S8	0.36	0.39	0.34	8.33%	-5.56%
S10	0.27	0.28	0.27	3.70%	0.00%
S11	0.16	0.16	0.15	0.00%	-6.25%
S12	0.44	0.46	0.42	4.55%	-4.55%
S9_1	0.18	0.19	0.17	5.56%	-5.56%
S9_2	0.48	0.5	0.45	4.17%	-6.25%
S7_1	0.33	0.35	0.31	6.06%	-6.06%
S7_2	0.45	0.49	0.42	8.89%	-6.67%
				6.41%	-5.19%

Table C-2: Summary of Sensitivity Analysis Results – Overland Slope

Catchment Name	Peak Runoff (m ³ /s) - Baseline	Peak Runoff (m ³ /s) - 90% of Baseline	Peak Runoff (m ³ /s) - 110% of Baseline	Percent Difference - 90% of Baseline	Percent Difference - 110% of Baseline
S1	2.27	2.22	2.32	-2.20%	2.20%
S2	0.3	0.3	0.31	0.00%	3.33%
S3	0.35	0.34	0.36	-2.86%	2.86%
S4	0.2	0.19	0.21	-5.00%	5.00%
S5	0.2	0.2	0.21	0.00%	5.00%
S6	0.34	0.33	0.36	-2.94%	5.88%
S8	0.36	0.35	0.37	-2.78%	2.78%
S10	0.27	0.27	0.28	0.00%	3.70%
S11	0.16	0.15	0.16	-6.25%	0.00%
S12	0.44	0.43	0.45	-2.27%	2.27%
S9_1	0.18	0.18	0.19	0.00%	5.56%
S9_2	0.48	0.46	0.49	-4.17%	2.08%
S7_1	0.33	0.32	0.34	-3.03%	3.03%
S7_2	0.45	0.44	0.47	-2.22%	4.44%
				-2.41%	3.44%

Table C-3: Summary of Sensitivity Analysis Results – Percent Impervious

Catchment Name	Peak Runoff (m³/s) - Baseline	Peak Runoff (m³/s) - 90% of Baseline	Peak Runoff (m³/s) - 110% of Baseline	Percent Difference - 90% of Baseline	Percent Difference - 110% of Baseline
S1	2.27	2.08	2.47	-8.37%	8.81%
S2	0.3	0.29	0.32	-3.33%	6.67%
S3	0.35	0.34	0.36	-2.86%	2.86%
S4	0.2	0.2	0.2	0.00%	0.00%
S5	0.2	0.2	0.21	0.00%	5.00%
S6	0.34	0.34	0.34	0.00%	0.00%
S8	0.36	0.35	0.38	-2.78%	5.56%
S10	0.27	0.26	0.29	-3.70%	7.41%
S11	0.16	0.15	0.16	-6.25%	0.00%
S12	0.44	0.41	0.47	-6.82%	6.82%
S9_1	0.18	0.18	0.19	0.00%	5.56%
S9_2	0.48	0.45	0.5	-6.25%	4.17%
S7_1	0.33	0.32	0.34	-3.03%	3.03%
S7_2	0.45	0.45	0.46	0.00%	2.22%
				-3.10%	4.15%

Table C-4: Summary of Sensitivity Analysis Results – Impervious Surface Roughness

Catchment Name	Peak Runoff (m ³ /s) - Baseline	Peak Runoff (m ³ /s) - 90% of Baseline	Peak Runoff (m ³ /s) - 110% of Baseline	Percent Difference - 90% of Baseline	Percent Difference - 110% of Baseline
S1	2.27	2.28	2.27	0.44%	0.00%
S2	0.3	0.31	0.3	3.33%	0.00%
S3	0.35	0.35	0.35	0.00%	0.00%
S4	0.2	0.2	0.2	0.00%	0.00%
S5	0.2	0.2	0.2	0.00%	0.00%
S6	0.34	0.34	0.34	0.00%	0.00%
S8	0.36	0.36	0.36	0.00%	0.00%
S10	0.27	0.27	0.27	0.00%	0.00%
S11	0.16	0.16	0.16	0.00%	0.00%
S12	0.44	0.44	0.44	0.00%	0.00%
S9_1	0.18	0.18	0.18	0.00%	0.00%
S9_2	0.48	0.48	0.48	0.00%	0.00%
S7_1	0.33	0.33	0.33	0.00%	0.00%
S7_2	0.45	0.45	0.45	0.00%	0.00%
				0.27%	0.00%

Table C-5: Summary of Sensitivity Analysis Results – Pervious Surface Roughness

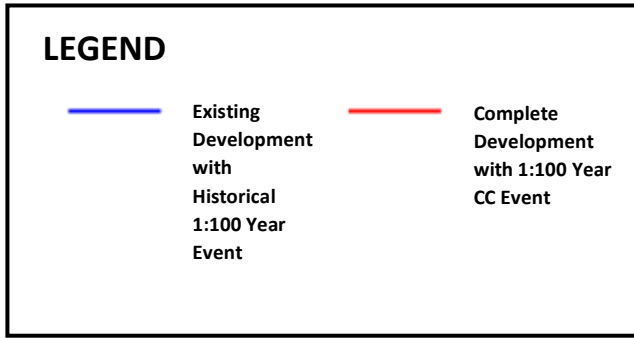
Catchment Name	Peak Runoff (m³/s) - Baseline	Peak Runoff (m³/s) - 90% of Baseline	Peak Runoff (m³/s) - 110% of Baseline	Percent Difference - 90% of Baseline	Percent Difference - 110% of Baseline
S1	2.27	2.36	2.19	3.96%	-3.52%
S2	0.3	0.32	0.29	6.67%	-3.33%
S3	0.35	0.38	0.33	8.57%	-5.71%
S4	0.2	0.22	0.19	10.00%	-5.00%
S5	0.2	0.22	0.19	10.00%	-5.00%
S6	0.34	0.37	0.31	8.82%	-8.82%
S8	0.36	0.39	0.34	8.33%	-5.56%
S10	0.27	0.28	0.27	3.70%	0.00%
S11	0.16	0.16	0.15	0.00%	-6.25%
S12	0.44	0.46	0.42	4.55%	-4.55%
S9_1	0.18	0.19	0.17	5.56%	-5.56%
S9_2	0.48	0.5	0.45	4.17%	-6.25%
S7_1	0.33	0.35	0.31	6.06%	-6.06%
S7_2	0.45	0.49	0.42	8.89%	-6.67%
				6.38%	-5.16%

Table C-5: Summary of Sensitivity Analysis Results – Runoff Curve Number (CN) Value

Catchment Name	Peak Runoff (m ³ /s) - Baseline	Peak Runoff (m ³ /s) - 90% of Baseline	Peak Runoff (m ³ /s) - 110% of Baseline	Percent Difference - 90% of Baseline	Percent Difference - 110% of Baseline
S1	2.27	2.08	2.51	-8.37%	10.57%
S2	0.3	0.26	0.36	-13.33%	20.00%
S3	0.35	0.29	0.43	-17.14%	22.86%
S4	0.2	0.16	0.26	-20.00%	30.00%
S5	0.2	0.17	0.26	-15.00%	30.00%
S6	0.34	0.26	0.45	-23.53%	32.35%
S8	0.36	0.31	0.42	-13.89%	16.67%
S10	0.27	0.25	0.3	-7.41%	11.11%
S11	0.16	0.14	0.17	-12.50%	6.25%
S12	0.44	0.38	0.51	-13.64%	15.91%
S9_1	0.18	0.15	0.23	-16.67%	27.78%
S9_2	0.48	0.41	0.56	-14.58%	16.67%
S7_1	0.33	0.26	0.41	-21.21%	24.24%
S7_2	0.45	0.35	0.57	-22.22%	26.67%
				-15.68%	20.79%

Appendix D

PCSWMM Profile – Lebreau Creek Brook



- Conduit C10_1 (31.173)
- Conduit C4_2 (13.91)
- Conduit C4_3 (31.184)
- Conduit C54_2 (34.963)
- Conduit C108 (34.941)
- Conduit C109_1 (34.908)
- Conduit C112_1 (34.919)
- Conduit C114 (36.776)
- Conduit C104 (42.377)
- Conduit C105 (42.263)
- Conduit C99 (42.518)
- Conduit C7 (42.534)
- Conduit C101_4 (18.801)
- Conduit C95 (57.305)
- Conduit C97_1 (57.271)
- Conduit C93_3 (67.882)
- Conduit C93_4 (67.866)
- Conduit C89 (67.76)
- Conduit C11 (68.815)
- Conduit C121 (68.932)
- Conduit C41 (68.809)
- Conduit C83 (68.961)
- Conduit C87 (68.96)
- Conduit C79 (68.955)

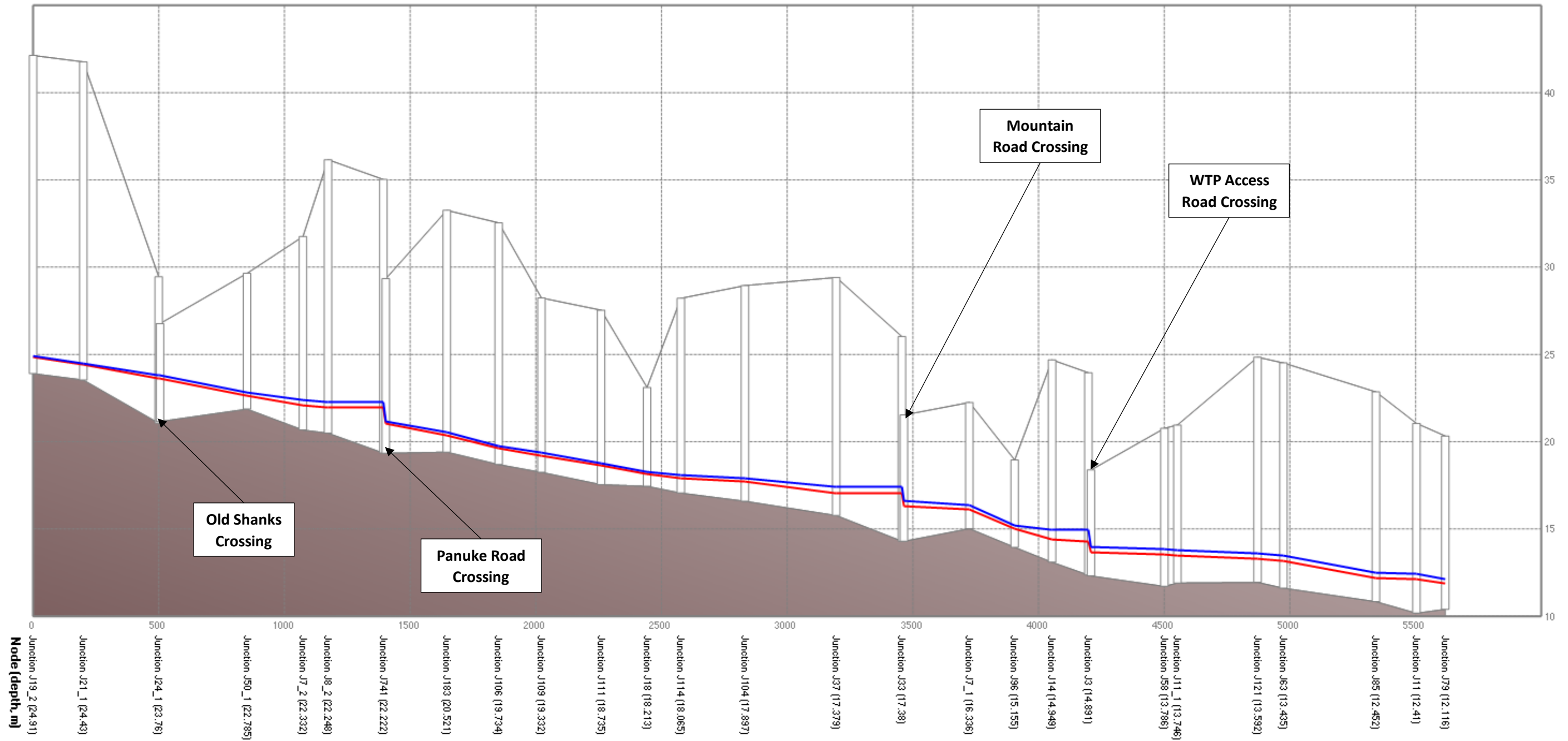


Figure D-1: Lebreau Creek Brook Profile with Water Level for Existing Development with the 1:100 Year Historical Event and Complete Development with the 1:100 year Climate Change Event