MUNICIPALITY OF WEST HANTS

Hantsport Stormwater Management Study

July 2018 – 17-6276

July 3, 2018

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Municipality of West Hants
Hantsport Stormwater Management Study

Attention: Mr. Brad Carrigan, Director of Public Works

Dear Mr. Carrigan,

Dillon Consulting Limited (Dillon) is pleased to present the following report entitled Hantsport Stormwater Management Study.

The attached report provides the methodology and hydrologic and hydraulic simulation results for the drainage network in Hantsport. 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.

Sincerely,

DILLON CONSULTING LIMITED

SDevereaux

Sarah Devereaux, M.Eng., P.Eng., FEC
Partner, Project Manager

SLD:jgc

Our file: 17-6276

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Executive Summary

The Municipality of West Hants (the Municipality) has retained Dillon Consulting Limited (Dillon) to undertake a Stormwater Management Study within the Community of Hantsport (Community). The Community is located along the west coast of mainland Nova Scotia in the vicinity of the Avon River estuary near the Bay of Fundy. The Community has a population of approximately 1,200 people. Historically, the Community has experienced drainage issues in the following areas:

- Chittick Avenue;
- Foundry Road;
- Riverview Road;
- Library/Playground on Main Street;
- Avon Street (no existing drainage infrastructure); and,
- Maple Drive.

The purpose of this study is to assess the performance of the current drainage system for a series of scenarios including existing and future 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.);
- Mitigate damage to public infrastructure; 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 storm and combined sewer, along with numerous overland drainage routes. Many of the roadways within the Community do not appear to be constructed as a dual-drainage network (i.e. major/minor drainage). However some of the recently upgraded road sections include new curb and gutter and would provide a degree of minor and major conveyance.

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 was used to assess the drainage network in Hantsport for six (6) 24-hour rainfall events with return periods ranging from two (2) years to 100 years. These rainfall events were developed using historical rainfall data as well as developed with climate change adjustments assuming a moderate greenhouse gas emissions scenario (RCP 4.5).

The future land use model was developed based on discussion the Municipality on November 22nd, 2017. The Municipality indicated a projected annual growth over a 50 year horizon of approximately two

(2) homes per year along with the potential construction of a nursing home. The expected developments were assumed to take place at the following locations:

- Extension of Faulkner Drive to Bog Road – Estimated 16 new homes;
- Connection of Chestnut Avenue to Evangeline Drive – Estimated 15 new homes;
- Extension of Mariner Drive – Nursing home;
- Extension of Alders Avenue to the west – Estimated 48 new homes; and,
- Extension of McCully Crescent and Mariner's Drive – Estimated 18 new homes.

The future land use model incorporates an estimated 97 new homes and a nursing home.

A series of simulations for both existing and future conditions were completed to assess the performance of the existing drainage works. The simulations scenarios included existing conditions with historical climate, future land use with historical climate, and future land use with climate change conditions. These simulations identified nine (9) areas potentially vulnerable to flooding, including:

- Holmes Hill Road – At the intersection with Riverbank Drive;
- Riverview Road – In the vicinity of Civic #8 and #23, and the ditch near the intersection with Rand Street;
- Maple Avenue – In the vicinity of Civic #24 and #30;
- Main Street Bridge Crossing;
- Chittick Avenue, Main Street, and School Street (Downtown) – In the vicinity of Civic #22 and #47 on Chittick Avenue, Civic #25 and #39 on Main Street, and Civic #12 on School Street;
- Foundry Road – In the vicinity of the soccer field;
- Prince Street and William Street – In the vicinity of Civic #11 on Prince Street and Civic #14 on William Street;
- Willow Street – At the intersection with Main Street; and,
- Faulkner Drive – In the vicinity of Civic #80.

A set of conceptual drainage improvement options were prepared for the vulnerable areas; improvements were also recommended for Avon Street and the library/playground area on Main Street (as noted by the Municipality). Recommendations include separation of the combined sewer systems on Willow Street, Prince Street, and William Street, and increased storm sewer sizes along Holmes Hill Road, Riverview Road, and Foundry. Storm sewers were sized to convey the 5-year return period event with climate change adjustments.

The aboiteau in the lower Halfway River failed in November 2017. The failure was concerning due to the potential of increased flood risk associated with storm surge from the Bay. Based on review of potential future water levels and surrounding topography, the estimated freeboard between the extreme water level elevation and the lowest residence adjacent to the river is approximately 8.8 m. The risk of riverine flooding to the residents of Hantsport is expected to be minimal.

1.0 Introduction

The Municipality of West Hants (the Municipality) has commissioned Dillon Consulting Limited (Dillon) to undertake a Stormwater Management Study within the Community of Hantsport (Community). This report describes the methodology and findings of the study, and puts forth recommendations to improve drainage conditions within the Community. The objectives of recommended drainage improvements are as follows:

- Protect citizens from flooding dangers (loss of life, loss of property, etc.);
- Mitigate damage to public infrastructure; 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.

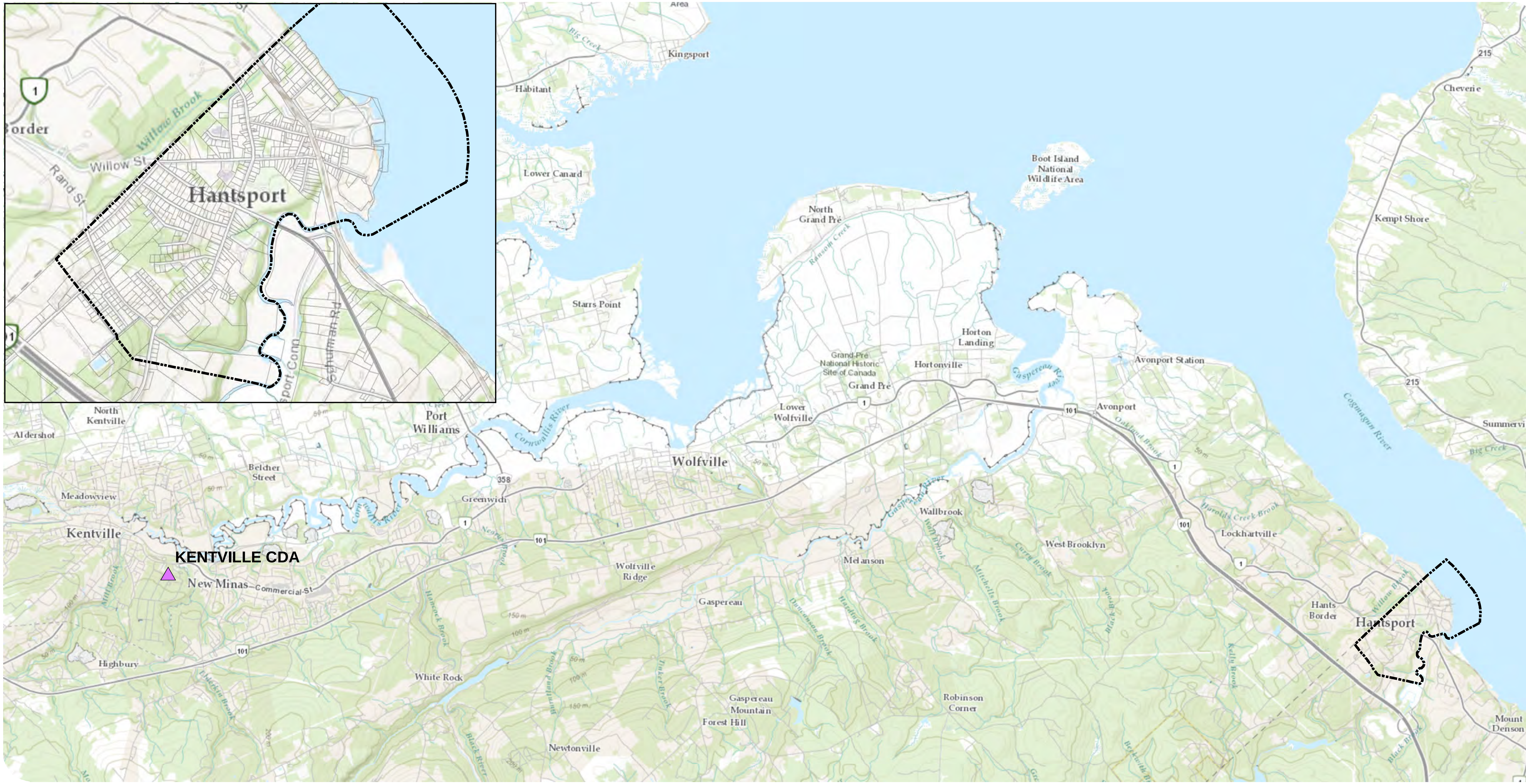
2.0 Background

The Community of Hantsport is located along the west coast of mainland Nova Scotia in the vicinity of the Avon River estuary near the Bay of Fundy. The general location of Hantsport and municipal boundaries are presented in Figure 2-1. The community has a population of approximately 1,200 people and is home to the CKF Food Services and Packaging Plant which is a significant employer in the area.

Historical significant flooding within the Community is limited to reports of localized flooding along the route connecting NS Highway 101 (Exit 8) to Route 1. The most recent instance of flooding was reported in late September 2017 where Exit 8 was closed and traffic was redirected. This flooding is probably the result of increased levels along the Halfway River. Furthermore, this section of highway near the Halfway River has been identified as at risk due to the effects of storm surge and projected sea level rise (Proosdij, 2009). Additionally, drainage issues have been noted throughout the Community at the following locations:

- Chittick Avenue;
- Riverview Road;
- Avon Street (no existing drainage infrastructure); and,
- Maple Drive.

Of special note, the existing aboiteau located at the mouth of Halfway River was reported to be in disrepair (Proosdig, 2009), allowing for partially restricted tidal flow to propagate up the Halfway River. In the fall of 2017 the Municipality reported that the aboiteau had completely collapsed. A photograph of the post-failure aboiteau taken on November 22 is presented in Figure 2-2. The failure of the



MUNICIPALITY OF THE DISTRICT OF WEST HANTS

HANTSPORT SWM STUDY

REGIONAL CONTEXT

FIGURE 2-1

WEST HANTS NOVA SCOTIA

KENTVILLE CDA METEOROLOGICAL STATION (ID: 8202800)

MUNICIPAL BOUNDARY

LAND PARCEL

DILLON CONSULTING

MAP DRAWING INFORMATION:
DATA PROVIDED BY ESRI & WEST HANTS

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

0 1 2 4 Kilometers

SCALE 1:30,000

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

PROJECT: 17-6276 STATUS: FINAL DATE: 18/04/27

aboiteau is expected to impact flood levels along the Halfway River, potentially increasing the effects of storm surge and/or restricting drainage along the Halfway River watershed to the Bay of Fundy.

Figure 2-2: Photograph of Post-Failure Aboiteau near Hantsport (taken November 22, 2017)

Drainage infrastructure within the Community consists of a combination of storm and combined sewer, along with numerous semi formalized overland drainage routes. Many of the roadways within the Community do not appear to be constructed as a dual-drainage network (i.e. provision for major/minor drainage). However some of the recently upgraded road sections include new curb and gutter and would be expected to provide a degree of minor and major flow conveyance. Examples of these road sections within the Community are presented in Figure 2-3.

Dual drainage networks are generally designed such that stormwater runoff is evacuated from the drainage area and conveyed through a combination of overland infrastructure, such as roadways (i.e. the major system), and sewer infrastructure (i.e. the minor system). The minor system is commonly sized to accommodate a 1:5 year or 1:10 year flood without surcharging into the roadway. For more intense flood events (e.g. the 1:100 year flood), the sewers would be expected to surcharge to the roadway and combined with excess flow from within the right of way (ROW) would be conveyed by the roadway. The roadway section is expected to provide safe conveyance without flooding areas outside the road ROW. Maximum ponding depths along the roadway are commonly designed to be no greater than ~ 0.3 m during a 1:100 year flood. The advantage of a dual-drainage system is that the size of underground sewers can be significantly reduced (i.e. limited to 1:5 or 1:10 year level of service) and not require significant stormwater management facilities.

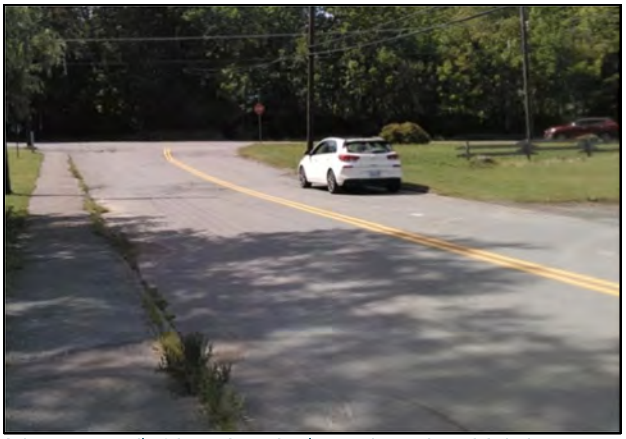
 

Figure 2-3: Photos of Recently Upgrade Roadway on Chittick Avenue (left) and typical Road section (right)

Referring to Figure 2-3, the recently upgraded roadway section (e.g. Chittick Avenue) would be expected to perform as a dual-drainage network. However, the older road sections without curb and gutter would be expected to spill onto private property once the capacities of the catchbasins and/or sewer are exceeded. This may result in uncontrolled overland flow routes, potentially impacting low-lying private property.

To address flood risk within the Community and to identify mitigation options to improve drainage, an assessment of existing stormwater infrastructure has been undertaken. This assessment seeks to identify both minor and major drainage routes within the study area and to identify hydraulic constrictions (i.e. bottlenecks) where surcharging occurs. Numerical simulation of design rainfall events has been undertaken to facilitate this assessment.

This study has also evaluated, at a conceptual level, the flood risk posed by the Halfway River as a result of the recent aboiteau failure.

3.0 Existing Drainage Network

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 the fall of 2017. 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 drainage model (described in subsequent sections of this report). A summary of each site visit is provided in the sections below.

3.1.1 September 5th – 8th, 2017

The first site visit took place on September 5th with objectives including the installation of a water level logger, and a targeted survey of the drainage network. The water level logger was installed along

Halfway River downstream of the Highway 1 bridge crossing (see Figure 4-3). The purpose of the logger was to record a period of water level fluctuations and obtain an understanding of the tidal influences on the lower river reach. The targeted survey investigated areas that have historically exhibited drainage issues, as discussed in the kickoff meeting which took place on August 29th. These areas include the following:

- Chittick Avenue;
- Riverview Road;
- Avon Street (no existing drainage infrastructure); and,
- Maple Drive.

A detailed survey was completed along Chittick Avenue on September 5th. The street consists of a curb/gutter and piped system. Dimensions and measurements were also collected along the Willow Brook crossing on Main Street.

The second site visit (undertaken on September 8th) involved a continued investigation and survey of the areas noted in the kickoff meeting. The ditch at the southern end of Riverview Road was noted as historically exhibiting drainage issues. Field observations revealed the ditch to be overgrown with an outlet pipe that was partially blocked due to soil/debris, leading the potential for roadway flooding.

3.1.2 October 4th, 2017

The site visit undertaken on October 4th involved relocation of the water level logger along with a survey of the select sites within the Community. The water level logger was relocated to Willow Brook slightly upstream of the Tannery Road culvert crossing (see Figure 4-3) to collect data for rainfall events to calibrate the PCSWMM model (described further in Section 4.5).

This survey focused on manhole inspections to investigate both their condition as well as obtain estimates of pipe diameters and depth. The purpose of the survey was also to confirm drainage routes due to limited records available for the drainage network. Based on this survey it was determined that the majority of the Community discharges to Willow Brook while the remainder discharges to Halfway River or directly to the Avon River. The location of these rivers can be seen in Figure 4-1.

Based on the site visits conducted in September and October it was observed that several catchbasins throughout Hantsport were full of leaves and debris significantly limiting the inlet capacity of the catchbasins which can potentially lead to roadway flooding. Performing regular maintenance flushing and cleaning would reduce this risk. Streets that contained a significant number of blocked structures included:

- Riverview Road;
- Willow Street;
- Maple Avenue;
- Alders Avenue;

- Cedar Avenue; and,
- Northern portion of Rand Street.

3.1.3 November 22nd, 2017

The site was visited to remove the water level logger from the Willow Brook monitoring location (deployed October 4th) and to undertake additional targeted survey of select drainage infrastructure. The survey was intended to infill some of the data gaps identified during model development.

A progress meeting with the Municipality was also organized during this visit. The meeting entailed discussions concerning the preliminary simulation results for existing conditions. At the time, municipal staff also provided an account of anecdotal flooding reports within the community during this meeting. The following areas were noted as areas of concern for flooding:

- Chittick Avenue – existing ditch located adjacent to a dentist office has caused localized flooding in the past;
- Chittick Avenue – flooding in the vicinity of the Community municipal storage yard has been reported;
- Library/Playground located along Main Street has experienced flooding in the past;
- The section of Cottage Street near the Soccer Field and adjacent to the CKF plant;
- Localized flooding at the top of Riverview Road has been reported; and,
- Flooding along Chestnut Avenue has been reported in the past – however, a new French drain was constructed and is believed to have mitigated the issue.

In addition to the above historical flood reports, the Municipality also identified the following drainage concerns:

- Ball Field located at the end of Porters Avenue has been used as a snow dump in the past and has been the site of bank failures; and,
- The failure of the Halfway River aboiteau was discussed, and noted as an issue of particular concern to Municipal staff.

3.2 Hydrometric Observations

3.2.1 Halfway River (September 5th – October 4th)

The water level data at the Halfway River monitoring station were collected and readings were compensated for changes in barometric pressure. A summary of water level fluctuations during the observation period is presented in Figure 3-1. Water levels at the observation site are clearly tidally influenced, and are indicative of the semi-diurnal tide cycles dominate in the Bay of Fundy. Figure 3-1 also includes the coincident observations at the Department of Fisheries and Oceans (DFO) gauge located in Saint John, NB.

It is noteworthy that the Halfway River level logger is not submerged during periods of low tidal fluctuations; this can be seen in Figure 3-1 where the level observations have a minimum value of approximately 2.5 m. Given the significant tidal range, the level logger could not safely be installed in a location that allowed for capture of the complete tidal cycle.

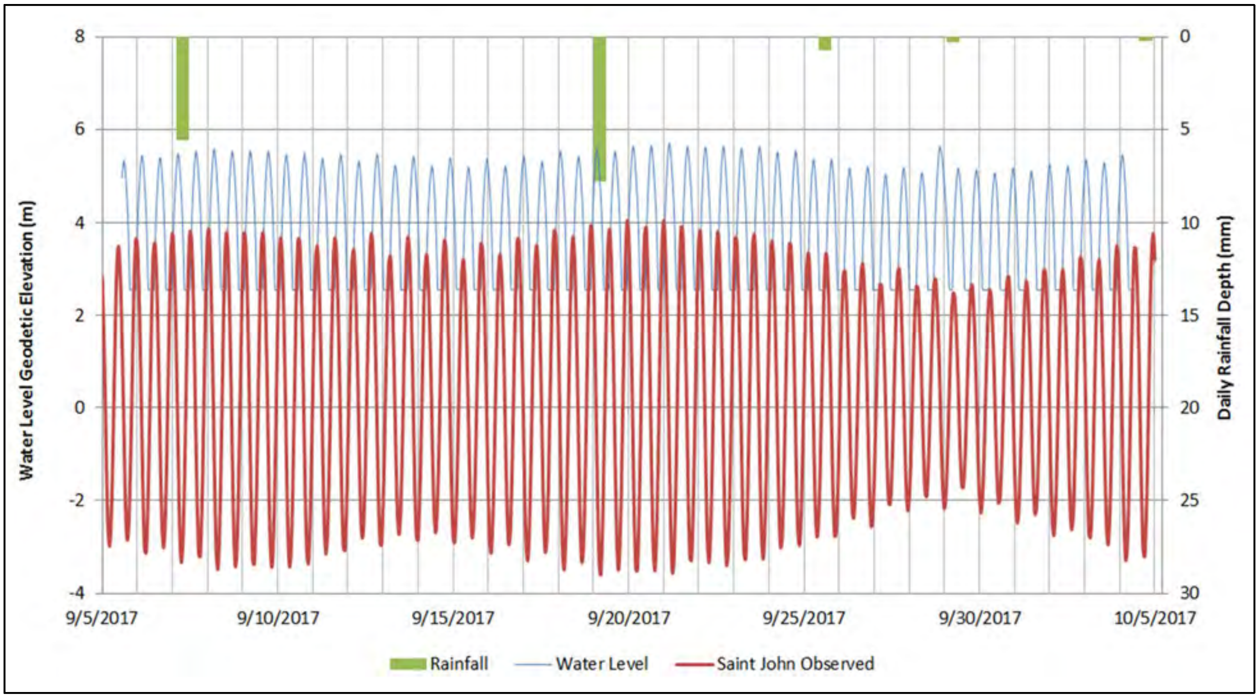


Figure 3-1: Halfway River Water Level Observations

It can be seen in the above figure that variation between the Fisheries and Oceans Canada (DFO) and Halfway River observations is evident. A refined view of a typical 24-hour observation period is also presented in Figure 3-2 to further demonstrate this deviation. This variation suggests that the Halfway River water level is frequently more than 2 m higher than levels in the Bay of Fundy. There is also a delay of roughly 3.5 hours between the high water level in the Halfway River compared to the Bay of Fundy at Saint John. Some of this variation is presumably due to the considerable distance between the two gauges (~150 km) and the backwatering effects along the channels; however the impact of runoff from the Halfway River and hydraulic constriction at the aboiteau is also a possible factor.

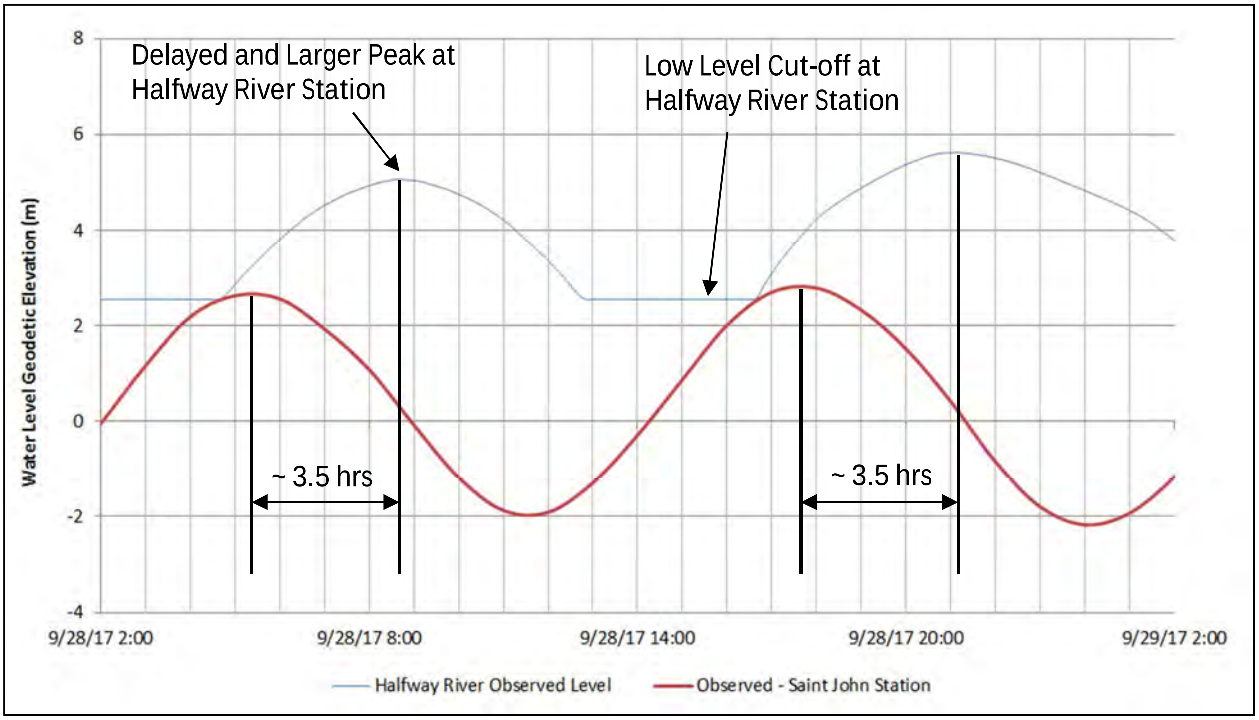


Figure 3-2: 24-Hour Water Level Observation Summary

It is important to note that the typical tidal fluctuation (high to low) along the Halfway River was significant (>2 m) leaving the monitoring station unsubmerged at times during the tidal cycle; as discussed above, this results in a low level cut-off of approximately 2.5.

It is noted that these observations were made prior to the recent failure of the Halfway River aboiteau. The failure has created a free outlet to the Avon River. It is expected that the current condition of the aboiteau (see Figure 2-2) could potentially increase flood risk associated with storm surge from the Bay. However the free outlet may slightly decrease flood risk associated with high flows in the Halfway River, since discharges to the Avon River are no longer restricted by the capacity of the aboiteau.

3.2.2 Willow Brook (October 4th – November 22nd)

As noted previously the water level logger was relocated on October 4th from the Halfway River to Willow Brook. The location of the logger is presented in Figure 4-3. A photograph of the Willow Brook level logger station is presented in Figure 3-3. Willow Brook receives runoff from a substantial portion of the Hantsport study area and therefore provides a suitable model calibration/validation point for evaluating peak flows within the watershed.

The logger was installed immediately upstream of the existing pre-cast concrete box culvert structure conveying flows under Tannery Road. The conveyance structure provides a controlled flow section, thus allowing for a simple estimation of the discharge through the structure using the observed water levels recorded upstream.

A summary of water level observations and rainfall records from the Environment Canada Kentville climate station (#8202810) are presented in Figure 3-4. The two largest rainfall events occurred on October 26 and 30th with 14.9 and 15.7 mm of daily rainfall reported, respectively. While higher intensity rainfall events (50+ mm) would provide a stronger indication of rainfall-runoff response, these two events were the highest observed daily rainfall accumulations during the monitoring period. These rainfall data will be used to support subsequent model validation calibration (Section 4.5).

Figure 3-3: Level Logger Deployment in Willow Brook (left), and Downstream Open Bottom Pre-Cast Structure Facing Downstream (right)

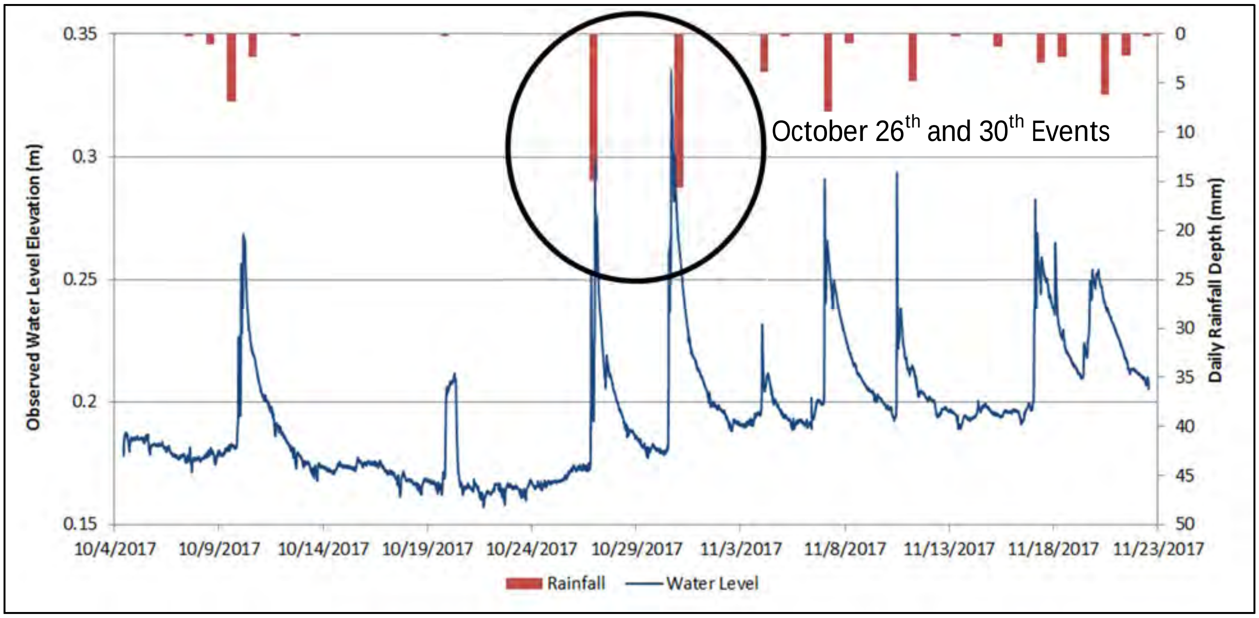


Figure 3-4: Willow Brook Water Level Observations (Geodetic Elevation)

3.2.3 Flood Frequency Analysis

A review of other 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. 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 - 2013	44	63.2
Great Village River near Scrabble Hill	01DJ005	1993 - 2013	21	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 >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 event to obtain a relationship between peak flow and drainage area for the west coast of Nova Scotia. Using these relationships, the theoretical peak flow for each return period flood may be estimated for watersheds of various sizes within the west coast of Nova Scotia. The log-log graph of the six regional stations is presented in Figure 3-5.

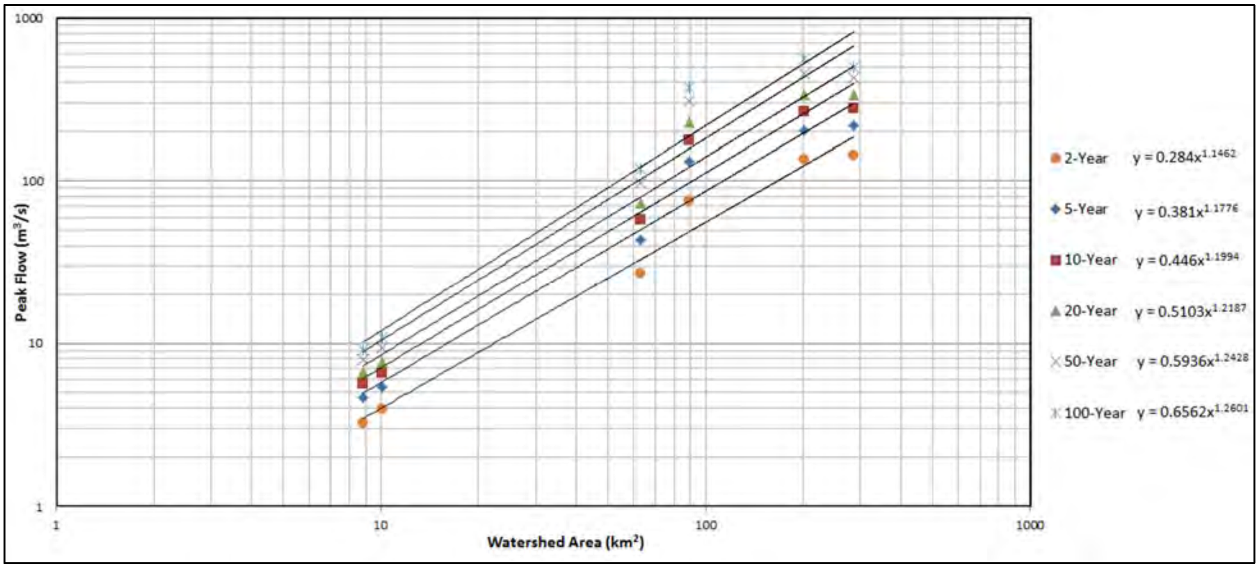


Figure 3-5: Regional Frequency Analysis Theoretical Peak Flow Plot

Table 3-2 presents a summary of the estimated peak flow values for Willow Brook and Halfway River using regional frequency analysis approach. The total drainage areas for Willow Brook and Halfway River were approximately 3.30 km^2 and 201 km^2, respectively.

Table 3-2: Regional Flood Frequency Peak Flow Estimates for Willow Brook and Halfway River

Return Period (Years)	Regional Flood Frequency Results for Willow Brook (m^3/s)	Regional Flood Frequency Results for Halfway River (m^3/s)
100	201	2.30
50	168	2.00
20	129	1.70
10	104	1.50
5	80.1	1.20
2	51.7	0.90

It is noteworthy that flows in Willow Brook are expected to be higher than above due to the degree of urbanization in the Willow Brook catchment area.

4.0 Hydrologic and Hydraulic Analysis

Hydrologic/hydraulic modelling was undertaken to simulate the performance of the current 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

4.1.1 Catchment Parameters

4.1.1.1 Catchment Delineation

Catchment delineation was initially completed using the existing LiDAR data (obtained from GeoNova) for the Hantsport study area. The initial coarse sub-catchment delineation was further refined based on the observations made during the site visits. The basin was delineated into 56 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. This is often impacted by impervious covers, such as asphalt, concrete and building roofs. 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).

Losses of precipitation due to infiltration, depression storages and interception in the sub-catchments were incorporated into the model through CN values. A review of topography within the study area was also undertaken to estimate the effective runoff from each sub-catchment. This review identified significant natural landform barriers and/or depressions that diverted or collected large amounts of surface runoff. These estimates were then included in the simulated sub-catchments.

A summary of the runoff parameters assigned to each sub-catchment is provided in Table A-1 in Appendix A.

4.1.2 Meteorological Inputs

Rainfall intensity-duration-frequency (IDF) statistics for the Kentville Environment Canada climate station (#8202800) were retrieved. The Kentville station is approximately 25 km from Hantsport with about 32 years (1960 – 1995) of continuous climate observations, including precipitation. Estimated existing rainfall depths for a range of short duration (24-hour) rainfall events are presented in Table 4-1.

A potential future rainfall climate change scenario has also been 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 intensity. A comparison of historical and future rainfall for the Kentville station is also presented in Table 4-1.

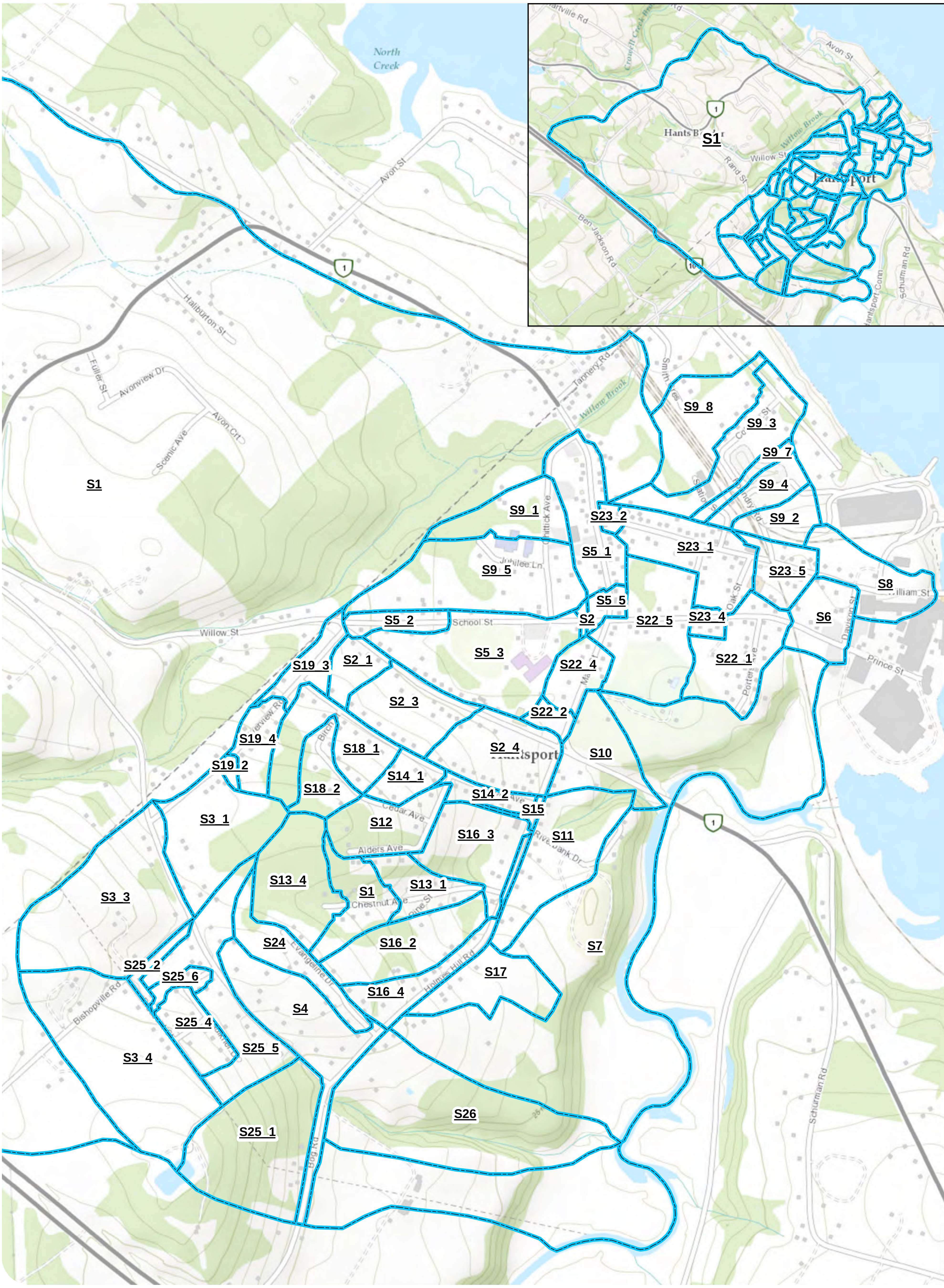
Table 4-1: Summary of Historical and Estimated Future Rainfall Depth for 24-hour Rainfall Events at Kentville, NS

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%

[1]Assumes Moderate Emissions Scenario – RCP 4.5

It can be seen that for a range of return periods, and for a storm duration of 24-hours, an increase of roughly 14 – 30% is expected under estimated future conditions. It is noteworthy that the largest increase in total rainfall accumulation is associated with the lowest frequency event (i.e. 100-year return period). The implications of such increases in rainfall intensity can be significant and must be considered when assessing drainage under future scenarios.

The total 24-hour rainfall depths presented in Table 4-1 were distributed using the Alternating Block Method (Chow, 1988). The Alternating Block Method allows for the development of a design hyetograph from regional IDF statistics. The rainfall time series generated includes rainfall accumulation for the 10-minute, up to the 24-hour storm duration. Thus, the Alternating Block Method allows for some flexibility with respect to the estimation of lag time within the watershed. An example of this distribution for the 100-year historical rainfall event is presented in Figure 4-2.



MUNICIPALITY OF THE DISTRICT OF WEST HANTS
HANTSPORT SWM STUDY

SUB-CATCHMENT DELINEATIONS
FIGURE 4-1

SUB-CATCHMENT
S1 SUB-CATCHMENT IDENTIFIER FOR PCSWMM MODEL

MAP DRAWING INFORMATION:
DATA PROVIDED BY ESRI & WEST HANTS

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

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

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SCALE 1:7,500

PROJECT: 17-6276 STATUS: FINAL DATE: 18/04/27

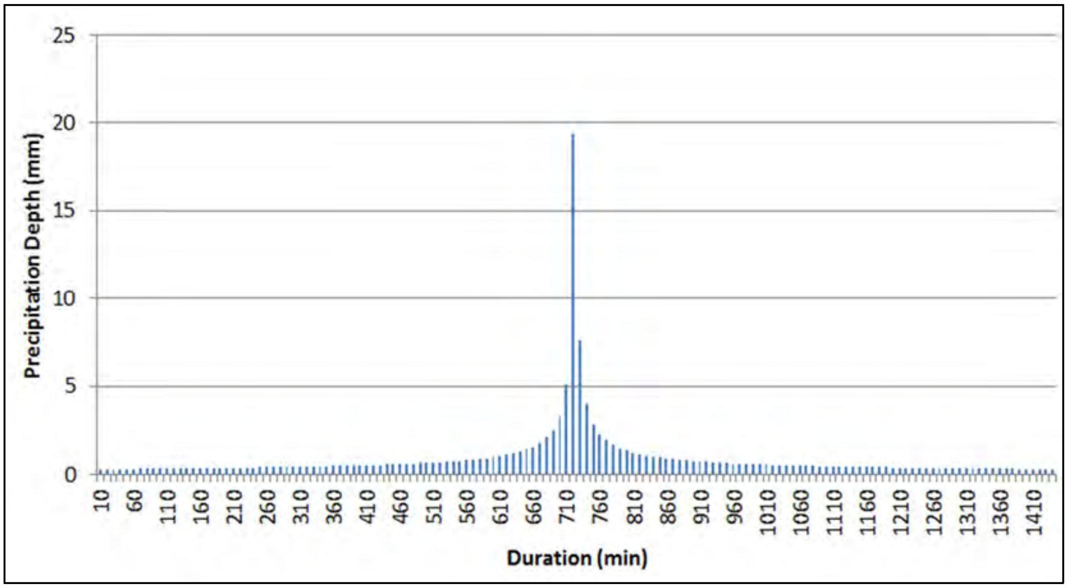


Figure 4-2: 24-Hour Alternating Block Method Distribution for 100-year Historical Event

4.1.3 Water Level Data

The water level boundary conditions were determined by reviewing the observed data along the Halfway River, and comparing to observed water levels in the Bay of Fundy (DFO station #65 at Saint John), see Figure 3-2. The confluence of the Halfway and Avon rivers is located approximately 7 km from the Minas Basin (Bay of Fundy); the downstream boundary condition for the study area is located approximately 0.5 km upstream of the confluence. Due to the proximity of the study area to the Minas Basin, it has been assumed that water levels in lower Halfway River are tidally influenced, as demonstrated in Figure 3-2. The tidal outlet boundary conditions considered in model development include the ditch outlets near Mariner's Drive and adjacent to NS Route 1.

Generally, an ordinary high water condition would be expected to coincide with a typical spring tide for the area. A moderate high water condition was determined to be comparable with the 10-year water level estimated using frequency analysis of the water level observations at Saint John. The frequency analysis was completed using 41 years (1977 – 2017) of complete record.

The estimated extreme tide condition was determined by obtaining the 100-year water level and adding a value of 0.5 m for storm surge (assumed typical surge for extreme event). The boundary conditions considered for this assessment are summarized in Table 4-2. The boundary conditions assume that the recently failed aboiteau provides no protection from storm surge; rehabilitation of the aboiteau would presumably reduce impacts of storm surge.

In addition to changes in future precipitation intensity, sea levels are also expected to increase along the coastal areas of the Bay of Fundy (Daigle, 2011). The estimated future projected sea level rise for over the next 80 years is approximately 1.10 m ($^+/_-$ 0.48 m) for coastal sections of Hantsport according to Daigle (2011). The estimated future projected water level elevation is presented in Table 4-2.

Table 4-2: Downstream Boundary Condition Summary for Study Area

Event	Peak Water Level Geodetic Elevation(m)
Typical Boundary Condition (normal spring tide)	4.60
Moderate Boundary Condition (1 in 10 year)	4.77
Extreme Boundary Condition (1 in 100 year + 0.5 m surge)	5.43
Future Projected Boundary Condition – Year 2100 (1 in 100 year + 0.5 m surge)	6.53

The estimated water levels presented above have been used as downstream boundary inputs for the hydraulic model. It is important to note that the water levels presented in Table 4-2 have been increased by 2 m in the hydraulic model to remain consistent with the difference in water level observations collected at the Saint John station and the Halfway River monitoring station (see Figure 3-3).

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 Hantsport with six (6) rainfall events.

The drainage network observed during the site visits in September and October was used to develop the model in PCSWMM for analysis. The drainage network in the Community of Hantsport consists of combined sewers, storm sewers, culverts and open ditches/watercourses. It is important to note that the PCSWMM model assumed the drainage network to be operating under ideal conditions (i.e. manholes have been cleaned and sewers have been flushed). A visual of the Community's drainage network is presented in Figure 4-3.

Combined sewers, which convey both storm and sanitary flows, run along six (6) streets within the Community which are listed below:

- Willow Street
- Prince Street;
- William Street;
- A portion of Main Street; and,

- Davidson Street.

The storm water makes its way into the system through catchbasins which connect laterally to the trunk sewer. Based on the site visits that took place in September 2017 and October 2017, these combined sewers eventually discharge to open watercourses included Halfway River and the Avon River.

Dedicated storm sewers are the primary method of storm water conveyance in Hantsport. Storm sewers convey all storm water from the residential lands located southwest of Willow Street as well as School Street, Chittick Avenue and Main Street. The majority of these storm sewers discharge to Willow Brook through open ditches.

The peak sanitary flow in the combined sewer systems were estimated using the Atlantic Canada Wastewater Design Guidelines (2006). These calculations assumed an average number of persons per dwelling of 2.3 based on the findings of the 2011 Census of Population and Statistics Canada. The Willow Street combined sewer was assumed to convey wastewater from 30 residential dwellings with an estimated peak daily flow of 1.0 L/s. The combined sewer system encompassing Prince Street, William Street, and Main Street assumed wastewater conveyance from 100 residential dwelling with an estimated peak daily flow of 4.0 L/s. It is important to note that these peak sanitary flows are small when compared to the estimated stormwater runoff flows.

It is important to note that the drainage network was not surveyed in its entirety. A targeted survey of the network was completed during the site visits carried out between September 2017 and November 2017 (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:

- Bridge slab thickness of 0.7 meters (Main Street and Tannery Road);
- Sewer grades equal to the grade of the roadway or a minimum of 1.0%;
- Watercourse and ditch centerlines that weren't collected by high resolution GPS have been estimated from aerial imagery;
- Watercourse and ditch cross sections have been estimated from the LiDAR; and,
- Catchbasin leads that were not surveyed are assumed to be 200mm diameter concrete pipe.

The assumptions described above were considered acceptable based on field observations noted during the site visits in the fall of 2017.

The following additional assumptions were also necessary to estimate catchment parameters:

- Hantsport soil has been given a Hydraulic Classification B - based on provincial surficial geologic mapping;
- Manning's roughness value (n) for concrete pipes and PVC pipes is 0.015 and 0.010, respectively (Chow, 1959); and,

- Effective catchment areas were used in the model; these reduced catchment areas were calculated assuming low lying areas did not contribute stormwater runoff to the drainage system. A topographical review revealed these low lying areas.

4.3 Hydraulic Model Calibration

4.3.1 Willow Brook

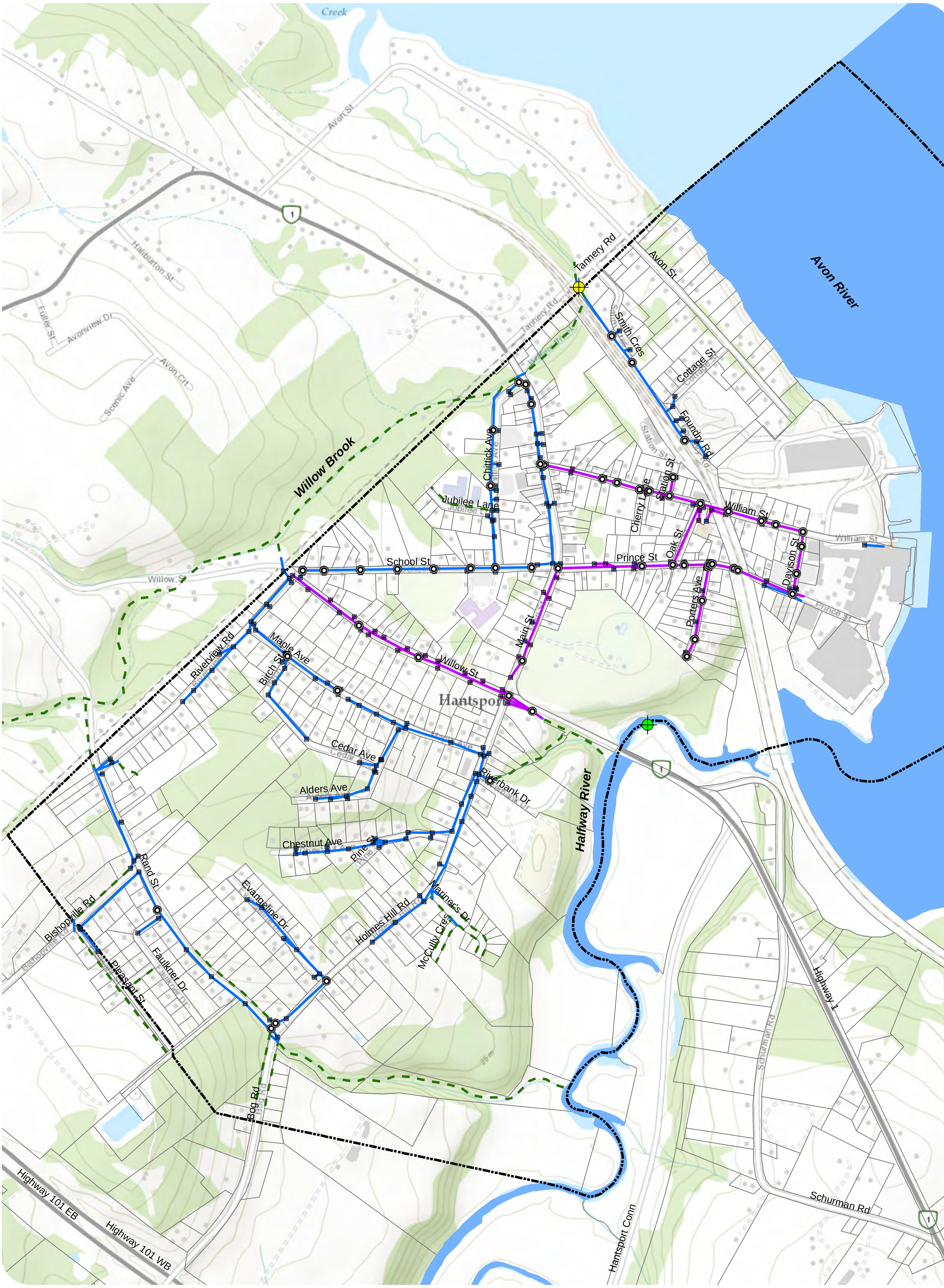
The water level observations discussed in Section 3.2.2 were used to estimate flow through the concrete box culvert at the Tannery Road crossing of Willow Brook. The Manning Equation was used to develop a rating curve for the Tannery Road crossing using the following parameters, collected during field survey and derived from LiDAR:

- Average bed slope of 3.6 %;
- Culvert Opening width of 3.66 m;
- Upstream Culvert Invert Elevation of 9.84 m;
- Downstream Culvert Invert Elevation of 9.86 m.

The Manning Equation assumes that flow is unobstructed by drainage infrastructure such as, bridges and culverts. This assumption was considered acceptable due to the low water levels associated with the daily rainfall on October 26th and 30th (14.9 and 15.7 mm, respectively). The recorded water levels for the October 26th and 30th rainfall events were approximately 0.30 and 0.34 m, corresponding to an HGL elevation of 10.14 and 10.18 m, respectively. Using the Manning's Equation the discharge during the two October events was estimated to be approximately 0.65 and 0.80 m^3/s.

A comparison of the simulated and observed runoff hydrograph for Willow Brook at the Tannery Road crossing during the October 30th event is presented in in Figure 4-4. The model results at this crossing encompass discharge generated from a large portion of the study area including sections of Rand Street, Riverview Road, Chittick Avenue, Main Street and Foundry Road (~0.6 km^2).

Observed rainfall depths at the Kentville climate station were requested from Environment Canada, and included 15-minute rainfall depth observations. These rainfall data were used to create a rainfall timeseries in the PCSWMM model.



MUNICIPALITY OF THE DISTRICT OF WEST HANTS
HANTSPORT SWM STUDY

EXISTING DRAINAGE NETWORK
FIGURE 4-3

WEST HANTS
NOVA SCOTIA

STORM SEWER	CATCHBASIN
COMBINED SEWER	MANHOLE
WATERCOUSE AND/OR OPEN DITCH	MUNICIPAL BOUNDARY

LAND PARCEL
HALFWAY RIVER WATER LEVEL LOGGER
WILLOW BROOK WATER LEVEL LOGGER

DILLON CONSULTING

MAP DRAWING INFORMATION:
DATA PROVIDED BY ESRI & WEST HANTS

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

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

0 50 100 200 Meters SCALE 1:7,500

N W E S

PROJECT: 17-6276 STATUS: FINAL DATE: 18/04/27

The following updates to the numerical model were made to simulate the Willow Brook response to the October 30th rainfall event:

- The PCSWMM model was updated to account for the observed baseline discharge leading up to the October 30th rainfall event (0.283 m^3/s) – this base flow was maintained for all subsequent simulations;
- The runoff curve number (CN) value was reduced to reflect the dry antecedent moisture leading up to the October rainfall events, from 65 (AMC II – Average) to 40 (AMC I – Dry) – this was in response to the unseasonably dry summer and fall months, whereby September recorded a total monthly rainfall depth of 15.9 mm compared to a historical average of 87.3 mm;
- Sections of sewer contributing to Willow Brook that were observed to be severely blocked with debris were restricted for this simulation (see Section 3.1).

The model required a slight calibration based on the comparison of observed and simulated flow. The depression storage values for pervious and impervious drainage areas were increased to 5 mm and 1 mm, respectively. This reduced the simulated peak flow slightly to be more in line with the observed peak flow. The simulated hydrograph in Figure 4-4 is the calibrated simulation results.

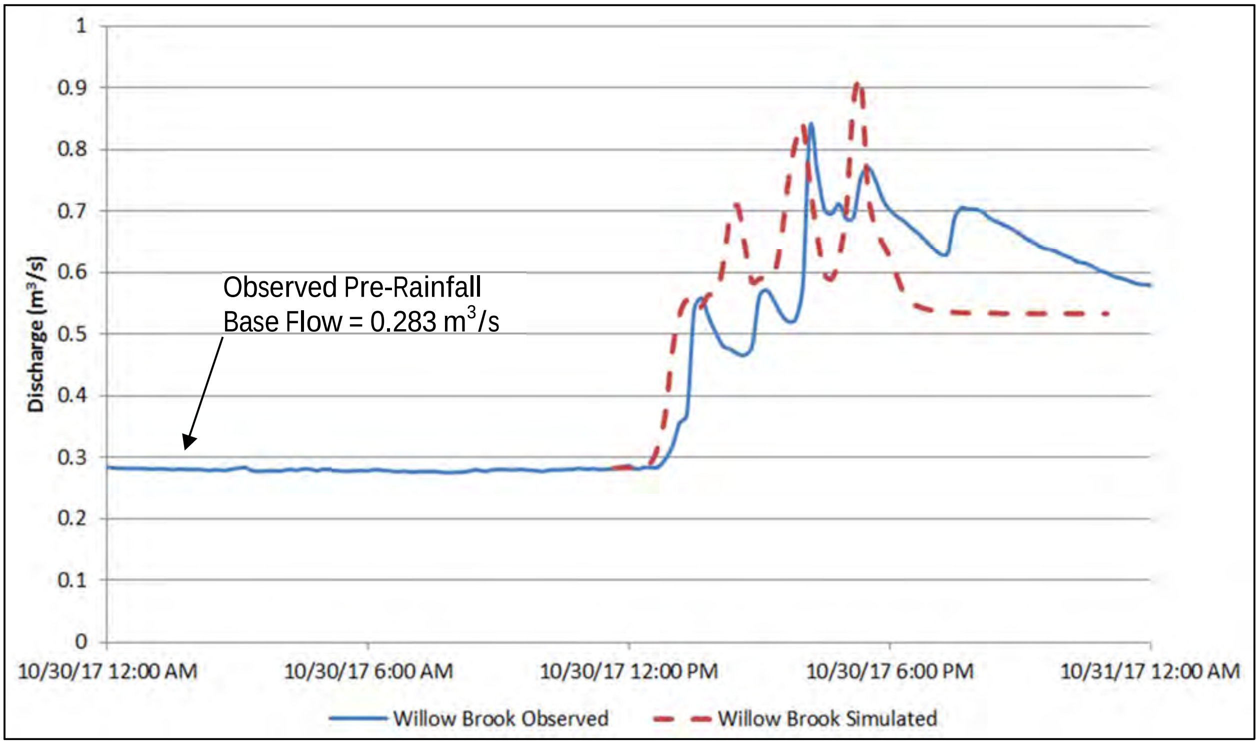


Figure 4-4: Comparison of Willow Brook Simulated and Observed Hydrograph for October 30th, 2017 Event

The simulated peak flow was approximately 0.08 m^3/s (+9.5%) greater than the observed peak value. It is expected that there was a lag between the rainfall recorded in Kentville and that which fell within the Willow Brook gauge watershed. Therefore the beginning of the October 30th rainfall event was delayed by 1.5 hours to compensate for the distance between Kentville and the study watershed (25 km). This delay allowed for the simulated and observed runoff responses to occur at roughly the same time.

It is important to note that the Halfway River was not used for model calibration. The Halfway River watershed is large (~201 km^2) resulting in significant flows (as shown in Section 3.2.3). The Halfway River conveys approximately 1.1 km^2 of stormwater runoff from within the Community. The stormwater runoff volumes from the Community are expected to have negligible impacts on water levels in the Halfway River. By comparison, the Willow Brook watershed (~3.3 km^2) conveys approximately 0.6 km^2 of stormwater runoff from within the Community. Willow Brook was deemed more suitable for model calibration.

5.0 Simulation Results

The following sections present the simulation results for the Hantsport drainage network under both existing conditions and future development conditions. The drainage network was assessed for six (6) 24-hour rainfall events with return periods ranging from two (2) years to 100 years. For the purposes of summarizing these results, the drainage network has been divided into eight (8) areas based on outlet location (see Figure 5-1).

Tables 5-1 through 5-20 provide the estimated water level elevations for the 5-year and 100-year return period events at areas noted as “drainage challenges” for three (3) different simulation scenarios. It is important to note that values in red indicate that water levels surcharge above the ground surface. The simulation results for all rainfall events are provided in Appendix B. Simulated water level profiles for select areas are presented in Appendix C, with peak flow results summarized in Appendix F.

The maximum ponding depth considered for the PCSWMM simulation was 0.3 m. A review of stormwater management guidelines in other municipalities suggests that 0.3 m is a common maximum for acceptable ponding depth in major drainage networks for the 100-year flood condition (e.g. the cities of Waterloo and Windsor, Ontario). It is noted that any amount of roadway ponding during the 5-year flood condition 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.

Generally, the maximum roadway ponding assumption was not applicable to roadways with curb and gutter, and sufficient grade to convey storm flows within the roadway. Accordingly, the maximum ponding depth of 0.3 m was most relevant to areas where road grades suggest the potential for the accumulation of runoff (e.g. accumulation of runoff in road sags).

5.1 Existing Performance

5.1.1 Downtown – Chittick Avenue, Main Street & School Street

The downtown core of Hantsport is comprised of three main streets which include School Street, Chittick Avenue, and Main Street. Drainage is conveyed through a series of storm sewers and ultimately discharges to Willow Brook at the Main Street bridge crossing.

Simulations of the downtown area suggest that the system conveys runoff for the 5-year rainfall event with only minor ponding in localized areas along Chittick Avenue, School Street, and Main Street. Under more extreme rainfall events (i.e. 100-year), the model shows increasing flooding along School Street and Main Street. Flood levels are most concerning near the Main Street Bridge crossing.

Drainage issues in this area are presumably highly influenced by the water levels in Willow Brook. Estimated peak water levels along Chittick Avenue, Main Street, and School Street are presented in Table 5-1. Simulated water levels profiles along Chittick Avenue and Main Street are provided in Appendix C.

Table 5-1: Summary of Simulated Water Level Elevations for Existing Conditions – Downtown

Location	Existing Development Conditions Peak Water Level (m)	
Return Period Rainfall Event	5 yr	100 yr
Main Street – Civic #25 (CB98)	18.16	18.38[2]
Main Street – Civic #39 (CB93)	18.27	19.38[2]
Main Street Bridge (J46)	14.38	15.75[1]
Chittick Avenue – Civic #22 (J1)	14.57	15.01[2]
Chittick Road - Civic #47 (CB84)	14.87[2]	14.87[2]
School Street – Civic #12 (J26)	16.68[2]	16.68[2]

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

5.1.2 Riverview Road, Maple Avenue & Birch Street

The area comprised of Riverview Road, Maple Avenue, and Birch Street, conveys runoff through a series of storm sewers. The system outlets to Willow Brook at the west end of School Street.

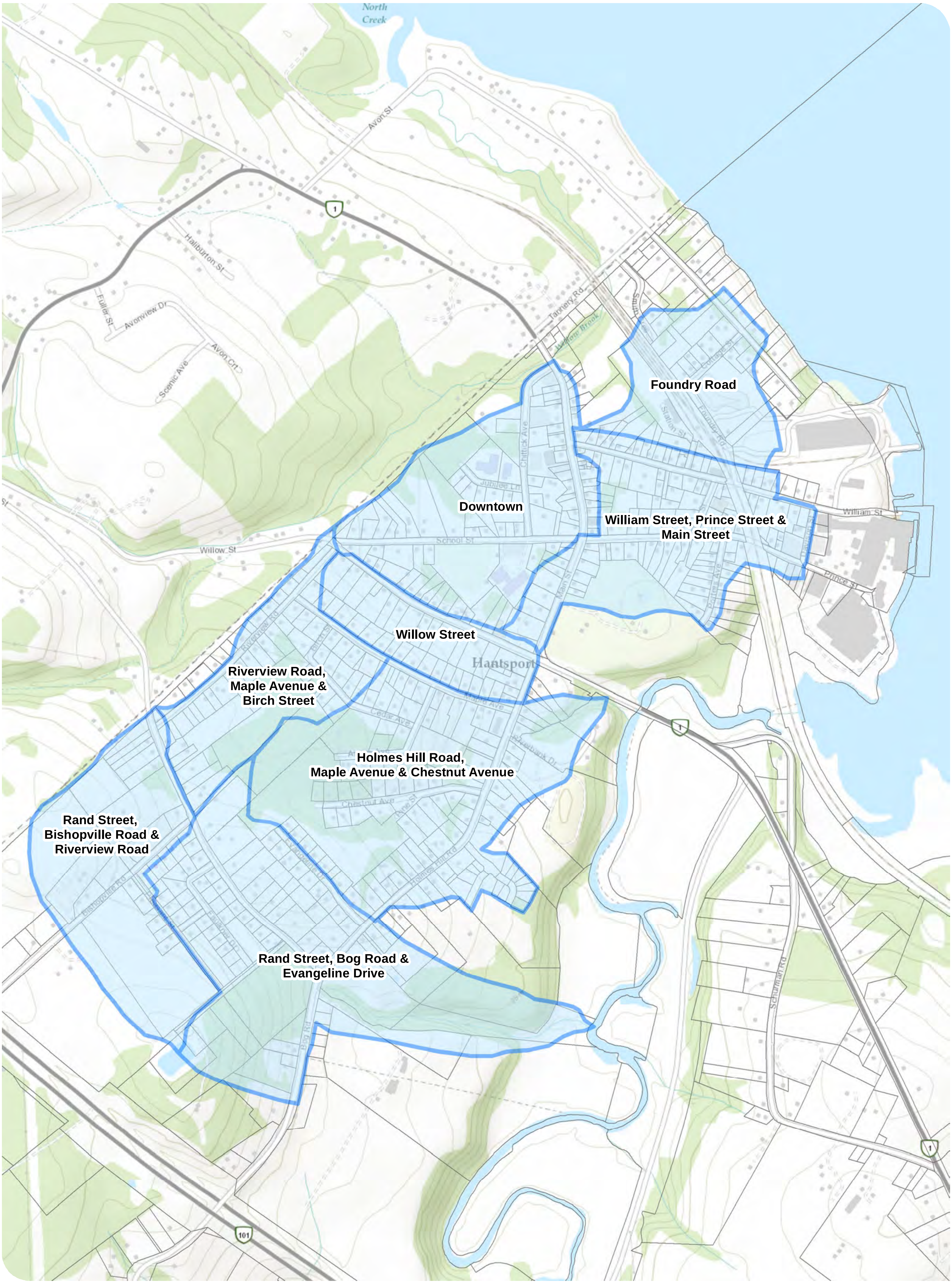
Simulation of the Riverview Road/Maple Avenue area suggests flooding during the 5-year rainfall event along areas of Maple Avenue and Riverview Drive in the vicinity of Civic #23. The flooding along Riverview Drive is presumable a result of the undersized (100 mm) diameter storm sewer lead from CB168. Estimated peak water levels at this location are presented in Table 5-2. Simulated water levels profiles along Riverview Road and Maple Avenue are provided in Appendix C.

Table 5-2: Summary of Simulated Water Level Elevations for Existing Conditions – Riverview Road, Maple Avenue & Birch Street

Location	Existing Development Conditions Peak Water Level (m)	
Return Period Rainfall Event	5 yr	100 yr
Maple Avenue – Civic #24 (J77)	40.71[2]	40.71[2]
Maple Avenue – Civic #30 (MH169)	40.39	40.52[2]
Riverview Road – Civic #8 (CB158)	36.84	37.45
Riverview Road – Civic #23 (CB168)	54.42[2]	54.42[2]

[2]Roadway ponding greater than 0.3 m in depth.

No significant challenges to adequate drainage were noted in this area.



MUNICIPALITY OF THE DISTRICT OF WEST HANTS

HANTSPORT SWM STUDY

SIMULATION RESULTS: CATCHMENT AREAS

FIGURE 5-1

CATCHMENT AREA - SIMULATION RESULTS

LAND PARCEL

MAP DRAWING INFORMATION:
DATA PROVIDED BY ESRI & WEST HANTS

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

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

SCALE 1:8,000

0 50 100 200 Meters

PROJECT: 17-6276 STATUS: FINAL DATE: 18/04/27

5.1.3 Foundry Road

Foundry Road conveys stormwater runoff from Cottage Street and Avon Street. The system picks up stormwater runoff from Smith Crescent before discharging to Willow Brook near the Tannery Road bridge crossing.

Simulation of the Foundry Road area suggests that the system does not effectively convey stormwater runoff for the 5-year storm. The model indicates roadway flooding near the soccer field and along Cottage Street. It is also important to note that the 100 mm diameter storm sewer which crosses Foundry Road in front of the Fire Station is undersized for estimated stormwater runoff volumes. The Foundry Road area is noted as an area requiring drainage upgrades to adequately convey stormwater runoff. Table 5-3 presents the estimated peak water level simulated near the soccer field on Foundry Road.

Table 5-3: Summary of Simulated Water Level Elevations for Existing Conditions – Foundry Road

Location	Existing Development Conditions Peak Water Level (m)	
Return Period Rainfall Event	5 yr	100 yr
Soccer Field (J137)	15.40[2]	15.40[2]

[2]Roadway ponding greater than 0.3 m in depth.

5.1.4 William Street, Prince Street & Main Street

The collection system encompassing William Street, Prince Street and Main Street includes primarily combined sewers with some minor storm sewer. The system converges at the intersection of Davidson Street and Prince Street and ultimately discharges to the Avon River.

As discussed in Section 4.4, peak sanitary flows were estimated for the combined sewer systems using the Atlantic Canada Wastewater Guidelines Manual (2006). The combined sewer along William Street, Prince Street, and Main Street was modelled with a peak sanitary flow of 0.004 cms assuming 100 residential dwellings. Simulations suggest that the area experiences flooding for the 5-year rainfall event particularly at the following locations:

- Main Street at the intersection with Prince Street
- Prince Street in the vicinity of Civic #11;
- William Street in the vicinity of Civic #17; and,
- William Street at the intersection with Oak Street.

In general, the locations noted above are low lying areas which will tend to collect stormwater runoff under major rainfall events. These areas are concerning as surcharging of the combined sewer system could potentially result in sanitary waste on the roadway. Table 5-4 presents the estimated peak water levels simulated at these locations.

Table 5-4: Summary of Simulated Water Level Elevations for Existing Conditions – William Street, Prince Street & Main Street

Location	Existing Development Conditions Peak Water Level (m)	
Return Period Rainfall Event	5 yr	100 yr
William Street – Civic #14 (CB57)	16.67[2]	16.67[2]
Prince Street – Civic #11 (CB40)	17.19[2]	17.19[2]
William Street – Intersection with Oak Street (CB47)	15.30	15.43[2]
Prince Street – Intersection with Davidson Road (CB23)	14.09[2]	14.09[2]

[2]Roadway ponding greater than 0.3 m in depth.

5.1.5 Willow Street

The Willow Street stormwater runoff is conveyed through a combined sewer system which outlets to Halfway River upstream of the N.S. Highway 1 bridge crossing.

The combined sewer was modelled with an estimated peak sanitary flow of 1 L/s assuming 30 residential dwellings. Simulations suggest that roadway flooding occurs for the 5-year rainfall event near the intersection with Main Street. Roadway flooding is expected to be minimized due to the existing grade along Willow Street and conveying stormwater runoff along the roadway network to Halfway River.

Table 5-5: Summary of Simulated Water Level Elevations for Existing Conditions – Willow Street

Location	Existing Development Conditions Peak Water Level (m)	
Return Period Rainfall Event	5 yr	100 yr
Willow Street – Intersection with Main Street (CB137)	19.93[2]	19.93[2]

[2]Roadway ponding greater than 0.3 m in depth.

The combined sewer system presents a drainage challenge as surcharging of the system could result in spillage of sanitary flows on the roadway.

5.1.6 Holmes Hill Road, Maple Avenue & Chestnut Avenue

The Holmes Hill Road drainage system conveys stormwater runoff from Maple Avenue, Chestnut Avenue, and Mariner's Drive, through a series of storm sewers. The system discharges to a watercourse on Riverbank Drive and ultimately discharges to Halfway River.

The model results indicate that the Chestnut Avenue storm sewer conveys stormwater flows effectively. Simulations indicate potential flooding along Holmes Hill Road for the 5-year rainfall events; the simulation also suggests minor flooding along Alders Avenue for the 5-year event. Due to the existing grade along Holmes Hill Road and Alders Avenue it is expected that any buildup of storm water flows overland eventually discharging to the open ditch along Riverbank Drive. Table 5-6 summarizes the estimated peak water levels simulated along Holmes Hill Road and Alders Avenue. Simulated water levels profiles along Alders Avenue are provided in Appendix C.

Table 5-6: Summary of Simulated Water Level Elevations for Existing Conditions – Holmes Hill Road, Maple Avenue & Chestnut Avenue

Location	Existing Development Conditions Peak Water Level (m)	
Return Period Rainfall Event	5 yr	100 yr
Alders Avenue – Intersection with Cedar Avenue (CB177)	41.94	43.57[2]
Holmes Hill Road – Intersection with Riverbank Drive (J130)	20.70[2]	20.70[2]
Mariner's Drive (J53)	33.04	34.01

[2]Roadway ponding greater than 0.3 m in depth.

5.1.7 Rand Street, Bog Road & Evangeline Drive

The drainage network encompassing Rand Street (south), Bog Road, and Evangeline Drive includes a combination of storm sewers and open ditches. The stormwater runoff from this area ultimately discharges to Halfway River.

Simulations of the area suggest that the majority of the system can convey stormwater runoff volumes for the 100-year rainfall event with minimal drainage issues. The model indicates flooding along Faulkner Drive; however, due to the relatively steep grade throughout the catchment area ponding is expected to be minimal. Table 5-6 presents the estimated peak water levels simulated on Faulkner Drive under existing conditions.

Table 5-7: Summary of Simulated Water Level Elevations for Existing Conditions – Rand Street, Bog Road & Evangeline Drive

Location	Existing Development Conditions Peak Water Level (m)	
Return Period Rainfall Event	5 yr	100 yr
Faulkner Drive – Civic #80 (CB225)	64.49[2]	64.49[2]

[2]Roadway ponding greater than 0.3 m in depth.

5.1.8 Rand Street, Bishopville Road & Riverview Road

The area comprised of Rand Street (north), Bishopville Road, and Riverview Road conveys stormwater runoff through a series of open ditches with some minor storm sewer infrastructure. The system discharges to Willow Brook.

Simulations suggest that the system experiences minor flooding for the 5-year rainfall event. The model indicates increased flooding along Pleasant Drive and Riverview Road during more extreme rainfall events (i.e. 100-year). Table 5-7 presents the estimated peak water levels simulated at these locations under existing conditions. Simulated water levels profiles along Riverview Road are provided in Appendix C.

Table 5-8: Summary of Water Level Elevations for Existing Conditions – Rand Street, Bishopville Road & Riverview Road

Location	Existing Development Conditions Peak Water Level (m)	
Return Period Rainfall Event	5 yr	100 yr
Pleasant Drive (J61)	71.27	71.55[2]
Riverview Road – Ditch Inlet (J40)	55.55	56.19[2]
Rand Street – Intersection with Bishopville Drive (CB227)	61.55	61.71

[2]Roadway ponding greater than 0.3 m in depth.

No significant challenges to adequate drainage were noted in this area.

5.2 Future Performance

Based on discussions with the Municipality of the District of West Hants on November 22nd, 2017, the expected annual projected growth over the next 50 years is an average two (2) new homes per year along with the potential construction of a nursing home. Three potential locations were identified during these discussions for the projected 100 new homes and nursing home. These locations are listed below and presented in Figure 5-2.

- Extension of Faulkner Drive to Bog Road (16 new homes);
- Connection of Chestnut Avenue to Evangeline Drive (15 new homes); and,
- Extension of Mariner Drive (new nursing home).

These three locations may not provide sufficient land to allow for the projected 50 year growth of 100 new homes and a nursing home. Dillon has assumed that all future development will take place in the southwest part of the Community. The following locations were also considered for potential future development during modelling:

- Extension of Alders Avenue to the west (48 new homes); and,
- Extension of McCully Crescent and Mariner's Drive (18 new homes).

Four (4) of the eight (8) drainage systems in the Community are expected to be influenced by future development. The following set of simulation results assesses the future performance of the drainage network with an estimated 97 new homes and a nursing home. A summary of the updated runoff parameters assigned to each sub-catchment are provided in Appendix A.

Figure 5-2 identifies flood vulnerable areas resulting from future development.

5.2.1 Downtown – Chittick Avenue, Main Street & School Street – Future Land Use Conditions

The downtown area is not expected to convey stormwater runoff from new developments over the next 50 years. The simulation results for future land use conditions are the same as existing conditions (see Table 5-1).

5.2.2 Riverview Road, Maple Avenue & Birch Street – Future Land Use Conditions

The Riverview Road drainage system is expected to collect stormwater runoff from future developments along the Alders Avenue extension. Refer to Appendix A for the projected increase in impervious surface parameters to the associated catchment areas.

Under future development conditions, simulations of the Riverview Road drainage area suggest potential flooding issues. Similar to the existing conditions simulations, future flooding is expected for the 5-year rainfall events particularly along Maple Avenue in the vicinity of Civic #30, and along Riverview Road in the vicinity of Civic #23. The existing grade along Riverview Road and Maple Avenue is expected to minimize issues resulting from roadway ponding.

The estimated water levels observed at the locations above are summarized in Table 5-9. Most notably, increased flooding along Maple Avenue is expected for the 5-year rainfall event.

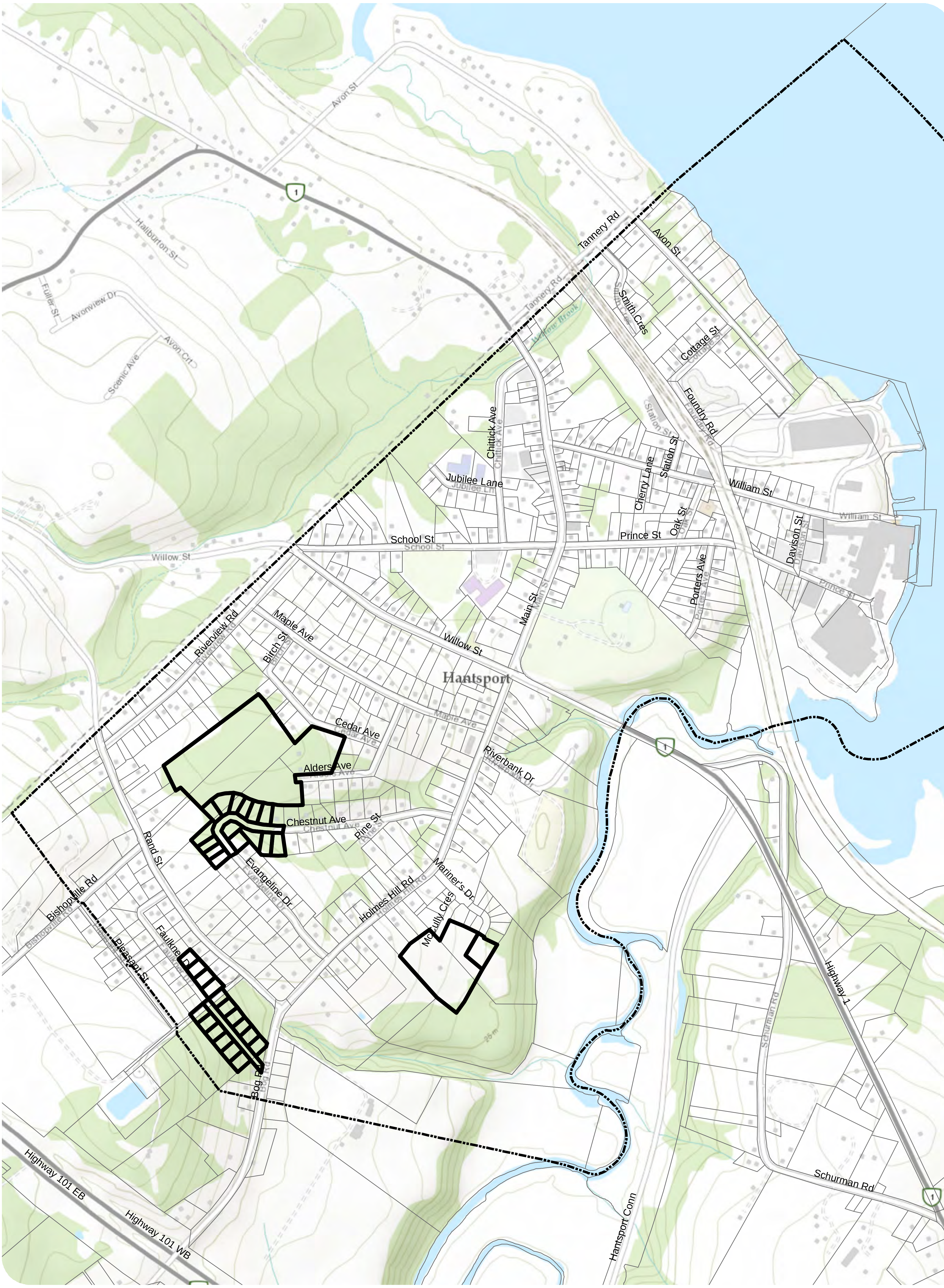
Table 5-9: Summary of Simulated Future Water Level Elevation Impacts for Historical Climate – Riverview Road, Maple Avenue & Birch Street

Location	Existing Development Conditions Peak Water Level (m)		Future Development Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Maple Avenue – Civic #24 (J77)	40.71[2]	40.71[2]	40.71[2]	40.71[2]	0.00	0.00
Maple Avenue – Civic #30 (MH169)	40.39	40.52[2]	40.52[2]	40.52[2]	+0.13	0.00
Riverview Road – Civic #8 (CB158)	36.84	37.45	36.85	37.72[2]	+0.01	+0.27
Riverview Road – Civic #23 (CB168)	54.42[2]	54.42[2]	54.42[2]	54.42[2]	0.00	0.00

[2]Roadway ponding greater than 0.3 m in depth.

5.2.3 Foundry Lane – Future Land Use Conditions

The Foundry Lane area is not expected to convey stormwater runoff from new developments over the next 50 years. The simulation results for future land use conditions are the same as existing conditions (see Table 5-3).



MUNICIPALITY OF THE DISTRICT OF WEST HANTS

HANTSPORT SWM STUDY

FUTURE DEVELOPMENT

FIGURE 5-2

WEST HANTS NOVA SCOTIA

FUTURE DEVELOPMENT AREAS (97 HOMES, 1 NURSING HOME)

LAND PARCEL

MUNICIPAL BOUNDARY

DILLON CONSULTING

MAP DRAWING INFORMATION:
DATA PROVIDED BY ESRI & WEST HANTS

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

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

0 50 100 200 Meters
SCALE 1:7,500

N E S W

PROJECT: 17-6276 STATUS: FINAL DATE: 18/04/27

5.2.4 William Street, Prince Street & Main Street – Future Land Use Conditions

The drainage system is not expected to convey stormwater runoff from new developments over the next 50 years. The simulation results for future land use conditions are the same as existing conditions (see Table 5-4).

5.2.5 Willow Street – Future Land Use Conditions

The Willow Street area is not expected to convey stormwater runoff from new developments over the next 50 years. The simulation results for future land use conditions are the same as existing conditions (see Table 5-5).

5.2.6 Holmes Hill Road, Maple Avenue & Chestnut Avenue – Future Land Use Conditions

This area has been considered as a primary location for development over the next 50 years. As noted above, the extension of Chestnut Avenue and Alders Avenue would accommodate approximately 63 new homes. It has been assumed that the majority of stormwater runoff from the new developments will be conveyed through this system.

The model indicates that these developments are not expected to significantly in increase roadway flooding from pre-development. Simulations suggest primary areas of concern include Alders Avenue and Holmes Hill Road.

The estimated water levels observed at these locations above summarized in Table 5-10.

Table 5-10: Summary of Simulated Future Water Level Elevation Impacts for Historical Climate – Holmes Hill Road, Maple Avenue & Chestnut Avenue

Location	Existing Development Conditions Peak Water Level (m)		Future Development Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Alders Avenue – Intersection with Cedar Avenue (CB177)	41.94	43.57[2]	42.00	43.57[2]	+0.06	0.00
Holmes Hill Road – Intersection with Riverbank Drive (J130)	20.70[2]	20.70[2]	20.70[2]	20.70[2]	0.00	0.00
Mariner's Drive (J53)	33.04	34.01	33.04	34.01	0.00	0.00

[2]Roadway ponding greater than 0.3 m in depth.

Based on the existing grade within this catchment area, any significant buildup of stormwater runoff will presumably flow overland and eventually discharge to the open ditch along Riverbank Drive.

5.2.7 Rand Street, Bog Road & Evangeline Drive – Future Land Use Conditions

The connection of Evangeline Avenue and Chestnut Avenue has been noted as a location for development in the next 50 years. This development is expected to accommodate approximately 15 new homes resulting in additional impervious surface in the Evangeline Drive catchment area.

The simulations suggest that the majority of the existing system is adequate to convey projected stormwater runoff volumes for the 100-year rainfall event. Similar to the existing conditions assessment, an area of concern is Faulkner Drive in the vicinity of Civic #80. Due to the relatively steep grade throughout the catchment area, it is expected that roadway ponding will be minimal.

The estimated water levels observed at the locations above are summarized in Table 5-11.

Table 5-11: Summary of Simulated Future Water Level Elevation Impacts for Historical Climate - Rand Street, Bog Road & Evangeline Drive

Location	Existing Development Conditions Peak Water Level (m)		Future Development Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Faulkner Drive – Civic #80 (CB225)	64.49[2]	64.49[2]	64.49[2]	64.49[2]	0.00	0.00

[2]Roadway ponding greater than 0.3 m in depth.

5.2.8 Rand Street, Bishopville Road & Riverview Road – Future Land Use Conditions

The extension of Alders Avenue is expected to result in 48 additional homes. A portion of the stormwater runoff from this new development is expected to be conveyed through Riverview Road and Rand Street collection systems. The model indicates a couple of areas of potential concern along Pleasant Street and Riverview Road.

The estimated water levels observed at these locations are summarized in Table 5-12.

Table 5-12: Summary of Simulated Future Water Level Elevation Impacts for Historical Climate – Rand Street, Bishopville Road & Riverview Road

Location	Existing Development Conditions Peak Water Level (m)		Future Development Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Pleasant Drive (J61)	71.27	71.55[2]	71.27	71.55[2]	0.00	0.00
Riverview Road – Ditch Inlet (J40)	55.55	56.19[2]	55.72	56.19[2]	+0.17	0.00
Rand Street – Intersection with Bishopville Drive (CB227)	61.55	61.71	61.55	61.71	0.00	0.00

[2]Roadway ponding greater than 0.3 m in depth.

5.3 Future Performance with Climate Change Conditions

This set of simulations assesses the performance of the existing system assuming future land use for a range of rainfall events that have been adjusted for climate change. As noted in Section 4.1.2, the climate change adjustments assume a moderate greenhouse gas emissions scenario over the next 50 years (RCP 4.5).

The rainfall distributions used in the model are provided in Appendix D. Figure 6-2 identifies flood vulnerable areas resulting from future development with climate change conditions.

5.3.1 Downtown – Chittick Avenue, Main Street & School Street – Future Land Use Conditions with Climate Change

The future development model indicates potential flooding issues along Chittick Avenue, School Street, and Main Street during the 5-year rainfall event. More extreme precipitation events (i.e. 100-year) result in increased flooding on all three streets. Water levels in Willow Brook are suspected to influence the potential of flooding.

Under future climate conditions, simulations suggest the potential for increased flooding during the 5-year rainfall event along Chittick Avenue and Main Street. The Main Street Bridge crossing is an area of significant concern with respect to flooding. Simulation results indicate flooding in the following additional areas:

- Chittick Avenue in vicinity of Civic #22 and #47;
- School Street in the vicinity of Civic #12; and,
- Main Street in the vicinity of Civic #25 and Civic #39.

The estimated water levels observed at the locations above are summarized in Table 5-13.

Table 5-13: Summary of Simulated Future Water Level Elevation Impacts for Future Development and Climate Change Conditions – Downtown

Location	Historical Climate Conditions Peak Water Level (m)		Climate Change Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Main Street – Civic #25 (CB98)	18.16	18.38[2]	18.38[2]	18.38[2]	+0.22	0.00
Main Street – Civic #39 (CB93)	18.27	19.38[2]	19.23	19.38[2]	+0.96	0.00
Main Street Bridge (J46)	14.38	15.75[1]	15.38	15.75[1]	+1.00	0.00
Chittick Avenue – Civic #22 (J1)	14.57	15.01[2]	15.01[2]	15.01[2]	+0.44	0.00
Chittick Road - Civic #47 (CB84)	14.87[2]	14.87[2]	14.87[2]	14.87[2]	+0.00	0.00
School Street – Civic #12 (J26)	16.68[2]	16.68[2]	16.68[2]	16.68[2]	0.00	0.00

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

5.3.2 Riverview Road, Maple Avenue & Birch Street – Future Land Use Conditions with Climate Change

The future development simulation results suggest potential flooding at the top of Riverview Road and Maple Avenue. The 100-year event indicated increased flooding along Riverview Road and Maple Avenue.

Simulations of the Riverview Road area assuming the effects of climate change suggest increased flooding issues during the 5-year rainfall event. The model shows minor flooding throughout the system particularly along Maple Avenue in the vicinity of Civic #24 and Civic #30, and along Riverview Road in the vicinity of Civic #23. The existing grade along Riverview Road and Maple Avenue is expected to help minimize flooding.

The estimated water levels observed at the locations above are summarized in Table 5-14.

Table 5-14: Summary of Simulated Future Water Level Elevation Impacts for Future Development and Climate Change Conditions – Riverview Road, Maple Avenue & Birch Street

Location	Historical Climate Conditions Peak Water Level (m)		Climate Change Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Maple Avenue – Civic #24 (J77)	40.71[2]	40.71[2]	40.71[2]	40.71[2]	0.00	0.00
Maple Avenue – Civic #30 (MH169)	40.52[2]	40.52[2]	40.52[2]	40.52[2]	0.00	0.00
Riverview Road – Civic #8 (CB158)	36.85	37.72[2]	36.86	37.72[2]	+0.01	0.00
Riverview Road – Civic #23 (CB168)	54.42[2]	54.42[2]	54.62[2]	54.62[2]	0.00	0.00

[2]Roadway ponding greater than 0.3 m in depth.

5.3.3 Foundry Lane – Future Land Use Conditions with Climate Change

The Foundry Lane area is of significant concern for potential flooding during the historical 5-year return period event. Increased flooding is expected for future climate conditions due to increased rainfall amounts. It is also noteworthy that the simulations indicate an undersized catchbasin lead (100 mm) from CB12 (in front of the Fire Station).

The estimated water levels observed in the vicinity of the Soccer Field are summarized in Table 5-15.

Table 5-15: Summary of Simulated Future Water Level Elevation Impacts for Future Development and Climate Change Conditions – Foundry Lane

Location	Historical Climate Conditions Peak Water Level (m)		Climate Change Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Soccer Field (J137)	15.40[2]	15.40[2]	15.40[2]	15.40[2]	0.00	0.00

[2]Roadway ponding greater than 0.3 m in depth.

5.3.4 William Street, Prince Street & Main Street – Future Land Use Conditions with Climate Change

Under future development conditions, the William Street, Prince Street, and Main Street combined system is expected to experience drainage issues for the 5-year event. The model indicates localized flooding along William Street and Prince Street. The 100-year rainfall event shows increased flooding along William Street and Prince Street.

Similar to the results indicated during historical rainfall events, minor flooding is expected along Prince Street and William Street for the projected 5-year climate change event with increased flooding during the 100-year event.

The estimated water levels observed at the locations above are summarized in Table 5-16.

Table 5-16: Summary of Simulated Future Water Level Elevation Impacts for Future Development and Climate Change Conditions – William Street, Prince Street & Main Street

Location	Historical Climate Conditions Peak Water Level (m)		Climate Change Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
William Street – Civic #14 (CB57)	16.67[2]	16.67[2]	16.67[2]	16.67[2]	0.00	0.00
Prince Street – Civic #11 (CB40)	17.19[2]	17.19[2]	17.19[2]	17.19[2]	0.00	0.00
William Street – Intersection with Oak Street (CB47)	15.30	15.43[2]	15.35	15.43[2]	+0.05	0.00
Prince Street – Intersection with Davidson Road (CB23)	14.09[2]	14.09[2]	14.09[2]	14.09[2]	+0.00	0.00

[2]Roadway ponding greater than 0.3 m in depth.

The locations identified above are primarily low lying areas which will tend to act as collection points during extreme rainfall events. As discussed in Section 5.1.4, separation of the combined sewer should be considered for this area.

5.3.5 Willow Street – Future Land Use Conditions with Climate Change

Similar to the results of the existing conditions simulations, the model suggests that the system continues to experience flooding at the intersection of Main Street for the 5-year event. As discussed in Section 5.1.5, it is expected that the roadway system minimizes ponding by conveying stormwater runoff along Willow Street and ultimately discharging to Halfway River.

Estimated water levels observed at the intersection of Main Street are summarized in Table 5-17.

Table 5-17: Summary of Simulated Future Water Level Elevation Impacts for Future Development and Climate Change Conditions – Willow Street

Location	Historical Climate Conditions Peak Water Level (m)		Climate Change Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Willow Street – Intersection with Main Street (CB137)	19.93[2]	19.93[2]	19.93[2]	19.93[2]	0.00	0.00

[2]Roadway ponding greater than 0.3 m in depth.

As discussed in Section 5.1.5, separation of the combined sewer should be considered for Willow Street.

5.3.6 Holmes Hill Road, Maple Avenue & Chestnut Avenue – Future Land Use Conditions with Climate Change

Flooding is a concern at the intersection of Holmes Hill Road with Riverbank Drive during the historical 5-year rainfall event. Increased flooding is expected for the 100-year rainfall event. The effects of climate change indicate that the Alders Avenue system provides less than a 5-year level of service.

The estimated water levels observed at the locations above are summarized in Table 5-18. Notably, Mariner's Drive is expected to experience increased flooding for the future 100-year rainfall event.

Table 5-18: Summary of Simulated Future Water Level Elevation Impacts for Future Development and Climate Change Conditions – Holmes Hill Road, Maple Avenue & Chestnut Avenue

Location	Historical Climate Conditions Peak Water Level (m)		Climate Change Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Alders Avenue – Intersection with Cedar Avenue (CB177)	42.00	43.57[2]	43.57[2]	43.57[2]	+1.57	0.00
Holmes Hill Road – Intersection with Riverbank Drive (J130)	20.70[2]	20.70[2]	20.70[2]	20.70[2]	0.00	0.00
Mariner's Drive (J53)	33.04	34.01	33.25	34.25[2]	+0.21	+0.24

[2]Roadway ponding greater than 0.3 m in depth.

Based on the existing grade within this catchment area, any significant buildup of stormwater runoff will presumably flow overland and eventually discharge to the open ditch off Riverbank Drive.

5.3.7 Rand Street, Bog Road & Evangeline Drive – Future Land Use Conditions with Climate Change

The Rand Street and Bog Road drainage system is expected to experience minor flooding during the historical 5-year rainfall event. An area of concern is along Faulkner Drive in the vicinity of Civic #80. The combined impact of future development and climate change conditions do not indicate any additional flooding concerns in this area.

The estimated water levels observed at the locations above are summarized in Table 5-19.

Table 5-19: Summary of Simulated Future Water Level Elevation Impacts for Future Development and Climate Change Conditions – Rand Street, Bog Road & Evangeline Drive

Location	Historical Climate Conditions Peak Water Level (m)		Climate Change Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Faulkner Drive – Civic #80 (CB225)	64.49[2]	64.49[2]	64.49[2]	64.49[2]	0.00	0.00

[2]Roadway ponding greater than 0.3 m in depth.

5.3.8 Rand Street, Bishopville Road & Riverview Road – Future Land Use Conditions with Climate Change

Simulation results indicate the system can adequately convey stormwater runoff from the 5-year rainfall event assuming future development conditions. Localized flooding becomes an issue during more extreme rainfall events (i.e. historical 100-year). The model indicates minor flooding for the projected 5-year climate change rainfall event.

The estimated water levels observed at the locations above are summarized in Table 5-20. It is noteworthy that the intersection of Bishopville Drive with Rand Street is not at risk of flooding during future development alone; however, this area is a potential flood concern for the climate change adjusted 100-year event.

Table 5-20: Summary of Simulated Future Water Level Elevation Impacts for Future Development and Climate Change Conditions – Rand Street, Bishopville Road & Riverview Road

Location	Historical Climate Conditions Peak Water Level (m)		Climate Change Conditions Peak Water Level (m)		Water Level Impact (m)	
Return Period Rainfall Event	5 yr	100 yr	5 yr	100 yr	5 yr	100 yr
Pleasant Drive (J61)	71.27	71.55[2]	71.42	71.55[2]	+0.15	0.00
Riverview Road – Ditch Inlet (J40)	55.72	56.19[2]	56.35	56.19[2]	+0.63	0.00
Rand Street – Intersection with Bishopville Drive (CB227)	61.55	61.71	61.58	63.09[2]	+0.03	+1.38

[2]Roadway ponding greater than 0.3 m in depth.

6.0 Assessment of Future Flood Risk Vulnerability

6.1 Urban Drainage Flood Risk

As previously discussed, the drainage network in the Community of Hantsport is comprised of storm sewer, combined sewer, and open ditching. The simulation results suggest that the majority of the drainage system can adequately convey stormwater runoff under existing conditions. The future development model incorporates an estimated 97 new homes and a nursing home. These developments result in increased impervious surfaces leading to increased runoff. The following three (3) locations are expected to experience drainage issues as a result of future development:

- Holmes Hill Road – The model indicates localized flooding along Holmes Hill Road at the intersection with Riverbank Road. It is expected that flooding is minimized however due to the existing grade in the area which would tend to convey runoff overland to the ditch on Riverbank Drive.
- Riverview Road – The model indicates two areas at risk of potential flooding along Riverview Road. The ditch located at the southern end of the road has an estimated 2-year level of service under future development conditions. The model indicates that the ditch has inadequate capacity to handle projected stormwater flows and the existing 300 mm diameter inlet pipe in undersized. Simulations suggest localized flooding in the vicinity of Civic #8 and #23 due to an undersized storm sewer.
- Maple Avenue – Localized low areas along Maple Avenue have been identified as potential flood risks. Most notably, the model indicates flooding in the vicinity of Civic #24 and #30.

Under future land use conditions with climate change, simulations suggest some inadequate drainage infrastructure. The results indicate six (6) locations of potential flood risk:

- Main Street Bridge Crossing – The Main Street Bridge crossing area has been noted for potential flood risk. Based on the simulation results, it is suspected that this issue is related to the channel capacity of Willow Brook in the area. The limited channel capacity is expected to influence and further impact flood vulnerability further upstream along Chittick Avenue, Main Street, and School Street.

- Chittick Avenue, Main Street, and School Street – The simulation results suggest that Chittick Avenue, Main Street and School Street are potential flood risks. It is expected that the downstream Main Street Bridge crossing has an influence on flooding at these sites. Further simulations were completed to identify undersized drainage infrastructure along these streets. These streets are at low elevations relative to the surrounding topography.
- Foundry Road – The Foundry Road area presents a significant challenge to provide adequate drainage. The simulation results show that this area experiences flooding during an estimated level of service of 2 years. Detailed investigation of topographical data along Foundry Road indicates that it is a low lying area which acts as a probable collection point of stormwater runoff. Undersized drainage infrastructure is also inadequate to convey the estimated stormwater runoff volumes.
- Prince Street and William Street – Localized low areas along Prince Street and William Street have been identified as potential flood risks. Most notably, the model indicates flooding along William Street in the vicinity of Civic #14, and along Prince Street in the vicinity of Civic #11.
- Willow Street – The model indicates localized flooding at the intersection of Willow Street with Main Street. The Willow Street system is combined therefore flooding could potentially result in sanitary flows on the roadway.
- Faulkner Drive – The simulation results indicate potential flooding along Faulkner Drive due to undersized drainage infrastructure. The catchbasin lead from CB225 (in the vicinity of Civic #80) is undersized for the estimated stormwater runoff volumes.

The Municipality also noted areas of concern during the meetings on August 29th and November 22nd. The additional two (2) areas were not assessed during the modeling exercise due to lack of stormwater infrastructure (Avon Street) and insufficient information (library/playground). These locations are described below:

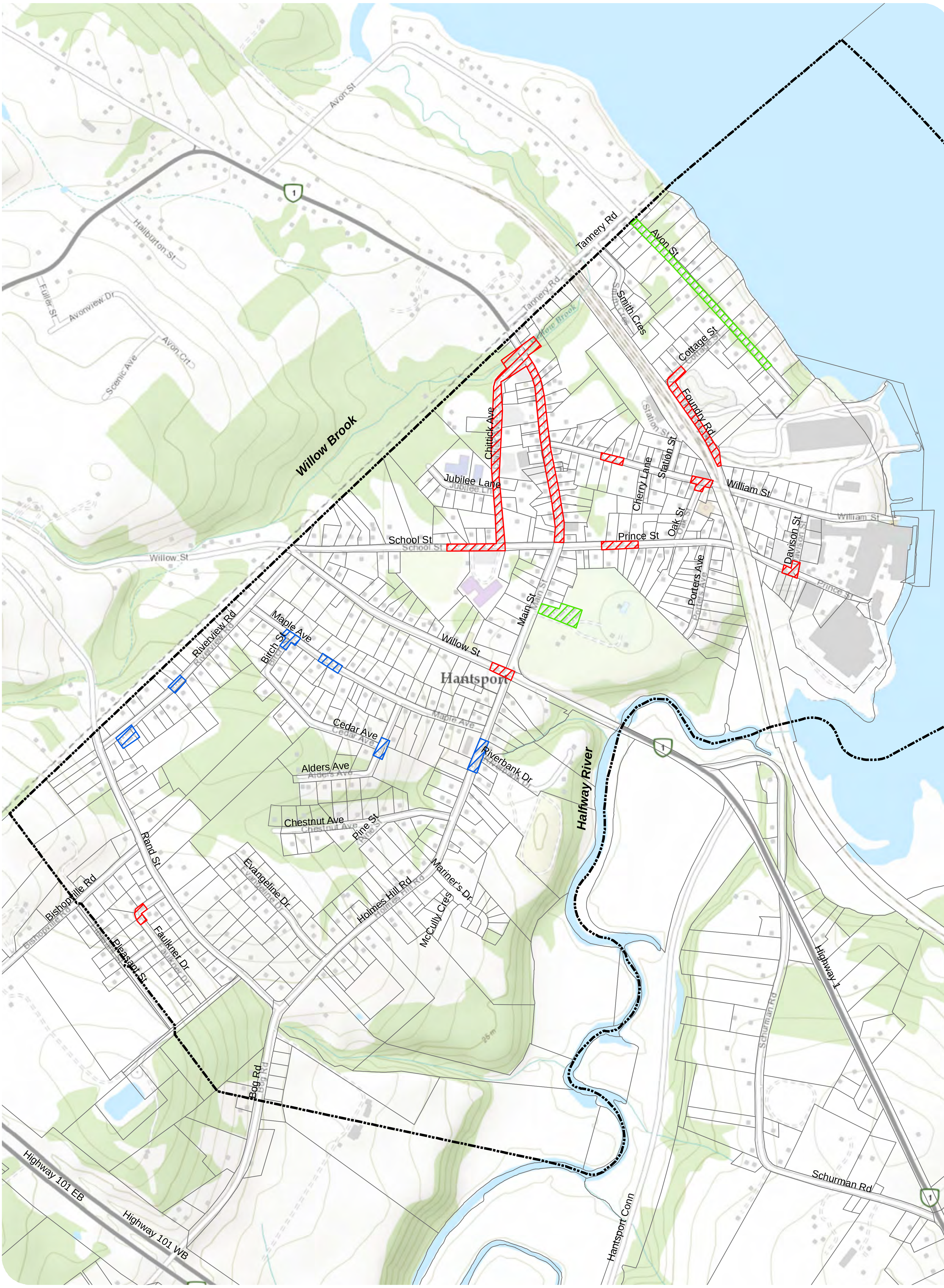
- Library/Playground (Main Street) – The playground behind the library on Main Street experiences drainage issues annually. A site visit on November 22nd revealed that drainage from the area is collected by a single catchbasin which is full of sediment/debris and a raised cover. These factors limit the inlet capacity of the structure which contributes to the flooding issue. Figure 6-1 provides photos of the catchbasin.
- Avon Street – Avon Street does not have any existing drainage infrastructure and frequently experiences flooding during heavy rainfall events. These flooding events are typically short lived as water quickly infiltrates into the ground.

Figure 6-1: Photographs of Catchbasin in Library/Playground Area (taken November 22, 2017)

6.2 Halfway River Flood Risk

As mentioned previously, the aboiteau crossing the existing railway embankment has failed. The hydraulic regime of the lower Halfway River is expected to have changed as a result of this failure. The collapsed aboiteau (see Figure 2-2) is expected to allow migration of extreme tides and storm surge into the lower Halfway River. Similarly, peak flows from the Halfway River watershed are expected to flow reasonably unimpeded into Avon River. Some hydraulic restriction as a result of the collapsed structure is expected, however this restriction would be expected to decrease over time as the collapsed material is exposed to tidal action and high flood flows from the river causing erosion.

There is also a set of four overflow culverts located east of the aboiteau, shown in Figure 6-3. These culverts appear to be operational, though quite deteriorated. The culverts appear to consist of 900 mm corrugated steel pipe (CSP). These culverts act as an overflow mechanism to prevent overtopping of the railway and risk of flooding along Willow Street/Highway 1 in the event of high water levels in the lower Halfway River.



MUNICIPALITY OF THE DISTRICT OF WEST HANTS

HANTSPORT SWM STUDY

FUTURE FLOOD VULNERABLE AREAS

FIGURE 6-2

WEST HANTS NOVA SCOTIA

FLOOD VULNERABLE AREAS (DUE TO FUTURE LAND USE)

FLOOD VULNERABLE AREAS (DUE TO FUTURE LAND USE AND CLIMATE CHANGE)

FLOOD VULNERABLE AREAS (ADDITIONAL IDENTIFIED BY THE MUNICIPALITY)

LAND PARCEL

MUNICIPAL BOUNDARY

DILLON CONSULTING

MAP DRAWING INFORMATION:
DATA PROVIDED BY ESRI & WEST HANTS

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

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

SCALE 1:7,500
0 50 100 200 Meters

N S E W

PROJECT: 17-6276 STATUS: FINAL DATE: 18/04/27

Figure 6-3: Existing Aboiteau Overflow at Halfway River Railway Crossing

The collapse has created a free outlet to the Avon River. It is expected that the current condition of the aboiteau could potentially increase flood risk associated with storm surge from the Bay. However the free outlet may slightly decrease flood risk associated with high flows in the Halfway River, since discharges to the Avon River are no longer restricted by the capacity of the aboiteau.

The flood risk along Willow Street from the Halfway River is expected to be minimal. A topographical review of the area suggests that the lowest lying residences have an elevation in the order of 16.8 m (Willow Street Civic #1). Notably, the historical 100-year water level in the Bay of Fundy is expected to be in the order of 5.4 m, with projected future (year 2100) 100-year level of roughly 6.5 m with sea level rise.

Based on observations collected along the Halfway River (Section 3.2), the HGL in the lower river are typically 2 m higher than the bay. This suggests the potential future extreme water level in the lower Halfway River in the order of 8.5 m. However, it is noteworthy that extreme flood flows combined with storm surge conditions would also presumably increase the HGL beyond this level.

The estimated freeboard between the extreme water level elevation and the lowest residence adjacent to the river is approximately 8.8 m, therefore the risk of riverine flooding to the residents of Hantsport is expected to be minimal. Flooding is presumably limited to downstream areas outside the limits of the Community.

7.0 Preliminary Drainage Improvement Options

Based on the observations from the site visits (September 2017 – November 2017), as well as the PCSWMM simulation results, a set of conceptual drainage improvement options were prepared and are described in the subsequent sections. It is noteworthy that these upgrades have been developed using the PCSWMM model assuming future build-out conditions and the projected future climate rainfall depths distributed using the Alternating Block rainfall distribution (described in Section 4.2).

A Community wide overview of the recommended preliminary improvements is presented in Table 7-1. A prioritization ranking has been assigned to each drainage challenge based on the extent of the issue and the potential risk to Community infrastructure.

Table 7-1: Summary of Preliminary Drainage Improvement Options

Drainage Improvement Option	Description	Prioritization Rank
Main Street Bridge Crossing	Based on the model results, it is estimated that the channel capacity of Willow Brook near Main Street provides a 2-year level of service. This limited channel capacity results in drainage issues throughout the downtown area (Chittick Avenue, Main Street, and School Street). Based on these findings, it is recommended that hydraulic modelling be completed for Willow Brook. This exercise would involve flood extent mapping along Willow Brook to determine the optimal channel dimensions for stormwater conveyance. Furthermore, this analysis would investigate the potential impact of these changes to the upstream and downstream reaches of Willow Brook. The recommended hydraulic modelling software for this analysis is HEC-RAS.	1
Combined Sewer Separation	Results indicate localized flooding of the combined sewers along William Street, Prince Street, and Main Street. The separation of these combined systems as well as the system along Willow Street should be considered. Based on preliminary model results, the following trunk sewers have been sized to adequately convey the future 5-year event: · William Street – 600 mm · Prince Street – 600 mm · Main Street – 375 mm · Willow Street – 375 mm	2
Foundry Road Improvements	Model simulations indicate significant roadway flooding along Foundry Road area. Further analysis revealed that the trunk sewer is potentially undersized and catchbasin intake capacity is not adequate for the stormwater runoff volumes. Preliminary model results indicate that the trunk sewer along Foundry Lane should be increased to 600 mm (from MH215) and catchbasin leads should be increase to 375mm. Additional catchbasins are also recommend along Foundry Road in the vicinity of Civic #3 for increased inlet capacity to reduce roadway flooding.	3

Drainage Improvement Option	Description	Prioritization Rank
Chittick Avenue, School Street, and, Main Street Improvements	The model indicates flooding along Chittick Avenue, School Street, and Main Street. Based on preliminary model results, the following trunk sewers have been sized to adequately convey the future 5-year event: · Chittick Avenue – 900 mm · School Street – 750 mm (between Civic #27 and Hantsport School) · Main Street – 750 mm	4
Holmes Hill Road	The model suggests flooding along Holmes Hill Road in the vicinity of the Riverbank Road intersection. Based on preliminary model results, the following recommendations have been developed: · Increase storm sewers along Holmes Hill Road to 450 mm (south of Riverbank Drive); · Increase storm sewers on Homes Hill Road to 750 mm (north of Riverbank Drive); · Increase storm sewers along Alders Avenue and Maple Avenue to 375 mm; and, · Additional catchbasins at the intersection of Riverbank Drive to minimize roadway flooding.	5
Library/Playground Area	The existing catchbasin providing stormwater runoff collection for this area is full of sediment/debris with a raised cover. The outlet location of this storm sewer could not be confirmed through field investigation; however, it is assumed to outfall to Halfway River (east). Based on field investigation, the following is recommended: · Reset cover and cleanout the existing catchbasin; · Addition of new catchbasins; and, · Regrade the area to catchbasins. Preliminary calculations indicate that a 250 mm sewer could adequately convey projected future 5-year runoff volumes.	6
Avon Street Improvements	Avon Street currently does not have any existing drainage infrastructure and the area experiences flooding during extreme rainfall events. It is recommended to install a storm sewer system along this road to prevent these flooding events. Preliminary model results indicate that a 375 mm diameter trunk sewer (assuming slope of 2%) will adequately convey the projected future 5-year return period flows.	7

Drainage Improvement Option	Description	Prioritization Rank
Riverview Road Ditch	As previously discussed, the model indicates that the Riverview Road ditch (in the vicinity of Civic #37 is undersized. Based on preliminary model results, the ditch inlet should be replaced with a 450 mm diameter pipe assuming a grade of 1% grade.	8
Faulkner Drive	The model indicates localized flooding along Faulkner Drive in the vicinity of Civic #80. A review of the simulated 5-year return period flows suggests that the existing 200 mm diamter pipe is undersized. Based on preliminary model results, the existing storm sewer on Faulkner should be replaced with a 375 mm diameter storm sewer to adequately convey future 5-year return period flows.	9
Riverview Road	The model indicates localized flooding along Faulkner Drive in the vicinity of Civic #23. A review of the simulated 5-year return period flows suggests that the existing 100 mm diameter pipe is undersized. Based on preliminary model results, the existing storm sewer on Riverview Drive should be replaced with a 250 mm diameter storm sewer to adequately convey future 5-year return period flows.	10

8.0 Conclusion

This report provides a review of the performance of the existing drainage infrastructure in the Community of Hantsport. Hydrologic and hydraulic numerical simulation of the system was completed to investigate the performance of existing drainage works. The system was assessed under existing conditions, future development conditions, and projected future climate change conditions. Rainfall events with return periods ranging from two (2) to 100 years were used to assess the stormwater system. The projected future climate change events were adjusted for climate change assuming a moderate greenhouse gas emissions scenario (RCP 4.5). The future development model incorporated the construction of an estimated 97 new homes and a nursing home. Updated runoff parameters for the future development model are presented in Appendix A.

Based on the simulation, the following 10 areas/systems were identified as concerning with respect to adequate drainage (see Figure 6-2).

- Main Street Bridge Crossing
- Chittick Avenue, Main Street, and School Street
- Foundry Road
- Combined Sewers Systems
- Holmes Hill Road

- Riverview Road
- Maple Avenue
- Faulkner Drive
- Avon Street
- Library/Playground Area (Main Street)

The future development model (with climate change adjustments) was then updated to include the recommended drainage improvements summarized in Table 7-1. The recommended pipe sizes are based on providing adequate conveyance of the future 5-year return period event with climate change adjustment. Additional catchbasins have also been recommended in some cases to minimize roadway flooding during the 100-year event. It is important to note that these recommended improvements are preliminary and a detailed analysis would be required prior to construction.

As noted in the report, the aboiteau in the low Halfway River failed in November 2017. The failure was concerning due to the potential of increased flood risk associated with storm surge from the Bay. Based on review of potential future water levels and surrounding topography, the estimate freeboard between the extreme water level elevation and the lowest residence adjacent to the river is approximately 8.8 m. The risk of riverine flooding to the residents of Hantsport is expected to be minimal.

Appendix A

PCSWMM Catchment Parameters

Table A-1: PCSWMM Catchment Parameters – Existing & Future Conditions

Catchment	Catchment Area (ha)	Effective Catchment Area (ha)	Slope (%)	Imperviousness – Existing (%)	Imperviousness – Future (%)
S1	275.73	275.40	9.0	4.7	4.7
S2	0.12	0.10	1.9	48.9	48.9
S2_1	1.16	1.16	10.1	16.1	16.1
S2_3	2.88	2.88	12.0	24.8	24.8
S2_4	3.40	3.40	7.2	25.9	25.9
S3_1	4.42	4.42	4.0	9.1	19.1
S3_3	7.74	7.74	11.6	7.5	7.5
S3_4	8.65	8.65	7.5	5.9	5.9
S4	4.81	4.81	11.2	20	20
S5_1	3.10	2.97	4.4	70.5	70.5
S5_2	0.95	0.95	3.8	41.9	41.9
S5_3	5.91	5.85	4.1	23.8	23.8
S5_5	0.54	0.47	4.0	93.7	93.7
S6	1.97	1.97	4.3	41	41
S7	37.15	37.06	11.4	1.4	3.8
S8	2.66	2.56	9.3	43	43
S9_1	2.34	2.34	2.5	29.3	29.3
S9_2	1.28	1.28	3.5	3.6	3.6
S9_3	3.20	2.95	2.7	21.4	21.4
S9_4	1.32	1.32	3.5	12.2	12.2
S9_5	6.08	5.90	2.9	22.4	22.4
S9_7	0.60	0.60	2.9	13.8	13.8
S9_8	3.70	2.80	3.8	19.7	19.7
S10	2.83	2.83	7.9	23	23
S11	4.27	4.27	9.9	7.8	7.8
S12	2.55	2.55	10.8	23	38.6
S13_1	1.68	1.68	15.3	30.5	30.5
S13_3	2.09	2.09	11.6	15.1	20.7
S13_4	3.62	3.62	6.8	5	40.3
S14_1	1.07	1.07	12.3	27.5	27.5

Catchment	Catchment Area (ha)	Effective Catchment Area (ha)	Slope (%)	Imperviousness – Existing (%)	Imperviousness – Future (%)
S14_2	0.81	0.81	10.0	44.5	44.5
S15	0.21	0.21	6.4	55	55
S16_2	3.33	3.33	11.4	7.4	7.4
S16_3	3.14	3.12	11.2	16	16
S16_4	2.05	2.05	9.2	39.3	39.3
S17	2.58	2.58	8.3	25	25
S18_1	3.17	3.17	10.9	20.8	23.3
S18_2	1.59	1.59	11.3	20.1	32.1
S19_2	0.22	0.22	8.9	22.6	22.6
S19_3	1.27	1.25	7.3	30.5	30.5
S19_4	1.45	1.45	9.9	18.7	18.7
S22_1	3.24	2.99	5.2	29.8	29.8
S22_2	0.50	0.50	3.7	39.6	39.6
S22_4	1.43	1.39	3.8	32.6	32.6
S22_5	4.65	4.41	4.2	22.2	22.2
S23_1	3.54	3.51	3.8	45.2	45.2
S23_2	0.27	0.27	3.2	54.9	54.9
S23_4	0.42	0.40	3.3	45.2	45.2
S23_5	1.55	1.55	2.9	37.5	37.5
S24	2.11	2.11	7.4	26	31.3
S25_1	6.31	6.25	16.9	3.1	10.6
S25_2	0.45	0.45	6.6	22.2	22.2
S25_4	1.66	1.65	13.4	30.2	30.2
S25_5	4.36	4.36	10.3	14.5	19.9
S25_6	0.47	0.47	5.8	38.2	38.2
S26	9.92	9.92	18.4	1.3	1.3

Note: - Imperviousness values in red indicate changes under future development conditions.

Appendix B

Additional Simulation Results

Table B-2: Summary of Simulated Future Water Level Elevations for Historical Climate - Downtown

Location		Return Period Rainfall Event					
		2	5	10	25	50	100
Main Street – Civic #25 (CB98)	Existing Development Conditions Peak Water Level (m)	17.57	18.16	18.38[2]	18.38[2]	18.38[2]	18.38[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A
Main Street – Civic #39 (CB93)	Existing Development Conditions Peak Water Level (m)	17.17	18.27	19.26	19.38[2]	19.38[2]	19.38[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A
Main Street Bridge (J46)	Existing Development Conditions Peak Water Level (m)	14.06	14.38	15.35	15.75[1]	15.75[1]	15.75[1]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A
Chittick Avenue – Civic #22 (J1)	Existing Development Conditions Peak Water Level (m)	14.34	14.57	15.01[2]	15.01[2]	15.01[2]	15.01[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A
Chittick Road - Civic #47 (CB84)	Existing Development Conditions Peak Water Level (m)	14.87[2]	14.87[2]	14.87[2]	14.87[2]	14.87[2]	14.87[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A
School Street – Civic #12 (J26)	Existing Development Conditions Peak Water Level (m)	16.68[2]	16.68[2]	16.68[2]	16.68[2]	16.68[2]	16.68[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-3: Summary of Simulated Future Water Level Elevations for Future Development – Downtown

Location		Return Period Rainfall Event					
		2	5	10	25	50	100
Main Street – Civic #25 (CB98)	Historical Climate Conditions Peak Water Level (m)	17.57	18.16	18.38[2]	18.38[2]	18.38[2]	18.38[2]
	Climate Change Conditions Peak Water Level (m)	18.38[2]	18.38[2]	18.38[2]	18.38[2]	18.38[2]	18.38[2]
Main Street – Civic #39 (CB93)	Historical Climate Conditions Peak Water Level (m)	17.17	18.27	19.26	19.38[2]	19.38[2]	19.38[2]
	Climate Change Conditions Peak Water Level (m)	17.87	19.23	19.38[2]	19.38[2]	19.38[2]	19.38[2]
Main Street Bridge (J46)	Historical Climate Conditions Peak Water Level (m)	14.06	14.38	15.35	15.75[1]	15.75[1]	15.75[1]
	Climate Change Conditions Peak Water Level (m)	14.29	15.38	15.75[1]	15.75[1]	15.75[1]	15.75[1]
Chittick Avenue – Civic #22 (J1)	Historical Climate Conditions Peak Water Level (m)	14.34	14.57	15.01[2]	15.01[2]	15.01[2]	15.01[2]
	Climate Change Conditions Peak Water Level (m)	14.62	15.01[2]	15.01[2]	15.01[2]	15.01[2]	15.01[2]
Chittick Road - Civic #47 (CB84)	Historical Climate Conditions Peak Water Level (m)	14.87[2]	14.87[2]	14.87[2]	14.87[2]	14.87[2]	14.87[2]
	Climate Change Conditions Peak Water Level (m)	14.87[2]	14.87[2]	14.87[2]	14.87[2]	14.87[2]	14.87[2]
School Street – Civic #12 (J26)	Historical Climate Conditions Peak Water Level (m)	16.68[2]	16.68[2]	16.68[2]	16.68[2]	16.68[2]	16.68[2]
	Climate Change Conditions Peak Water Level (m)	16.68[2]	16.68[2]	16.68[2]	16.68[2]	16.68[2]	16.68[2]

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-4: Summary of Simulated Future Water Level Elevations for Historical Climate – Riverview Road, Maple Avenue & Birch Street

Location	Return Period Rainfall Event						
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Maple Avenue – Civic #24 (J77)	Existing Development Conditions Peak Water Level (m)	40.31	40.71[2]	40.71[2]	40.71[2]	40.71[2]	40.71[2]
	Future Development Conditions Peak Water Level (m)	40.65	40.71[2]	40.71[2]	40.71[2]	40.71[2]	40.71[2]
Maple Avenue – Civic #30 (MH169)	Existing Development Conditions Peak Water Level (m)	39.66	40.39	40.52[2]	40.52[2]	40.52[2]	40.52[2]
	Future Development Conditions Peak Water Level (m)	39.71	40.52[2]	40.52[2]	40.52[2]	40.52[2]	40.52[2]
Riverview Road – Civic #8 (CB158)	Existing Development Conditions Peak Water Level (m)	36.80	36.84	36.88	36.89	36.89	37.45
	Future Development Conditions Peak Water Level (m)	36.82	36.85	36.88	36.89	36.90	37.72[2]
Riverview Road – Civic #23 (CB168)	Existing Development Conditions Peak Water Level (m)	53.37	54.42[2]	54.42[2]	54.42[2]	54.42[2]	54.42[2]
	Future Development Conditions Peak Water Level (m)	54.12	54.42[2]	54.42[2]	54.42[2]	54.42[2]	54.42[2]

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-5: Summary of Simulated Future Water Elevations for Future Development – Riverview Road, Maple Avenue & Birch Street

Location	Return Period Rainfall Event						
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Maple Avenue – Civic #24 (J77)	Historical Climate Conditions Peak Water Level (m)	40.65	40.71[2]	40.71[2]	40.71[2]	40.71[2]	40.71[2]
	Climate Change Conditions Peak Water Level (m)	40.65	40.71[2]	40.71[2]	40.71[2]	40.71[2]	40.71[2]
Maple Avenue – Civic #30 (MH169)	Historical Climate Conditions Peak Water Level (m)	39.71	40.52[2]	40.52[2]	40.52[2]	40.52[2]	40.52[2]
	Climate Change Conditions Peak Water Level (m)	40.52[2]	40.52[2]	40.52[2]	40.52[2]	40.52[2]	40.52[2]
Riverview Road – Civic #8 (CB158)	Historical Climate Conditions Peak Water Level (m)	36.82	36.85	36.88	36.89	36.90	37.72[2]
	Climate Change Conditions Peak Water Level (m)	36.83	36.86	36.91	37.59	37.72[2]	37.72[2]
Riverview Road – Civic #23 (CB168)	Historical Climate Conditions Peak Water Level (m)	54.12	54.42[2]	54.42[2]	54.42[2]	54.42[2]	54.42[2]
	Climate Change Conditions Peak Water Level (m)	54.12	54.42[2]	54.42[2]	54.42[2]	54.42[2]	54.42[2]

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

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Table B-6: Summary of Simulated Future Water Level Elevations for Historical Climate – Foundry Road

Location	Return Period Rainfall Event						
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Soccer Field (J137)	Existing Development Conditions Peak Water Level (m)	15.40[2]	15.40[2]	15.40[2]	15.40[2]	15.40[2]	15.40[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-7: Summary of Simulated Future Water Level Elevations for Future Development – Foundry Road

Location	Return Period Rainfall Event						
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Soccer Field (J137)	Historical Climate Conditions Peak Water Level (m)	15.40[2]	15.40[2]	15.40[2]	15.40[2]	15.40[2]	15.40[2]
	Climate Change Conditions Peak Water Level (m)	15.40[2]	15.40[2]	15.40[2]	15.40[2]	15.40[2]	15.40[2]

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-8: Summary of Simulated Future Water Levels for Historical Climate – William Street, Prince Street & Main Street

Location	Return Period Rainfall Event						
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
William Street – Civic #14 (CB57)	Existing Development Conditions Peak Water Level (m)	16.67[2]	16.67[2]	16.67[2]	16.67[2]	16.67[2]	16.67[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A
Prince Street – Civic #11 (CB40)	Existing Development Conditions Peak Water Level (m)	17.19[2]	17.19[2]	17.19[2]	17.19[2]	17.19[2]	17.19[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A
William Street – Intersection with Oak Street (CB47)	Existing Development Conditions Peak Water Level (m)	15.27	15.30	15.35	15.43[2]	15.43[2]	15.43[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A
Prince Street – Intersection with Davidson Road (CB23)	Existing Development Conditions Peak Water Level (m)	14.02	14.09[2]	14.09[2]	14.09[2]	14.09[2]	14.09[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-9: Summary of Simulated Future Water Level Elevations for Future Development – William Street, Prince Street & Main Street

Location		Return Period Rainfall Event					
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
William Street – Civic #14 (CB57)	Historical Climate Conditions Peak Water Level (m)	16.67[2]	16.67[2]	16.67[2]	16.67[2]	16.67[2]	16.67[2]
	Climate Change Conditions Peak Water Level (m)	16.67[2]	16.67[2]	16.67[2]	16.67[2]	16.67[2]	16.67[2]
Prince Street – Civic #11 (CB40)	Historical Climate Conditions Peak Water Level (m)	17.19[2]	17.19[2]	17.19[2]	17.19[2]	17.19[2]	17.19[2]
	Climate Change Conditions Peak Water Level (m)	17.19[2]	17.19[2]	17.19[2]	17.19[2]	17.19[2]	17.19[2]
William Street – Intersection with Oak Street (CB47)	Historical Climate Conditions Peak Water Level (m)	15.27	15.30	15.35	15.43[2]	15.43[2]	15.43[2]
	Climate Change Conditions Peak Water Level (m)	15.28	15.35	15.43[2]	15.43[2]	15.43[2]	15.43[2]
Prince Street – Intersection with Davidson Road (CB23)	Historical Climate Conditions Peak Water Level (m)	14.02	14.09[2]	14.09[2]	14.09[2]	14.09[2]	14.09[2]
	Climate Change Conditions Peak Water Level (m)	14.05	14.09[2]	14.09[2]	14.09[2]	14.09[2]	14.09[2]

[1] Overtopping of bridge deck.
[2]Roadway ponding greater than 0.3 m in depth.

Table B-10: Summary of Simulated Future Water Levels for Historical Climate – Willow Street

Location		Return Period Rainfall Event					
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Willow Street – Intersection with Main Street (CB137)	Existing Development Conditions Peak Water Level (m)	19.93[2]	19.93[2]	19.93[2]	19.93[2]	19.93[2]	19.93[2]
	Future Development Conditions Peak Water Level (m)	N/A	N/A	N/A	N/A	N/A	N/A

[1] Overtopping of bridge deck.
[2]Roadway ponding greater than 0.3 m in depth.

Table B-11: Summary of Simulated Future Water Level Elevations for Future Development – Willow Street

Location	Return Period Rainfall Event						
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Willow Street – Intersection with Main Street (CB137)	Historical Climate Conditions Peak Water Level (m)	19.93[2]	19.93[2]	19.93[2]	19.93[2]	19.93[2]	19.93[2]
	Climate Change Conditions Peak Water Level (m)	19.93[2]	19.93[2]	19.93[2]	19.93[2]	19.93[2]	19.93[2]

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-12: Summary of Simulated Future Water Levels for Historical Climate – Holmes Hill Road, Maple Avenue & Chestnut Avenue

Location	Return Period Rainfall Event						
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Alders Avenue – Intersection with Cedar Avenue (CB177)	Existing Development Conditions Peak Water Level (m)	41.90	41.94	41.97	43.57[2]	43.57[2]	43.57[2]
	Future Development Conditions Peak Water Level (m)	41.93	42.00	43.57[2]	43.57[2]	43.57[2]	43.57[2]
Holmes Hill Road – Intersection with Riverbank Drive (J130)	Existing Development Conditions Peak Water Level (m)	20.70[2]	20.70[2]	20.70[2]	20.70[2]	20.70[2]	20.70[2]
	Future Development Conditions Peak Water Level (m)	20.70[2]	20.70[2]	20.70[2]	20.70[2]	20.70[2]	20.70[2]
Mariner's Drive (J53)	Existing Development Conditions Peak Water Level (m)	32.97	33.04	33.25	33.53	33.75	34.01
	Future Development Conditions Peak Water Level (m)	32.97	33.04	33.25	33.53	33.75	34.01

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-13: Summary of Simulated Future Water Level Elevations for Future Development – Holmes Hill Road, Maple Avenue & Chestnut Avenue

Location		Return Period Rainfall Event					
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Alders Avenue – Intersection with Cedar Avenue (CB177)	Historical Climate Conditions Peak Water Level (m)	41.93	42.00	43.57[2]	43.57[2]	43.57[2]	43.57[2]
	Climate Change Conditions Peak Water Level (m)	41.94	43.57[2]	43.57[2]	43.57[2]	43.57[2]	43.57[2]
Holmes Hill Road – Intersection with Riverbank Drive (J130)	Historical Climate Conditions Peak Water Level (m)	20.70[2]	20.70[2]	20.70[2]	20.70[2]	20.70[2]	20.70[2]
	Climate Change Conditions Peak Water Level (m)	20.70[2]	20.70[2]	20.70[2]	20.70[2]	20.70[2]	20.70[2]
Mariner's Drive (J53)	Historical Climate Conditions Peak Water Level (m)	32.97	33.04	33.25	33.53	33.75	34.01
	Climate Change Conditions Peak Water Level (m)	32.99	33.25	33.56	34.25[2]	34.25[2]	34.25[2]

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-14: Summary of Simulated Future Water Levels for Historical Climate – Rand Street, Bog Street & Evangeline Drive

Location		Return Period Rainfall Event					
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Faulkner Drive – Civic #80	Existing Development Conditions Peak Water Level (m)	64.49[2]	64.49[2]	64.49[2]	64.49[2]	64.49[2]	64.49[2]
	Future Development Conditions Peak Water Level (m)	64.49[2]	64.49[2]	64.49[2]	64.49[2]	64.49[2]	64.49[2]

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-15: Summary of Simulated Future Water Level Elevations for Future Development – Holmes Hill Road, Maple Avenue & Chestnut Avenue

Location	Return Period Rainfall Event						
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Faulkner Drive – Civic #80	Historical Climate Conditions Peak Water Level (m)	64.49[2]	64.49[2]	64.49[2]	64.49[2]	64.49[2]	64.49[2]
	Climate Change Conditions Peak Water Level (m)	64.49[2]	64.49[2]	64.49[2]	64.49[2]	64.49[2]	64.49[2]

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-16: Summary of Simulated Future Water Level Elevations for Historical Climate – Rand Street, Bishopville Street & Riverview Road

Location	Return Period Rainfall Event						
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Pleasant Drive (J61)	Existing Development Conditions Peak Water Level (m)	71.08	71.27	71.42	71.55[2]	71.55[2]	71.55[2]
	Future Development Conditions Peak Water Level (m)	71.08	71.27	71.42	71.55[2]	71.55[2]	71.55[2]
Riverview Road – Ditch Inlet (J40)	Existing Development Conditions Peak Water Level (m)	55.42	55.55	55.82	56.19[2]	56.19[2]	56.19[2]
	Future Development Conditions Peak Water Level (m)	55.45	55.72	56.19[2]	56.19[2]	56.19[2]	56.19[2]
Rand Street – Intersection with Bishopville Drive (CB227)	Existing Development Conditions Peak Water Level (m)	61.49	61.55	61.58	61.63	61.67	61.71
	Future Development Conditions Peak Water Level (m)	61.49	61.55	61.58	61.63	61.67	61.71

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Table B-17: Summary of Simulated Future Water Level Elevations for Future Development – Rand Street, Bishopville Street & Riverview Road

Location		Return Period Rainfall Event					
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Pleasant Drive (J61)	Historical Climate Conditions Peak Water Level (m)	71.08	71.27	71.42	71.55[2]	71.55[2]	71.55[2]
	Climate Change Conditions Peak Water Level (m)	71.12	71.42	71.55[2]	71.55[2]	71.55[2]	71.55[2]
Riverview Road – Ditch Inlet (J40)	Historical Climate Conditions Peak Water Level (m)	55.45	55.72	56.19[2]	56.19[2]	56.19[2]	56.19[2]
	Climate Change Conditions Peak Water Level (m)	55.49	56.19[2]	56.19[2]	56.19[2]	56.19[2]	56.19[2]
Rand Street – Intersection with Bishopville Drive (CB227)	Historical Climate Conditions Peak Water Level (m)	61.49	61.55	61.58	61.63	61.67	61.71
	Climate Change Conditions Peak Water Level (m)	61.51	61.58	61.64	61.72	62.11	63.09[2]

[1] Overtopping of bridge deck.
[2] Roadway ponding greater than 0.3 m in depth.

Appendix C

PCSWMM Profiles

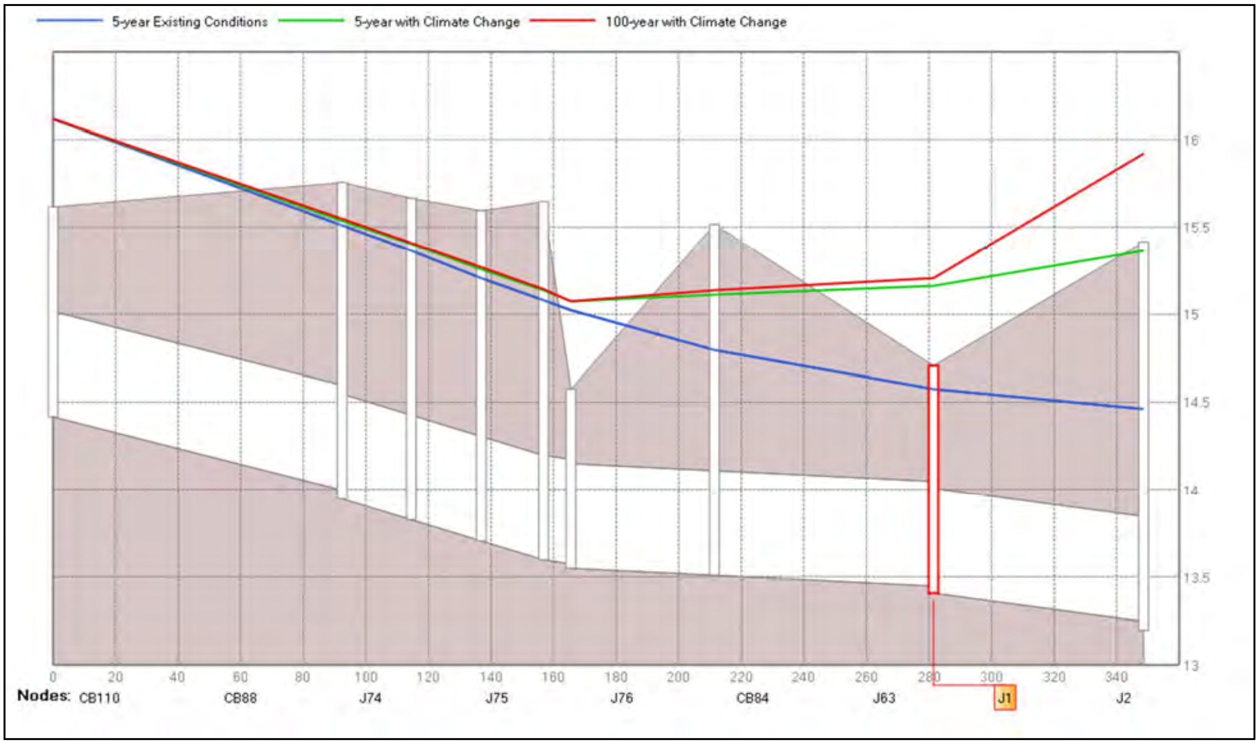

Figure C-1: Comparison of Simulated Water Level Elevations on Chittick Avenue – 5-year Existing Conditions, 5-year with Climate Change & 100-year with Climate Change

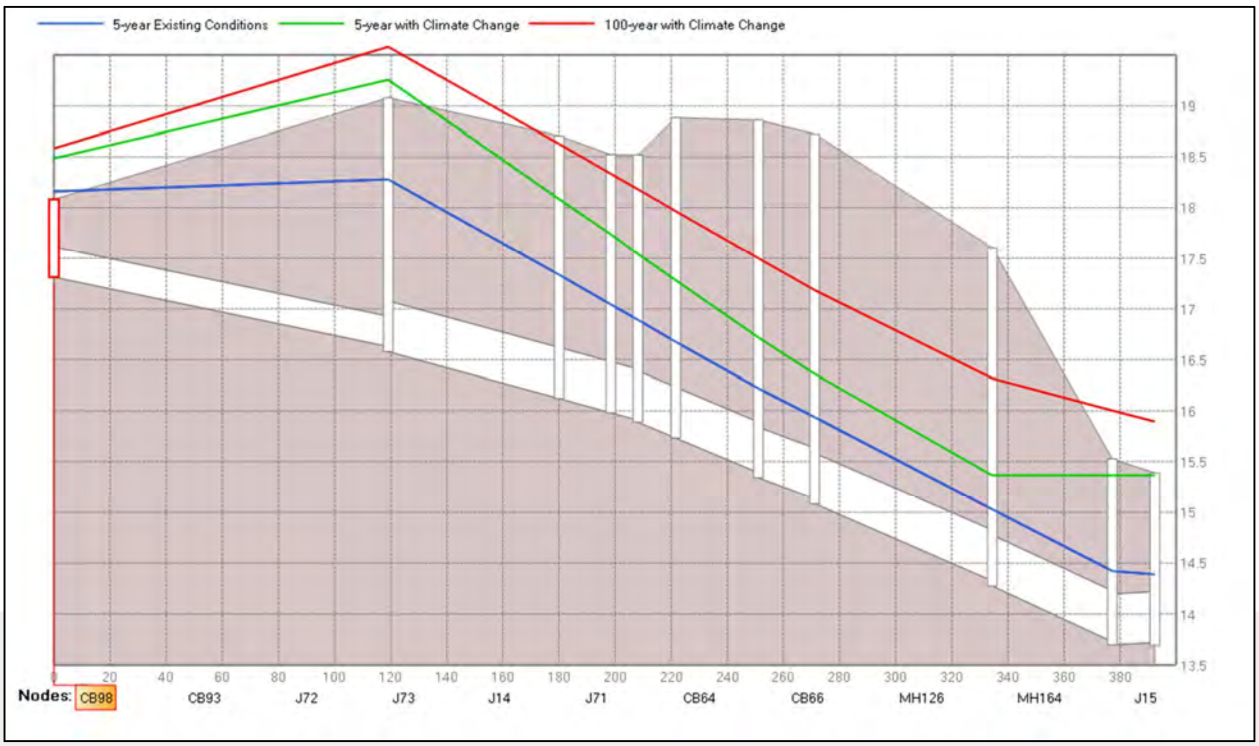

Figure C-2: Simulated Water Level Elevations on Main Street Avenue – 5-year Existing Conditions, 5-year with Climate Change & 100-year with Climate Change

Figure C-3: Simulated Water Level Elevations at Main Street Bridge Crossing – 5-year Existing Conditions, 5-year with Climate Change & 100-year with Climate Change

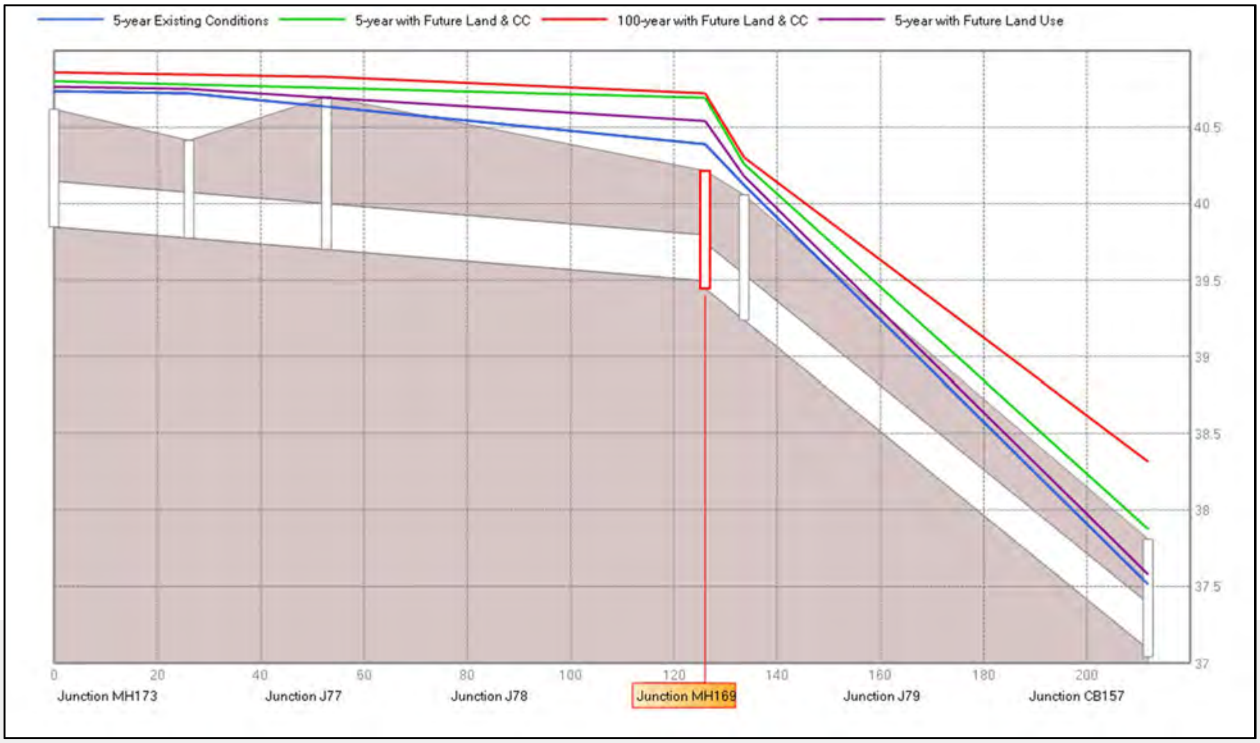

Figure C-4: Simulated Water Level Elevations on Maple Avenue – 5-year Existing Conditions, 5-year with Future Land Use, 5-year with Future Land Use (CC) & 100-year with Future Land Use (CC)

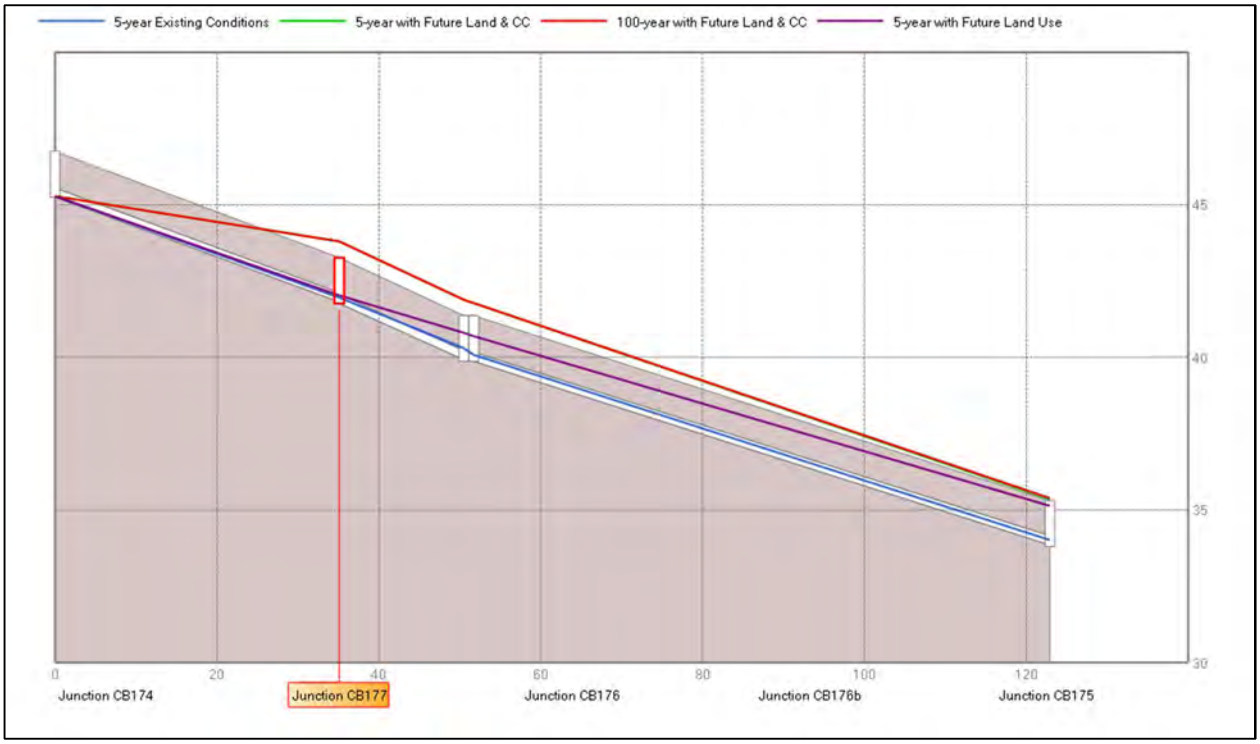

Figure C-5: Simulated Water Level Elevations on Alders Avenue – 5-year Existing Conditions, 5-year with Future Land Use, 5-year with Future Land Use (CC) & 100-year with Future Land Use (CC)

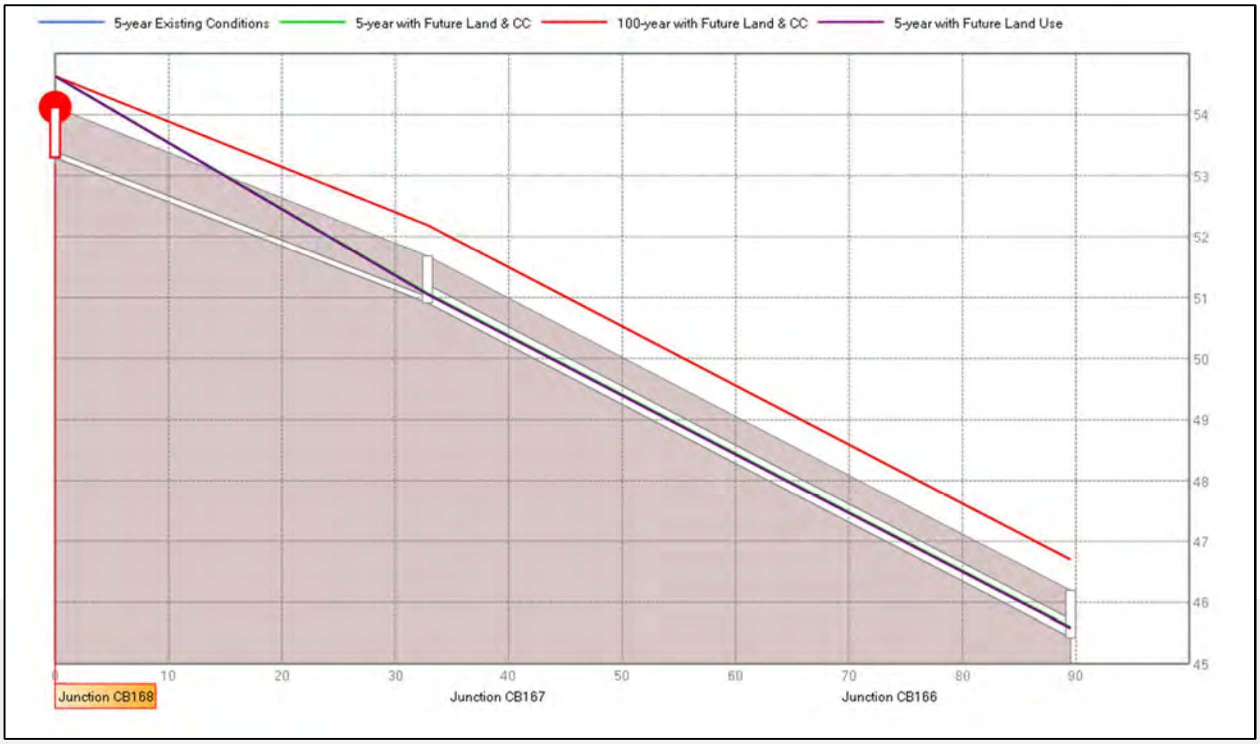

Figure C-6: Simulated Water Level Elevations at Riverview Road – 5-year Existing Conditions, 5-year with Future Land Use, 5-year with Future Land Use (CC) & 100-year with Future Land Use (CC)

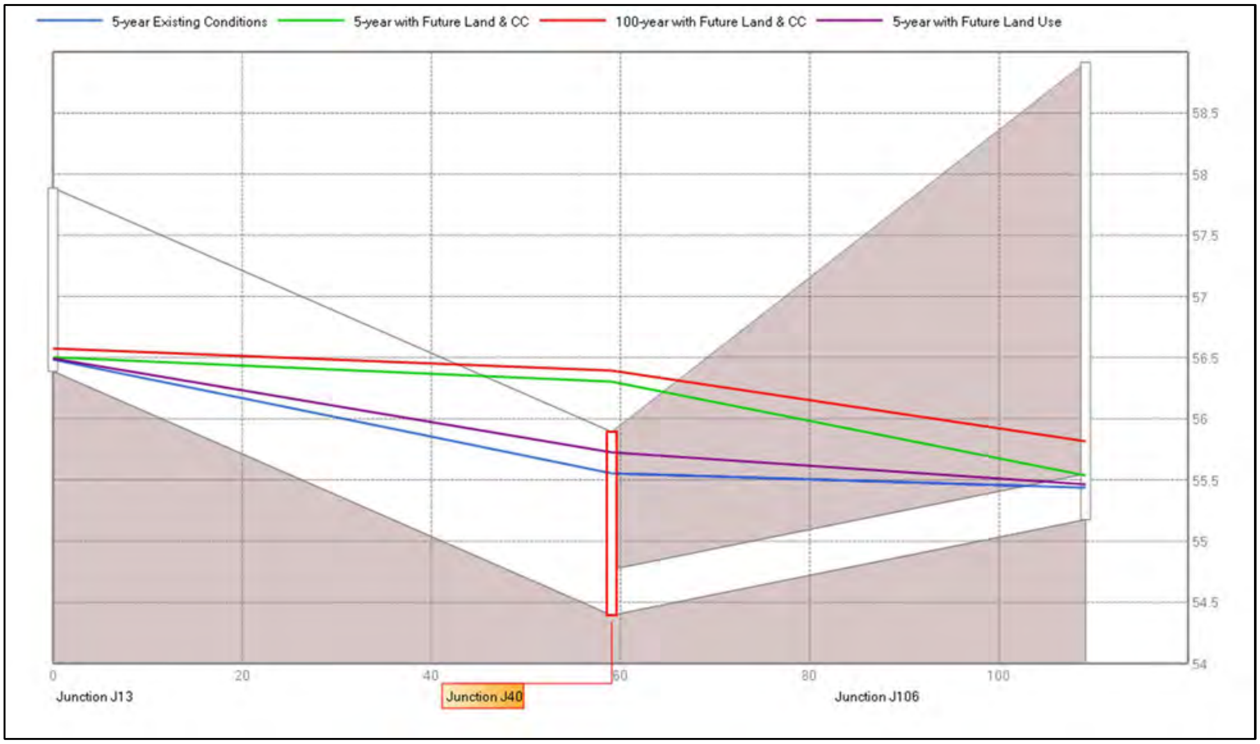

Figure C-7: Simulated Water Level Elevations at Riverview Road Ditch – 5-year Existing Conditions, 5-year with Future Land Use, 5-year with Future Land Use (CC) & 100-year with Future Land Use (CC)

Appendix D

Rainfall Distributions

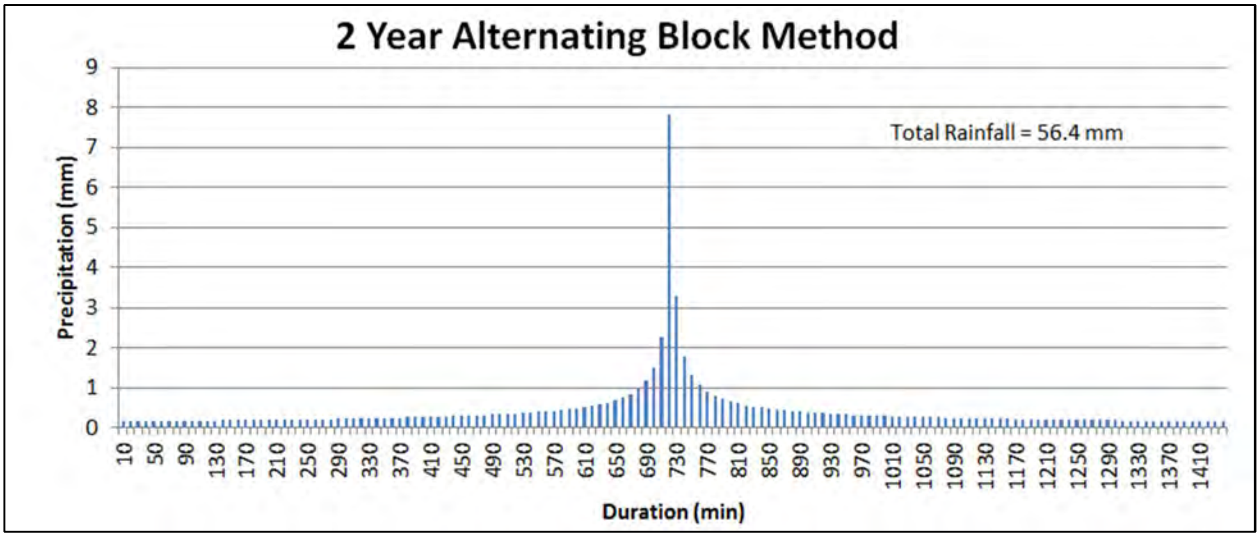

Figure D - 1: 24-Hour Alternating Block Method Distribution for 2-year Historical Event

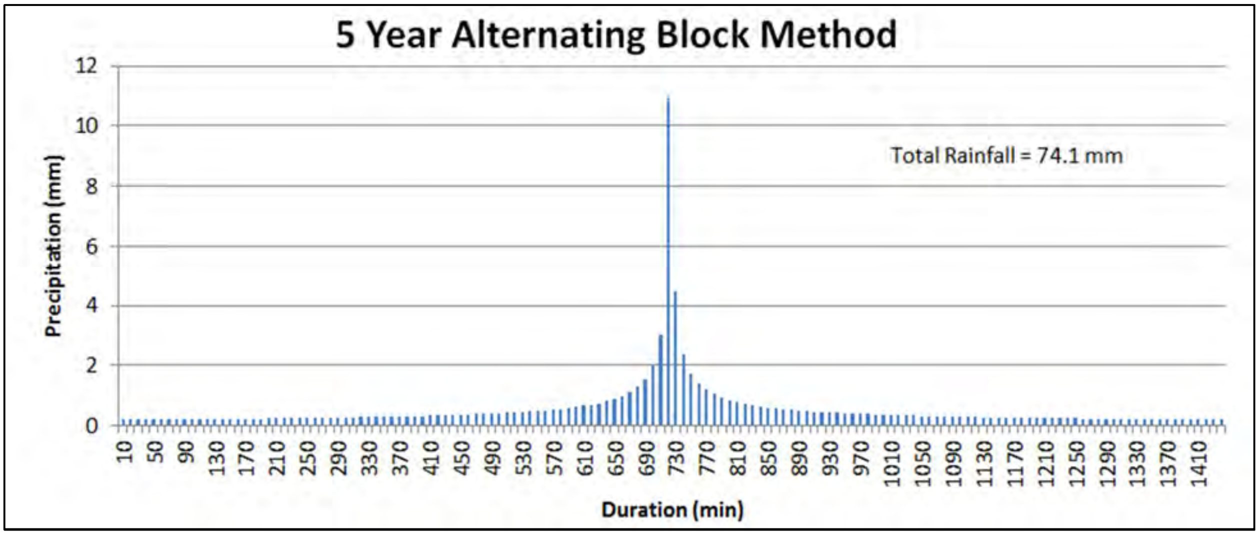

Figure D - 2: 24-Hour Alternating Block Method Distribution for 5-year Historical Event

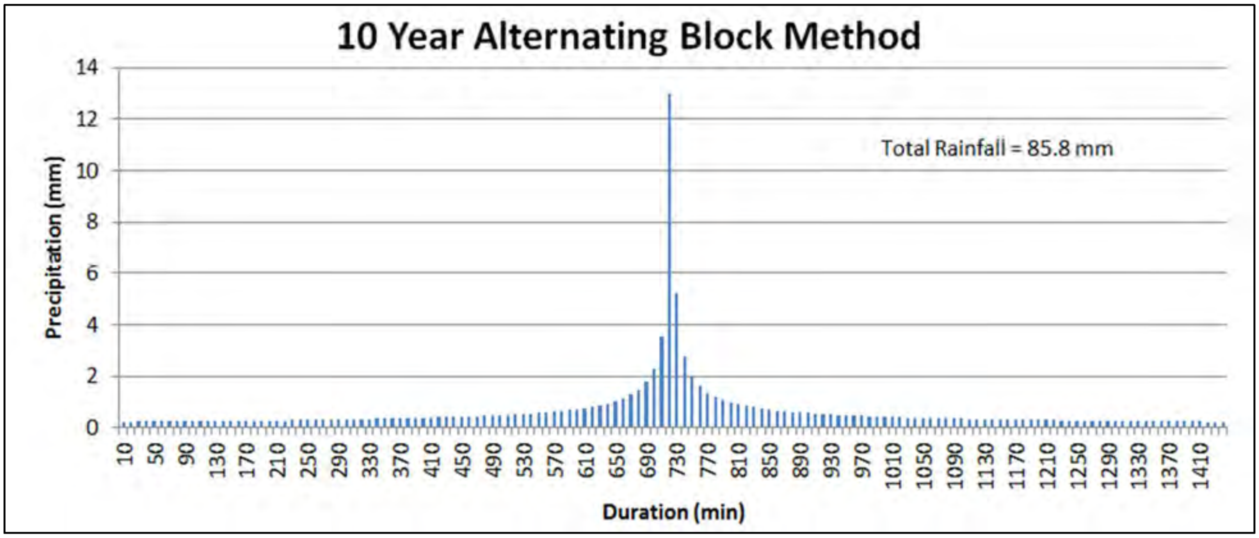

Figure D - 3: 24-Hour Alternating Block Method Distribution for 10-year Historical Event

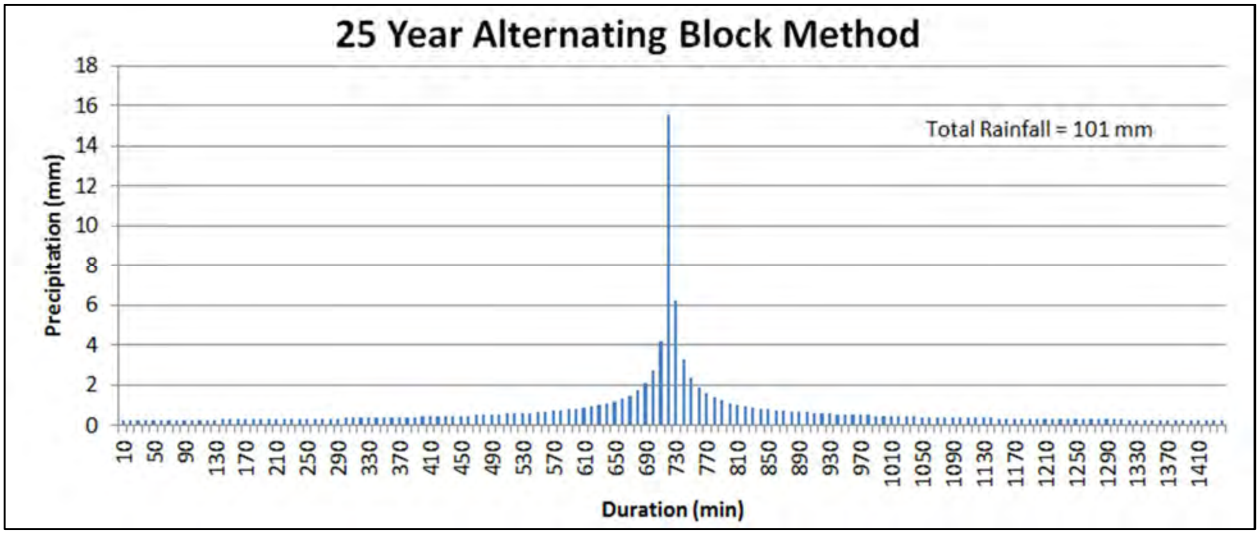

Figure D - 4: 24-Hour Alternating Block Method Distribution for 25-year Historical Event

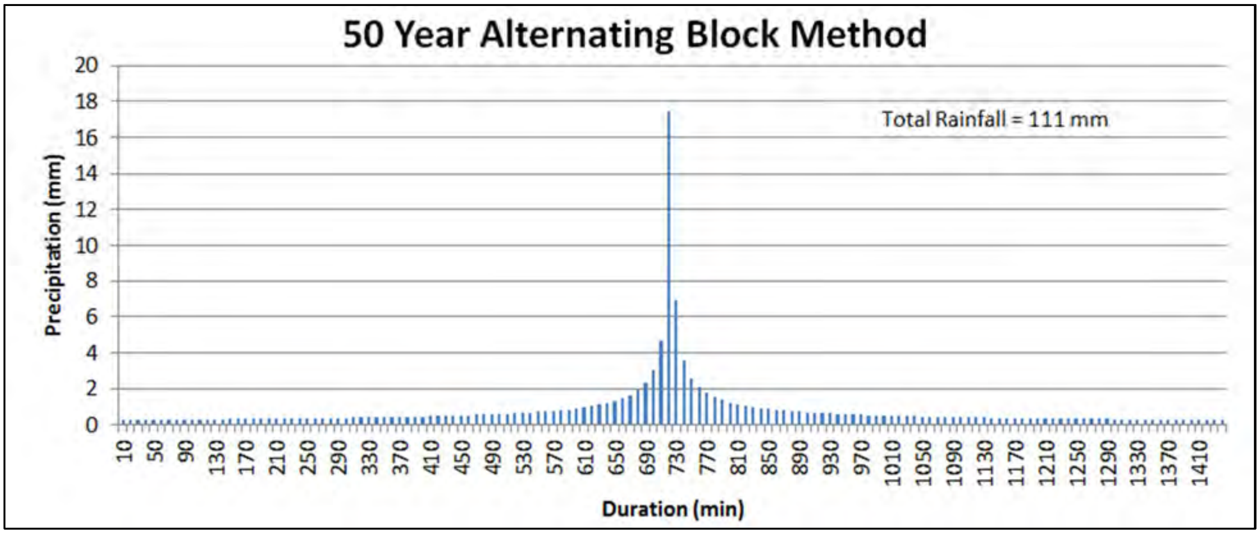

Figure D - 5: 24-Hour Alternating Block Method Distribution for 50-year Historical Event

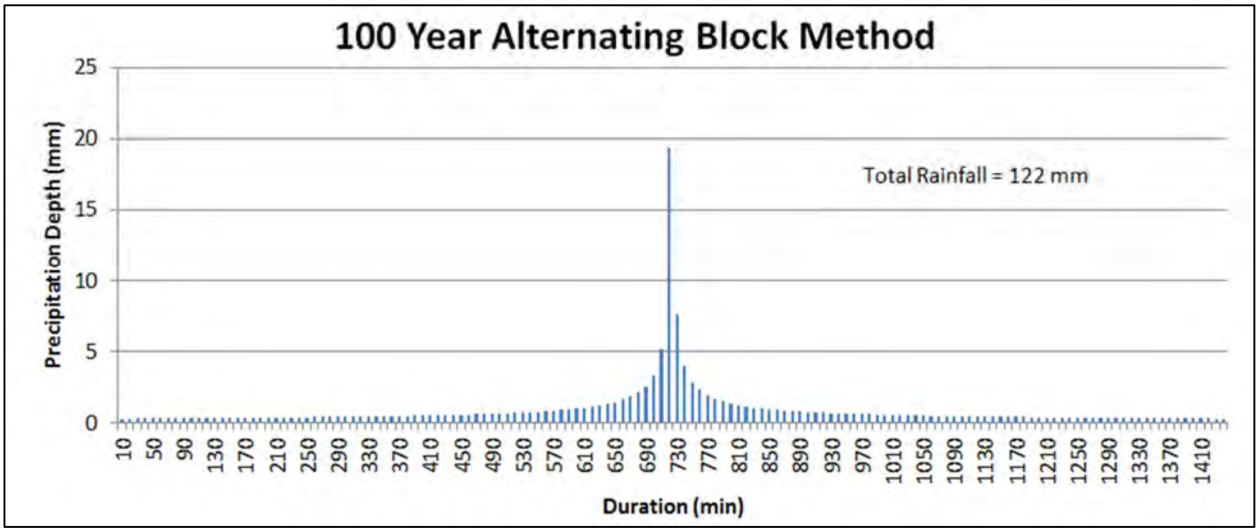

Figure D - 6: 24-Hour Alternating Block Method Distribution for 100-year Historical Event

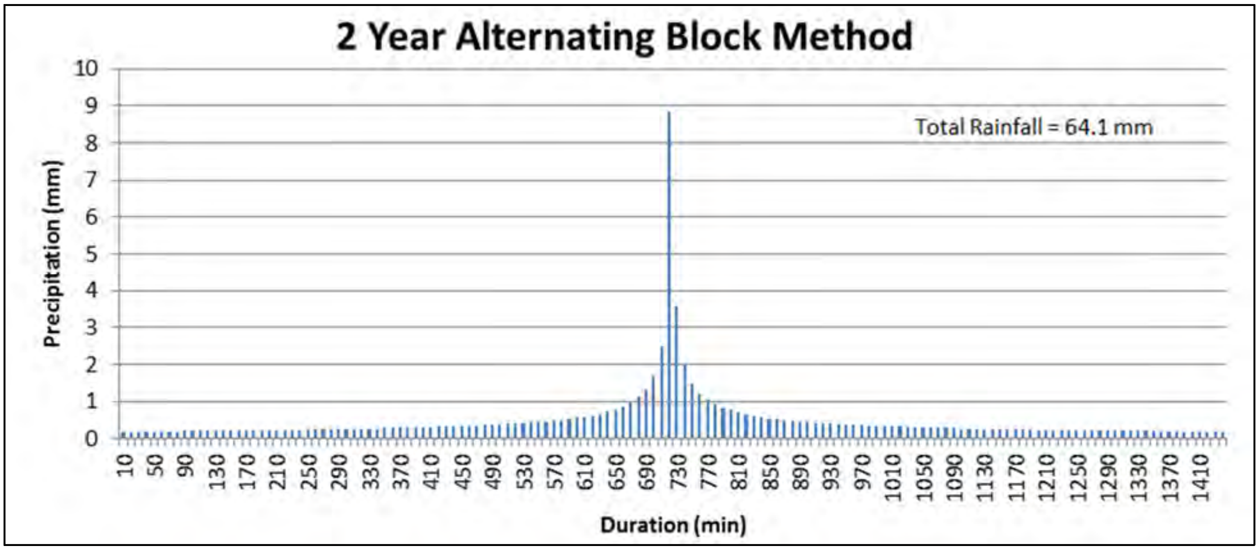

Figure D - 7: 24-Hour Alternating Block Method Distribution for 2-year Climate Change (Future) Event

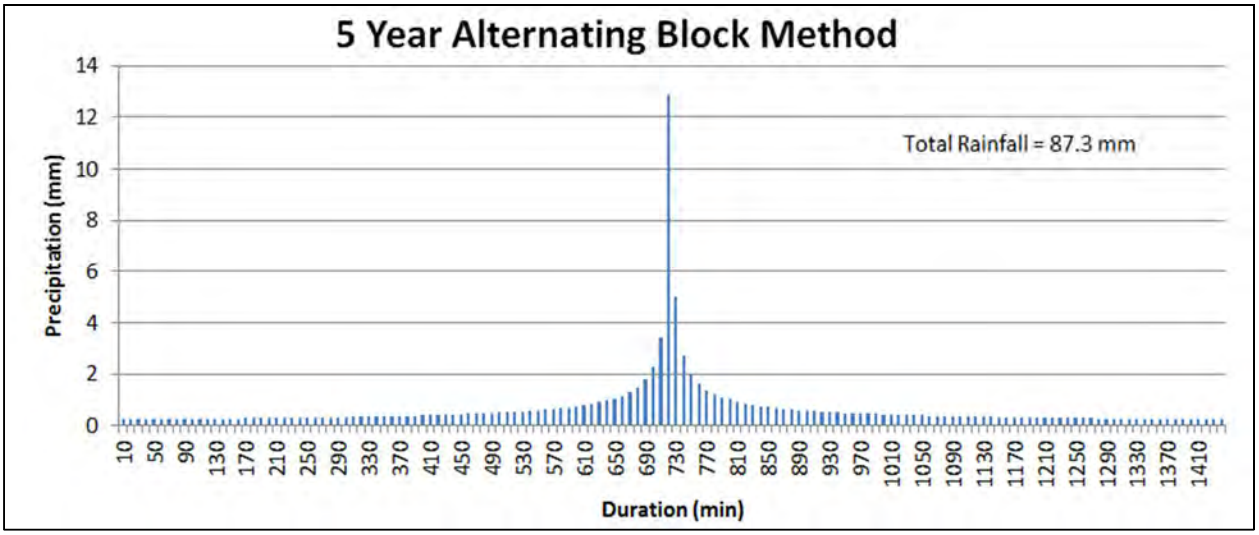

Figure D - 8: 24-Hour Alternating Block Method Distribution for 5-year Climate Change (Future) Event

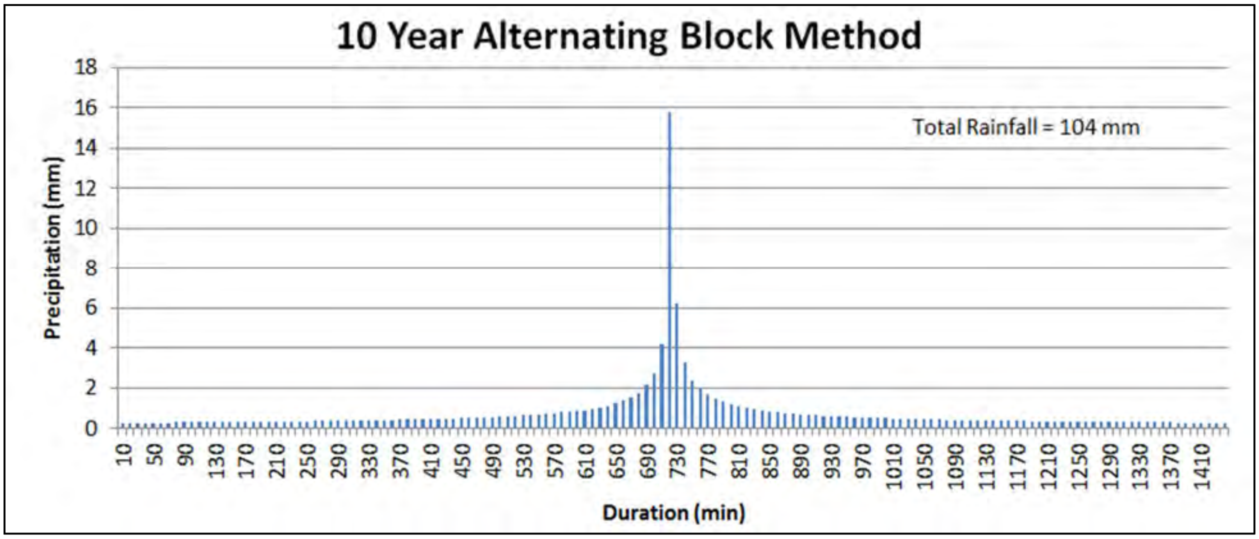

Figure D - 9: 24-Hour Alternating Block Method Distribution for 10-year Climate Change (Future) Event

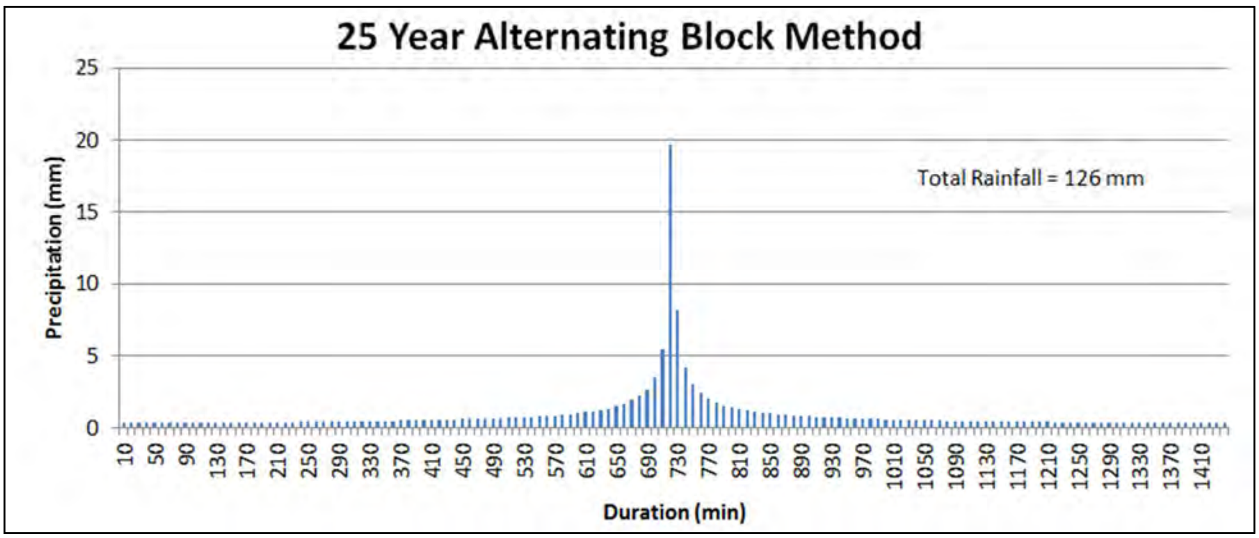

Figure D - 10: 24-Hour Alternating Block Method Distribution for 25-year Climate Change (Future) Event

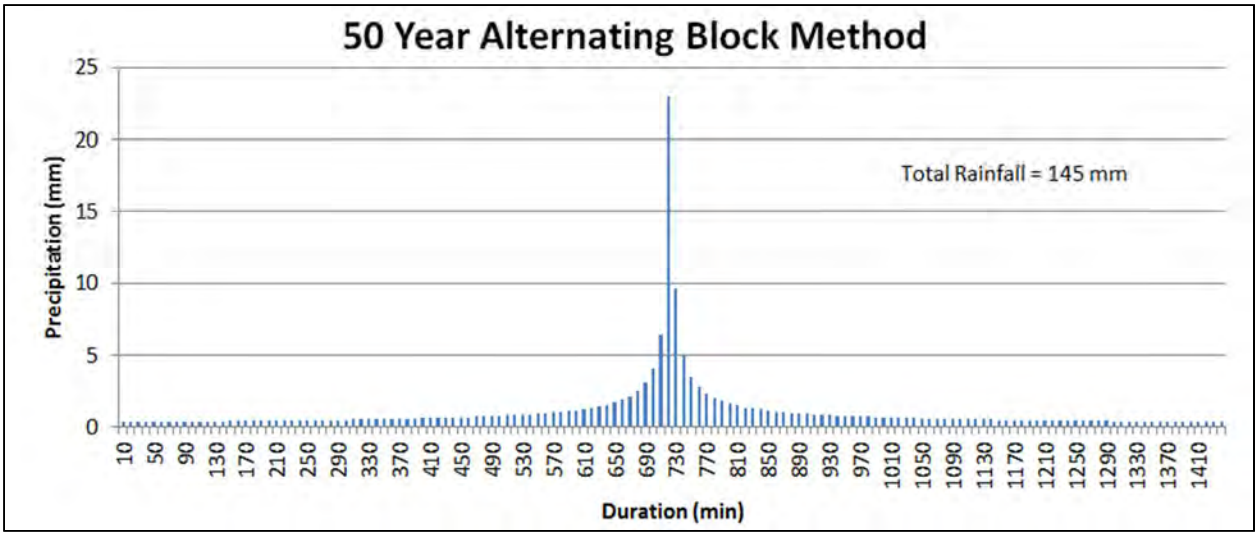

Figure D - 11: 24-Hour Alternating Block Method Distribution for 50-year Climate Change (Future) Event

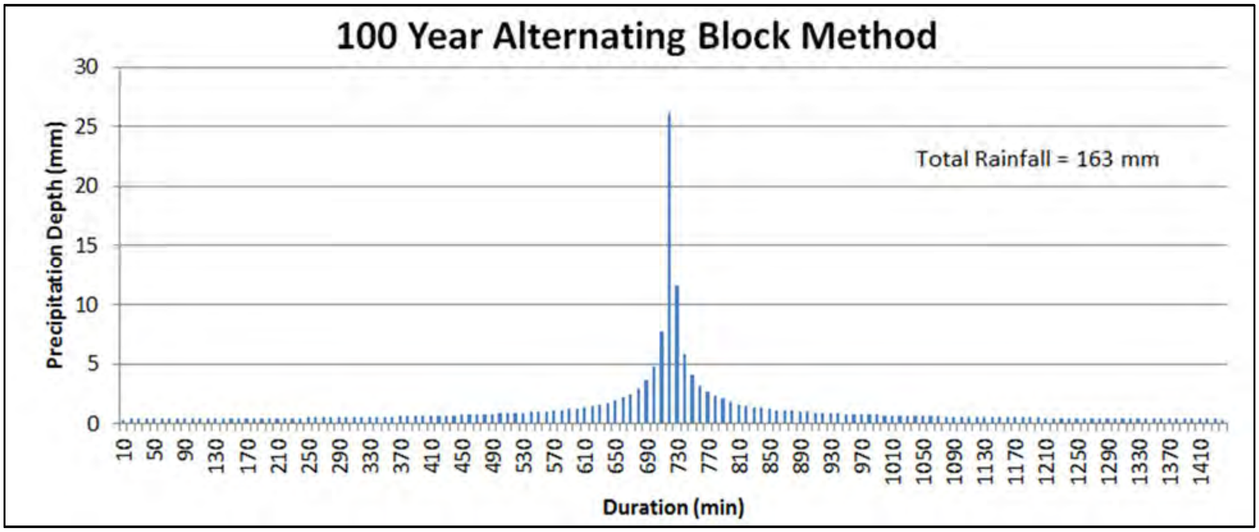

Figure D - 12: 24-Hour Alternating Block Method Distribution for 100-year Climate Change (Future) Event

Appendix E

Meeting Minutes

MEETING MINUTES

DILLON CONSULTING

Subject:	Start-up Meeting, Hantsport Stormwater Management Study
Date and Time:	Tuesday, August 29, 2017, 9:00 am
Location:	Hantsport Office, Municipality of the District of West Hants
Our File:	17-6276-1000

Attendees

Brad Carrigan	Director Public Works, West Hants
Karrie Ritchie	Public Works Admin, West Hants
Peter Johnston	Public Works Superintendent, West Hants
Martin Kehoe	Water Plant Operator, West Hants
Saira Shah	Planner, West Hants
Sarah Deveraux	Project Manager, Dillon
Annamarie Burgess	Water Resources Engineer and Planner, Dillon

Notes

Item	Discussion	Action By
1.	Introduction & Review of scope of work	
1.1.	Scope of work of the assignment was reviewed and confirmed as consistent with Dillon's proposal.	
1.2.	West Hants confirmed their priority for the study is to: • Develop Best Management Practices to guide future development • Identify/confirm infrastructure priorities and approaches to achieve best life cycle with future infrastructure works • Work towards a 'future scenario' of final segments of combined sewer and stormwater pipes being separated	
1.3.	West Hants confirmed Brad Carrigan to be main point of contact for the study	
1.4.	West Hants identified that they have also planned to conduct a full documentation of existing stormwater infrastructure and modeling of the existing and proposed system. West Hants to consider adding this to Dillon's scope of work, recognizing that there will be inherent efficiency in tying the two projects together.	
2.	Description of Existing/Historical Areas of Concern	

2.1. West Hants provided an overview of areas of concern inside the community of Hantsport that frequently experience flooding, as well as additional commentary about the existing infrastructure in the area. Areas discussed include:

- Riverview Road has water on sidewalks and across the road every spring, and after heavy rains
- Avon Street has no SWM infrastructure. It floods after heavy rains but the water quickly infiltrates and a few hours after a rain evet there is no longer any standing water.
- Maple Drive has French drains
- The sluice gate at the old rail tracks has failed and has not been repaired
- Older houses are tied into the stormwater sewer, though newer houses are not
- Roads as they are rebuilt are constructed with separated sanitary and storm sewers, however, the system is incomplete and all separated system re-converge

3. Available Information & Field Review

3.1. West Hants to provide available information:

- infrastructure sizes
- LiDAR coverage of Hantsport
- Gauge data of the Avonmore River collected by CBCL in Falmouth study

3.2. Dillon will install stream gauge as soon as possible. *Post Meeting Note: Dillon Staff were installing the stream gauge September 5, 2017 on Halfway River.*

3.3. Dillon will complete a field review of the study area and gather supplemental survey as required.

4. Proposal Debrief

4.1. Dillon requested any feedback West Hants could provide on the proposal submission in order to improve future submissions. West Hants identified strengths in having strong representative projects (relevant size and scope), and clearly written and easy to follow approach.

Errors and/or Omissions

These minutes were prepared by Annamarie Burgess who should be notified of any errors and/or omissions.

Appendix F

Capacity of Undersized Pipes

Table F-1 through F-3 provide an indication of the percentage of capacity of undersized pipes for existing conditions, future land use conditions, and future land use with climate change conditions. A percentage of capacity over 100% is a pipe that is undersized by that relative amount. For example a pipe at 167% capacity has a flow that exceeds its capacity by 67%. The pipes identified in these tables are limited to those that have resulted in roadway flooding for the 5-year return period rainfall event. The purpose of this information is to assist the Municipality in prioritizing improvements and upgrades.

The theoretical pipe capacities were calculated assuming pipes flowing full (e.g. the flow depth of a 300 mm diameter pipe is 300 mm) for the 5-year return period rainfall event. It is noteworthy that the stormwater network was modeled as a system with surcharging resulting from inflows, downstream water levels, or a combination thereof.

Table F-1 presents the estimated percentage of capacity of the undersized infrastructure for existing conditions.

Table F-1: Summary of Undersized Pipe Infrastructure for Existing Conditions

Location	Pipe Diameter (mm)	Theoretical Capacity (m^3/s)	Simulated Inflow (m^3/s)	Percentage of Capacity(%)[1]
Main Street – Civic #25 (CB98)	300	0.063	0.105	167%
Main Street – Civic #39 (CB93)	500	0.283	0.445	157%
Main Street Bridge (J46)	NA			
Chittick Avenue – Civic #22 (J1)	600	0.261	0.383	147%
Chittick Road - Civic #47 (CB84)	600	0.160	0.596	373%
Maple Avenue – Civic #24 (MH171)	300	0.044	0.074	168%
Maple Avenue – Civic #30 (MH169)	300	0.139	0.172	124%
Riverview Road – Civic #8 (CB158)	300	0.230	0.175	76%
Riverview Road – Civic #23 (CB168)	100	0.012	0.018	150%
Soccer Field (J137)	200	0.037	0.048	130%
William Street – Civic #14 (CB57)	450	0.150	0.133	89%
Prince Street – Civic #11 (CB40)	300	0.057	0.195	342%
William Street – Intersection with Oak Street (CB47)	450	0.166	0.233	140%
Prince Street – Intersection with Davidson Road (CB23)	CB INTAKE CAPACITY INSUFFICIENT			
Willow Street – Intersection with Main Street (CB137)	CB INTAKE CAPACITY INSUFFICIENT			

Alders Avenue – Intersection with Cedar Avenue (CB177)	300	0.290	0.188	65%
Holmes Hill Road – Intersection with Riverbank Drive (J130)	CB INTAKE CAPACITY INSUFFICIENT			
Mariner's Drive (J53)	2 x 250	0.442	0.198	45%
Faulkner Drive – Civic #80 (CB225)	200	0.032	0.137	428%
Pleasant Drive (J61)	300	0.155	0.266	172%
Riverview Road – Ditch Inlet (J40)	375	0.021	0.179	852%
Rand Street – Intersection with Bishopville Drive (CB227)	600	1.257	0.469	37%

[1]These values are based on the simulated system capacity and would be expected to vary in response to upstream/downstream upgrades. It is recommended that the impact on overall system performance be evaluated for the preferred upgrade scenarios.

Table F-2 shows the estimated percentage under capacity of pipes for future land use conditions. These results assume the construction of approximately 97 new homes and a nursing home over the 50 year horizon.

Table F-2: Summary of Undersized Pipe Infrastructure for Future Development

Location	Pipe Diameter (mm)	Theoretical Capacity (m^3/s)	Simulated Inflow (m^3/s)	Percentage of Capacity (%)[1]
Maple Avenue – Civic #24 (MH171)	300	0.044	0.074	168%
Maple Avenue – Civic #30 (MH169)	300	0.139	0.187	135%
Riverview Road – Civic #8 (CB158)	300	0.230	0.18	78%
Riverview Road – Civic #23 (CB168)	100	0.012	0.018	150%
Alders Avenue – Intersection with Cedar Avenue (CB177)	300	0.290	0.249	86%
Holmes Hill Road – Intersection with Riverbank Drive (J130)	CB INTAKE CAPACITY INSUFFICIENT			
Mariner's Drive (J53)	2 x 250	0.442	0.198	45%
Faulkner Drive – Civic #80 (CB225)	200	0.032	0.137	428%
Pleasant Drive (J61)	300	0.155	0.266	172%
Riverview Road – Ditch Inlet (J40)	375	0.021	0.254	1210%
Rand Street – Intersection with Bishopville Drive (CB227)	600	1.257	0.469	37%

[1]These values are based on the simulated system capacity and would be expected to vary in response to upstream/downstream upgrades. It is recommended that the impact on overall system performance be evaluated for the preferred upgrade scenarios.

Table F-3 summarizes the estimated percentage under capacity of pipes identified for future land use conditions with climate change.

Table F-3: Summary of Undersized Pipe Infrastructure for Future Development with Climate Change

Location	Pipe Diameter (mm)	Theoretical Capacity (m^3/s)	Simulated Inflow (m^3/s)	Percentage of Capacity (%)[1]
Main Street – Civic #25 (CB98)	300	0.063	0.164	260%
Main Street – Civic #39 (CB93)	500	0.283	0.522	184%
Main Street Bridge (J46)	NA			
Chittick Avenue – Civic #22 (J1)	600	0.261	0.385	148%
Chittick Road - Civic #47 (CB84)	600	0.160	0.627	392%
School Street – Civic #12 (J26)	375	0.178	0.266	149%
Maple Avenue – Civic #24 (MH171)	300	0.044	0.074	168%
Maple Avenue – Civic #30 (MH169)	300	0.139	0.205	147%
Riverview Road – Civic #8 (CB158)	300	0.230	0.222	97%
Riverview Road – Civic #23 (CB168)	100	0.012	0.023	192%
Soccer Field (J137)	200	0.037	0.073	197%
William Street – Civic #14 (CB57)	450	0.150	0.165	110%
Prince Street – Civic #11 (CB40)	300	0.057	0.216	379%
William Street – Intersection with Oak Street (CB47)	450	0.166	0.286	172%
Prince Street – Intersection with Davidson Road (CB23)	CB INTAKE CAPACITY INSUFFICIENT			
Willow Street – Intersection with Main Street (CB137)	CB INTAKE CAPACITY INSUFFICIENT			
Alders Avenue – Intersection with Cedar Avenue (CB177)	300	0.290	0.310	107%
Holmes Hill Road – Intersection with Riverbank Drive (J130)	CB INTAKE CAPACITY INSUFFICIENT			
Mariner's Drive (J53)	2 x 250	0.442	0.254	57%
Faulkner Drive – Civic #80 (CB225)	200	0.032	0.173	541%
Pleasant Drive (J61)	300	0.155	0.362	234%
Riverview Road – Ditch Inlet (J40)	375	0.021	0.327	1557%
Rand Street – Intersection with Bishopville Drive (CB227)	600	1.257	0.578	46%

[1]These values are based on the simulated system capacity and would be expected to vary in response to upstream/downstream upgrades. It is recommended that the impact on overall system performance be evaluated for the preferred upgrade scenarios.