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City-Wide Flood Risk Assessment and Storm Outlet Review

Final Report

Submitted by:

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1 INTRODUCTION

1.1 General

The City of Pembroke (herein referred to as “the City” or “Pembroke”) is situated along the Ottawa River and is traversed by Muskrat River and its tributary, Indian River, which converge at the northern end of the city. Historically, Pembroke has experienced significant amounts of flooding, as a result of both riverine flooding and storm sewers reaching capacity during large storm events, which have in turn caused damage to storm sewer outlets. In the City of Pembroke Official Plan (City of Pembroke, 2016), the City recognized that riverine flooding stems from high water elevations in the Ottawa river, as well as within the Muskrat River and the Indian River, but that 100-year flood elevations were only defined for the Ottawa River.

The City retained Aquafor Beech Limited (“Aquafor”) in 2021 to undertake the City-Wide Flood Risk Assessment and Storm Outlet Review, which is partially funded by the National Disaster Mitigation Program. This study presents opportunities for not only addressing historical flooding issues, but also regulating future development in flood-prone areas and rehabilitating/replacing storm outlets at risk of failure.

1.2 Study Area

The study area is shown in **Figure 1-1** and includes flooding issues in riverine areas, flooding caused by insufficient sewer conveyance in certain locations, and deteriorating storm sewer outlets.

Figure 1-1: Study Area

1.3 Study objectives

The key objectives of this study are as follows:

- Characterize the Muskrat River watershed
- Select and utilize appropriate hydrologic and hydraulic models for the watershed
- Generate 100-year flood elevations in the Indian and Muskrat rivers
- Identify areas sensitive to riverine flooding under the 100-year flood event
- Create a storm sewer hydrologic and hydraulic model
- Evaluate storm sewer performance under the 5-year and 100-year rainfall events
- Identify storm sewer system deficiencies and associated flood risks
- Propose solutions to urban flooding issues caused by undersized sewer pipes
- Identify the most at-risk storm sewer outlets through field assessments, and design remedial measures

2 BACKGROUND

The City provided a number of background documents and files to support the study. Images providing a general overview of flooding problems throughout Pembroke are shown in **Figure 2-1**, with additional photos shown in **Appendix A**. Of particular note is flooding that occurred during on August 1st and August 10th, 2003, which had rainfall volumes of 100 mm and 110 mm, respectively. Both events occurred over a 2-3 hr time period, thereby exceeding the 100-year return period. As a result, flows exceeded the storm sewer capacity and caused damage to private and municipal property. In addition, several houses along Doran Street were inundated with water due to flooding of the Indian River, and a portion of the road was submerged.

According to the City's 2016 Official Plan, the 100-year flood elevation of the Ottawa River at Pembroke is 113.9 m. A map provided by the City showing flooded areas under the 100-year Ottawa River flow event is presented in **Figure 2-2**. More detailed maps of flooding under this event are shown in **Appendix B**.

Other materials provided by the City included manhole, catchbasin, and storm sewer pipe network GIS files, as well as a number of as-built drawings containing information about sewer pipes. The analysis of storm sewer performance was limited to 4 study areas, shown in **Figure 1-1** above: the Trafalgar Road area, the Lake Street area, the Doran Street area, and the Angus Campbell Drive area. The City also identified 17 storm outfalls to be assessed (**Figure 1-1**).

The sewer pipe GIS files contained pipe size information for 44% of the pipe network, but did not contain any invert elevations. Upon review of the built drawings, Aquafor determined that 39 of the 201 manholes (19%) located within the study areas had sufficient elevation and sizing data for all pipes shown in the drawings, leaving 162 manholes to be surveyed.

Other pertinent information was retrieved from other sources, including: LiDAR elevation data, road networks, waterbodies, stream networks, geological information, land use, and soil classifications obtained from the Ministry of Natural Resources and Forestry; rainfall data collected by Environment Canada; and flow data provided by Water Resources Canada.

Various Locations of Flooding

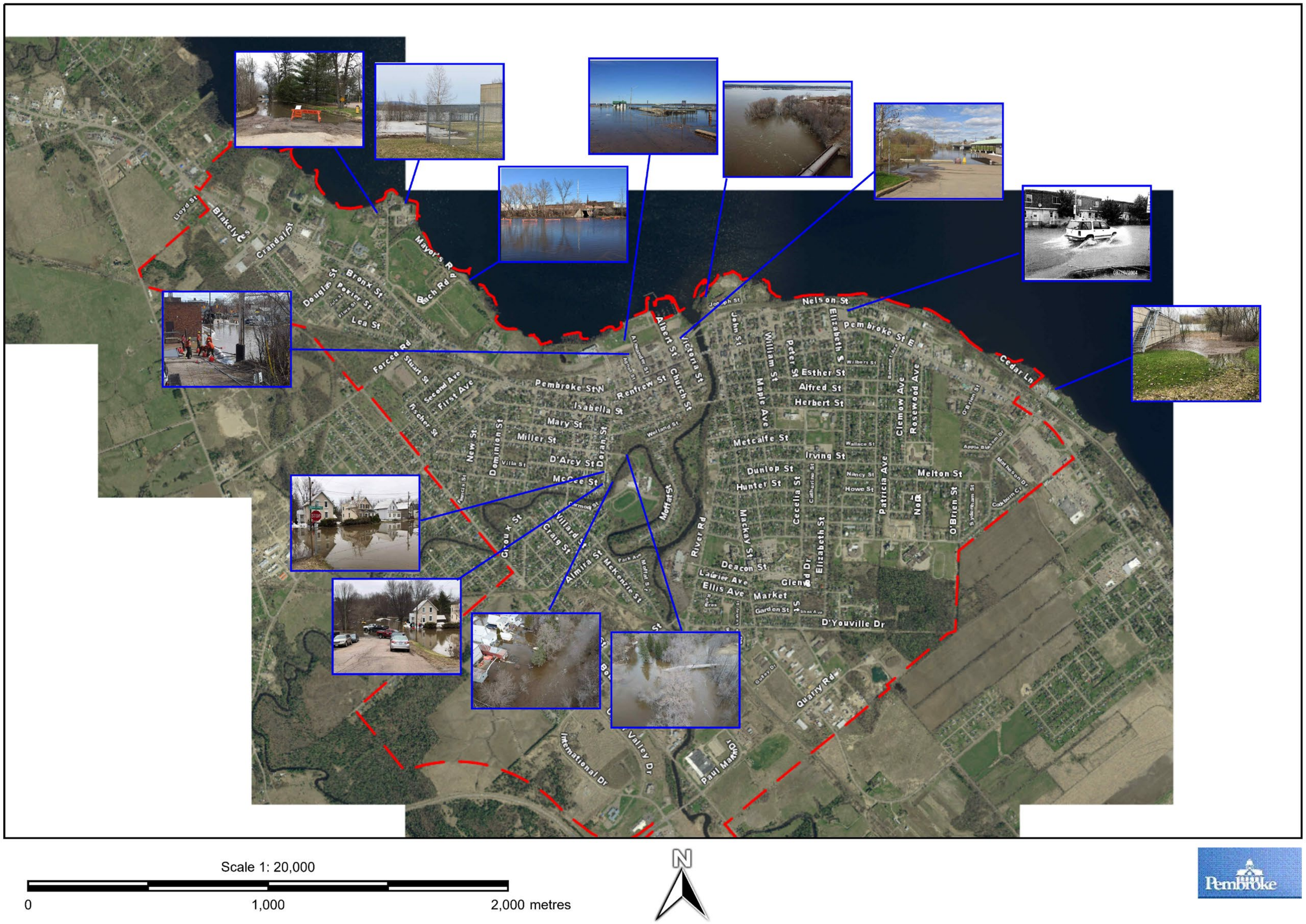


Figure 2-1: Historical Flooding within Pembroke

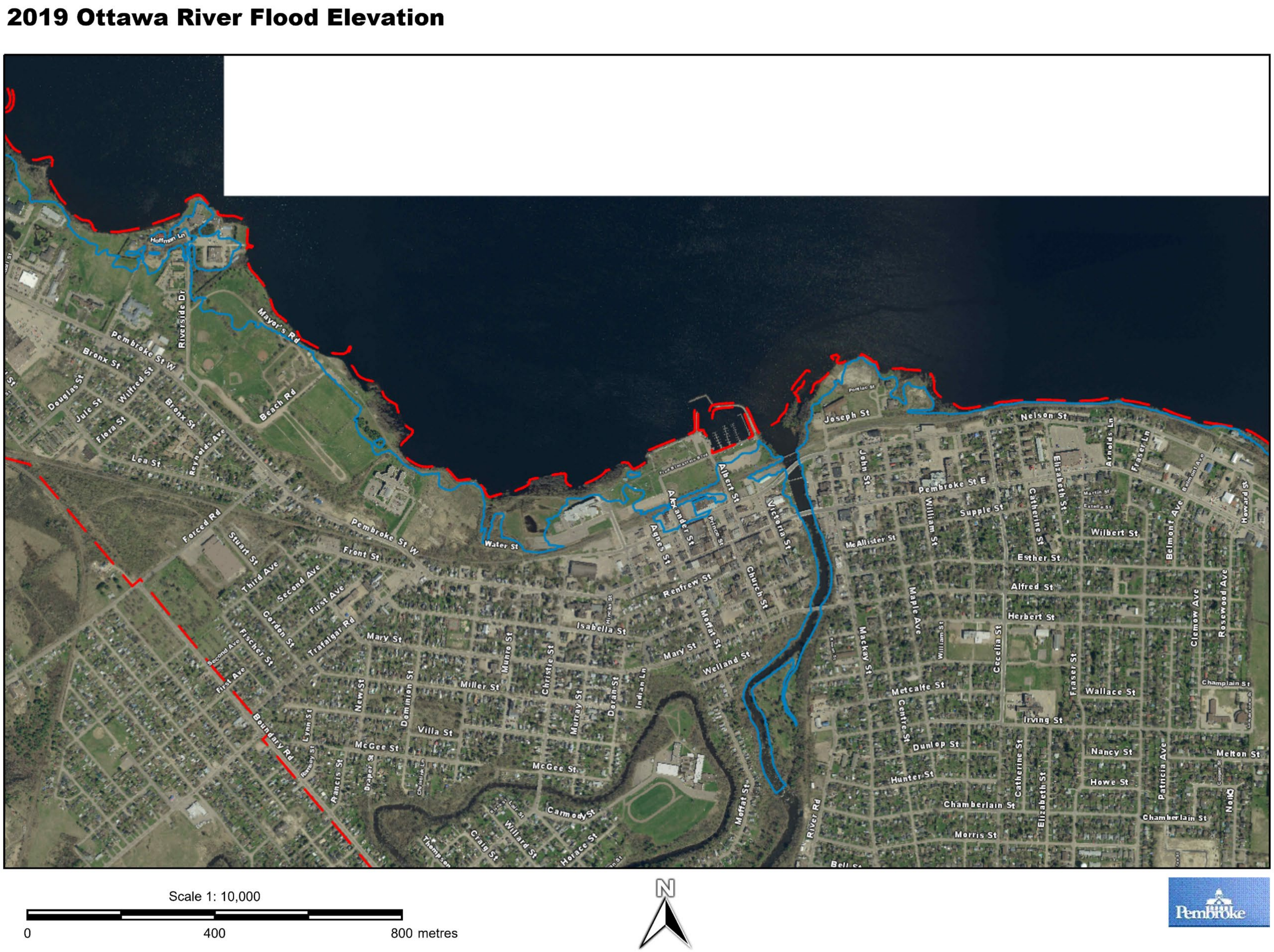


Figure 2-2: Ottawa River Floodlines (in blue) within the Study Area (dashed red lines) for the 100-year Flow Event

3 HYDROLOGICAL MODEL SETUP FOR FLOODPLAIN MAPPING

3.1 General

This section describes the hydrological model that was developed in HEC-HMS to predict riverine peak flows throughout Pembroke. The HEC-HMS model accounts for precipitation runoff from the entire watershed that drains into Muskrat River and Indian River, because the majority of the water flowing through these rivers originates upstream of Pembroke. The results from the hydrological model were used as inputs for the HEC-RAS hydraulic model described in **Section 4**, which was used to define the extent and location of existing flooding followed by an assessment of alternatives to alleviate the issues. The primary focus with respect to riverine flooding was along Muskrat and Indian Rivers, as discussed in the start-up meeting. Flooding issues with respect to the Ottawa River will be limited to assessing alternatives for protecting municipal properties.

A third model (described in **Section 1**) incorporating both urban hydrology and pipe hydraulics was developed in PC-SWMM to assess the capacity of the storm sewer system and associated roadways, from which recommendations were made to reduce flooding risks.

3.2 Watershed Characterization

3.2.1 Drainage Network

The Muskrat River drains a watershed area of approximately 1144 km^2 in size. As illustrated in **Figure 3-1**, the river is characterized by a main drainage branch and a tributary – Indian River – which converge at the southern side of Pembroke. The main branch ultimately discharges into the Ottawa River 1.8 km downstream of the confluence with Indian River.

The Muskrat River extends approximately 99 km from its headwaters to its outlet at the Ottawa River. Upstream of the confluence, it drains an area of roughly 674 km^2 that contains two large lakes – Doré Lake and Muskrat Lake – as well as a number of smaller lakes. In total, lakes occupy an area of 52 km^2, i.e., 7.7 % of the main branch drainage area upstream of the confluence.

The Indian River extends roughly 75 km from its headwaters to the confluence with Muskrat River, draining an area of approximately 468 km^2. The subwatershed contains numerous small lakes that occupy a total area of 25 km^2, corresponding to 5.4 % of the Indian River subwatershed area.

A summary of slopes of the Indian River and the main branch of Muskrat River is show in **Table C.1** (**Appendix C**). The Indian River begins with a high slope at its headwaters (0.031 m/m), then becomes more gradual (0.0020 m/m), before flattening out prior to the confluence with Muskrat River. This flat portion of the river has an average slope of 0.00058 m/m but is composed of a series of alternating flat reaches (slopes <0.00022 m/m) and riffles (slopes >0.0055 m/m).

Muskrat River follows a similar pattern: slopes are highest at the upstream extents (0.012 m/m), then become more gradual (0.001 - 0.003 m/m, not including Lake Doré and Muskrat Lake), and finally decrease to 0.0002 m/m before discharging into the Ottawa River.

Figure 3-1: Muskrat River and Indian River Drainage Areas, Stream Network, and Monitoring Stations

3.2.2 Geological Setting

The bedrock (Paleozoic) geology within Pembroke consists of the Gull River, Bobcaygeon, and Rockcliffe Formations. The Gull River and Bobcaygeon Formations extend along the eastern side of the Muskrat River watershed, with undifferentiated Precambrian formations occupying the rest of the watershed.

Surficial geology is shown in **Figure 3-2**. Bedrock complexed with till and glaciofluvial Quaternary deposits is found throughout much of the watershed, dominating the western portion that drains into the Indian River. Fine-textured glaciomarine deposits and till are prevalent along the eastern side of the watershed, surrounding the main branch of Muskrat River and its associated lakes.

Figure 3-2: Surficial Geology within the Muskrat River Watershed

3.2.3 Land Use and Hydrological Soil Groups

Figure 3-3 shows that land uses within the watershed are predominantly rural in nature, consisting primarily of forests (59%) and agriculture (25%). Impervious surfaces attributed with urbanization (roads, buildings, parking lots, etc.) are mostly concentrated in Pembroke and only account for ~1% of the total watershed area. Other land use types include open water (7%), wetlands (6%), and open space such as lawns (1%).

As illustrated **Figure 3-4**, soil classified as Hydrological Soil Group B dominates the western portion of the watershed that is drained by Indian River, and generally coincides with complexes of near-surface bedrock, till, and glaciofluvial Quaternary deposits. Areas in the western portion of the watershed that had missing hydrological soil group data were assigned Soil Group B, given that nearby areas were classified as Group B and had similar land use and quaternary geology. In total, Soil Group B occupies 66% of the watershed area. Soil Group D accounts for 19% of the area and is associated with fine, poorly drained soils located in the eastern portion of the watershed. Soil Groups A and C are also present, comprising 13% and 2% of the total area, respectively.

The use of soil hydrological classification and land use for calculating runoff is described in **Section 3.11**, with CN values shown in **Table 3.4** and **Table 3.5**.

Figure 3-3: Land Use within the Muskrat River Watershed

Figure 3-4: Hydrological Soil Group Distribution within the Muskrat River Watershed

3.3 Background Data Acquisition

A LiDAR digital elevation model (DEM) collected by the Natural Resources Canada between 2019-2020 as part of the "Rivière Outaouais" ("Ottawa River") mapping project was available for all areas draining into the main branch of the Muskrat River. However, LiDAR data was not available for the majority of the watershed draining into the Indian River. Wherever possible, LiDAR data was used for determining elevations and slopes of model components. In areas where LiDAR data was not available, Aquafor opted to use a 30 m hydrology-enforced DEM that was produced by the Ontario Ministry of Natural Resources and Forestry (MNRF). The DEM was created by first compiling various elevation maps throughout the province, then forcing the DEM such that flow accumulated along mapped watercourses. The enforced DEM was selected instead of MNRF's 2 m imagery-derived DEM because Aquafor's assessment of the imagery-derived DEM showed that it could not reliably predict watershed boundaries and flow accumulation.

Two hydrometric gauges (shown in **Figure 3-1**) collect flow data within the watershed and are located upstream of the confluence of the Indian River and the Muskrat River. The first flow gauge measures flow through the main branch of the Muskrat River and is located 2.4 km upstream of the confluence; the other measures flow through the Indian River and is located 2.1 km upstream of the confluence. Data reviewed by the Water Survey of Canada was available in 1-hour intervals or less between 2007-2020 for Indian River, and from 1969-1978 and 2008-2020 for Muskrat River.

The Environment Canada "Pembroke Climate" station (ID #6106367), located at the Pembroke & Area Airport, was the only nearby climate station that collected rainfall data on an hourly basis. Data at this station was available between 2014 and 2021. Daily rainfall data was also available over this period at the Charteris climate station (ID #7031315) and was retrieved to help identify storms that were sufficiently widespread for calibration purposes – that is, storms that produced significant rainfall at both the Pembroke Climate and Charteris stations.

3.4 Design Storm Events

Intensity-duration-frequency (IDF) curves at the centroid of the Muskrat River watershed were retrieved using the Ontario Ministry of Transportation (MTO) IDF Curve Lookup Tool (MTO, 2021) and area shown in **Appendix C**. AES 30% storm distributions for Southern Ontario were applied because they are typically more accurate than SCS or Chicago distributions (OMNR, 2002). Storm duration was selected to be 12 hrs because 12-hr storms produced the highest flows. Longer duration storms (e.g., 24 hrs) were not considered since AES storms are not defined for durations greater than 12 hrs. Using this approach, design storms were created for the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year return periods to assess the hydrology, hydraulics, and potential flooding issues under a gamut of storm intensities. Regulatory floodplain mapping, however, is to be completed using only the 100-year event. The intensities and associated rainfall depths for each return period are summarized in **Table 3.1**.

Table 3.1: Rainfall Intensities and Depths for Various 12-hr Storm Return Periods

Return period	Intensity (mm/hr)	Depth (mm)
2-yr	3.5	42.5
5-yr	4.7	56.4
10-yr	5.5	65.7
25-yr	6.4	77.1
50-yr	7.1	85.8
100-yr	7.9	94.2

3.5 Model Selection and Setup

The hydrologic model selected for application to the riverine flood study was HEC-HMS (ver. 4.8). HEC-HMS was selected because it is a publicly available and is widely used for floodplain mapping studies, incorporating a variety of loss, transform, and routing methods.

The model was set up using the NAD83 UTM Zone 18N horizontal coordinate system and the CGVD2013 vertical datum. All associated GIS files used the same coordinate system and vertical datum.

3.6 Timestep

The control specification time step was selected to be 6 mins (0.1 hrs) because it is a relatively small timestep that can improve accuracy. The time interval for Muskingum-Cunge routing is dependent on reach index flow, which was selected as approximately the average between baseflow (assumed to be negligeable, i.e., 0 m^3/s) and peak flow within each reach, as per the HEC-HMS Technical Reference Manual (US Army Corps of Engineers, 2000).

3.7 Subbasins

The watershed was discretized into 33 subbasins based on a stream delineation process that required a minimum drainage area of 20 km^2, as shown in **Figure 3-5.** Originally, only 31 subbasins were delineated, but it was deemed necessary to increase the number of subbasins immediately upstream of the confluence in order to separate urban areas from rural catchments and to increase flow discretization within Pembroke. Subbasin properties and model inputs are shown in **Appendix C**.

Figure 3-5: Delineated Subbasins in the HEC-HMS Model

3.8 Reach Routing

Since LiDAR cannot accurately capture stream bathymetry, the LiDAR DEM was augmented with stream topographic surveys conducted by Aquafor during the period of November 8^{th}-19^{th}. This allowed for accurate cross-sections to be generated for reaches within Pembroke. Furthermore, in areas where LiDAR was not available, the 30 DEM could not adequately capture stream meandering; as such, reaches were manually defined to follow MNRF's mapped stream network.

A review of LiDAR elevation data revealed that certain reaches are characterised by wide floodplains, particularly along the main branch of Muskrat River. In addition, some of the reaches are characterized by low slopes (<0.0001 m/m). Therefore, the Muskingum-Cunge routing method was selected and was applied using 8-point cross-sections that were defined for each reach. This approach accounts for overbank flow and is appropriate for modelling flow in subbasins with low slope (US Army Corps of Engineers, 2000, 2021). Manning's n was selected to be 0.08 for the floodplains give that these areas are typically highly vegetated. This is in agreement with the typical Manning's n coefficients put forth in the MTO Drainage Management Manual (MTO, 1997), which suggests values of 0.06-0.08 for light brush and trees in summer and 0.01-0.16 for medium to dense vegetation in summer. Manning's n was selected to be 0.035 for the channel, based on recommended values for open channels (**Table 3.2**).

The locations of the reaches and the junctions connecting them are shown in **Figure 3-6**. A summary of reach parameters is provided in **Appendix C**.

Table 3.2: Standard Manning Roughness Coefficients for Open Channels (from CLOCA et al., 2017)

Land cover	Standard 'n' Value
Overbank	
Woods	0.080
Meadows	0.055
Lawns (or Agriculture*)	0.045
Wetlands*	0.080
Channel	
Natural	0.035
Grass	0.030
Natural Rock	0.035
Armour Stone	0.025
Concrete	0.015
Articulated Block	0.020
Gabions	0.025
Woods	0.012

*Modified by Aquafor as per the MTO Drainage Management Manual (MTO, 1997)

Figure 3-6: HEC-HMS Reaches and Junctions

3.9 Transform Method and Lag Time

The SCS unit hydrograph transform method was selected. Lag time, t_{lag} was calculated from time of concentration, t_c, using the equation:

$$t_{lag} = 0.6t_c$$

As per the recommendations in the United States Department of Agriculture (USDA) TR-55 report (USDA, 1986), time of concentration was calculated for each subbasin as the sum of travel times from sheet flow, shallow concentrated flow, and channel flow along the longest flowpath. Sheet flow length was estimated using the McCuen-Spiess equation, and travel time was estimated using the Manning's roughness values shown in **Table 3.3** that were recommended by CLOCA et al. (2017). Channel length coinciding with the longest flowpath was determined using the MNRF's stream network, and travel time was estimated by assuming using a natural channel roughness of 0.035, as per **Table 3.2**. Shallow concentrated flow length was calculated by subtracting channel length and sheet flow length, and travel time was estimated based on land use.

Table 3.3: Standard Manning Roughness Coefficients for Overland Flow (from CLOCA et al., 2017)

Land cover	Standard 'n' Value
Impervious	0.013
Lawns	0.250
Cultivated	0.300
Meadows	0.350
Woods	0.600

3.10 Lakes

The two largest lakes, Doré Lake and Muskrat Lake, were modelled as reservoirs. Flows were routed through the lakes using the Outflow Structures method, and lake outlets were modelled as non-level dam crests with a weir overflow discharge coefficient of 1.84. Elevation-storage curves and lake outlet geometries were defined for each lake using the LiDAR DEM. The simplified 8-point cross-sections representing the outlets are shown in A**ppendix C**, along with the elevation-storage curves.

The effect of the numerous other small lakes on the watershed hydrology was accounted for by increasing subbasin lag times during the calibration process, described in **Section 3.13**.

3.11 Runoff Calculation

The SCS curve number (CN) method for infiltration loss was adopted for estimating runoff. CN values are defined based on both land use and hydrological soil group. Typical CN values under average moisture conditions (AMC II) proposed by CLOCA et al. (2017) are shown in **Table 3.4**. The Ontario Land Classification layer was updated by Aquafor by overlaying roads from the Ontario Road Network, buildings from Google Earth, and waterbodies from the Ontario Hydro Network. Average road widths were determined by manually sampling roads using areal imagery. Average urban road width was determined to be 9.6 m (n=35), while average rural road width was determined to be 11.8 m (n=20). Other impervious areas, such as parking lots, were manually identified, along with gravel lots and areas of disturbance. Additional CN values under AMC II were defined for wetlands and are recorded in **Table 3.5**.

A single composite CN value was determined for each subbasin by calculating the weighted average of CN values, based on the area occupied by each CN value. Similarly, composite values for initial abstraction and impervious fraction were calculated for each subbasin. Initial abstraction values for each land use type are shown in **Table 3.6**. Impervious areas (roads, houses, paved lots, etc.) located within Pembroke were considered to be directly connected to the stream via storm drains and were not included in the calculation of composite CN values. Outside of Pembroke, impervious areas were included in the calculation of composite CN values but were not considered to be directly connected. Waterbodies were considered to be impervious throughout the entire watershed.

Table 3.4: Uncalibrated Standard SCS Curve Numbers for each Land Use Type and Soil Hydrological Group under AMC II (from CLOCA et al., 2017)

Land use	A	B	C	D
Woods	32	60	73	79
Meadows	38	65	76	81
Cultivated	62	74	82	86
Lawns	49	69	79	84

Table 3.5: Other Uncalibrated SCS Curve Numbers for each Land Use Type and Soil Hydrological Group under AMC II

Land use	A	B	C	D
Dirt Lot*	72	82	87	89
Gravel Lot*	76	85	89	91
Bedrock	80	80	80	80
Bog	10	10	10	10
Fen	10	10	10	10
Marsh	50	50	50	50
Swamp	32	60	73	79

*Values obtained from the TR-55 report (USDA, 1986)

Table 3.6: Initial Abstraction Values for Each Land Use (from CLOCA et al., 2017)

Land use	Initial Abstraction (mm)
Woods	10
Meadows	8
Cultivated	7
Lawns	5
Impervious Areas	2
Bog	15
Fen	15
Marsh	15
Swamp	15
Gravel*	4
Dirt*	4
Lakes*	0
Bedrock*	5

*Values defined by Aquafor

3.12 Flood Frequency Analyses

Single-station flood frequency analyses were completed for the Indian River and Muskrat River flow gauges. In order to account for the effect of snowmelt on peak flows, separate analyses were undertaken for summer (June-November) and winter (December-May) periods. For each seasonal period and flow gauge, peak flow data was ranked from highest to lowest flow, then the estimated return period, T_{est}, was calculated as:

$$T_{est} = \frac{n + 0.12}{m - 0.44}$$

where n is the total number of years of record and m is rank. Theoretical return period under the Gumbel distribution was then calculated. Peak flow was plotted against both estimated and theoretical return periods using a log scale on the x-axis (**Figures 3-7 to 3-10**). A logarithmic curve of best fit was fitted to the "straight" portion of the theoretical data, in order to estimated the 100-year peak flow. Using the equation of the curve of best fit, the expected 100-year summer peak flow at the Indian River station was calculated to be:

$$Q_{100,summer} = 9.6131 \times \ln(100) + 6.3873 = 50.7 \ \mathrm{m^3/s}$$

The 100-year winter peak flow at the Indian River station was calculated to be:

$$Q_{100,winter} = 23.533 \times \ln(100) + 52.225 = 160.6 \ \mathrm{m^3/s}$$

Using the same approach for Muskrat River, the summer and winter peak flows were estimated to be 37.7 m^3/s and 88.3 m^3/s, respectively.

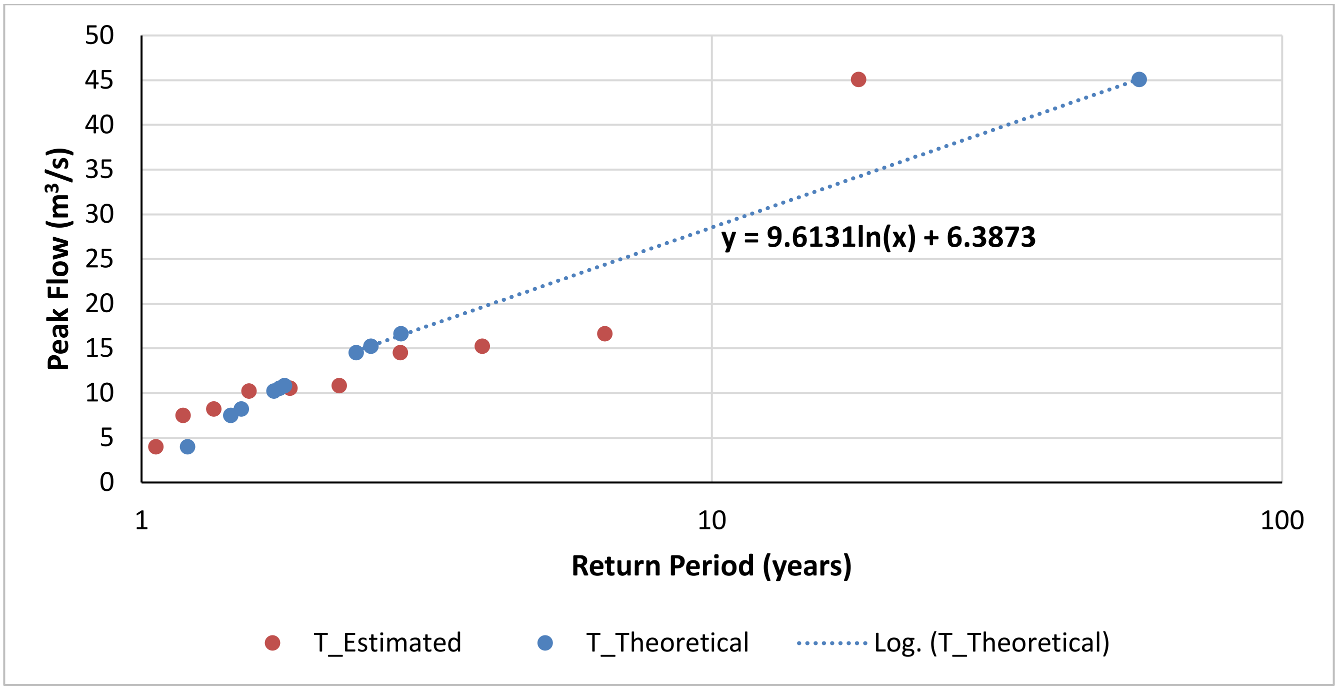


Figure 3-7: Indian River Summer Peak Flows as a Function of Estimated and Theoretical Return Periods

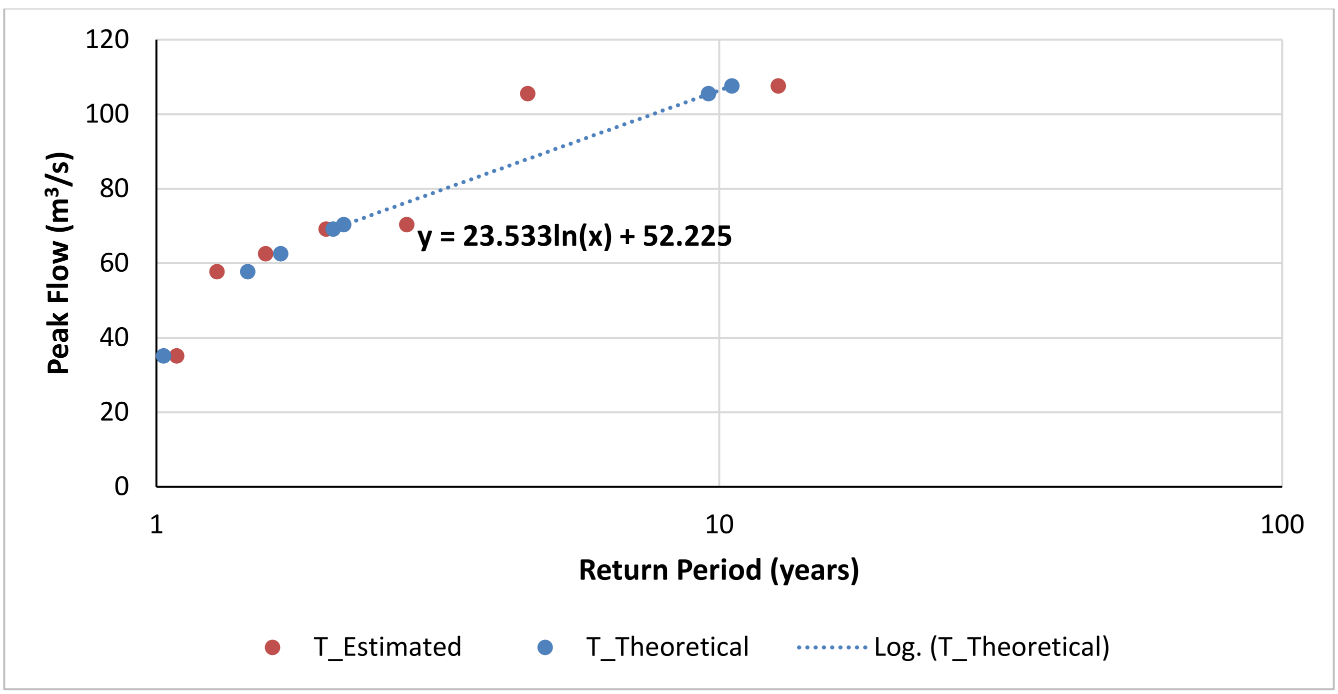


Figure 3-8: Indian River Winter Peak Flows as a Function of Estimated and Theoretical Return Periods

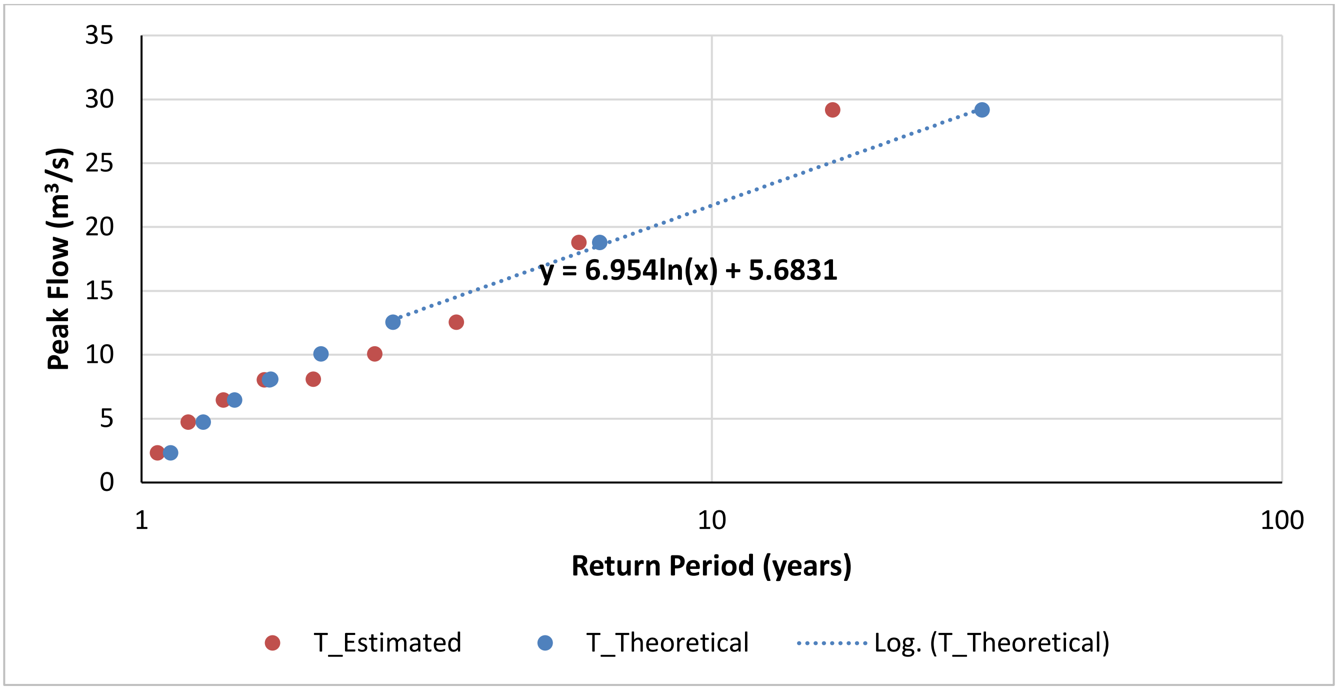


Figure 3-9: Muskrat River Summer Peak Flows as a Function of Estimated and Theoretical Return Periods

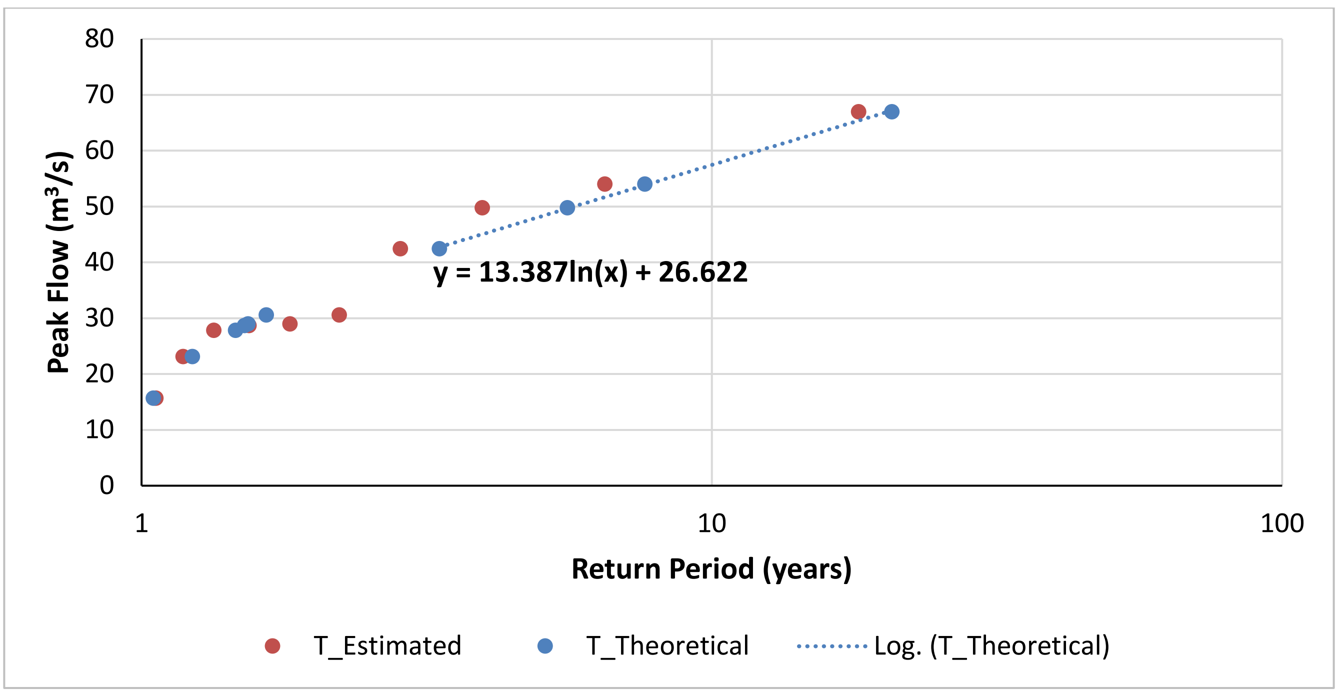


Figure 3-10: Muskrat River Winter Peak Flows as a Function of Estimated and Theoretical Return Periods

3.13 Calibration

The HEC-HMS model was calibrated using 3 storm events (summarize in **Table 3.7)** that produced large flow responses in the Indian River. Only summer events were selected for calibration, in order to remove confounding factors such as ice jams, frozen ground, and snowmelt – all of which lack data to be used in the hydrologic model. Graphs comparing modelled and measured flows within Indian River and Muskrat River are shown in **Appendix C**.

Based on the MTO Drainage Management Manual (MTO, 1997), antecedent moisture conditions should be considered as dry (AMC I) for all 3 events since precipitation was less than 35.5 mm over the 5 days preceding each event. However, during the calibration process, it was found that the model produced the best results when moderately dry conditions were applied – that is, when CN values halfway between AMC I (dry) and AMC II (average) were applied. Variations in CN based on antecedent moisture content are specified in the Technical Guidelines for Flood Hazard Mapping report (CLOCA et al., 2017) and are listed in **Table C.4** (**Appendix C**).

CN values were also changed for individual land uses: CN was increased for forested areas and was decreased for agricultural land use. A summary of the calibrated CN values for AMC I, AMC II, AMC III, and moderately dry (between AMC I and II) scenarios are shown in **Appendix C**.

Lag time was increased for most subbasins to account for the effect of lakes and wetlands on delaying the travel of water. In addition, the routing lengths of Reaches 9-1 and 12 were increased because they passed through flat, ponded areas that greatly increase the travel time of water. Lastly, slope was decreased for reaches containing long, flat stretches, which can disproportionately affect flow routing and reduce peak flows. Final subbasin and reach parameters are listed in **Appendix C**.

Table 3.7: Summary of Storm Events used for Calibration of the HEC-HMS Model

Event ID	Rainfall Depth (mm)	Start Date/Time	End Date/Time	5-Day Prior Rainfall (mm)	Indian River Peak Flow (m^3/s)	Peak Flow Return Period
Cal-1	37.3	2014-06-17 16:00	2014-06-17 20:00	1.8	15.3	2.5-year
Cal-2	38.3	2018-06-03 16:00	2018-06-04 19:00	9.1	14.6	2.5-year
Cal-3	32.4	2019-06-13 12:00	2019-06-15 16:00	18.6	10.6	1.5-year

3.14 Model Scenarios and Results

Although regulatory floodplain mapping is normally performed using summer events, it was clear from the flood frequency analyses (**Section 3.12**) that peak flows were substantially higher in the winter months as a result of frozen soil and soil saturation from snowmelt, which increase precipitation runoff. For this reason, Aquafor created separate hydrologic model scenarios for the summer (June-November) and winter (December-April) periods. For the summer events, CN values corresponding to moderately dry antecedent moisture conditions (average of AMC I and AMC II) were used; for winter events, CN values for wet antecedent conditions (AMC III) were used.

Typical baseflows of 5.0 m^3/s were applied to Indian River and Muskrat River when modelling summer events; for winter events, typical baseflows of 2.1 m^3/s and 2.5 m^3/s were applied to the Indian River and Muskrat River, respectively.

The calibrated hydrologic model was run for the summer and winter scenarios using the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year (Regulatory) design storms. Peak flows at key locations in Pembroke are presented in **Table 3.8** below, and results at all junctions are shown in **Tables C.9 and C.10** (**Appendix C**).

Table 3.8: Modelled Peak Flows at Key Locations in Pembroke under the 2-yr, 5-yr, 10-yr, 25-yr, 50-yr, and 100-yr Summer and Winter Events

Junction ID	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Summer Peak Flows (m^3/s)						
Jun-17 (Indian River, entering Pembroke)	12.4	25.8	38	57.4	72.3	87.5
Jun-18 (Muskrat River, entering Pembroke)	13.4	20.3	25.8	33.5	39.9	46.5
Jun-2 (at Confluence)	30.3	51.4	69.2	95.4	116.9	138.1
Jun-2 (from Indian River only)	12.5	26.5	38.9	57.8	73	87.9
Jun-2 (from Muskrat River only)	13.2	20.1	25.6	33.2	39.6	46.2
Jun-1 (at Ottawa River)	24.9	45.8	63.6	89.7	111.2	132.3
Winter Peak Flows (m^3/s)						
Jun-17 (Indian River, entering Pembroke)	34.5	69.6	94.3	127.4	153.6	179.8
Jun-18 (Muskrat River, entering Pembroke)	22.6	36.6	47.1	60.9	71.9	83
Jun-2 (at Confluence)	59.6	107.8	141.7	186.8	223.4	259.9
Jun-2 (from Indian River only)	35.3	70	93.9	126	151.7	177.4
Jun-2 (from Muskrat River only)	22.7	36.8	47.3	61	72.1	83.2
Jun-1 (at Ottawa River)	57.2	105.4	139.1	183.7	219.9	256.1

4 RIVERINE HYDRAULIC MODEL AND FLOODPLAIN MAPPING EXTENSION

Hydraulic modelling was completed to determine the floodplain extents of the Indian River and Muskrat River throughout the study area, using peak flow results from the hydrologic model. The following section outlines the field survey and structure inventory, hydraulic model setup, model results, and recommendations for mitigating riverine flooding.

4.1 Field Survey and Structure Inventory

Bridge crossings along the Indian River and Muskrat River within the study area were identified from a preliminary review of aerial imagery. A total of 8 bridge structures were identified.

A field inventory and topographic survey using GPS equipment were conducted by Aquafor during the period of November 8th-19th for the 8 crossings. The information collected during the structure surveys was incorporated into the hydraulic model for the study, as described in **Section 4.2.2**. The topographic surveys included capturing stream bathymetry at the bridges, as well as at representative locations between the bridges (shown in **Figure 4-1**), in order to augment the LiDAR data and increase the accuracy of the hydraulic model. Structure inventories documented the following information:

- Structure type
- Material
- Opening shape and dimensions
- Pier dimensions (if applicable)
- Parapet dimensions (if applicable)
- Bridge skew (if applicable)
- Photographs of the upstream and downstream faces of the structure

A sample structure inventory sheet created for the Boundary Road crossing over Indian River is shown in **Figure 4-2**. The full results of the structure inventory are presented in **Appendix D**.

Figure 4-1: Hydraulic Model Reaches, Structures, and Surveyed Intermediary Cross-Sections

Hydraulic Structure Data Sheet

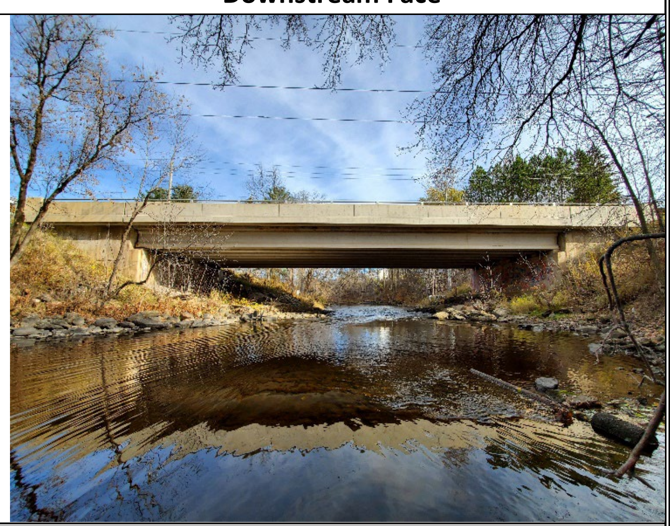
General Information	
Date (Month DD, YYYY)	**November 9, 2021**
Field Crew Initial	**D.M., G.D.**
Municipality	**City of Pembroke**
Watershed Name	**Muskrat**
Creek or Tributary Name	**Indian River**
UTM Co-ordinates	**N: 5075651.4598 m, E: 334600.0107 m**
Street Name	**Boundary Road**
Structure Information	
Structure Type	**Bridge**
Number of Cells	**1**
Each Cell Shape	**Flat**
Each Cell Material	**Concrete**
Each Cell Dimension in Meters (Height x Width or Diameter)	**4.88 m x 26.04 m**
Open Footing (Y/N)	**Y**
Structure Length in Meters (inlet to outlet)	**11.55 m**
Additional Information where applicable	
Pier Width in Meters	
Parapet Present (Y/N)	**Y**
Parapet Length in Meters	**38.91 m**
Parapet Height in Meters	**0.83 m**
Skew Angle in Degrees	
Photographs	
Upstream Face	**Downstream Face**
	
Additional Comments	

Figure 4-2: Sample Hydraulic Structure Sheet (Boundary Road Bridge Crossing)

4.2 Hydraulic Model Setup

GeoHECRAS was utilized to create a georeferenced HEC-RAS (ver. 5.0.7) hydraulic model of the Indian River and Muskrat River within the study area. As with the hydrologic model, the hydraulic model was set up using the NAD83 UTM Zone 18N horizontal coordinate system and the CGVD2013 vertical datum.

The following section outlines the information input into the hydraulic model.

4.2.1 Cross-sections

A base model was created in GeoHECRAS using topographical information from the LiDAR DEM provided by Natural Resources Canada, along with the MNRF stream network. This spatial data was used to define channel cross-sections, stream centrelines, and overbank locations for the watercourse reaches. Cross-sections were spaced to account for changes in channel geometry, meanders, and bridge structures.

Manning's roughness coefficients for cross-section overbanks were applied based on the updated land use layer that was used in the hydrologic model. The Manning's "n" values used for each land use type are shown in **Table 4.1** and were derived from the MTO Drainage Management Manual (MTO, 1997). Stream bed roughness was set to 0.035, as per the Technical Guidelines for Flood Hazard Mapping (CLOCA et al., 2017).

Table 4.1: Hydraulic Model Roughness Values for Cross-Section Overbanks

Land cover	Manning's n
Woods	0.09
Agriculture	0.04
Impervious Areas	0.015
Open Space (Lawns)	0.05

Following the initial model setup, stream bathymetry was corrected using the topographic survey data collected by Aquafor staff. The base model was further refined through the addition of obstructions within the cross-sections to represent buildings that may be within the flow path.

4.2.2 Bridge Structures

Bridge geometry was defined using the LiDAR DEM and topographic survey data, along with other measurements and notes (such as bridge length and material) taken during the structure inventory. Ineffective flow areas were applied using a 1:1 contraction and expansion ratio to account for flow restriction near bridge openings. An expansion coefficient of 0.3 and a contraction coefficient of 0.5 were applied to the boundary cross-sections located immediately upstream and downstream of each bridge, as per the values recommended by Chow (1959; see **Table 4.2**). For all other cross-sections, the contraction and expansion ratios used in the model were 0.1 and 0.3, respectively.

Table 4.2: Contraction and Expansion Coefficients (from Chow, 1959)

Flow Type	Contraction	Expansion
No transition loss computed	0.0	0.0
Gradual transitions	0.1	0.3
Typical bridge cross-sections	0.3	0.5
Abrupt transitions and culverts	0.6	0.8

4.2.3 Flood Flow Rates and Boundary Conditions

The resultant 100-year design flows from the hydrological model (**Section 3.14**) were input into the hydraulic model to determine floodplain extents. The downstream starting water surface elevation (WSE) at the edge of the Ottawa River was selected to be 111.082 m, as measured in the field by Aquafor staff on November 9th, 2021. The sensitivity of the lower reaches of Muskrat River to fluctuations in Ottawa River WSE were evaluated in **Section 4.4**.

4.3 Model Results and Floodplain Mapping

The GeoHECRAS model was run for using the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year (Regulatory) design storms for both the summer and winter scenarios. Full model results for all cross-sections and design storms are recorded in **Appendix E**. Using the water surface elevations generated for the 100-year storms, floodplain maps were created for the summer and winter events and are plotted in **Figure 4-3**.

The hydraulic model for both the summer and winter scenarios predicted that the Indian River would flood houses along Doran Street McGee Street, and Murray Street, as well as portions of the roads themselves. This is consistent with substantial flooding in this area that was noted following the August 2003 rainfall events (**Figure 2-1** and **Appendix A**). All affected properties along the Indian River are located near this area, with the exception of a shed located at 499 Boundary Road East that is flooded under the winter scenario but not under the summer scenario.

Muskrat River did not cause flooding of any structures except for a small portion of the City Hall building. Water surface elevation in this this area was 112.57 m, meaning that ponding depth next to City Hall would be approximately 0.88 m.

Figure 4-3: Floodlines for the 100-year Summer and Winter Storm Events

4.4 Sensitivity Analysis

A sensitivity analysis was performed to evaluate the effect of the Ottawa River water surface elevation (WSE) – that is, the downstream boundary condition – on floodlines along Muskrat River and Indian River. As previously mentioned, the WSE measured by Aquafor was 111.082 m (CGVD2013 datum). This is was taken as the lower bound in the sensitivity analysis, while the 100-year Ottawa River high-water elevation was taken as the upper bound. The 100-year Ottawa River WSE was specified as 113.9 m in the City of Pembroke Official Plan (City of Pembroke, 2016), but this elevation is referenced to the CGVD28 datum, rather than the newer CGVD2013 datum that was used in the hydraulic model. The 100-year WSE was therefore converted to the CGVD2013 datum and was determined to be 113.621 m. In total, three WSE's were tested in the sensitivity analysis: 111.082 m (Summer Scenario 1), which was measured by Aquafor staff; 112.0 m (Summer Scenario 2); and 113.621 m (Summer Scenario 3), i.e., the WSE for the Ottawa River 100-year flow event.

Floodlines for the three Summer Scenarios were nearly identical throughout most of the Indian and Muskrat Rivers, but differed slightly at the downstream extents, as shown in **Figure 4-4** and **Figure 4-5**. Flood waters encroached slightly farther into City Hall property under Summer Scenario 3, with the WSE at this location increasing from 112.57 m (Scenarios 1 and 2) to 113.84 m (Scenario 3), such that ponding depth next to City Hall was roughly 2.15 m under Scenario 3. Therefore, the Ottawa River is responsible for most of the flooding at the downstream extents of Muskrat River when the Ottawa River WSE is high (113.621 m), whereas flows from Muskrat River have a more pronounced effect when the Ottawa River WSE is lower (e.g., 111.082 m and 112.0 m).

Furthermore, the high Ottawa River WSE under Summer Scenario 3 caused flooding of the Pembroke Farmer's Market grounds, whereas the lower Ottawa River WSE's did not. Flooding predicted using the 100-year Ottawa River WSE (Scenario 3) is consistent with floodlines previously generated along the Ottawa River, which showed flooding of the Farmer's Market and of the City Hall AC unit (**Appendix B**).

Figure 4-4: 100-Year Summer Floodlines for Various Downstream Water Surface Elevation Boundary Conditions (Model Extents)

Figure 4-5: 100-Year Summer Floodlines for Various Downstream Water Surface Elevation Boundary Conditions (Downstream Extents)

4.5 Mitigation

Flood remediation alternatives were developed for the identified flood hazard areas. Overviews of the considered alternatives are presented below.

4.5.1 Flood Remediation Alternatives

Structural Flooding
Landowners can apply structural treatments to their buildings to reduce or eliminate the possibility of flood damage to the structure and its contents. Residential buildings should be dry-floodproofed to prevent flooding up to and including the 100 Year event, and prevent structural movement as a result of flood flows. Buildings can be floodproofed by sealing or filling low openings which are flood susceptible. Flood proofing is a viable alternative in areas outside of municipal authority and where buildings cannot be feasibly removed from the 100 Year Floodplain.

Dykes/Berms
Dykes or berms can be constructed adjacent to private property to prevent flood waters from reaching dwellings. Associated risks include increased channel velocities during flood conditions, and downstream erosion. This alternative was considered in several locations where flood risk was attributable to proximity of buildings to the floodplain, but was determined to be infeasible in most locations due to limited space between at-risk buildings and the top of the channel bank

Watercourse Capacity Re-grading/Enlargement
This alternative includes measures that are designed and implemented based on engineering-based flood mitigation measures that would increase the conveyance capacity of watercourses, thus reducing the flow of water beyond the channel banks during storm events. The measures are generally applied on a stream reach basis and include stream rehabilitation using natural or engineered channel design principles and naturalization of stream riparian zones using native materials. They may also include individual approaches such as streambank re-grading, gradient controls and floodplain contouring to address specific flooding problems. Riparian plantings and open space re-vegetation can be implemented concurrently to improve the function of stream corridors. In addition to reducing overbank flood flows, these approaches improve water quality, slow runoff, moderate stream temperatures, reduce erosion and improve aquatic and terrestrial habitat conditions.

4.6 Recommended Flood Remediation Alternatives

4.6.1 Flooding from the Indian River and Muskrat River

As previously noted, riverine flooding primarily affects buildings located along McGee Street and Doran Street, adjacent to the Indian River, as well as City Hall. Provided below is a conceptual approach for protecting the properties in the Doran and McGee Street area. It should be noted that the approach shown is, as noted, conceptual and various steps will be required to confirm feasibility. This would include discussions with land owners, acquisition of easements as required, confirmation of impacts on the sewer system (backflow valves may be required to ensure that flows from the Indian River do not spill into the dry side of the berm). These steps would be followed by preliminary and detailed design.

Aquafor recommends constructing a continuous 1 m high berm between the river and affected properties to prevent water from passing into the floodplain and/or applying structural floodproofing to structures located within the floodplain. **Figure 4-6** shows the extent of the recommended berm, which is roughly 310 m in length. It has been assumed that a 1 m high berm with 3:1 side slopes and 1 m plateau would be implemented. If the berm option were to be adopted, it may be necessary for the City to procure parts of certain properties, as existing buildings are located within (or very close to) the footprint of the proposed berm. It should be noted that the property at 375 Doran Street is already enclosed by a berm that may help prevent flooding.

City Hall is also affected by flooding, with up to 2.15 m of ponding occurring during the 100-year summer event when the Ottawa River WSE is 113.621 m. Aquafor recommends flood-proofing the building and/or re-grading the property to protect against flooding.

No mitigation alternatives are proposed for the shed affected by the 100-year winter event, as it is neither a primary dwelling unit nor a commercial unit. In addition, no mitigation measures were proposed for the Pembroke Farmer's Market building, because it is an open-air structure.

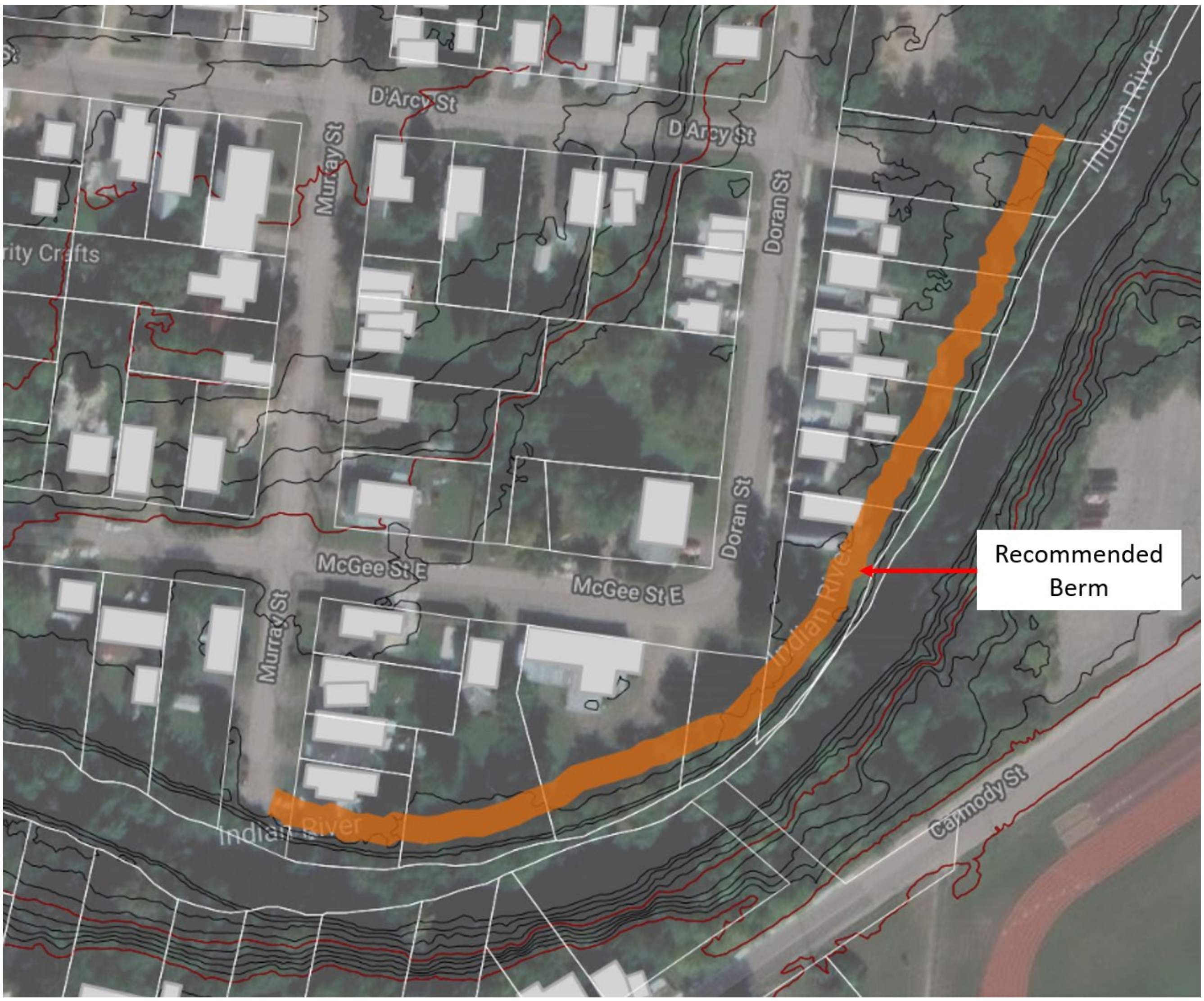


Figure 4-6: Extent of the Recommended Berm

4.6.2 Flooding from the Ottawa River

The Ottawa River is also responsible for flooding during the 100-year flow event (**Figure 2-1** and **Appendix B**). Historically, flooding issues caused by high waters within the Ottawa River has been addressed by sandbagging buildings and sensitive equipment, sealing maintenance holes and fuel tanks, shutting down pump stations, and blocking roads with berms.

As previously discussed, City Hall is impacted by flows within both the Muskrat River and the Ottawa River. Other municipal properties impacted by flooding of the Ottawa River include:

- Riverside Park, including the pump station
- Coronation Park
- Waterfront Park
- The Supples Landing pump station

Aquafor recommends flood-proofing municipal buildings and pump stations located within these flood areas, as well as deploying sandbags or inflatable bags during high flow events.

Mitigation of flooding on private property caused by high waters within the Ottawa River is beyond of the scope of this study. Nonetheless, Aquafor recommends that the City work in partnership with private property owners to prevent the release of hydrocarbons from gas stations, fuel tanks, and automotive shops located within the Ottawa River 100-year floodlines.

5 STORM SEWER INFRASTRUCTURE MODEL

5.1 Stormwater Infrastructure and Flooding

Hydrologic and hydraulic modelling of the sewer network was undertaken. The PCSWMM software was selected as the most appropriate tool for modeling of the City of Pembroke sewer systems.

The objective of the modeling was to assess the capacity of the existing systems, identify potential flooding problems and to provide insight for defining suitable alternatives. The following sections explain the steps that were undertaken in order to set up the model.

5.2 Modelling Area and Database Assessment

Discussions were held with the city at the onset of the study to define areas where historical flooding had occurred. **Figure 5-1** illustrates the boundary of the four Pembroke models, and includes the storm sewer network which was incorporated into the models. For the purposes of this model, only the storm sewers with diameters that are 300 mm or greater were considered (total length of 9,640 m).

Data that was obtained from the City's geodatabase included drawings and reports. As was noted in the Request for Proposal, a considerable amount of information related to the storm sewer system is missing. A data gap assessment was therefore performed to identify inconsistencies / gaps in the data. Gaps included:

- Missing pipe/manholes invert elevations;
- Number and locations of some catchbasins;
- Negative pipe or zero pipe gradients; and
- Inconsistencies between maintenance hole cover elevations and the DEM.

A summary of storm sewer data assessment in the study area and related gaps is presented in **Table 5.1** to **Table 5.4**.

5.2.1 Survey Investigations

In order to obtain the missing information a survey of the sewer infrastructure was conducted by Aquafor during the periods of November 15th-19th, 2021 and February 7th-11th, 2022. This included using GPS instrumentation to collect missing elevations for manhole rims, pipe inverts, and pipe obverts. In addition, photographs and notes were collected for catchbasins to determine lid inlet capacity limits. The number of the surveyed sewer pipes and manholes are presented in **Table 5.1** to **Table 5.4.**

5.3 Network Data

The Pembroke study areas dual drainage water collection system model consists of two components, including the major and minor systems. The major system represents overland flow paths such as roadways and ditches, while the minor system is predominately defined as the underground pipe network. The major system is connected to the minor system through catchbasins, which are defined as an "Outlet" in the PCSWMM model.
Subcatchment delineation was completed on a maintenance hole to maintenance hole basis utilizing the property parcels (**Figure 5-2**).

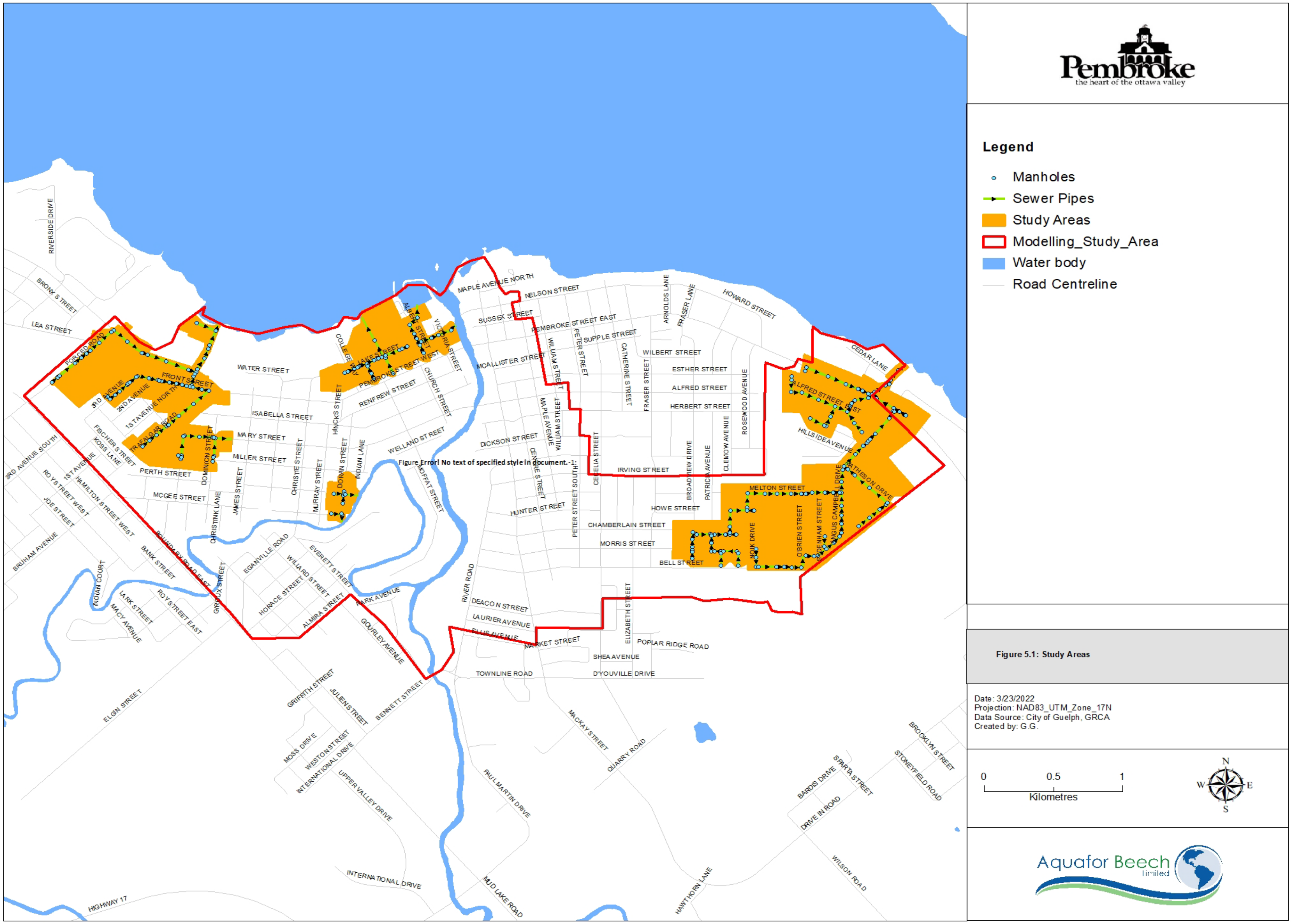


Figure 5-1: Pembroke study areas and storm sewer network schematic

Table 5.1: Summary of the storm sewer network and gaps for Amalgamated area

Infrastructure type		Sewer Pipes	Manholes
In Model after screening*		**117**	**117**
Missing Information	Upstream elevation	73	-
	Downstream elevation	71	-
	Diameter	20	
	No invert elevation	-	115
	No rim elevation	-	2
		80	50

Table 5.2: Summary of the storm sewer network and gaps for Trafalgar area

Infrastructure type		Sewer Pipes	Manholes
In Model after screening*		**68**	**68**
Missing Information	Upstream elevation	36	-
	Downstream elevation	37	-
	Diameter	0	-
	No invert elevation	-	40
	No rim elevation	-	8
		26	30

Table 5.3: Summary of the storm sewer network and gaps for Darcy area

Infrastructure type		Sewer Pipes	Manholes
In Model after screening*		**10**	**10**
Missing Information	Upstream elevation	7	-
	Downstream elevation	4	-
	Diameter	0	-
	No invert elevation	-	8
	No rim elevation	-	1
		2	4

Table 5.4: Summary of the storm sewer network and gaps for Lake Street area

Infrastructure type		Sewer Pipes	Manholes
In Model after screening*		**35**	**35**
Missing Information	Upstream elevation	23	-
	Downstream elevation	21	-
	Diameter	0	-
	No invert elevation	-	35
	No rim elevation	-	1
		15	14

**the values in this row present the number of features with a diameter ≥300mm*

***Some of the manholes are catch basin manholes*

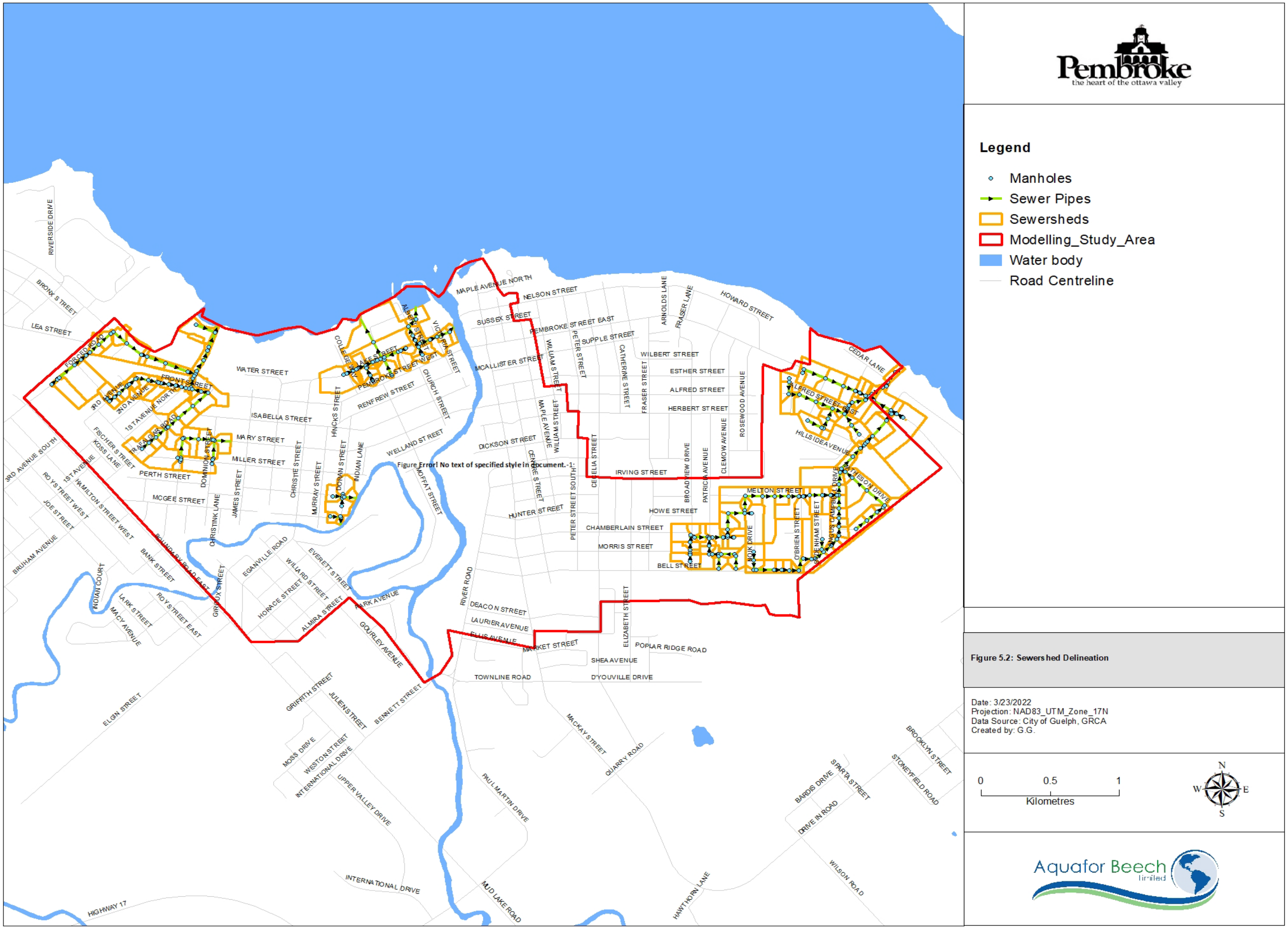


Figure 5-2: Pembroke PCSWMM model sewershed delineation

5.4 Minor System

Proper network development of the model was critical to ensure that each sewer system element was representative of the current physical collection system.

Aquafor therefore developed a new hydrologic model using the PCSWMM model. For this purpose, GIS information related to the storm sewer network which was provided by the City of Pembroke was utilized, along with data gathered from the survey.

As noted, the sewer network was initially setup based on the City's GIS database and survey investigations by Aquafor. The system contains the sewer network and maintenance hole as-built information including pipe diameters, invert elevations, pipe lengths, and maintenance hole ground elevations.

To confirm the accuracy of the data once imported, extensive quality checks were completed, and data gaps were filled in through review of as-built information, field investigations, and use of best professional judgement to enhance the model accuracy. Updated and revised data were flagged and documented in the model for future reference.

All maintenance hole cover elevations were updated using the city-provided DEM. Any missing invert and ground elevations were filled in using the inference tool in PCSWMM.

5.5 Major System

5.5.1 Overland Flow Paths

In the developed model, overland flow and surface area characteristics were considered for every subcatchment. The overland runoff system was then added as an additional link between nodes as represented by the street cross sections. The overland flow system typically consisted of streets with flows constrained by the curb along both sides of the street.

The streets were modelled as wide shallow open channels to reflect the appropriate geometry, cross section and channel roughness. The overland channel invert levels were set at the manhole cover levels such that flows in the overland channels can occur when there is flooding out of the manholes from the minor drainage system or when the flow is restricted into the minor system at the catchbasin based on the catchbasin inlet capture capacity.

The typical roadway channels were defined to represent local and collector roads consisted of user defined cross sections. One typical cross section was used in the study area including a road right-of-way (ROW) width of 20.1 metres with a height of 0.30 metres for local roads. Adjustments were made to the network as necessary, such as additional nodes, overland segments, invert adjustments, etc., to replicate the overland flow paths predominately associated with roadways.

5.5.2 Catchbasins (Curb Inlets)

The catchbasin locations were provided by the City of Pembroke in ArcGIS shapefile format. The catchbasin type within the study area were verified by orthophoto. The catchbasins that were not in the City records were located and digitized to upgrade the missing data.

The major system is connected to the minor system through inlets, or catchbasins. The number of catchbasins was adjusted in the database and the type of catchbasin cover was considered using the information obtained from the survey.

The maximum inflow contribution of each catchbasin was adopted from various road drainage studies (i.e., Inlet Control Devices for Stormwater Catchbasins, Road and Bridge Deck Drainage Systems, etc.). The applicable inlet capture rate was then assigned to the downstream manholes based on the type and number of catchbasins within each of the storm sub-catchment areas.

With the completion of the major system network, tests were undertaken to ensure network continuity between the overland network (major) and pipe network (minor) behaved as expected. The end result was a dual drainage model of the storm drainage network.

Volume that is not captured by the inlet of the catch basin is either stored along the road surface until the inlet rate drops below the maximum allowable capacity of the catch basin or is bypassed to the next downstream inlet.

5.5.3 Model Parameters

Sewershed delineations were based on the sewer segment, closest land parcel and were assigned to the upstream node of the sewer segment. Subcatchments were parametrized based on similar land use and soil classifications in the original model. The Dynamic Wave approach in PCSWMM was selected. The PCSWMM model parameters including subcatchments and storm sewer network parameters are presented in **Appendix F**.

5.5.3.1 Infiltration Parameters

Subcatchment infiltration is the process of rainfall infiltration into the pervious area of the ground surface into the unsaturated soil zone of pervious subcatchment areas. The method selected for the Pembroke models was Horton’s Equation. Horton’s Equation input parameters include: the maximum and minimum infiltration rates, a decay coefficient that describes how fast the rate decreases over time, and the time it takes a fully saturated soil to completely dry (used to compute the recovery of infiltration rate during dry periods). Soil characteristics are presented in **Table 5.5:**.

Table 5.5: Soil Characteristics

Texture	Max Infiltration (mm/hr)	Min Infiltration (mm/hr)
Clay	25.4	0.25
Sand	127	120
Silt, sand, gravel	120	55
Clay, silt	50	0.51
Sand, gravel	150	5
Organic deposits	90	3.5
Clay, silt, sand	75	1.6

5.5.3.2 Imperviousness

The percent of imperviousness land in each subcatchment was calculated using the land use layer using the general imperviousness values which are presented in **Table 5.6**. Once the subcatchments were discretized, the percent of impervious land in each subcatchment was calculated using aerial photographs.

Table 5.6: Typical Imperviousness for a Defined Land Use

Land use	Imperviousness (%)
Agriculture	0.05
Open Space/Environmental	0.05
Employment	80
Low Density Residential	50
Medium Density Residential	60
High Density Residential	65
Residential	55 (51-60)
Medium and High Density Residential	70 (65-75)
Commercial/Industrial	75-90

Source: Design Specifications & Requirements Manual (2003; Updated 2019)

5.5.4 Design Requirements

The capacity of the minor storm sewer systems shall typically be designed to carry the peak flow resulting from a 5- year rainfall event, while the major system shall convey the runoff from infrequent storm events, typically greater than 5-year design storm and up to a 100-year design storm, that exceeds the minor system capacity.

The major system design shall be based on a one in 100-year rainfall event and should include assessment of road sags and boulevard overflows into stormwater management ponds and watercourses. The maximum ponding depth shall not exceed 300 mm as measure to the curbline.

5.5.4.1 Design Storm Events

Intensity-duration-frequency (IDF) curves at the centroid of the Muskrat River watershed were retrieved using the Ontario Ministry of Transportation (MTO) IDF Curve Lookup Tool (MTO, 2021) and are shown in **Appendix C**. Storm duration was selected to be 2-hrs for the purpose of pipe capacity assessment in this study. The 5-year event was used to assess the minor sewer systems and the 100-year rainfall event was used for major system. The intensities and associated rainfall depths for each return period are summarized in **Table 5.7**. Based on the availability of data, one historical event on July 2017 was also selected for further modeling analysis.

Table 5.7: Rainfall Intensities and Depths for Various 2-hr Storm Return Periods

Return period	Intensity (mm/hr)	Depth (mm)
2-yr	12.4	24.8
5-yr	16.4	32.9
10-yr	19.2	38.3
25-yr	22.5	45
50-yr	25	50
100-yr	27.5	54.9

Figure 5-3 to **Figure 5-5** show the model simulation results of the 5-year and 100-year events as well as 2017 event. Each figure indicates the state of surcharging at each maintenance hole and sewer pipe through the following colour coding:

Minor System (5-year event):

- Green Manhole: Not surcharged
- Red Manhole: Surcharged
- Green Pipe: Below Capacity
- Red Pipe: At Capacity or higher

Major System (100-year event):

- Red Street: Surface Flooded

The results showed that under 5-year event nearly 60% of the minor system is in the state of surcharge. Under 100-year event, about 70% of the storm system is surcharged beyond the conveyance capacity. The results are summarized in **Table 5.8** to **Table 5.11**.

Table 5.8: Length of Storm System Surcharged/Surface Flooded in Amalgamated area

Conditions	Length of System Surcharged (m)	
	Minor System (5-Year Design Storm)	Major System (100-Year Design Storm)
Surcharged	2798.682	NA
Not Surcharged	2428.859	NA
Surface Flooding	NA	3745.098
Total	5227.541	4597.865

Table 5.9: Length of Storm System Surcharged/Surface Flooded in Daarcy area

Conditions	Length of System Surcharged (m)	
	Minor System (5-Year Design Storm)	Major System (100-Year Design Storm)
Surcharged	239.88	NA
Not Surcharged	52.145	NA
Surface Flooding	NA	171.067
Total	292.025	184.05

Table 5.10: Length of Storm System Surcharged/Surface Flooded in Lake Street area

Conditions	Length of System Surcharged (m)	
	Minor System (5-Year Design Storm)	Major System (100-Year Design Storm)
Surcharged	614.432	NA
Not Surcharged	634.745	NA
Surface Flooding	NA	542.955
Total	1249.177	965.877

Table 5.11: Length of Storm System Surcharged/Surface Flooded in Trafalgar area

Conditions	Length of System Surcharged (m)	
	Minor System (5-Year Design Storm)	Major System (100-Year Design Storm)
Surcharged	2147.686	NA
Not Surcharged	723.596	NA
Surface Flooding	NA	1748.286
Total	2871.282	2438.933

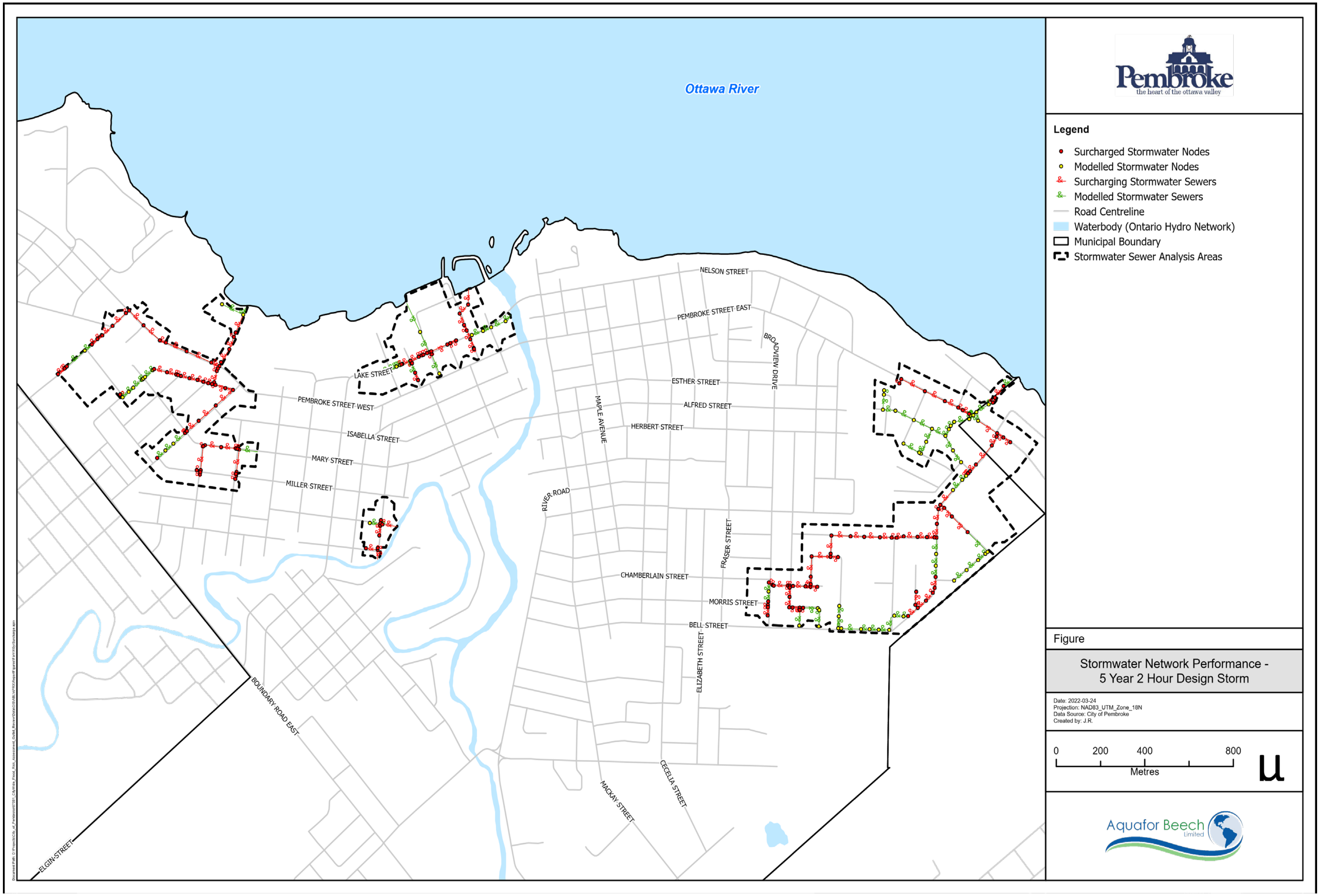


Figure 5-3: Minor System Performance under 2-hour, 5-year Design Storm

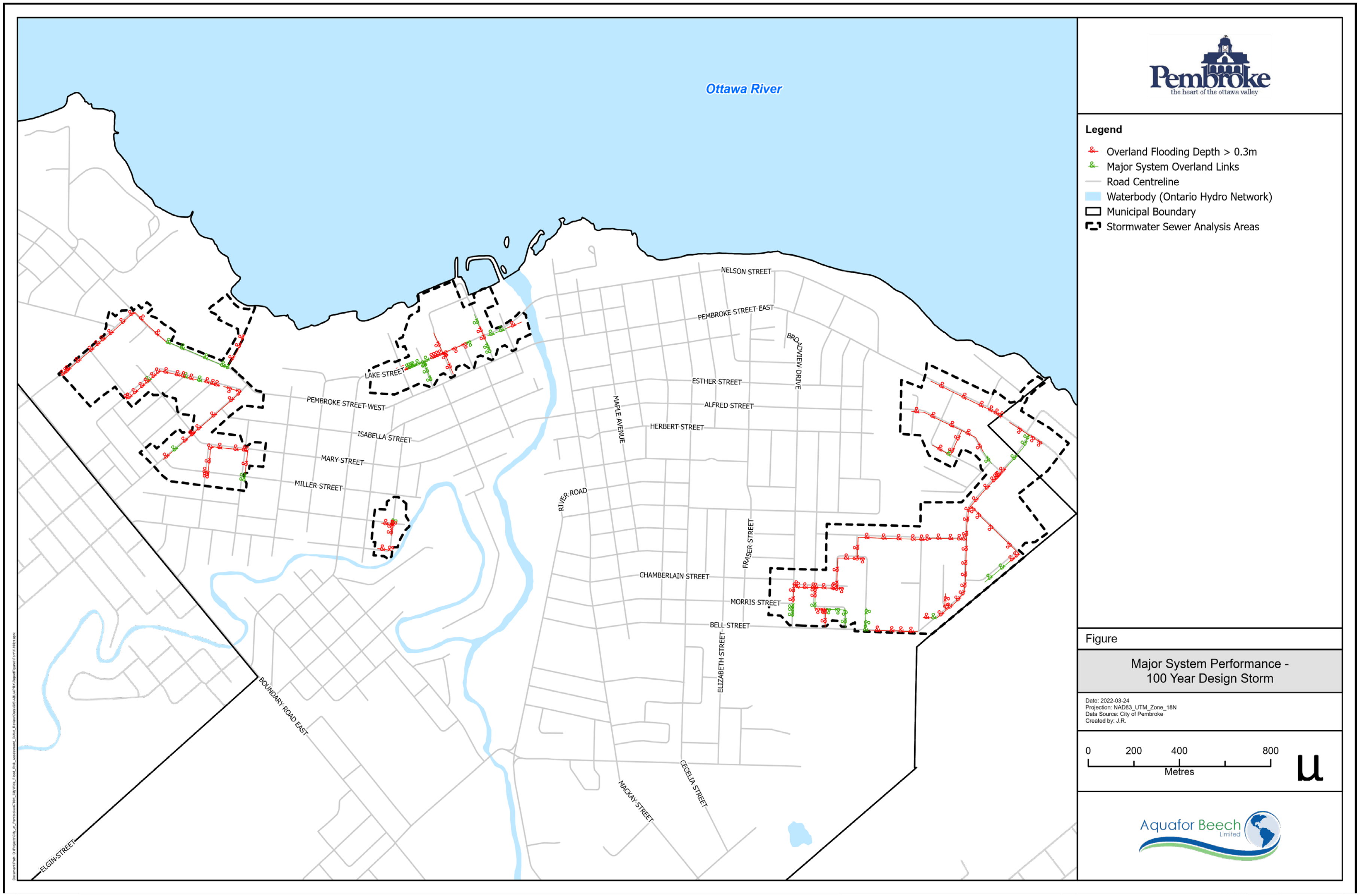


Figure 5-4: Major System Performance under 2-hour, 100-year Design Storm

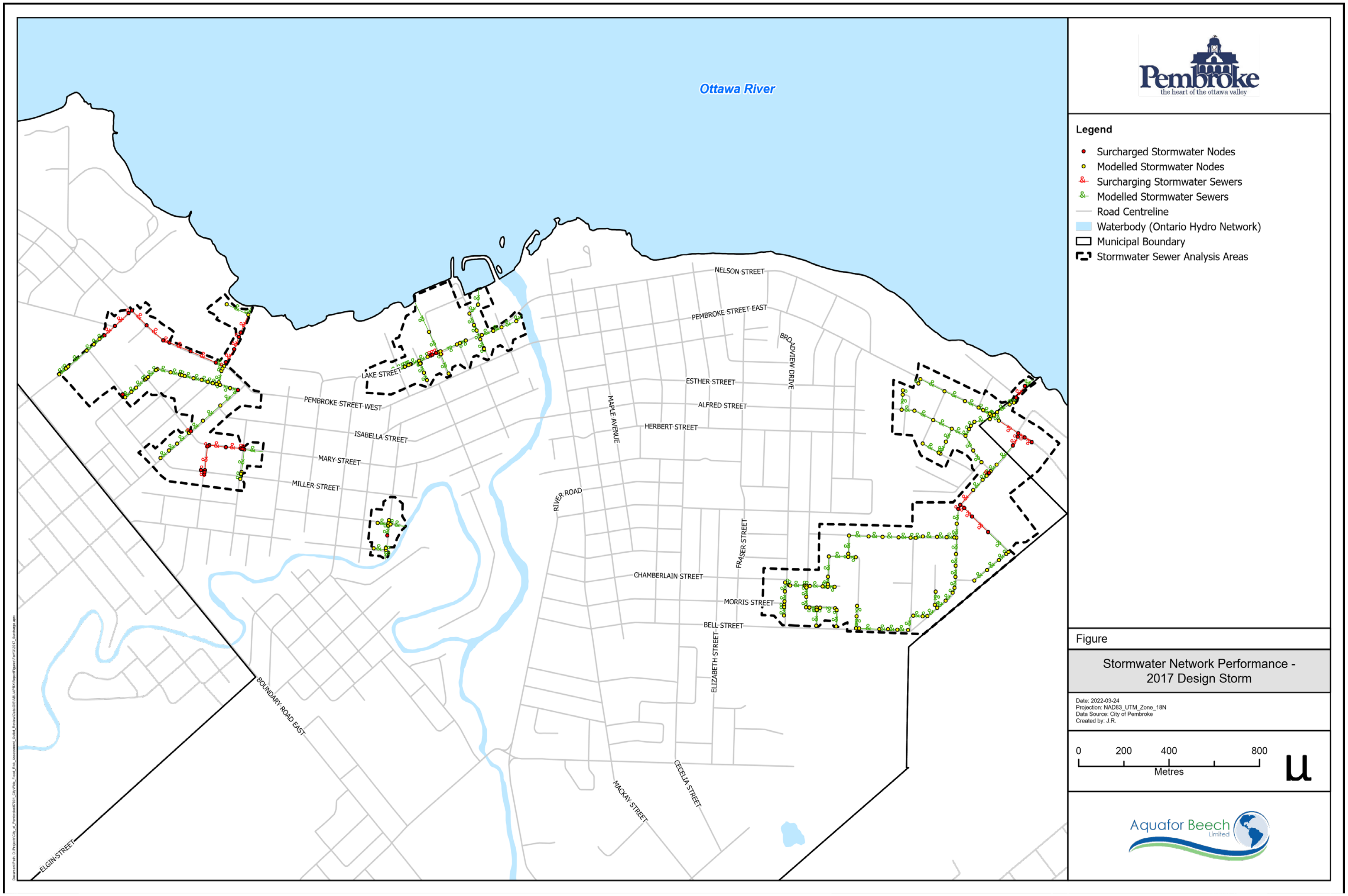


Figure 5-5: Minor System Performance under July 2017 Event

6 STORM OUTFALL ASSESSMENT AND REHABILITATION

This task involved the assessment of 17 storm outfalls, with the intention of designing remedial measures for 7 priority sites. Upon review of the assessment results, the City decided to implement detailed design and tender documents for 9 priority sites. The locations of these priority sites and of with the other 8 assessed outfalls are shown in **Figure 6-1**. A summary of the assessment results is provided in **Table 6.1**, and field records of the assessments are located in **Appendix G**.

Detailed design drawings for each of the 9 sites are provided under a separate cover, and **Appendix H** provides accompanying design briefs.

As the construction of the 9 priority sites will not be undertaken presently, Aquafor also highlighted items that need to be considered that are common to all 9 sites. These items include:

- The acquisition of easements
- Confirmation of staging and access routes
- Approvals and permits

Figure 6-1: Location of Assessed Outfalls and Selected Priority Outfalls

Table 6.1: Summary of Outfall Assessment Results

Priority Rank	Outfall ID	Description of Deterioration and Performance Problems	Size (mm)
1	27	Manhole appears to have fallen, resulting in ~50 m of CSP failure and severe slope erosion. Outlet is now an upstream cast iron pipe. Large concrete blocks at outfall of exposed upstream cast iron pipe	300 diam.
2	41	Outflanked and has two cracks from settling of the foundation: • 1" wide and stretching along 3/4 of the pipe circumference, leakage from crack • 1" wide crack at the seam	600 diam.
3	35	Original design appears to have been a box culvert that collects water from ditches upstream of an old road. Box culvert extended ~15-20 m downstream of the road and eventually lead into a round concrete pipe, which discharged into an outfall channel. Unclear whether box and circular culvert were originally directly connected or joined by an open-air channel. The ~15-20 m of box culvert extension has completely failed, giving rise to a channel running along the side of the box culvert remains. Several sections of the round concrete pipe have also become disconnected from each other and the box culvert.	1230 W x 1250 H (box culvert) 1050 diam. (circular)
4	# Unknown (Paul Martin Dr.)	Bottom of CSP is rusted through at the outlet. Roundstone over geotextile was used as scour protection but the slope is very steep, especially a few metres downstream of the outfall, and has washed out downstream/into the Muskrat River.	300 diam.
5	31	CSP half-pipe conveys water from outfall to channel. CSP is in poor conditions: 1 section is downstream is undermined, 1 has fallen. Some rust corrosion of CSP.	300 diam. (PVC), 525 diam. (CSP Half-Pipe)
6	45	30" of pipe end broke off at the seam, pipe is in good condition at the seam but is recessed into the bank	200 diam.
7	42	Wingwalls failing. Erosion along downstream portion of outfall channel	600 diam.
8	46	Submerged pipe, resident claims the pipe has been decommissioned	250 diam.
9	56	Headwall destroyed Pipe end is partially buried and slightly crushed	300 diam.
10	43	Could not find outfall, but found remnants of broken concrete pipe. Outfall may be located under dumped vegetation/grass clippings	N/A
11	55	Mitered, slight drop but no scour protection, concrete around pipe outlet is critical condition	200 diam.
12	44	Cracking, partially submerged	300 diam.
13	47	Minor rust on grate nuts. Some scour behind headwall.	525 diam.
14	28	Minor rust on grate. Partially submerged in ponded area. Overland flow erosion behind headwall	900 diam.
15	30	Cable-crete scour protection	250 diam.
16	62	No comments	300 diam.
17	29	No comments	450 diam.

7 EVALUATION OF ALTERNATIVE SOLUTIONS

7.1 General

This Chapter provides a description of the alternative solutions that were considered in order to mitigate the problems identified in Chapter 5 and assess the opportunities. There are four alternatives discussed for each problem area, with "Do nothing" as the first alternative and followed by sewer pipe upsizing, inline/offline storage, and a combination of all three solution types. The evaluations of the alternative solutions for each problem area will be discussed and a preferred alternative will be provided in the following sections.

7.2 Level of Service for the Problem Areas

Following a discussion with City Staff on September 2021, the proposed solutions will provide the City a standard of a 100-year level of service (major and minor) for the four identified problem areas which were flooded during the July 2017 event. This will enhance the current level of service within the four problem areas.

7.3 Description of Alternative Solutions

Four general alternatives were considered for each of the problem areas. These include:

1. <u>Alternative 1: Do Nothing</u>
   No mitigation measures would be taken for this alternative, with the exception of ongoing operation and maintenance activities together with emergency measures.

2. <u>Alternative 2: Pipe Upsizing within the Existing Storm System</u>
   Existing sewers where the rate of inflow is greater than the current sewer capacity would be upsized to accommodate more inflow in order to alleviate existing flooding problems.

3. <u>Alternative 3: System Storage (In-line / Off-line) within the Existing Storm System</u>
   This alternative involves restricting the rate of inflow to certain existing sewers to the existing capacity. Flows in excess of the capacity of the existing sewers are directed to localized storage tanks or are temporarily stored within the road right of way.

4. <u>Alternative 4: Pipe Upsizing and System Storage (In-line / Off-line) within the Existing Storm System</u>
   This alternative will include pipe upsizing and local storage measures at strategic locations to improve the system performance and mitigate existing flooding problems.

The above alternatives were considered for all areas and evaluated based on the evaluation criteria in **Section 7.4**.

7.4 Evaluation Criteria

In order to evaluate the alternative solutions identified in the previous sections, four general categories of criteria were considered. The four (4) categories are natural environment, social / cultural environment, economical / financial, and technical, and the detailed list of criteria for each category is summarized in **Table 7.1**. The evaluation of the alternatives based on these criteria would form the basis and justification for the selection of the preferred alternative(s).

Table 7.1: Summary of Evaluation Criteria

Category	Criteria	Description of Criterial
Natural Environment	Impact of existing vegetation	Potential to impact existing vegetation.
	Impact on surface flooding	Potential to decrease surface flooding.
	Impact on erosion	Potential to mitigate existing erosion issues.
Social / Cultural Environment	Potential disruption to community	Potential for the proposed alternative to impact residents as a result of construction practices, rerouting of traffic or items associated with proposed construction (e.g. noise, dust, mud, etc.).
	Potential benefit to community	Potential for the proposed alternative to provide positive impact to residents.
Economical / Financial	Estimated construction cost	The relative capital cost as compared to the other alternatives.
	Estimated operation & maintenance cost	The relative cost of maintaining the works in short-term and long-term based on factors such as access/ egress, ownership implications, future risks due to failures or flooding, overall operation frequency and intensity.
	Potential requirements for property acquisition / easements	Potential cost to acquire any lands that may be necessary in order to construct or maintain proposed infrastructure.
Technical	Feasibility of alternative	The relative ease with which the alternative can be implemented taking into consideration of approvals, community landowner acceptance, and length of time to implement.

7.5 Selection and Description of the Preferred Alternative

Based on the results of the alternative evaluations and consultation, the preferred alternative for each problem was selected and presented in **Table 7.2**. The EA Schedule for all proposed undertakings associated with the preferred alternatives is also shown in the table below. A detailed description of each Preferred Alternative is provided in the following sections. The proposed works including storm sewer pipes and catch basins of each area are tagged as “MOD” in the PCSWMM model, and they are also listed in **Appendix J**.

Table 7.2: Summary of Preferred Alternatives

Problem Area	Preferred Alternative	Municipal Class EA Schedule
Area 1 – Amalgamated Area	Alternative 2 – Pipe Upsizing and deepening	Schedule A
Area 2 – Darcy St / Doran St	Alternative 2 – Pipe Upsizing and deepening	Schedule A
Area 3 – Lake St	Alternative 2 – Pipe Upsizing and deepening	Schedule A
Area 4 – Trafalgar Rd / Mary St	Alternative 2 – Pipe Upsizing and deepening	Schedule A

7.5.1 Area 1 – Amalgamated Area

Alternative 2 – Pipe Upsizing within the Existing Storm System

This alternative considered the replacement of the existing 300 mm to 750 mm storm sewers with 375 mm to 1050 mm storm pipes from Broadview Dr through Chamberlain St, Noik Dr, Cooper St to Melton St. Existing storm sewers ranging from 600 mm to 900 mm would be replaced with 900 mm to 1650 mm on Angus Campbell Dr, with 675 mm storm sewers upsized to sewers ranging from 825 mm to 1200 mm on Matheson Dr. The existing storm sewers 525 mm to 600 mm on Alfred St E west of Kyie Ave would be replaced with 675 mm to 750 mm storm sewers. The existing storm sewers ranging from 375 mm to 1350 mm would be upsized with 525 mm to 1650 mm storm sewers on Pembroke St E. Also, the storm sewers ranging from 900 mm to 1200 mm north of Alfred St E to the outlet would be upsized with 1350 mm to 1800 mm storm sewers. These sewer upsizing would increase the flow capacity of the sewer system and avoid flow surcharging of the storm pipes. In addition, due to the shallow existing storm sewer, storm pipe on Bell St would be replaced with 1 m depth ditch. Storm sewer deepening on Broadview Dr, Chamberlain St, Noik Dr, Cooper St, Melton St, Angus Campbell Dr, Matheson Dr and Pembroke St E is also necessary in Amalgamated area to meet the minimum of 1.5 m coverage over the storm pipes. The proposed works are illustrated in **Figure 7-1**.

Other alternatives considered were in-line system storage on Pembroke St E, Angus Campbell Dr, and Melton St with pipe upsizing on Broadview Dr, Chamberlain St, Noik Dr, Matheson Dr, and Alfered St E, but was deemed not technically possible due to steep surface slope and shallow existing pipe system, not able to meet outlet discharge restriction of sewer pipe size on private property, and not cost effective.

7.5.2 Area 2 – Darcy St / Doran St

Alternative 2 – Pipe Upsizing within the Existing Storm System

This alternative includes deepening and replacement of existing 200 mm to 300 mm storm sewers with 300 mm to 525mm pipes on Dracy St, Doran St, and McGee St E. Sewer upsizing can provide more capacity to convey flows, and sewer pipe deepening will help meet the 1.5 m minimum coverage requirement. Also, three catchbasins on Doran St were replaced with higher discharge catchbasins in order to allow more surface water to enter the sewer pipe system and decrease surface flooding. The proposed works are shown in **Figure 7-2**.

This sewer upgrade alternative was the only alternative considered for Area 2 as the solution is the least complex and effectively addresses flood risk.

7.5.3 Area 3 – Lake St

Alternative 2 – Pipe Upsizing within the Existing Storm System

This alternative involves storm sewer deepening on Lake St and Agnes St and upsizing from existing 300 mm to 450 mm storm sewer pipes with 375 mm to 600 mm pipes on Lake St, 300 mm storm sewer with 375 mm to 450 mm pipes on Agnes St, 750 mm pipes on Alexander St north of Lake St with 1050 mm to 1200 mm sewer pipes, and 300 mm to 375 mm pipes with 375 mm to 600 mm pipes on Albert St. There are three catchbasins require upgrade with higher inlet catchbasins to reduce the surface flooding, and they are on Alexander St and Albert St. A plan view of the proposed works is illustrated in **Figure 7-3**.

This sewer upgrade alternative was the only alternative considered for Area 2 as the solution is the least complex and effectively addresses flood risk.

7.5.4 Area 4 – Trafalgar Rd / Mary St

Alternative 2 – Pipe Upsizing within the Existing Storm System

The proposed works for Trafalgar Rd area include Storm Sewer deepening on Forced Rd, Pembroke St W, Trafalgar Rd, and pipes from Front St to Pembroke St through private property in order to meet the 1.5 m of minimum coverage. The existing 375 mm storm sewers on Forced Rd would be replaced with 450 mm pipes, existing 900 mm sewer pipes on Pembroke St W would be replaced with 1050 mm to 1200 mm sewer pipes, 300 mm to 450 mm storm sewer pipes on Trafalgar Rd would be replaced with 450 mm to 750 mm pipes, 450 mm to 750 mm storm pipes on Front St would be replaced with 750 mm to 900 mm storm pipes, and 200 mm to 375 mm sewer pipes on Third Ave would be replaced with 375 mm to 450 mm sewer pipes. Also, storm sewers from Front St junction to the outlet upsizing would be from existing 600 mm to 900 mm with 825 mm to 1350 mm sewer pipes. Some catchbasin inlet structures need to be replaced with larger inlet catchbasin type in this area to provide more discharge to the storm sewer system and reduce surface flooding.

The alternative for Mary St area involves storm sewer upsizing of all existing storm sewer pipes from 200 mm to 450 mm and would be replaced with 300 mm to 900 mm sewer pipes. The rest of this sewer system were not mentioned as flooded area in the meeting with city staff. Therefore, an assumption was made that the downstream of this sewer system could handle the storm water from the proposed works and would remain no change at this point.

The proposed works of Trafalgar Rd and Mary St area are shown in **Figure 7-4**.

System storage alternative was considered on Front St, Forced Rd and Trafalgar Rd, but because of the steep slope on Forced Rd and Trafalgar Rd and the limited flow capacity of the sewer pipes between Front St and Pembroke St W, system storage would not meet the low flow discharge and would cost more on pipe upsizing.

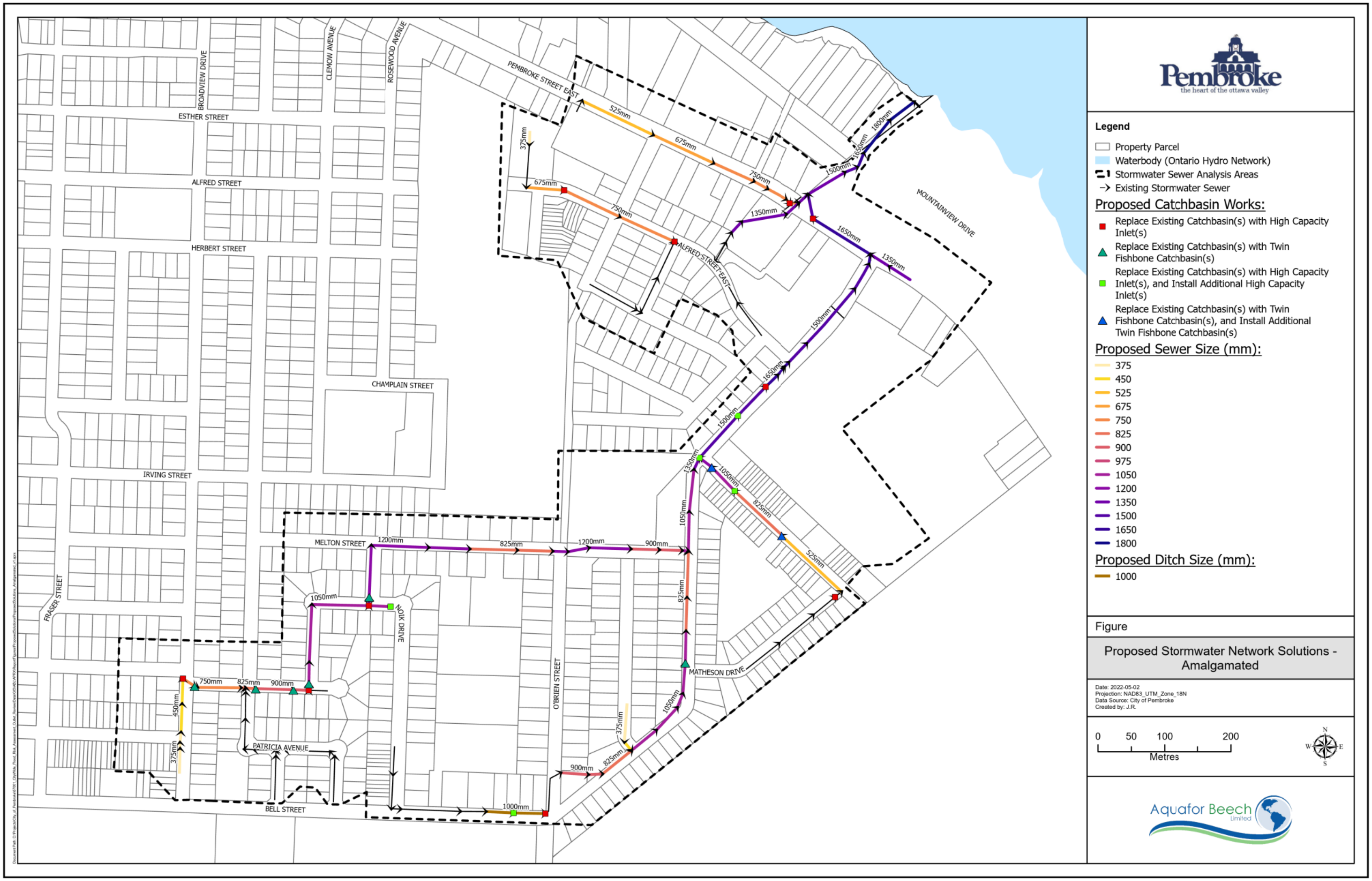


Figure 7-1: Proposed Stormwater Network Solutions of Amalgamated Area

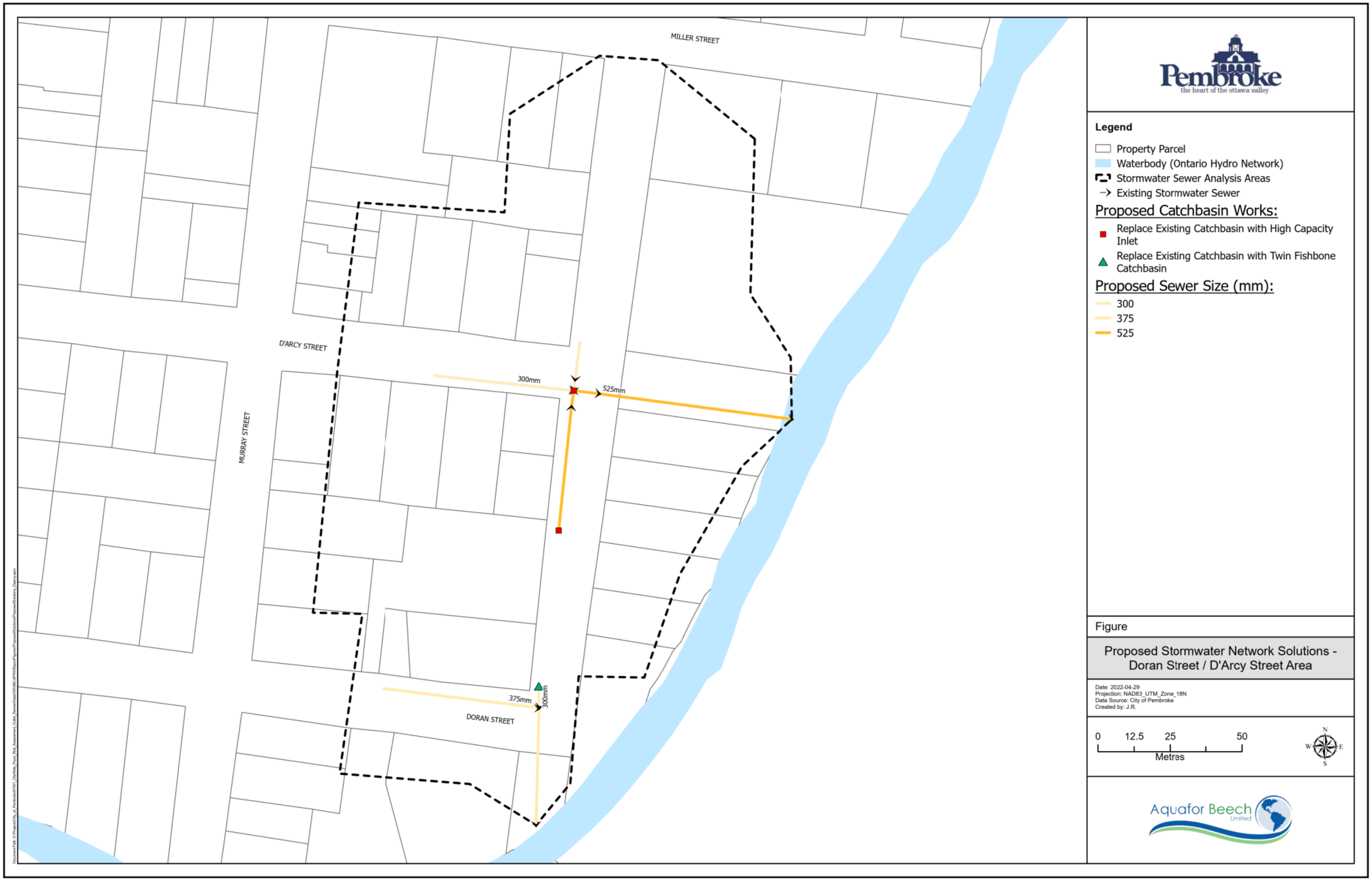


Figure 7-2: Proposed Stormwater Network Solutions of Darcy St Area

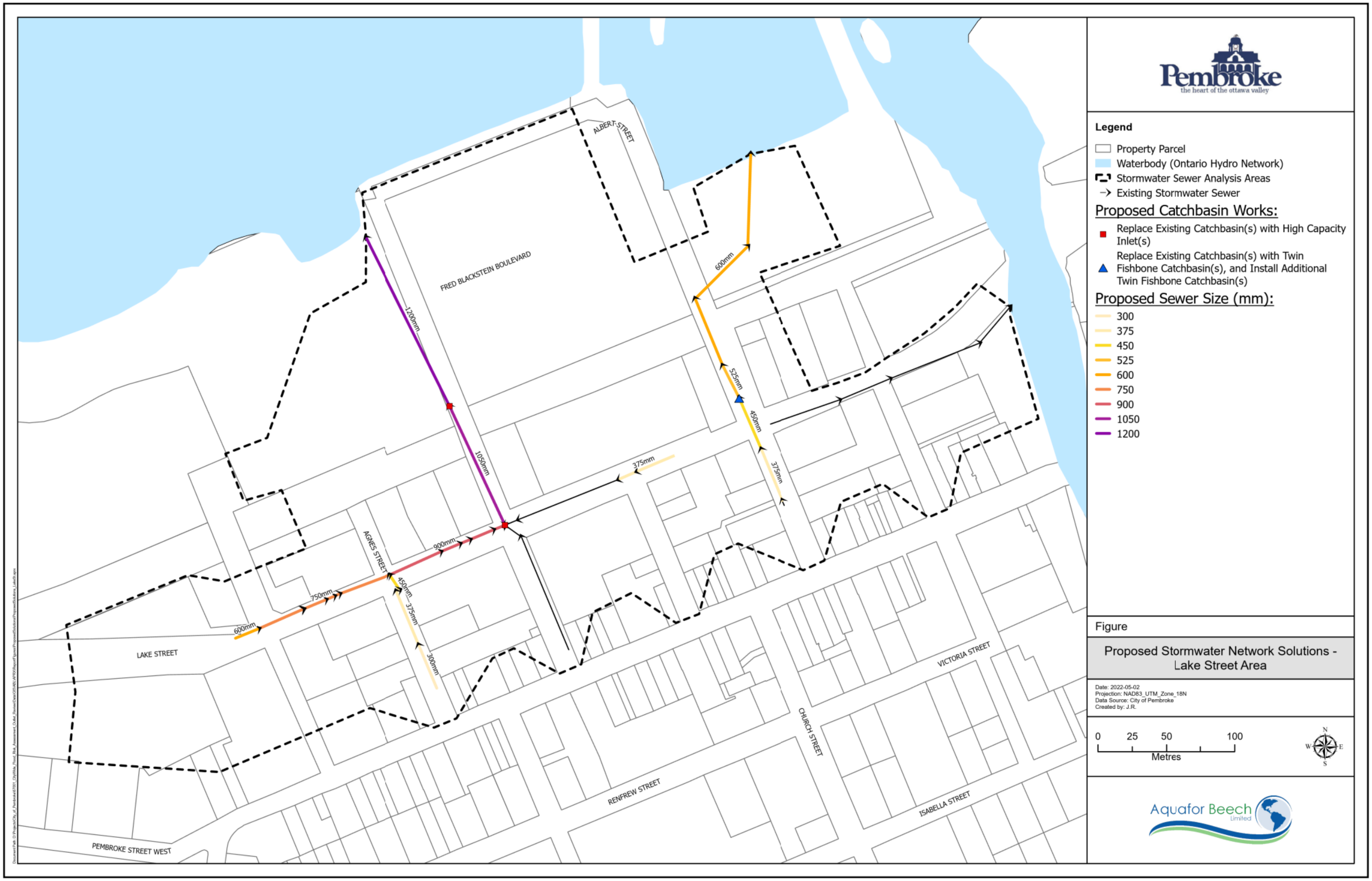


Figure 7-3: Proposed Stormwater Network Solutions of Lake Street Area

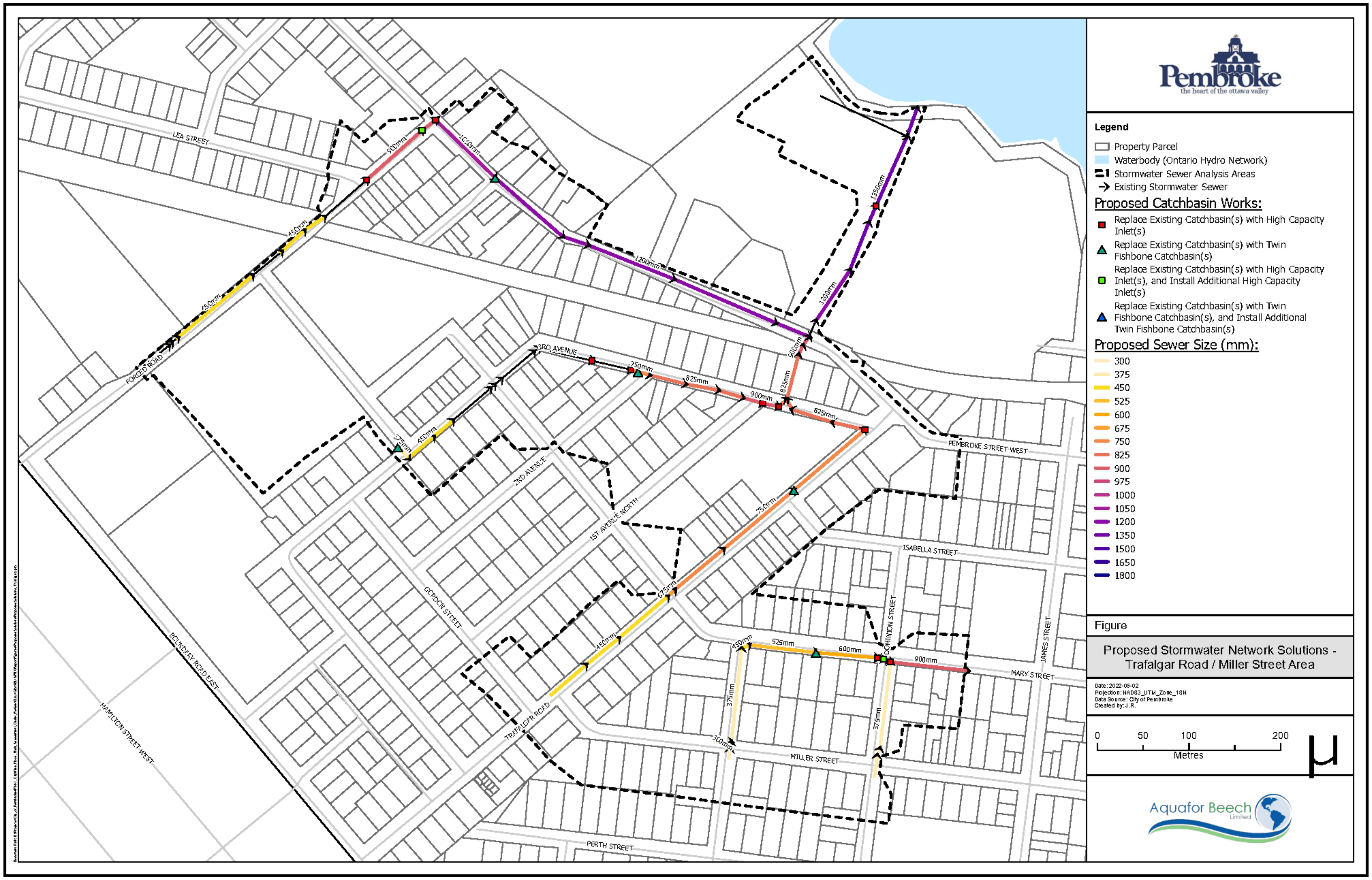


Figure 7-4: Proposed Stormwater Network Solutions of Trafalgar Area

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