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TOWN OF TECUMSEH

Storm Drainage Master Plan

VOLUME 2 - FINAL TECHNICAL MODELLING REPORT

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1.0 Introduction

The Town of Tecumseh has experienced several significant storm events over the years that have resulted in widespread surface and basement flooding. Since the Town has been keeping records of surface and basement flooding calls, extreme rainfall events resulting in flooding have been noted in September 1981, July 1983, February 1985, February 1990, June 2010, September 2011, July 2013, September 2016 and August 2017. Based on reported surface and basement flooding throughout the Town, a number of storm and sanitary sewer assessment studies were completed in the early 2000's up to 2016 to review the existing municipal infrastructure. A Sanitary Sewage Collection System Improvements Class Environmental Assessment (Dillon Consulting, 2013) was amongst the studies included in this time. Upon finalizing the Sanitary Class EA, the Town began discussions of completing a Storm Drainage Master Plan for the areas serviced by means of storm pump stations within the Town. Other motivators for the examination of the storm drainage system is due to a number of factors, including reports of frequent surface flooding in low lying urbanized areas and aging infrastructure in which improvements would be necessary in the near future.

Based on these discussions, a proactive decision was made to begin reviewing the condition of the existing storm pump stations and storm drainage in service areas where road reconstruction was proposed in the future. Three studies were finalized in 2016 and included a Pump and Metering Station Condition Assessment Report (Dillon, November 2016), PJ Cecile (Kensington) Storm Pump Station – Review of Drainage Area and Contributing Flow (Dillon, September 2016) and St. Mark's and Scully (Edgewater) Storm Pump Stations – Review of Drainage Areas and Storm Servicing Alternatives (Dillon, August 2016). These initial studies paved the way for a Town wide Storm Drainage Master Plan.

Prior to commencement of the Storm Drainage Master Plan, an extreme rainfall event hit the region on September 28/29th 2016, which well exceeded a 1:100 year event over a 24 hour period. This rainfall event overwhelmed the existing storm sewer system and storm pump stations and led to widespread surface flooding along both municipally owned roadways and on private property. Severe surface flooding in lower lying areas caused temporary road closures. During this time, the extent of surface flooding affected the municipal sanitary system through direct inflow and infiltration, leading to extensive basement flooding. Following this extreme rainfall event, the Town of Tecumseh initiated the Storm Drainage Master Plan Class Environmental Assessment Study (EA) *(hereafter referred to as Storm Drainage MP)*.

1.1 Study Purpose and Scope

The purpose of the Storm Drainage MP is to address the impacts of surface flooding on the mainly urbanized residential areas of the Town located along the northern and eastern limits. This includes the assessment of storm pump stations, gravity outfalls and the respective service area minor (sewer) and major (roadway and overland) systems discharging out to Lake St. Clair and Pike Creek. The Storm Drainage MP was initiated to:

- Confirm the factors contributing to surface flooding resulting from significant storm events that exceed the guidelines;
- Determine existing condition problem areas throughout the study area with surface ponding exceeding acceptable levels;
- Identify areas of future development and incorporate the future level of service design into a future conditions model;
- Identify and evaluate alternative solutions within the future conditions model to reduce the risk and impacts of surface flooding;
- Identify recommended design solutions based on a traditional level of service for the design;
- Simulate the effects of climate change on the recommended solutions to consider a more enhanced the level of service, if warranted; and
- Outline a recommended long-term implementation strategy for the preferred surface flooding solutions.

The Storm Drainage MP included the development of an integrated 1-Dimensional/2-Dimensional Stormwater Management (SWM) model using PCSWMM software, which integrates the use of comprehensive dynamic rainfall-runoff simulations. PCSWMM is widely used throughout the world for the analysis of complex hydrologic and hydraulic solutions and assessments for urban and rural areas. The model constructed for this study integrated the linear storm infrastructure throughout the Town, including storm sewers, catch basins, pump station and municipal drains as a 1-Dimensional hydraulic model for stormwater runoff. The advanced 2-Dimensional modelling component was used to integrate the existing ground surface (roadways, overland flow routes, private property land topography) using digital elevation data obtained during the background investigation work to more accurately simulate the dynamic movement of storm runoff along the surface during simulated rainfall events.

The Storm Drainage MP follows the requirements of the Municipal Class Environmental Assessment (Class EA) (2000, as amended) - Approach No. 2 and the requirements of Phases 1 and 2 of the Class EA, including requirements for any Schedule B projects. The preferred surface flooding solutions are designed to a functional level of detail.

Only broader-based surface flooding solutions are considered for the review of alternatives and the respective evaluations through the master planning process, unless specific localized surface flooding solutions are included based on their impact on private property and the community.

This Storm Drainage MP does not directly focus on basement flooding resulting from sanitary sewer surcharging, which the Town of Tecumseh has been addressing separately through other studies, initiatives, and subsidy programs since 2010. The study also did not take into consideration surface flooding due to high Lake Levels, which is expected to be addressed in a future study outlined within the Town's Flood Mitigation Strategy.

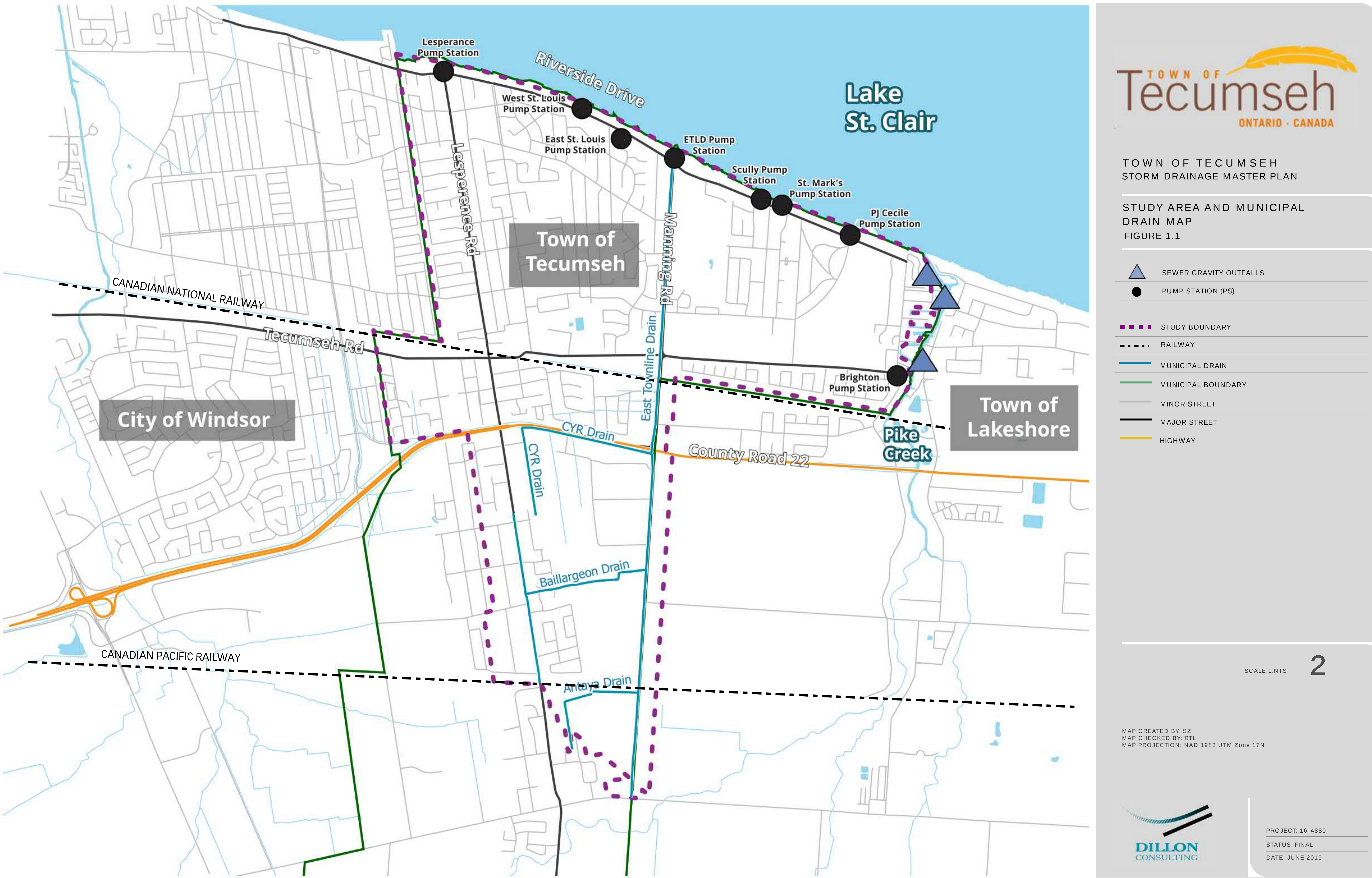
1.2 Study Area

As shown in Figure 1.1, the Storm Drainage MP Study Area has an approximate area of 12.78 km^2.It is bordered by Lake St. Clair to the north, Pike Creek and Town of Lakeshore to the east, County Road 42 to the south and City of Windsor to the west, and encompasses the residentially urbanized portion of the Town of Tecumseh north of County Road 42, which includes portions of the former Township of Sandwich South, the Village of St. Clair Beach and the Town of Tecumseh.

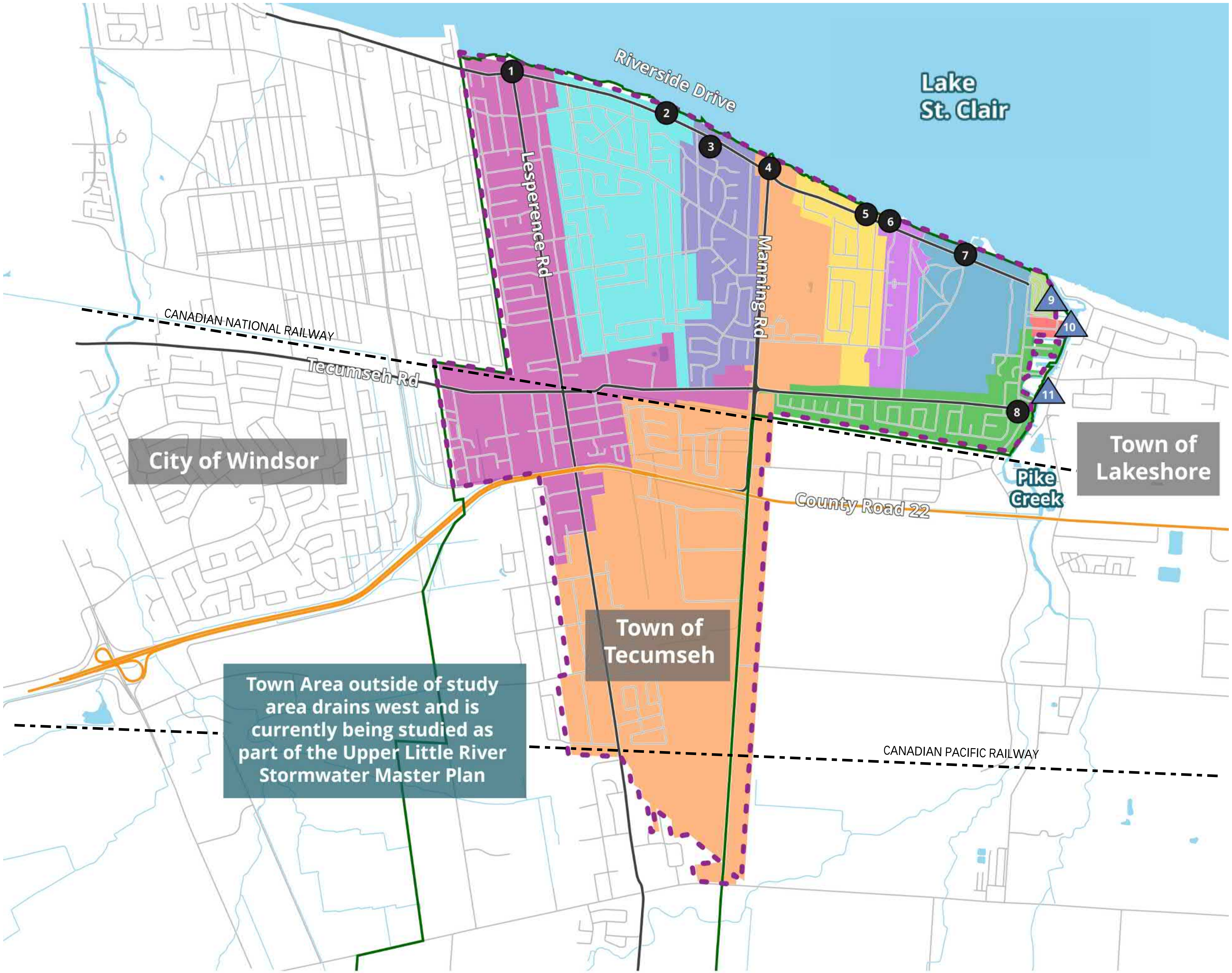
Within the study area, there are a total of five (5) municipal drains: the East Townline Drain, the Manning Road Drain, the Baillargeon Drain, the Cyr Drain and the Antaya Drain. The full study area is within the Lake St. Clair Watershed and is currently serviced by eight (8) storm pump stations and three (3) gravity outfalls. This includes:

- Lesperance Storm Pump Station (273.50 ha);
- West St. Louis Storm Pump Station (174.90 ha);
- East St. Louis Storm Pump Station (91.40 ha);
- East Townline Drain (ETLD)/Manning Road Storm Pump Station (480.95 ha);
- Scully (Edgewater) Storm Pump Station (55.90 ha);
- St. Mark's Storm Pump Station (33.90 ha);
- PJ Cecile (Kensington) Storm Pump Station (79.01 ha);
- Brighton Storm Pump Station (73.47 ha);
- Pilots Cove Storm Gravity Outfall (5.15 ha);
- Southwind/Starwood Storm Gravity Outfall (3.96 ha); and
- Mei-Lin Storm Gravity Outfall (0.65 ha).

The service areas for each outlet are further delineated within the project study area in Figure 1.2.



Lesperance Pump Station
Riverside Drive
West St. Louis Pump Station
East St. Louis Pump Station
ETLD Pump Station
Scully Pump Station
St. Mark's Pump Station
PJ Cecile Pump Station
Brighton Pump Station
Lake St. Clair
Lesperence Rd
Manning Rd
Town of Tecumseh
CANADIAN NATIONAL RAILWAY
Tecumseh Rd
East Townline Drain
City of Windsor
CYR Drain
County Road 22
Pike Creek
Town of Lakeshore
Baillargeon Drain
Antaya Drain
CANADIAN PACIFIC RAILWAY
TOWN OF Tecumseh ONTARIO · CANADA
TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN
STUDY AREA AND MUNICIPAL DRAIN MAP
FIGURE 1.1
SEWER GRAVITY OUTFALLS
PUMP STATION (PS)
STUDY BOUNDARY
RAILWAY
MUNICIPAL DRAIN
MUNICIPAL BOUNDARY
MINOR STREET
MAJOR STREET
HIGHWAY
SCALE 1:NTS
2
MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N
DILLON CONSULTING
PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019



TOWN OF Tecumseh
ONTARIO · CANADA

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

STORM PUMP STATION SERVICE AREA AND GRAVITY OUTFALL MAP
FIGURE 1.2

SEWER GRAVITY OUTFALLS
PUMP STATION (PS)
STUDY BOUNDARY
RAILWAY
MUNICIPAL DRAIN
MUNICIPAL BOUNDARY
MINOR STREET
MAJOR STREET
HIGHWAY

PUMP STATION CATCHMENT AREAS
1. LESPERANCE PUMP STATION
2. WEST ST. LOUIS PUMP STATION
3. EAST ST. LOUIS PUMP STATION
4. EAST TOWNLINE DRAIN PUMP STATION
5. SCULLY PUMP STATION
6. ST. MARK'S PUMP STATION
7. PJ CECILE PUMP STATION
8. BRIGHTON PUMP STATION

OUTFALL CATCHMENT AREAS
9. PILOTS COVER OUTFALL
10. SOUTHWIND/STARWOOD OUTFALL
11. MEI-LIN OUTFALL

SCALE 1:NTS
2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

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2.0 Design Criteria and Guidelines

As part of this study, a number of storm drainage guidelines were reviewed, both regionally and provincially, to determine the proposed level of service design criteria to be used for both the storm sewer and overland flow conveyance of storm runoff. The investigation of other municipality standards assist with identifying drivers on how to assess existing surface flooding and determine proper design criteria for surface flooding solutions during larger storm events within the Town of Tecumseh. This section details the guidelines and development manuals reviewed, as well as the aspects of the storm drainage system within the Town.

2.1 Storm Drainage System

2.1.1 Storm Sewers (Minor System)

Storm sewer systems are underground systems for the conveyance of storm runoff. A storm sewer system is traditionally designed to the municipally accepted level of service at the time of implementation for the conveyance of more frequent rainfall events. The design of the minor system is traditionally completed using static calculations based on rainfall information provided through Intensity-Duration-Frequency (IDF) curves provided by Environment Canada at regional weather stations. Design standards typically allow for an accepted level of service for the storm sewer systems ranging from a 1:2 year event (50% chance of occurrence in a given year), 1:5 year event (20% chance of occurrence in a given year) to even a 1:10 year event (10% chance of occurrence in a given year), as required by municipalities.

2.1.2 Overland Drainage Systems (Major System)

Overland drainage systems (roadways, surface conveyance) are typically required to be designed to convey flows beyond the capacity of the storm sewer (minor system) level of service. Overland flow routes, including roadway networks and low lying areas are to be designed to ensure that runoff along the surface is safely conveyed to the respective storm outlets during storm events up to and including the 1:100 year event (1% chance of occurrence in a given year). Current provincial design standards (MECP Stormwater Management Planning and Design – 2003) state that ponding depths along municipally owned Right-of-Ways are to be limited to less than 0.30 m during the 1:100 year event. Previous design analysis for overland drainage systems entailed a review of the proposed static grading design to establish overland flow networks.

The Windsor-Essex area, including the study area within the Town of Tecumseh is relatively flat in nature with no natural or manmade overland flow routes to Lake St. Clair. As such, during larger storm events when the minor system is at capacity, pump stations are relied on to drawdown the storm sewer system to allow surface ponding along roadways to be kept below the acceptable depth of 0.30m (as per the MOECC Stormwater Management Planning and Design Manual -2003), and eventually conveyed through the storm sewer system once conveyance capacity is available. The Windsor/Essex Region Stormwater Management Standards Manual (December 2018) identifies that dynamic modelling is necessary for new developments to more accurately assess overland flow routes and surface ponding depths.

2.1.3 Municipal Drains

A municipal drain is traditionally an open drainage system that is primarily located in rural agricultural areas of the province. The municipal drains have been a fixture of rural Ontario's infrastructure since the 1800's and most municipal drains were constructed to improve the drainage of agricultural land, but as development has occurred, they have now become part of the urban drainage system in many communities, including the Town of Tecumseh. They generally provide a 1:2 year level of service based on an undeveloped condition, and may also be located in naturally lower lying areas that provide a degree of major storm runoff conveyance. Some drains within more urbanized areas are enclosed, which can be designed for a greater level of service than the 1:2 year.

2.1.4 Gravity Storm Outfalls

Storm gravity outfalls are typically implemented where the upstream storm sewer system is sufficiently higher than the outlet water level conditions that they may be drained without the risk of surface flooding for more frequent storm events. Gravity outfalls within this region are limited due to the relatively flat topography of the area.

2.1.5 Storm Pump Stations

Storm pump stations are typically constructed at the most downstream end of a system and are required in low lying areas where gravity storm outlets are not feasible due to grading constraints. Pump stations are also used in areas where the outlet of the storm sewer system discharges to a waterway in which water levels interfere with the functionality of the storm sewer. In the past, pump stations have typically been designed to handle expected level of service flows within the storm sewer system. Some municipal design standards throughout the region now require that pump stations are designed to limit significant sewer surcharging and surface (roadway) ponding to below 0.30m during more infrequent large storm events. This new design standard is typically used where overland flow routes to a respective drainage outlet cannot be achieved due to the natural topography of the land. Stormwater runoff is expected to be temporarily storage along roadways until the storm sewer system has available capacity to drawdown roadway surface ponding and safely convey flows to the pump station outlet.

2.2 Stormwater Management Planning and Practices

As land development increases throughout municipalities in Ontario, proper SWM planning and design is becoming an increasingly important aspect of land development and roadway drainage design. Due to the evolution of SWM design principles in the 1990's and 2000's, the Ministry of the Environment, Conservation and Parks (MECP), formerly known as the Ministry of the Environment, issued the "Stormwater Management Planning and Design Manual (March 2003)". This guidance document was created to provide technical and procedural guidance for the planning, design and review of stormwater management practices.

Like most municipalities, the Town of Tecumseh has adopted these provincial guidelines for managing land drainage. Consultants across the region use the manual as a tool for establishing the design requirements of stormwater management and drainage solutions. Many municipalities and conservation authorities have developed complimentary SWM guidelines and design criteria to assist practitioners with SWM regulations and standards that are more specifically tailored to the local environment. A summary of the criteria applicable in several other Ontario municipalities plan, design and manage their storm drainage systems is provided below:

Table 2.1: Ontario Municipalities Storm Drainage Design Criteria

Municipality	Minor System Design Criteria	Major System Design Criteria	Climate Change Design Criteria
City of Windsor*	1:5 year - HGL below ground	1:100 year – Surface ponding < 0.30m	Increased storage volume and major system conveyance under 1:100 year 24 hour event + 39% (1 hour time step)
City of London	1:2 year – HGL below obvert of sewer	1:100 year – Surface ponding maintained within ROW.	Overland flow safely conveyed to outlet during 1:250 year event
City of Hamilton	1:5 year – Sewer not surcharged (< 85% of sewer capacity)	1:100 year – Surface ponding < 0.15m above crown of road	
City of Toronto	1:2 year – HGL below obvert of sewer	1:100 year – Surface ponding < 0.15m above crown of road	
City of Ottawa	1:5 year– HGL below obvert of sewer	1:100 year – Surface ponding < 0.30m	Increased storage volume and major system conveyance under 1:100 year event + 20%.

** Design Criteria taken from Windsor/Essex Region Stormwater Management Standards Manual (December, 2018)*

The subsections provided below outline the stormwater infrastructure within the Town of Tecumseh.

2.3 Existing Conditions Storm Drainage Level of Service

The level of service of a storm drainage system is based on the provincial and locally accepted level of service standards at the time of implementation. As part of this study, an assessment was completed to confirm the existing level of service. The following sections identify the level of service that the Town of Tecumseh has historically applied in the planning and design of their storm drainage system.

2.3.1 Storm Sewers (Minor System)

The majority of the municipal storm sewer system in the Town of Tecumseh was constructed from 1950 – 1990 and was designed for a 1:2 year level of service based on the local IDF information used at that time. This level is service is typical for municipalities throughout the region. The design of the municipal storm sewer system would have typically been completed using the modified rational method based on a free flow conveyance for a 1:2 year storm event to an appropriate outlet. Outlet boundary conditions and capacity limitations would have been considered based on available information, or reasonable assumptions.

Based on the existing storm sewer level of service, it is expected that there would be limited surface ponding on roadways for a 1:2 year design storm event. The original design of the storm sewer system would not have considered dynamic modelling of the drainage system.

2.3.2 Overland Flow (Major System)

Overland flow routes throughout the Town of Tecumseh consist of roadway networks that were designed to temporarily store runoff within the municipal right-of-way during larger storm events above and beyond a 1:2 year storm. Based on the flat topography of the study area, overland flow routes to convey storm runoff to the respective storm outlet, in this case Lake St. Clair, during larger storm events are not feasible. Barrier landforms throughout the study area, including County Road 42, Manning Road/County Road 19 and the CN Railway limit overland flow conveyance during large storm events. Municipal drains and the traditional storm sewer systems are relied upon to drain most urbanized areas throughout the Town. As stated in Section 2.1.2, pump stations can be relied on to drawdown the storm sewer system to allow surface ponding along roadways to be kept to below the acceptable depth of 0.30m and eventually conveyed through the storm sewer system once conveyance capacity is available.

During larger, more extreme rainfall events, the Town of Tecumseh has allowed for no more than 0.30 m of surface ponding during the 1:100 year event based on static grading design. The roadway grading design can generally meet this design criterion by having road sags no more than 0.30 m lower than the nearest highpoint elevation. In the past, there was no mandatory overland flow modelling requirements across the province or within the Town to dynamically analyze the overland flow network and confirm that roadway ponding depths did not exceed 0.30 m for the 1:100 year event.

2.4 Provincial Guidelines for Low Impact Development (LID)

The Ontario Ministry of Environment, Conservation and Parks (MECP) produced a LID Stormwater Management Guidance Manual (Draft) in 2017 to provide guidance on the design, construction and maintenance of LIDs. This manual was intended to act as a companion to the Stormwater Management Planning and Design Manual (MECP, 2003), which advocated a “treatment train” approach to treat and store stormwater runoff. This approach encouraged the use of lot-level and conveyance controls along with end-of-pipe measures to manage stormwater runoff. The principles of Green Infrastructure (GI) and

LID promote a shift towards an ecosystem-based water balance approach in lieu of rapid conveyance using sub-surface pipes and end-of-pipe controls.

The LID Stormwater Management Guidance Manual [Draft] (MECP, 2017) defines specific Runoff Volume Control Targets (RVC_T) for new development, redevelopment, infill-development, re-urbanization, linear infrastructure and retrofits in Ontario. It requires limiting of runoff volume to 10% (or less) of total rainfall volume, which corresponds to control of 90% of the average annual rainfall, determined through the use of the 90^{th} percentile event. The Recommended future RVC_T for the Windsor region is specified to be 32 mm based on the 90^{th} percentile rainfall event.

At this time, the future RVC_T for the Windsor area is not taken into consideration as part of this study, but is recommended to be further reviewed once detail design commences.

2.4.1 Linear Development Volume Control Exemption

The LID Stormwater Management Guidance Manual [Draft] (MECP, 2017) provides an exemption to the volumetric control criteria defined above for linear development projects like roadway resurfacing. These kind of projects are not classified as new linear projects in the manual and are not mandated to follow the volume control regulations defined. They are encouraged to achieve volume control to Maximum Extent Possible (MEP).

MEP is defined in the manual as "maximum achievable volume control, beyond the water balance requirement, using all known, available and reasonable, including the methods as described in the manual, given the site restriction."

2.5 Future Conditions Storm Drainage Level of Service

The focus of this study in assessing drainage systems for existing developed areas is to establish solutions that would provide an acceptable level of service for surface flooding. It was determined that this study would not specifically focus on improving the level of service of the existing storm sewer system for frequent storm events, but rather to focus on addressing surface flooding solutions under the more infrequent, extreme storm events, which may also involve a degree of minor storm sewer system improvements.

It was identified at the onset of the Tecumseh Drainage MP that surface flooding problem areas within the study area would be identified based on the dynamic modelling analysis of surface ponding depths within the municipal right-of-way where surface ponding exceeds 0.30 m during the 1:100 year storm event. This criterion is consistent with the recently completed Windsor/Essex Region Stormwater Management Standards Manual (December, 2018).

2.5.1 Storm Sewers (Minor System)

Based on discussions with the Town and the review of the newly adopted Windsor/Essex Region Stormwater Management Standards Manual (December 2018), future storm sewers for new developments within the study area are proposed to be designed to a 1:5 year level of service with no surface ponding along roadways during the minor system event. Existing municipal storm sewers being replaced in the future are also to be designed to a 1:5 year level of service, if considered practical and if there is sufficient downstream capacity to accept the increased conveyance of flows.

2.5.2 Overland Flow (Major System)

Based on discussions with the Town and the review of the newly adopted Windsor/Essex Region Stormwater Management Standards Manual (December 2018), overland flow networks, including roadway sags for any future development areas, are to maintain dynamic surface ponding depths to below 0.30 m during rainfall events up to and including the 1:100 year event. Existing roadways proposed to be reconstructed in the future are to incorporate best efforts to reduce surface ponding depths to below 0.30 m.

2.6 Provincial Climate Change Guidelines and Local Considerations

During the past three decades, there have been improvements to watershed management practices throughout the province that require a greater understanding of the watersheds themselves. An evolution in stormwater management has occurred in Ontario and planners, engineers and designers must now address a broad set of technical issues relating to the development of land, including the maintenance of hydrologic processes and the mitigation of the observed and forecasted impact of climate change. Climate Change is now playing an important role in the design of new storm drainage infrastructure and the development of drainage solutions in existing developed areas that experience surface flooding. At this time, there is no clear and consistent provincial guidance with regard to climate change and the impact that the increased amount and intensity of rainfall has on the municipal storm drainage system. Due to a number of extreme rainfall events in the local region and across the province, a number of guidelines have been updated and adopted to adapt to climate changes both regionally and provincially.

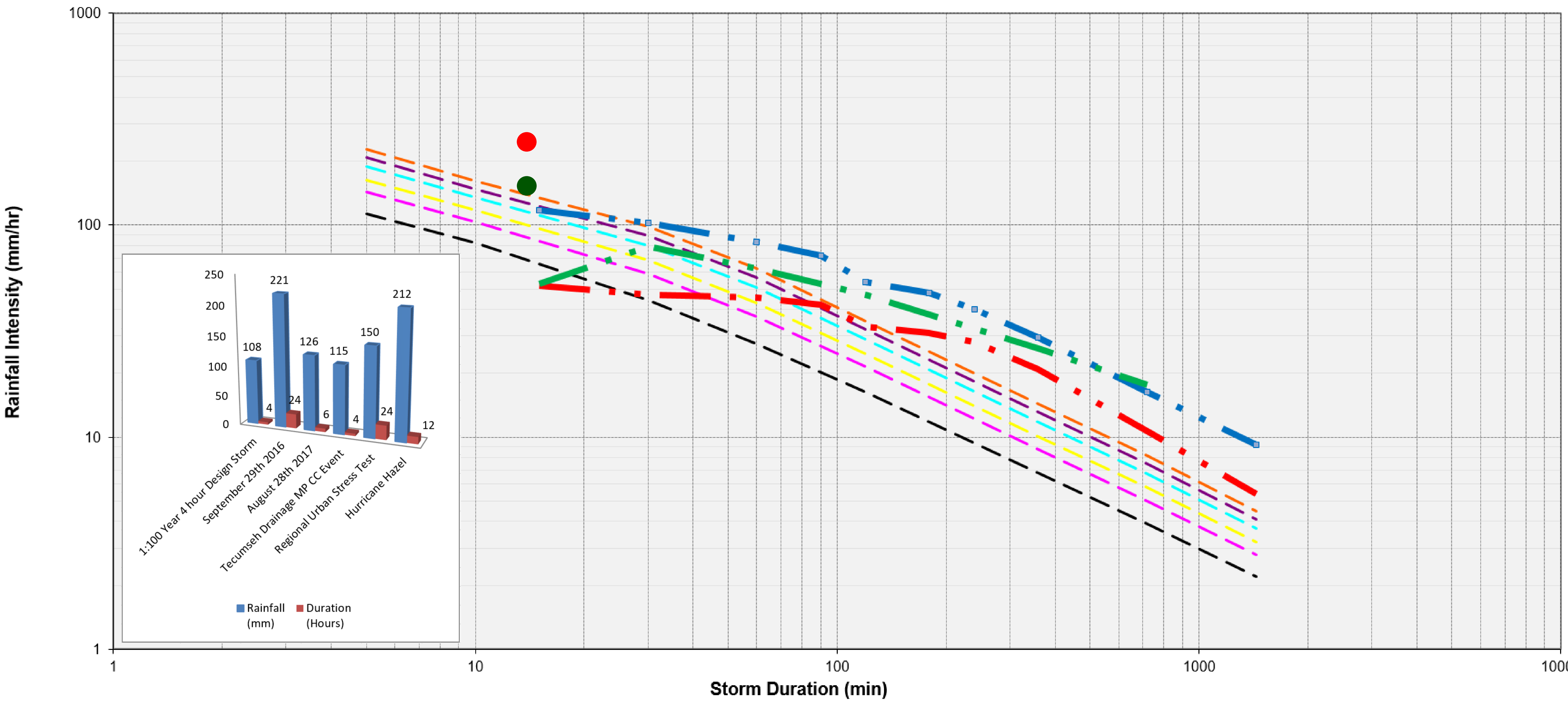
The Ministry of the Environment, Conservation and Parks (MECP) have issued Draft Low Impact Development Stormwater Management Guidelines and Recommended Minimum Runoff Volume Control Targets for Ontario which provide updates to the previous stormwater management guidance manual. It has become clear that there is a need to consider climate change in determining the resiliency and vulnerability of stormwater infrastructure under this changing trend in rainfall conditions.

2.6.1 Climate Change Design Events

In December 2018, the Essex Region Conservation Authority (ERCA), on behalf of the regions municipalities, released the Windsor/Essex Region Stormwater Management Standards Manual (December 2018). The standards manual was developed to identify technical standards for stormwater management design in the area and was developed to present best practises in the region to provide practical and prescriptive design criteria for local practitioners. The standards manual developed a rainfall event to stress test urban systems and was defined as 150mm of rainfall uniformly distributed over a Chicago 1:100 year 24 hour distribution with a max 15 minute intensity of 145 mm/hr. This equated to a uniform increase of 39% to the design storm event. The stress test analysis is not required to be designed to, but to strictly test the resiliency of the proposed storm infrastructure system. New developments are required to store the runoff volume from this event on-site.

The existing municipal storm drainage system throughout the study area was to be assessed for climate change and a decision making framework was therefore developed to determine the required level of service for recommended surface flooding solutions. The framework took into account the need for added resiliency based on a selected climate change event of a Chicago 1:100 year 4 hour storm event distribution + 40% incremental intensity. This rainfall event produced a total rainfall volume of 115 mm with a maximum 10 minute intensity of 241 mm/hr. Although the total rainfall volume is below the now published regional urban stress test identified above (115 mm compared to 150 mm), the climate change analysis chosen for this study identifies an event where the rainfall is falling for a shorter period of time (4 hours compared to 24 hours) at a significantly higher maximum intensity (241mm/hr compared to 145 mm/hr) over the respective storm duration.

Through a review of both climate change analysis options, the recommended surface flooding solutions were assessed against both the climate change analysis used within the solution decision framework (high intensity event) and the regional urban stress test (high rainfall volume event). Provided below in Figure 2.1 is a summary of both climate change events in comparison to the regional Windsor_A IDF curves and recent extreme rainfall events we have had in the past 5 years.



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

CLIMATE CHANGE AND HISTORICAL STORM EVENT COMPARISON
FIGURE 2.1

TECUMSEH STORM DRAINAGE MP CLIMATE CHANGE EVENT MAXIMUM INTENSITY (241 mm / hr)
REGIONAL URBAN STRESS TEST EVENT MAXIMUM INTENSITY (145 mm / hr)
1:2 - YEAR (WINDSOR IDF)
1:5 - YEAR (WINDSOR IDF)
1:10 - YEAR (WINDSOR IDF)
1:25 - YEAR (WINDSOR IDF)
1:50 - YEAR (WINDSOR IDF)
1:100 - YEAR (WINDSOR IDF)
SEPTEMBER 29TH 2016 EVENT
AUGUST 28TH 2017 EVENT
HURRICANE HAZEL

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MAP CREATED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

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PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

3.0 Background Review and Data Collection

3.1 Previously Completed Studies

The Town of Tecumseh has completed several studies since 1970 which assessed their stormwater drainage infrastructure throughout the Town (both private and public). These studies included detailed design of residential developments, existing and future condition assessments of pump stations and their respective service areas, as well as a full condition assessment of all municipal pump station facilities to confirm their condition and prioritize rehabilitation and replacement efforts.

A background investigation report was completed to summarize key findings from these previous stormwater servicing and stormwater management (SWM) studies. Each document was reviewed to confirm the respective study area, issues identified and solutions that were recommended. The following studies were reviewed as part of this Storm Drainage MP:

- The Village of St. Clair Beach Report on Storm Drainage (M.M. Dillon, January 1970);
- Tecumseh Hamlet Storm Drainage Study for the Township of Sandwich South (M.M. Dillon, June 1979);
- St. Clair Beach Stormwater Pump Study (M.M. Dillon, October 1983);
- Township of Sandwich South Master Drainage Plan (N.K. Becker and Associates, December 1987);
- Shawnee Road and Arbour Street Area Improvements Class EA (Dillon, September 2009);
- Town of Tecumseh MRSPA SWM Study Class EA ESR Report (Dillon, April 2010);
- Town of Tecumseh Sanitary Sewer Assessment Report (Dillon, May 2011);
- Town of Tecumseh East Townline Drain Hydrology and Hydraulic Study Report (Dillon, June 2012);
- Lakewood Park South Design Brief for Channel Design (Odan Detech, October 2014);
- Town of Tecumseh MRSPA Functional Servicing Report (Dillon, April 2015);
- Town of Tecumseh MRSPA SWM ESR Addendum Final Report (Dillon, April 2015);
- Town of Tecumseh St. Mark's and Scully (Edgewater) Storm Pump Stations – Review of Drainage Areas and Storm Servicing Alternatives (Dillon, August 2016);
- PJ Cecile (Kensington) Storm Pump Station – Review of Drainage Area and Contributing Flow (Dillon, September 2016); and
- Town of Tecumseh 2016 Pump and Metering Station Condition Assessment Report (Dillon, November 2016).

Part of the background report also includes an investigation of the September 28/29th, 2016 rainfall event, which provides the following information:

- Review of the regional IDF curves with respect to rainfall data beyond 2012;
- Statistical rainfall analysis on the September 28/29th, 2016 storm event; and
- Summary of the observed surface and basement flooding reports from the event.

General findings from the review of the previous reports identified above include:

- Areas with semi-urban roadway cross sections proposed to be reconstructed to a fully urban curb and gutter roadway and traditional storm sewer system require upgrades to the pump station outlet. The pump stations are recommended to be upgraded, at a minimum, based on the corresponding level of service of the storm drainage infrastructure;
- There is limited overland flow routes to a major system surface outlet for the majority of the Town area due to flat land topography; and
- Roadside ditches within residential areas with semi-urban cross sections have been known to either be filled in by residents or replaced with local sewers to reduce local surface drainage problems. In most cases, the size and slope of the existing sewers are insufficient to service the actual drainage areas. The absence of manholes also makes it difficult to carry out any regular maintenance program.

The full background document completed for this study is provided in Appendix A.

3.2 Study Area Topographic Survey

As part of the Storm Drainage MP, Dillon Consulting Limited retained Mosaic 3D(A Division of Geniarp Group), to complete a ground digital elevation map (DEM) of the study area through an aerial remote sensing method, known as Light Detection and Ranging (LiDAR). This form of surveying measures distance from an aerial craft to a target by illuminating the target with pulsed laser light and measuring the reflected pulses with a sensor. This was completed using a helicopter to determine the topographic elevations of the ground surface within the study area. The elevation map was then created as a consolidated DEM for the full study area.

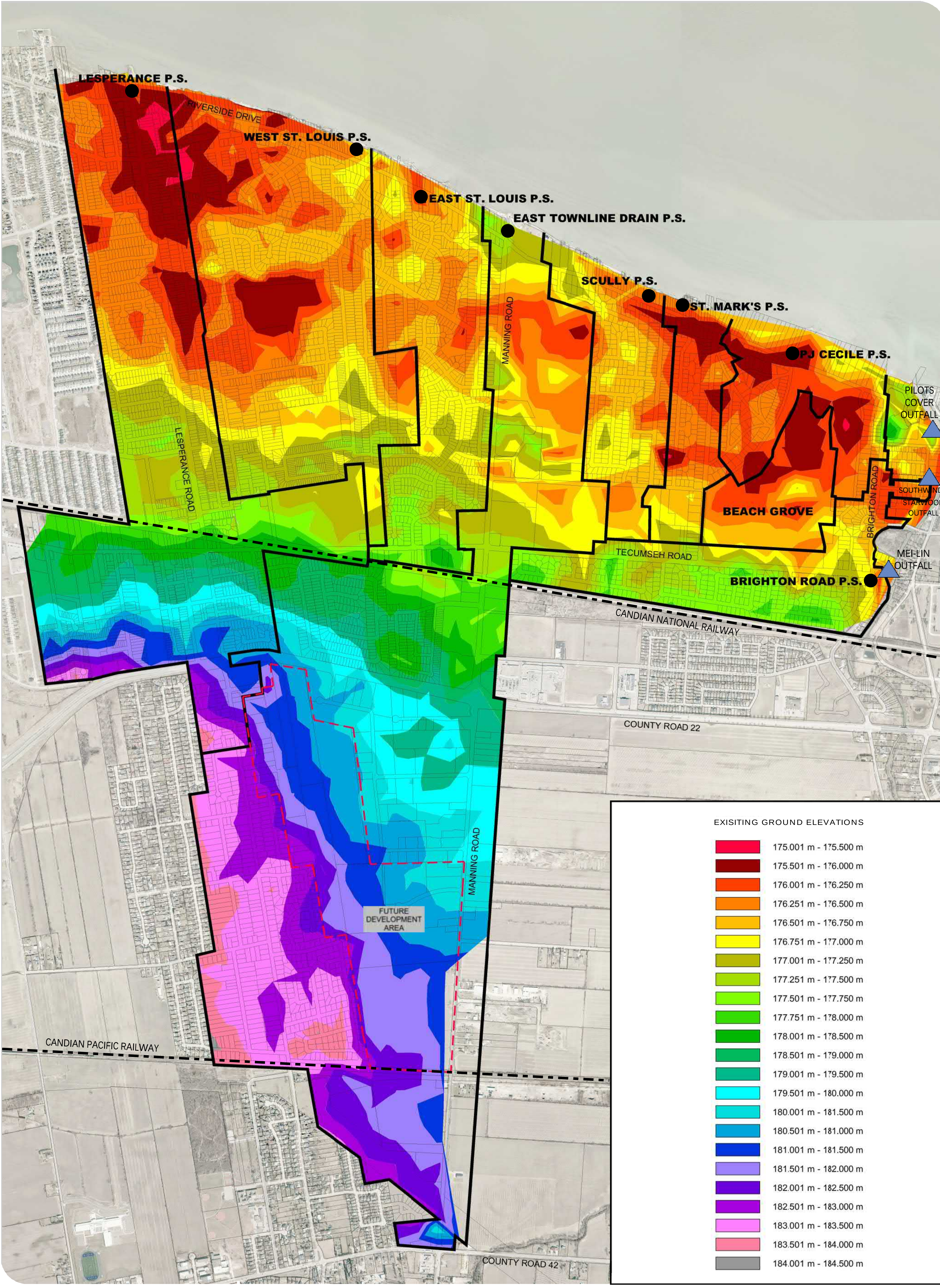
The DEM was used during the initial stages of the study to identify general surface flooding problem areas based on existing land topography and resident complaints. The DEM was also used in the PCSWMM model to create a 2-Dimensional mesh to represent the ground surface, including all overland flow routes (i.e. roadways and conveyance routes through private lands) and low lying areas to ensure a more complete understanding of surface flooding in the Town.

A visual representation of the LiDAR mapping has been provided on the topographic elevation heat map in Figure 3.1.

3.3 Soils Investigation

A soils inventory was completed for the study area using the Ontario Ministry of Agriculture, Food and Rural Affairs (OMAFRA) provincial data soil information. The soil maps, and their respective classifications of soil and land attributes, have been digitized and electronically assembled to produce a single digital file for the region. The digital version of this map was provided by Land Information Ontario and used in determining the hydrologic properties of the study area.

Based on a review of the OMAFRA soil classifications, the majority of the study area is primarily Brookston Clay, which is considered hydrologic “Type D” soil. This type of soil has a very low infiltration rate, which in turn produces a high runoff rate. Portions of the study area along the shoreline of Lake St. Clair show trace amounts of Wauseon soil, which is considered a hydrologic “Type C” soil. This type of soil has a low infiltration rate, which in turn results in a moderate to high runoff rate. A visual representation of the OMAFRA soils mapping for the study area has been provided in Figure 3.2.



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

STUDY AREA TOPOGRAPHIC LiDAR MAP

FIGURE 3.1

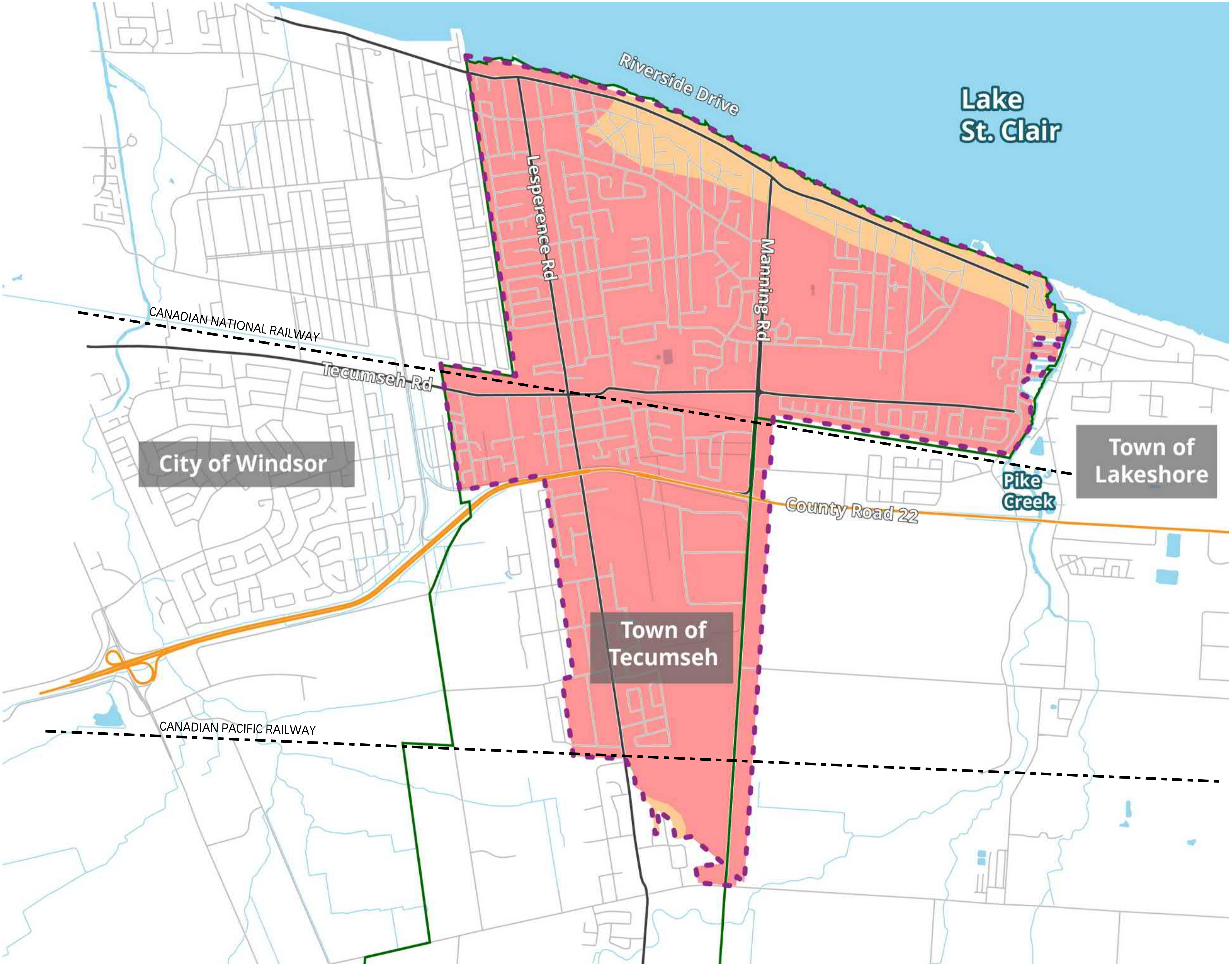
● PUMP STATION LOCATION (P.S.)
▲ GRAVITY OUTFALLS
PUMP STATION CATCHMENT AREAS
FUTURE DEVELOPMENT AREA
RAILWAY

MAP CREATED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019



TOWN OF Tecumseh ONTARIO · CANADA

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

OMAFRA SOILS MAP
FIGURE 3.2

HYDROLOGIC SOIL TYPE C
HYDROLOGIC SOIL TYPE D
STUDY BOUNDARY
RAILWAY
WATER COURSE
MUNICIPAL BOUNDARY
MINOR STREET
MAJOR STREET
HIGHWAY

SCALE 1:NTS

2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

DILLON CONSULTING

PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019

3.4 Land Use and Zoning

The study areas existing land use zoning was reviewed during the background investigation to assist with identifying areas where potential private stormwater management controls are currently in place. Due to the amalgamation of municipalities with the Town, the Town of Tecumseh is currently separated into 3 Official Plan Areas: Tecumseh, St. Clair Beach and Sandwich South. The Official Plan Land Use figures were provided by the Town of Tecumseh and 2017 aerial imaging was referenced to identify and confirm each land use category for existing and future uses. The land uses throughout the Town are provided below with the official Land Use figures provided in Appendix A.

3.5 Private Stormwater Management (Industrial, Commercial, Institutional Lands)

3.5.1 Residential Development On-Site Controls

Through field investigation and a review of as-built drawings, it was determined that several residential developments have oversized storm sewers and outlet restrictions to control outflow into the downstream storm trunk sewers. The following residential streets have controlled outlets to the downstream trunk sewers:

- Jason Court;
- Village Grove Drive/Tuscany Crescent;
- Calvary Crescent;
- Lessard Street;
- Chornoby Crescent/Westlake Drive;
- Papineau Court;
- Gauthier Drive/Valente Court;
- Gauthier Drive/Oliver Drive;
- Southfield Drive; and
- Carmelita Court.

The applicable SWM solution for each site was reflected in the model. The locations of the residential SWM controls incorporated in the PCSWMM model are illustrated in Figure 3.3.

3.5.2 Industrial/Commercial/Institutional Development On-Site Controls

Through the background investigation, it was determined that on-site quantity control exists for several existing Industrial, Commercial and Institutional (ICI) developments within the study area. The project team and the Town of Tecumseh reviewed all available as-built drawings and municipal building permit files to gather servicing drawings and SWM reports which may provide more information on the SWM strategy and design used for each site. A limited number of private property design files were found through the search due to the age of most of most ICI developments within the study area.

The following developments appear to have on-site quantity control measures in place:

- Lakewood Condominiums Phase 2 – 200 Manning Road, Tecumseh, ON;
- A.V. Graham Public School Parking Lot Reconstruction – 815 Brenda Crescent, Tecumseh, ON;
- Tecumseh EMS – 975 Lesperance Avenue, Tecumseh, ON;
- Apartment Building Complex – 1310 Lesperance Road, Tecumseh, ON;
- Ecole Saint-Antoine Parking Lot – 1317 Lesperance Road, Tecumseh, ON;
- Jamsyl Centre Mini Mall – 1606 Sylvestre Drive, Tecumseh, ON;
- Tecumseh Home Hardware – 1613 Lesperance Road, Tecumseh, ON;
- Commercial Development (McDonalds) – 1613 Manning Road, Tecumseh, ON;
- Parking Lot Expansion – 1620 Sylvestre Drive, Tecumseh, ON;
- Manning West Commercial Development (Golf Town Plaza) – 1695 Manning Road, Tecumseh, ON;
- Villa Pia Investments Commercial Development (Frank's Brewery) – 12002 Tecumseh Road, Tecumseh, ON;
- Commercial Development Addition – 13039 Tecumseh Road, Tecumseh, ON;
- Tecumseh Medical Centre Commercial Plaza – 13278 Tecumseh Road, Tecumseh, ON;
- BK Cornerstone Sales Office – 13405 Desro Drive, Tecumseh, ON; and
- Tecumseh Retirement Residence – 13500 Riverside Drive, Tecumseh, ON;
- New Commercial Plaza – 14306 Tecumseh Road, Tecumseh, ON; and
- Zehrs Tecumseh Plaza – Tecumseh, ON.

The locations of ICI on site SWM controls that are incorporated in the PCSWMM model are illustrated in Figure 3.3.

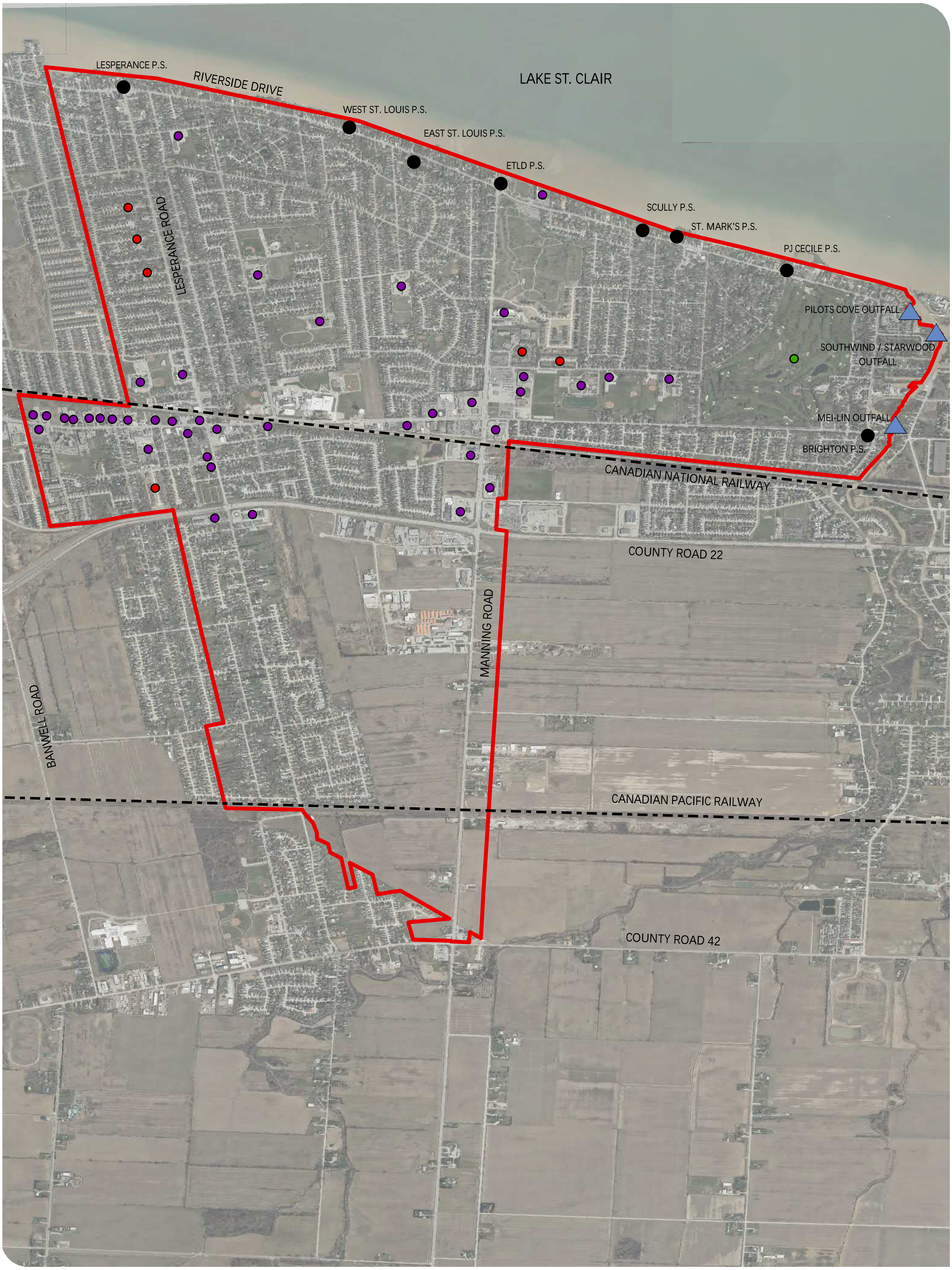
3.5.3 Residential Rear Yard Areas

Based on discussions with Town staff, only newer portions of the Town within the study area have rear yard catch basins to convey storm runoff from the rear yard to the municipal storm sewer system. The majority of the residential rear yard areas experience some surface ponding within depression storage until it can be infiltrated, evaporated and/or conveyed through overland flow routes based on private lot grading.

3.6 Existing Semi-Urban Roadway Cross Section Areas

The majority of the roadways in the study area have an urban roadway cross section, which includes a curb and gutter, roadway catch basins and storm sewer system. There are, however, a number of existing residential neighbourhoods within the study area which have semi-urban and rural roadway cross sections. For these roads, stormwater runoff is conveyed primarily through roadside ditches. In some cases, sub drain pipe systems have been implemented to supplement the capacity for the drainage system. Catch basins serve to connect the two systems to an appropriate downstream outlet. These areas are highlighted in Figure 3.4 and include the following:

- Streets designated within the “Kensington Dish” Area;
- Streets designated within the “Coronado Dish” Area;
- Arlington Boulevard;
- St. Marks Road;
- Edgewater Boulevard;
- St. Anne Area including:
  - St. Anne Street between North Pacific Avenue and Gouin Street.
  - Portions of North Pacific Avenue, Intersection Road, Maisonneuve Street and Gouin Street within the study area.
- Tecumseh Road; and
- Pentilly Road.



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

MODELLED EXISTING SWM CONTROLS: RESIDENTIAL/ PRIVATE ICI LANDS
FIGURE 3.3

TOWN OF Tecumseh ONTARIO · CANADA

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SCALE 1:NTS

2

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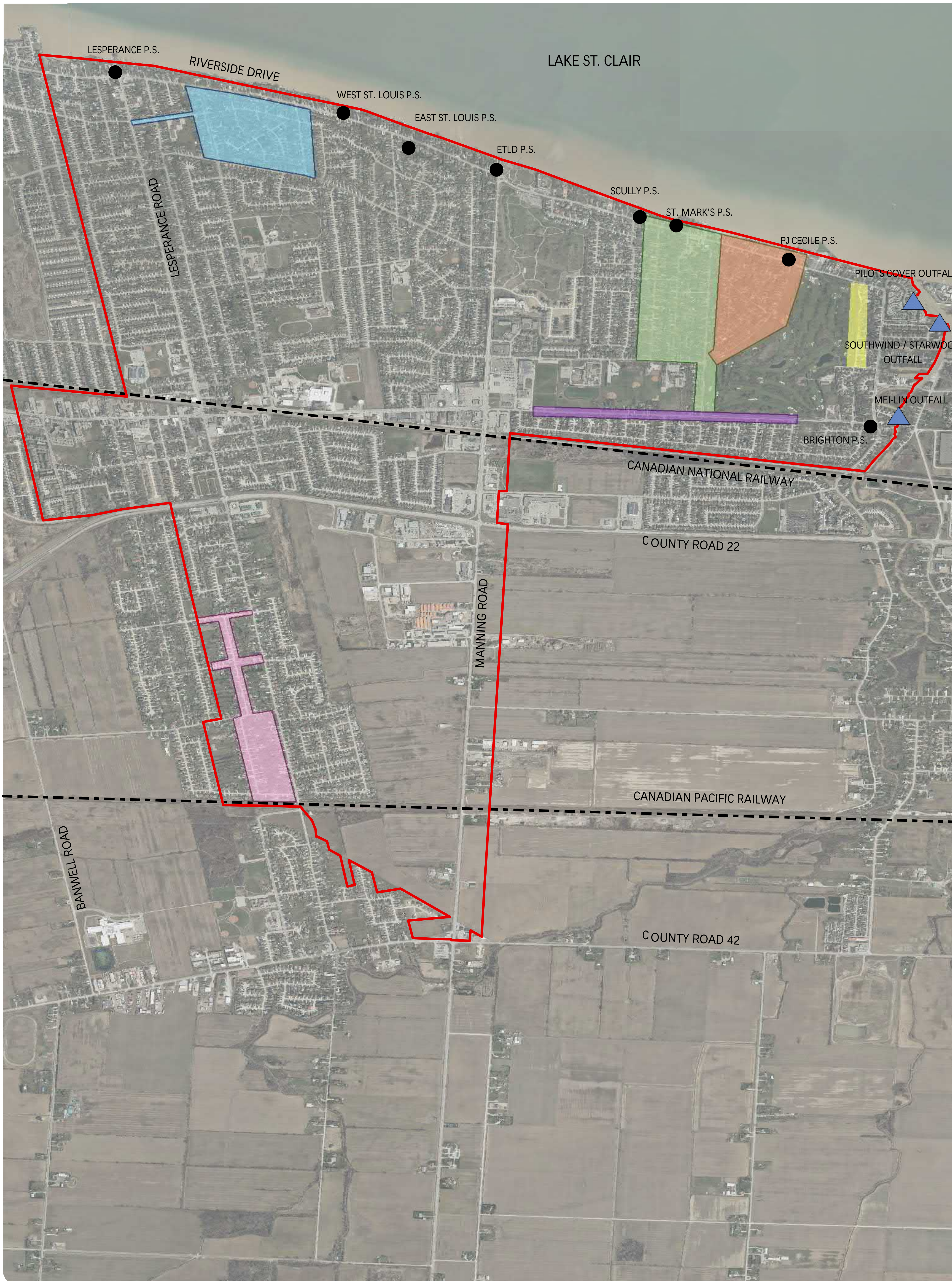


TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

STUDY AREA ROADWAYS WITH SEMI-URBAN CROSS SECTIONS
FIGURE 3.4

SEWER GRAVITY OUTFALL	TECUMSEH ROAD	EDGEWATER/ ST. MARKS/ ARLINGTON AREA
PUMP STATIONS (P.S.)	ST. ANNE AREA	KENSINGTON DISH AREA
STUDY AREA	CORONADO DISH AREA	
RAILWAY	PENTILLY ROAD (ALREADY RECONSTRUCTED)	

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

3.7 Low Impact Development Design for Stormwater Management

Low Impact Development (LID) measures are defined by the United States Environmental Protection Agency (U.S. EPA) as "systems and practices that use or mimic natural processes that result in the infiltration, evapotranspiration or use of stormwater in order to protect water quality and associated aquatic habitat". The U.S. EPA further clarifies that applying LID measures entails working with nature to manage stormwater as close to its source as possible.

The Ontario Ministry of Environment, Conservation and Parks (MECP) produced a LID Stormwater Management Guidance Manual (Draft) in 2017 for the design, construction and maintenance of LIDs. This manual was intended to act as a companion to the Stormwater Management Planning and Design Manual (MECP, 2003), which advocated a "treatment train" approach to treat and store stormwater runoff. This approach encouraged the use of lot-level and conveyance controls, along with end-of-pipe measures, to manage stormwater runoff. The principles of Green Infrastructure (GI) and LID promote a shift towards an ecosystem-based water balance approach in lieu of rapid conveyance using sub-surface pipes and end-of-pipe controls. Table 3.1 outlines a list of known LID Best Management Practices (BMPs) and their applications, which have been reproduced from Table 1.5.1 in the LID Stormwater Management Guidance Manual [Draft] (MECP, 2017).

Table 3.1: LID BMPs

<table>
<tr><th></th><th>Source Control</th><th>Conveyance Control</th><th>Notes</th></tr>
<tr><td rowspan="12">LID BMPs</td><td>Rainwater Harvesting</td><td></td><td></td></tr>
<tr><td>Green Roofs</td><td></td><td></td></tr>
<tr><td>Downspout Disconnection</td><td></td><td></td></tr>
<tr><td>Soakaways, Infiltration Trenches and Chambers</td><td></td><td rowspan="9">Suitable for use within the road right-of-way</td></tr>
<tr><td>Bioretention (a.k.a Rain Gardens)</td><td></td></tr>
<tr><td>Vegetated Filter Strips</td><td></td></tr>
<tr><td>Permeable Pavements</td><td></td></tr>
<tr><td colspan="2">Enhanced Grass Swales (a.k.a. Vegetated Swales)</td></tr>
<tr><td colspan="2">Dry Swales (a.k.a Bioswales)</td></tr>
<tr><td></td><td>Perforated Pipe Systems</td></tr>
<tr><td>Tree BMPs</td><td></td></tr>
<tr><td>Soil Amendments</td><td></td></tr>
</table>

The LID Stormwater Management Guidance Manual [Draft] (MECP, 2017) defines Runoff Volume Control Targets (RVC_T) for new development, redevelopment, infill-development, re-urbanization, linear infrastructure and retrofits in Ontario. It requires limiting of runoff volume to 10% (or less) of total rainfall volume. This corresponds to control of 90% of the average annual rainfall, determined through the use of the 90th percentile event. The Recommended future RVC_T for the Windsor region is specified to be 32 mm based on the 90th percentile rainfall event. The provincial guideline with this specified design criteria is currently in draft format and is recommended to be further reviewed during detail design for new developments and municipal road and storm sewer replacement projects. This study assessed a sample area with the incorporation of LID's and is further discussed in Section 12.3 of the report.

4.0 Existing Condition Summary

4.1 Storm Sewers (Minor System)

As identified in Section 2.3.1, the majority of the municipal storm sewer system in the Town of Tecumseh was constructed from 1950 – 1990. The majority of the Town of Tecumseh's storm sewer system was designed for a 1:2 year level of service for the conveyance of storm runoff for more frequent storm events. Within the study area, the storm sewer system consists of the following:

- Approximately 67.0 km of storm sewer greater than or equal to 450 mm in diameter; and
- Approximately 40.4 km of storm sewer less than 450 mm in diameter.

Some areas within the study have semi-urban or rural cross sections, as identified in Section 3.6, where roadside ditches are used as the means of runoff conveyance. These areas are known to experience temporary surface ponding within the roadside ditches during frequent and infrequent storm events. This is due to a number of factors including sediment buildup within the open ditches and restricted outlet capacity.

4.2 Overland Flow (Major System)

The main form of overland flow routes are municipal roadways that are typically designed to convey flows during more infrequent storm events once the minor system capacity has been exceeded. As outlined in Section 2.3.2, a typical roadway has a curb and gutter and consists of "sawtooth" grading to create sags (i.e. roadway low points) with catch basin inlets positioned on a continuous grade or within the sag of the roadway gutter to direct roadway runoff to the storm sewer system. Due to the flat topography throughout the study area, major system flow routes are not always clearly evident, and in some cases direct overland flows to local, low lying areas. During larger, more infrequent storm events, storm runoff is temporarily stored on the roadway until the storm sewer system has available capacity to convey flows to the respective outlets.

Based on feedback from the Town staff and Tecumseh residents, it has been identified that surface ponding along roadways occur during more frequent storm events in lower lying areas along the northwestern boundary in the Lesperance and West St. Louis storm pump station service areas This coincides with the lower lying areas identified within the LiDAR. Surface ponding during more frequent storm events is also prevalent in areas of semi-urban cross sections. These areas of semi-urban and rural roadway cross sections convey storm runoff through open roadside ditches which may also overflow into lower lying private property. This includes the areas within the Kensington Dish, St. Anne's Street area and along Lacasse and Little River Boulevard.

4.3 Topographic Assessment

As identified in Section 3.2, a ground digital elevation map (DEM) of the study area was developed through an aerial remote sensing method called Light Detection and Ranging (LiDAR) to determine the topographic elevations of the ground surface within the study area.

During the initial existing condition review, the DEM was used to identify the general surface flooding problem areas based on existing land topography and resident complaints. The study area is relatively flat, falling from south to north. Lower lying areas which were identified to be susceptible to surface flooding were noted along the northwest (from Lesperance Road to Lacasse Blvd.) and northeast (from Edgewater Blvd. to Brighton Road). Based on a review of the DEM, potential areas of surface flooding were also noted along the central (Lemire and Lanoue Street) and south end (St. Anne's street area) of the study area where barrier land forms, such as rail lines, and collector and arterial roadways could limit overland flow conveyance and result in localized surface ponding.

The visual representation of the existing topography has been illustrated in the LiDAR Mapping Figure 3.1.

5.0 Modelling Methodology and Development

5.1 Modelling Platform

Hydrologic and hydraulic modelling for both existing and future conditions was undertaken using the PCSWMM 2017 software. PCSWMM is a modelling software for stormwater, wastewater, and watershed drainage systems that includes a graphical user interface (GUI) for the United States Environment Protection Agency's Stormwater Management Model (EPA SWMM).

The model was used to simulate the existing flow conditions of the minor (sewers) and major (overland) systems. The minor system was modelled using a 1-Dimensional (1D) linear model network, while the major (overland) system was modelled using a 2-Dimensional (2D) approach. This integrated 1D-2D approach was used to model the ground surface in 2D free surface flow, while still using the 1D Saint Venant equations for storm sewer and municipal drain flow. The 2D characteristics of the overland flow network are defined by the underlying DEM model layer.

5.2 Hydrology

For the model development component of the project, a representation of the watershed hydrologic parameters for the study area were calculated based on accurate impervious data extraction from aerial mapping, review of local soil types and best engineering judgement of the study area conditions. The hydrologic parameter calculations were undertaken to more accurately improve the reliability of the surface runoff contributing to both the minor (sewers) and major (overland). The hydrology component was developed by determining the following:

- Review of current drainage conditions and subcatchment delineation; and
- Subcatchment land use parameters.

5.2.1 Existing Drainage Conditions and Subcatchment Delineation

To determine the nature of stormwater runoff, an understanding of the conditions is required. Subcatchments are areas of land that drain to a respective inlet to a storm drainage system, which could be a catch basin or a point at which concentrated flows leave a larger area. Detailed subcatchment delineation was completed through an extensive review of the current storm drainage network (storm sewers for roadway catchment areas), current municipal drainage reports and the DEM generated from the LiDAR mapping (rural lumped catchment areas) and pump station reports (overall pump station service area catchment areas). A review of the existing topographic LiDAR assisted in delineating the subcatchments to a catch basin level.

Locations of roadway manholes and catch basins were compared with respect to roadway high and low points to determine contributing runoff. Where catch basins were on continuous grade, roadway catchments were delineated from manhole to manhole. Where catch basins were at sag points, roadway catchments were delineated based on roadway highpoints.

Private property drainage areas were delineated based on the following:

- Review of topographic LiDAR to determine existing overland flow routes which would allow runoff to enter the roadway network and contribute to the catch basin inlets;
- Private storm site connections based on as-built drawings, field investigations and proximity to manholes;
- Where no SWM controls were determined for private ICI lands, the site was assumed to connect to the upstream node of the sewer segment and assumed to convey flows uncontrolled into the municipal storm system; and
- Lands adjacent to municipal drains connected to the open drainage system based on existing overland flow routes identified through the DEM.

Subcatchment areas upstream of the storm sewer network generated within the baseline model were lumped together to a single catch basin inlet.

5.2.2 Subcatchment Parameters

Land use parameters were identified for the subcatchments to setup the hydrologic conditions of the study area. The primary hydrologic parameters for each subcatchment area included the surface area, flow length and equivalent width, average ground slope, percent imperviousness, and infiltration losses. The following is a summary on how each parameter was determined.

Subcatchment Area

Delineation of the subcatchments completed within GIS to accurate represent the catchment area in hectares.

Subcatchment Flow Length and Equivalent Width

Calculated flow lengths were established based on LiDAR DEM, while subcatchment width parameters were calculated based on the area of the catchment divided by the flow length.

Average Ground Slope

An average slope of 0.25% was used throughout the study area. This assumption was determined based on review of average roadway slopes and private property grades from the LiDAR DEM.

Percent Imperviousness

Impervious area shapefiles were developed by delineating the areas of imperviousness (i.e. roadways, sidewalks, building footprints, driveways and backyard hard surfaces) based on 2017 aerial mapping. Through a review of typical residential lots within the study area, a calculation of approximately 20% of the impervious area for each subcatchment was identified to not be directly connected to the storm sewer system. This includes backyard hard surface areas (porches, pathways) and/or rooftops that are not directly connected to the municipal storm sewer system. The 20% of the impervious area for each subcatchment was simulated in the model to route through grassed area prior to getting into the storm sewer system.

Using GIS software and intersection analysis through the two shapefiles, the impervious area shapefile was merged with the subcatchment delineation shapefile. The area of imperviousness and percent impervious values for each subcatchment were then accurately calculated.

Infiltration

To represent surface infiltration, the curve number (CN) method was selected at the onset of the project, which has been widely accepted throughout the region. This parameter accounts for the initial accumulation of rainfall which represents interception, depression storage, and infiltration before the start of runoff begins. Based on the Hydrologic Group D Soil for the study area, an average curve number of 85 was used in the model to represent the pervious spaces in the urbanized area. This curve number was generated based on the range of residential lots ranging from 0.05 ha to 0.20 ha in size with impervious values ranging from 25% to 65% (Runoff Curve Numbers for Urban Land Uses, SCS – 1986)

Other Hydrologic Attributes

Based on a review of the study area conditions, the completed LiDAR topographic mapping and regional SWM guidelines, the following hydrologic attributes were determined for the study area subcatchments:

- Impervious N Value = 0.014 ;
- Pervious N Value = 0.35;
- Impervious Depression Depth Storage = 1.57; and
- Pervious Depression Depth Storage = 4.67.

Based on the model for this study incorporating a 2-dimensional component to take into account the existing topography of the land, surface depression storage within the subcatchments were already taken into consideration. The depression storage depths stated above were therefore used and are lower than what is identified in the recently released Windsor/Essex Region Stormwater Management Standards Manual (December 2018).

The input files for the existing condition PCSWMM model are included in Appendix B. Output files are available on request.

5.3 Hydraulics

The storm sewer drainage network was incorporated in the model based on available information. The components of the storm drainage system include:

- Storm sewers;
- Manholes;
- Catchbasins;
- Municipal Drains;
- Private Storm Connections;
- Pump Stations; and
- Gravity Outfalls.

A visual representation of the storm drainage network is provided in Appendix B.

5.3.1 Storm Sewers, Manholes and Catch Basins

For the model development of the existing storm drainage system, the project team used the GIS data sets provided by the Town for storm sewers, storm sewer manholes, catch basin inlets and leads. The information included pipe diameters, invert elevations, pipe lengths, and maintenance hole ground elevations. Catch basin lead sizes ranged from 150mm diameter to 300mm diameter based on the Town's GIS data provided. The model buildout included the following:

- Storm sewers greater than or equal to 450 mm in diameter;
- Storm sewers less than 450 mm in diameter where warranted;
- Respective manholes and catch basins for the imported storm sewer system; and
- Catch basin lead pipes from the catch basin to the municipal storm sewer system.

Modelling of the storm drainage network for the Storm Drainage MP incorporated all 67 km of storm sewers larger than 450 mm in diameter. Approximately 16.5 km of the 40.40 km of storm sewers less than 450mm in diameter were modelled where known residential surface flooding has been reported.

The storm sewers were represented as conduits and the storm manholes and catch basin were represented as junctions within the model. The catch basin leads were considered the governing inlet restriction into the storm sewer system and were modelled to represent the catch basin connections to the storm sewer. The representation of catchbasin leads in the model was development to more accurately reflect the inlet rates from storm runoff along the surface into the storm sewer system. The LiDAR bare earth DEM was used to determine storm maintenance hole and catch basin rim elevations.

To confirm the accuracy of all data imported into the baseline model, an extensive quality check was completed, and data gaps were filled in through review of as-built information and field survey investigations.

5.3.2 Municipal Drains

All municipal drains identified within Section 2.1.3 were incorporated in the model using either 1-Dimensional conduits or 2-dimensional mesh. Conduits can represent both closed pipes and open channels in PCSWMM. The conduits representing each of the municipal drains noted below were open channel and assigned the respective in-bank cross sections and in-stream bottom of drain elevations based on the following information:

- *East Townline Drain (1-D):* In-field survey collected from previous municipal drainage studies and review of previous municipal drainage reports;
- *Baillargeon Drain (1-D):* In-field survey collected from previous municipal drainage studies and review of previous municipal drainage reports;
- *Cyr Drain (1-D):* In-field survey collected from previous design studies and review of previous municipal drainage reports;
- *Antaya Drain (1-D):* Design information taken from previous municipal drainage reports; and
- *Manning Road Drain (2-D):* Drain attributes extracted from the 2017 Topographic LiDAR.

5.3.3 Rear Yard Storm Connections

As identified in Section 3.5.3, the majority of the study area residential rear yard areas do not have a storm sewer connection. The Town agreed that residential rear yard areas should be assumed to be connected to the municipal storm sewer system in both existing and future conditions models. Although portions of the Town do not currently have rear yard residential connections, this assumption would provide conservative results under existing conditions when determining surface flooding problem areas.

Under future conditions modelling and the development of surface flooding solutions, the assumption of connected rear yard areas to the municipal storm sewer system allow for a factor of safety in proposed solutions and potential municipal storm infrastructure needs where all residential properties are constructed with rear yard catch basins that connect to the municipal storm sewers.

5.3.4 Private Stormwater Management (Residential, Industrial, Commercial, Institutional Lands)

Based on the areas identified in Figure 3.3, the SWM design for each site was built out in the PCSWMM model through the documented as-built drawings or design reports. For private developments with no available stormwater management reports, the minor system from the development was restricted to the known size of the outlet pipe into the municipal storm sewer system. The model included known information regarding the private storm sewer system such as pipe size, invert elevation, and known on-site controls. For private surface storage or where no SWM design information was found, the LiDAR topographic mapping was used to approximate available surface ponding areas. If the storm sewer size for the site outlet was unknown, the development was assumed to release uncontrolled flows into the municipal storm sewer system.

5.3.5 Pump Stations

As part of the background investigation, an assessment was completed to determine the existing pump station capacities of the storm pump stations within the study area. This included the following:

- Field investigation for each of the eight (8) pump stations to determine the current operating characteristics, including lead, duty and standby orientation, on/off elevations, and to confirm the operating process for each station;
- Review of all available as-built information, including drawings, design reports and operations manuals;
- Review of available pump curves; and
- Spaans Babcock assessment completed in 2018 to confirm the theoretical capacity of the screw pumps in the Lesperance, West St. Louis and East St. Louis pump stations based on measurements of the existing pumps.

Table 5.1 summarizes the operating characteristics for each storm pump station in relation to the individual pumps. Note that the screw pump capacities were based on the Spaans Babcock assessment for the Lesperance, West and East St. Louis pump stations. The pump station data provided below was incorporated into the model.

Table 5.1: Existing Pump Station Operating Characteristics

Station	Town Pump #	Individual Pump Capacity (m^3/s)	On Elevation (m)	**Off** Elevation (m)	Comments
Lesperance Pump Station *(Firm Capacity of 3.143 m^3/s)*	1	1.740 m^3/s	173.20	172.20	Lag Pump #2
	2	1.740 m^3/s	172.20	171.20	Lag Pump #1
	3	1.403 m^3/s	171.20	169.90	Lead Screw Pump
West St. Louis Pump Station *(Firm Capacity of 1.690 m^3/s)*	1	1.690 m^3/s	170.11	169.33	Lead Screw Pump
	2	1.690 m^3/s	170.62	169.68	Lag Screw Pump
East St. Louis Pump Station *(Firm Capacity of 3.38 m^3/s)*	1	1.690 m^3/s	172.20	171.79	Lead Screw Pump
	2	1.690 m^3/s	172.65	172.13	Lag Screw Pump #1
	3	1.690 m^3/s	173.26	170.79	Lag Screw Pump #2
Manning ETLD Pump Station *(Firm Capacity of 7.47 m^3/s)*	1	2.490 m^3/s	173.38	173.10	Larger pumps run on a Lead-Lag System with 1 Lead, 2 Lag and 1 Standby
	2	2.490 m^3/s	172.75	172.00	
	3	2.490 m^3/s	173.38	173.10	
	4	2.490 m^3/s*	-	-	
	5	0.125 m^3/s	172.60	172.00	Auxiliary Pumps rotate between Lead and Standby
	6	0.125 m^3/s*	-	-	
Scully (Edgewater) Pump Station *(Firm Capacity Approximately 0.794 m^3/s)***	1	0.397 m^3/s	174.50	171.90	Pumps switch between Lead and Lag
	2	0.397 m^3/s	174.96	171.91	
	3	Unknown*	-	-	Third pump used for emergencies.
St. Mark's Pump Station *(Firm Capacity Approximately 0.347 m^3/s)***	1	0.347 m^3/s	173.54	172.62	Pumps switch between Lead and Lag
	2	0.347 m^3/s*	-	-	
PJ Cecile (Kensington) Pump Station *(Firm Capacity Approximately 0.397 m^3/s)***	1	0.397 m^3/s	174.10	171.91	Lead Pump
	2	0.397 m^3/s	174.96	171.91	Lag Pump
Brighton Road Pump Station *(Firm Capacity of 2.325 m^3/s)*	1	0.750 m^3/s	171.94	171.24	Larger pumps run on a Lead-Lag System with 1 Lead, 2 Lag and 1 Standby
	2	0.750 m^3/s	171.94	171.24	
	3	0.750 m^3/s	171.94	171.24	
	4	0.750 m^3/s*	-	-	
	5	0.075 m^3/s	170.44	169.55	Auxiliary Pumps rotate between Lead and Standby
	6	0.075 m^3/s*	-	-	

** Pump station operating elevations dependent on switching of lead/lag pump or manually turned on based on required standby.*

*** Firm capacity is unknown and has been approximated based on capacity of inlet and outlet storm sewers to and from the pump station.*

5.3.6 Storm Gravity Outfalls

A total of three (3) storm gravity outfalls are located along the north eastern boundary of the study area and discharge runoff into Pike Creek, which discharges to Lake St. Clair. Development of the PCSWMM model for each gravity outlet location included an assessment of historical high water elevations in Lake St. Clair to establish boundary conditions at each outlet, as required to simulate the impact of the tailwater conditions on the municipal storm sewer system and roadway surface flooding.

Through discussions with the Essex Region Conservation Authority (ERCA) and a review of the Environment Canada and NOAA historic water level data, several water levels were considered as a boundary condition at each of the gravity outfalls, as follows:

- ERCA Regulated Level @ Tecumseh Gauge (Shoreline Management Plan, N.K. Becker, 1986) – Lake St. Clair at Pike Creek = 176.44 m (578.90 ft);
- Environment Canada All-Time Average – Lake St. Clair = 175.16 m;
- Environment Canada All-Time Maximum – Lake St. Clair = 175.93 m;
- Environment Canada 2008 – 2017 Monthly Average – Lake St. Clair (Environment Canada) = 175.27 m;
- NOAA (Windmill Point, MI) Historic 100 Year Maximum Monthly Mean – Lake St. Clair at the mouth of the Detroit River = 175.90 m; and
- NOAA (St. Clair Shores, MI) Historic 100 Year Maximum Monthly Mean – Lake St. Clair = 175.97 m.

Based on a review of the historic Lake St. Clair water levels, it was determined that the ERCA governing water level of 176.44 m would be used within the model to simulate the boundary conditions at each of the gravity outfalls. This level was chosen due to the long term water levels in Lake St. Clair being high and water levels expected to rise in the future.

5.4 2-Dimensional Ground Surface Development

The major overland drainage system was of particular concern to the Town as it relates to surface flooding during major storm events. In order to better understand the existing conditions and propose reasonable solutions, a 2D mesh was created based on the DEM data collected during the initial stages of the study.

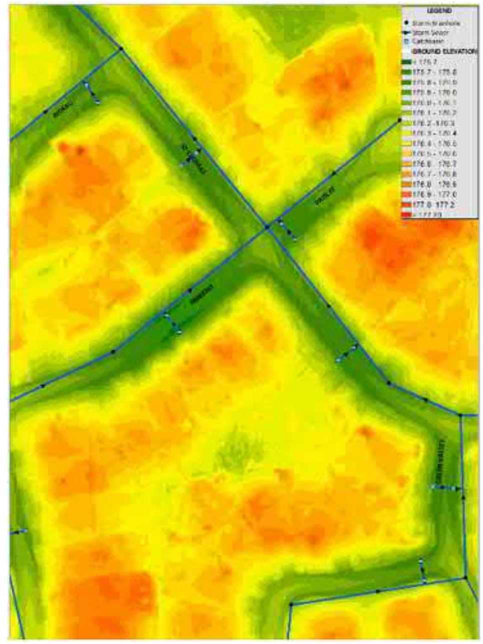
This 2D mesh was used to more accurately represent the major system flow within the municipal road right-of-way and through private properties. The 2D analysis of the major system for this study included the examination of surface ponding depths within depressed areas, localized roadway low points and ground surfaces outside of the municipal right-of-way.

Modelling the major and minor systems required combining the results from 1D and 2D model elements. The elements are connected at key locations to allow for the transfer of flow between the two elements, in particular ensuring that any flow that exceeds the capacity of the minor (1D) network can flow overland to the major (2D) system.

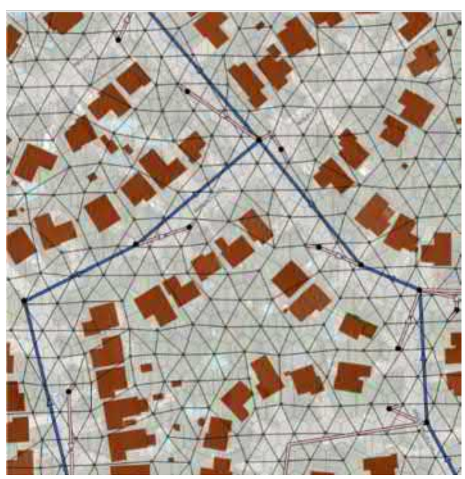
The model will dynamically compute overland flow routing and surface ponding in the 2D mesh throughout the duration of the simulated storm event. Obstructions were created within the 2D mesh to represent buildings based on building footprint shapefile.

Figure 5.1 illustrates how the DEM and 2D mesh elements connect with the 1D portions of the PCSWMM model to generate a fully integrated 1D/2D model

Municipal Storm Infrastructure Example



Storm Infrastructure with Tecumseh DEM



Developed 1D/2D PCSWMM Model with Municipal Storm Infrastructure Connected with 2D Ground Mesh w/Obstructions

Figure 5.1: Development of 2-Dimensional PCSWMM Mesh

5.5 Design Storms

The Chicago design storm distribution was used for the model simulations, and is consistent with the recently completed Windsor/Essex Region Stormwater Management Standards Manual (December 2018). The Chicago distribution is considered the preferred distribution when evaluating conveyance capacity of urban drainage systems. The Chicago design storm distributions were developed using the City of Windsor Airport Short Duration Rainfall Intensity-Duration-Frequency Data obtained from Environment Canada.

The following traditional design storms and respective duration and intensity time steps were used for the model simulations for both existing and future development conditions:

- Chicago 2 –year 4 hour event (10 minute time step);
- Chicago 5 –year 4 hour event (10 minute time step);
- Chicago 10 –year 4 hour event (10 minute time step); and
- Chicago 100 –year 4 hour event (10 minute time step).

5.5.1 Climate Change Considerations

A key component of the study was to consider climate change adaptation in the decision making process in the development of resilient solutions. As discussed in Section 2.6.1, two climate change events were reviewed:

1. *High Intensity Climate Change Event:* Chicago 1:100 year 4 hour storm event distribution + 40% incremental intensity. This rainfall event produced a total rainfall volume of 115 mm with a maximum 10 minute intensity of 241 mm/hr.

2. *High Volume Climate Change Event:* 150mm rainfall event with a 15 minute time step, representing a 39% increase in volume uniformly distributed across the rainfall event, as compared to the Windsor Airport 1:100 year 24-hour rainfall of 108mm. Maximum intensity of 145 mm/hr.

The existing and future conditions model have been simulated under both climate change storm events above. The high intensity climate change event was used first to design the recommended surface flooding solutions and the high volume climate change event was then used to validate and confirm the design.

6.0 Flow Monitoring and Rain Gauge Analysis

6.1 Overview

A flow monitoring program was developed to refine the original calculated watershed hydrologic values within the model. The refined model was developed to more accurately assess storm runoff and antecedent conditions in the study area during smaller, more frequent storm events. Based on the flow monitoring completed for the study areas storm sewer system, the refined model took into consideration any loss of runoff leaving or not entering the storm sewer system based on the majority of existing residential lands foundation drain and/or rooftops connected to the sanitary system, as identified in the earlier studies completed by the Town for the municipal sanitary system.

To further refine the model, confirming the system's responses to rain events is required. A flow monitoring program was implemented by the project team, along with AMG Environmental, to provide recorded hydrograph flow data for the subsequent model calibration and validation of the minor system. A total of two rain gauges were installed, along with one existing rain gauge in order to record variations in rainfall across the study area.

Rainfall events were selected to be similar to the regional 1:2 year 4 hour design storm and the model was to be calibrated to the respective observed flows.

The 7 month flow monitoring program began in April 1, 2018 and ended on October 31, 2018. The following tasks were completed:

- Supplied and installed 3 flow monitors in the storm sewer system;
- Supplied and installed 2 rain gauges within the study area;
- Conducted bi-weekly field verification and calibration of flow monitors;
- Collected data and stored the information in a web-based format;
- Collected and analyzed 7 months of flow monitoring data at each location; and
- Removed flow monitoring equipment at the conclusion of the monitoring program.

6.2 Flow Monitoring and Rain Gauge Location Selection

In order to confirm the performance of the storm sewer systems, the flow monitoring program was initiated at three locations within the sewer network. The flow monitoring devices were equipped with two depth sensors, two Doppler ultrasonic velocity sensors, one float switch, and an antenna. Sensor readings were taken every 5 minutes and data was stored on redundant servers and backed up daily using a managed backup system.

Flow monitors were installed at the following three (3) locations:

- FM01 on Lexham Gardens Road at the intersection of Estate Park Road;
- FM02 on Grant Avenue at the intersection of David Crescent; and
- FM03 on Lesperance Road between Evergreen Drive and Papineau Court.

The flow monitor locations are also summarized in Table 6.1.

Table 6.1 Summary of Flow Monitors in the Study Area

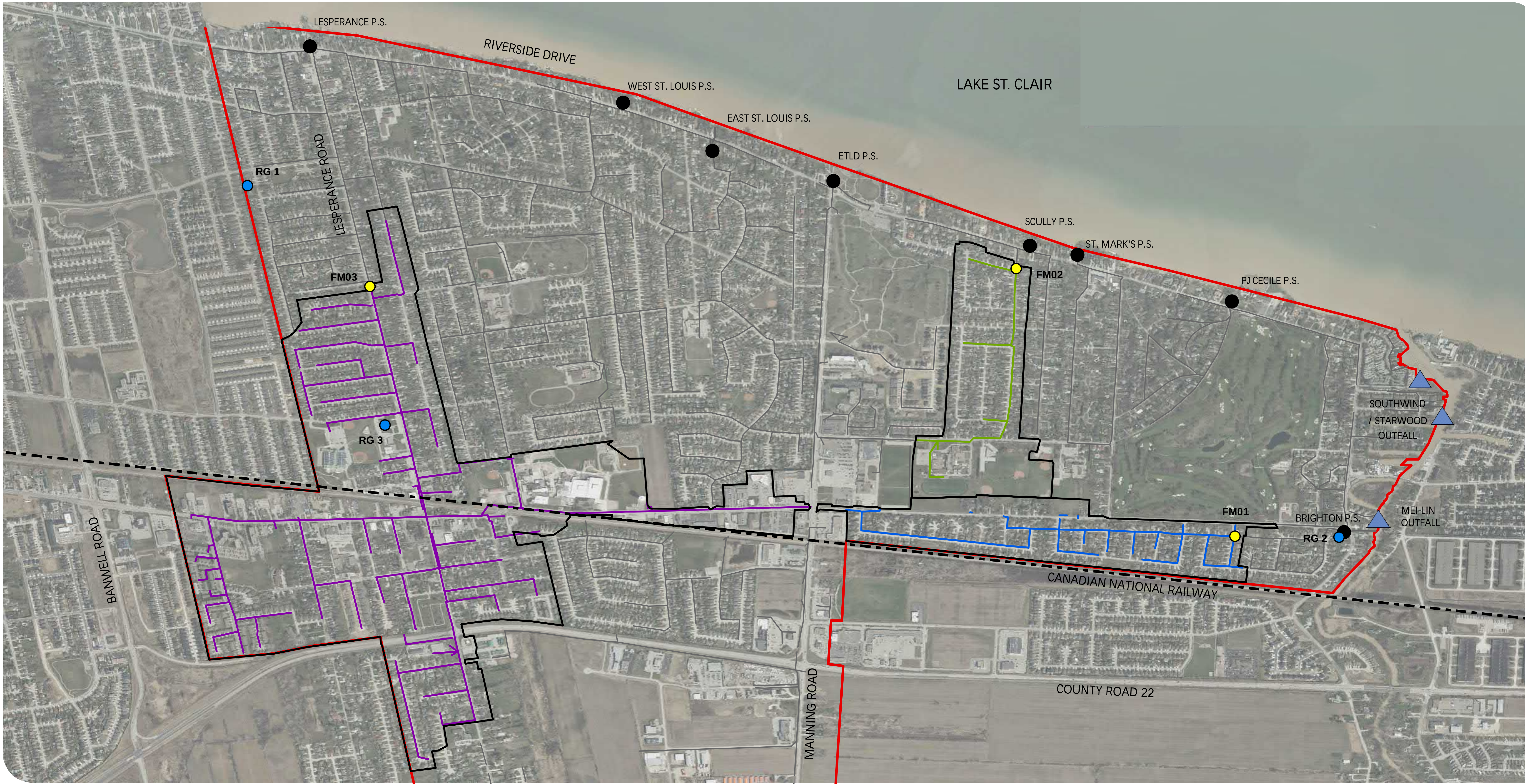
Monitor ID	Location	Manhole Number	Street Name	Sewer Size
FM01	Latitude:42.310985 Longitude: -82.850456	STM3912	Lexham Gardens and Estate Park	1200 mm
FM02	Latitude: 42.32269 Longitude: -82.86021	STM3236	Grant St.	1050 mm
FM03	Latitude: 42.322072 Longitude: -82.888801	STM73	Lesperance Rd	1950 mm

Two (2) rain gauges were installed, with a third existing rain gauge already in place at the following locations:

- Existing Rain Gauge 1 (RG01): Gauthier Sanitary Pump Station Location at the intersection of Little River Boulevard and Gauthier Drive;
- New Rain Gauge 2 (RG02): Installed on the Brighton Pump Station Rooftop at the intersection of Brighton Road and Tecumseh Road; and
- New Rain Gauge 3 (RG03): Installed on the Tecumseh Town Hall Rooftop at the intersection of Lesperance Road and McNorton Street.

The locations of each rain gauge and the flow monitoring locations are illustrated in Figure 6.1.

DILLON CONSULTING



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

FLOW MONITORING AND RAIN GAUGE LOCATION MAP
FIGURE 6.1

Town of Tecumseh Ontario · Canada

SEWER GRAVITY OUTFALL
PUMP STATIONS (P.S.)
FLOW MONITOR LOCATIONS
RAIN GAUGE LOCATIONS
STORM SEWERS CONTRIBUTING TO FM01
STORM SEWERS CONTRIBUTING TO FM02
STORM SEWERS CONTRIBUTING TO FM03
RAILWAY
STUDY AREA
CONTRIBUTING AREA TO FLOW MONITORS

DILLON CONSULTING

MAP CREATED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

6.3 Rain Gauge Data and Flow Monitoring Analysis

Rainfall data was collected from a total of three rain gauges RG01, RG02 and RG03 within the Town throughout the flow monitoring period. The data collected included rainfall volume (mm), intensity (mm/hr) and the number of tips from the rainfall device to calculate the rainfall volume. Each individual rainfall event was initially reviewed to select storms with at least 24-hours of an advance dry weather period.

From April 2018 to October 2018, several rainfall events were recorded, as summarized in Table 6.2.

Table 6.2 Summary of Reviewed Storm Events during the Monitoring Period

Rain Gauge	Rain Event ID	Storm Duration (hr)	Total Rainfall Volume (mm)	Peak Intensity (mm/hr) 15 min interval	Return Period 15 min interval Avg. Intensity (mm/hr)	Return Period Based on Total Rainfall and Duration (mm)
RG01	Apr 3 2018	9.5	25.4	14.20	< 2-year	< 2-year
RG01	Apr 14 2018	9.25	23.62	8.12	< 2-year	< 2-year
RG01	Apr 15 2018	14.75	42.93	24.36	< 2-year	< 2-year
RG01	May 2 2018	25	15.0	10.16	< 2-year	< 2-year
RG01	May11 2018	36	59.7	18.29	< 2-year	> 2-year
RG01	Jul 31 2018	3	27.43	36.56	< 2-year	< 2-year
RG01	Aug 6 2018	2.5	21.09	57.88	> 2-year	< 2-year
RG01	Sep 20 2018	4.25	52.6	52.83	> 2-year	> 5-year
RG01	Sep 25 2018	30.00	59.45	45.72	< 2-year	> 2-year
RG01	Oct 6 2018	11.75	26.16	28.44	< 2-year	< 2-year
RG01	Oct 6 2018	3	8.13	14.24	< 2-year	< 2-year
RG01	Oct 31 2018	7.75	22.86	8.12	< 2-year	< 2-year
RG02	Apr 3 2018	9.25	18.80	10.16	< 2-year	< 2-year
RG02	Apr 14 2018	9	20.58	5.08	< 2-year	< 2-year
RG02	Apr 15 2018	12.25	31.49	36.56	< 2-year	< 2-year
RG02	May 2 2018	25	23.6	25.40	< 2-year	< 2-year
RG02	May 11 2018	36	52.1	19.30	< 2-year	< 2-year
RG02	Jul 31 2018	2.75	16.77	30.48	< 2-year	< 2-year
RG02	Aug 6 2018	2.00	15.24	47.76	> 2-year	< 2-year
RG02	Sep 20 2018	4.25	43.9	31.50	< 2-year	> 2-year
RG02	Sep 25 2018	30.00	58.94	50.80	> 2-year	> 2-year

Rain Gauge	Rain Event ID	Storm Duration (hr)	Total Rainfall Volume (mm)	Peak Intensity (mm/hr) 15 min interval	Return Period 15 min interval Avg. Intensity (mm/hr)	Return Period Based on Total Rainfall and Duration (mm)
RG02	Oct 6 2018	10.75	13.21	13.20	< 2-year	< 2-year
RG02	Oct 6 2018	3.00	24.38	58.92	> 2-year	< 2-year
RG02	Oct 31 2018	7.75	23.62	10.16	< 2-year	< 2-year
RG03	Apr 3 2018	9.50	25.40	14.22	< 2-year	< 2-year
RG03	Apr 14 2018	9.25	23.62	7.11	< 2-year	< 2-year
RG03	Apr 15 2018	12.50	28.96	10.16	< 2-year	< 2-year
RG03	May 2 2018	25	34.0	22.35	< 2-year	< 2-year
RG03	May 11 2018	36	69.1	18.29	< 2-year	> 2-year
RG03	Jul 31 2018	3.50	28.44	64.00	> 2-year	< 2-year
RG03	Aug 6 2018	2.25	13.72	39.60	< 2-year	< 2-year
RG03	Sep 20 2018	4.25	47.8	49.78	> 2-year	> 2-year
RG03	Sep 25 2018**	N/A	N/A	N/A	N/A	N/A
RG03	Oct 6 2018	10.00	16.26	17.24	< 2-year	< 2-year
RG03	Oct 6 2018	3.00	19.81	49.76	> 2-year	< 2-year
RG03	Oct 31 2018	7.75	25.65	12.16	< 2-year	< 2-year

*The rainfall volumes are estimated based on Environment Canada Windsor Airport Station IDF data.
** Precipitation data on September 25th 2018 are unavailable in RG03 due to maintenance activities.

Based on the lack of larger storm events during the monitoring period (no greater than a 1:5 year), selected storms were compared with Environment Canada Windsor Airport Station IDF data and assessed against local 1:2 year storm events. A total of four events were selected for the calibration and validation of the model, including the rainfall events of May 2, 2018, May 11, 2018, September 20, 2018 and September 25, 2018.

These storm events are similar to a local standard 1:2 year design rainfall event except for the September 20, 2018 storm, which appears to be greater than a 1:5 year event at RG01 and slightly greater than a 1:2 year event at RG02 and RG03. The September 20, 2018 was also considered in this study. The total rainfall volume observed in RG01 on May 2, 2018 is significantly less than at RG03, based on which RG01 was not used for model calibration for the May 2, 2018 event.

A summary of the rainfall events selected for model calibration and validation are presented for each rain gauge in Table 6.3. All rainfall data were entered into the PCSWMM model in 5 minute time steps.

Table 6.3: Selected Storm Events for Model Calibration and Validation

Rain Gauge	Rain Event ID	Start Date /Time	End Date /Time	Storm Duration (hr)	Total Rainfall Volume (mm)	Peak Intensity (mm/hr) 15 min interval	Environment Canada Windsor A Station 1:2 Year Volume*	Return Period 15 min interval Avg. Intensity (mm/hr)	Return Period Based on Total Rainfall and Duration (mm)
Calibrated Events									
RG01	May 2 2018	2-May-2018 18:00	3-May-2018 18:45	25	15.0**	10.16	52.3	< 2-year	< 2-year
RG01	May11 2018	11-May-2018 17:10	13-May-2018 4:55	36	59.7	18.29	55.1	< 2-year	> 2-year
RG02	May 2 2018	2-May-2018 18:00	3-May-2018 18:45	25	23.6	25.40	52.3	< 2-year	< 2-year
RG02	May11 2018	11-May-2018 17:10	13-May-2018 4:55	36	52.1	19.30	55.1	< 2-year	< 2-year
RG03	May 2 2018	2-May-2018 18:00	3-May-2018 18:45	25	34.0	22.35	52.3	< 2-year	< 2-year
RG03	May11 2018	11-May-2018 17:10	13-May-2018 4:55	36	69.1	18.29	55.1	< 2-year	> 2-year
Validated Events									
RG01	Sep20 2018	20-Sep-2018 7:05	20-Sep-2018 11:05	4.25	52.6	52.83	38.8	> 2-year	> 5-year
RG01	Sep25 2018	24-Sep-2018 15:00	25-Sep-2018 20:29	30.00	59.45	45.72	53.7	< 2-year	> 2-year
RG02	Sep20 2018	20-Sep-2018 7:05	20-Sep-2018 11:05	4.25	43.9	31.50	38.8	< 2-year	> 2-year
RG02	Sep25 2018	24-Sep-2018 15:00	25-Sep-2018 20:29	30.00	58.94	50.80	53.7	> 2-year	> 2-year
RG03	Sep20 2018	20-Sep-2018 7:05	20-Sep-2018 11:05	4.25	47.8	49.78	38.8	> 2-year	> 2-year
RG03	Sep25 2018***	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

*The rainfall volumes are estimated based on Environment Canada Windsor Airport Station IDF data.
** The total volume at RG01 on May 02 2018 is significantly less than RG03 which is 1.37km within RG01 and thus not used.
*** Precipitation data on September 25th 2018 are unavailable in RG03 due to maintenance activities.

Figure 6.2 through Figure 6.13 illustrate the rainfall intensity, observed flow and relative depth versus time at each flow monitor during the selected rainfall events. Rain gauge RG02 was used for flow monitor FM01 and FM02. Due to field complications with RG03, RG03 or RG01 was used for FM03 depending on which rain gauge was collecting data at the time of the storm. These figures were downloaded from the AMG Environmental website and reproduced below. Each figure also labels the peak flow rate and maximum depth in the sewer at each monitor.

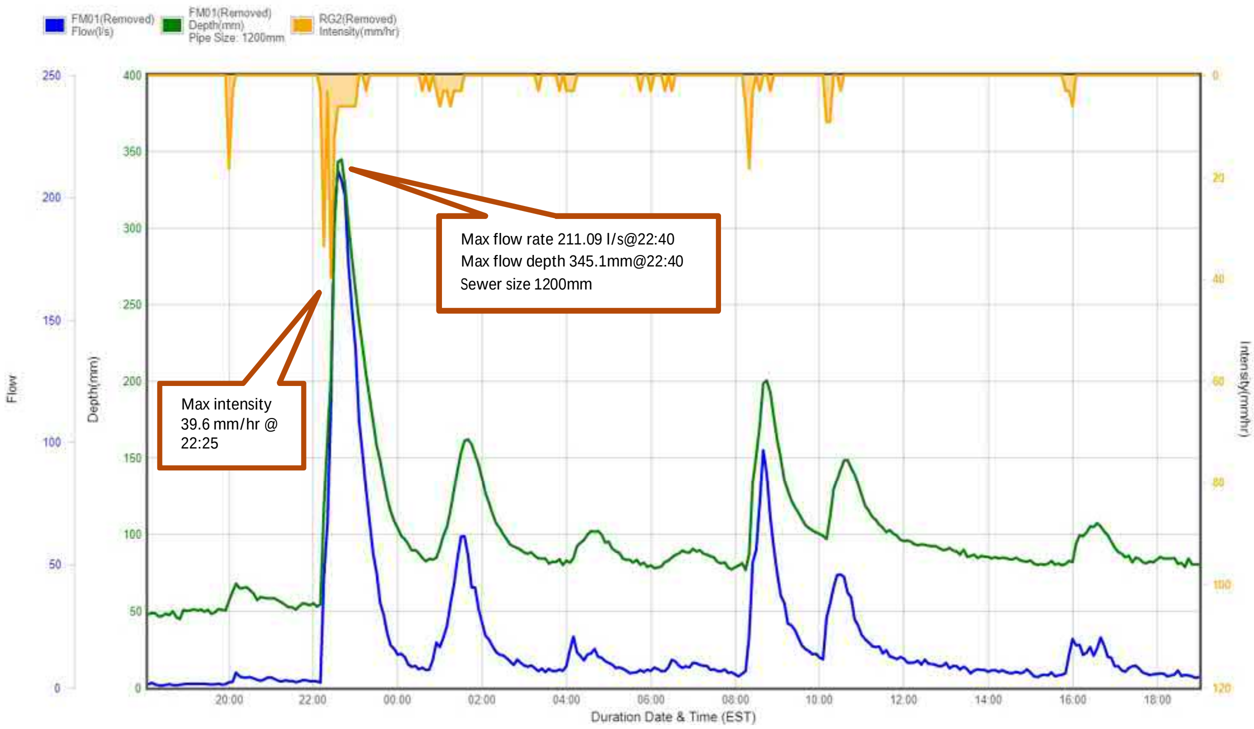


Figure 6.2 Observed Flow Results – May 2, 2018 at FM01 (RG02 Rainfall)

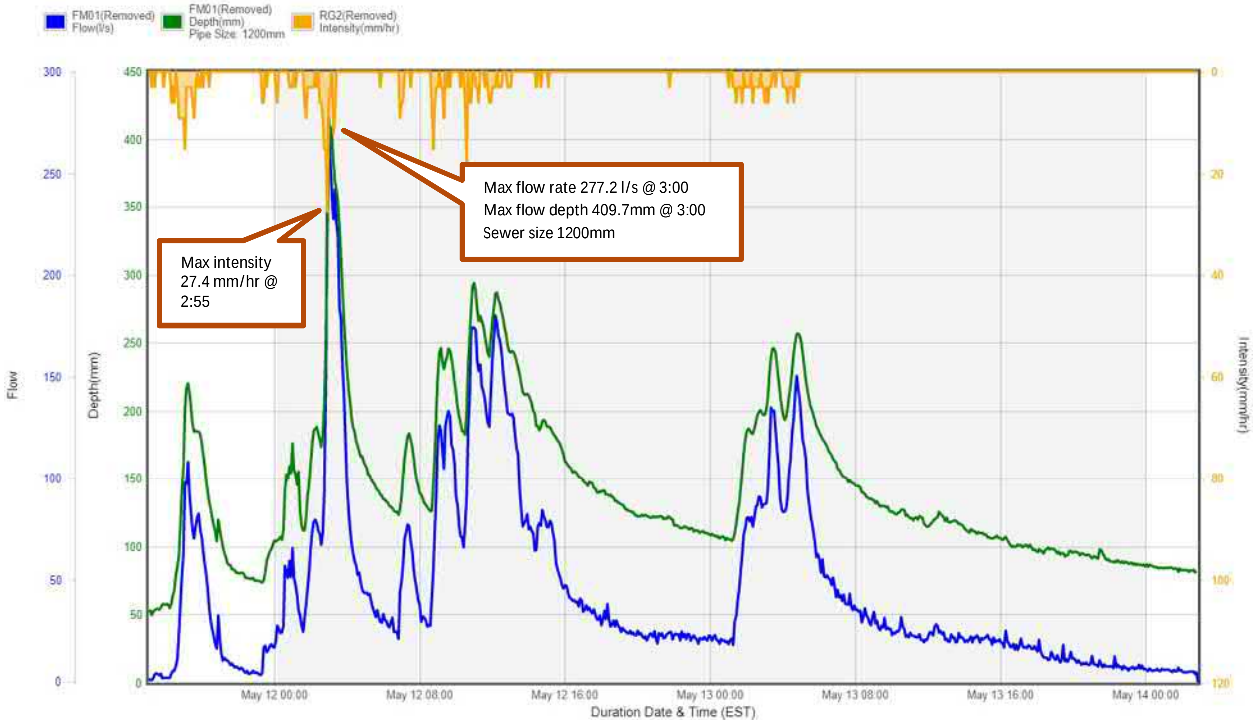


Figure 6.3 Observed Flow Results – May 11, 2018 at FM01 (RG02 Rainfall)

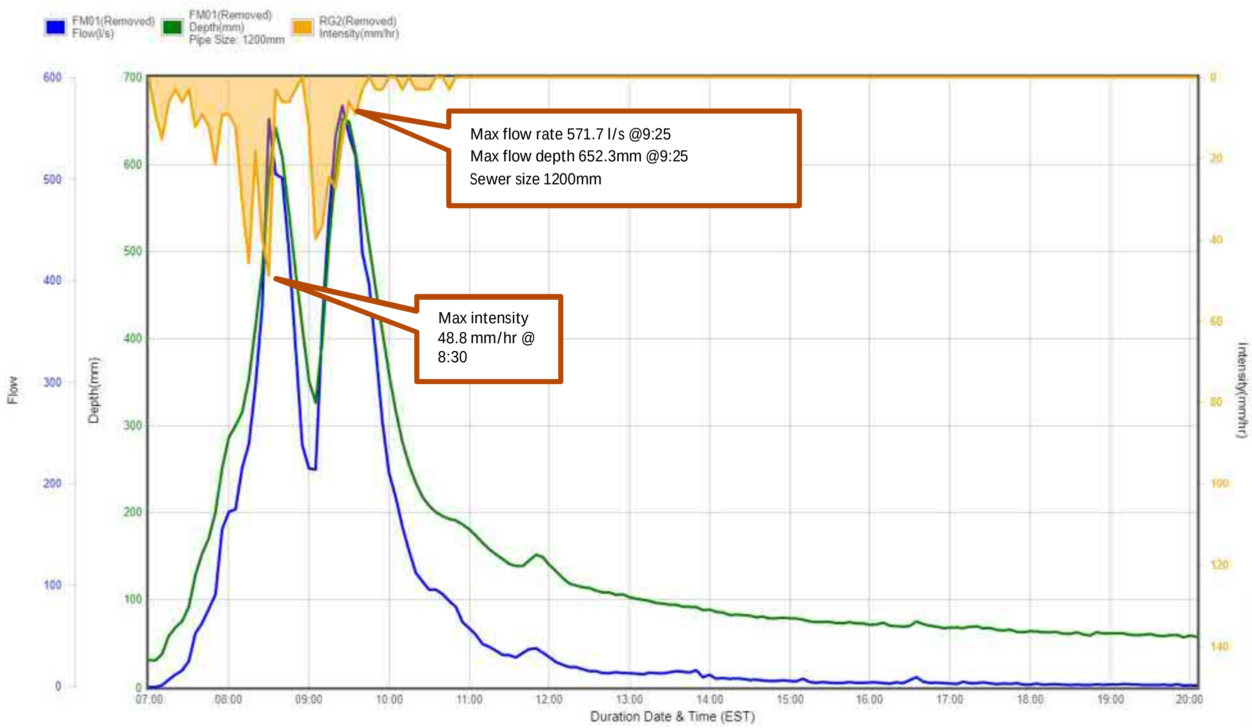


Figure 6.4 Observed Flow Results – September 20, 2018 at FM01 (RG02 Rainfall)

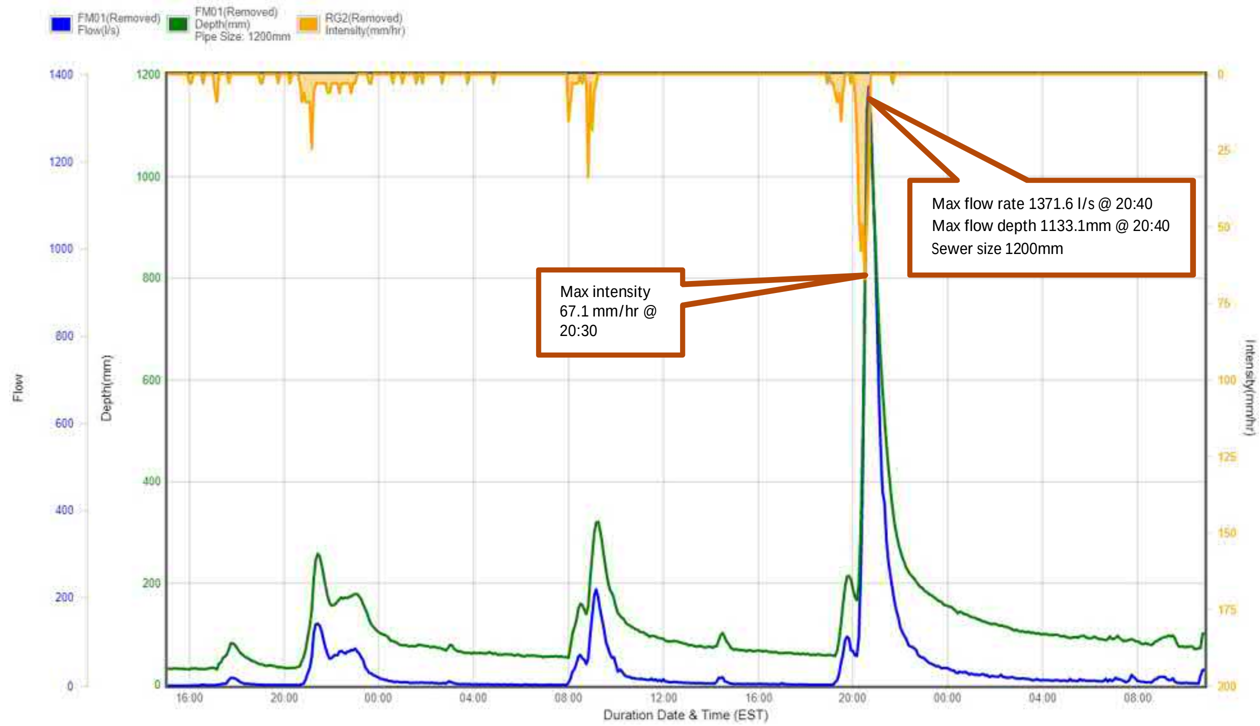


Figure 6.5 Observed Flow Results – September 25, 2018 at FM01 (RG02 Rainfall)

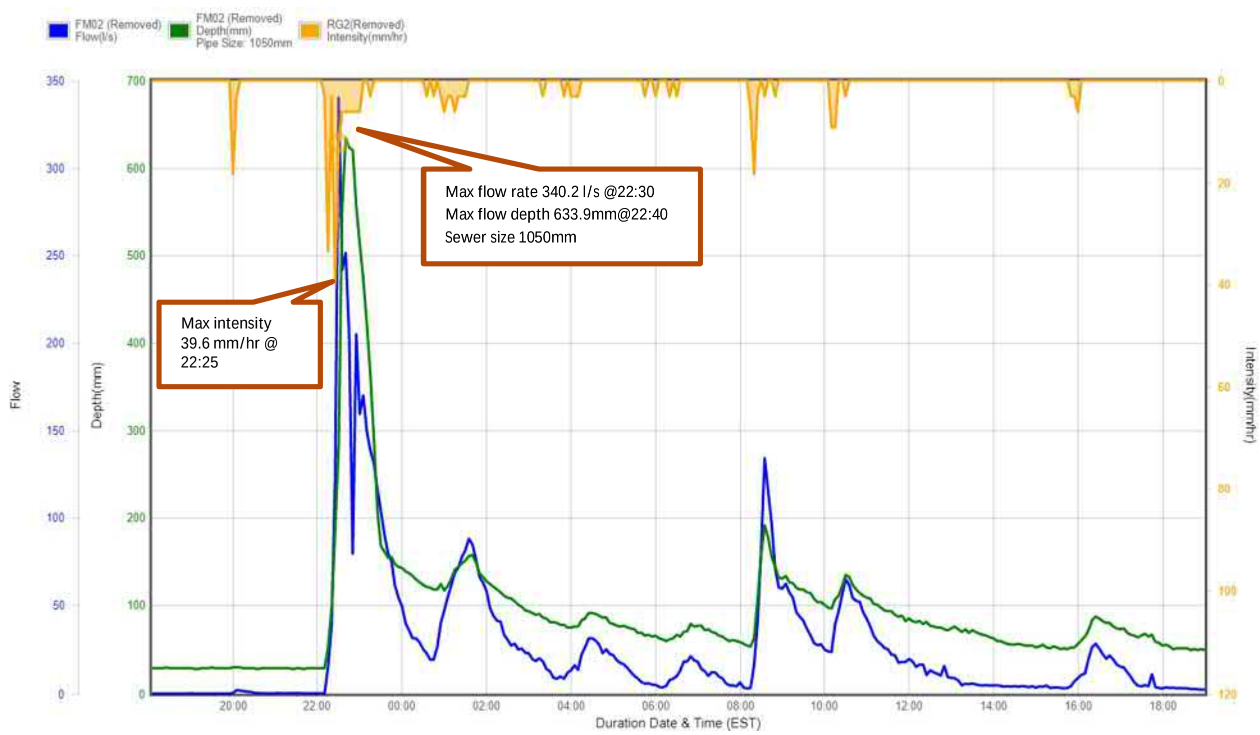


Figure 6.6 Observed Flow Results – May 2, 2018 at FM02 (RG02 Rainfall)

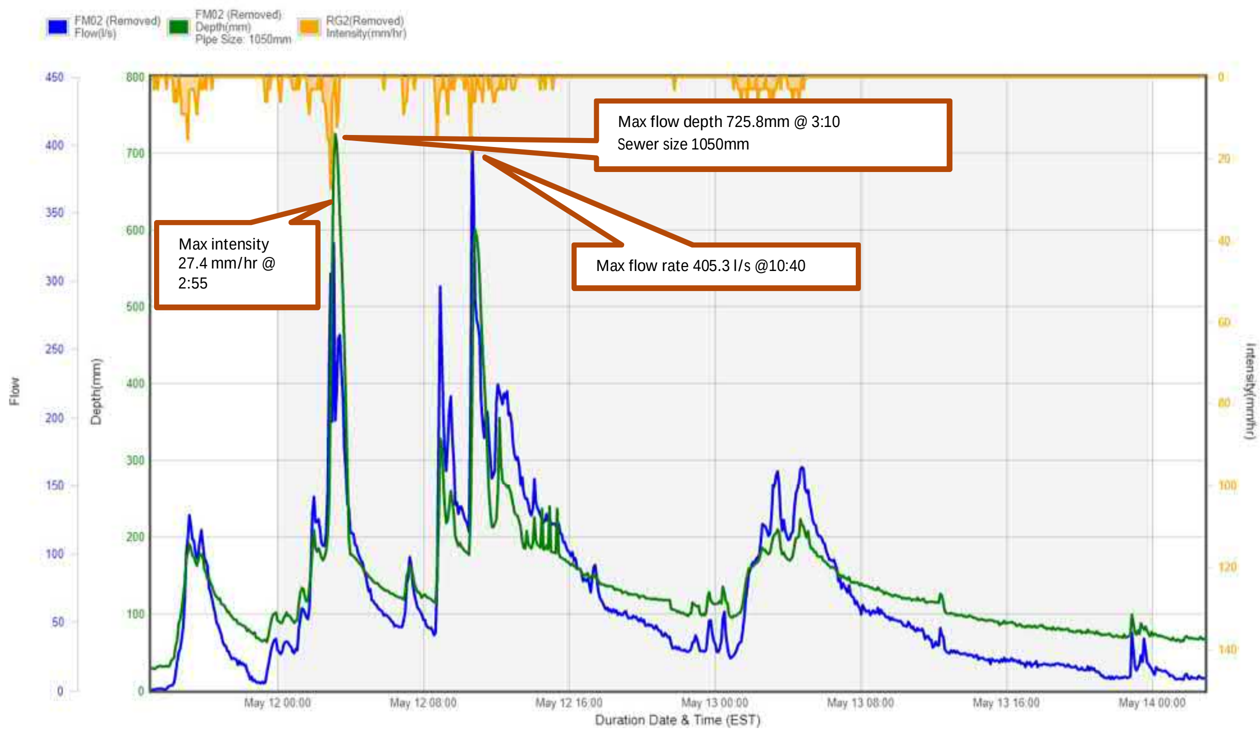


Figure 6.7 Observed Flow Results – May 11, 2018 at FM02 (RG02 Rainfall)

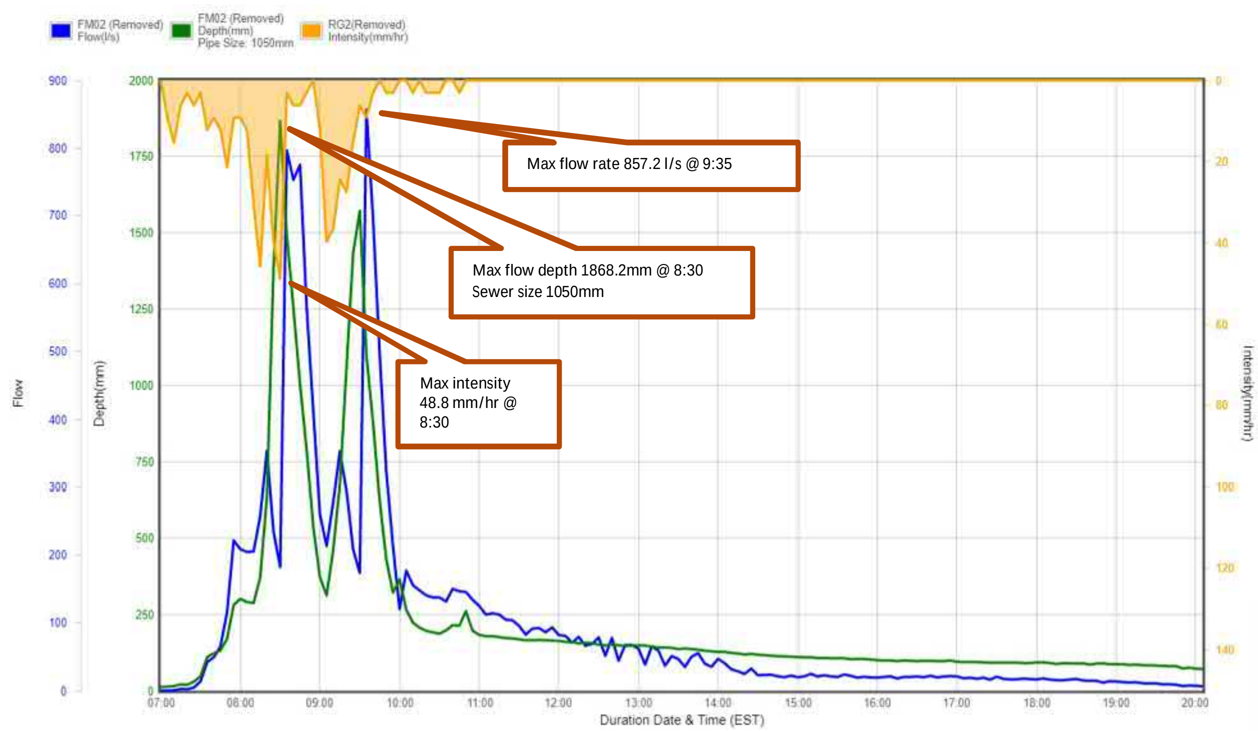


Figure 6.8 Observed Flow Results – September 20, 2018 at FM02 (RG02 Rainfall)

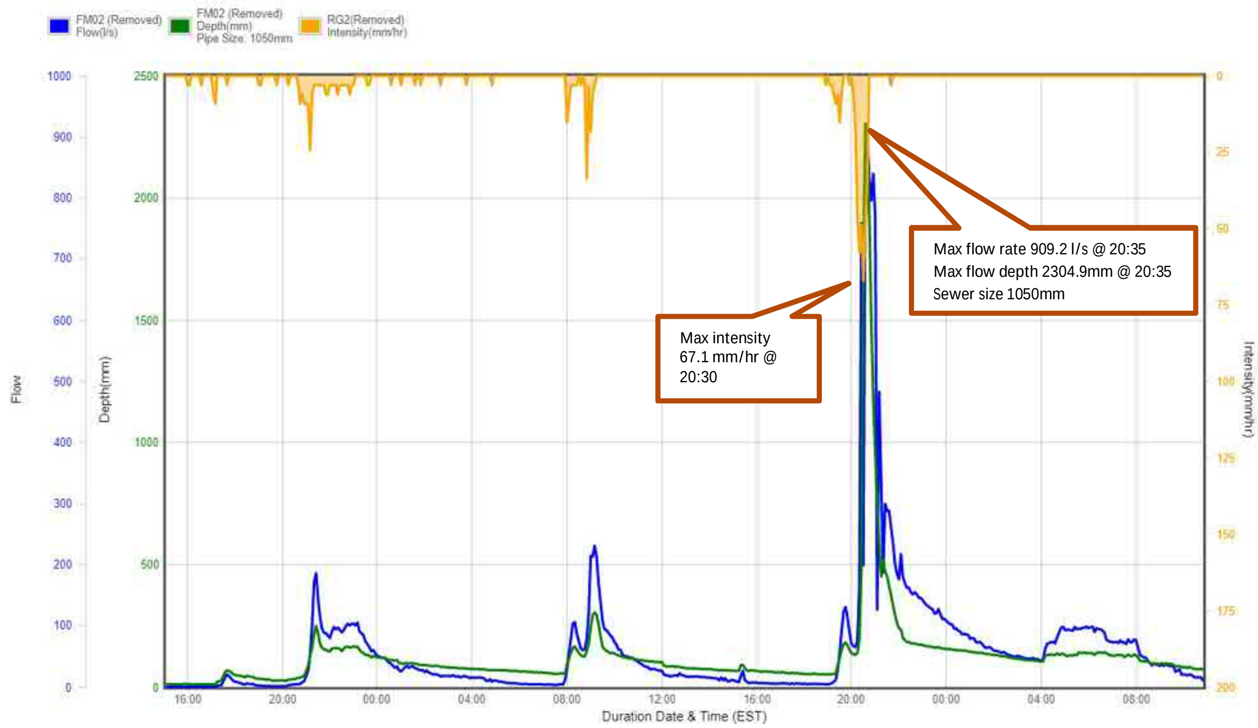


Figure 6.9 Observed Flow Results – September 25, 2018 at FM02 (RG02 Rainfall)

As outlined in Section 2.1.5, the majority of the study area is serviced by pump stations outlets for the storm sewer system. The proximity of the pump station in relation to the flow monitor location may influence the recorded data. During the monitoring period, obvious effects on the flow monitoring data were observed due to the pump cycling downstream of FM03 along Lesperance Road at the intersection of Evergreen Drive, approximately 1.18 km from the pump station for the May 2, 2018, May 11, 2018 and September 25, 2018 events, as illustrated in Figure 6.10, Figure 6.12 and Figure 6.13. Pumping effects were not found in FM03 for the larger September 20, 2018 event, which shows the pump running more continuously during the event. These discrepancies may be due to the configuration and operating characteristics of the Lesperance Pump Station.

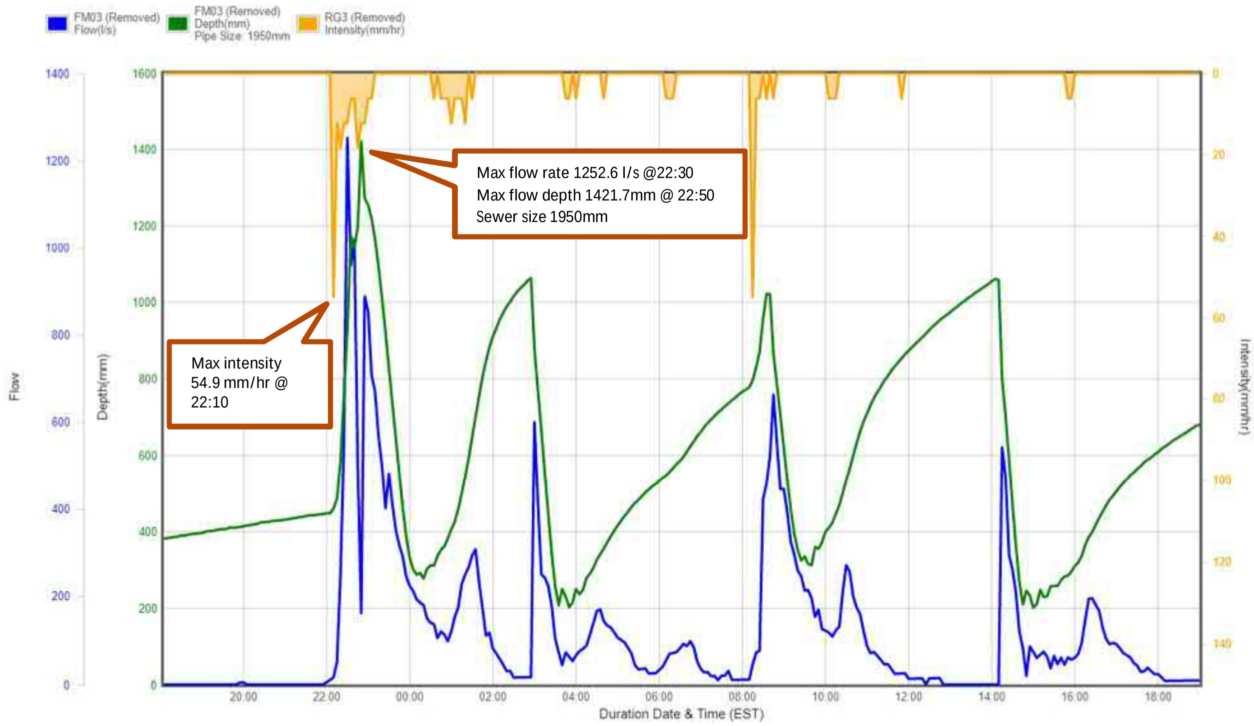


Figure 6.10 Observed Flow Results – May 2, 2018 at FM03 (RG03 Rainfall)

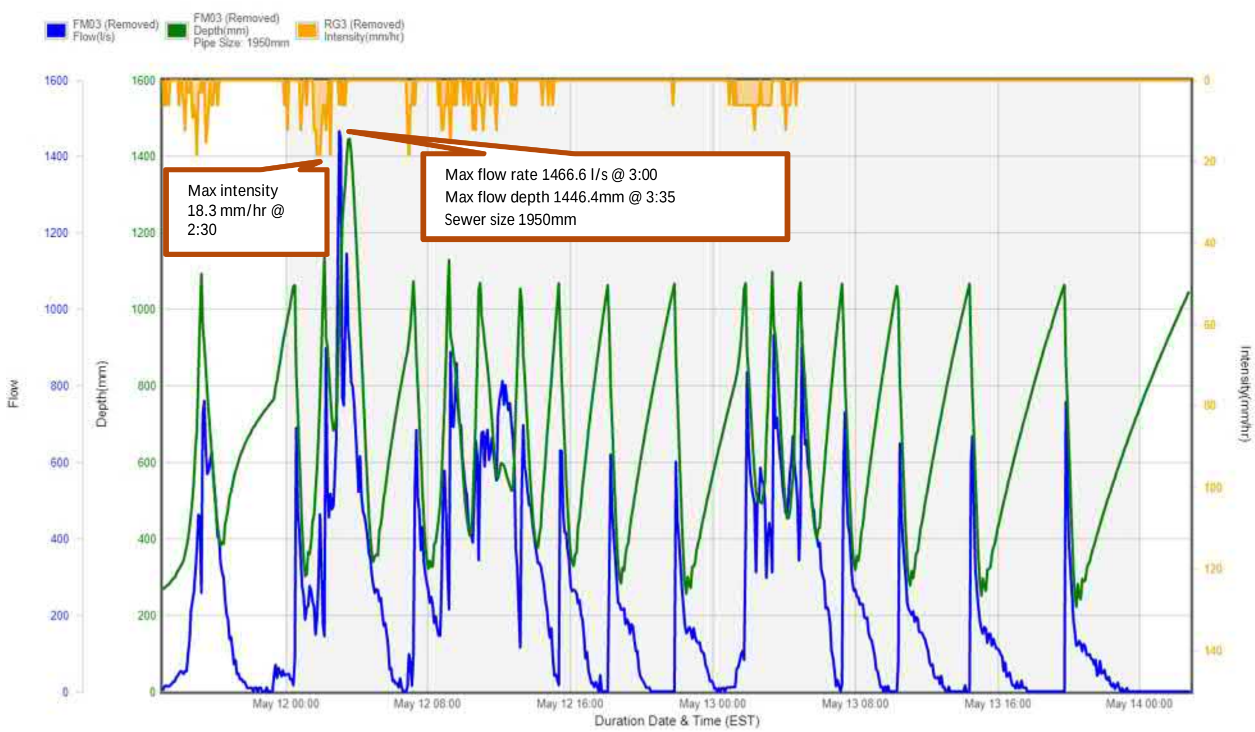


Figure 6.11 Observed Flow Results – May 11, 2018 at FM03 (RG03 Rainfall)

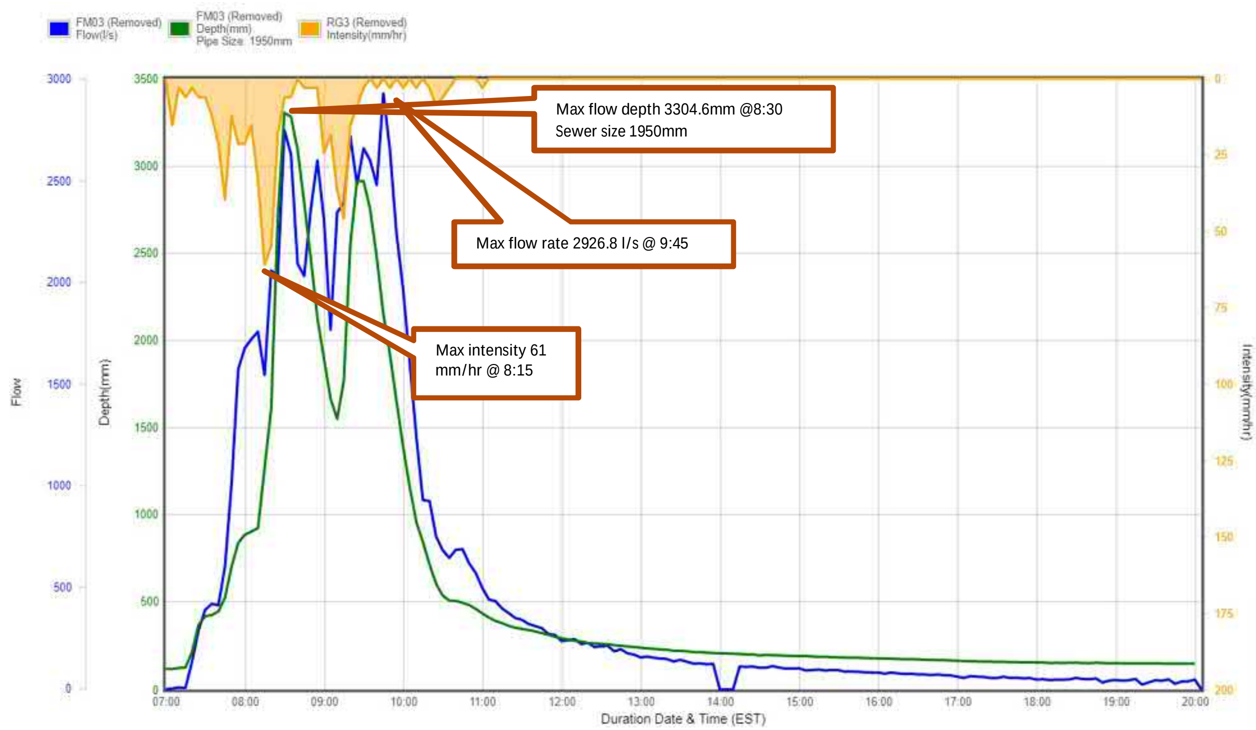


Figure 6.12 Observed Flow Results – September 20, 2018 at FM03 (RG03 Rainfall)

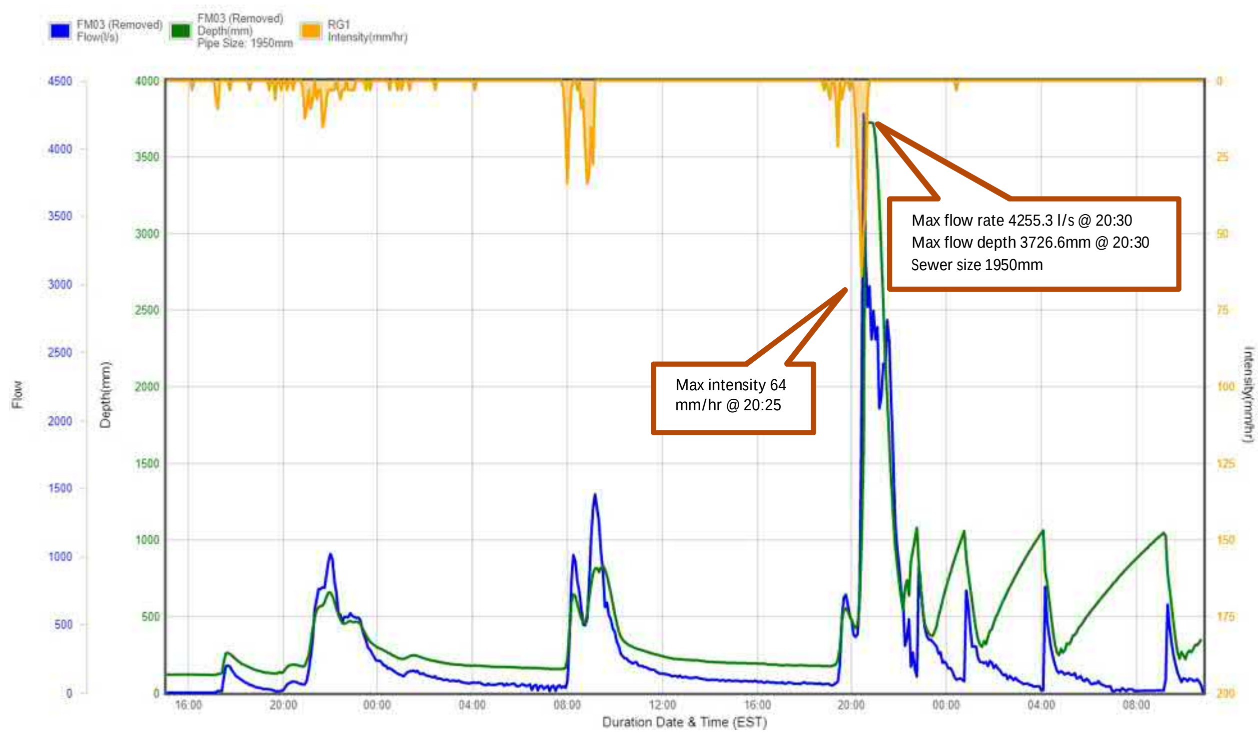


Figure 6.13 Observed Flow Results – September 25, 2018 at FM03 (RG01 Rainfall)

7.0 Model Calibration and Validation

7.1 Overview

As identified in Section 6.1, it was identified that a more refined model was to be developed to more accurately assess storm runoff and antecedent conditions in the study area during smaller, more frequent storm events. Although the original model developed for this study used calculated hydrologic watershed values, it did not account for a number of factors that can change the flow and runoff volume entering and conveying through the storm sewer system. This includes the known sources of inflow leaving or not entering the storm sewer system and conveying through the sanitary system. These sources have been identified in previous Town sanitary system studies, including storm inflow from sanitary manhole covers, private rooftop and foundation drain connections and potential commercial/industrial/institutional private drain connections.

It was determined at the onset of the study that the original model with the calculated hydrologic watershed values would be used to determine the surface flooding solutions. This approach takes a conservative measure to the solutions and was identified to be beneficial for the following reasons:

- Takes into consideration Town recommendations to disconnect rooftops and foundation drains from the sanitary system and connect to the local storm sewers; and
- Allows the Town to design the recommended surface flooding solutions to a level of resiliency where all respective storm runoff is entering the storm sewer system.

Although the study is primarily focused on surface flooding mitigation measures using the original model, the more refined model is to be used to understand the existing hydraulic characteristics through the municipal storm sewer system during the 1:2 year storm event. This model is recommended to be used, in certain areas, to determine the allowable release rates for future infill development areas where the re-connection of sources of inflow on the private side back to the storm sewer system is not feasible.

7.2 Model Calibration

A model is considered calibrated when the numerical simulation produces results deemed sufficiently similar to the information collected in the recorded flow data. The refined calibration of the original model focuses on volume, peak flow, the general hydrograph shape and timing of peaks. This study undertook a calibration process with the goal of evaluating the hydraulic characteristics in the existing storm sewer system. Model calibration was completed by adjusting model parameters to produce simulation results similar to the recorded data. A good calibration was determined once the difference between measured and modelled volumes and flows were within 20% - 25%.

Key parameters for the contributing subcatchments that were adjusted as part of the calibration process included impervious values, depression storage, catchment flow lengths/equivalent width and catchment area (if required). The catchment areas were only adjusted in the instance where other parameter adjustments seemed unreasonable and did not reasonably reflect the conditions in the subwatershed. A reduction in catchment area was deemed warranted where there were known inflow and infiltration (I&I) issues in the existing system and where portions of potential rooftop and foundation drain runoff may be directed to the sanitary sewer. These issues impact the storm sewer calibration as volume not getting to, or leaving the storm sewer system and not being measured at the monitoring locations. Parameters were adjusted until the model results compared favourably to the monitored data.

Once the flow monitoring subcatchment areas were calibrated, these parameters were applied to areas having similar characteristics. The model calibration was completed in order to enhance the ability of the PCSWMM model to represent existing hydrologic and hydraulic conditions under minor storm events. In most instances, 1 storm event achieved lower volume and/or peakflow levels in the model than the observed events, but were then counteracted by another event with high values, either within the model calibration or validation storms. The primary target during the model calibration was to achieve the closer values with respect to volume and achieve a balanced model.

The degree of calibration was characterized based on the following:

- Good Level of Calibration – model estimates are within 20% - 25% above or below the observed values of peak flow and/or volume.
- Average Level of Calibration – model estimates are within 40% above or below the observed values of peak flow and/or volume.
- Poor Level of Calibration– model estimate are beyond 40% of the observed values of peak flow and/or volume.

The May 2 and May 11, 2018 events were used for model calibration, with the remaining 2 events used for validation. The following figures identify the model calibration results as part of this study and comparison of the observed flows and modelled flows at each monitoring location.

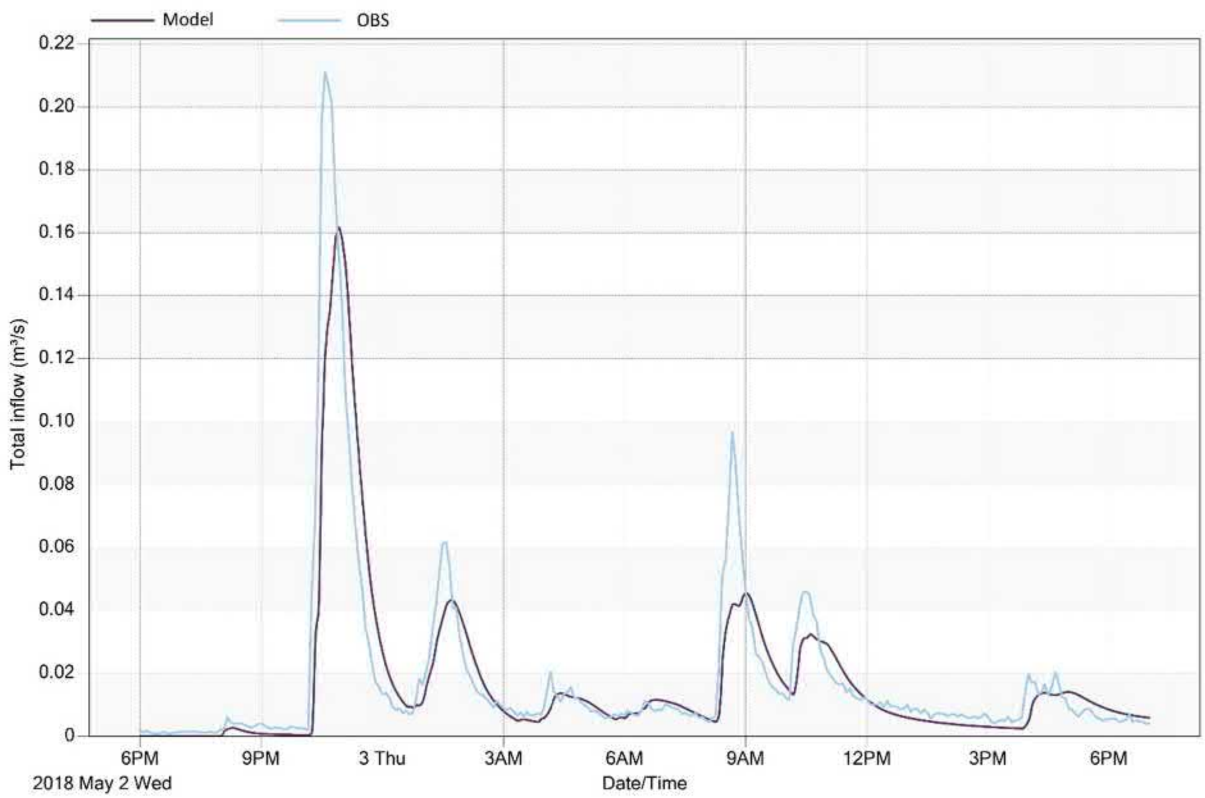


Figure 7.1 Calibration Results – May 2, 2018 at FM01

As shown in Figure 7.1, the predicted runoff volume is 9.4% lower than the observed and the predicted peak flow is 23.8% lower than the observed. The model fit well with the timing and shape of the peak hydrograph. As a result, the calibration results at FM01 are considered good for the May 2, 2018 storm.

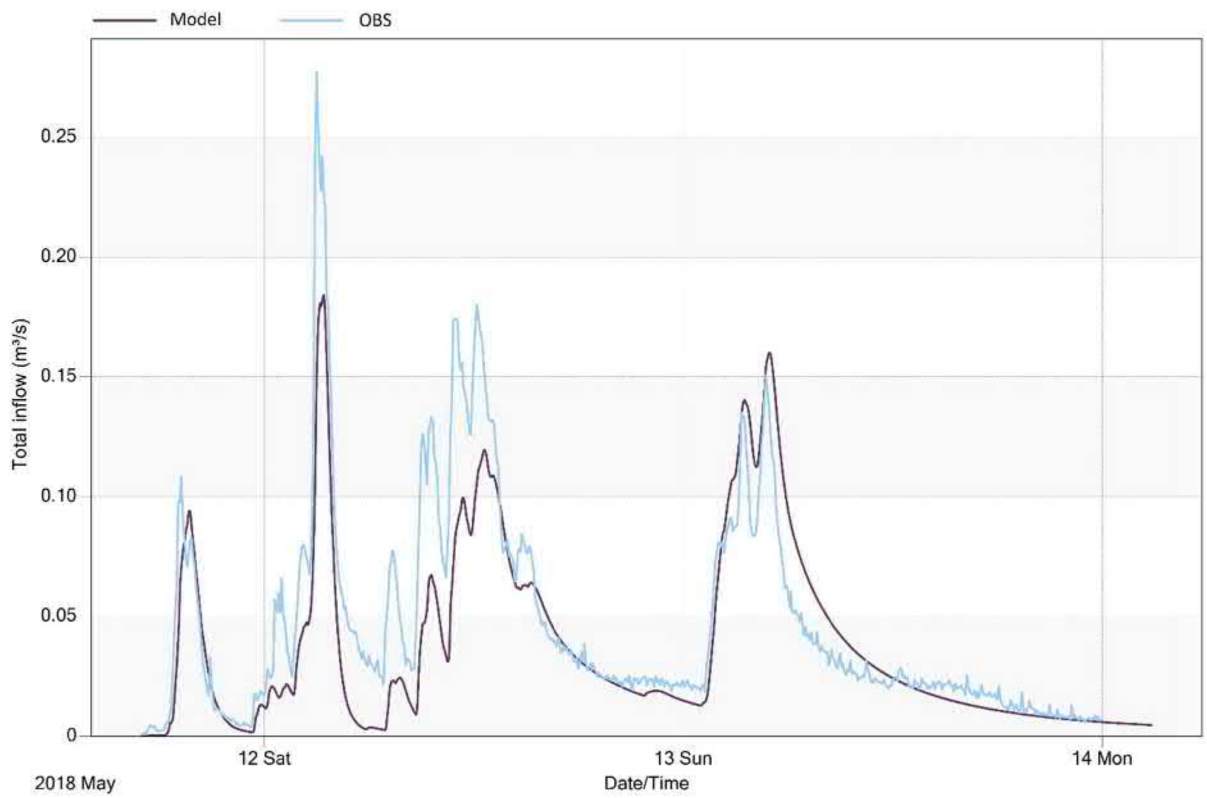


Figure 7.2 Calibration Results – May 11, 2018 at FM01

As shown in Figure 7.2, the predicted runoff volume is 17% lower than the observed and the predicted peak flow is 35.7% lower than the observed. The model fit well with the timing and shape of the peak hydrograph. As a result, the calibration results at FM01 are considered good to average for the May 11, 2018 storm.

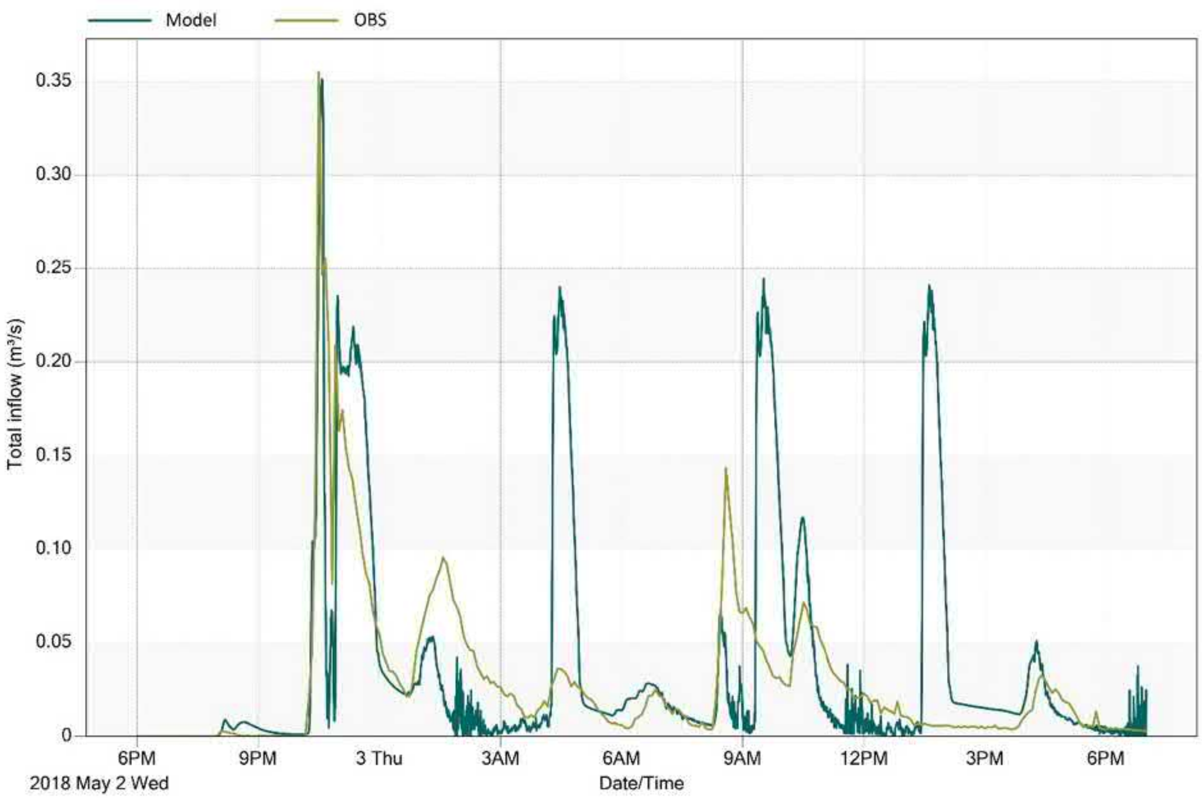


Figure 7.3 Calibration Results – May 2, 2018 at FM02

As shown in Figure 7.3, the predicted runoff volume is 24.5% higher than the observed and the predicted peak flow is 2.8% lower than the observed. Observed pump cycling at the Scully Pump Station is not reflected in the model results. During the background investigation, pump curves for this station could not be found. The spikes within the model results may be due to this. The pump station in the model was developed with an assumed pump outflow operational behaviour working together with the on/off elevations provided by Town staff. The model fit reasonably well with the timing and shape of the peak hydrograph, with the exception of the pump cycling. As a result, the calibration results at FM02 are considered good to average for the May 2, 2018 storm.

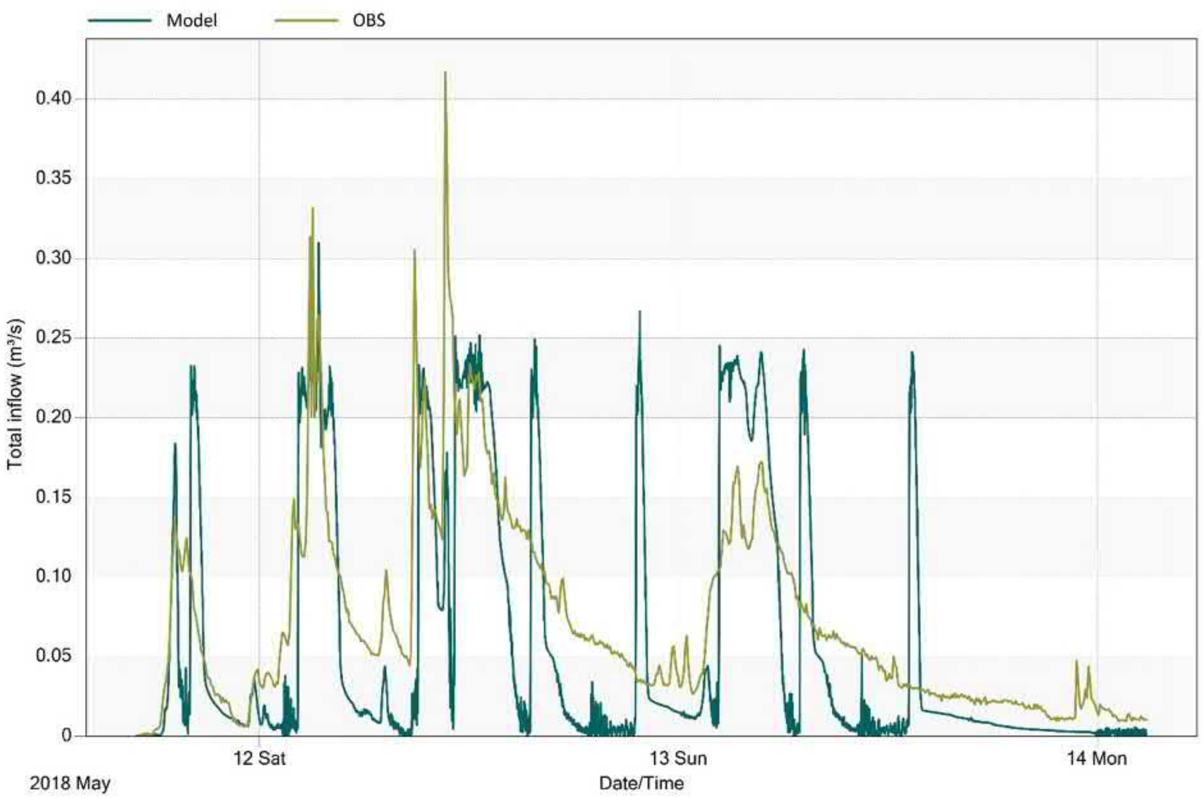


Figure 7.4 Calibration Results – May 11, 2018 at FM02

As shown in Figure 7.4, the predicted runoff volume is 19.7% lower than the observed and the predicted peak flow is 26.2% lower than the observed. The observed data does not exhibit the same pump cycling as noted in the model results. During the background investigation, pump curves for this station could not be found. The spikes within the model results may be due to this. The pump station in the model was developed with an assumed pump outflow operational behaviour working together with the on/off elevations provided by Town staff. The model fit reasonably well with the timing and shape of the peak hydrograph, with an exception of the pump cycling spikes. As a result, the calibration results at FM02 are considered good to average for the May 11, 2018 storm.

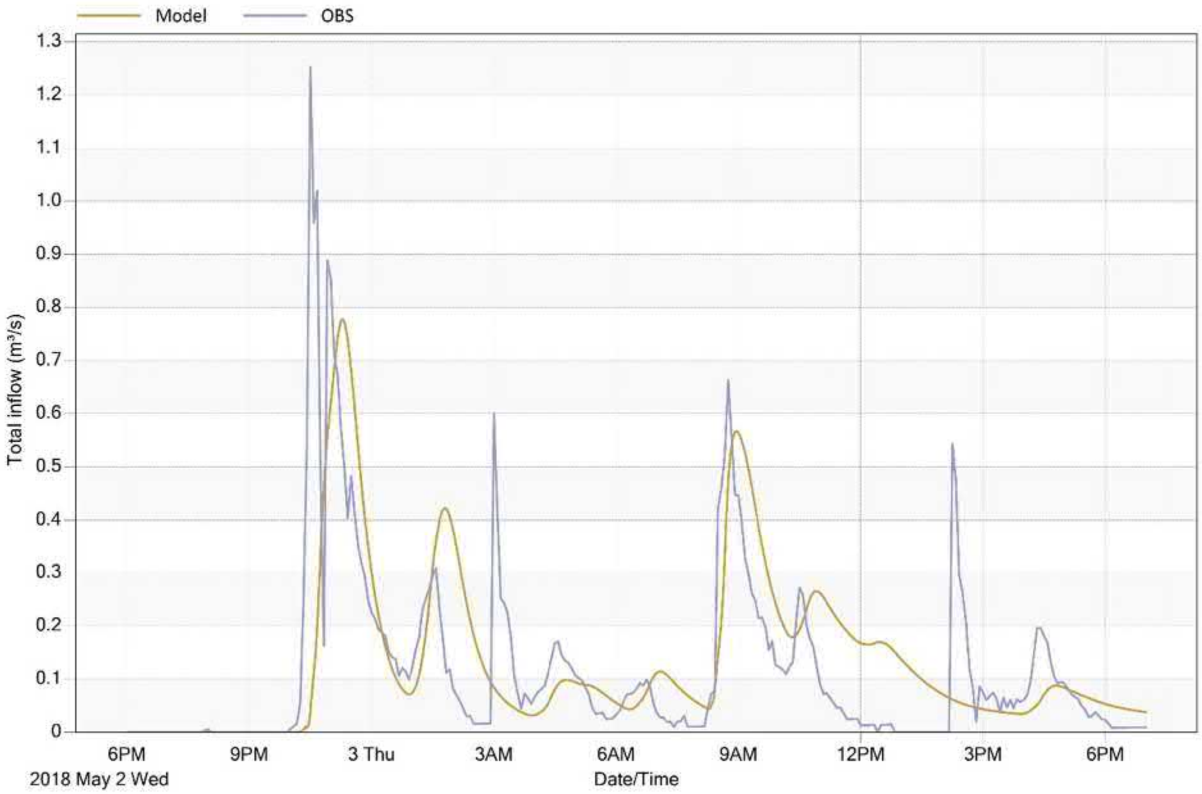


Figure 7.5 Calibration Results – May 02, 2018 at FM03

As shown in Figure 7.5, the predicted runoff volume is 11.6% higher than the observed and the predicted peak flow is 37.6% lower than the observed. The small spikes within the observed data were not identified in the model. This could be due to the accuracy of operating characteristics of the multiple pumps within the Lesperance Pump Station. The model fit well with the timing and shape of the peak hydrograph. As a result, the calibration results at FM03 are considered good to average for the May 2, 2018 storm.

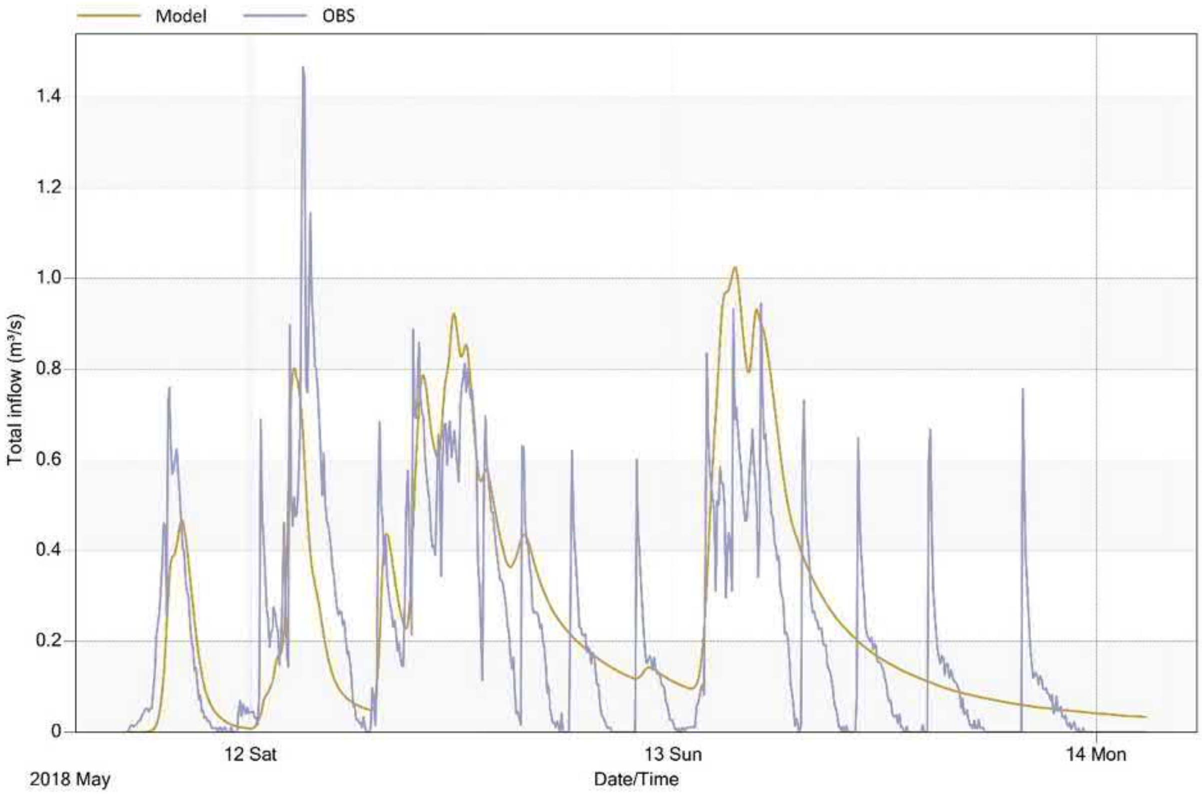


Figure 7.6 Calibration Results – May 11, 2018 at FM03

As shown in Figure 7.6, the predicted runoff volume is 15.8% higher than the observed and the predicted peak flow is 30.5% lower than the observed. The small spikes, which appear to indicate pump cycling, within the observed data were not replicated in the model. This could be due to the accuracy of operating characteristics of the multiple pumps within Lesperance Pump Station. The model fit reasonably well with the timing and shape of the peak hydrograph. As a result, the calibration results at FM03 are considered good to average for the May 11, 2018 storm.

The summary Table 7.1 illustrates the monitored versus modelled runoff volumes and peak flows comparison before and after calibration.

Table 7.1 Calibration Volume and Peak Flow Comparison

FM/ Event	VOLUME (m^3)					PEAK FLOW (m^3/s)				
	Observed Monitor Data	Model Before Calibration	Percent Difference Before Calibration (%)	Model After Calibration	Percent Difference After Calibration (%)	Observed Monitor Data	Model Before Calibration	Percent Difference Before Calibration (%)	Model After Calibration	Percent Difference After Calibration (%)
FM01/ MAY 2 2018	1683	4664	177.1	1524	-9.4	0.21	0.51	142.9	0.16	-23.8
FM01/ MAY11 2018	9445	14940	58.2	7837	-17.0	0.28	0.58	107.1	0.18	-35.7
FM02/ MAY 2 2018	2652	3690	39.1	3302	24.5	0.36	0.39	8.3	0.35	-2.8
FM02/ MAY11 2018	15060	12580	-16.5	12090	-19.7	0.42	0.46	9.5	0.31	-26.2
FM03/ MAY 2 2018	10780	37910	251.7	12030	11.6	1.25	2.03	62.4	0.78	-37.6
FM03/ MAY11 2018	47230	102000	116	54680	15.8	1.47	2.43	65.6	1.02	-30.5

Note: Modelled Runoff Error of 0 and Model Routing Error ranging from 2.3% to 6.2% during calibration.

Based on the above table, the model calibration generally identifies that for the May 2, 2018 and May 11, 2018 storm events, the model had good volume matches (9.4% to 24.5% difference) and good to average peak flow matches (2.8% to 37.6% difference). The calibration for the volume was prioritized in order to examine level of service requirements for the storm sewer system. Due to the discrepancies between pump station operations and modeling, pump cycling in the modeled hydrographs were not observed in FM02 and observed spikes were not predicted in the model at FM03.

To improve the model results, catchment parameters were adjusted to better reflect the observed conditions, as outlined in Table 7.2 .

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Table 7.2 Summary of Key Hydrology Calibration Parameters

Key Parameters	Calculated Watershed Values	Adjusted Values for Model Refinement
Drainage Area*	100 %	73%
% Impervious	47 %	27 %
Impervious Depression Storage	1.57 mm	2.82 mm
Pervious Depression Storage	4.67 mm	8.38 mm
Flow Length	114 m	208 m

** Drainage Area only reduced where other catchment parameters began to exceed realistic conditions.*

During the model calibration, certain hydrologic parameters were adjusted to limits where the values would not be realistic based on the existing land use (i.e. percent impervious). Based on the summary of key hydrologic parameters noted above, the drainage area parameter was reduced in some areas. This was to account for the following:

- Potential loss and/or retention of runoff volume due to unknown factors or stormwater controls within private commercial/industrial areas, residential rear yards and greenspace areas where runoff volume is not getting to the storm sewer system;
- Topographic constraints preventing smaller storm runoff from reaching the storm sewer; and
- Loss of runoff volume due to inflow and infiltration (I&I) in areas where known rooftop and foundation drains are connected to the sanitary system. Solutions to reduce I&I in the sanitary system would ultimately increase the amount of rainfall volume to the storm sewer system, which would impact the refined calibration of the model.

7.3 Model Validation

Two key validation events were chosen during the analysis of the rainfall data. Once again, events were chosen to be similar to a 1:2 year event to further confirm the level of service in the storm sewer system. The September 20 and September 25, 2018 events were used for validation of the minor system model. The validation process determines the performance of the calibrated model and consists of taking the additional events and comparing the results to the recorded flow data.

Model validation results are compared to observed data in the following figures. Commentary is provided below each figure to describe the effectiveness of the model calibration.

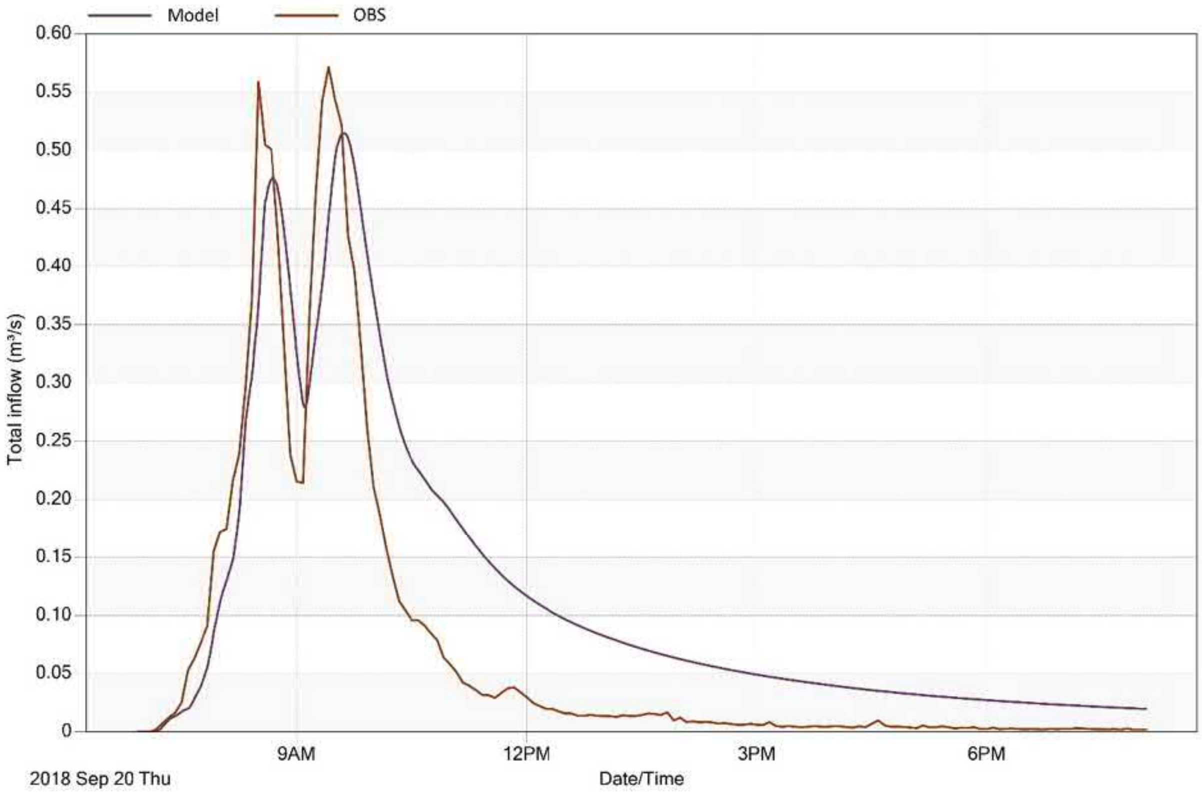


Figure 7.7 Validation Results – September 20, 2018 at FM01

As shown in Figure 7.7, the predicted runoff volume is 57.3% higher than the observed and the predicted peak flow is 10.5% lower than the observed. The model fit well with the timing and shape of the peak hydrograph except for the recession curve. The observed data identified a shorter recession than the modelled flows. This could be due to a number of factors including manual operating adjustments at the station during the storm event, the accuracy of the Brighton pump station operating characteristics entered in the model or the pumps within the station working at a higher efficiency than the design curve. As a result, the calibration results at FM01 are considered average to poor for the September 20, 2018 storm.

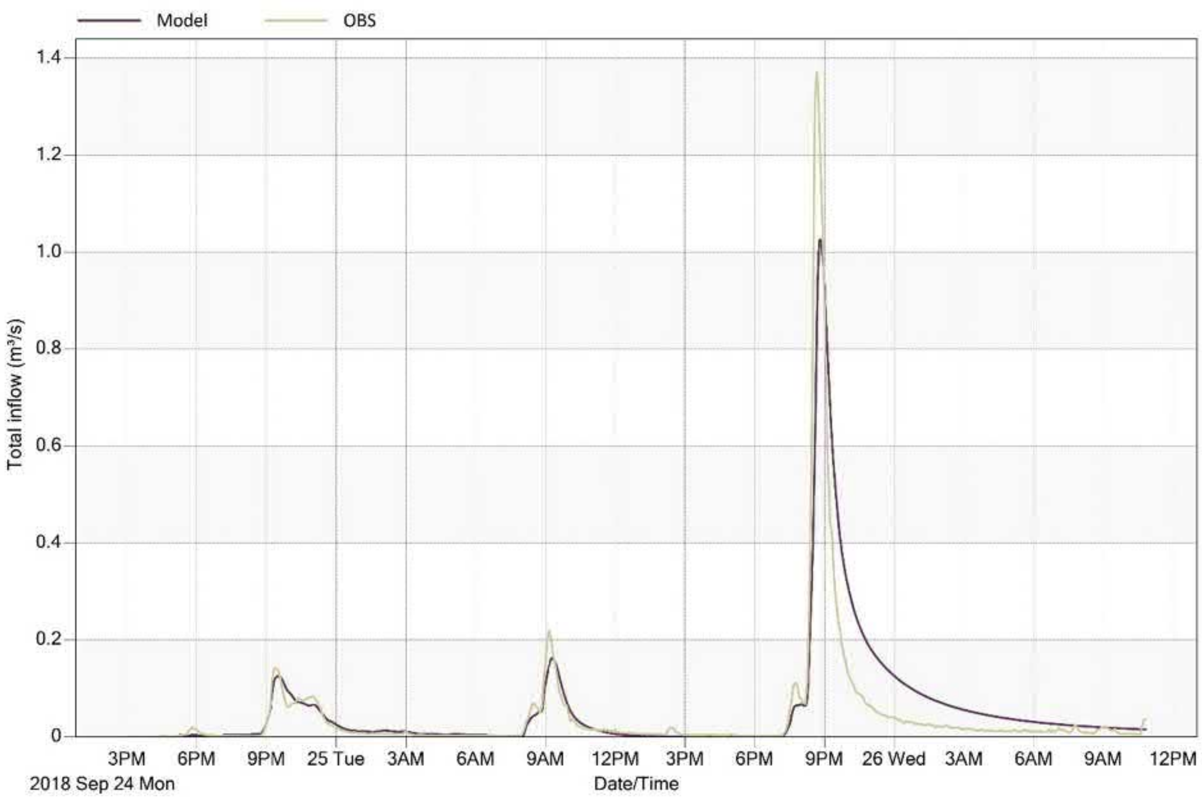


Figure 7.8 Validation Results – September 25, 2018 at FM01

As shown in Figure 7.8, the predicted runoff volume is 21.7% higher than the observed and the predicted peak flow is 24.9% lower than the observed. The model fit well with the timing and shape of the peak hydrograph except for the recession curve. The observed data identified a shorter recession than the modelled flows. This could be due to a number of factors including manual operating adjustments at the station during the storm event, the accuracy of the Brighton pump station operating characteristics entered in model or the pumps within the station working at a higher efficiency than the design curve. As a result, the calibration results at FM01 are considered good for the September 25, 2018 storm.

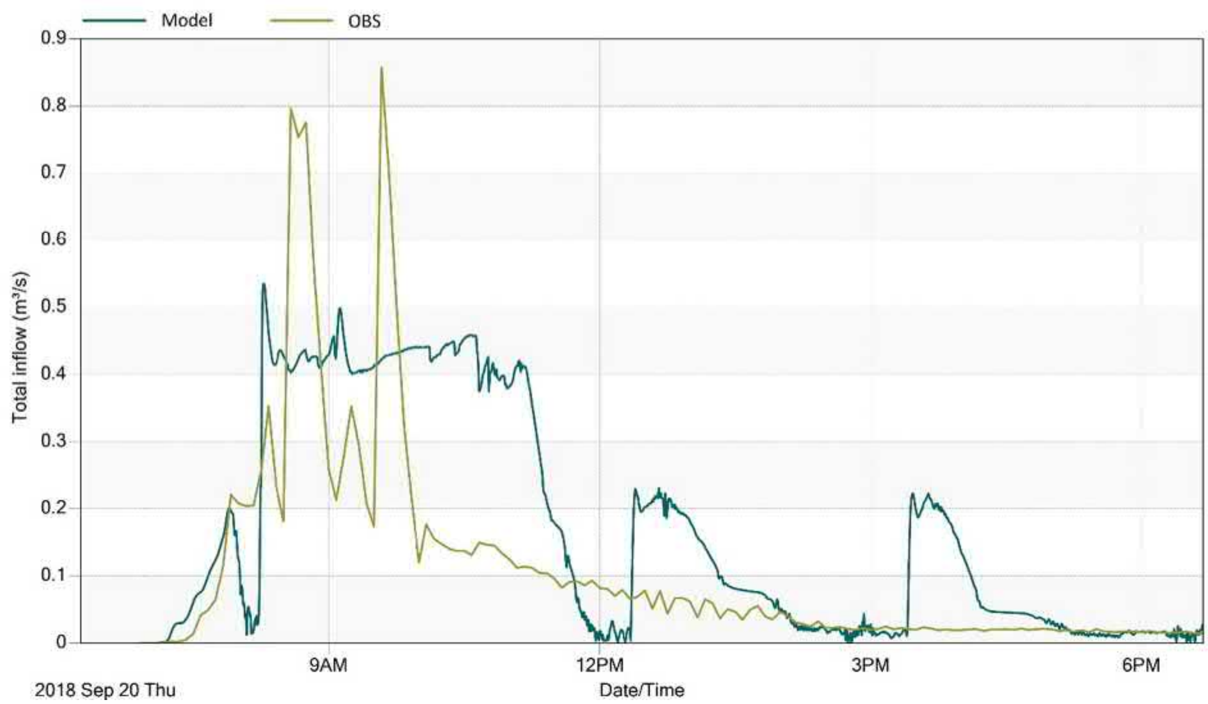


Figure 7.9 Validation Results – September 20, 2018 at FM02

As shown in Figure 7.9, the predicted runoff volume is 50.3% higher than the observed and the predicted peak flow is 38.4% lower than the observed. The September 20th event produced longer constant pumping times within the model, as shown above. During the background investigation, pump curves for this station could not be found. The spikes within the model results may be due to this. The pump station in the model was developed with an assumed pump outflow operational behaviour working together with the on/off elevations provided by Town staff. The model fit reasonably well with the timing and shape of the peak hydrograph, with an exception of the intermittent spikes. As a result, the calibration results at FM02 are considered poor for the September 20, 2018 storm.

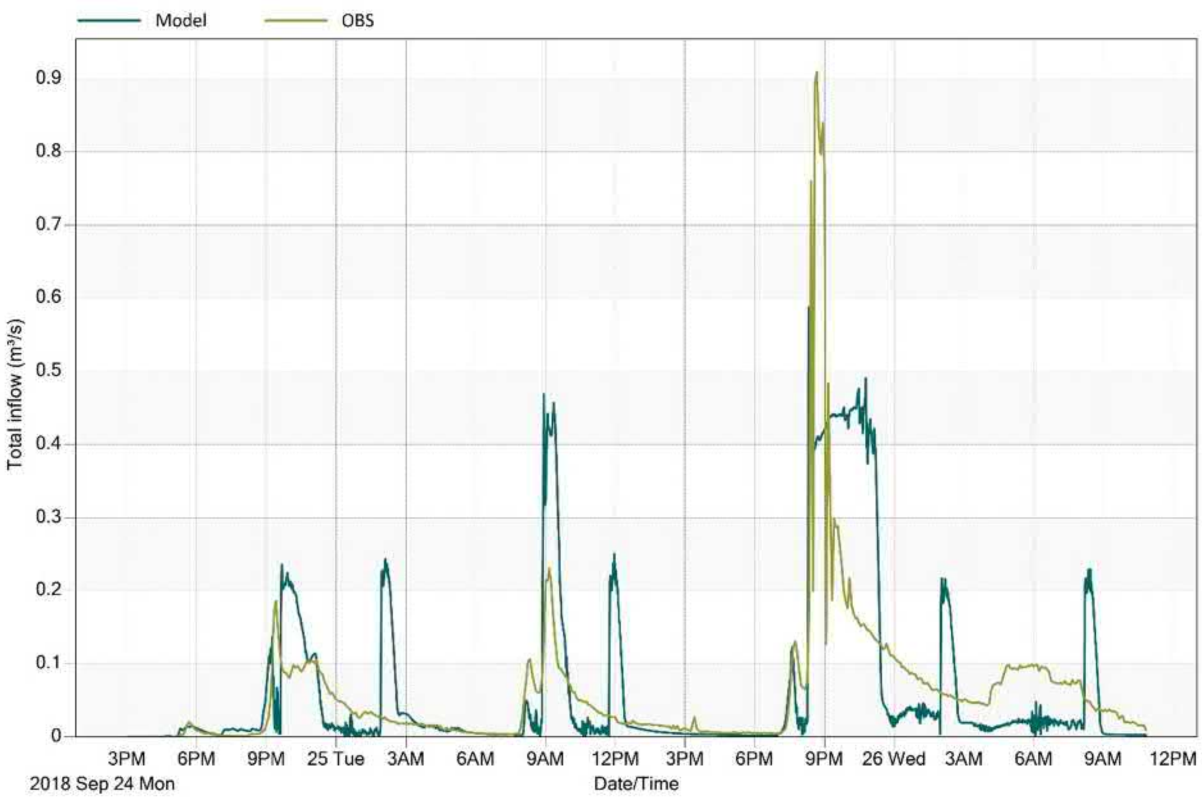


Figure 7.10 Validation Results – September 25, 2018 at FM02

As shown in Figure 7.10, the predicted runoff volume is 18.9% higher than the observed and the predicted peak flow is 35.2% lower than the observed. During the background investigation, pump curves for this station could not be found. The spikes within the model results may be due to this. The pump station in the model was developed with an assumed pump outflow operational behaviour working together with the on/off elevations provided by Town staff. The model fit reasonably well with the timing and shape of the peak hydrograph, with the exception of the intermittent spikes. As a result, the calibration results at FM02 are considered good to average for the September 25, 2018 storm.

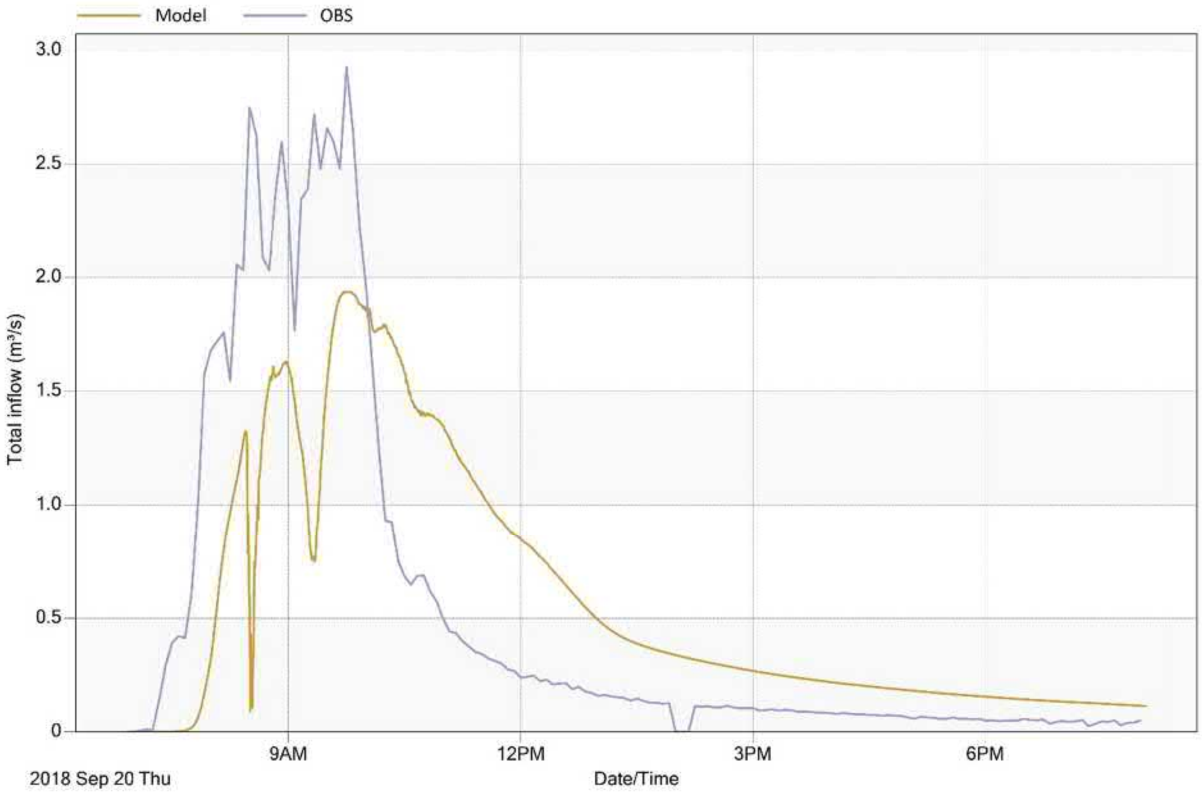


Figure 7.11 Validation Results – September 20, 2018 at FM03

As shown in Figure 7.11, the predicted runoff volume is 7.3% higher than the observed and the predicted peak flow is 33.8% lower than the observed. The model fit well with the timing and shape of the peak hydrograph except for the recession in the end. This could be due to a number of factors including manual operating adjustments at the station during the storm event, the accuracy of the Lesperance pump station operating characteristics entered in model or the pumps within the station working at a higher efficiency than the design curve. As a result, the calibration results at FM03 are considered good to average for the September 20, 2018 storm.

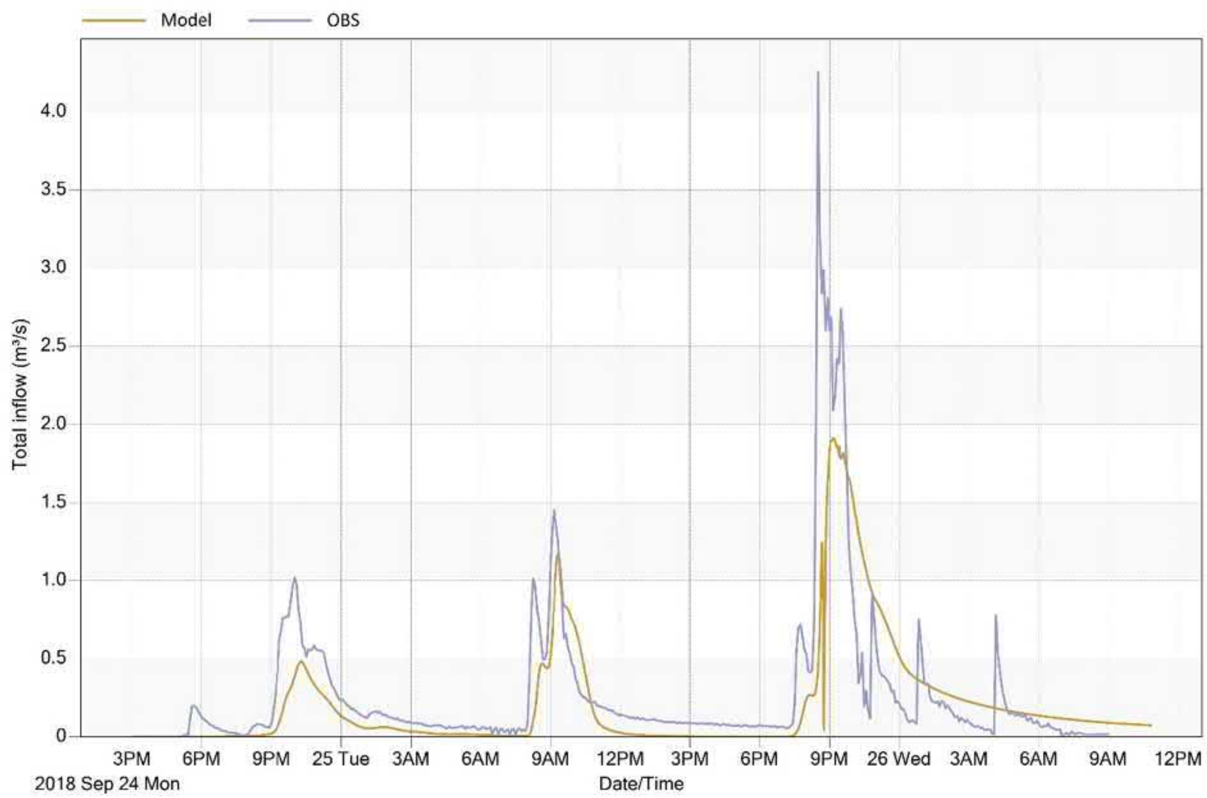


Figure 7.12 Validation Results – September 25, 2018 at FM03

As shown in Figure 7.12, the predicted runoff volume is 22.9% lower than the observed and the predicted peak flow is 54.8% lower than the observed. The model fit well with the timing and shape of the peak hydrograph except for the observed small spikes at the end of the storm event. This could be due to a number of factors including manual operating adjustments at the station during the storm event, the accuracy of the Lesperance pump station operating characteristics entered in model or the pumps within the station working at a higher efficiency than the design curve. As a result, the calibration results at FM03 are considered good to average for the September 25, 2018 storm.

As summarized in Table 7.3, the validation results indicate a more accurate volume matching for the September 25, 2018 event than September 20, 2018 event for the exception of FM03. As the calibration exercise was completed for more frequent rainfall events in proximity to a 1:2 year, model results from storm events greater than this, such as the September 20, 2018 event, are expected to have different hydrologic parameters taken into consideration during the calibration process. It is expected that the September 20, 2018 event would be further away from the minor system calibration completed as part of this study.

Table 7.3 Validation Volume and Peak Flow Comparison

FM/ Event	VOLUME(m^3)					PEAK FLOW(m^3/s)				
	Observed Monitored Data	Model Before Calibration	Percent Difference Before Calibration (%)	Model After Calibration	Percent Difference After Calibration (%)	Observed Monitored Data	Model Before Calibration	Percent Difference Before Calibration (%)	Model After Calibration	Percent Difference After Calibration (%)
FM01/ SEP20 2018	3534	11910	237	5560	57.3	0.57	1.63	186	0.51	-10.5
FM01/ SEP25 2018	6769	15180	124.3	8235	21.7	1.37	2.18	58.9	1.03	-24.9
FM02/ SEP20 2018	4612	7435	59.7	6930	50.3	0.86	0.53	-38.4	0.53	-38.4
FM02/ SEP25 2018	8071	10530	30.5	9594	18.9	0.91	0.59	-35.2	0.59	-35.2
FM03/ SEP20 2018	25320	56170	121.8	27180	7.3	2.93	3.32	13.3	1.94	-33.8
FM03/ SEP25 2018	42370	71930	69.8	32670	-22.9	4.25	4.39	3.3	1.92	-54.8

Note: Modelled Runoff Error of 0 and Model Routing Error ranging from 4.8% to 6.1% during validation.

7.4 Summary

Based on the flow monitoring program and refined model calibration/validation completed as part of this study, the overall results may be characterized as follows:

- FM01: Good Level of Calibration/Validation
  - Volume: 3 of 4 good calibrated/validated events
  - Peak flow: 3 of 4 good calibrated/validated events
- FM02: Average Level of Calibration/Validation
  - Volume: 2 of 4 good calibrated/validated events
  - Peak flow: 2 of 4 good calibrated/validated events
- FM03: Average Level of Calibration/Validation
  - Volume: 2 of 4 good calibrated/validated events
  - Peak flow: 2 of 4 good calibrated/validated events

A more refined existing conditions model was accurately calibrated for frequent storm events at or under a 1:2 year storm through the model calibration/validation process. With the Town completing ongoing sanitary system studies to further detail solutions to reduce I&I to the sanitary system and connect these sources from the private side back to the storm sewer network, the refined calibrated model would potentially represent an underestimation of the runoff volume entering the storm sewer system. The refined model was therefore used in this study to develop a more accurate assessment of the hydraulic conditions through the storm sewer system during smaller storm events and not to design surface flooding solutions during larger storm events.

The refined model recognizes the existing hydraulic water level conditions within the storm system throughout the study area and was therefore used for the following illustrations as part of this study:

- Existing Hydraulic Gradeline (HGL) conditions within the storm sewer system during a 1:2 year storm event; and
- Hydraulic Gradeline (HGL) conditions within the storm sewer system during a 1:2 year storm event with the implementation of all recommended surface flooding solutions identified through this study.

The future conditions model identifies the improvements to the conveyance of flow through the storm sewer system during the 1:2 year storm event based on the surface flooding solutions recommended.

As discussed in Section 7.1, the refined model is recommended to be used, in certain areas, to determine the allowable release rates for future infill development areas where the re-connection of sources of inflow on the private side back to the storm sewer system is not feasible.

8.0 Existing Condition Modelling Results

As discussed in previous sections, the original model developed from calculated watershed hydrologic values was to be used to assess existing surface ponding conditions and identify surface flooding problem areas where surface ponding is exceeding the regionally acceptable level of 0.30 m during the 1:100 year event. Any surface ponding depths beyond the regionally accepted level of 0.30 m was considered a surface flooding problem area which required further review to determine if an economic solution is feasible.

The surface flooding problem areas for the 1:100 year storm event were delineated and separated between those on the west and east sides of Manning Road for ease of reference. The surface flooding problem areas were determined based on either a regional surface flooding problem area or an isolated surface flooding problem area, as described below:

- *Regional Surface Flooding Problem Area*: Where the surface flooding was widespread throughout a large portion of the service area and surface flooding was dispersed over long lengths of roadway, or flooding was caused by a more regional issue (ie. trunk sewer capacity constraints, pump station capacities, regional surface grading). Localized issues are also identified within the problem area, such as bottlenecks and lack of overland flow routes; and
- *Isolated Surface Flooding Problem Area:* Surface flooding is more isolated in nature, where the spread of surface flooding is maintained within small portions of a roadway, or caused by only a localized issue (ie. roadway grading, sewer bottleneck, inlet capacity).

The following sections outline the analysis results for the study area, including the determination of surface flooding problem areas for the 1:100 year event and the hydraulic analysis of the municipal storm sewer system for the 1:2 year storm event from the refined calibrated model developed and discussed in Section 7.4.

8.1 Modelling Analysis - West of Manning Road/County Road 19

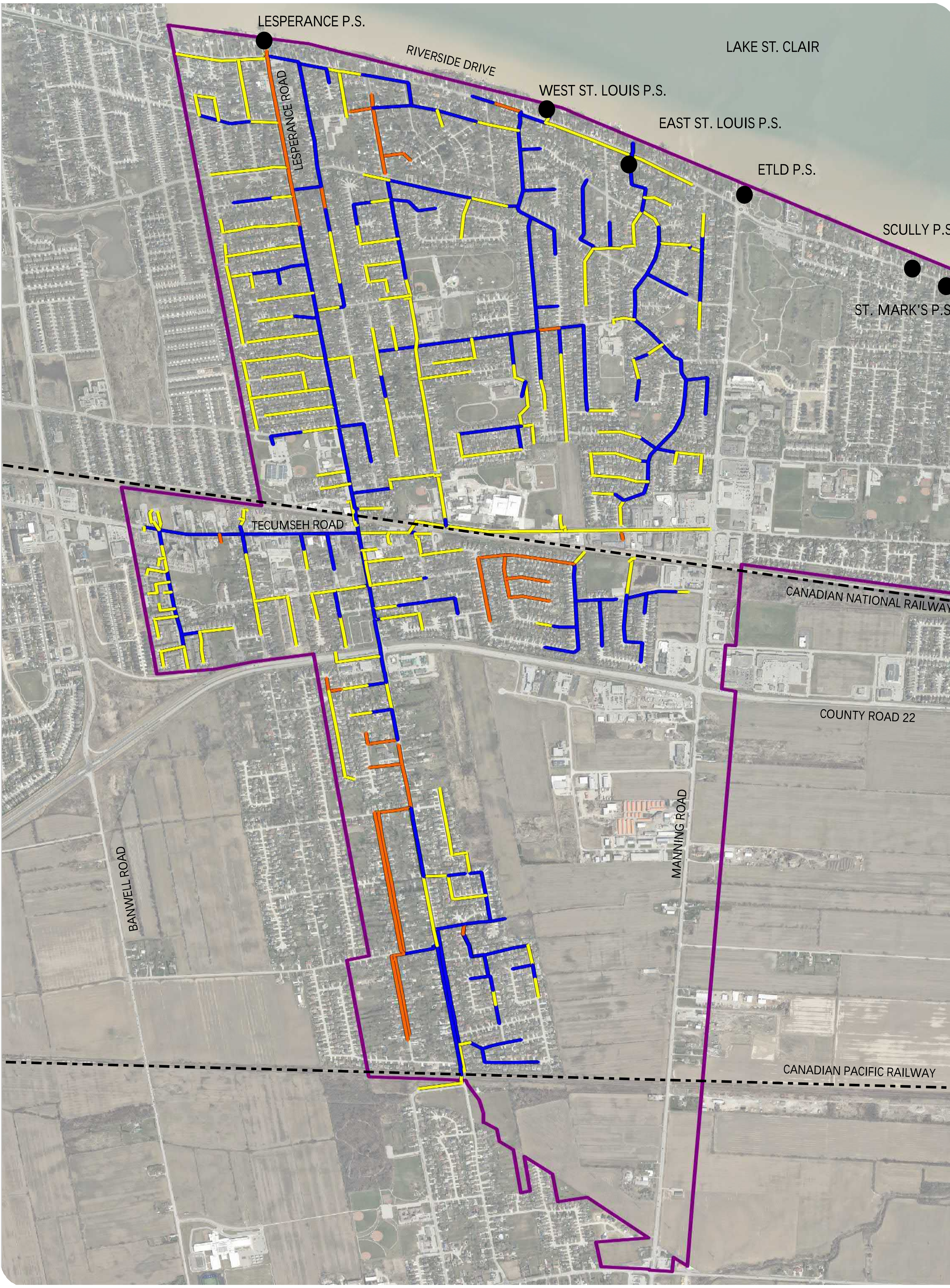
Under existing conditions, the Lesperance and West St. Louis pump station service areas have a storm sewer interconnection at the intersection of St. Pierre Street and Riverside Drive. The two service areas work together during larger storm events, with runoff (both minor and major system) contributing to both systems. The majority of the localized sewers within the West St. Louis pump station area connect to the Little River storm sewer at Michael Drive. The storm trunk sewer continues north along Michael Drive and through an existing easement between two residential properties. Flows are then conveyed east along Riverside Drive to the West St. Louis pump station.

8.1.1 Existing Conditions – Calibrated Minor System Analysis

The results from the refined calibrated model developed for a minor system analysis and discussed in Section 7.4 was used to illustrate the existing hydraulic gradeline (HGL) throughout the municipal infrastructure system. The estimated HGL for the minor storm sewer system in the Town of Tecumseh was assessed under a 1:2 year design storm event. A graphic representation of the water levels within the storm sewer was developed. The depth of flow conveyance within the storm sewer system was summarized in which the HGL elevation during the 1:2 year event was:

1. Maintained within the storm sewer with the sewer flow below 50% full;
2. Maintained within the storm sewer with the sewer flow between 50% and 99% full; and
3. Beyond the obvert of the pipe, but within regional design standards for storm sewer design (ie. no higher than 0.30m below the existing ground elevation).

Figure 8.1 illustrates the existing condition HGL results during the 1:2 storm events from the refined model for each of the storm sewer systems modelled within the study area west of Manning Road.



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

WEST OF MANNING ROAD EXISTING CONDITIONS ESTIMATED 1:2 YEAR STORM SEWER HYDRAULIC CONDITIONS
FIGURE 8.1

TOWN OF Tecumseh ONTARIO · CANADA

SEWER GRAVITY OUTFALL	CONDUIT FLOW < 50% FULL
PUMP STATION (P.S.)	CONDUIT FLOW BETWEEN 50% - 99% FULL
STUDY AREA	CONDUIT SURCHARGING ABOVE PIPE OBVERT WITHIN REGIONAL STANDARD LIMITS
RAILWAY	

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MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

8.1.2 Major System Analysis – 1:100 Year Surface Flooding Problem Area W-1

The majority of runoff through this problem area is conveyed through one trunk storm sewer along Lesperance Road to the Lesperance pump station at Riverside Drive.

Based on the existing condition 1:100 year storm event analysis, the Lesperance pump station causes a high tailwater condition on the upstream storm sewer system, which is expected during larger storm events. Flows back up through the storm sewers, surcharging both the storm trunk sewer and adjacent local sewers. This surcharging of the system within the minor system in turn causes an increase in surface ponding depths in low points along the roadways where major system overland flow routes are not viable due to local grading constraints and flat topography of the area. This is considered typical in our area where the storm pump stations are considered as the only outlet for the service areas into the respective ultimate drainage outlet. This appears to occur along several streets west of Lesperance Road. Other causes of surface flooding throughout the service area include roadway grading low points with a lack of an overland flow route to the downstream system.

The West St. Louis pump station also causes a high tailwater condition on the storm sewer system during larger storm events that is causing flows to back up through the storm sewers and increasing HGL's within both the storm trunk sewer and adjacent local sewers.

Figure 8.2 illustrates the extent of surface ponding for the 1:100 year event along the Lesperance and West St. Louis pump station service areas, which is identified as Regional Surface Flooding Problem Area W-1. Isolated surface flooding problem areas are also identified.

8.1.3 Major System Analysis - 1:100 Year Surface Flooding Problem Area W-2

Based on the 1:100 year design storm existing condition model simulation, the East St. Louis pump station service area was identified to have the least amount of surface flooding throughout the service areas west of Manning Road. The main storm trunk sewer is along Green Valley Drive, which conveys flows north to Dillon Drive and through an easement at the cul-de-sac along Shannon Place between two residential properties, ultimately discharging to the East St. Louis pump station.

The upstream portion of Green Valley Drive was identified to have the greatest extent of surface flooding within this service area.

Based on the existing condition 1:100 year design storm modelling analysis, the East St. Louis pump station is identified as not causing a high enough tailwater condition at the outlet on the storm sewer system to cause a significant HGL increase in the system to effect surface flooding within the service area. The problem area during the 1:100 year event is identified in the upstream portions of the service area, which is caused by a number of constraints within the system, including roadway grading low points, a lack of an overland flow route to the downstream system, and a potential bottleneck within the storm sewer system at the intersection of Green Valley Drive and St. Thomas Street. This is considered typical in our area where the storm pump stations are considered as the only outlet for the service areas into the respective ultimate drainage outlet. These factors in turn cause an increase in surface ponding depths in the upper ends of the system.

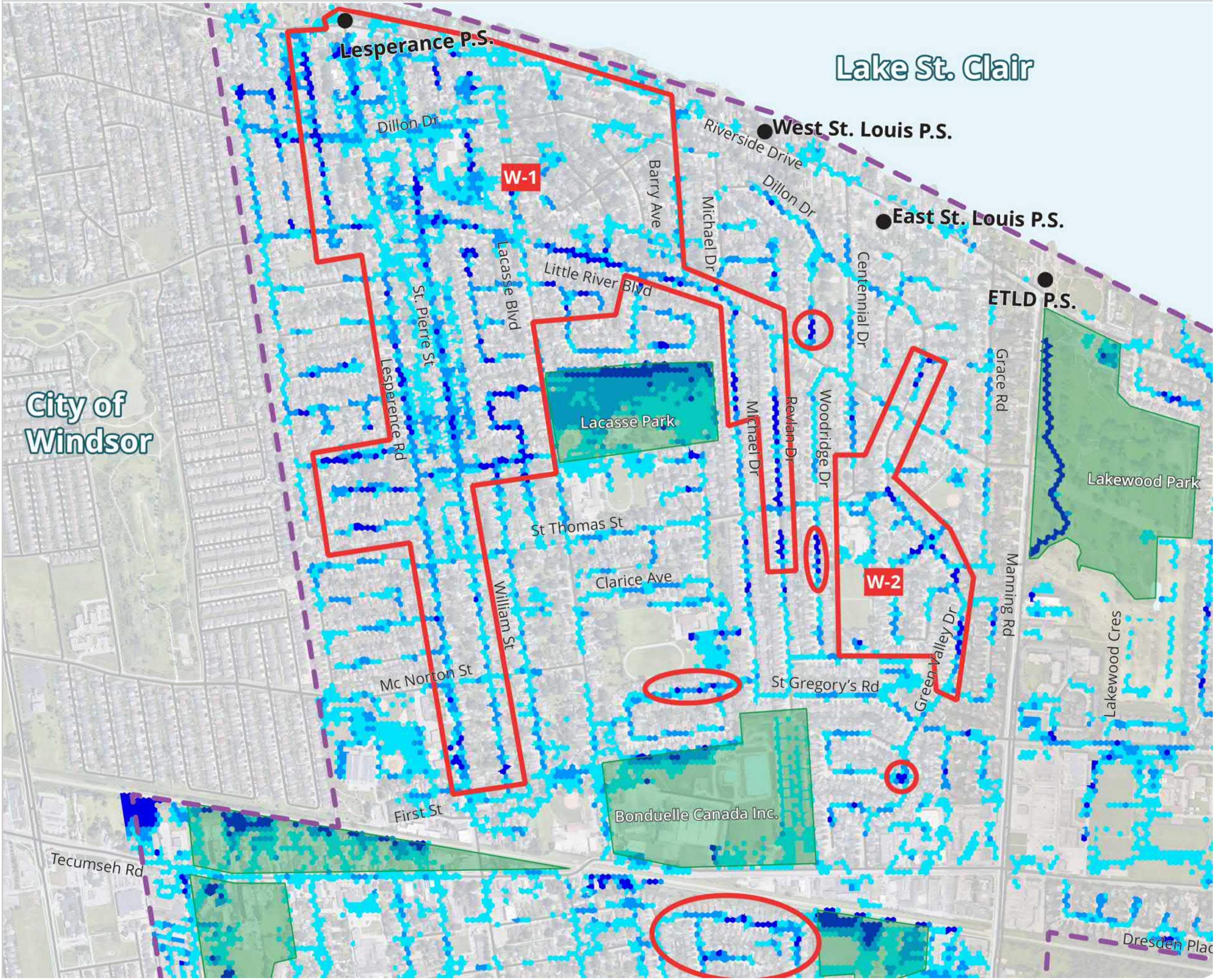
Figure 8.2 illustrates the extent of surface ponding along the East St. Louis pump station service area, which is identified as Regional Surface Flooding Problem Area W-2. Isolated surface flooding problem areas are also identified.

8.1.4 Major System Analysis - 1:100 Year Surface Flooding Problem Area W-3

Based on the 1:100 year design storm existing condition model simulation, the Baillargeon Drain service area was identified to have surface flooding along Lesperance Road and within the existing residential development east of Lesperance Road. The Baillargeon Drain ultimately discharges into the East Townline Drain where runoff is conveyed north to the East Townline Drain pump station.

Based on the existing condition 1:100 year modelling analysis, the majority of the surface flooding along Lesperance Road is caused by a lack of an overland flow route for a localized low point between North Pacific Avenue and Intersection Road. Surface flooding relief from this low point is currently to the west through existing residential properties. The more localized surface flooding areas along the residential development east of Lesperance Road are caused by a number of potential constraints within the drainage system, including roadway grading low points and a lack of overland flow route to the downstream system. These factors in turn cause an increase in surface ponding depths in the upper ends of the system.

Figure 8.3 illustrates the extent of surface ponding in the Baillargeon Drain service area, which is identified as Regional Surface Flooding Problem Area W-3. Isolated surface flooding problem areas are also identified.



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TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

SURFACE FLOODING PROBLEM AREAS - W-1 & W-2
FIGURE 8.2

- PUMP STATION (P.S.)
- ISOLATED SURFACE FLOODING PROBLEM AREAS
- STUDY AREA
- PARKLAND/ PRIVATE PROPERTY NOT TO BE ANALYZED
- STREETS
- REGIONAL SURFACE FLOODING PROBLEM AREAS

1:100 SURFACE PONDING
- LESS THAN 0.15m DEPTH
- BETWEEN 0.15m - 0.30 m DEPTH
- OVER 0.30m DEPTH

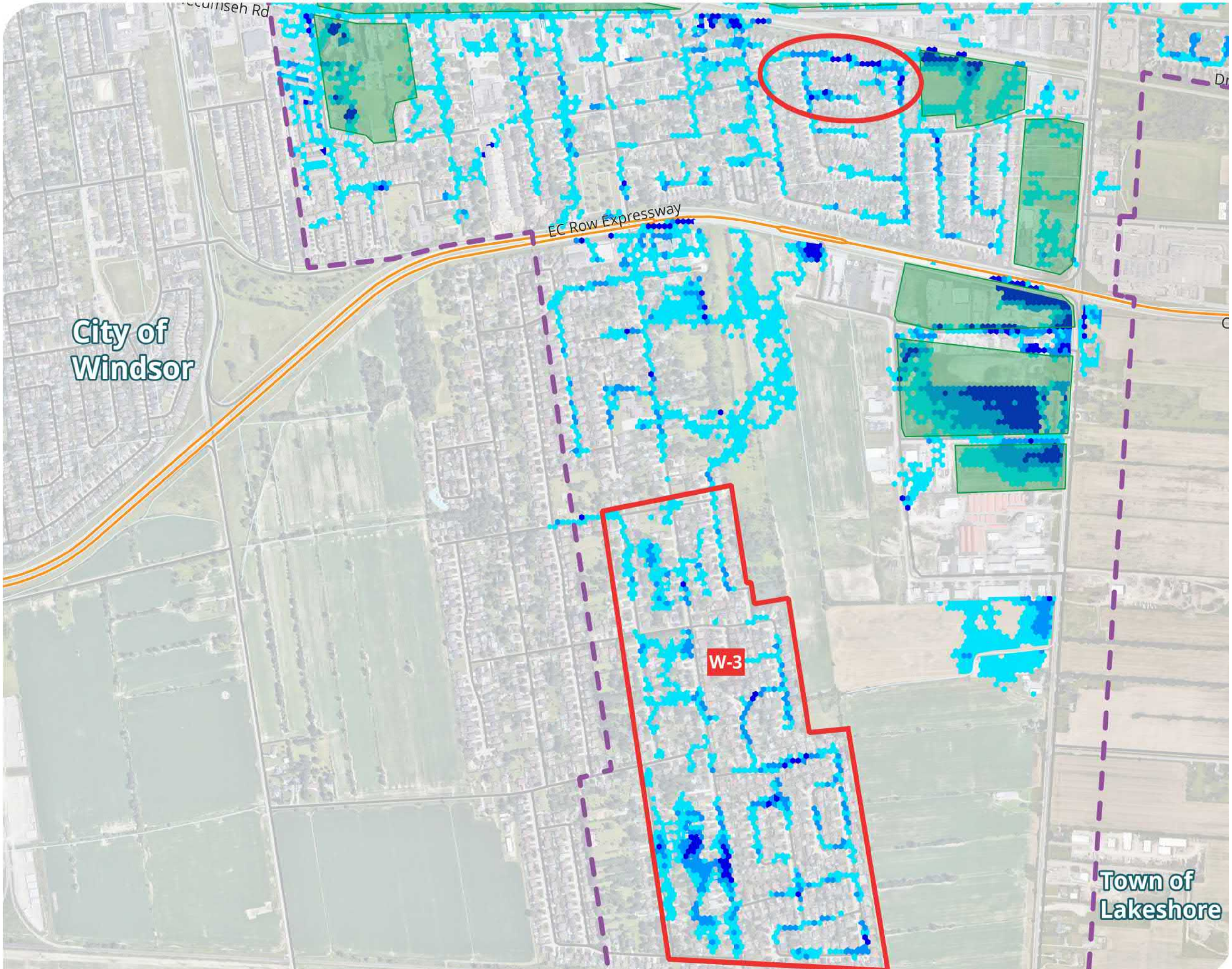
SCALE 1:NTS

2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

DILLON CONSULTING

PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019



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STORM DRAINAGE MASTER PLAN

SURFACE FLOODING PROBLEM AREAS - W-3
FIGURE 8.3

ISOLATED SURFACE FLOODING PROBLEM AREAS
STUDY AREA
PARKLAND/ PRIVATE PROPERTY NOT TO BE ANALYZED
STREETS
REGIONAL SURFACE FLOODING PROBLEM AREAS

1:100 SURFACE PONDING
LESS THAN 0.15m DEPTH
BETWEEN 0.15m - 0.30 m DEPTH
OVER 0.30m DEPTH

SCALE 1:NTS 2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

DILLON CONSULTING

PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019

8.1.5 Major System Analysis - 1:100 Year Isolated Surface Flooding Problem Areas

Several isolated surface flooding areas have been identified throughout the pump station service areas west of Manning Road, as identified in Figure 8.2 and Figure 8.3. The majority of the isolated surface flooding arises from local grading constraints and a lack of overland flow routes, along with inlet capacity constraints into the storm sewer system.

One of the main isolated surface flooding problem areas is identified in Figure 8.3, in which surface runoff along the roadway is utilizing lower lying private property elevations as an overland flow route to the VIA Rail railway ditch. This isolated surface flooding problem area was initially identified during the existing conditions modelling to warrant a more extensive analysis for a localized surface flooding solution.

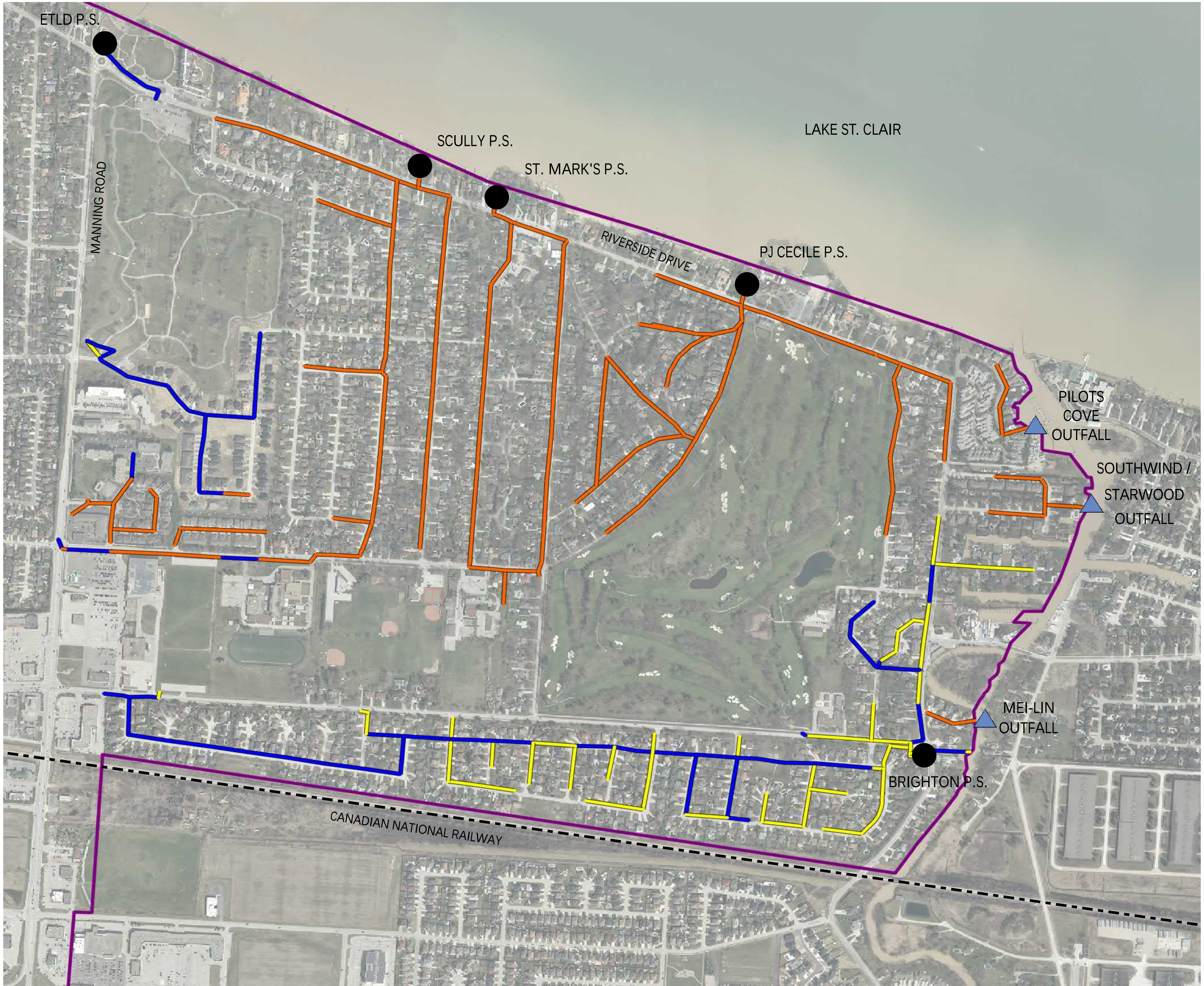
8.2 Modelling Analysis - East of Manning Road/County Road 19

8.2.1 Existing Conditions - Calibrated Minor System Analysis

The results from the refined calibrated model developed for a minor system analysis and discussed in Section 7.4, was used to illustrate the existing hydraulic gradeline (HGL) throughout the municipal infrastructure system. The estimated HGL for the minor storm sewer system in the Town of Tecumseh was assessed under a 1:2 year design storm event. A graphic representation of the water levels within the storm sewer was developed. The depth of flow conveyance within the storm sewer system was summarized in which the HGL elevation during the 1:2 year event was:

1. Maintained within the storm sewer with the sewer flow below 50% full;
2. Maintained within the storm sewer with the sewer flow between 50% and 99% full; and
3. Beyond the obvert of the pipe, but within regional design standards for storm sewer design. (ie. no higher than 0.30m below the existing ground elevation)

Figure 8.4 illustrates the existing condition HGL results during the 1:2 storm events from the refined model for each of the storm sewer systems modelled within the study area east of Manning Road.



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EAST OF MANNING ROAD - EXISTING CONDITIONS ESTIMATED 1:2 YEAR STORM SEWER HYDRUALIC CONDITIONS
FIGURE 8.4

▲	SEWER GRAVITY OUTFALL
●	PUMP STATION (P.S.)
yellow line	CONDUIT FLOW < 50% FULL
blue line	CONDUIT FLOW BETWEEN 50% - 99% FULL
orange line	CONDUIT SURCHARGING ABOVE PIPE OBVERT WITHIN REGIONAL STANDARD LIMITS
dashed line	RAILWAY
purple box	STUDY AREA

SCALE 1:NTS 2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

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PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019

8.2.2 Major System Analysis - 1:100 Year Surface Flooding Problem Area E-1

Based on the existing condition 1:100 year storm event analysis, the surface flooding problem area was delineated over three pump station service areas; the Scully, St. Mark's and PJ Cecile pump station areas. These three service areas consist of, for the most part, roadways with semi-urban and rural cross sections where the drainage network consists of roadside ditches. Based on previous discussions with the Town and completed studies for this area east of Manning Road, Edgewater Drive, St. Mark's Road, Arlington Boulevard, their connecting adjacent streets and the streets within the Kensington Dish area are proposed to be reconstructed. As part of these previously completed studies, a number of alternative solutions have already been documented, including pump station improvements.

The majority of the surface flooding is due to the limited capacity of the existing shallow roadside ditches. The urbanized roadway cross section areas west of Edgewater Drive along Grant Avenue and Cada Crescent have surface flooding due to a lack of overland flow routes, as well as a bottleneck in the storm sewer system along Hayes Avenue at the Grant Avenue storm inlet where the storm sewer is currently aligned through a maintenance easement between two residential properties. Lack of overland flow routes are considered typical in our area where the storm pump stations are considered as the only outlet for the service areas into the respective ultimate drainage outlet due to the flat topography.

Based on previously completed studies, it was determined that to reconstruct some roadways east of Manning Road to a more traditional curb and gutter roadway cross section, pump stations would need to be improved. The modelling has identified that under existing conditions, the Scully pump station is causing a high tailwater condition on the municipal storm sewer system within the Grant Avenue area during larger storm events. This is expected during larger storm events which ultimately cause flows to back up through the storm sewers and increase the HGL's within the local sewers. This in turn causes an increase in surface ponding depths in lower roadway areas.

Figure 8.5 identifies the extent of surface ponding along the Scully, St. Mark's and PJ Cecile pump station service areas, which is identified as Regional Surface Flooding Problem Area E-1. Isolated surface flooding problem areas are identified as well.

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SURFACE FLOODING PROBLEM AREAS - E-1
FIGURE 8.5

PUMP STATION (P.S.)
SEWER GRAVITY OUTFALLS
ISOLATED SURFACE FLOODING PROBLEM AREAS
STUDY AREA
PARKLAND/ PRIVATE PROPERTY NOT TO BE ANALYZED
STREETS
REGIONAL SURFACE FLOODING PROBLEM AREAS

1:100 SURFACE PONDING
LESS THAN 0.15m DEPTH
BETWEEN 0.15m - 0.30 m DEPTH
OVER 0.30m DEPTH

SCALE 1:NTS 2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

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PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019

8.2.3 Major System Analysis - 1:100 Year Isolated Surface Flooding Problem Areas

A number of isolated surface flooding areas have been identified throughout the pump station service areas east of Manning Road, as illustrated in Figure 8.5. The majority of the isolated surface flooding is caused by local grading constraints and inlet capacity constraints into the storm sewer system. Some of the areas experience high HGL elevations throughout the localized storm sewer system, causing increased ponding depths.

One of the main isolated surface flooding problem areas is identified in Figure 8.5 within the Southwind Crescent/Starwood Lane area. This development is currently serviced by a gravity storm sewer outlet into Pike Creek. Under the historic high water level simulation for the 1:100 year design storm event, surface flooding is identified along Southwind Crescent due to the tailwater condition caused by high lake levels. This isolated surface flooding problem area is considered to warrant a more extensive analysis for a localized surface flooding solution.

8.3 Existing Condition Surface Ponding Summary

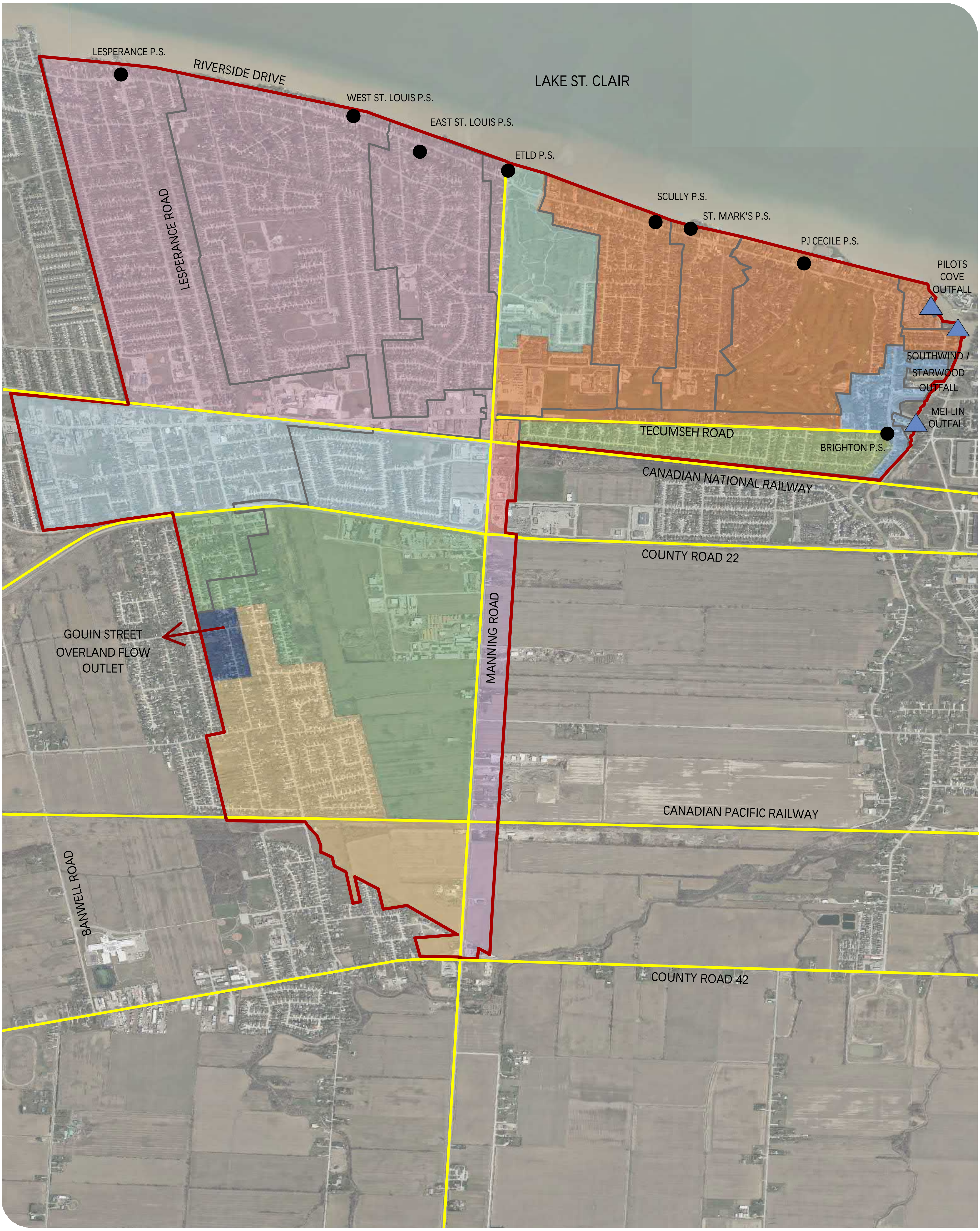
Beyond identifying the existing condition surface flooding problem areas for the 1:100 year, 4 hour storm event, a review of surface ponding throughout municipal roadways for the other design storm events was completed using the original existing conditions model based on calculated hydrologic watershed values. This included an assessment of surface ponding during the following design storm events:

- 1:5 Year 4 Hour Storm Event;
- 1:10 Year 4 Hour Storm Event;
- 1:100 year 4 Hour +40% Storm Event (115mm rainfall with a maximum intensity of 241mm/hr); and
- 1:100 year 24 hour +39% Storm Event (150mm rainfall with a maximum intensity of 145mm/hr).

Surface ponding during these events are outlined in Appendix C.

8.4 Major System Drainage Areas

Through a review of the 1:100 year 4 hour storm event existing conditions model results, a major system drainage area plan was developed to identify the extent of overland flow contributions from one service area to another. Barrier landforms throughout the study area were identified to restrict overland flow conveyance downstream. The major system drainage area plan is provided in Figure 8.6.



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EXISTING CONDITION MAJOR SYSTEM DRAINAGE AREA PLAN

FIGURE 8.6

TOWN OF Tecumseh ONTARIO · CANADA

PUMP STATION (P.S.)	MAJOR SYSTEM DRAINAGE AREA 2
SEWER GRAVITY OUTFALL	MAJOR SYSTEM DRAINAGE AREA 3
PUMP STATION SERVICE AREA BOUNDARIES	MAJOR SYSTEM DRAINAGE AREA 4
MAJOR SYSTEM FLOW BARRIER LANDFORMS	MAJOR SYSTEM DRAINAGE AREA 5
STUDY AREA	MAJOR SYSTEM DRAINAGE AREA 6
MAJOR SYSTEM DRAINAGE AREA 1	MAJOR SYSTEM DRAINAGE AREA 7
MAJOR SYSTEM DRAINAGE AREA 8	MAJOR SYSTEM DRAINAGE AREA 9
MAJOR SYSTEM DRAINAGE AREA 10	MAJOR SYSTEM DRAINAGE AREA 11

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MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

9.0 Surface Flooding Solution Decision Making Framework

The Storm Drainage MP has identified solutions that would improve the resiliency of the storm drainage infrastructure, taking into consideration the impacts of climate change. Based on the assessment of the current and future level of service standards in the Town of Tecumseh for storm infrastructure and roadway overland flow design, a surface flooding solution decision framework was developed. This framework provides for a more adaptive level of design approach to developing solutions that address both the required level of service and an added resiliency for each surface flooding problem area, as appropriate.

The framework was developed to determine the scope of the preferred design solution and identify areas that require either a traditional or an adaptive level of service having added resiliency, as required to suit the risks and vulnerability of the area.

Traditional Design Solution

- Static design criteria established by regulatory agencies; and
- Standard level of service and flood risk mitigation.

Adaptive Design Solution

- Assessment and review of higher risk areas susceptible to surface flooding and provide a flexible and sustainable solution that accounts for a reasonable degree of uncertainty due to climate change; and
- Enhanced and variable level of service and flood risk mitigation for the respective higher risk surface flooding areas.

Figure 9.1 illustrates the decision framework used to determine the scope of the preferred solutions. The design process includes a required climate change analysis of the proposed design in areas where surface flooding is problematic. As identified in Section 5.5.1, the initial climate change event using the Chicago 1:100 year 4 hour + 40% storm was used to complete a more extensive analysis to review whether a surface flooding solution warranted an enhanced level of service or not. The urban stress test from the Windsor/Essex Region Stormwater Management Standards Manual (December 2018) was then analyzed under the preferred solution and the design was adjusted as needed.

Although recommended solutions will improve the risk and impact of surface flooding, some private and public properties in lower-lying areas may still be susceptible to localized flooding during extreme rainfall events.

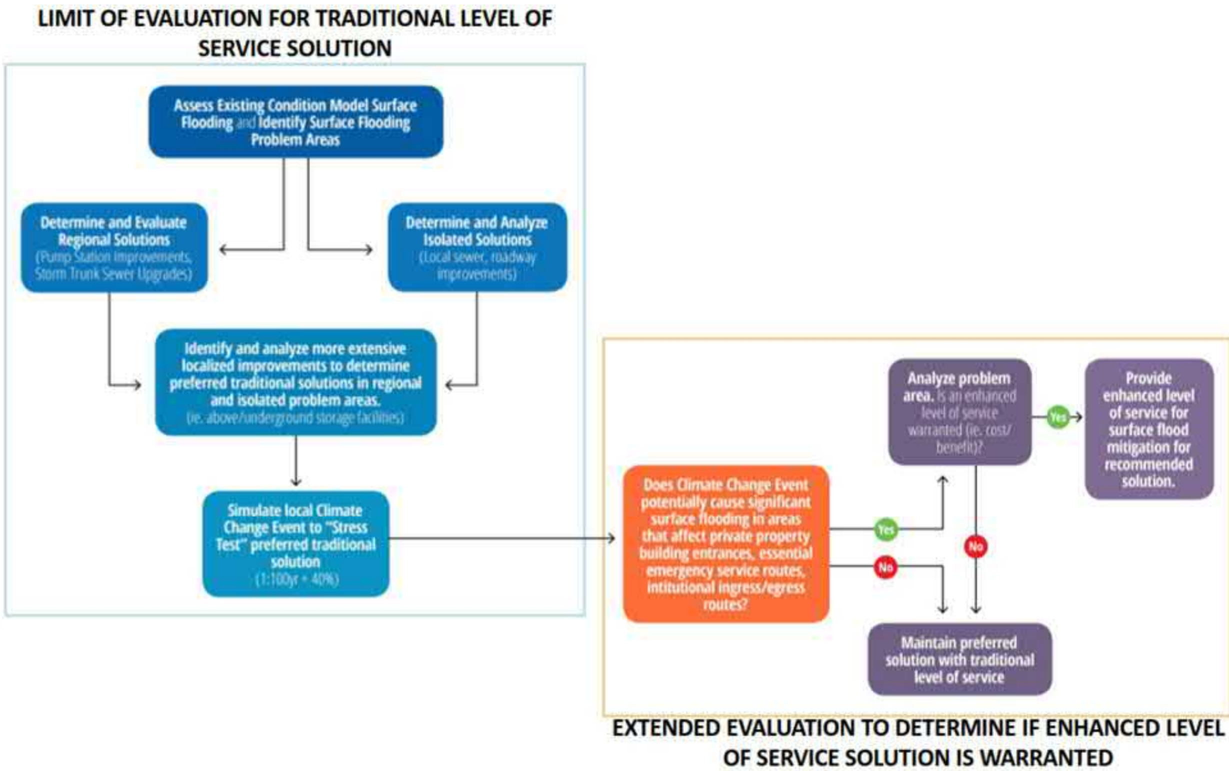


Figure 9.1: Surface Flooding Solution Decision Framework

10.0 Future Conditions Alternative Solutions and Evaluation

During the future condition modelling analysis, the model was adjusted to account for known future developments and road and sewer reconstruction projects proposed within the study area. The model was used to re-analyze each surface flooding problem area and determine alternative regional surface flooding solutions. The surface flooding solutions were developed using the decision framework outlined in Section 9.0 through model simulation of both the 1:100 year storm event, as well as the climate change analysis to determine whether the solution warranted either a traditional or adaptive analysis for a more enhanced level of service.

Localized surface flooding solutions were also developed in more isolated surface flooding problem areas for the 1:100 year, 4 hour storm event. The decision framework was then used to identify localized areas which may warrant an enhanced level of service.

10.1 Future Conditions Model Development

As a preliminary effort to construct the future conditions model, the project team first incorporated all known future private development and municipal roadway and storm sewer improvements into the calibrated existing conditions model. This was completed to simulate a future condition scenario where the known areas of development or roadway/sewer reconstruction were taken into consideration when designing the regional and localized surface flooding solutions. The following was incorporated in the model:

Future Private Development

- 103 hectare development consisting of a mixture of residential, commercial, and institutional development referred to as the Manning Road Secondary Planning Area;
- SWM design as per the "Town of Tecumseh MRSPA SWM Environmental Study Report Addendum (April 2015) which assumed the redirection of the Baillargeon Drain into the MRSPA storm trunk sewer and proposed SWM Facility; and
- One SWM facility with a controlled 500 L/s pump release rate into the East Townline Drain for the proposed development.

Two of the larger property owners within the MRSPA expressed their desire to review the SWM strategy for the MRSPA development to consider alternative solutions that may vary from the approved MRSPA SWM Class EA Addendum Report (2015). Once the developers have initiated the detailed design of the SWM solution for this site, the PCSWMM Storm Drainage MP Model may be used to confirm that the development does not result in any adverse impacts on the downstream drainage system.

Future Municipal Roadway and Sewer Reconstruction

Based on discussions with the Town, proposed municipal roadway and storm sewer reconstruction areas have been identified within the study area. These municipal roadway and sewer reconstruction improvements impact the amount of runoff to the respective outlets and were taken into consideration when designing each alternative surface flooding solution.

As identified in Section 3.6 and illustrated in Figure 3.4, the following areas have been identified by the Town of Tecumseh as roadways with semi-urban cross sections that in the future are to be reconstructed with associated sewer improvements.

1. Streets within the “Kensington Dish” Area;
2. Streets within the “Coronado Dish” Area;
3. Arlington Boulevard;
4. St. Marks Road;
5. Edgewater Boulevard;
6. St. Anne Area including:
    - St. Anne Street between North Pacific Avenue and Gouin Street;
    - Portions of North Pacific Avenue, Intersection Road, Maisonneuve Street and Gouin Street within the study area;
7. Tecumseh Road Storm Sewer Extension; and
8. Enclosure of the East Townline Drain, incorporation of a new local Manning Road sewer and re-direction of flow to the existing Lakewood Park Drainage Channel (hereafter referred to as Manning Road Phase 2 Drain Enclosure).

Previously completed studies included the following storm sewer improvements that have been incorporated in the future conditions model:

- Storm sewer upgrades to provide 1:5 year level of service for the PJ Cecile Strom Pump Station (Peter Cecile (PJ Cecile) Storm Pump Station – Review of Drainage Area and Contributing Flow (Dillon, September 2016));
- Storm sewer upgrades to provide 1:5 year level of service for St. Mark’s and Scully Storm Pump Stations (Town of Tecumseh St. Mark’s and Scully Storm Pump Stations – Review of Drainage Areas and Storm Servicing Alternatives (Dillon, August 2016));
- Storm sewers designed for the 1:2 year level of service along Lesperance Road (1997). The allowable release rates from the St. Anne Area to the Lesperance storm sewer was limited to the capacity of the receiving sewer and based on the initially determined capacity from the previously noted study;
- Storm sewer Design for Tecumseh Road (Brighton Road Reconstruction and Pump Station Design in (Dillon, 2007)); and
- Manning Road Phase 2 and Phase 3 - Stormwater Analysis Technical Memo (October 2018).

Based on the storm sewer design criteria for future development areas, the proposed storm sewer design for the Coronado Dish Area is to be designed for a 1:5 year level of service.

10.2 Future Conditions Evaluation

Each storm sewer network noted above was further analyzed for the 1:100 year storm event and both climate change events and designed accordingly for either a traditional or adaptive level of service design based on the decision framework, as outlined in Figure 9.1 and detailed in Section 9.0.

10.3 Solutions Development

Based on the regional and isolated surface flooding problem areas identified in Section 8.0, several solutions were assessed for the surface flooding problem areas based on the existing conditions analysis and the incorporation of the future conditions identified above. The surface flooding solutions for the study were divided into two types: regional solutions and localized solutions, as identified in Table 10.1.

Table 10.1: Initial Surface Flooding Solution Alternatives

Regional Surface Flooding Solutions	Localized Surface Flooding Solutions
Storm Trunk Sewer Upgrades	Local Storm Sewer Upgrades
Pump Station Upgrades	Aboveground/Underground Relief Storage
Redirection of Storm Drainage	Roadway Grading
Sewer Overflows	Backflow Prevention
	Catch basin Inlet Improvements

10.4 West of Manning Road/CR19 Alternative Surface Flooding Solutions

The alternative solutions for each of the surface flooding problem areas west of Manning Road/CR19 have been separated into the respective pump station service areas. Alternative regional surface flooding solutions were determined for each of the following:

- Regional Problem Area W-1: Lesperance Pump Station Service Area (Storm sewer system interconnected with West St. Louis Pump Station Service Area)
- Regional Problem Area W-1: West St. Louis Pump Station Service Area (Storm sewer system interconnected with Lesperance Pump Station Service Area)
- Regional Problem Area W-2: East St. Louis Pump Station Service Area
- Regional Problem Area W-3: Baillargeon Drain Area (East Townline Drain Pump Station Service Area)

This section provides details of the proposed improvements within each problem area for each alternative.

10.4.1 Problem Area W-1 Solutions – Lesperance Pump Station Service Area

As identified previously, the Lesperance and West St. Louis Pump Station service areas are interconnected by a shared manhole along Riverside Drive, directly east of St. Pierre Street. Under each alternative, it is proposed to remove this interconnection.

The existing Lesperance storm pump station was identified as requiring capacity improvements. This was recommended for all alternatives evaluated within the regional problem area.

Four alternatives were considered for Problem Area W-1:

1) Do nothing
2) Alternative 1: Improvements to the Lesperance Trunk Sewer and Pump Station
3) Alternative 2: Improvements to St. Pierre Trunk Sewer and Pump Station (RECOMMENDED)
4) Alternative 3: Improvements to St. Pierre and Lesperance Trunk Sewers and Lesperance Pump Station

Pump station improvements considered for all alternatives include improving the pump capacity of the existing station. Increasing the pump capacity was considered to improve the level of service in the catchment area. Two options for pump capacity were considered; increasing the pump capacity to a traditional level of service (8 m^3/s) or increasing the pump capacity to an enhanced level of service (9 m^3/s).

Based on the analysis of the service area and the decision framework for the solutions, the Lesperance storm pump station was determined to warrant an enhanced level of service with a firm pump capacity upgrade to 9 m^3/s.

The alternatives for this problem area are further illustrated in Figure 10.1.

10.4.1.1 Do Nothing

Under this alternative, no regional or localized solutions are proposed within the problem area where widespread surface flooding is identified. The evaluation of this alternative is required by the EA process; however as part of this study, the "Do Nothing" approach does not reduce surface flooding in the identified problem areas. With the risk of more intense storm events in the future and the aging infrastructure in the problem area, this alternative is not recommended.

10.4.1.2 Alternative 1: Lesperance Trunk Sewer and Pump Station Improvements

This alternative scenario includes improvements to the existing Lesperance storm pump station and the existing Lesperance storm trunk sewer from St. Jacques Street directly north of the VIA railway to the pump station at the intersection of Riverside Drive and Lesperance Road. For this alternative, the existing Lesperance storm trunk sewer is proposed to be upgraded from an 1800 mm – 1950 mm diameter circular storm sewer to a 2400 mm – 3600 mm diameter circular storm sewer. The existing storm sewer connections along the east of Lesperance Road from the St. Pierre storm sewer system to the Lesperance storm trunk sewer are to be improved as follows:

- 600 mm diameter sewer from Clapp Street upgraded to a 975 mm diameter;
- 600 mm diameter sewer from Wood Street upgraded to a 975 mm diameter; and
- 675 mm diameter sewer from Riverside Drive upgraded to a 1350 mm diameter.

As part of this regional solution, additional localized solutions are proposed within the regional surface flooding area to provide the added resiliency to improve surface flooding in the service area. This includes:

- Storm sewer conveyance, roadway grading and catch basin inlet improvements along Meander Crescent and Clapp Street; and
- Catch basin inlet improvements along Oakpark Drive.

The improvements identified above would provide increased storm conveyance capacity and level of service within the Lesperance storm sewer system during frequent storm events. Local sewers discharging into the Lesperance storm trunk sewer would also be provided increased conveyance capacity due to the reduction in tailwater conditions from the Lesperance storm trunk sewer. During larger, more infrequent storm events, the alternative solution would provide resiliency in the system to reduce surface ponding depths to within the acceptable levels of less than 0.30 m during the 1:100 year event for the majority of the Lesperance storm pump station service area.

10.4.1.3 Alternative 2: New St. Pierre Trunk Sewer and Pump Station Improvements (RECOMMENDED)

This alternative scenario introduces a new storm trunk sewer along St. Pierre Street from Clapp Street to Riverside Drive and along Riverside drive west of St. Pierre to the proposed improved Lesperance storm pump station. Under existing conditions, the St. Pierre storm system comprises smaller local sewers which convey flows to the Lesperance storm trunk sewer through three separate outlets along Clapp Street, Wood Street and Riverside Drive. The three storm connections to the existing Lesperance storm trunk sewer are proposed to be abandoned with the incorporation of the new storm trunk sewer. For this alternative, the St. Pierre storm trunk sewer is proposed to be constructed as a 1200 mm – 1500 mm diameter circular storm sewer.

As part of this regional solution, additional localized solutions are proposed within the regional surface flooding area to provide the added resiliency to improve surface flooding in the service area. This includes:

- Storm sewer conveyance, roadway grading and catch basin inlet improvements along Meander Crescent and Clapp Street;
- Catch basin inlet improvements along Oakpark Drive;
- Underground/aboveground stormwater storage behind the existing Tecumseh Townhall; and
- Underground storage along Evergreen and Gauthier Drive.

The Lesperance storm trunk sewer would provide increased conveyance capacity due to the disconnection of runoff entering the system along the eastern service area boundary. During larger, more infrequent storm events, the alternative solution would provide resiliency in the system to reduce surface ponding depths to within the acceptable levels of less than 0.30 m during the 1:100 year event for the majority of the Lesperance storm pump station service area.

10.4.1.4 Alternative 3: New St. Pierre Trunk Sewer, Lesperance Trunk Sewer and Pump Station Improvements

This alternative scenario includes both the introduction of a new storm trunk sewer along St. Pierre Street and improvements to the existing Lesperance storm trunk sewer. For this alternative, the St. Pierre storm trunk sewer is proposed to be constructed as a 1200 mm – 1500 mm diameter circular storm sewer. The existing Lesperance storm trunk sewer is proposed to be upgraded to a 2400 mm – 2700 mm diameter circular storm sewer.

As part of this regional solution, localized solutions are reduced from previous alternatives. This includes:

- Storm sewer conveyance, roadway grading and catch basin inlet improvements along Meander Crescent and Clapp Street; and
- Catch basin inlet improvements along Oakpark Drive.

During larger, more infrequent storm events, the alternative solution would provide resiliency in the system to reduce surface ponding depths to within the acceptable levels of less than 0.30 m during the 1:100 year event for the majority of the Lesperance storm pump station service area.

10.4.2 Problem Area W-1 Solutions– West St. Louis Pump Station Service Area

For each solution within the West St. Louis pump station service area, the future Coronado Dish Area roadway and storm sewer improvements were incorporated in each of the solutions. The future storm sewer improvements for the Coronado Dish Area were initially designed with a 1:5 year level of service. Through the process of assessing the storm sewer design within the surface flooding decision framework, it was identified that the storm sewer systems for each roadway required larger diameter sewers for an enhanced level of service due to the following:

- Surface flooding along adjacent lower lying residential lands along each roadway was identified to affect private property building entrances during the 1:100 year event; and
- Based on proximity to Lake St. Clair, there was limited opportunity to reduce road grades and utilize surface ponding along road sags.

Based on the increase in storm runoff from the proposed road reconstruction area, the West St. Louis Pump Station is proposed to be improved.

Details of the storm sewer improvements for the Coronado Dish area are further outlined in Section 11.1.2 of this report.

Based on the future road and storm sewer improvements proposed within the Coronado Dish area, two regional alternatives were therefore considered for the West St. Louis Pump Station area within Problem Area W-1:

1) Do nothing
2) Alternative 1: West St. Louis Pump Station Improvements (RECOMMENDED)

The alternatives for this problem area are further illustrated in Figure 10.1.

10.4.2.1 Do Nothing

Under this alternative, no regional or localized solutions are implemented within the problem area where widespread surface flooding is identified. The evaluation of this alternative is required by the EA process; however as part of this study, the "Do Nothing" approach would not reduce surface flooding in the identified problem areas. With the risk of more intense storm events in the future and the known aging infrastructure in the problem area, this alternative is not recommended.

10.4.2.2 Alternative 1: West St. Louis Pump Station Improvements (RECOMMENDED)

The existing West St. Louis storm pump station was identified to be a required improvement within the regional problem area. The pump station design was assessed to provide either a traditional level of service or a more adaptive analysis for an enhanced level of service. The existing pump capacity of 3.38 m^3/s was determined to be improved based on the following options:

- Traditional Level of Service firm capacity upgrade to 5 m^3/s; and
- Enhanced Level of Service firm capacity upgrade to 7 m^3/s.

Based on the analysis of the service area and the decision making process illustrated previously, the West St. Louis pump station was determined to warrant an enhanced level of service with a firm pump capacity upgrade to 7 m^3/s.

For the West St. Louis pump station service area, no significant regional surface flooding solutions (other than the pump station improvements detailed above) were required to reduce surface flooding within the problem area. The proposed storm sewer and roadway improvements within the "Coronado Dish" area and along Barry Avenue improve the level of service to convey all future condition runoff into the Riverside Drive storm trunk sewer to the improved West St. Louis Pump Station.

As part of this regional solution, additional localized solutions are proposed to provide the added resiliency to improve surface flooding in the service area. This includes:

- Storm sewer conveyance, roadway and catch basin inlet improvements within the Coronado Dish Area, Lacasse Boulevard, Dillon Drive, Kimberly Drive and Jelso Place;
- Underground storage along Little River Boulevard from Lacasse Boulevard to Barry Avenue;

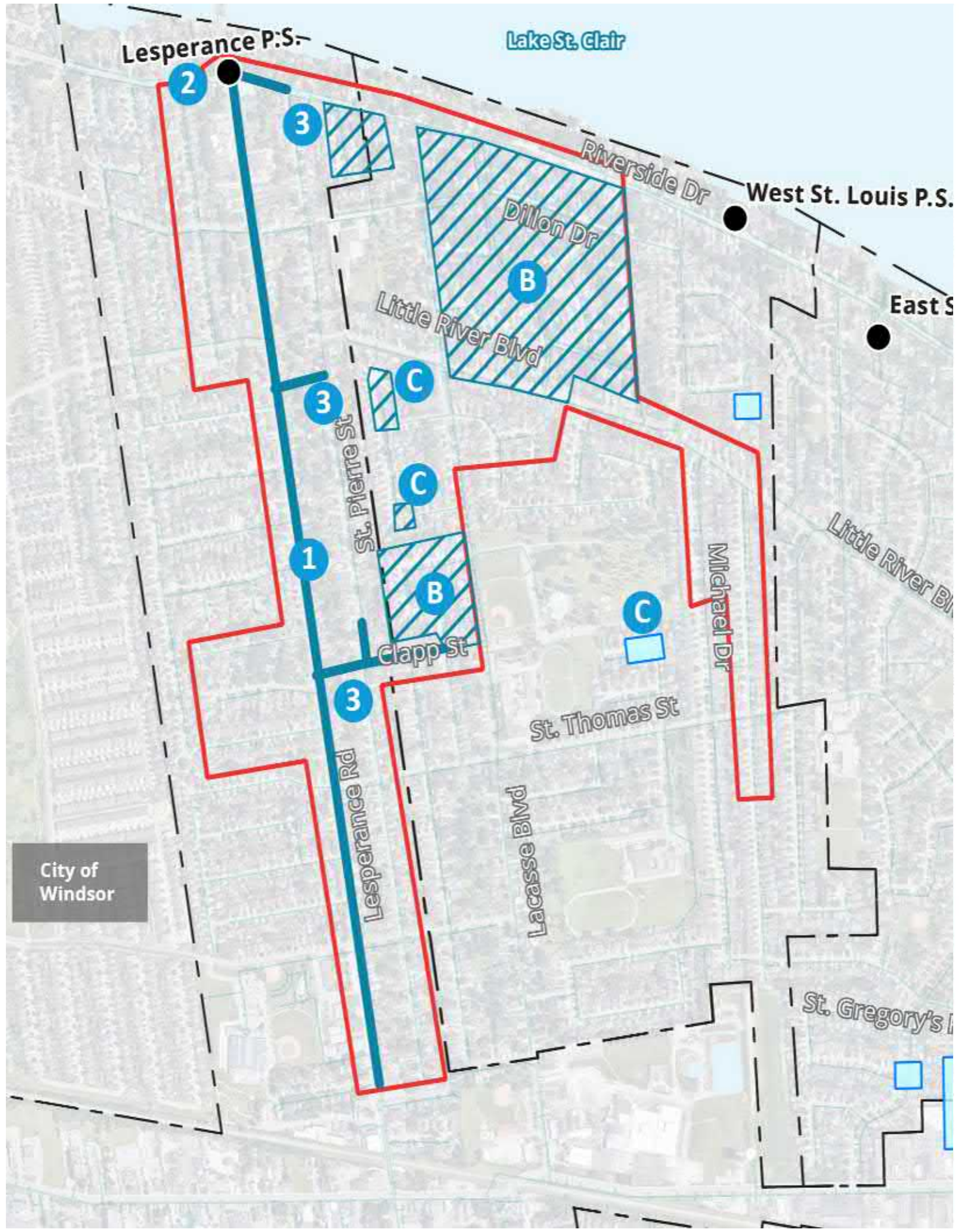
- Installation of a backflow prevention device at the storm sewer interconnection along Riverside Drive of the Lesperance and West St Louis storm pump station service areas between St. Pierre Street and Lacasse Boulevard; and
- Catch basin inlet improvements along Michael, Revland and Woodridge Drive south of Little River.

The improvements identified above would increase storm conveyance capacity and level of service within the localized areas identified above during frequent storm events. During larger, more infrequent storm events, this alternative solution would provide resiliency in the system to reduce surface ponding depths to within the acceptable levels of less than 0.30 m during the 1:100 year event for the majority of the West St. Louis storm pump station service area.

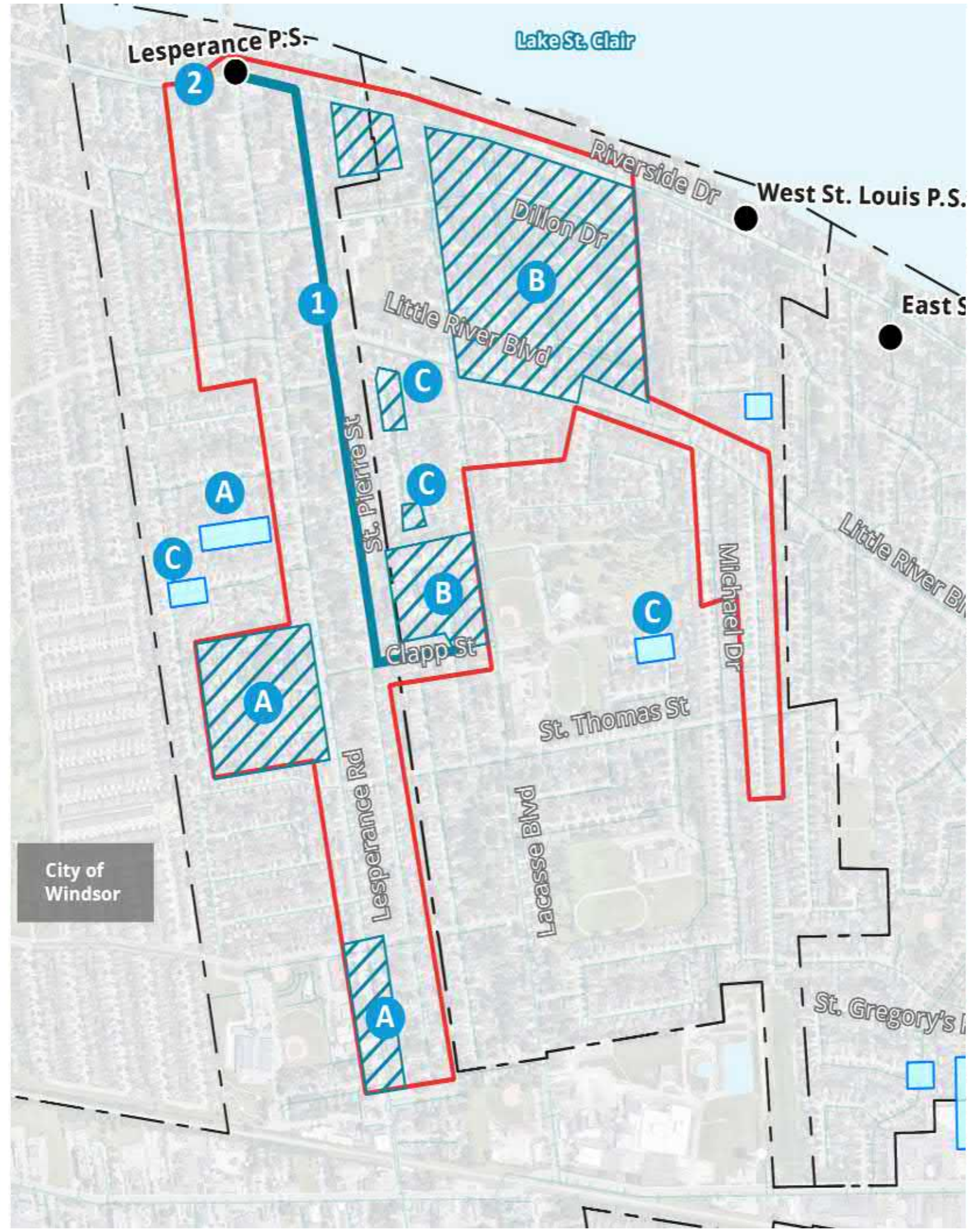
10.4.3 Evaluation of Alternative Solutions – Problem Area W-1

Beyond the evaluation of alternatives identified within the Storm Drainage Master Plan Class Environmental Assessment Study Report, a visual representation of each regional solution alternative within Regional Problem Area W-1 with the respective significant localized solutions is provided in Figure 10.1. Further evaluations with expanded alternatives for each Schedule B project within the problem area is provided in Appendix D.

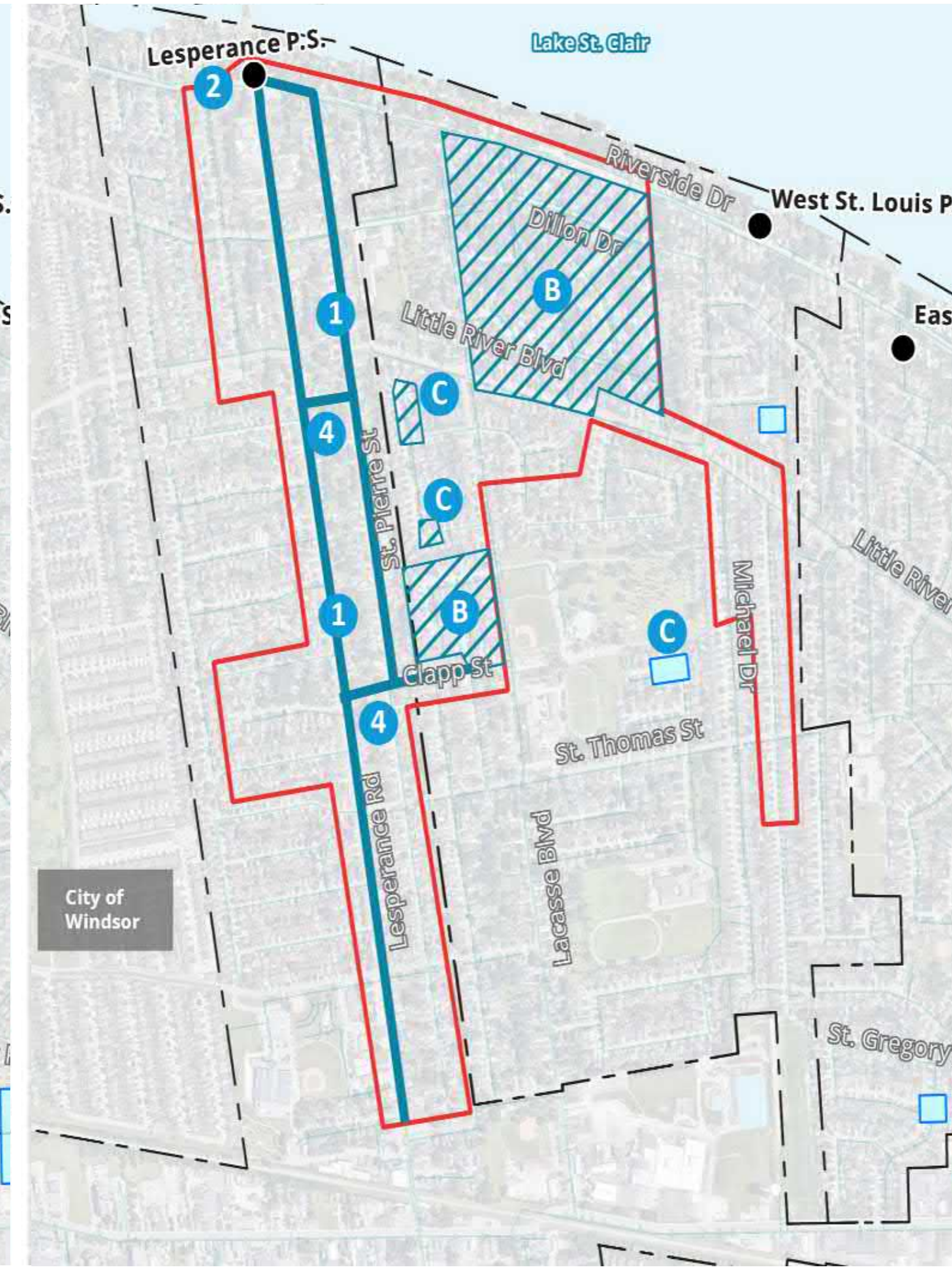
ALTERNATIVE 1



ALTERNATIVE 2



ALTERNATIVE 3



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ALTERNATIVE SURFACE FLOODING SOLUTIONS PROBLEM AREA - W-1
FIGURE 10.1

PUMP STATION (P.S.)
PUMP STATION SERVICE AREA
EXISTING STORM SEWER ALIGNMENT
REGIONAL SURFACE FLOODING PROBLEM AREA
REGIONAL SURFACE FLOODING SOLUTIONS
LOCALIZED SURFACE FLOODING SOLUTIONS
ISOLATED LOCAL SURFACE FLOODING SOLUTIONS

REGIONAL ALTERNATIVE SOLUTIONS
1 STORM TRUNK SEWER IMPROVEMENTS
2 PUMP STATION IMPROVEMENTS
3 STORM SEWER OUTLET IMPROVEMENTS TO PUMP STATION
4 STORM OVERFLOW SEWER TO LESPERANCE TRUNK STORM SEWER

LOCALIZED SOLUTIONS
A UNDERGROUND STORAGE/ STORM OUTLET IMPROVEMENTS
B STORM SEWER CONVEYANCE/ ROAD GRADING IMPROVEMENTS
C CATCH BASIN INLET IMPROVEMENTS

SCALE 1:NTS

2

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PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

10.4.4 Problem Area W-2 Solutions – East St. Louis Pump Station Service Area

Four regional alternatives were considered for Problem Area W-2:

1) Do nothing
2) Alternative 1: St. Thomas Storm Sewer Overflow to Lakewood Park Drainage Channel (RECOMMENDED)
3) Alternative 2: St. Thomas Storm Sewer Overflow to Proposed Manning Road Local Sewer
4) Alternative 3: East St. Louis Trunk Sewer and Pump Station Improvements

The alternatives for this problem area are further illustrated in Figure 10.2.

10.4.4.1 Do Nothing

Under this alternative, no regional or localized solutions would be implemented within the problem area where widespread surface flooding is identified. The evaluation of this alternative is required by the EA process; however as part of this study, the "Do Nothing" approach would not reduce surface flooding in the identified problem areas. With the risk of more intense storm events in the future and the known aging infrastructure in the problem area, this alternative is not recommended.

10.4.4.2 Alternative 1: St. Thomas Storm Sewer Overflow to Lakewood Park Drainage Channel (RECOMMENDED)

This alternative included the incorporation of a 1050 mm diameter storm sewer overflow within the existing 1350 mm diameter storm trunk sewer along St. Thomas Street at the intersection with the Green Valley Drive storm sewer. The overflow sewer is proposed to convey flows easterly down St. Thomas Street and through the proposed 1.8 m x 3.0 m box culvert crossing Manning Road to the Lakewood Park Drainage Channel, eventually discharging into the existing East Townline Drain storm pump station. The box culvert is proposed to be constructed as part of the Manning Road Phase 2 and 3 project, which also includes the enclosure of the East Townline Drain south of St. Thomas Street and the redirection of flows to the Lakewood Park Drainage Channel through a new box culvert. The overflow sewer is not expected to convey flows unless the St. Thomas storm trunk sewer is surcharged.

As part of this regional solution, additional localized solutions are proposed to provide the added resiliency to improve surface flooding in the service area. This includes:

- Catch basin inlet improvements upstream of the St. Thomas overflow sewer along Green Valley Drive, Brunelle Crescent, St. Gregory's Drive, Primrose Place and Harvest Lane; and
- Installation of a backflow prevention device along St. Thomas Street within the 1050 mm diameter overflow sewer directly west of the connection with the proposed 1.8 m x 3.0 m box culvert crossing Manning Road.

During larger, more infrequent storm events, the alternative solution would provide resiliency in the system to reduce surface ponding depths to within the acceptable levels of less than 0.30 m during the 1:100 year event for the East St. Louis storm pump station service area.

10.4.4.3 Alternative 2: St. Thomas Storm Sewer Overflow to Proposed Manning Road Local Sewer

This alternative includes the same 1050mm diameter overflow sewer proposed in Alternative 1. Alternative 2, however, proposes to convey flows easterly down St. Thomas and through the proposed localized Manning Road storm sewer to be constructed along the west side of Manning Road, eventually discharging into the existing East Townline Drain storm pump station. The local Manning Road storm sewer is to be constructed as part of the Manning Road Phase 2 and 3 project to provide conveyance of runoff for the residential properties along the west side of Manning Road to the pump station. This solution includes increased storm sewer designs for the local Manning Road storm sewer from St. Thomas Street to the East Townline Drain pump station to accommodate the runoff from the St. Thomas overflow sewer during the 1:100 year storm event. The proposed local Manning Road storm sewer is proposed to be upgraded from an original design ranging from 825mm – 975 diameter circular storm sewer to a 1050mm – 1350mm diameter circular storm sewer.

As part of this regional solution, additional localized solutions are proposed to provide the added resiliency to improve surface flooding in the service area. This includes:

- Catch basin inlet improvements upstream of the St. Thomas overflow sewer along Green Valley Drive, Brunelle Crescent, St. Gregory's Drive, Primrose Place and Harvest Lane.

During larger, more infrequent storm events, the alternative solution would provide resiliency in the system to reduce surface ponding depths to within the acceptable levels of less than 0.30 m during the 1:100 year event for the East St. Louis storm pump station service area.

10.4.4.4 Alternative 3: East St. Louis Trunk Sewer and Pump Station Improvements

This alternative includes improvements to the existing East St. Louis storm pump station and the existing storm trunk sewer for the service area along Green Valley Drive. The storm trunk sewer improvements are proposed beginning directly north of the intersection with Green Valley Drive and Brunelle Crescent and continuing along Green Valley Drive northerly down St. Thomas Drive, Dillon Drive and Shannon Place to the East St. Louis storm pump station located along Riverside Drive.

For this alternative, the East St. Louis storm trunk sewer is proposed to be upgraded from a 1200mm – 1650mm diameter circular storm sewer to a 1500mm – 2100mm diameter circular storm sewer.

Under this alternative, the proposed storm trunk sewer upgrades for the service area increased the amount of runoff entering the East St. Louis storm pump station. Based on this solution, the existing East St. Louis storm pump station was identified to be a required improvement. The pump station design was assessed to provide either a traditional level of service or a more adaptive analysis for an enhanced level of service. The existing pump capacity of 5.07 m^3/s was identified to be upgraded based on the following options:

- Traditional Level of Service firm capacity upgrade to 6 m^3/s; and
- Enhanced Level of Service firm capacity upgrade to 8 m^3/s.

As part of this regional solution, additional localized solutions are proposed to provide the added resiliency to improve surface flooding in the service area. This includes:

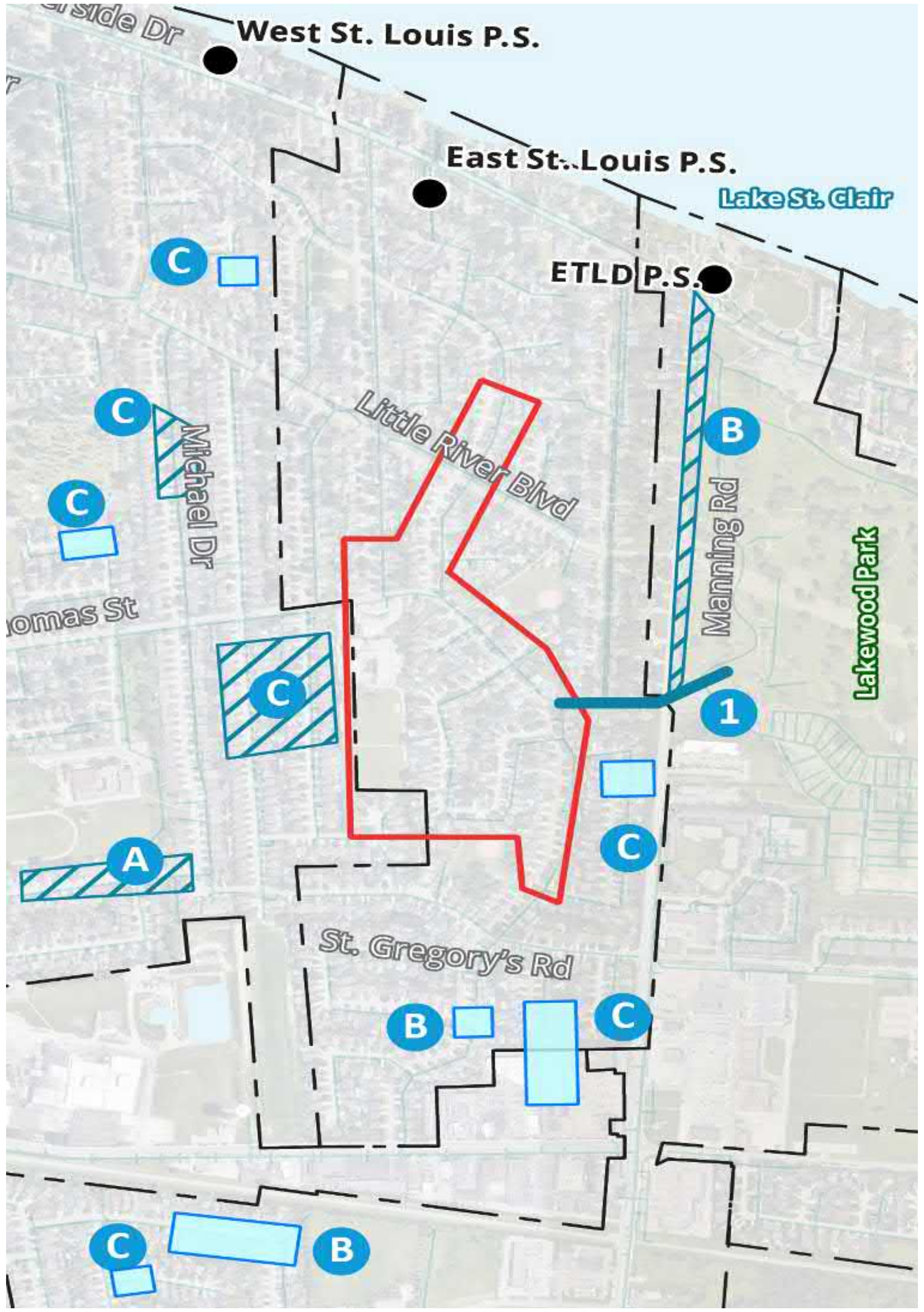
- Catch basin inlet improvements upstream of the St. Thomas overflow sewer along Green Valley Drive, Brunelle Crescent, St. Gregory's Drive, Primrose Place and Harvest Lane.

The improvements identified above would provide an increased storm conveyance capacity and level of service within the East St. Louis service area storm sewer system during frequent storm events. Local sewers discharging into the storm trunk sewer would also be provided increased conveyance capacity due to the reduction in tailwater conditions. During larger, more infrequent storm events, the alternative solution would provide resiliency in the system to reduce surface ponding depths to within the acceptable levels of less than 0.30m during the 1:100 year event for the majority of the East St. Louis storm pump station service area.

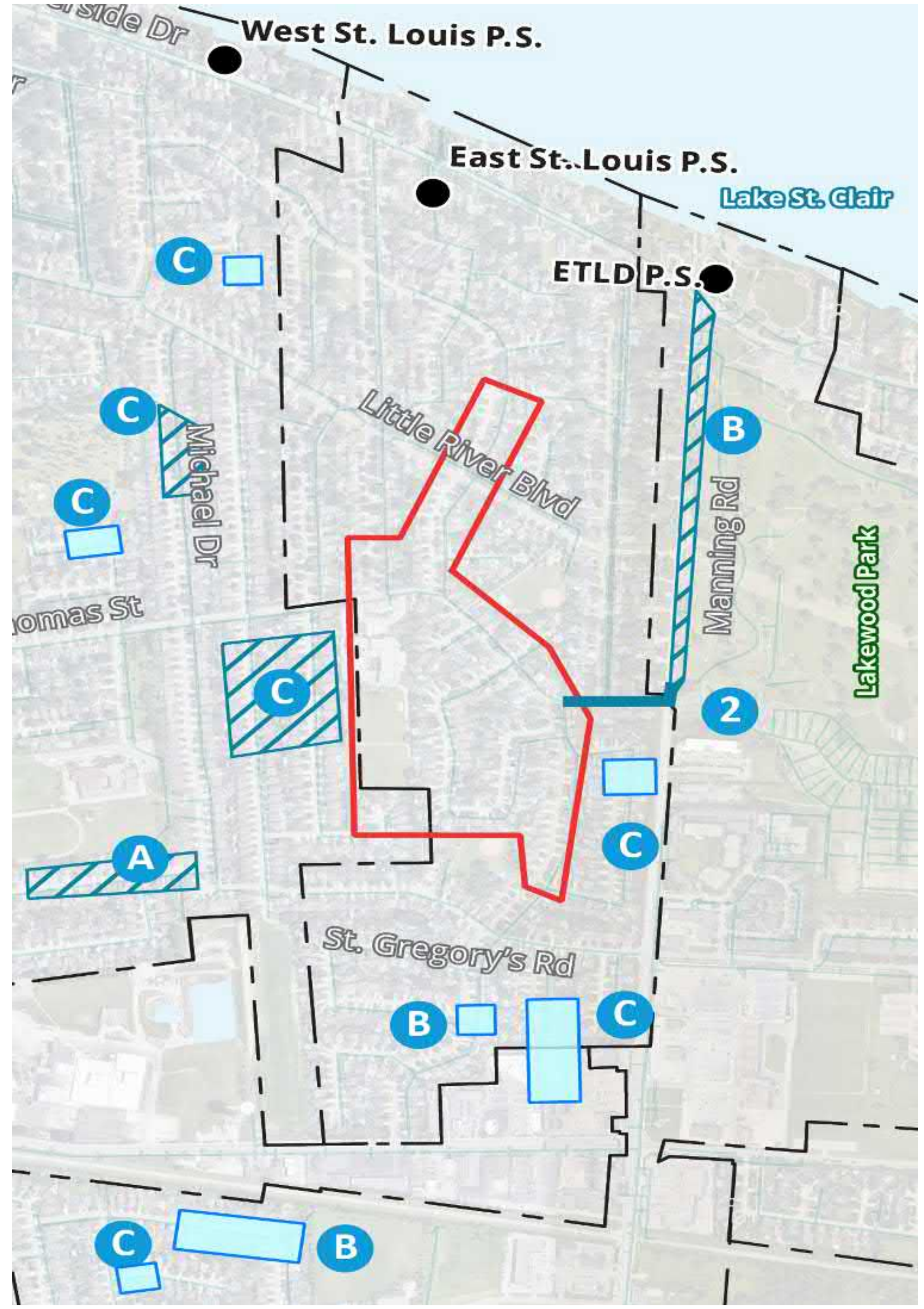
10.4.5 Evaluation of Alternative Solutions – Problem Area W-2

Beyond the evaluation of alternatives identified within the Storm Drainage Master Plan Class Environmental Assessment Study Report, a visual representation of each regional solution alternative within Regional Problem Area W-2 with the respective localized solutions is provided in Figure 10.2. Further evaluations with expanded alternatives for each Schedule B project within the problem area is provided in Appendix D.

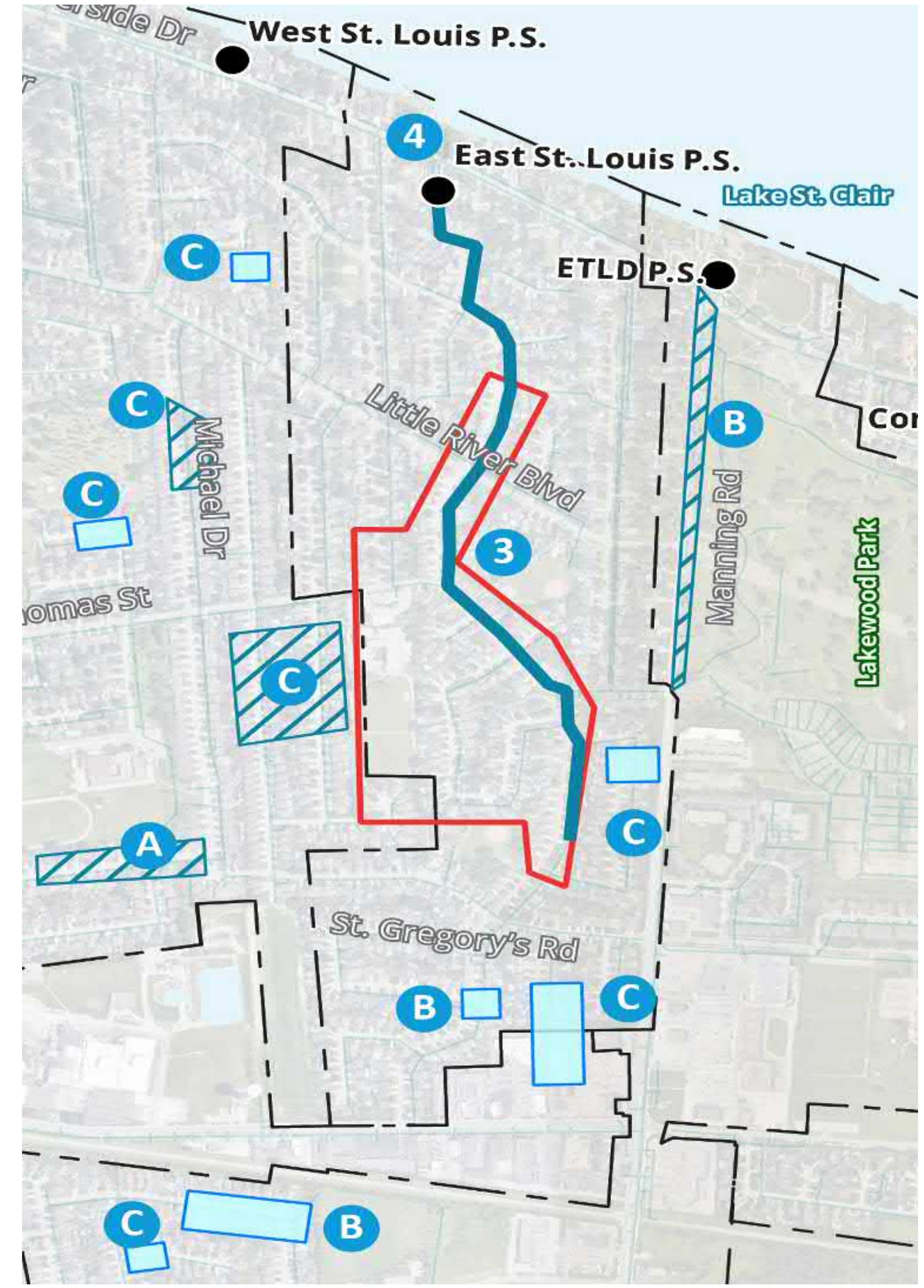
ALTERNATIVE 1



ALTERNATIVE 2



ALTERNATIVE 3



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

ALTERNATIVE SURFACE FLOODING SOLUTIONS PROBLEM AREA - W-2
FIGURE 10.2

Town of Tecumseh Ontario - Canada

PUMP STATION (P.S.)
PUMP STATION SERVICE AREA
EXISTING STORM SEWER ALIGNMENT
REGIONAL SURFACE FLOODING PROBLEM AREA
REGIONAL SURFACE FLOODING SOLUTIONS

LOCALIZED SURFACE FLOODING SOLUTIONS
ISOLATED LOCAL SURFACE FLOODING SOLUTIONS

REGIONAL ALTERNATIVE SOLUTIONS
1 STORM OVERFLOW SEWER TO LAKEWOOD PARK CHANNEL
2 STORM OVERFLOW SEWER TO LOCAL FUTURE MANNING ROAD SEWER
3 STORM TRUNK SEWER IMPROVEMENTS
4 PUMP STATION IMPROVEMENTS

LOCALIZED SOLUTIONS
A UNDERGROUND STORAGE
B STORM SEWER CONVEYANCE/ROAD GRADING IMPROVEMENTS
C CATCH BASIN INLET IMPROVEMENTS

DILLON CONSULTING

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

10.4.6 Problem Area W-3 Solutions – Baillargeon Drain Service Area

For each solution within the Baillargeon Drain service area, the future St. Anne Area roadway and storm sewer improvements were incorporated in each of the solutions. Details of the storm sewer improvements for this area are further detailed in Section 11.1.6.

Three regional alternatives were considered for Problem Area W-3 within this service area:

1) Do nothing
2) Alternative 1: Charlene Storm Relief Sewer to MRSPA Development (RECOMMENDED OPTION 1)
3) Alternative 2: Localized Underground Storage(RECOMMENDED OPTION 2)

The alternatives for this problem area are further illustrated and summarized in Figure 10.3.

For both Alternatives 1 & 2, the existing 1350 mm diameter Baillargeon Drain outlet at Candlewood Drive is assumed to discharge into the proposed future development storm trunk sewer (as identified within the 2015 MRSPA Functional Servicing Report and EA Report Addendum). The existing open drainage channel currently conveys flows from the upstream developed lands through the area of future development and discharges flows into the East Townline Drain.

10.4.6.1 Do Nothing

Under this alternative, no regional or localized solutions would be implemented within the problem area where widespread surface flooding is identified. The evaluation of this alternative is required by the EA process; however as part of this study, the “Do Nothing” approach would not reduce surface flooding in the identified problem areas. With the risk of more intense storm events in the future and the known aging infrastructure in the problem area, this alternative is not recommended.

10.4.6.2 Alternative 1: Charlene Storm Relief Sewer to MRSPA Development (RECOMMENDED)

As part of this alternative, a storm relief sewer is proposed along Charlene Lane to intercept all storm sewer runoff East of Lesperance Road and south of Charlene Lane from the existing residential lands to reduce surface flooding within the regional problem area. The Charlene relief sewer design ranges from 675mm – 900mm in diameter and is proposed to convey flows east through the existing 4.0 m storm easement between two residential properties and into the future development storm trunk sewer. A 600mm diameter overflow sewer is also proposed to connect to the Charlene relief sewer from the Lesperance storm sewer system at the intersection of Charlene Lane and Lesperance Road. Under existing conditions, the 4.0m storm easement is used as an overland flow route for the residential development along Charlene Lane to the currently undeveloped future development lands to the east.

As part of the 2015 MRSPA Functional Servicing Report and MRSPA SWM Class EA Addendum Report, the Baillargeon Drain service area is to incorporate a new storm outlet from Gouin Street into the future MRSPA storm trunk sewer. This outlet consists of a 900 mm diameter sewer along Gouin Street at Deslippe Drive. As part of this study, the Gouin Street storm outlet is not required to be directed into the

MRSPA storm trunk sewer to reduce the surface flooding in the area, but recommended as part of this alternative.

The improvements identified above, along with the localized storm sewer conveyance, road grading and catch basin improvements within the St. Anne Area, would provide an increased storm conveyance capacity and level of service within the storm sewer system during frequent storm events. During larger, more infrequent storm events, the alternative solution would provide resiliency in the system to reduce surface ponding depths to within the acceptable levels of less than 0.30m during the 1:100 year event for the majority of the service area. Some localized surface flooding beyond 0.30m would still occur within isolated road sags along Lesperance Road directly north of Meconi Drive, but flooding depths would be significantly reduced based on these improvements

10.4.6.3 Alternative 2: Localized Underground Storage (RECOMMENDED)

As part of this alternative, underground storage is proposed along Charlene Lane, St. Martin Crescent and St. Agnes Crescent to reduce surface flooding within the regional problem area. The underground storage proposed includes:

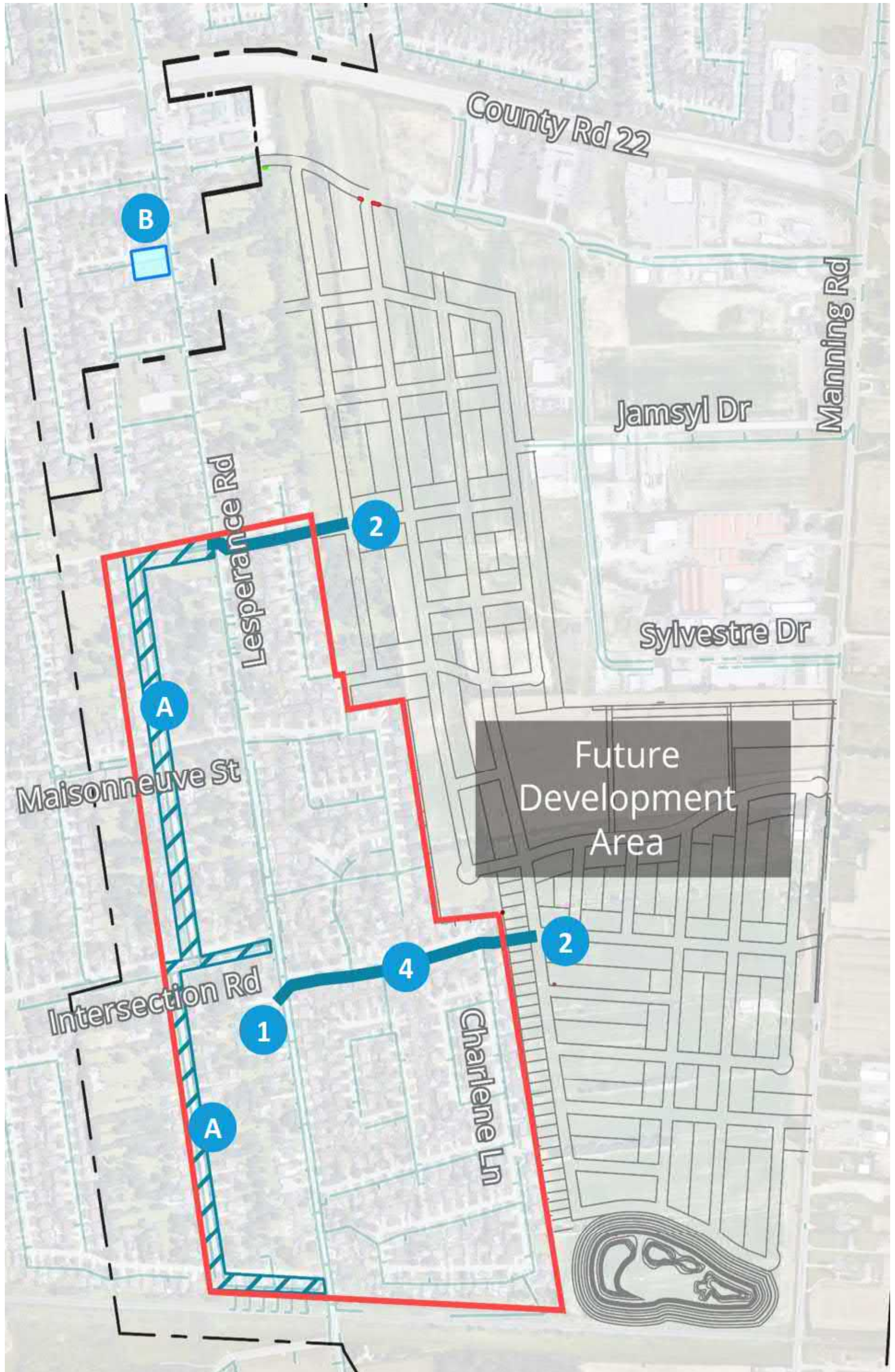
- 825 mm diameter underground storage sewer along St. Martin directly upstream of the existing 675 mm diameter storm sewer outlet from St. Martin Crescent to Charlene Lane;
- 900 mm diameter underground storage sewer Charlene Lane directly north of Eugeni Street; and
- 900 mm x 1800 mm underground storage rectangular box chambers along Charlene Lane and St. Agnes upstream of the existing 1200 mm diameter enclosed Baillargeon Drain.

The improvements identified above, along with the localized storm sewer conveyance, road grading and catch basin improvements within the “St. Anne Area”, would provide an increased storm conveyance capacity and level of service within the storm sewer system during frequent storm events. During larger, more infrequent storm events, the alternative solution would provide resiliency in the system to reduce surface ponding depths to within the acceptable levels of less than 0.30m during the 1:100 year event for the majority of the service area. Some localized surface flooding beyond 0.30m would still occur within isolated road sags along Lesperance Road directly north of Meconi Drive, but flooding depths significantly reduced based on the these improvements

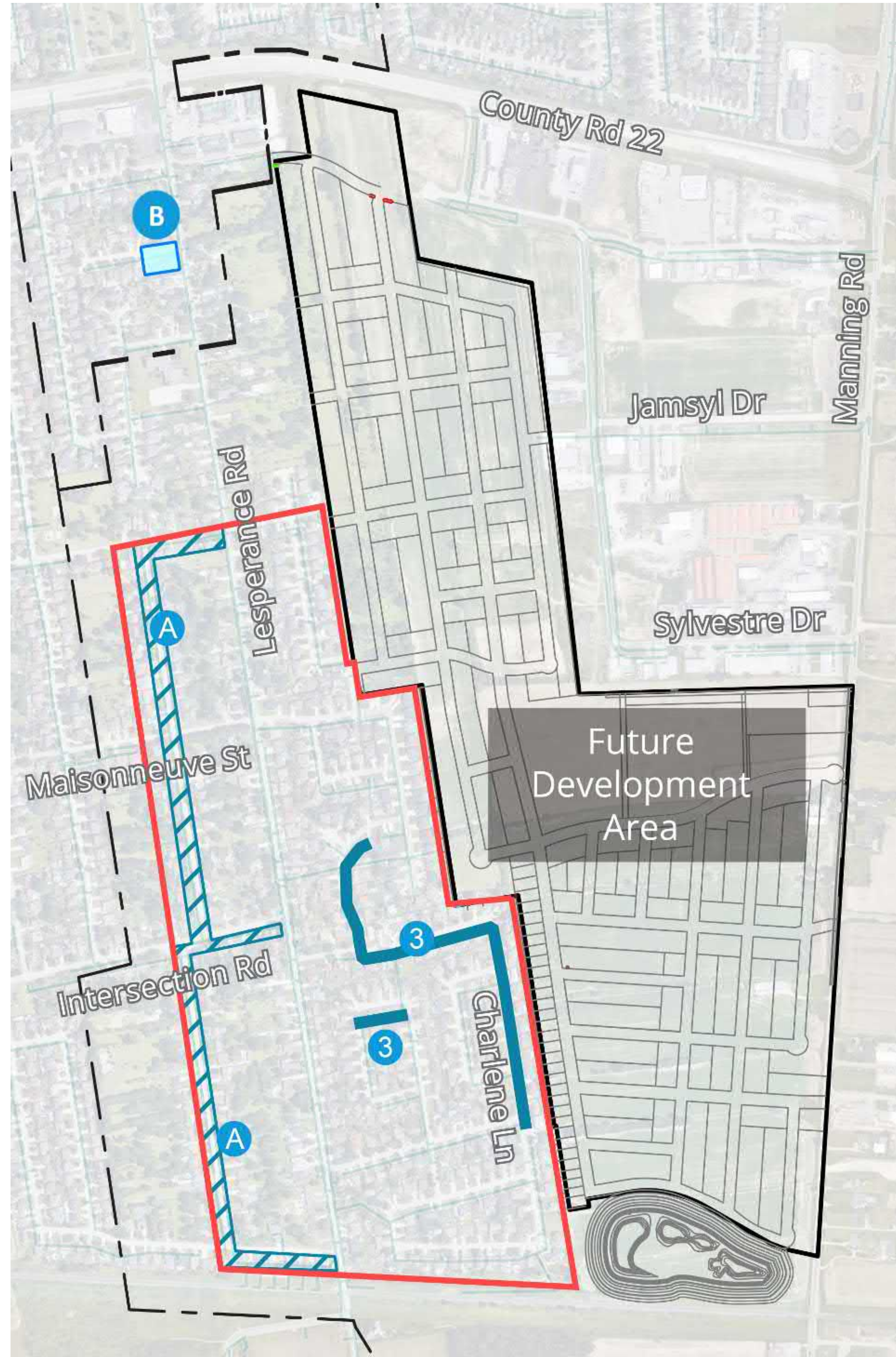
10.4.7 Evaluation of Alternative Solutions – Problem Area W-3

Beyond the evaluation of alternatives identified within the Storm Drainage Master Plan Class Environmental Assessment Study Report, a visual representation of each regional solution alternative within Regional Problem Area W-3 with the respective localized solutions is provided in Figure 10.3.

ALTERNATIVE 1



ALTERNATIVE 2



TOWN OF Tecumseh ONTARIO · CANADA

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

ALTERNATIVE SURFACE FLOODING SOLUTIONS PROBLEM AREA - W-3
FIGURE 10.3

- PUMP STATION SERVICE AREA
- EXISTING STORMSEWER ALIGNMENT
- REGIONAL SURFACE FLOODING PROBLEM AREA
- REGIONAL SURFACE FLOODING SOLUTIONS
- LOCALIZED SURFACE FLOODING SOLUTIONS
- ISOLATED LOCAL SURFACE FLOODING SOLUTIONS

REGIONAL ALTERNATIVE SOLUTIONS

1. LESP. STM OVERFLOW SEWER TO CHARLENE RELIEF SEWER
2. STM SEWER OUTLET TO FUTURE DEVELP. AREA TRUNK SEWER
3. UNDERGROUND STORAGE
4. STORM SEWER CONVEYANCE IMPROVEMENTS

LOCALIZED SOLUTIONS

A. UNDERGROUND STORAGE/ STORM SEWER CONVEYANCE/ ROAD GRADING IMPROVEMENTS
B. CATCH BASIN INLET IMPROVEMENTS

SCALE 1:NTS 2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

DILLON CONSULTING

PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019

10.4.8 Expanded Localized Surface Flooding Solutions

Based on the extent of regional surface flooding solutions identified through this study, more enhanced localized solutions were incorporated in areas where the surface flooding is isolated where a regional surface flooding solution would not be warranted or within a regional problem area to reduce the design for the regional solution.

The localized solutions identified within this section followed the same decision making framework outlined within Section 9.0 to identify whether the solution warranted a traditional or more enhanced level of service.

10.4.8.1 Lemire Street and Lanoue Street

As part of the study, it was identified based on the decision framework that the climate change analysis would be assessed within the model to further determine problem areas and potentially enhance the level of service of initial surface flooding solutions identified. Based on this analysis, a more extensive localized surface flooding solution was required within the Lemire and Lanoue Street area to reduce surface flooding along both the localized roadways and within existing residential private properties. Under existing conditions, the Lemire and Lanoue Street storm sewer drainage is directed into the existing CN Railway ditch through two outlets; a 900 mm diameter outlet at the intersection of Lemire Street and Northfield Way and a second 900 mm outlet within the existing pathway to Buster Reaume Park. To reduce surface flooding within the area, two alternatives were reviewed, as follows:

Alternative 1 (RECOMMENDED):

- Redirection of storm drainage and upgrade of local storm sewers (675 mm – 825 mm diameter) along Lanoue and Lemire street through to Buster Reaume Park and discharge flows into the existing 900 mm diameter CN Railway Ditch outlet;
- Construction of a 0.80 m depression along the southwestern portion of Buster Reaume Park to provide approximately 4,100 m^3 of aboveground surface storage with a connection to the upgraded municipal storm sewers;
- Installation of a catch basin and a 750 mm diameter storm sewer outlet from the Buster Reaume Park aboveground storage to the existing 900 mm diameter storm sewer to the CN Railway ditch; and
- Installation of a backflow prevention device at the outlet of the 900 mm diameter sewer to the CN Railway ditch within the Buster Reaume pathway from Lemire Street.

Alternative 2

- 551 meters of 3048 mm x 2438 mm (a total of 4,100 m^3 of storage) underground rectangular box chambers along Lemire and Lanoue Street within the municipal right-of-way upstream of the existing 900 mm diameter outlet through the pathway into the CN Railway ditch; and
- Installation of a backflow prevention device at the outlet of the 900 mm diameter sewer to the CN Railway ditch within the Buster Reaume pathway from Lemire Street.

Due to the complexity and extensive improvements proposed for the localized surface flooding area, a high level evaluation was completed for the two alternatives detailed above. Advantages and Disadvantages of each were identified and illustrated for Pubic Information Centre #2. This high level evaluation is included in Appendix D.

10.5 East of Manning Road/CR19 Alternative Surface Flooding Solutions

The following service areas were examined:

- Scully Pump Station Service Area;
- St. Mark's Pump Station Service Area;
- PJ Cecile Pump Station Service Area;
- Brighton Pump Station Service Area; and
- Starwood/Southwind Gravity Outfall Service Area.

This section includes details of the proposed improvements within each problem area for each alternative.

10.5.1 Regional Problem Area E-1 Solutions – Scully, St. Mark's, PJ Cecile Pump Station Service Area

Each alternative for Regional Problem Area E-1 considered storm sewer improvements in the following locations:

- Edgewater Boulevard (Existing Scully Storm Pump Station Service Area);
- St. Marks Road (Existing St. Mark's Storm Pump Station Service Area);
- Arlington Boulevard (Existing St. Mark's Storm Pump Station Service Area); and
- Streets within the Kensington Dish Area (Existing PJ Cecile Storm Pump Station Service Area).

The future storm sewer improvements for the above roadways were initially designed with a 1:5 year level of service. Through the process of assessing the storm sewer design within the surface flooding decision framework, the storm sewer systems for each roadway were deemed to require larger sewers due to the following:

- Surface flooding along adjacent lower lying residential lands along each roadway was identified to affect private properties including potential building entrances during the 1:100 year event; and
- Based on proximity to Lake St. Clair, there was limited opportunity to reduce road grades and utilize surface ponding along road sags.

Details of the storm sewer improvements for the areas are further detailed in Section 11.2.1.

The existing Scully, St. Mark's and PJ Cecile storm pump stations have been identified to require improvements for all alternatives evaluated within the regional problem area. For each alternative solution, several alternative pump station upgrades were assessed. Each pump station improvement alternative was analyzed to identify if either a traditional level of service or a more adaptive analysis for an enhanced level of service was warranted.

Four regional alternatives were considered for Problem Area E-1 within this service area:

1) Do nothing
2) Alternative 1: Scully, St. Mark's and PJ Cecile PS Upgrades
3) Alternative 2: Consolidated Scully/St. Mark's PS and PJ Cecile PS Upgrades (RECOMMENDED)
4) Alternative 3: Consolidated Scully/St. Mark's/PJ Cecile PS Upgrades

The recommended improvements to the pump stations under each alternative is detailed below and further illustrated in Figure 10.4.

10.5.1.1 Do Nothing

Under this alternative, no regional or localized solutions would be implemented within the problem area where widespread surface flooding is identified. The evaluation of this alternative is required by the EA process; however as part of this study, the "Do Nothing" approach would not reduce surface flooding in the identified problem areas. With the risk of more intense storm events in the future and the known aging infrastructure in the problem area, this alternative is not recommended.

10.5.1.2 Alternative 1: Scully, St. Mark's and PJ Cecile PS Upgrades

This alternative consisted of the improvement to each of the three pump stations and respective service area storm sewers. The service areas for each of the three pump stations were analyzed under the following upgrade options:

Traditional Level of Service Upgrades

- Scully pump station firm capacity upgrade to 2.47 m^3/s;
- St. Mark's pump station firm capacity upgrade to 1.89 m^3/s; and
- PJ Cecile pump station firm capacity upgrade to 2.2 m^3/s.

Enhanced Level of Service Upgrades

- Scully pump station firm capacity upgrade to 3.5 m^3/s;
- St. Mark's pump station firm capacity upgrade to 2.7 m^3/s; and
- PJ Cecile pump station firm capacity upgrade to 3.0 m^3/s.

Based on the analysis of the service areas and following the surface flooding decision framework process illustrated previously, the three pump stations were determined to warrant an enhanced level of service based on the firm pump capacity upgrades outlined above.

10.5.1.3 Alternative **2**: Consolidated Scully/St. Mark's PS and PJ Cecile PS Upgrades (RECOMMENDED)

This alternative considered decommissioning St. Mark's storm pump station and consolidating that service area into that of the Scull storm pump station. This alternative included a new consolidated Scully/St. Mark's storm pump station on the Scully site and upgrades to improve the PJ Cecile Storm Pump Station.

The service areas for the proposed two pump stations were analyzed under the following upgrade options:

Traditional Level of Service Upgrades

- Consolidated Scully/St. Mark's pump station firm capacity upgrade to 4.0 m^3/s; and
- PJ Cecile pump station firm capacity upgrade to 2.2 m^3/s.

Enhanced Level of Service Upgrades

- Consolidated Scully/St. Mark's pump station firm capacity upgrade to 6.0 m^3/s; and
- PJ Cecile pump station firm capacity upgrade to 3.0 m^3/s.

Based on the analysis of the service areas and following the surface flooding decision framework process illustrated previously, the two pump stations were determined to warrant an enhanced level of service with the firm pump capacity upgrades noted above.

The respective service area storm sewers noted in Section 10.5.1 were identified to be reconstructed and were further analyzed based on the proposed pump station improvements. Storm trunk sewer improvements and the redirection of storm drainage along Riverside Drive are required between Arlington Boulevard and the proposed consolidated storm pump station on the Scully site to convey flows from the existing St. Mark's storm pump station service area. The design of the storm trunk along Riverside Drive ranges from 1350 mm to 1500 mm in diameter.

With the proposed improvements of the storm trunk sewer along Riverside Drive and redirection of flows from the St. Mark's service area to the location of the consolidated Scully/St. Mark's storm pump station within the existing Scully site, sanitary sewer improvements are required along Riverside Drive due to potential conflicts with private drain connections of properties fronting Riverside Drive. To accommodate the new Riverside Drive storm trunk sewer from Arlington Boulevard to the new storm pump station at the Scully storm pump station site, the following works are required beyond the storm sewer improvements:

- Lowering of the local Riverside Drive sanitary sewer by approximately 0.50 m between Arlington Boulevard and St. Mark's Road which currently outlets into the St. Mark's sanitary sewer. This local Riverside Drive sanitary sewer is proposed to be redirected to the Arlington sanitary sewer; and
- Lowering of the local Riverside Drive sanitary sewer by approximately 0.50 m between St. Mark's Road and Edgewater Drive and maintain the existing outlet to the Edgewater sanitary sewer.

10.5.1.4 Alternative 3: Consolidated Scully/St. Mark's/PJ Cecile PS Upgrades

This alternative considered decommissioning both the Scully and the PJ Cecile storm pump stations and consolidating the service areas with that of St. Mark's storm pump station. The scenario would then include a new consolidated Scully/St. Mark's/PJ Cecile storm pump station on the St. Mark's site.

The pump station improvement for this alternative of one consolidated pump station included the following upgrade options:

Traditional Level of Service Upgrades

- Consolidated Scully/St. Mark's/PJ Cecile pump station firm capacity upgrade to 6.2 m^3/s.

Enhanced Level of Service Upgrades

- Consolidated Scully/St. Mark's/PJ Cecile pump station firm capacity upgrade to 8.7 m^3/s.

Based on the analysis of the service areas and following the surface flooding decision framework process illustrated previously, consolidation of the three pump stations to one large station was determined to warrant an enhanced level of service with a firm pump capacity of 8.7 m^3/s.

The respective service area storm sewers noted in Section 10.5.1 were identified to be reconstructed and were also analyzed based on the pump station improvements. Storm trunk sewer improvements and the redirection of storm drainage along Riverside Drive are required between Kensington Boulevard and the proposed consolidated storm pump station on the St. Mark's site to convey flows from the existing PJ Cecile and St. Mark's storm pump station service area. The design of the storm trunk along Riverside Drive ranges from 1350mm to 2100mm in diameter.

With the improvements of the storm trunk sewer along Riverside Drive and redirection of flows from the PJ Cecile and St. Mark's service area to the location of the consolidated Scully/St. Mark's storm pump station within the existing St. Mark's site, sanitary sewer improvements are required along Riverside Drive due to potential conflicts with private drain connections of properties fronting Riverside Drive. To accommodate the new Riverside Drive storm trunk sewer from Kensington Boulevard to the new storm pump station, the following works are required beyond the storm sewer improvements:

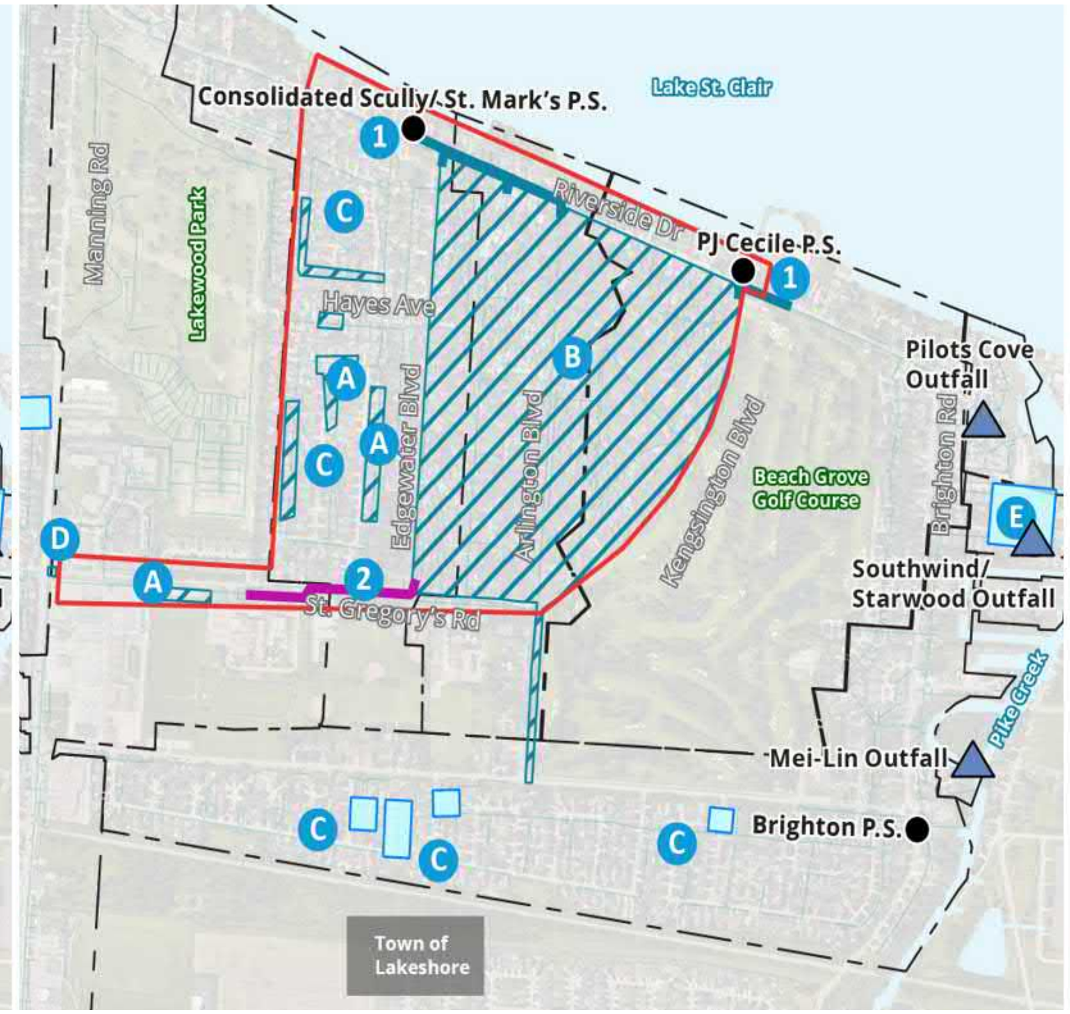
- Lowering of the local Riverside Drive sanitary sewer by approximately 2.0 m between Arlington Boulevard and St. Mark's Road which currently outlets into the St. Mark's sanitary sewer. This local Riverside Drive sanitary sewer is proposed to be redirected to outlet to the Arlington sanitary sewer; and
- Lowering of the local Riverside Drive sanitary sewer by 1.5 m between St. Mark's Road and Edgewater Drive and maintain the existing outlet to the Edgewater sanitary sewer.

10.5.2 Evaluation of Alternative Solutions – Problem Area E-1

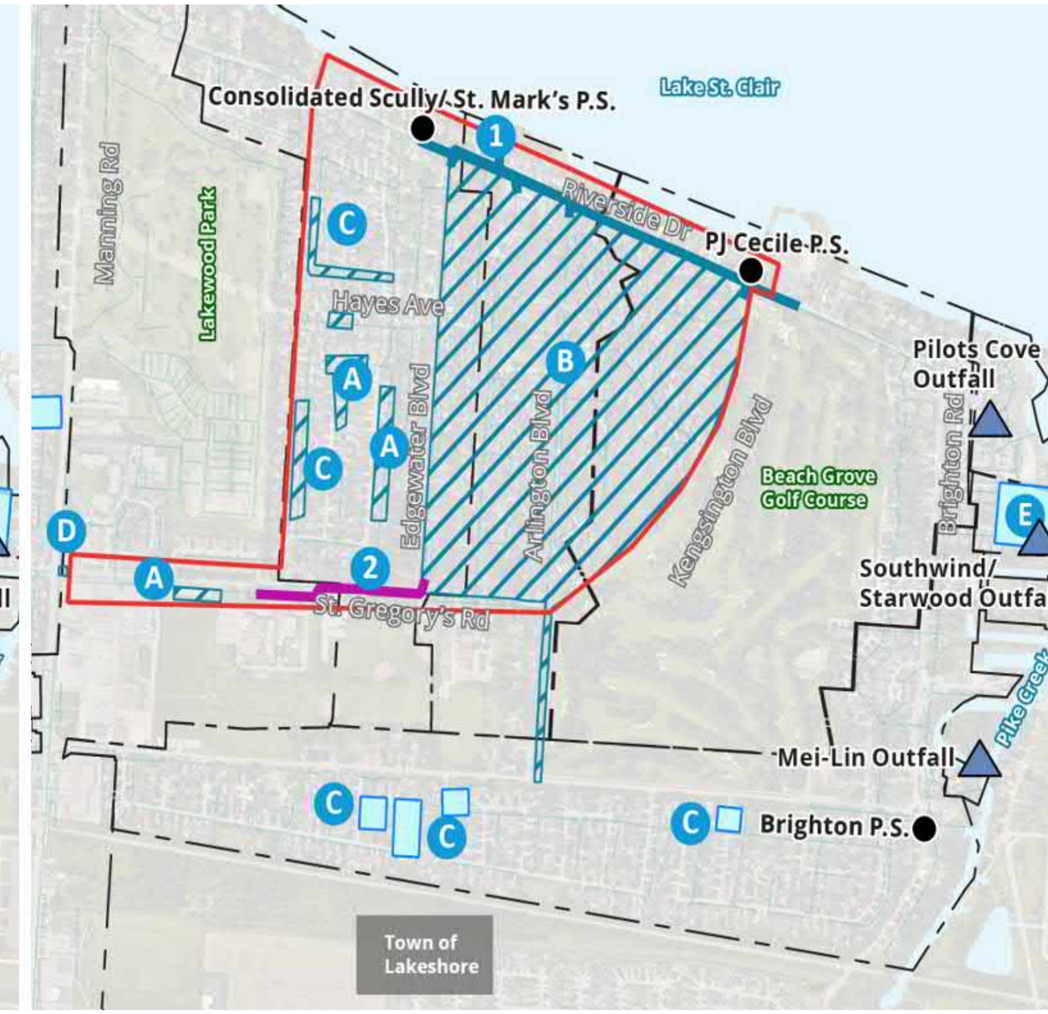
Beyond the evaluation of alternatives identified within the Storm Drainage Master Plan Class Environmental Assessment Study Report, a visual representation of each regional solution alternative within Regional Problem Area E-1 with the respective localized solutions is provided in Figure 10.4. Further evaluations with expanded alternatives for each Schedule B project within the problem area is provided in Appendix D.

ALTERNATIVE 1

ALTERNATIVE 2



ALTERNATIVE 3



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

SEWER GRAVITY OUTFALL
PUMP STATION (P.S.)
PUMP STATION SERVICE AREA
EXISTING STORM SEWER ALIGNMENT
REGIONAL SURFACE FLOODING PROBLEM AREA

REGIONAL SURFACE FLOODING SOLUTIONS
SUPPLEMENTARY SURFACE FLOODING SOLUTIONS
LOCALIZED SURFACE FLOODING SOLUTIONS
ISOLATED LOCAL SURFACE FLOODING SOLUTIONS

REGIONAL ALTERNATIVE SOLUTIONS
1 PUMP STATION AND RIVERSIDE DRIVE STORM SEWER TRUNK IMPROVEMENTS
2 STORM SEWER REDIRECTION TO EDGEWATER STORM SEWER SYSTEM

LOCALIZED SOLUTIONS
A UNDERGROUND STORAGE/ ROAD GRADING IMPROVEMENTS
B STORM SEWER CONVEYANCE/ ROAD GRADING IMPROVEMENTS
C CATCH BASIN INLET IMPROVEMENTS
D BACKFLOW PREVENTION FROM EAST TOWNLINE DRAIN
E PUMP STATION TO PIKE CREEK

ALTERNATIVE SURFACE FLOODING SOLUTIONS PROBLEM AREA - E-1
FIGURE 10.4

Town of Tecumseh
ONTARIO · CANADA

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MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

10.5.3 Localized Surface Flooding Solutions

10.5.3.1 St. Gregory's Road

As part of the study, it was identified during the climate change analysis that a more extensive localized surface flooding solution beyond the traditional level of service was required along St. Gregory's Road between Village Grove Drive and Cada Crescent. A traditional solution is proposed along St. Gregory's which includes the following:

- Incorporation of a backflow prevention device at the St. Gregory's Road storm outlet to the East Townline Drain at the intersection of St. Gregory's and Manning Road;
- Elimination of the storm sewer interconnection along St. Gregory's Road fronting L'Essor School with the East Townline Drain and Scully storm pump station service areas; and
- Redirection of the storm sewer outlet for St. Gregory's Road, East of L'Essor School to extend further east from the Cada Crescent storm outlet to the proposed Edgewater Drive storm sewer.

During extreme rainfall events, the model simulation identified that extensive surface flooding occurs specifically within the ingress and egress routes of the L'Essor Highschool. To reduce surface flooding within the area and provide, two alternatives were reviewed. This included:

Alternative 1 (RECOMMENDED):

- Construction of a 0.70 m depression of the existing northern soccer fields within the Tecumseh Soccer Fields Park currently owned by the L'Essor high school and Conseil Scolaire Catholique (CSC) Providence to provide approximately 3,200 m^3 of aboveground surface storage with a 200mm diameter storm sewer connection to the existing 750 mm diameter storm sewer along St. Gregory's Road.

Alternative 2

- 430 meters of 3048 mm x 2438 mm (providing 4100 m^3 of storage) underground rectangular box chambers along St. Gregory's Road within the municipal right-of-way.

Due to the complexity and extensive improvements proposed for the localized surface flooding area, a high level evaluation was completed for the two alternatives detailed above. Advantages and Disadvantages of each were identified and illustrated for Pubic Information Centre #2. This high level evaluation and illustration for each alternative is provided in Appendix D.

10.5.3.2 Starwood Lane/Southwind Crescent

For each of the localized solution alternatives for the Starwood/Southwind area, a new pump station is proposed to provide a hydraulic disconnect of the storm system from the water levels at the existing outlet along Pike Creek. A traditional level of service was warranted for the new pump station with a firm capacity of 0.2 m^3/s for each alternative. The alternatives reviewed as part of this localized solution included the location of the proposed pump station.

Alternative Location # 1 (RECOMMENDED)

- Construction of a 0.2 m^3/s storm pump station located within existing 4.5 m storm easement with a connection to the existing storm outlet; and
- Incorporation of a backflow prevention device at the existing storm outlet.

Alternative Location # 2

- Construction of a 0.2 m^3/s storm pump station located within the existing Southport Sailing Club lands;
- New pump station storm outlet within the existing Southport Sailing Club lands; and
- Incorporation of a backflow prevention device at the existing storm outlet.

Alternative Location #3

- Construction of a 0.2 m^3/s storm pump station located within existing Southwind Crescent municipal right-of-way with a connection to the existing storm outlet; and
- Incorporation of a backflow prevention device at the existing storm outlet.

Due to the impact on the adjacent residents and directly affected property owners for localized solution, an additional high level evaluation was completed for the two alternatives detailed above. Advantages and Disadvantages of each were identified and illustrated for Pubic Information Centre #2. This high level evaluation and illustration for each alternative is provided in Appendix D.

10.6 Evaluation of Alternative Regional Surface Flooding Solutions

As part of the Environmental Assessment (EA) process, each regional and Schedule B alternative was evaluated based on a number of factors, including technical, social/economic impacts, environmental impacts and cost. The evaluation took into consideration discussions with any applicable stakeholders and comments received from the public during the consultation process and public information centres.

The evaluation of each alternative is further detailed within the main body of the Storm Drainage Master Plan Class Environmental Assessment Study Report.

11.0 Recommended Functional Design Solutions

Through the Environmental Assessment (EA) process, an evaluation of each alternative regional surface flooding solution was completed to identify the preferred solution for each problem area. The preferred regional surface flooding solutions for this study are as follows:

Problem Area W-1 Regional Solution:
- Lesperance Pump Station Service Area: ALTERNATIVE 2; and
- West St. Louis Pump Station Service Area: ALTERNATIVE 1.

Problem Area W-2 Regional Solution:
- East St. Louis Pump Station Service Area: ALTERNATIVE 1

Problem Area W-3 Regional Solution:
- Baillargeon Drain Service Area: ALTERNATIVE 1 or ALTERNATIVE 2

Problem Area E-1 Regional Solution:
- Scully/St. Mark's/PJ Cecile Pump Station Service Area: ALTERNATIVE 2

Where extensive local surface flooding solutions were identified, these solutions were further assessed for the preferred solution to reduce surface flooding. This included localized solutions in the following areas:

Localized Solutions West of Manning Road/County Road 19
- Underground and Surface Storage along Lesperance Road within Tecumseh Centre Park;
- Underground Storage along Evergreen Drive and Gauthier Drive;
- Underground Storage, Storm Sewer and Roadway Grading Improvements along Meander Crescent and Clapp Street;
- Underground Storage along Little River Boulevard between St. Pierre Street and Barry Avenue;
- Manning Road Phase 2 Drain Enclosure; and
- Surface storage along Lemire Street and Lanoue Street at Buster Reaume Park.

Localized Solutions East of Manning Road/County Road 19
- Tecumseh Road Storm Sewer Extension;
- Surface Storage along St. Gregory's Road at Tecumseh Soccer Fields; and
- Starwood Lane/Southwind Crescent Pump Station.

Expanded details of each design are outlined in the following sections. The input files for the final future condition PCSWMM model are included in Appendix B. Output files are available on request.

11.1 West of Manning Road/CR19 Recommended Surface Flooding Solutions

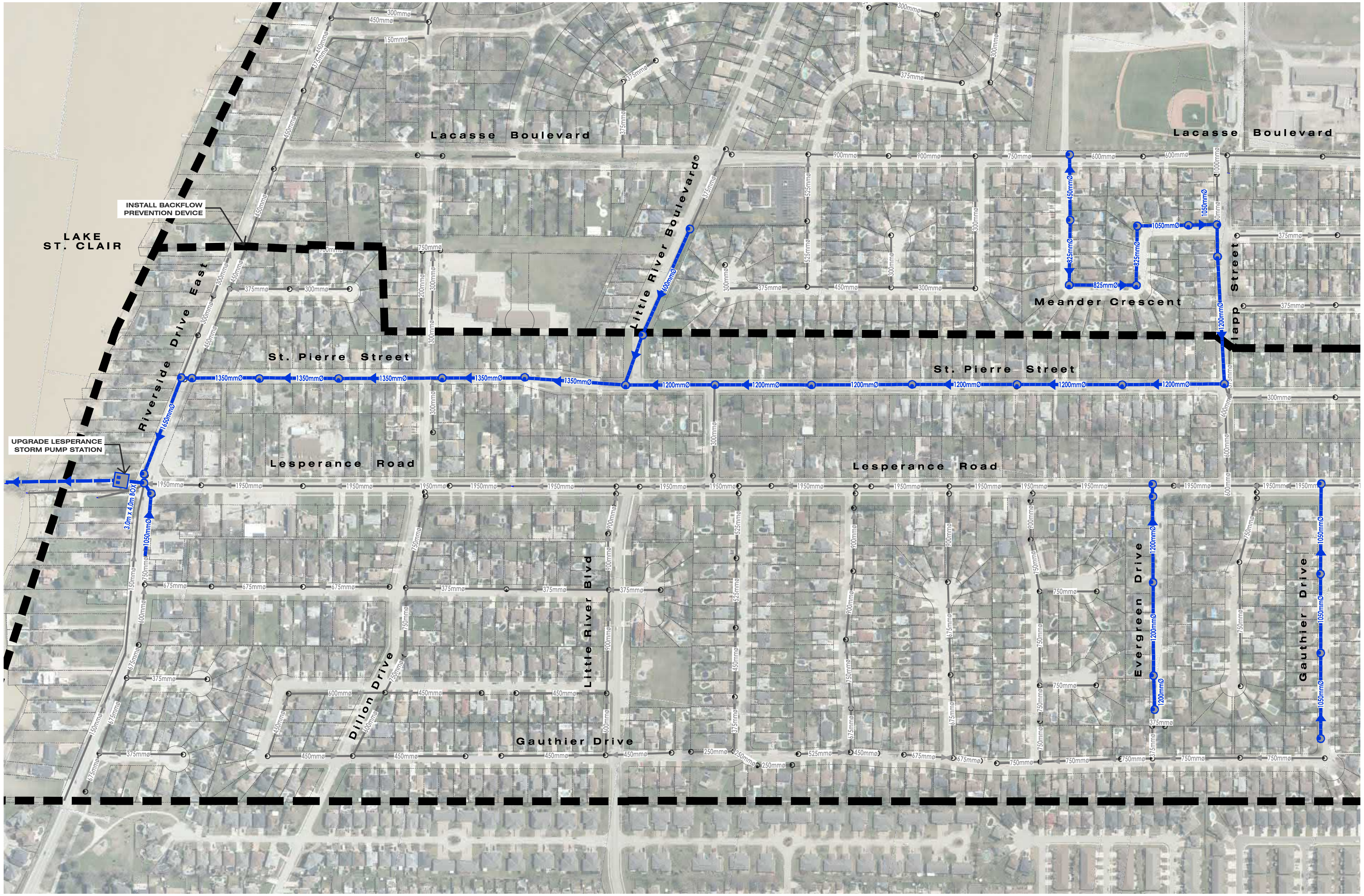
11.1.1 W-**1** Regional Solutions: Lesperance Pump Station Service Area: ALTERNATIVE **2**

The recommended regional surface flooding solution for the Lesperance pump station service area include upgrades to the Lesperance storm pump station and the construction of a new storm trunk sewer along St. Pierre Street. The solutions for this alternative were evaluated for both a traditional and adaptive analysis for an enhanced level of service based on the decision framework outlined in Section 9.0 of the report.

This solution included the introduction of a new storm trunk sewer along St. Pierre Street from Clapp Street to Riverside Drive and along Riverside drive from St. Pierre Street to the proposed improved Lesperance storm pump station. Based on the analysis of the service area and the decision making process, the Lesperance storm pump station was determined to warrant an enhanced level of service with a firm pump capacity upgrade to 9 m^3/s. The enhanced level of service was determined for the pump station improvements due to the effect the pumping capacity has on the HGL throughout the municipal storm sewer system during larger storm events and the pump stations being the primary storm outlet for the large urbanized storm service area.

Under existing conditions, the St. Pierre storm system would be directed to the Lesperance storm trunk sewer through three separate storm connections along Clapp Street, Wood Street and Riverside Drive. The three storm sewer connections are proposed to be abandoned with the incorporation of the new storm trunk sewer. The St. Pierre storm trunk sewer is proposed to range in storm sewer sizes of 1200mm – 1500mm diameter circular storm sewer.

The new St. Pierre trunk sewer improvements would provide an increased storm conveyance capacity and level of service within the local St. Pierre Street storm sewer system during frequent storm events. Surface ponding depths in some isolated road sags within the problem area are still identified as being slightly higher than the recommended maximum of 0.30 m during the 1:100 year design simulation under the recommended solution future model conditions. These areas are localized and isolated in nature and do not have clear overland flow paths to relieve the major system. Grading improvements around these areas may not be feasible due to adjacent existing private property grades and the proximity of the roadways to Lake St. Clair since elevations along the roadways cannot be designed to be lower than 0.30 m below the 1:100 year Lake St. Clair water surface elevations. Figure 11.1 represents the recommended solutions within the Lesperance pump station service area, including all localized storm sewer improvements.



PROPOSED MANHOLE

PUMP STATION

450mmØ PROPOSED STORM SEWER

EXISTING MANHOLE

100mmØ EXISTING STORM SEWER

P.S. SERVICE AREA BOUNDARY

TOWN OF TECUMSEH

STORM DRAINAGE MASTER PLAN

RECOMMENDED SURFACE FLOODING MITIGATION SOLUTION: WEST OF MANNING ROAD - LESPERANCE PS SERVICE AREA

FIGURE 11.1

TOWN OF Tecumseh ONTARIO · CANADA

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11.1.2 W-1 Regional Solutions: West St. Louis Pump Station Service Area: ALTERNATIVE 1

Regional solutions recommended in the West St. Louis storm pump station service area include upgrading the West St. Louis storm pump station capacity. The solutions within this alternative were evaluated for both a traditional and a more adaptive analysis for an enhanced level of service based on the decision framework discussed in Section 9.0 of the report

Based on the analysis of the service area and the decision making process illustrated previously, the West St. Louis storm pump station was determined to warrant an enhanced level of service with a firm pump capacity upgrade to 7 m^3/s. The enhanced level of service was determined for the pump station improvements due to the effect the pumping capacity has on the HGL throughout the municipal storm sewer system during larger storm events and the pump stations being the primary storm outlet for the large urbanized storm service area.

For the West St. Louis pump station service area, no significant regional surface flooding solutions other than the pump station improvements detailed above were identified to be required to reduce surface flooding within the problem area. Localized road and storm sewer upgrades based on the Town of Tecumseh's long term capital works plan and underground storage within the regional surface flooding area were recommended and further designed to provide added resiliency to improve surface flooding in the area. Some areas within the service area were identified to require an enhanced level of service, as outlined in previous sections of the report. Recommended solutions within the service area are listed below.

<u>Traditional Level of Service Design Solutions</u>

- Coronado Dish Area:
  - The "Coronado Dish Area" currently has a semi-urban roadway cross-section, with roadside ditches and underdrains for stormwater conveyance. In proposed conditions, an urban cross-section is proposed with roadway grading improvements including roadway catch basins and storm sewers to convey runoff. Proposed storm sewer sizes range from 375 mm to 900 mm diameter.
  - Storm sewers along Lacasse Boulevard, north of Little River Boulevard, was upgraded to provide better conveyance of runoff within the localized area. Storm sewer sizes in the proposed conditions ranged from 375 mm to 525 mm diameter.
  - Storm sewers along Dillon Drive between Coronado Drive and St. Pierre Street are proposed to be upgraded to range in size from 375 mm to 675mm in diameter.
- Catch basin inlet capacity improvements are recommended along Michael Drive, Revland Drive and Woodbridge Drive; and
- Installation of a backflow prevention device at the storm sewer interconnection of the Lesperance and West St Louis storm pump station service areas along Riverside Drive, between St. Pierre Street and Lacasse Boulevard.

Enhanced Level of Service Design Solutions

- Storm sewers along Barry Avenue and Riverside Drive were upgraded to provide an enhanced level of service to provide relief from surface flooding and better conveyance to the upgraded West St. Louis storm pump station. Proposed storm sewer sizes ranged from 675 mm to 1350 mm diameter.

Figure 11.2 represents the recommended solutions within the West St. Louis pump station service area, including localized improvements.

(symbol)	PROPOSED MANHOLE	(symbol)	EXISTING MANHOLE	(symbol)	P.S. SERVICE AREA BOUNDARY
(symbol)	PUMP STATION	100mmØ	EXISTING STORM SEWER		
450mmØ	PROPOSED STORM SEWER				

TOWN OF TECUMSEH

STORM DRAINAGE MASTER PLAN

RECOMMENDED SURFACE FLOODING MITIGATION SOLUTIONS: WEST OF MANNING ROAD - WEST ST. LOUIS PS SERVICE AREA

FIGURE 11.2

TOWN OF Tecumseh ONTARIO · CANADA

DILLON CONSULTING

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11.1.3 W-1 Localized Solutions

11.1.3.1 Lesperance Road within Tecumseh Centre Park

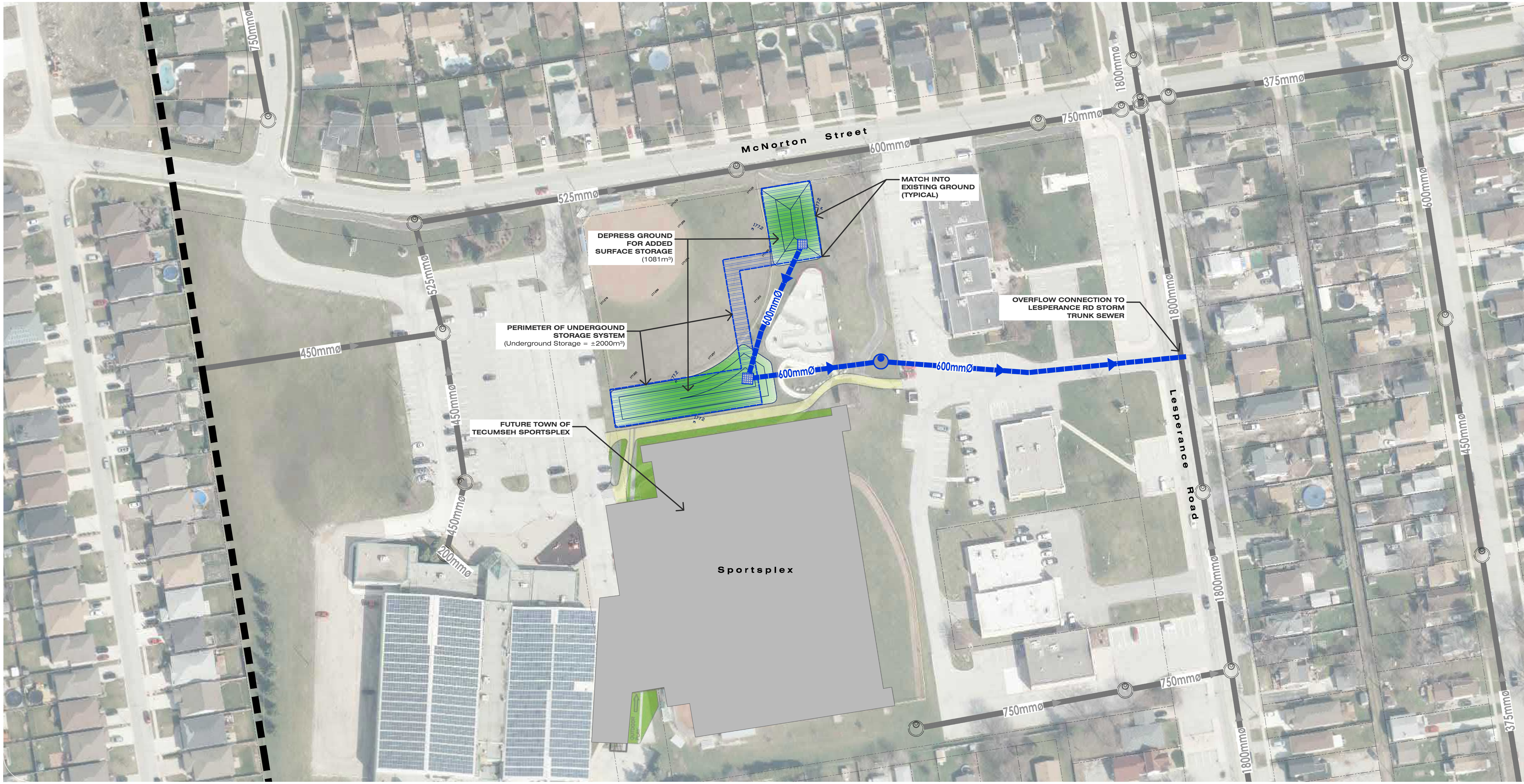
Underground and surface storage of stormwater runoff in the park within Tecumseh Centre Park is proposed to provide added resiliency to improve surface flooding in the Lesperance pump station service area, specifically along Lesperance Road fronting essential emergency service buildings including the Tecumseh OPP Police Station and Tecumseh Fire.

The following SWM solution has been recommended:

- Depression of open space green areas for approximately 1,081 m^3 of surface storage within Tecumseh Centre Park; and
- Incorporation of approximately 2,000 m^3 of underground chamber system storage (Modelled as a series of Stormtech MC4500 units).

The SWM solution is proposed to be connected to the Lesperance storm trunk sewer through an overflow sewer from the municipal system. The surface storage and underground storage chambers were assessed under the decision framework for an enhanced level of service and it was determined that the solution only warranted a traditional level of service based on the extent of surface flooding along Lesperance adjacent to the Town Fire Hall and Police Station directly south of Tecumseh Town Hall. This stormwater storage system is proposed to only be used during larger storm events where water levels in the Lesperance storm trunk sewer can surcharge through the storm connection and into the underground storage chambers. Surface storage will not be used until the water levels within the system reach the surface and the underground storage is fully utilized.

Figure 11.3 provides a visual representation of the location of the surface and underground storage areas and the overflow storm sewer connecting them to the Lesperance trunk storm sewer.



PROPOSED MANHOLE

450mmØ PROPOSED STORM SEWER

PROPOSED CATCH BASIN

EXISTING MANHOLE

100mmø EXISTING STORM SEWER

DEPRESSED LAND

UNDERGROUND STORAGE CHAMBERS (STORMTECH MC-4500)

P.S. SERVICE AREA BOUNDARY

TOWN OF TECUMSEH

STORM DRAINAGE MASTER PLAN

RECOMMENDED SOLUTION: LESPERANCE ROAD LOCALIZED SOLUTION (TECUMSEH CENTRE PARK)

FIGURE: 11.3

TOWN OF Tecumseh ONTARIO · CANADA

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11.1.3.2 Evergreen Drive and Gauthier Drive

Localized solutions to reduce surface flooding along Evergreen Drive and Gauthier Drive are recommended as part of this study and were designed for a traditional level of service. These solutions include underground storage in storm sewer pipes and catch-basin inlet restrictions to reduce the HGLs in the storm sewers.

Proposed improvements include:

- Storm sewers along Evergreen Drive upgraded to 1200 mm diameter;
- Storm sewers along Gauthier Drive upgraded to 1050 mm diameter; and
- Existing orifices installed at the outlet to the Lesperance Road storm trunk sewer is maintained.

Figure 11.1 illustrates the proposed improvements along both Evergreen Drive and Gauthier Drive.

11.1.3.3 Meander Crescent and Clapp Street

As part of the solutions for the Lesperance storm pump station service area within Problem Area W-1, a localized solution for a traditional level of service along Meander Crescent and Clapp Street is proposed to reduce surface flooding.

Proposed improvements include:

- Storm sewer, catch basin and roadway grading improvements along Meander Crescent. Proposed storm sewer sizes along Meander Crescent range from 450 mm to 1050 mm diameter;
- Redirection of storm sewers along Meander Crescent from outletting to the Lacasse Boulevard storm sewer to the new storm trunk sewer along St. Pierre Street through the Clapp Street storm sewer;
- Intersection grading improvements at Meander Crescent and Lacasse Boulevard and Meander Crescent and Clapp Street to limit overland flow contributions during larger storm events from the adjacent roadways into Meander Crescent; and
- 1350mm diameter storm sewer as underground storage and catch basin improvements along Clapp Street, from Meander Crescent to St. Pierre Street.

Figure 11.1 illustrates the proposed improvements along Meander Crescent.

11.1.3.4 Little River Boulevard

Additional localized solutions are proposed within the West St. Louis storm pump station service area within Problem Area W-1 to improve surface flooding in the service area. The solutions within this alternative were evaluated for both a traditional and a more adaptive analysis for an enhanced level of service based on the decision framework discussed in Section 9.0 of the report

Based on the analysis of the service area and the decision making process illustrated previously, the localized Little River Boulevard solution requires an enhanced level of service. The enhanced level of service was warranted in this area due to the effect the HGL within the storm sewer system has on surface flooding along the roadway and the overland runoff to adjacent lower lying areas along private property.

Proposed improvements include:

- Underground storage along Little River Boulevard, from Lacasse Boulevard to Michael Drive, in the form of 1500 mm diameter storm sewers;
- Storm sewer upgrades along Jelso Place, Kimberly Drive and Lacasse Boulevard. Storm sewer sizes in the proposed conditions ranged from 525 mm diameter to 1200 mm diameter;
- 600 mm diameter overflow sewer from the Little River Boulevard storm sewer to the new storm trunk sewer along St. Pierre Street; and
- Increased catch basin inlets along Jelso Place, Kimberly Drive, Lacasse Boulevard and Little River Boulevard.

Figure 11.1 and Figure 11.2 illustrate the proposed improvements along Little River Boulevard.

11.1.3.5 Lesperance/West St. Louis Storm Sewer Interconnection

- Incorporation of a backflow prevention device at the shared storm manhole between the two service areas along Riverside Drive directly east of St. Pierre Street, as identified on Figure 11.1.

11.1.4 W-2 Regional Solution: East St. Louis Pump Station Service Area: ALTERNATIVE 1

This recommended solution in the East St. Louis storm pump station service area includes the incorporation of a 1050 mm diameter storm sewer overflow for the existing 1350 mm diameter storm trunk sewer along St. Thomas Street at the intersection with the Green Valley Drive storm sewer. The overflow sewer has been designed for a traditional level of service for the service areas and is proposed to convey flows easterly down St. Thomas Street and through the proposed 1.8 m x 3.0 m box culvert crossing Manning Road to the Lakewood Park Drainage Channel, eventually into the existing East Townline Drain storm pump station. The box culvert is proposed to be constructed as part of the Manning Road Phase 2/3 project which also includes the enclosure of the East Townline Municipal Drain south of St. Thomas Street and redirection of flows to the Lakewood Park Drainage Channel through the box culvert.

Figure 11.4 illustrates the recommended solution in the East St. Louis storm pump station service area.

11.1.5 W-2 Localized Solution

11.1.5.1 Manning Road Phase 2 Drain Enclosure

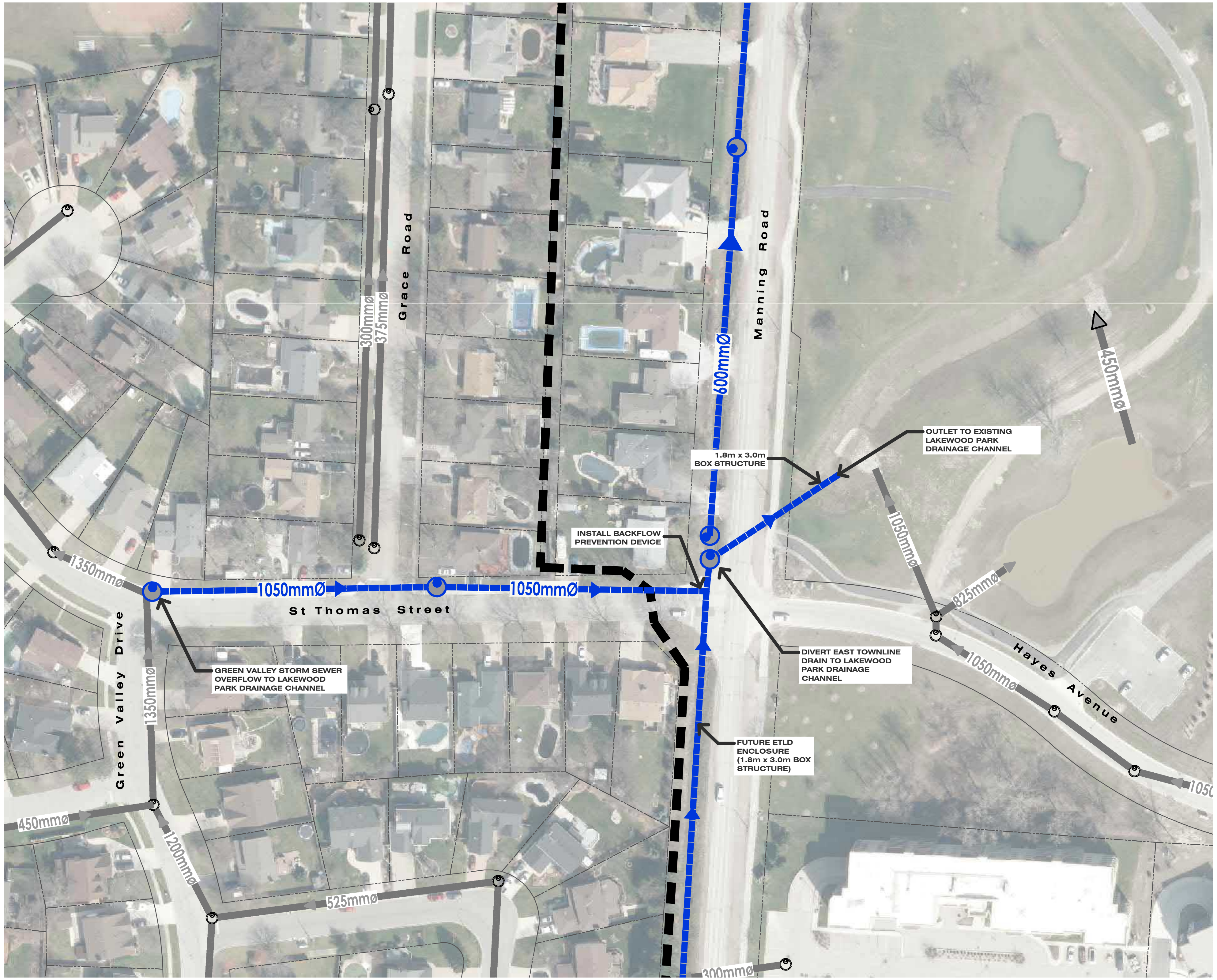
The following storm sewer drainage improvements are proposed to satisfy the final stages of the ultimate stormwater management strategy for the East Townline Drain and Manning Road. These improvements were incorporated into the future conditions model and consist of the following:

- Enclosure of the East Townline Drain (ETLD) between the existing culvert outlet north of St. Gregory's Road, to the proposed outlet at St. Thomas Street to the Lakewood Park Drainage Channel. The completion of this culvert extension will re-direct the ETLD flows from the existing ditch on the west side of Manning Road to the newly constructed open channel in the Lakewood Park; and
- Construction of a local storm sewer system servicing Manning Road residential properties, between Riverside Drive and St. Thomas Street, including the backfill of the existing swale on the west side of Manning Road.

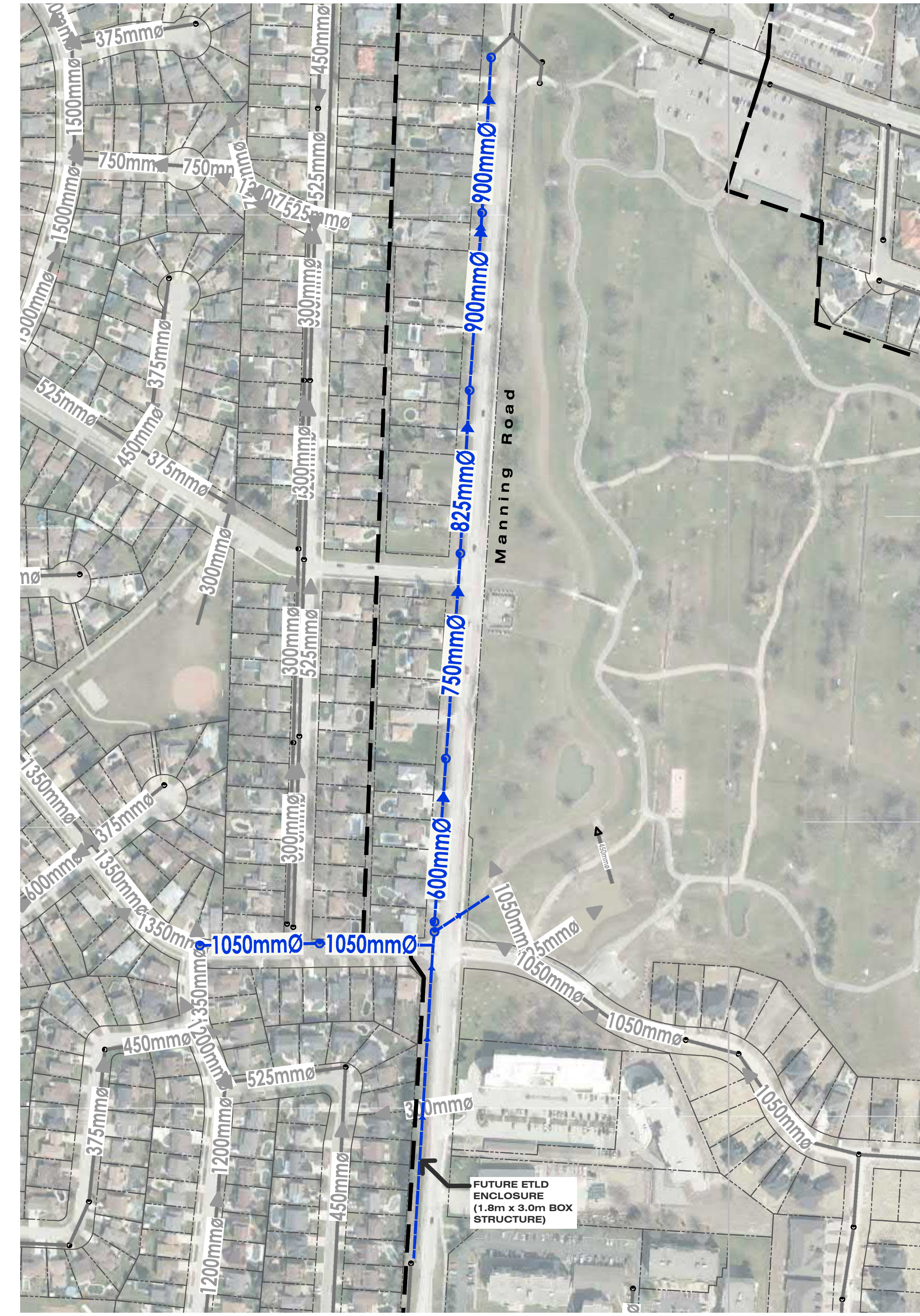
Phase 3 of the project includes the reconstruction of Manning Road between Riverside Drive and St. Thomas Street, including the construction of two roundabouts, re-grading and upgrade to an urban cross section.

Figure 11.4 illustrates the proposed drainage improvements as part of this project.

EAST ST. LOUIS PS SERVICE AREA RECOMMENDED SOLUTION



MANNING ROAD PHASE 2 DRAIN ENCLOSURE DESIGN



PROPOSED MANHOLE
450mmØ PROPOSED STORM SEWER
EXISTING MANHOLE
100mmø EXISTING STORM SEWER
P.S. SERVICE AREA BOUNDARY

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

RECOMMENDED SURFACE FLOODING MITIGATION SOLUTIONS: WEST OF MANNING ROAD - EAST ST. LOUIS/ EAST TOWNLINE DRAIN PS SERVICE AREA
FIGURE 11.4

TOWN OF Tecumseh ONTARIO · CANADA

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11.1.6 W-3 Regional Solution: Baillargeon Drain Service Area: ALTERNATIVE 1 or ALTERNATIVE 2

The St. Anne Area within the Baillargeon Drain service area currently has a semi-urban roadway cross-section, with roadside ditches and subdrains for stormwater conveyance. For the future conditions, an urban cross section is proposed with roadway and storm sewer improvements within the St. Anne Area, including new catch basins. The St. Anne Area includes the following areas:

- St. Anne Street between North Pacific Avenue and Gouin Street; and
- Portions of North Pacific Avenue, Intersection Road, Maisonneuve Street and Gouin Street within the study area;

The solutions for this alternative were evaluated for both a traditional and an adaptive analysis for a more enhanced level of service based on the decision framework discussed in Section 9.0 of the report. It was identified that the solutions warranted a traditional level of service.

Storm sewer sizes in the St. Anne Area range from 450 mm to 1050 mm diameter under proposed conditions. The existing 1350 mm diameter Baillargeon Municipal Drain outlet at Candlewood Drive is expected to be accommodated through the proposed future development, whether through a future enclosure to the East Townline Drain or through the future development trunk sewer and into the future pond (as identified within the 2015 MRSPA Functional Servicing Report and EA Report Addendum).

The final recommended regional solution for this area beyond the design stated above for the St. Anne area is dependent on the availability of utilizing the proposed future development MRSPA area as a storm relief option for the Baillargeon Drain Area.

The initial recommended alternative at this time are as follows:

ALTERNATIVE 1

- Storm relief sewer designed for a traditional level of service ranging from 675 mm to 900 mm diameter along Charlene Lane to intercept storm sewer runoff east of Lesperance Road and south of Charlene Lane from the existing residential lands;
- Relief sewer outlet proposed through the existing 4.0 m storm easement between two residential properties and into the future development storm trunk sewer;
- 600 mm diameter overflow sewer proposed to connect to the Charlene relief sewer from the Lesperance storm sewer system at the intersection of Charlene Lane and Lesperance Road; and
- New 900 mm diameter storm outlet from Gouin Street into the future development storm trunk sewer (As part of the 2015 MRSPA Functional Servicing Report and EA Report Addendum).

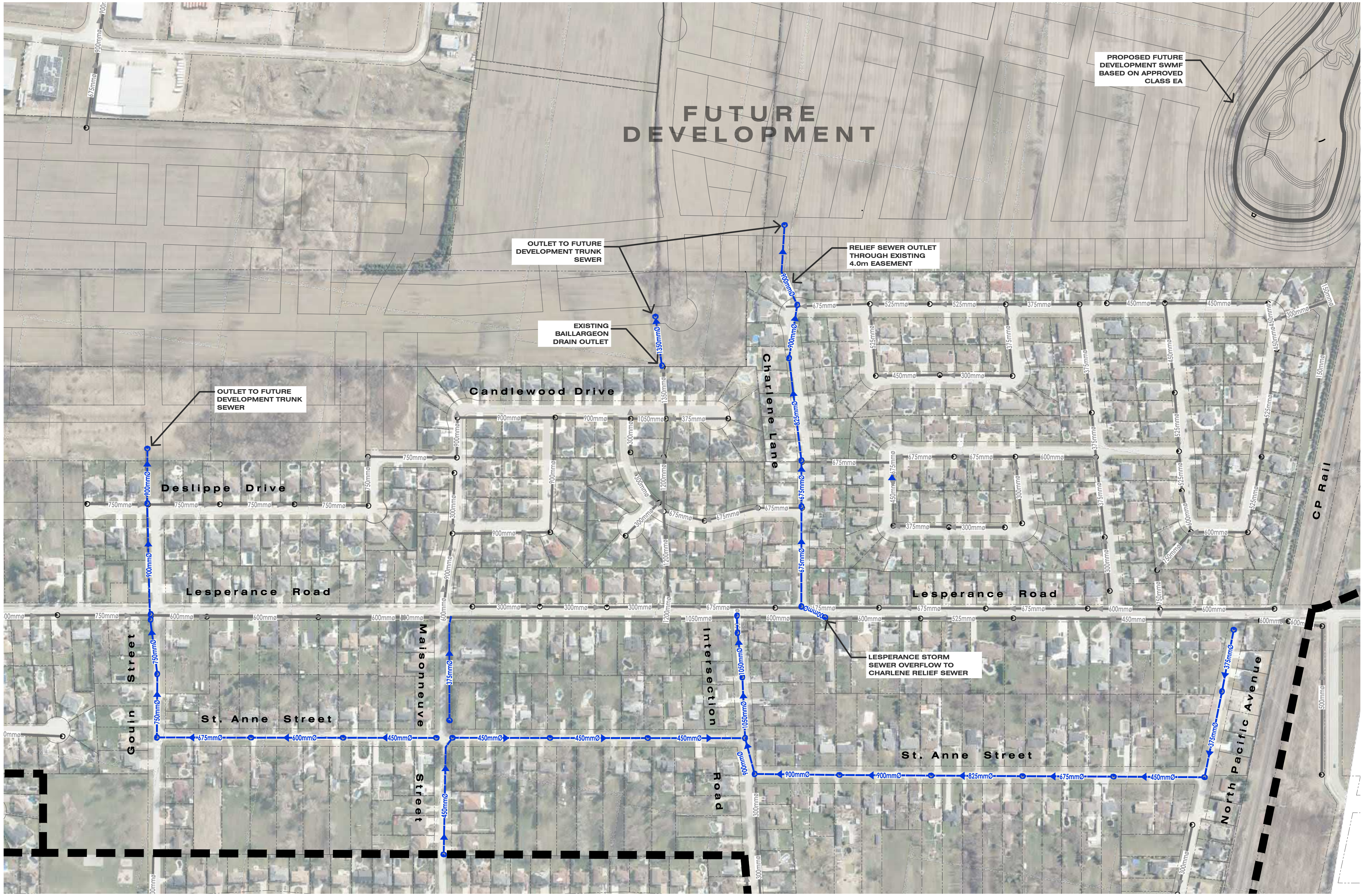
This recommended alternative above is considered the preferred for this area, but is entirely dependent on agreements with the land owners and developers of the future development lands. Alternative 2 has therefore also been presented as a secondary recommended option and detailed below:

ALTERNATIVE 2

- 825 mm diameter underground storage sewer along St. Martin directly upstream of the existing 675 mm diameter storm sewer outlet from St. Martin Crescent to Charlene Lane;
- 900 mm diameter underground storage sewer Charlene Lane directly north of Eugeni Street; and
- 900 mm x 1800 mm underground storage rectangular box chambers along Charlene Lane and St. Agnes upstream of the existing 1200 mm diameter enclosed Baillargeon Drain.

Minor localized surface flooding beyond 0.30 m would still occur within isolated road sags along Lesperance Road directly north of Meconi Drive, but flooding depths have been significantly reduced based on the incorporation of the improvements.

Figure 11.5 and Figure 11.6 represent the two recommended improvements in the Baillargeon Drainage Area.



PROPOSED MANHOLE	EXISTING MANHOLE	P.S. SERVICE AREA BOUNDARY
450mmØ PROPOSED STORM SEWER	100mmØ EXISTING STORM SEWER	

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

RECOMMENDED SURFACE FLOODING MITIGATION SOLUTION OPTION 1: WEST OF MANNING ROAD - BAILLARGEON DRAIN SERVICE AREA
FIGURE 11.5

TOWN OF Tecumseh ONTARIO · CANADA

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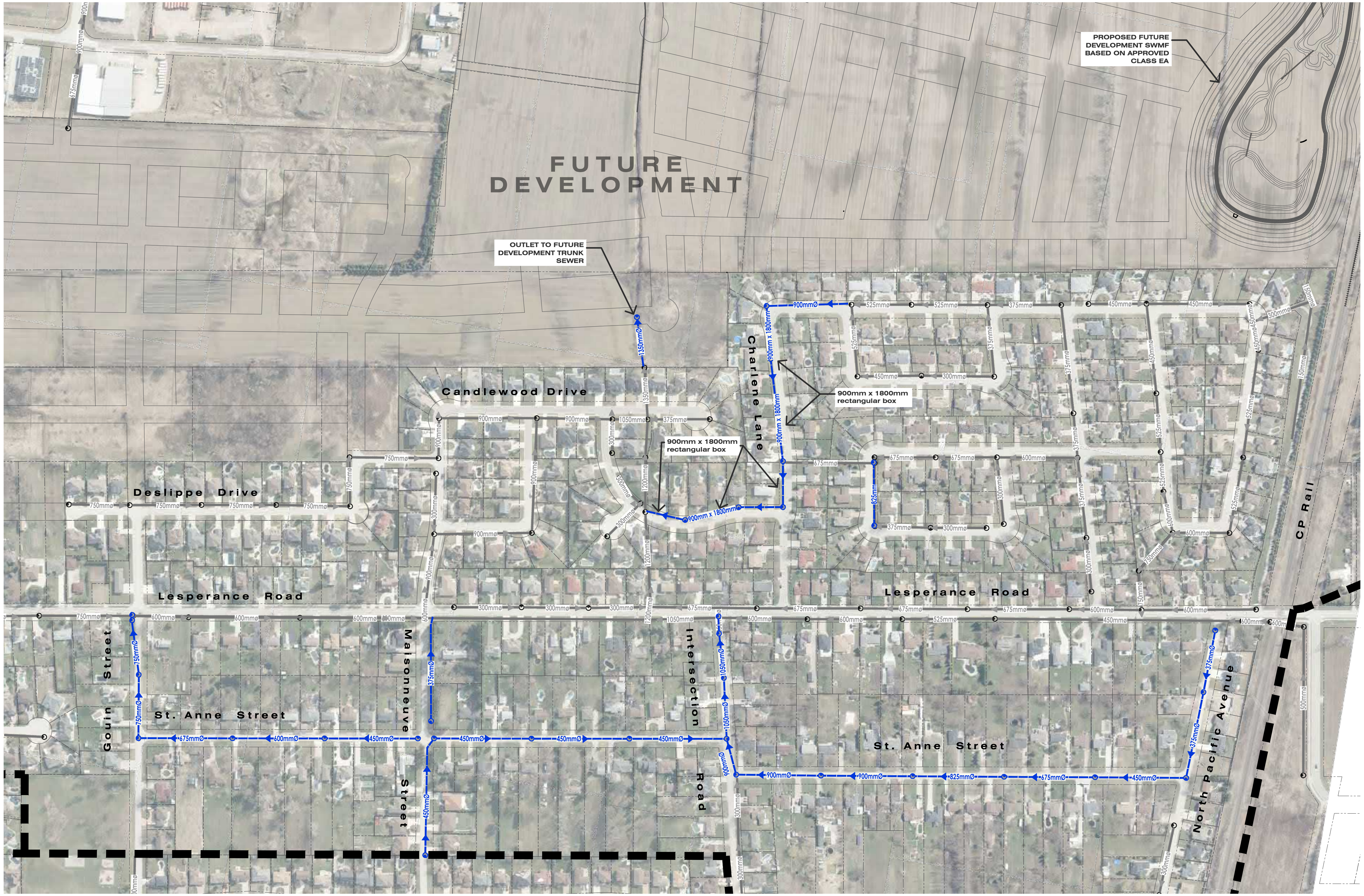

PROPOSED MANHOLE

EXISTING MANHOLE

P.S. SERVICE AREA BOUNDARY

450mmØ PROPOSED STORM SEWER

100mmØ EXISTING STORM SEWER

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

RECOMMENDED SURFACE FLOODING MITIGATION SOLUTION OPTION 2: WEST OF MANNING ROAD - BAILLARGEON DRAIN SERVICE AREA
FIGURE 11.6

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11.1.7 W-3 Localized Solutions

11.1.7.1 Buster Reaume Park/Lemire Street/Lanoue Street

Additional localized solutions are proposed within the East Townline Drain storm pump station service area within Problem Area W-2 to provide the added resiliency to improve surface flooding in the service area. The solutions within this alternative were evaluated for both a traditional and a more adaptive analysis for an enhanced level of service based on the decision framework discussed in Section 9.0 of the report. Based on the analysis of the service area and the decision making process illustrated previously, the Lemire Street and Lanoue Street area was determined to warrant an enhanced level of service. The enhanced level of service was warranted in this area due to the effect the HGL within the storm sewer system has on surface flooding along the roadway and the overland runoff to adjacent lower lying areas on private properties along Lanoue Street.

The following improvements are recommended in the Lemire Street and Lanoue Street area:

- Redirection of storm drainage and upgrade of local storm sewers (675 mm – 825 mm diameter) along Lanoue and Lemire street through to Buster Reaume Park and discharge flows into the existing 900 mm diameter CN Railway Ditch outlet;
- Construction of a 0.80 m depression along the southwestern portion of Buster Reaume Park to provide approximately 4,100 m^3 of aboveground surface storage with a connection to the upgraded municipal storm sewers;
- Installation of a catch basin and a 750 mm diameter storm sewer outlet from the Buster Reaume Park aboveground storage to the existing 900 mm diameter storm sewer to the CN Railway ditch; and
- Installation of a backflow prevention device at the outlet of the 900 mm diameter sewer to the CN Railway ditch within the Buster Reaume pathway from Lemire Street.

Figure 11.7 illustrates the recommended improvements in the Lemire and Lanoue Street area.

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

RECOMMENDED SOLUTION: LEMIRE/LANOUE STREET LOCALIZED SOLUTION (BUSTER REAUME PARK)
FIGURE 11.7

TOWN OF Tecumseh ONTARIO · CANADA

	PROPOSED MANHOLE		EXISTING MANHOLE		DEPRESSED LAND
450mmØ	PROPOSED STORM SEWER	100mmø	EXISTING STORM SEWER		P.S. SERVICE AREA BOUNDARY
	PROPOSED CATCH BASIN		EXISTING CATCH BASIN		

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SCALE: N.T.S.

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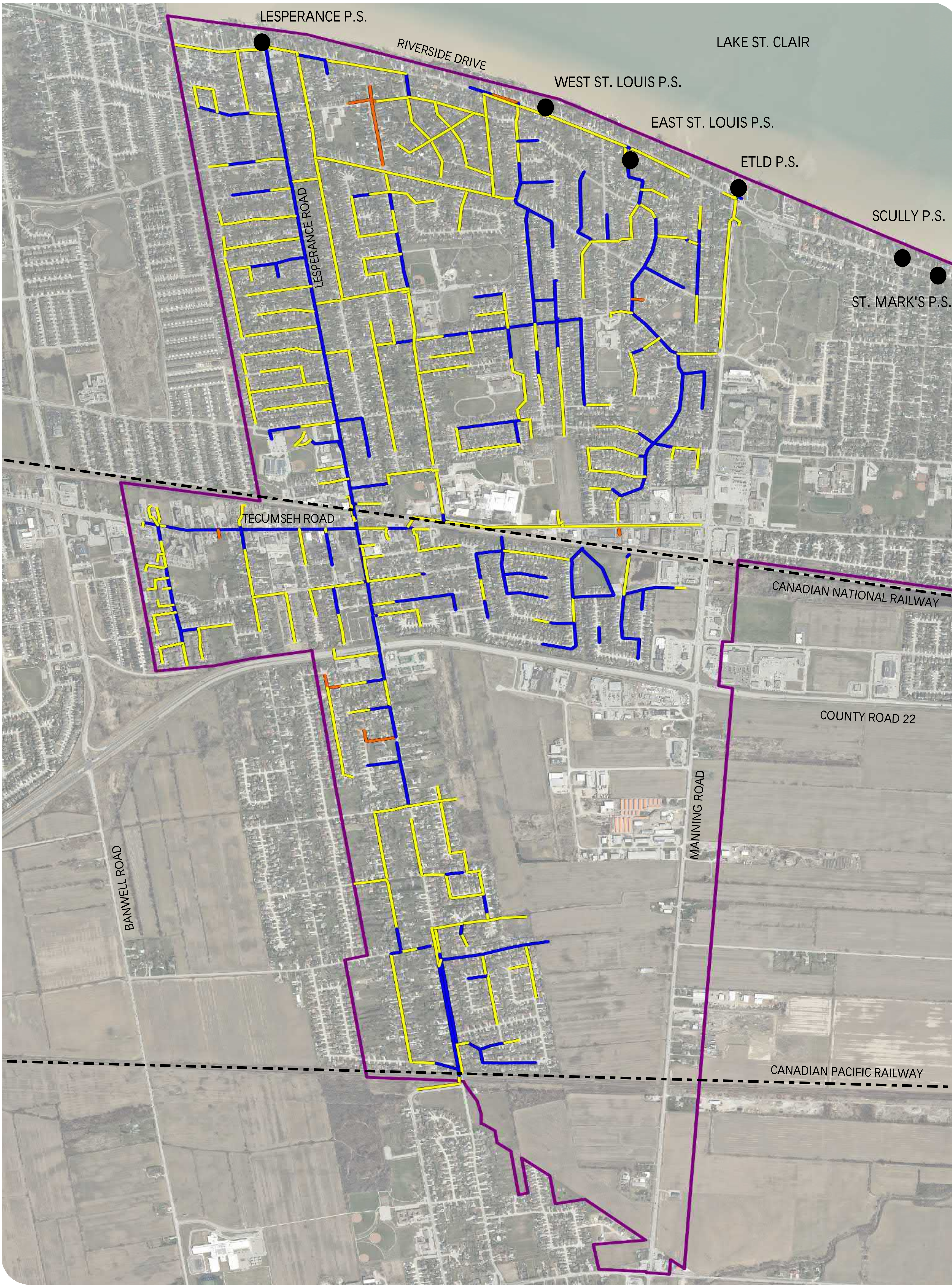
11.1.8 Future Conditions – Calibrated Minor System Analysis

The results from the refined calibrated model developed for a minor system analysis and discussed in Section 7.4 was used to illustrate the existing hydraulic gradeline (HGL) throughout the municipal infrastructure system. The estimated HGL for the minor storm sewer system in the Town of Tecumseh was assessed under a 1:2 year design storm event. A graphic representation of the water levels within the storm sewer was then developed. The depth of flow conveyance within the storm sewer system was summarized in which the HGL elevation during the 1:2 year event was:

1. Maintained within the storm sewer with the sewer flow below 50% full;
2. Maintained within the storm sewer with the sewer flow between 50% and 99% full; and
3. Beyond the obvert of the pipe, but within regional design standards for storm sewer design (ie. no higher than 0.30m below the existing ground elevation).

Based on the hydrologic changes from the revised model calibrated, the recommended surface flooding solutions were incorporated and the model was run for a 1:2 year storm event. Figure 11.8 illustrates the future condition HGL results during the 1:2 storm events from the refined calibrated based on the incorporation of all preferred surface flooding solutions west of Manning Road.

Figure 11.8 illustrates the existing condition HGL results during the 1:2 storm events from the refined model for each of the storm sewer systems modelled within the study area west of Manning Road.



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

WEST OF MANNING ROAD FUTURE CONDITIONS ESTIMATED 1:2 YEAR STORM SEWER HYDRAULIC CONDITIONS

FIGURE 11.8

Town of Tecumseh Ontario · Canada

SEWER GRAVITY OUTFALL	CONDUIT FLOW < 50% FULL
PUMP STATION (P.S.)	CONDUIT FLOW BETWEEN 50% - 99% FULL
STUDY AREA	CONDUIT SURCHARGING ABOVE PIPE OBVERT WITHIN REGIONAL STANDARD LIMIT
RAILWAY	

DILLON CONSULTING

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

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PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

11.2 East of Manning Road/CR19 Recommended Surface Flooding Solutions

11.2.1 E-1 Regional Solution: Scully/St. Mark's/PJ Cecile Pump Station Service Area: ALTERNATIVE 2

The recommended regional solution for Problem Area E-1 includes the decommissioning of the St. Mark's storm pump station and consolidation of its service area with that of the Scully storm pump station. The improvements to the pump stations as part of this solution include the construction of a new consolidated Scully/St. Mark's storm pump station on the Scully site and improvements to the PJ Cecile Storm Pump Station.

The solutions for this alternative were evaluated for both a traditional and a more adaptive analysis for an enhanced level of service based on the decision framework discussed in Section 9.0 of the report. The two pump stations were determined to warrant an enhanced level of service due to the effect the pumping capacity has on the HGL throughout the municipal storm sewer system during larger storm events and the pump stations being the primary storm outlet for the large urbanized storm service area. The two stations were identified to require the following firm pump capacity upgrades:

- Consolidated Scully/St. Mark's pump station firm capacity upgrade to 6.0 m^3/s; and
- PJ Cecile pump station firm capacity upgrade to 3.0 m^3/s.

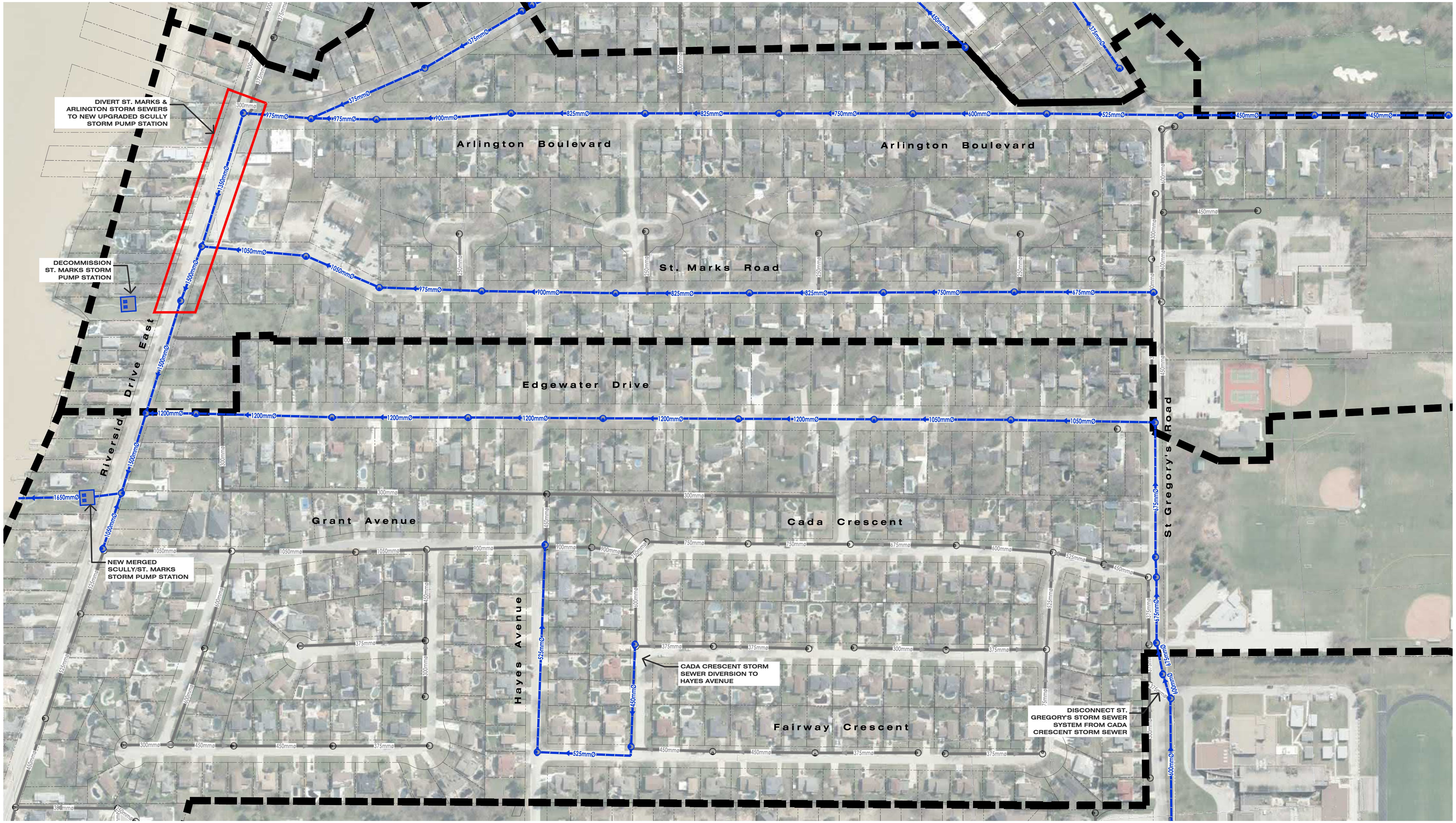
Storm trunk sewer improvements along Riverside Drive are recommended to convey stormwater runoff to the consolidated Scully/St. Mark's pump station and to re-route flows from the existing St. Mark's pump station site to the consolidated Scully/St. Mark's pump station. Storm sewer improvements are recommended along Riverside Drive, from Grant Avenue to Arlington Boulevard within the consolidated Scully/St. Mark's service area. The proposed sewer sizes range from 1050 mm diameter to 1500 mm diameter.

In addition, municipal roadway and storm sewer improvements are recommended in the following areas:

- Edgewater Boulevard – Proposed storm sewer sizes range from 1050 mm diameter to 1200 mm diameter;
- St. Marks Road – Proposed storm sewer sizes range from 675 mm diameter to 1050 mm diameter;
- Arlington Boulevard – Proposed storm sewer sizes range from 375 mm diameter to 1050 mm diameter; and
- Streets within the Kensington Dish Area – Proposed storm sewer sizes ranging from 375 mm diameter to 1050 mm diameter.

Figure 11.9 illustrates the proposed storm sewer improvements along Edgewater Boulevard, St. Mark's Road and Arlington Boulevard along with pump station improvements at the consolidated Scully/St. Mark's pump station site. Figure 11.10 illustrates the proposed storm sewer improvements for the "Kensington Dish" area and pump station improvements for the PJ Cecile pump station.

The areas identified above currently have a semi-urban roadway cross-section, with roadside ditches and underdrains for stormwater conveyance. In proposed conditions, at this time, an urban cross-section is proposed with storm sewers, roadway catch basin and roadway grading improvements. The solutions for this alternative were evaluated for both a traditional and a more adaptive analysis for an enhanced level of service based on the decision framework discussed in Section 9.0 of the report. The storm sewer infrastructure improvements within this area warranted an enhanced level of service due to the effect the HGL within the storm sewer system has on surface flooding along the roadway and the amount of overland runoff to adjacent lower lying areas along private property.



TOWN OF TECUMSEH

STORM DRAINAGE MASTER PLAN

RECOMMENDED SURFACE FLOODING MITIGATION SOLUTIONS: EAST OF MANNING ROAD - CONSOLIDATED SCULLY/ST.MARK'S PS SERVICE AREA

FIGURE 11.9

PROPOSED MANHOLE

PUMP STATION

450mmØ PROPOSED STORM SEWER

EXISTING MANHOLE

100mmØ EXISTING STORM SEWER

P.S. SERVICE AREA BOUNDARY

CREATED BY: JTB
CHECKED BY: RTL
DESIGNED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE: N.T.S.

File Location:
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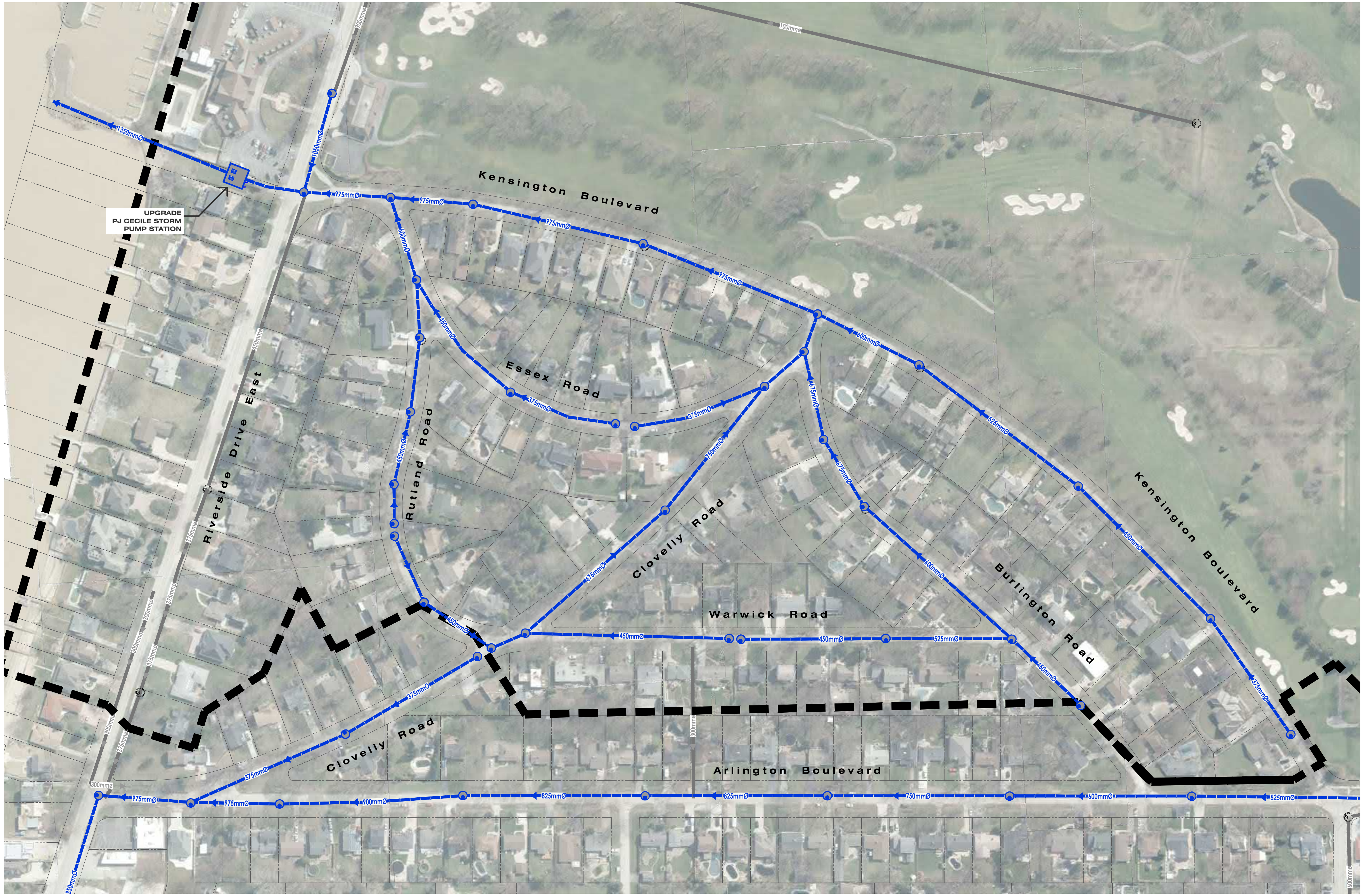
PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019



PROPOSED MANHOLE

PUMP STATION

450mmØ PROPOSED STORM SEWER

EXISTING MANHOLE

100mmØ EXISTING STORM SEWER

P.S. SERVICE AREA BOUNDARY

TOWN OF TECUMSEH

STORM DRAINAGE MASTER PLAN

RECOMMENDED SURFACE FLOODING MITIGATION SOLUTIONS: EAST OF MANNING ROAD - PJ CECILE PS SERVICE AREA

FIGURE 11.10

TOWN OF Tecumseh ONTARIO · CANADA

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DESIGNED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE: N.T.S.

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PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

11.2.2 E-1 Localized Solutions

11.2.2.1 Tecumseh Road Storm Sewer Extension

Tecumseh Road East (east of Manning Road), within the Brighton pump station service area currently has a semi-urban roadway cross-section, with boulevard catch-basins and smaller storm sewers to convey stormwater runoff. In proposed conditions, an urban cross-section is proposed with storm sewers sized for a traditional level of service, roadway catch basin and roadway grading improvements.

The Tecumseh road ditch is proposed to be replaced with storm sewers along Tecumseh Road East, from Dresden Place to tie-in to the existing 1050 mm diameter storm sewer, east of Lexham Gardens. Proposed storm sewer sizes range from 525 mm to 1050 mm in diameter. Figure 11.11 illustrates the proposed storm sewer improvements along Tecumseh Road East.

11.2.2.2 St. Gregory's Road at Tecumseh Soccer Fields

The solutions within this alternative were evaluated for both a traditional and a more adaptive analysis for an enhanced level of service based on the decision framework discussed in Section 9.0 of the report. A traditional solution is proposed along St. Gregory's which includes the following:

- Incorporation of a backflow prevention device at the St. Gregory's Road storm outlet to the East Townline Drain at the intersection of St. Gregory's and Manning Road;
- Elimination of the storm sewer interconnection along St. Gregory's Road fronting L'Essor School with the East Townline Drain and Scully storm pump station service areas;
- Redirection of the storm sewer outlet for St. Gregory's Road East of L'Essor School to extend further east from the Cada Crescent storm outlet to the proposed Edgewater Drive storm sewer; and
- Road grading improvements along St. Gregory's Road, from Village Grove Drive to L'Essor School to reduce surface flooding depths in this area of roadway.

Further analysis identified that an enhanced level of service was warranted to address the extensive surface flooding that occurs within the ingress/egress routes of the L'Essor High School. The following localized solution is therefore recommended for an enhanced design beyond the traditional solution provided above:

- Construction of a 0.70 m depression of the existing northern soccer fields within the Tecumseh Soccer Fields Park currently owned by the L'Essor high school and Conseil Scolaire Catholique (CSC) Providence to provide approximately 3,200 m^3 of aboveground surface storage with a 200mm diameter storm sewer connection to the existing 750 mm diameter storm sewer along St. Gregory's Road; and
- Grading improvements at the two (2) entrances to L'Essor High School.

Figure 11.12 illustrates the recommended expanded localized surface flooding solution along St. Gregory's Road within the Tecumseh Soccer Fields Park.

PROPOSED MANHOLE
EXISTING MANHOLE
P.S. SERVICE AREA BOUNDARY
450mmØ PROPOSED STORM SEWER
100mmø EXISTING STORM SEWER

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

RECOMMENDED SOLUTION: TECUMSEH ROAD LOCALIZED SOLUTION
FIGURE 11.11

CREATED BY: JTB
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DESIGNED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE: N.T.S.

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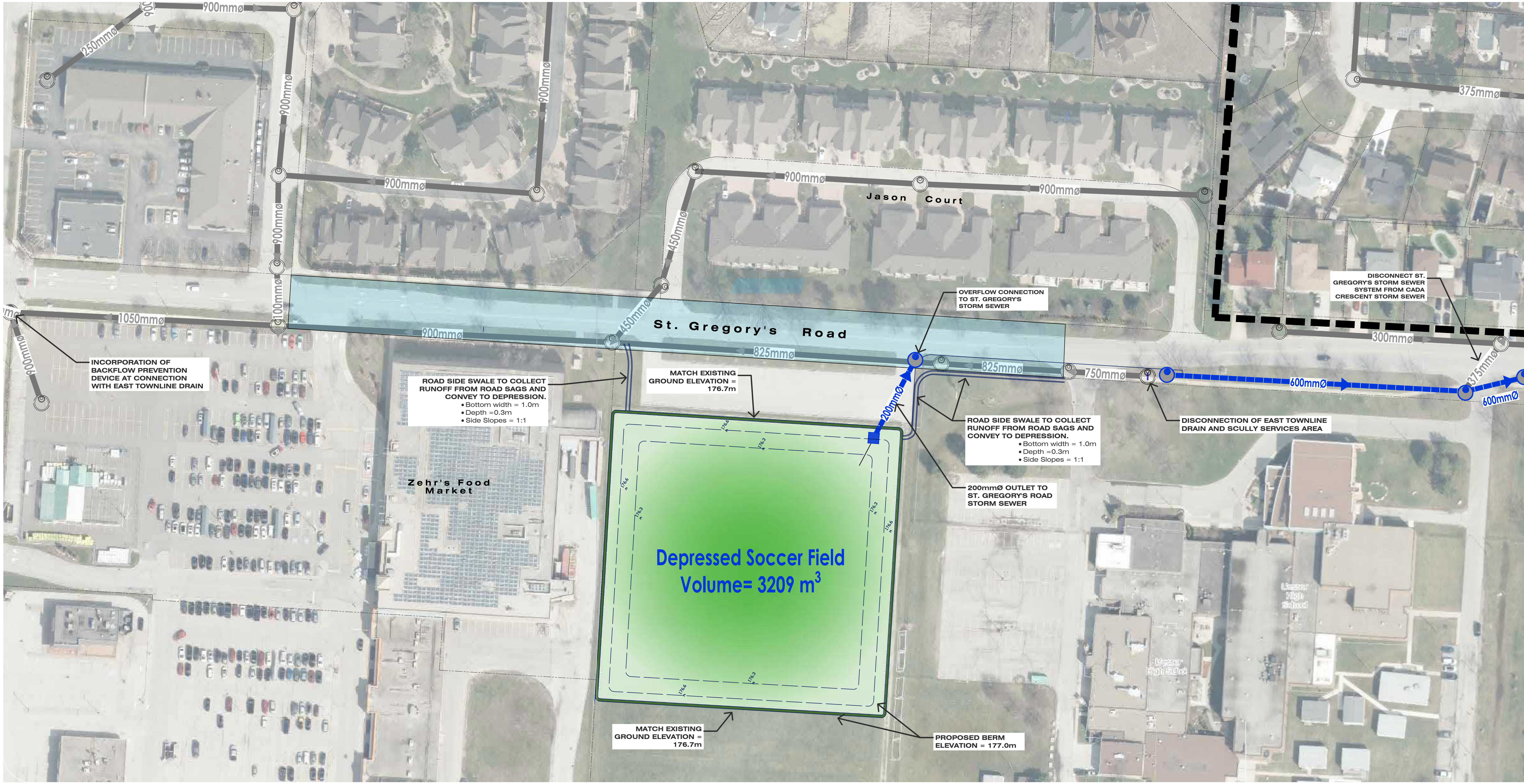

	PROPOSED MANHOLE		EXISTING MANHOLE
450mmØ	PROPOSED STORM SEWER	100mmø	EXISTING STORM SEWER
	PROPOSED CATCH BASIN		
	ROADWAY GRADING IMPROVEMENTS		P.S. SERVICE AREA BOUNDARY
	DEPRESSED LAND		

TOWN OF TECUMSEH

STORM DRAINAGE MASTER PLAN

RECOMMENDED SOLUTION: ST. GREGORY'S ROAD LOCALIZED SOLUTION (TECUMSEH SOCCER FIELDS)

FIGURE 11.12

TOWN OF Tecumseh ONTARIO · CANADA

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DESIGNED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE: N.T.S.

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PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

11.2.2.3 Starwood Lane/Southwind Crescent

A new pump station is proposed to hydraulically disconnect the Starwood Lane and Southwind Crescent storm sewer system from the water levels at the existing storm outlet along Pike Creek. A traditional level of service was warranted for the new pump station with a firm capacity of 0.20 m^3/s.

The proposed storm pump station is to be constructed within an existing 4.5 m wide storm easement with a connection to the existing storm outlet. Through discussions with the adjacent property owners, it was identified that the adjacent residential storm private drain connections are connected to the existing storm outlet pipe to Pike Creek. Under high lake levels, the residential sump pumps run constantly. It is therefore recommended that a backflow prevention device be incorporated at the existing storm outlet to reduce standing water within the existing outlet pipe.

Further details of the localized solution, including the recommended location for the proposed Southwind Crescent/Starwood Lane area pump station is provided in Section 13.5.

11.2.3 Future Conditions – Calibrated Minor System Analysis

The results from the refined calibrated model developed for a minor system analysis and discussed in Section 7.4 was used to illustrate the existing hydraulic gradeline (HGL) throughout the municipal infrastructure system. The estimated HGL for the minor storm sewer system in the Town of Tecumseh was assessed under a 1:2 year design storm event. A graphic representation of the water levels within the storm sewer was then developed. The depth of flow conveyance within the storm sewer system was summarized in which the HGL elevation during the 1:2 year event was:

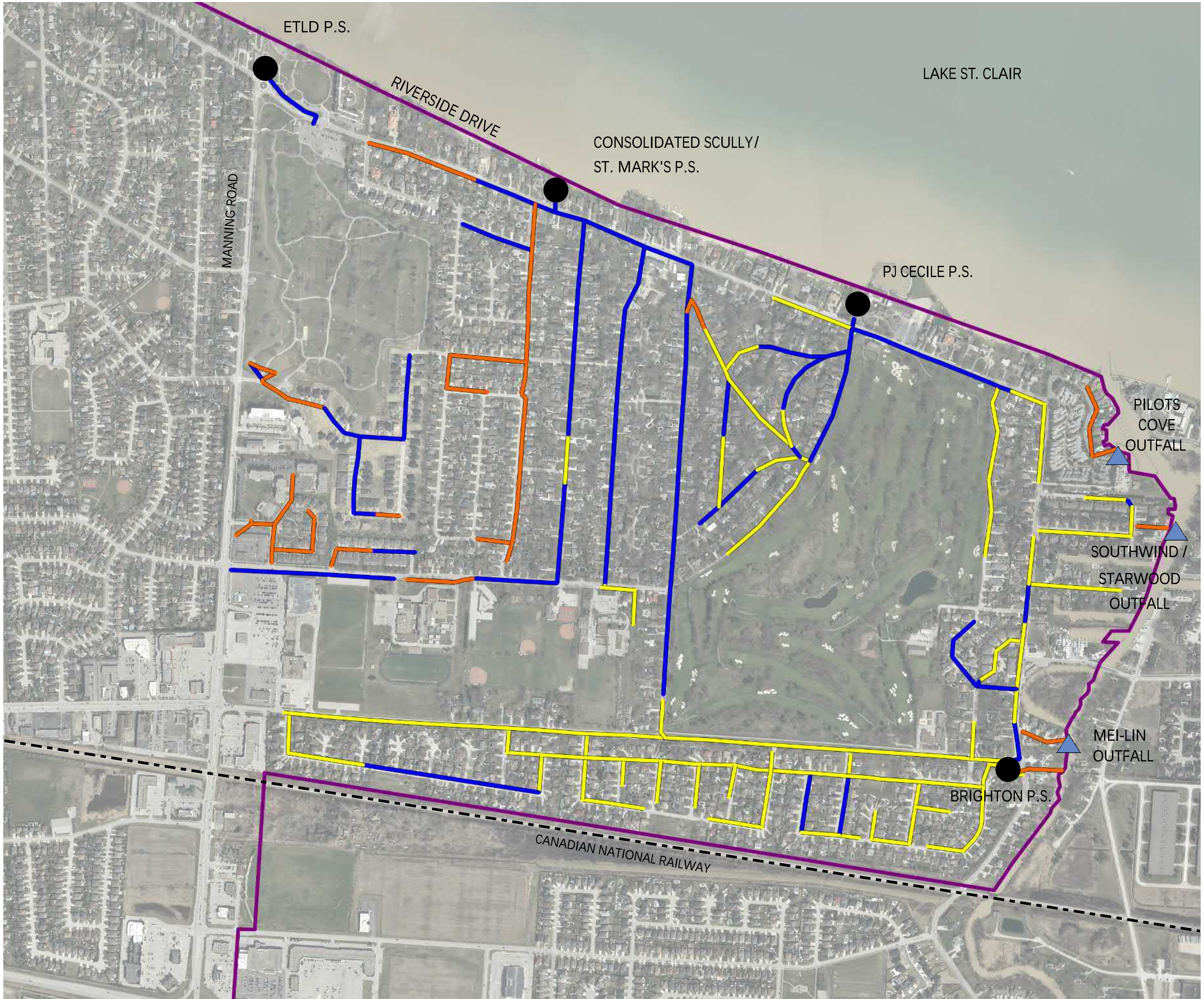
1. Maintained within the storm sewer with the sewer flow below 50% full;
2. Maintained within the storm sewer with the sewer flow between 50% and 99% full; and
3. Beyond the obvert of the pipe, but within regional design standards for storm sewer design (ie. no higher than 0.30m below the existing ground elevation)

Based on the hydrologic changes from the revised model calibrated, the recommended surface flooding solutions were incorporated and the model was run for a 1:2 year storm event. Figure 11.8 illustrates the future condition HGL results during the 1:2 storm events from the refined calibrated based on the incorporation of all preferred surface flooding solutions west of Manning Road.

Figure 11.13 illustrates the existing condition HGL results during the 1:2 storm events from the refined model for each of the storm sewer systems modelled within the study area west of Manning Road.

11.3 Localized Storm Inlet Improvements: East and West of Manning Road

The areas illustrated in Figure 11.14 have been identified to benefit from inlet modifications within the localized area to change the surface ponding conditions within the areas during larger storm events. These modifications include the incorporation of inlet control devices where surface ponding is currently well below the regional allowable storage depth of 0.30 m in localized areas and areas where additional catch basins are proposed to reduce surface ponding in areas currently shown to be above the regional requirements of 0.30 m.



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

EAST OF MANNING ROAD - FUTURE CONDITIONS ESTIMATED 1:2 YEAR STORM SEWER HYDRAULIC CONDITIONS
FIGURE 11.13

- SEWER GRAVITY OUTFALL
- PUMP STATION (P.S.)
- RAILWAY
- CONDUIT FLOW < 50% FULL
- CONDUIT FLOW BETWEEN 50% - 99% FULL
- CONDUIT SURCHARGING ABOVE PIPE OBVERT WITH REGIONAL STANDARD LIMITS
- STUDY AREA

SCALE 1:NTS

2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

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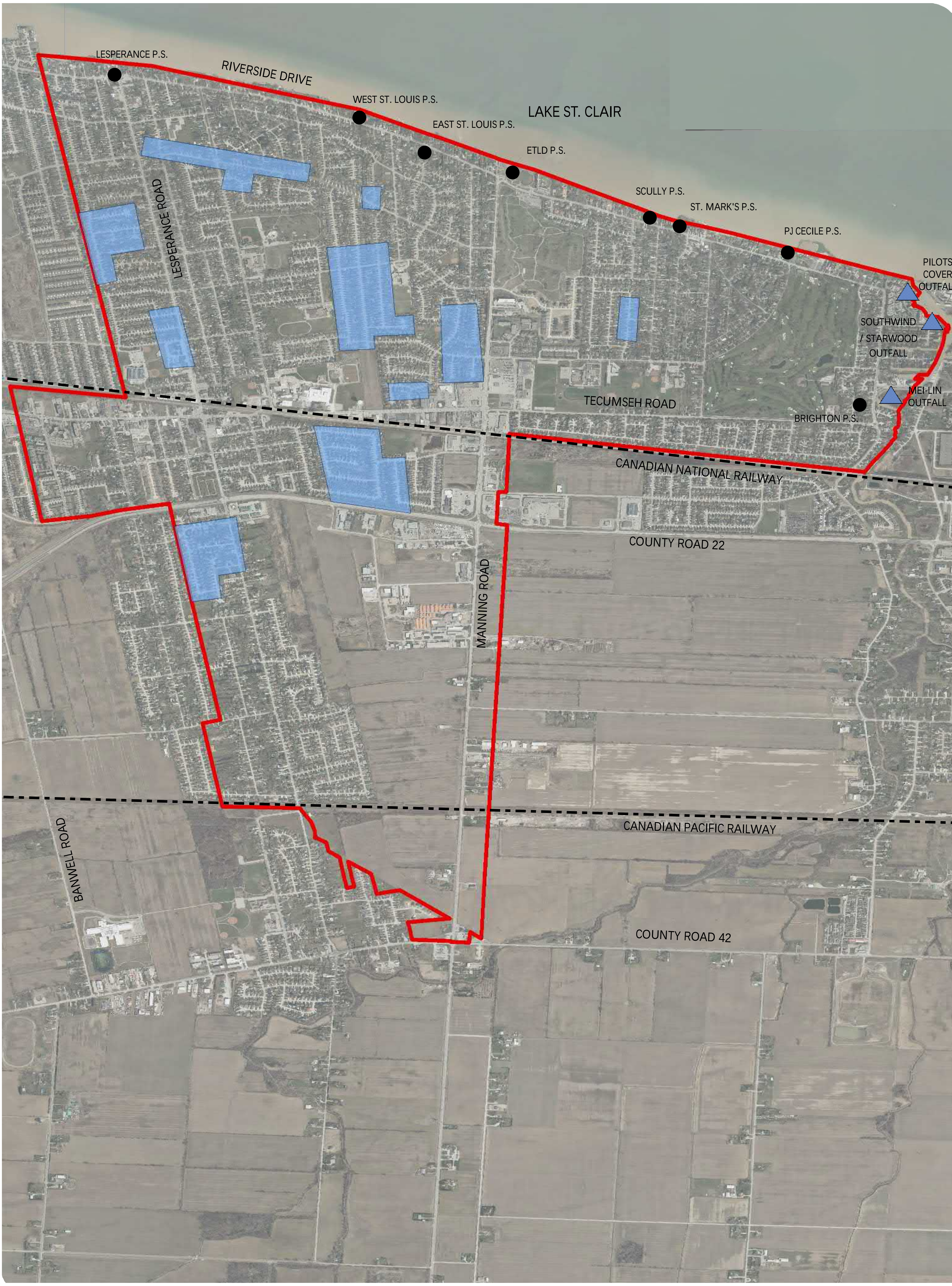


TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

RECOMMENDED LOCALIZED STORM INLET IMPROVEMENT LOCATIONS
FIGURE 11.14

SEWER GRAVITY OUTFALL
PUMP STATIONS (P.S.)
LOCATIONS OF LOCALIZED STORM INLET IMPROVEMENTS
STUDY AREA
RAILWAY

MAP CREATED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

2

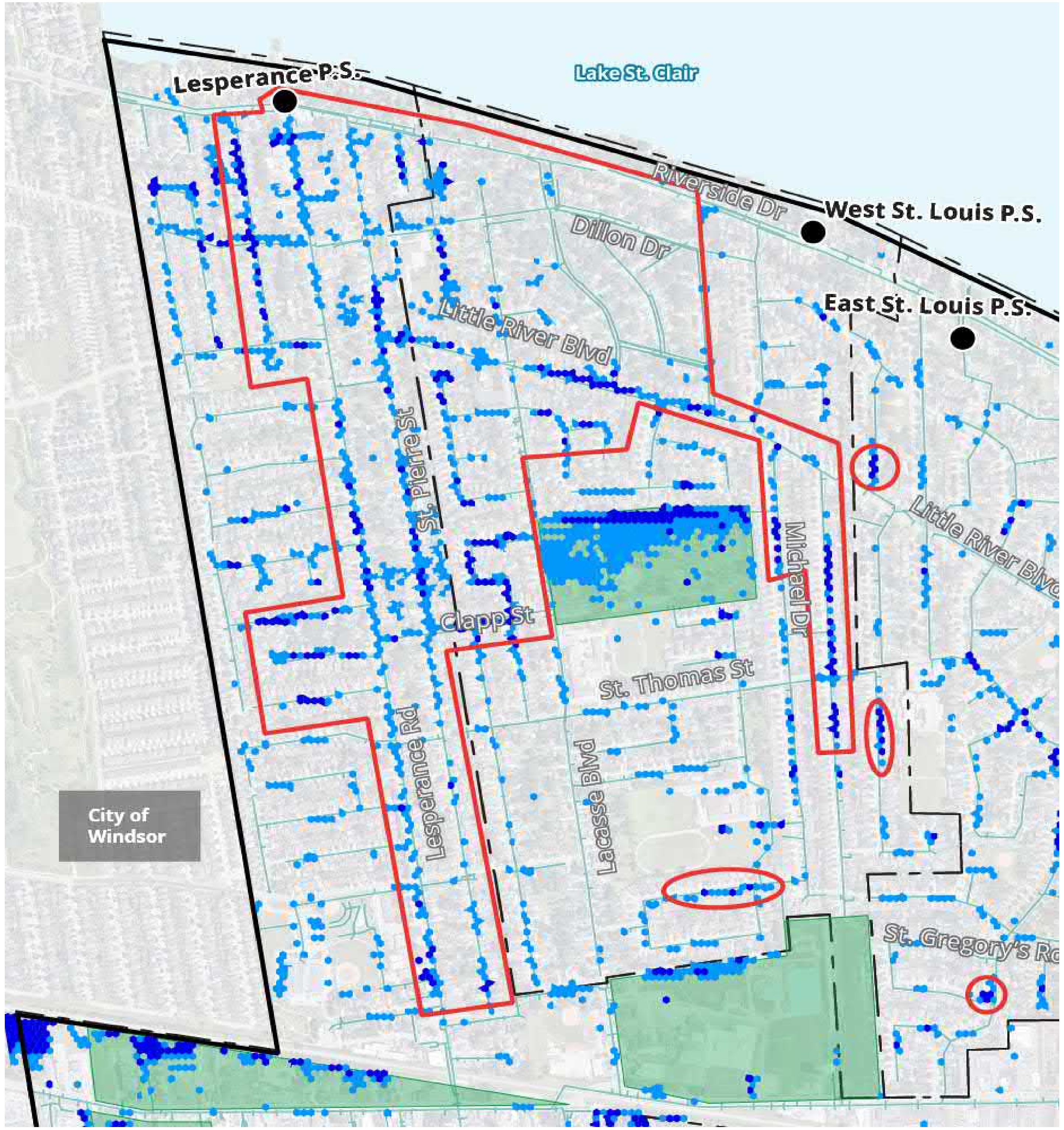
PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

11.4 Future Condition Surface Ponding Summary

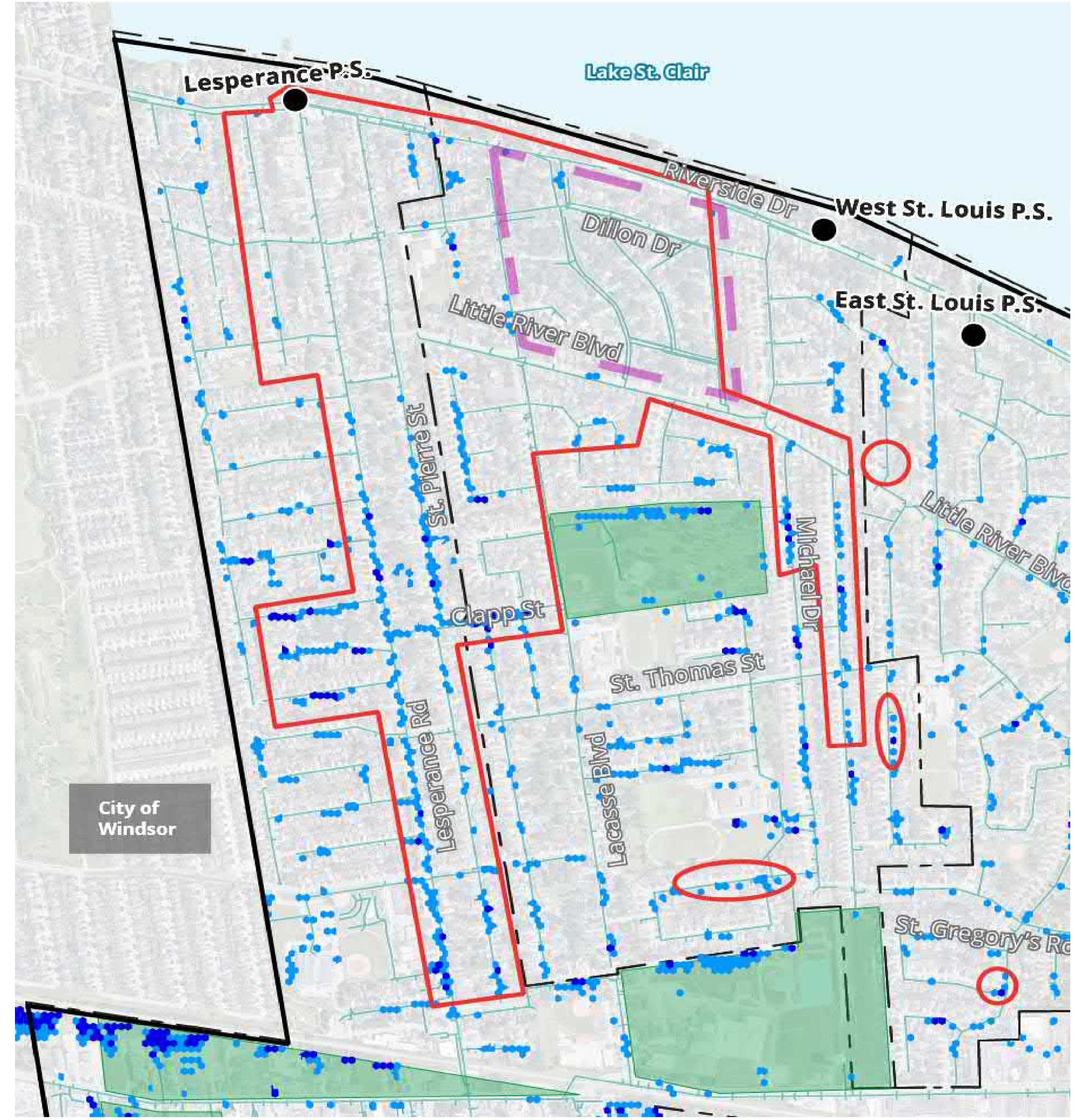
Comparisons of existing versus future surface flooding conditions during both the 1:100 year and 1:100 year + 40% (climate change) events based on the implementation of all recommended solutions throughout the study area are provided in the figures below. The future condition surface ponding during other storm events analyzed are provided in Appendix E for the following:

- 1:5 Year 4 Hour Design Storm Event;
- 1:10 Year 4 Hour Design Storm Event; and
- 1:100 year 24 hour +39% Storm Event (150mm rainfall with a maximum intensity of 145mm/hr).

EXISTING CONDITION (1:100 YEAR SURFACE PONDING SIMULATION)



FUTURE CONDITION (1:100 YEAR SURFACE PONDING SIMULATION)



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

1:100 YEAR EXISTING VS FUTURE CONDITION
SURFACE FLOODING - PROBLEM AREA W-1
FIGURE 11.15

Town of Tecumseh Ontario · Canada

- PUMP STATION (P.S.)
- PUMP STATION SERVICE AREA
- STUDY AREA
- SURFACE PONDING WITHIN AREA MAINTAINED BELOW 0.30m
- REGIONAL SURFACE FLOODING PROBLEM AREAS
- ISOLATED SURFACE FLOODING PROBLEM AREAS
- PARKLAND / PRIVATE PROPERTY NOT TO BE ANALYZED
- SURFACE PONDING BETWEEN 0.15m - 0.30m DEPTH
- SURFACE PONDING > 0.30m DEPTH

DILLON CONSULTING

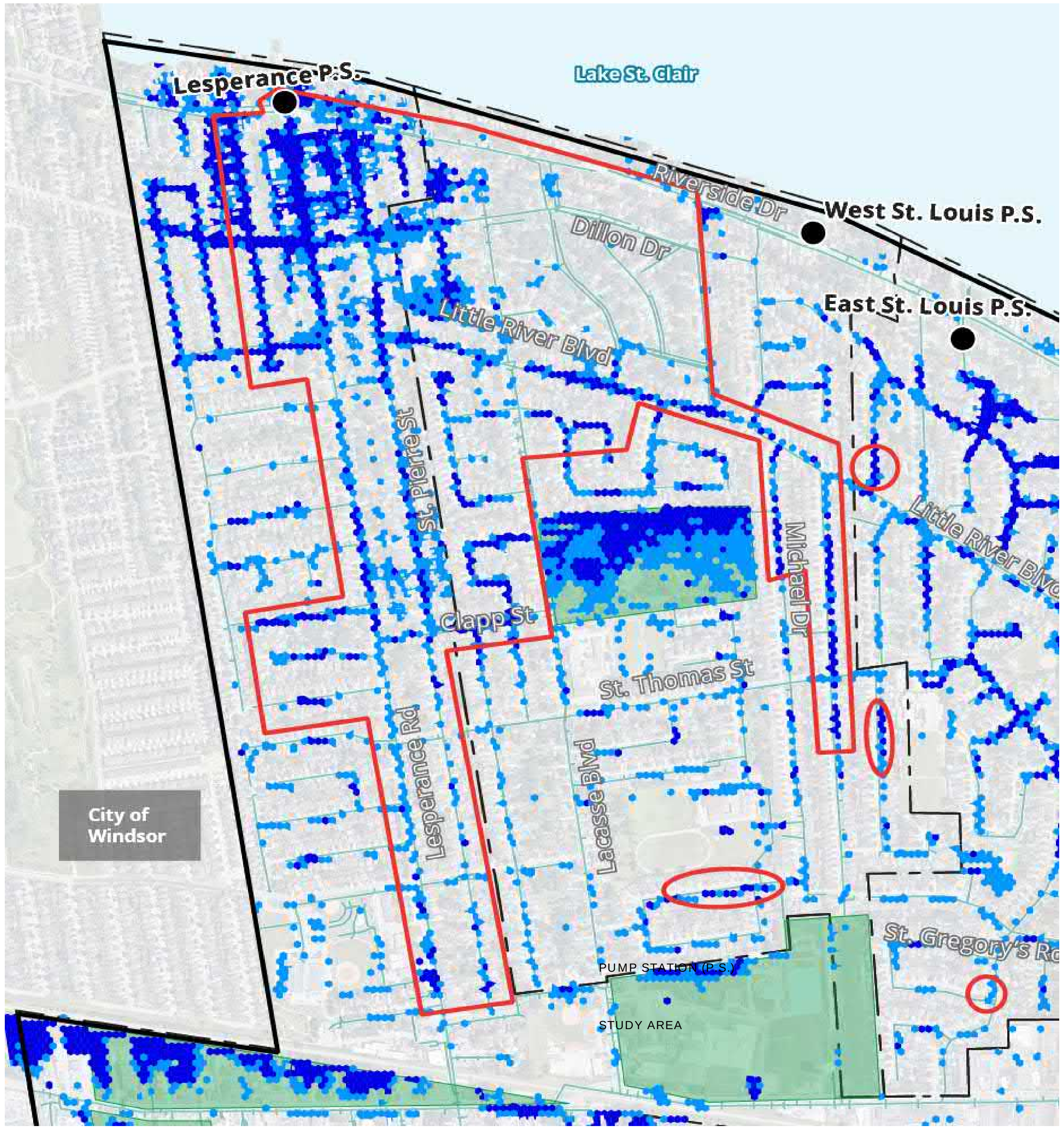
MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

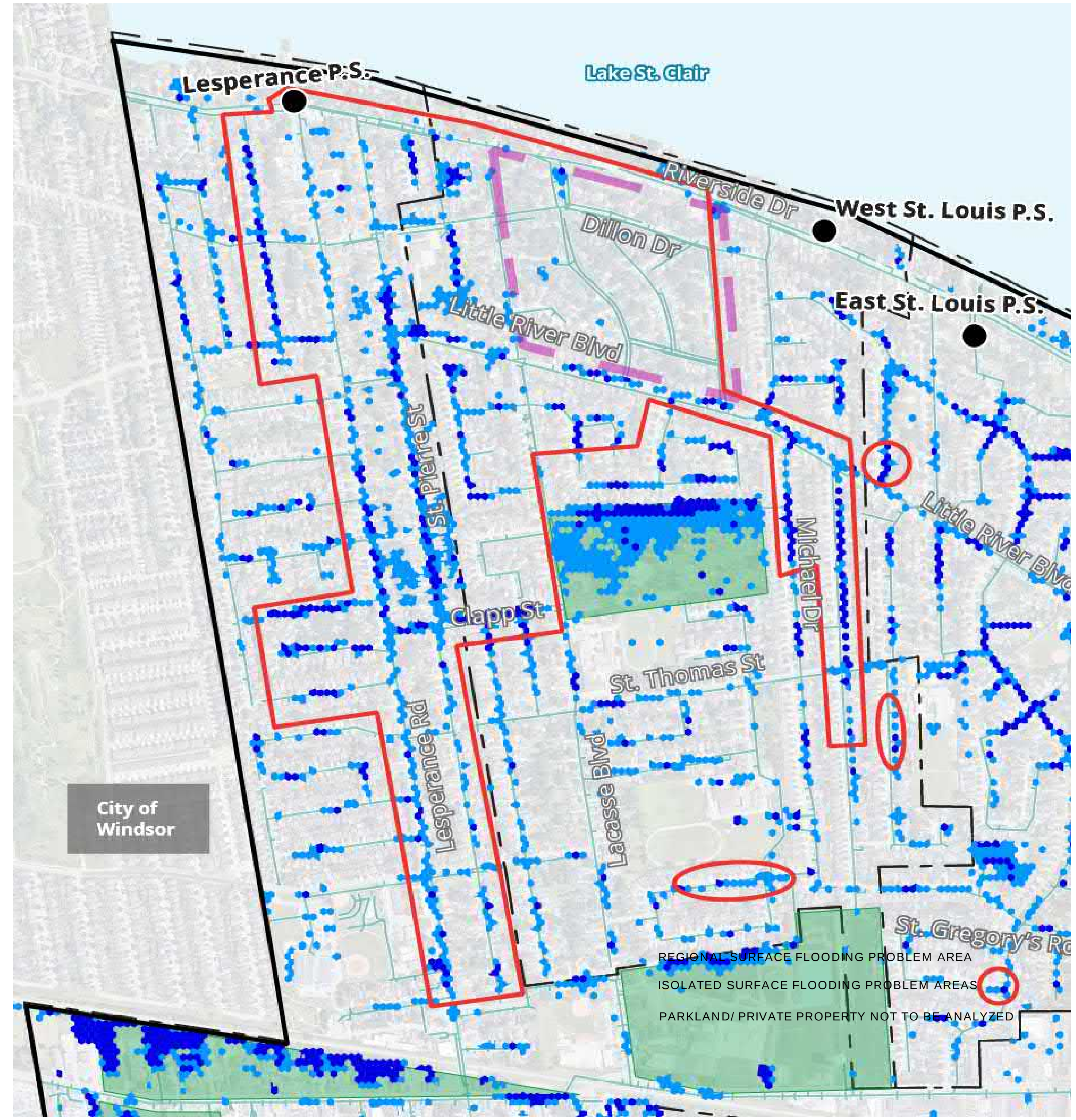
2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

EXISTING CONDITION (1:100 YEAR + 40% SURFACE PONDING SIMULATION)



FUTURE CONDITION (1:100 YEAR + 40% SURFACE PONDING SIMULATION)



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

1:100 YEAR + 40% EXISTING VS FUTURE CONDITION
SURFACE FLOODING - PROBLEM AREA W-1
FIGURE 11.16

Town of Tecumseh Ontario · Canada

PUMP STATION (P.S.)
PUMP STATION SERVICE AREA
STUDY AREA
SURFACE PONDING WITHIN AREA MAINTAINED BELOW 0.30m
REGIONAL SURFACE FLOODING PROBLEM AREAS
ISOLATED SURFACE FLOODING PROBLEM AREAS
PARKLAND / PRIVATE PROPERTY NOT TO BE ANALYZED
SURFACE PONDING BETWEEN 0.15m - 0.30m DEPTH
SURFACE PONDING > 0.30m DEPTH

DILLON CONSULTING

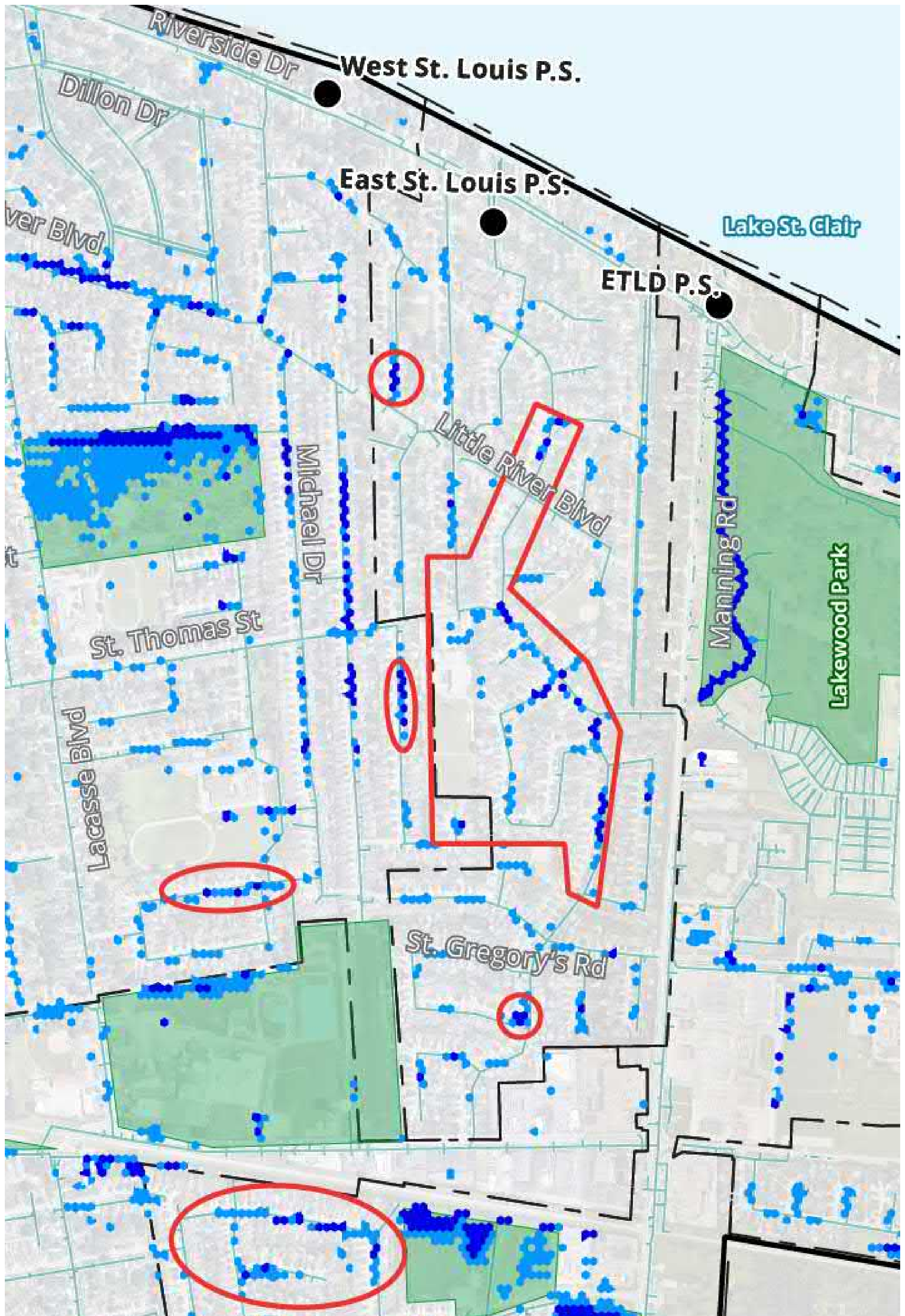
MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

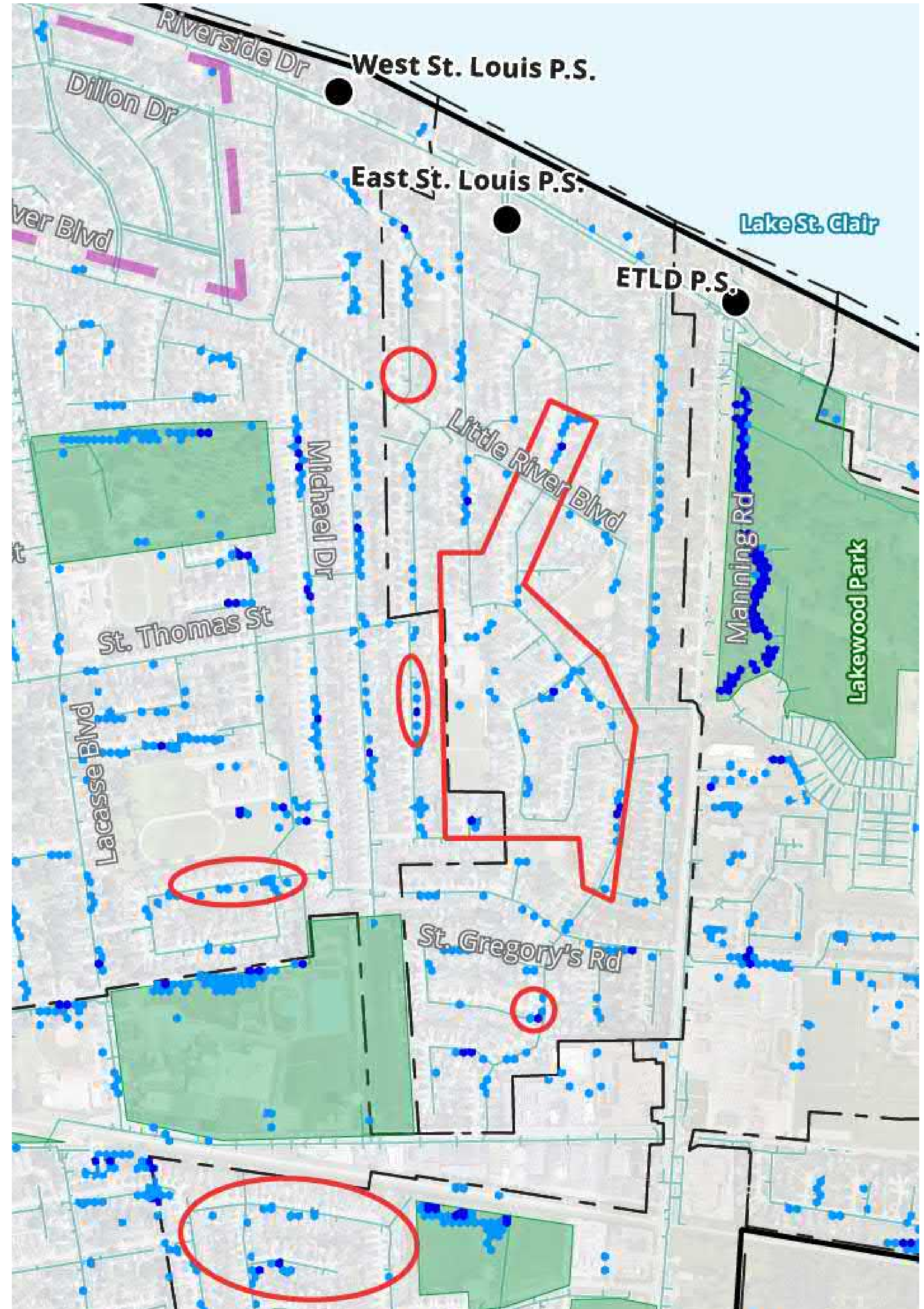
2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

EXISTING CONDITION
(1:100 YEAR SURFACE PONDING SIMULATION)



FUTURE CONDITION
(1:100 YEAR SURFACE PONDING SIMULATION)



TOWN OF Tecumseh
ONTARIO · CANADA

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

1:100 YEAR EXISTING VS FUTURE CONDITION SURFACE FLOODING - PROBLEM AREA W-2
FIGURE 11.17

- PUMP STATION (P.S.)
- PUMP STATION SERVICE AREA
- STUDY AREA
- SURFACE PONDING WITHIN AREA MAINTAINED BELOW 0.30m
- REGIONAL SURFACE FLOODING PROBLEM AREAS
- ISOLATED SURFACE FLOODING PROBLEM AREAS
- PARKLAND / PRIVATE PROPERTY NOT TO BE ANALYZED
- SURFACE PONDING BETWEEN 0.15m - 0.30m DEPTH
- SURFACE PONDING > 0.30m DEPTH

SCALE 1:NTS 2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

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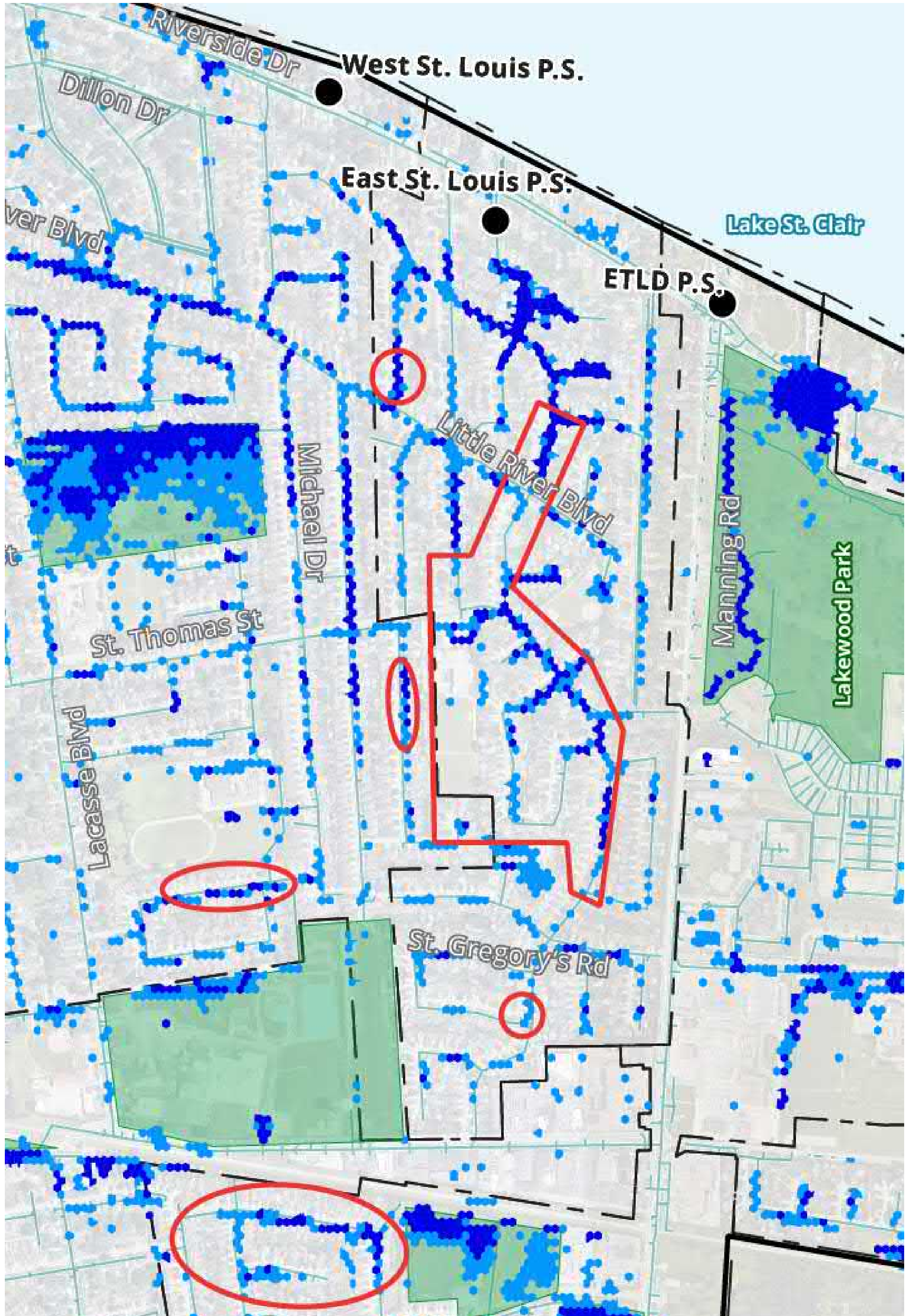
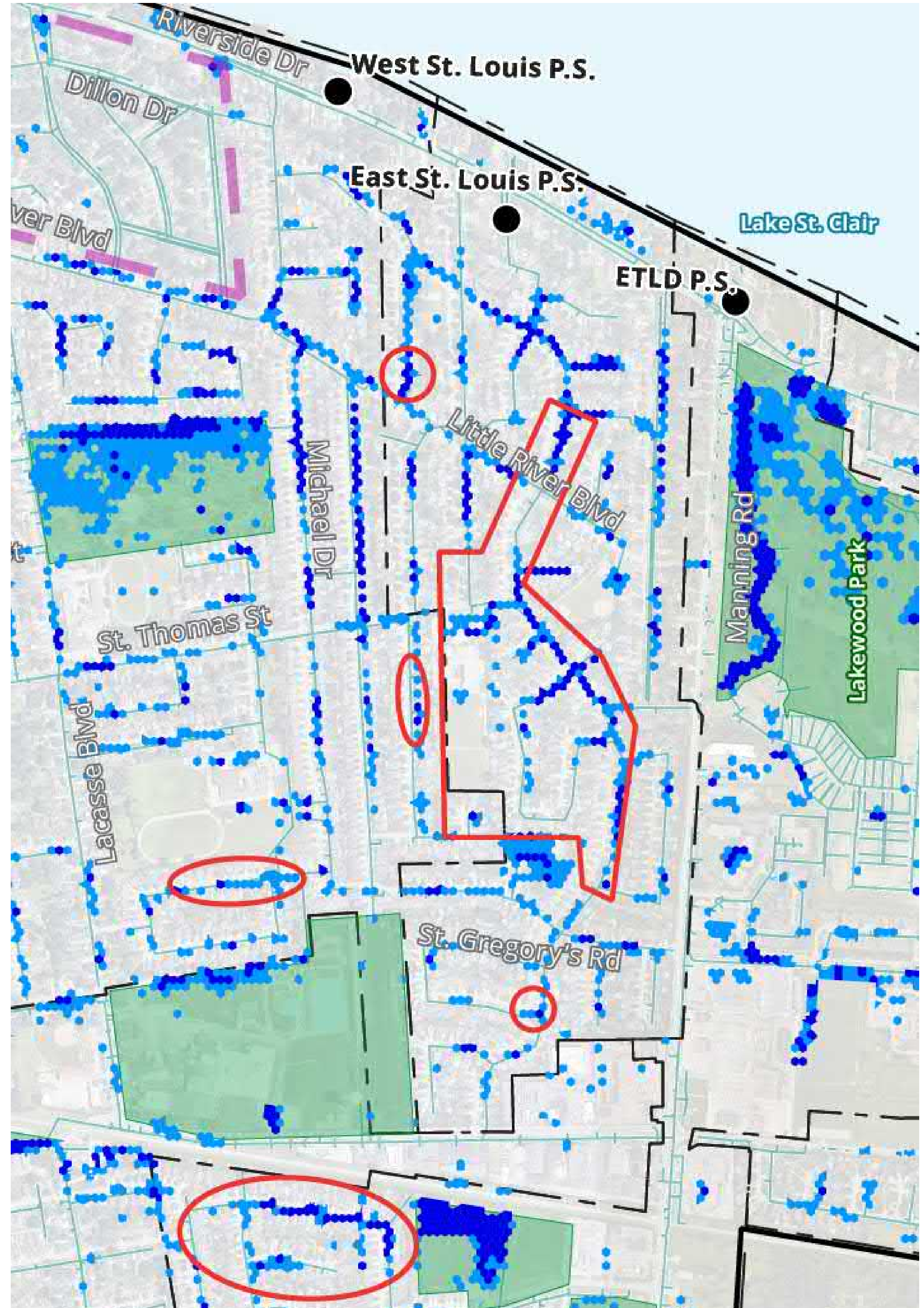
PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019

EXISTING CONDITION (1:100 YEAR + 40% SURFACE PONDING SIMULATION)



FUTURE CONDITION (1:100 YEAR + 40% SURFACE PONDING SIMULATION)



TOWN OF Tecumseh ONTARIO · CANADA

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

1:100 YEAR + 40% EXISTING VS FUTURE CONDITION SURFACE FLOODING -PROBLEM AREA W-2
FIGURE 11.18

- PUMP STATION (P.S.)
- PUMP STATION SERVICE AREA
- STUDY AREA
- SURFACE PONDING WITHIN AREA MAINTAINED BELOW 0.30m
- REGIONAL SURFACE FLOODING PROBLEM AREAS
- ISOLATED SURFACE FLOODING PROBLEM AREAS
- PARKLAND / PRIVATE PROPERTY NOT TO BE ANALYZED
- SURFACE PONDING BETWEEN 0.15m - 0.30m DEPTH
- SURFACE PONDING > 0.30m DEPTH

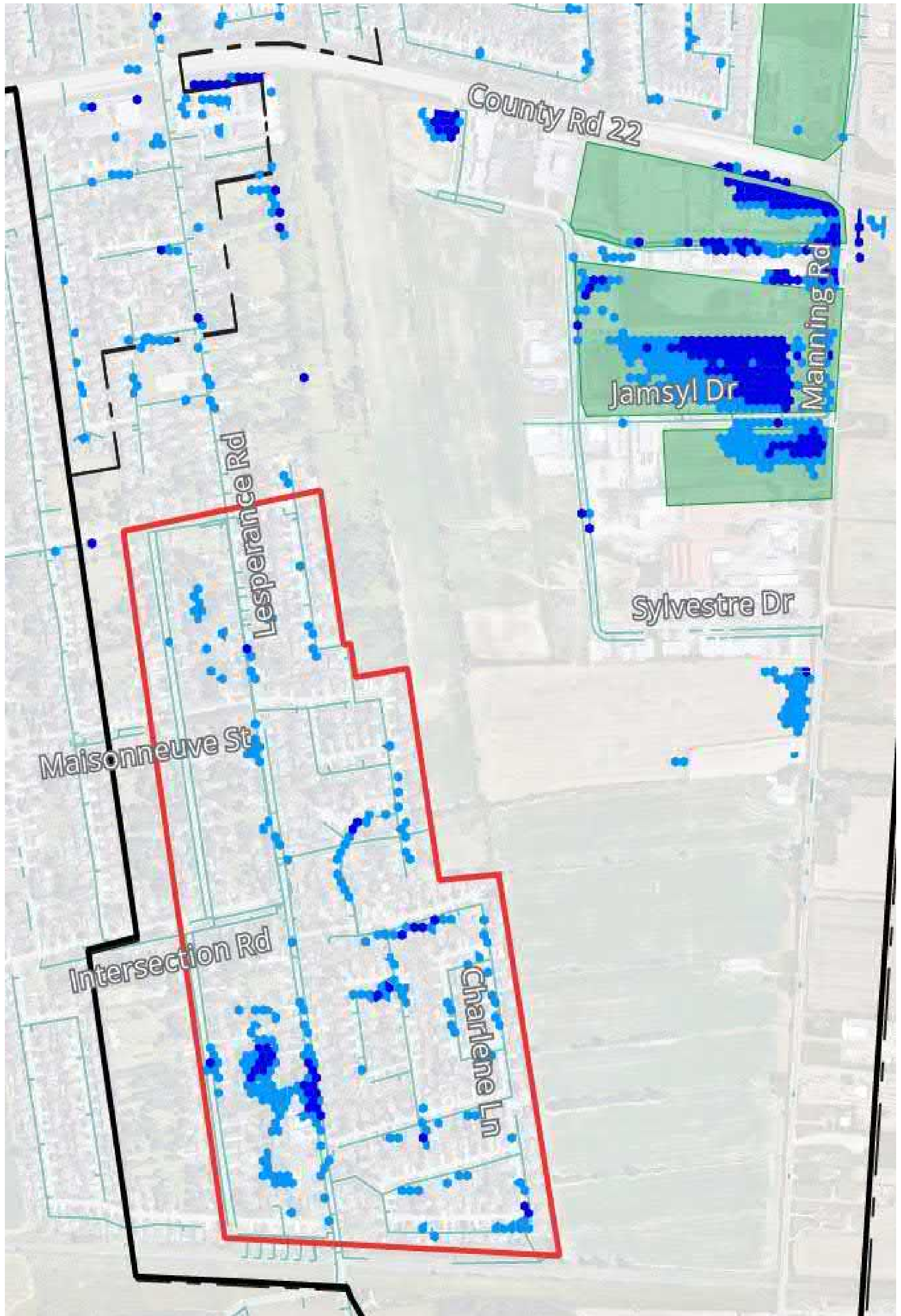
SCALE 1:NTS 2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

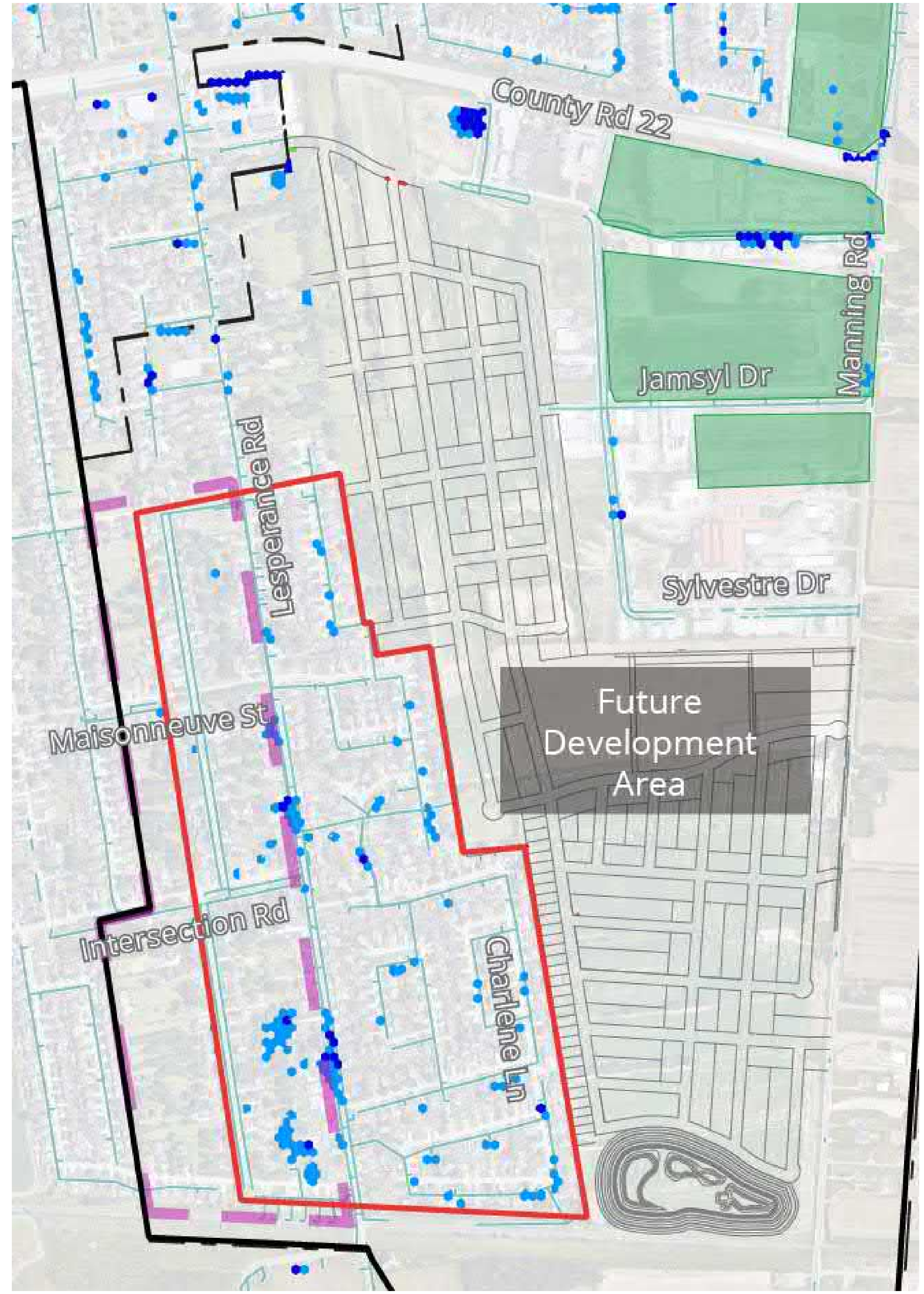
DILLON CONSULTING

PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019

EXISTING CONDITION
(1:100 YEAR SURFACE PONDING SIMULATION)



FUTURE CONDITION
(1:100 YEAR SURFACE PONDING SIMULATION)



TOWN OF Tecumseh
ONTARIO · CANADA

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

1:100 YEAR EXISTING VS FUTURE CONDITION SURFACE FLOODING - PROBLEM AREA W-3
FIGURE 11.19

- PUMP STATION SERVICE AREA
- STUDY AREA
- SURFACE PONDING WITHIN AREA MAINTAINED BELOW 0.30m
- REGIONAL SURFACE FLOODING PROBLEM AREAS
- ISOLATED SURFACE FLOODING PROBLEM AREAS
- PARKLAND / PRIVATE PROPERTY NOT TO BE ANALYZED
- SURFACE PONDING BETWEEN 0.15m - 0.30m DEPTH
- SURFACE PONDING > 0.30m DEPTH

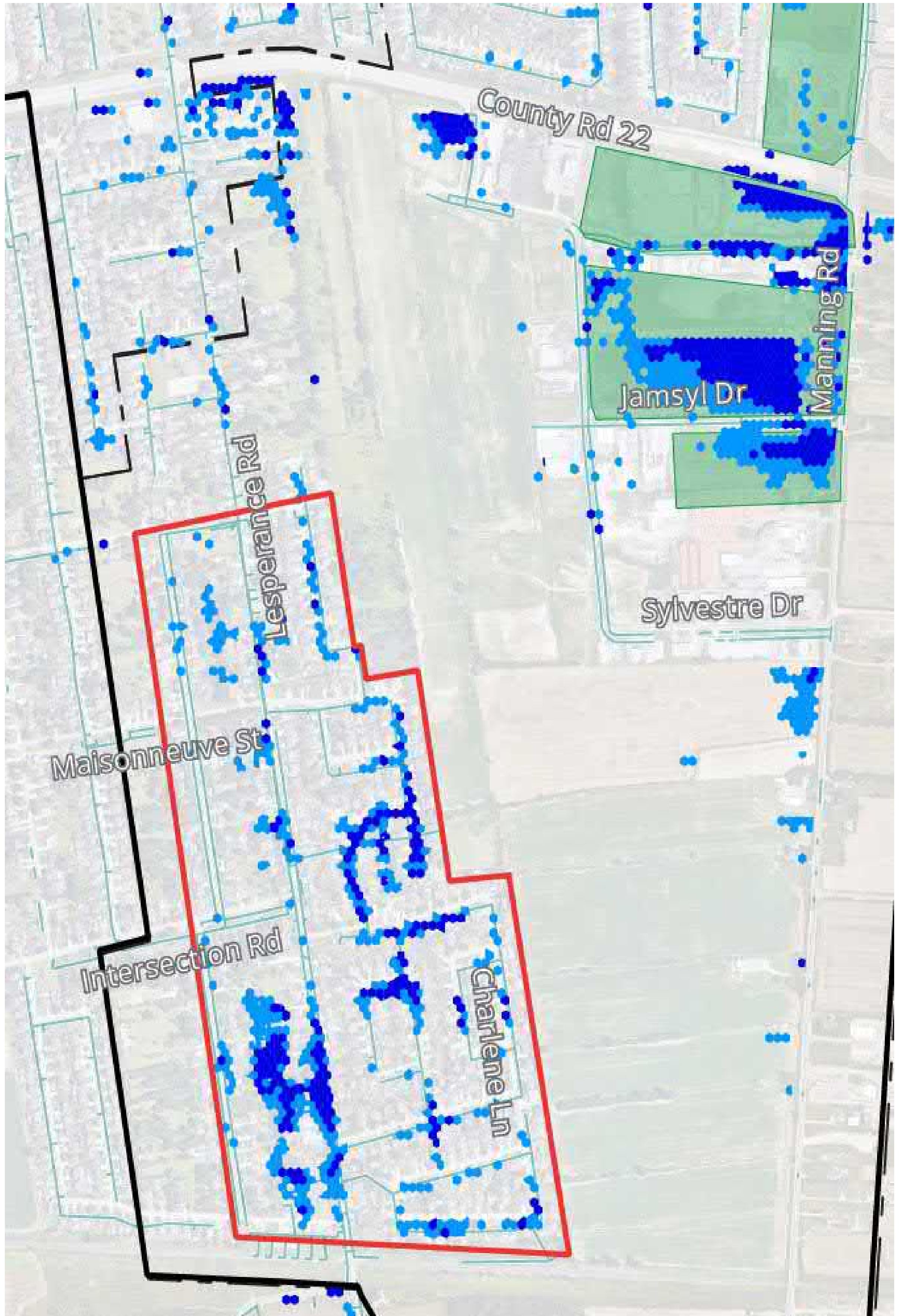
SCALE 1:NTS 2

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

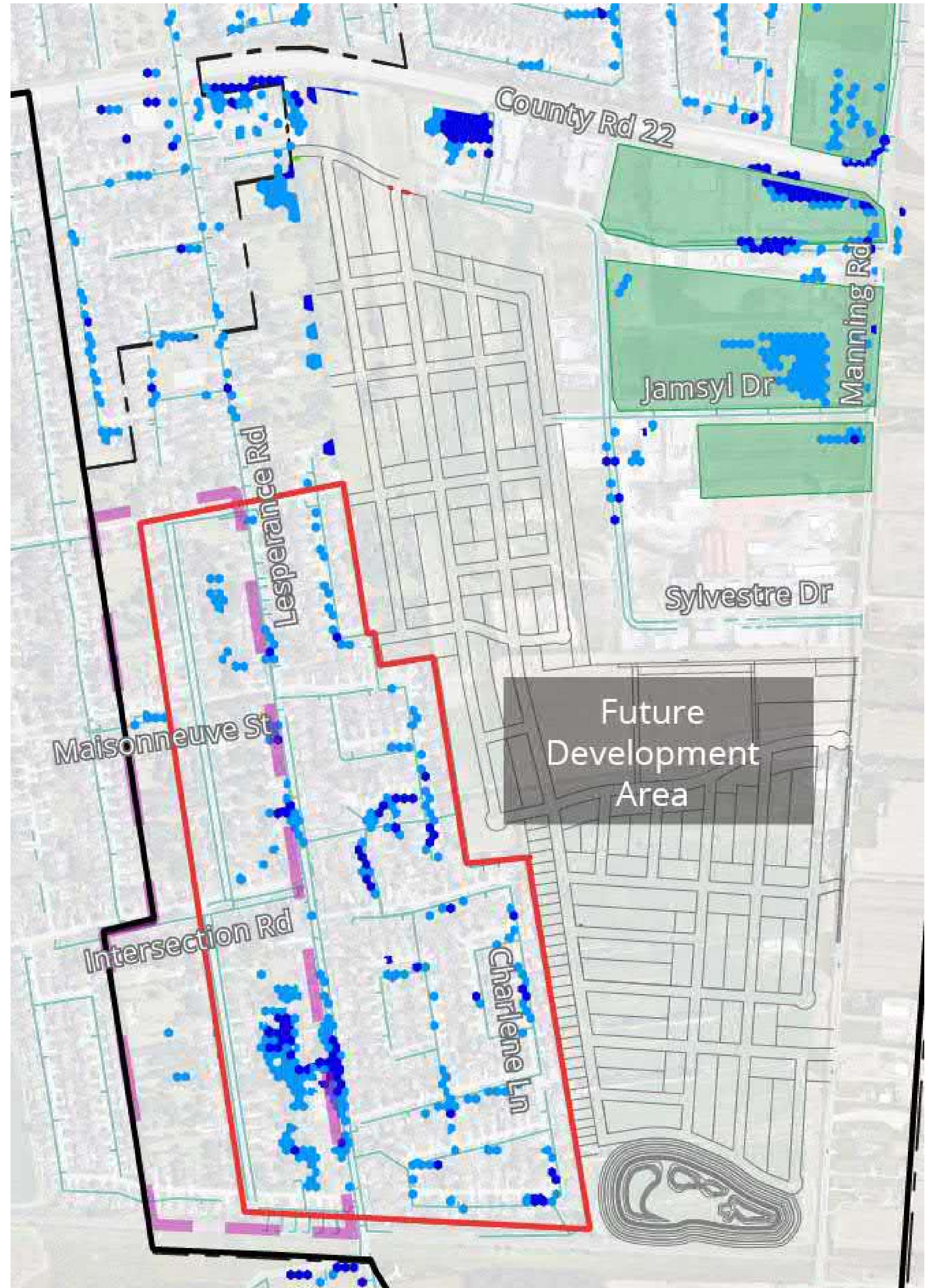
DILLON CONSULTING

PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019

EXISTING CONDITION
(1:100 YEAR + 40% SURFACE PONDING SIMULATION)



FUTURE CONDITION
(1:100 YEAR + 40% SURFACE PONDING SIMULATION)



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

1:100 YEAR + 40% EXISTING VS FUTURE CONDITION SURFACE FLOODING - PROBLEM AREA W-3

FIGURE 11.20

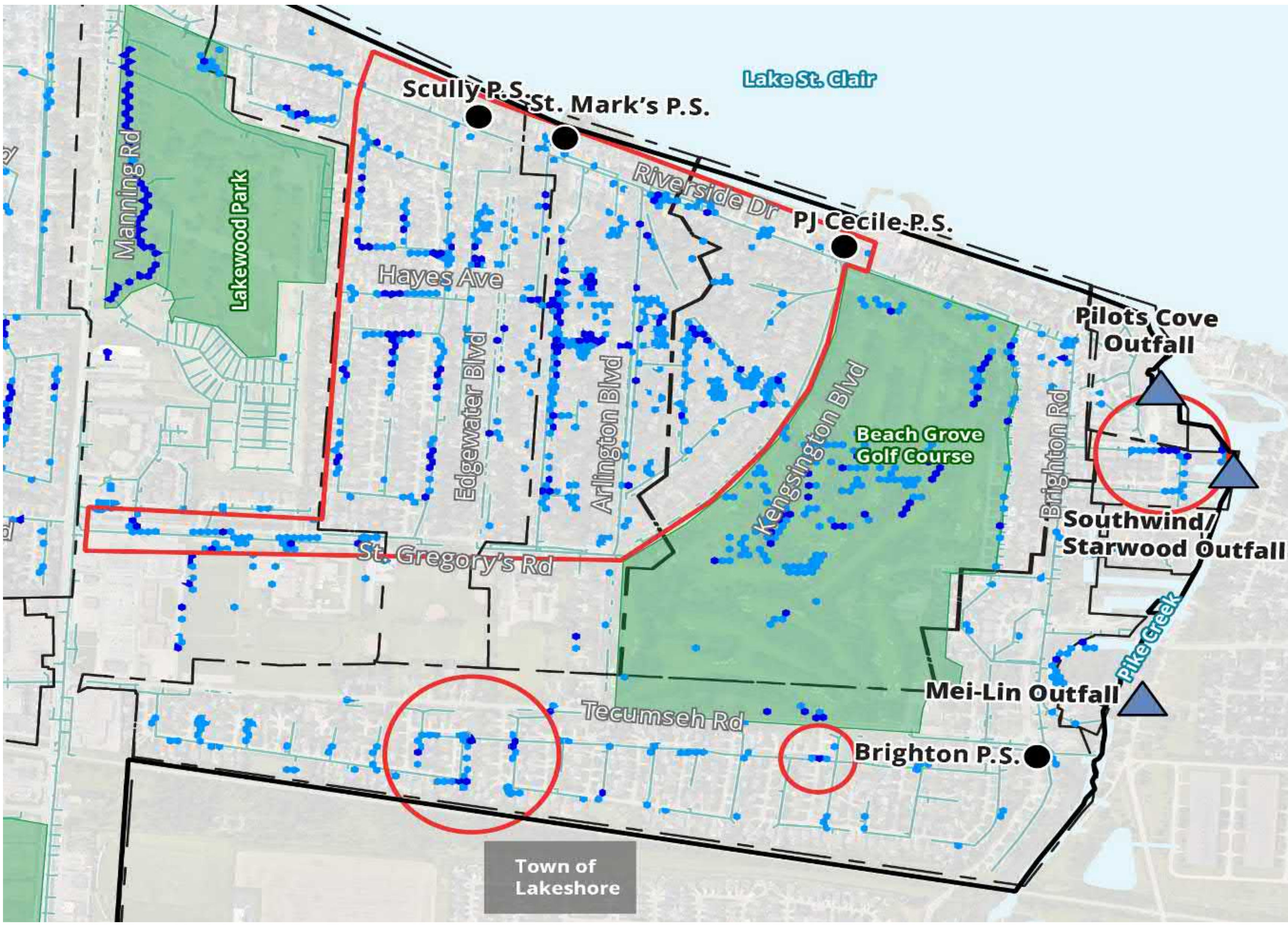
- PUMP STATION SERVICE AREA
- STUDY AREA
- SURFACE PONDING WITHIN AREA MAINTAINED BELOW 0.30m
- REGIONAL SURFACE FLOODING PROBLEM AREAS
- ISOLATED SURFACE FLOODING PROBLEM AREAS
- PARKLAND / PRIVATE PROPERTY NOT TO BE ANALYZED
- SURFACE PONDING BETWEEN 0.15m - 0.30m DEPTH
- SURFACE PONDING > 0.30m DEPTH

SCALE 1:NTS 2

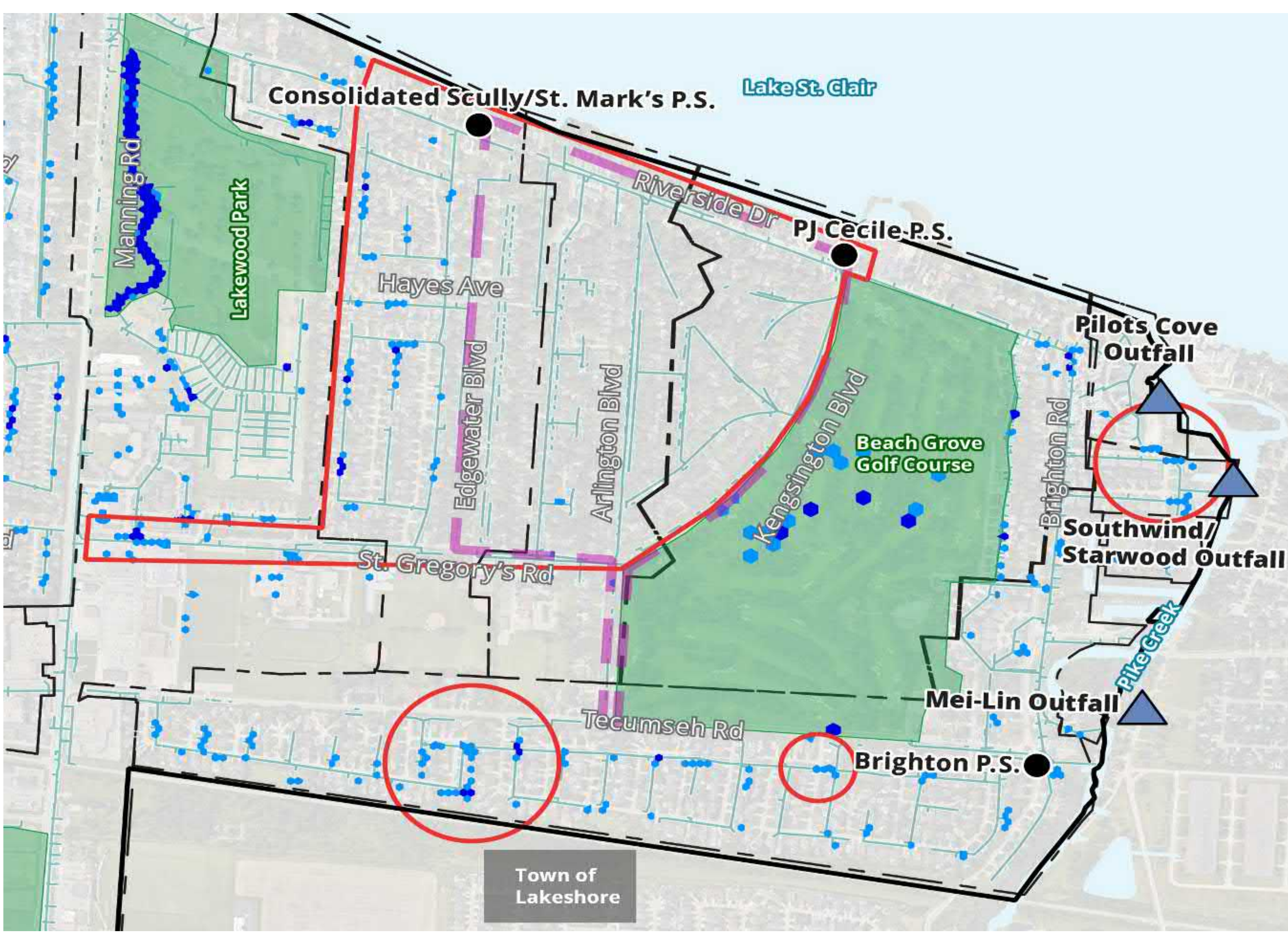
MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

PROJECT: 16-4880
STATUS: FINAL
DATE: JUNE 2019

EXISTING CONDITION (1:100 YEAR SURFACE PONDING SIMULATION)



FUTURE CONDITION (1:100 YEAR SURFACE PONDING SIMULATION)



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

1:100 YEAR EXISTING VS FUTURE CONDITION SURFACE FLOODING - PROBLEM AREA E-1
FIGURE 11.21

SEWER GRAVITY OUTFALL
PUMP STATION (P.S.)
PUMP STATION SERVICE AREA
STUDY AREA
SURFACE PONDING WITHIN AREA MAINTAINED BELOW 0.30m
REGIONAL SURFACE FLOODING PROBLEM AREAS
ISOLATED SURFACE FLOODING PROBLEM AREAS
PARKLAND / PRIVATE PROPERTY NOT TO BE ANALYZED
SURFACE PONDING BETWEEN 0.15m - 0.30m DEPTH
SURFACE PONDING > 0.30m DEPTH

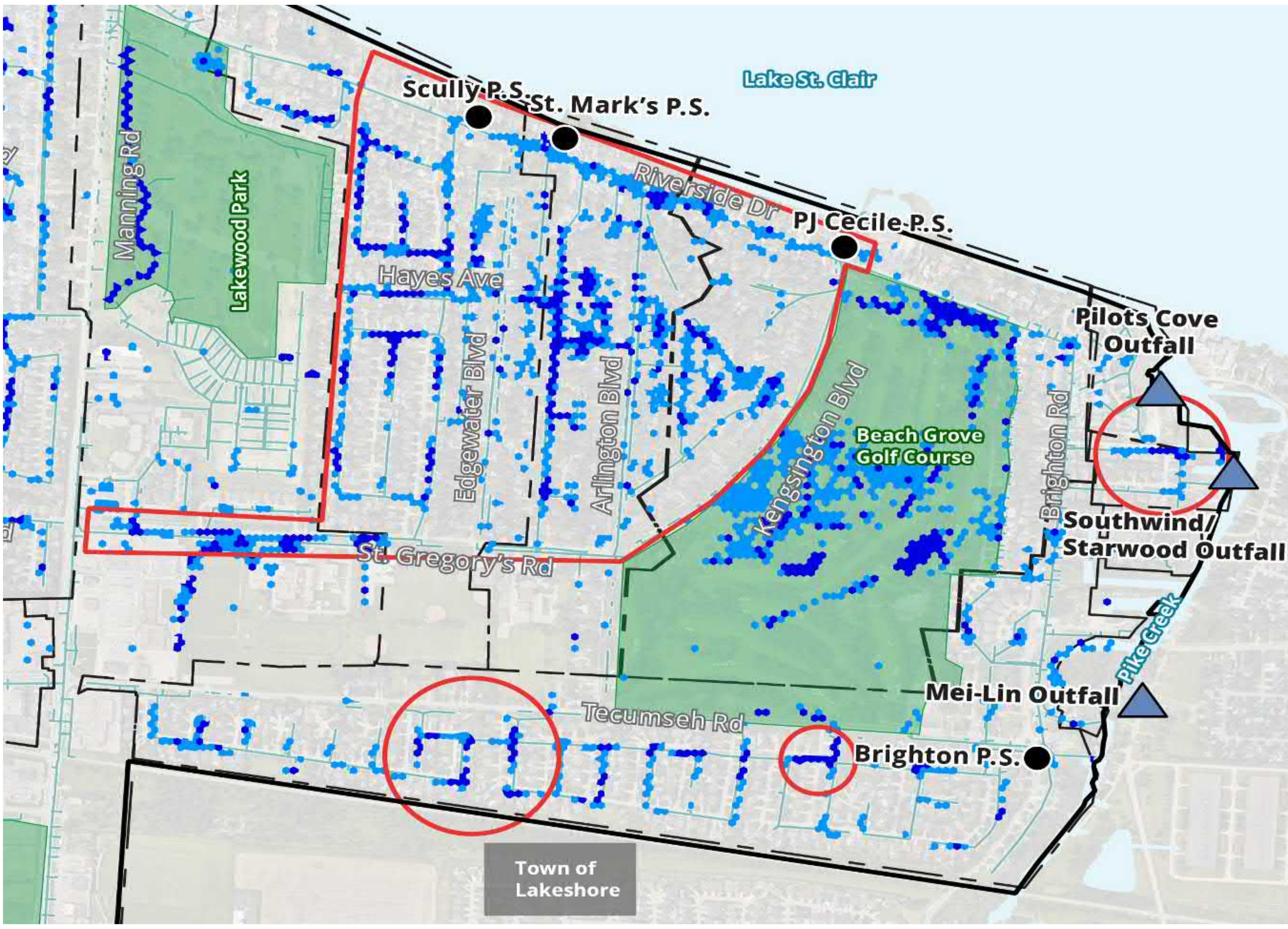
MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

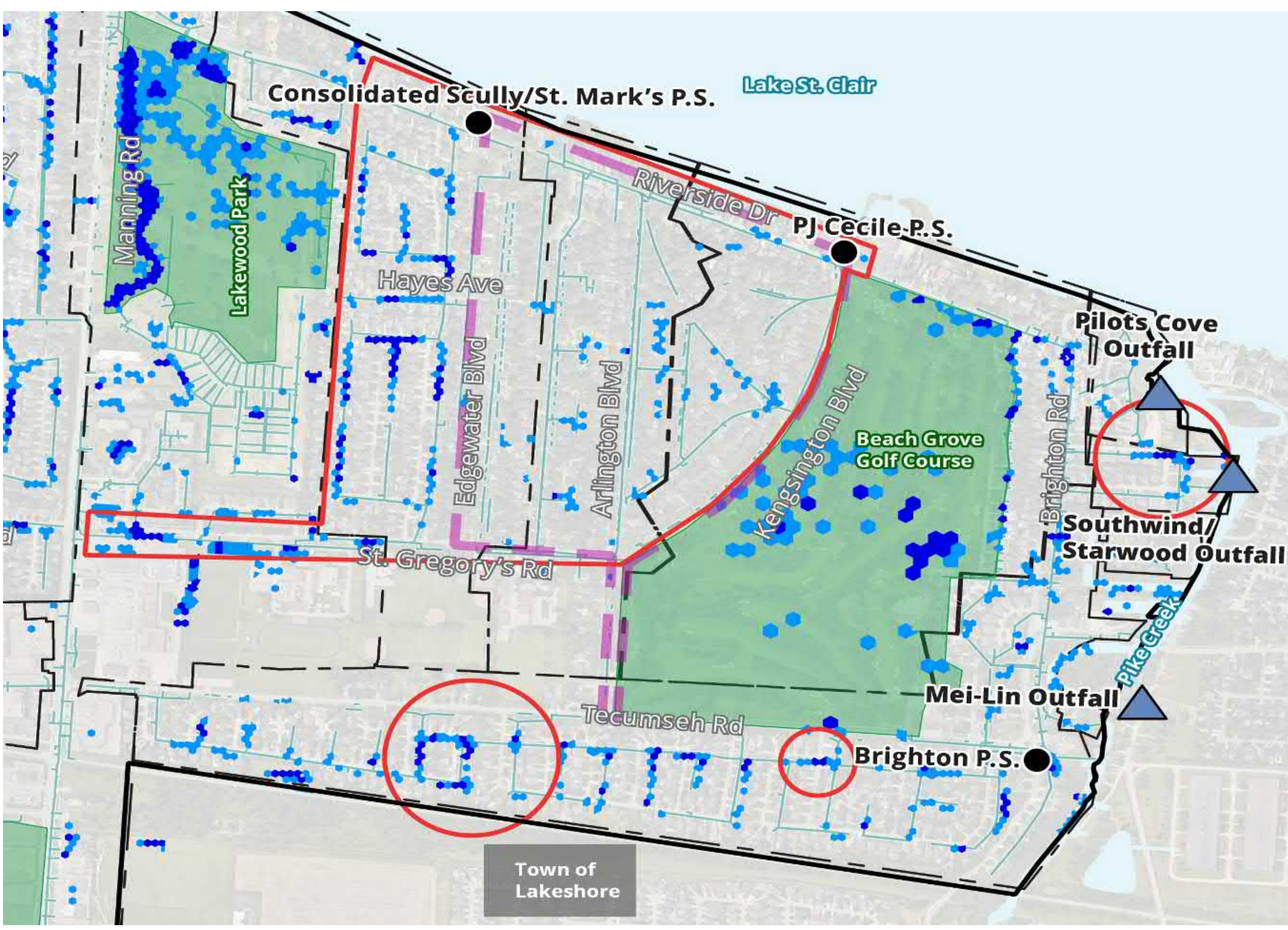
2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

EXISTING CONDITION (1:100 YEAR + 40% SURFACE PONDING SIMULATION)



FUTURE CONDITION (1:100 YEAR + 40% SURFACE PONDING SIMULATION)



TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

1:100 YEAR +40% EXISTING VS FUTURE CONDITION SURFACE FLOODING - PROBLEM AREA E-1
FIGURE 11.22

▲	SEWER GRAVITY OUTFALL		REGIONAL SURFACE FLOODING PROBLEM AREAS
●	PUMP STATION (P.S.)		ISOLATED SURFACE FLOODING PROBLEM AREAS
	PUMP STATION SERVICE AREA		PARKLAND / PRIVATE PROPERTY NOT TO BE ANALYZED
	STUDY AREA		SURFACE PONDING BETWEEN 0.15m - 0.30m DEPTH
	SURFACE PONDING WITHIN AREA MAINTAINED BELOW 0.30m		SURFACE PONDING > 0.30m DEPTH

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

12.0 Evaluation of Low Impact Development Techniques

12.1 Overview

As outlined in Section 2.4, the Ontario Ministry of Environment, Conservation and Parks (MECP) produced a DRAFT Low Impact Development (LID) Stormwater Management Guidance Manual in 2017 to provide future guidance on the design, construction and maintenance of LIDs to advocate a “treatment train” approach to treat and store stormwater runoff. This approach encouraged the use of lot-level and conveyance controls along with end-of-pipe measures to manage stormwater runoff.

To assess a case study area, the Coronado Dish was chosen within the overall study limits due to the Town proposing to reconstruct the municipal roadway and storm infrastructure from a semi-urban cross section to a more traditional storm sewer design and urbanized roadway cross section.

12.2 Evaluation of LID Controls within the Study Area

As identified above, LID controls were evaluated within the Coronado Dish area. The Coronado Dish was chosen as the representative area to test the effectiveness of LIDs as it is one of the areas where road reconstruction is recommended to upgrade roadway cross-sections from rural or semi-urban to an urban cross-section. Since the entire area drains through one storm sewer at the downstream end, it makes it easier to compare results from non-LID and LID scenarios.

LID stormwater control measures which can be implemented within municipal road rights-of-way are classified as “conveyance control measures”. These systems treat stormwater as it travels overland or through pipes on route to the downstream outlet. Traditional conveyance systems comprise curbs, gutters and buried concrete (or other) piping systems that carry stormwater away from a development area to a water body generally along the road network. In appropriate applications, LID conveyance control measures such as vegetated filter strips, enhanced swales or bio filtration units can be used to improve water quality conditions at a lower cost to the Town while still providing conveyance of the minor system.

Because residential streets account for a significant share of a community’s impervious surfaces, conveyance control measures present an important opportunity to improve downstream water quality conditions (e.g. sediment, nutrient, bacteria, oil/grit, thermal impact reduction, etc.), minimize watercourse erosion and reduce peak flows associated with urban flooding. Conveyance control measures can most feasibly be incorporated into existing right-of-way’s as part of planned road reconstruction works as storm sewers and inlets can be replaced and reconfigured during this process.

The LID conveyance control measure chosen for this exercise to integrate with the existing municipal road right-of-way are exfiltration systems due to their minimal maintenance requirements, low incremental cost and conventional streetscape aesthetic. The storage within the exfiltration trenches is expected to replace the storage in the existing roadside ditches, which convey runoff in the existing conditions rural roadway cross-section. The exfiltration system design evaluated as part of this exercise involved installing parallel perforated pipes in addition to the conventional storm sewer. Installing a perforated pipe that runs parallel to the conventional storm sewer will ensure that the majority of storm events are conveyed directly to the exfiltration trench, acting as additional on-site storage, but that a conventional storm sewer is still available when the infiltration and storage capacity of the exfiltration system is overwhelmed.

For this example system, each proposed catch basin drains to the exfiltration trench system through two catch basin lead pipes. A catch basin lead pipe at a lower elevation drains directly to the perforated pipes, allowing runoff to filter out into the trench and store within the voids of the granular fill material of the trench until it is infiltrated into the soil surrounding the trench or conveyed back to the storm sewers through the underdrain. During larger storm events, the second catch basin lead pipe, which is at a higher elevation, is utilised. This pipe drains directly to the storm sewer and stormwater runoff is conveyed downstream, similar to a conventional storm sewer conveyance system. For a scenario where infiltration capacity of the soils is limited (ie. compacted clays), an underdrain located at the bottom of the trench collects the runoff from the trench and conveys it to the downstream manhole. Figure 12.1 provides a typical cross section of the proposed exfiltration trench system.

The soils in the study area consist mostly of poorly drained hydrologic “Type D” soil, as mentioned in Section 3.3. As such, the exfiltration trench system is not expected to provide significant infiltration into the surrounding soil and is expected to provide additional storage during storm events, and help in attenuating the peak flows in the storm sewer. They would also provide quality benefits to stormwater that is conveyed to downstream infrastructure due to the initial runoff being conveyed to the trench directly, where suspended solids are trapped, before being conveyed downstream.

At this time, there are still many questions relating to the use of exfiltration trenches within the municipal right-of-way, including:

- Effectiveness of the system in future years;
- Cleanup of potential contamination of the trench during spills; and
- Feasibility of constructing exfiltration trenches in the area and determine the potential of increased infiltration or interconnections into the stone surrounding the sanitary sewers and laterals leading back to the foundations on private lands.

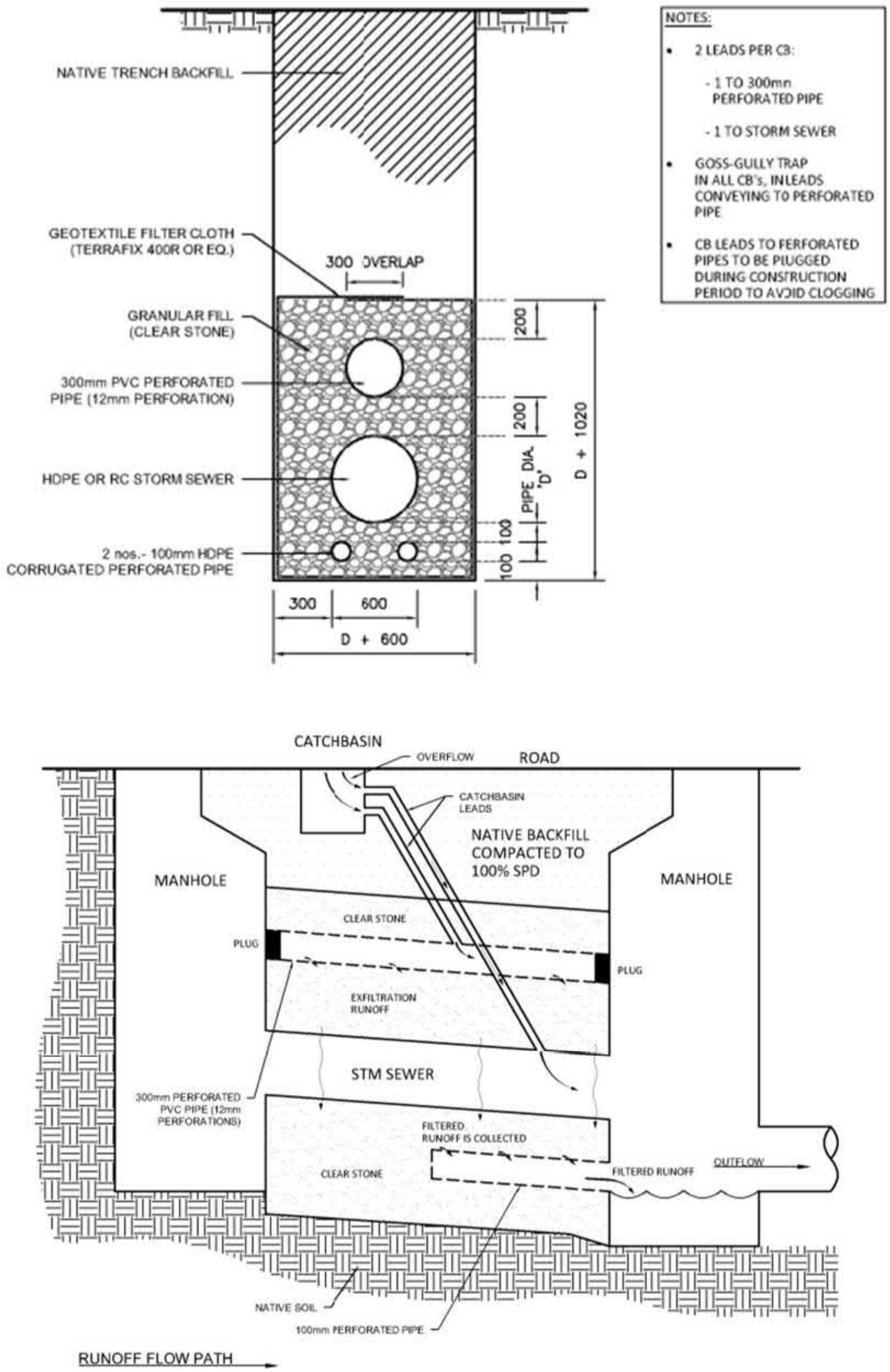


Figure 12.1: Typical section of an exfiltration trench system

12.2.1 Coronado Dish Area LID Modelling Approach

Under future development conditions, the Coronado Dish area is proposed to be upgraded from a semi-urban roadway cross-section to an urban cross-section with curbs and gutter and stormwater conveyance through catch basins and storm sewers. LID controls in the form of exfiltration trench systems were evaluated as an alternative solution.

The exfiltration trench systems were modelled as an additional conduit in the PCSWMM model. As discussed above, a lower lead pipe from the catch basins connect to the exfiltration trench conduit directly while another lead pipe, at a higher elevation, connects to the storm sewer. The conduit representing the exfiltration trench was modelled with a rectangular cross-section. The height of the rectangle represented the actual height of the trench, while the width of the rectangle in the model represented 40% of the actual width of the trench. This was done to account for storage in the trench in only the voids of the granular fill material. At this time, 40% void ratio was assumed in the granular material. The exfiltration trench was modelled to assume no infiltration in the soils, due to the lack of porosity testing of the soils in the area to accurately replicate infiltration conditions.

The conduit representing the exfiltration trench was connected to the downstream manhole through an orifice, which represented the underdrain through which the trench will be drained.

12.2.1.1 Model Results

The PCSWMM model with the LID controls incorporated was simulated for the 1:2 year, 1:5 year and 1:100 year design storm events. The peak flows at the downstream end of the Coronado Dish storm sewer system (storm sewer along Barry Avenue, south of Riverside Drive), were compared to the model results from the corresponding future development scenarios without the LID controls. The comparison in peak flows in the Barry Avenue storm sewer is presented below in Table 12.1.

Table 12.1: Model results comparing peak flows with/without LID controls

	Peak Flow (m^3/s)			Max. Storage in Trenches (m^3)
	Without LID controls	With LID controls	% change in peak flows due to LID controls	
1:2 year event	1.57	0.94	40.1% (reduction)	692.7
1:5 year event	1.94	1.23	36.7% (reduction)	955.9
1:100 year event	2.04	1.77	13.5% (reduction)	1,675.5

The storm sewer along Barry Avenue drains a total area of approximately 75.5 Ha, which includes the Coronado Dish area, areas south of Little River Boulevard, along Kimberly Drive, Jelso Place and Shawn Avenue, areas along Lacasse Boulevard and some areas along Riverside Drive east of Pinewood Crescent. The Coronado Dish area, where LID controls were modelled, accounts for 17.5 Ha of this area. This amounts to approximately 23% of the total area draining into the Barry Avenue storm sewer. The reduction in peak flow represented in Table 12.1, is due to the reduction in flows in 23% of the total catchment area only. No LID controls were modelled in the other areas draining into the Barry Avenue storm sewer.

As seen in the results, there is a 40% reduction in peak flows during the 1:2 year event, due to the incorporation of additional at source controls and storage within the exfiltration trenches in only 23% of the upstream areas draining into the Barry Avenue storm sewer. The proposed solution along Little River Boulevard, upstream of the Coronado Dish area, is designed as underground storage in the storm sewers with an overflow into the Barry Avenue storm sewer that is utilised during high flow events. During the 1:2 year and 1:5 year event simulations, this overflow is not utilised as the HGL in the Little River Boulevard storm sewer remains low and flows are routed to the storm sewer along Michael Drive. During these smaller design storm events, most of the flow in the Barry Avenue storm sewer consists of stormwater runoff from the Coronado Dish area.

During the 1:100 year event, notable improvements through the major system is observed. No roadway ponding was observed at the catch basin locations during the 1:100 year event in the scenario where LIDs were modelled. This is due to the additional lot level storage being provided within the exfiltration trenches.

12.3 LID Assessment Findings

Findings of the case study identify reductions in peak flow within the downstream sewer from the Coronado Dish area. During all storm events, a reduction in the HGLs in the storm sewers and reduction in roadway ponding was also observed due to the incorporation of the at-source storage. As identified previously, no infiltration was taken into consideration as part of this assessment.

Although the incorporation of LID controls show a reduction in HGL and peakflow at the downstream end of the system during all storm events, the use of LID techniques within a municipal system are not to be a basis for reducing the storm sewer design level of service. Any LID included in a municipal roadway storm design shall be for added resiliency in the system and be in addition to the traditional storm sewers.

At this time, there are concerns that the benefits of a design that promotes infiltration into the underlying soils do not outweigh the potential for increases in I&I into the sanitary system. Many residential areas within the Town of Tecumseh have foundation drains directly connected to the sanitary system, which already contribute a substantial amount of direct inflow to the sanitary sewers. This includes all areas currently with semi-urban roadway cross sections. The addition of an exfiltration trench design within an existing built up area in a new storm sewer system could further exacerbate problems of I&I and ultimately basement flooding.

The use of LID techniques is recommended to be further reviewed during detail design. This includes the completion of a detailed geotechnical investigation to determine the local soil type, porosity rates in the underlying soils and existing water table information to determine the feasibility of the use of LID techniques. Remedial measures to eliminate increased I&I into the sanitary system due to the installation of exfiltration trenches is also recommended to be reviewed in the future.

13.0 Pump Station Improvements

13.1 Lesperance Storm Pump Station Improvements

As identified in Section 5.3.5, there are currently three different types of pumps at the existing Lesperance pump station. The oldest pump is a vertical turbine pump with a horizontal electrical motor and a gearbox installed in 1957. The pump station was first expanded in 1986, when a single screw pump station was constructed beside the existing facility. The second expansion was completed in 2002 when a duplex submersible turbine pump station was constructed. The condition assessment completed in 2016 identified several improvements required to the screw pump station and also the recommendation to replace the 1957 vertical pumps which is nearing its end of life.

Based on the age of the pump station infrastructure at this location and through the modelling analysis, it is recommended that the 1957 vertical pump station and the screw pump station be demolished and the 2002 expansion pump station be kept in service. It is recommended to construct a new pump station expansion equipped with vertical submersible axial flow pumps similar to the 2002 pump station. The expansion of the pump station would be located east of the existing screw pump station. There would be two duty and one stand-by pump in the expansion pump station. The diesel generator would serve the existing pump station during the construction but will have to be relocated.

The outfall from the existing and expansion pump station will merge to a single outfall pipe connected to the existing outfall structure. The outfall structure will have to be modified within its existing footprint to accommodate a larger size outfall pipe and to convey the increased flow.

Capacity Summary

Required firm capacity	9.0 m^3/s
Existing 2002 PS capacity	2 x 1.4 m^3/s
Additional capacity required	6.2 m^3/s
New PS configuration	2 duty, 1 standby
New PS pump capacity	3.1 m^3/s each
New pump motors	350 kW (each)

Functional design drawings for the pump station improvement is provided in Appendix F.

13.2 West St. Louis Storm Pump Station Improvements

As identified in Section 5.3.5, there are currently two screw pumps installed at the pump station. No upgrades have been completed since the station was put into operation in 1991 and is currently in good condition. The existing pump station design identified an expansion to the station to the east and through a review of the as-built drawings, the outfall was originally constructed to accommodate and expansion.

It is recommended to leave the existing pump station in service and to construct an expansion to the pump station east of the existing structure. The expansion would utilize vertical submersible axial flow pumps consisting of one duty and one stand-by pump in the expansion structure. An interconnection to the new pump station would be constructed from the existing inlet chamber. The outfall would be connected to the existing outfall structure by new outfall pipes.

Capacity Summary

Required firm capacity	7.0 m^3/s
Existing PS capacity	2 x 1.7 m^3/s
Additional capacity required	3.6 m^3/s
New PS configuration	1 duty, 1 standby
New PS pump capacity	3.6 m^3/s each
New pump motors	400 kW (each)

Functional design drawings for the pump station improvement is provided in Appendix F.

13.3 Consolidated Scully/St. Mark's Storm Pump Station Improvements

As identified in Section 5.3.5, there are currently three vertical turbine pumps installed at the existing Scully pump station. No upgrades have been completed since the station was put into operation in 1974. The electrical equipment is approaching end of its life. The St. Mark's pump station currently has two vertical turbine pumps and was constructed in 1957 and is also reaching its end of its life. The current pump station structures on both sites cannot be expanded to accommodate the increased flow.

Based on the age of the pump station infrastructure at this location and through the modelling analysis, it is recommended that a new pump station is constructed at the Scully pump station site to handle flow from a consolidated service area of the Scully and St. Mark's pump stations. The new station would utilize vertical submersible axial flow pumps. The station would be located north of the existing structure and will require a new inlet and outfall pipe, and expanded outfall structure. The existing pump stations would be kept in service during construction.

Capacity Summary

Required firm capacity	6.0 m^3/s
New PS configuration	2 duty, 1 standby
New PS pump capacity	3.0 m^3/s each
New pump motors	350 kW (each)

Functional design drawings for the consolidated pump station is provided in Appendix F.

13.4 PJ Cecile Storm Pump Station Improvements

As identified in Section 5.3.5, there are currently two vertical turbine pumps installed at the pump station both equipped with 40 hp motors. No upgrades have been completed since the station was put into operation in 1974. The electrical equipment is approaching end of its life.

Based on the age of the pump station infrastructure at this location and through the modelling analysis, it is recommended that a new pump station is constructed at the PJ Cecile PS site. Due to site restraints, the construction of the new pump station is proposed to be constructed over the footprint of the existing structure. The new station would utilize vertical submersible axial flow pumps. The installation of temporary pumps using portable pump stations is recommended to provide servicing during the construction. A new outfall pipe will be required to provide increased flow capacity. At this time, it is recommended to extend the new outfall to the northern end of the jetty bank to eliminate additional flow from entering the Beach Grove harbour, which is the location of the existing outfall. The inlet pipe to the pump station will be replaced with a larger diameter pipe in the existing alignment.

Capacity Summary

Required firm capacity	3.0 m^3/s
New PS configuration	1 duty, 1 standby
New PS pump capacity	3.0 m^3/s each
New pump motors	350 kW (each)

Functional design drawings for the pump station improvement is provided in Appendix F.

13.5 New Southwind/Starwood Pump Station

Based on the modelling analysis completed under high lake levels, it is proposed that a new pump station is constructed for the existing gravity outfall servicing the Southwind and Starwood residential development. The station is proposed to be constructed within the existing easement directly east of the Southwind right-of-way. The pump station will comprise of a below grade wet well and an above grade electrical panel. The existing outfall pipe will be maintained as the outlet and structure will be constructed within the existing easement.

Capacity Summary

Required firm capacity	200 L/s
New PS configuration	1 duty, 1 standby
New PS pump capacity	200 L/s each
New pump motors	20 kW (each)

Functional design drawings for the new pump station is provided in Appendix F.

14.0 Estimated Capital Construction Costs

14.1 Cost Assumptions

The cost assumptions for all recommended improvements for each of the service areas include the following:

- Construction cost estimates, including labour are based on 2018 unit prices and the accuracy of each estimate is +/- 10% and dependent on the timing of implementation;
- 30% contingency added for Capital Construction Costs;
- Future engineering costs calculated as 15% of capital construction costs; and
- Future Geotechnical Investigations as 2% of capital construction costs.

As part of this study, it has been identified that no potential land acquisition is required to construct any of the recommended surface flooding solutions.

Storm Sewer Infrastructure Improvements

Storm sewer construction cost estimates for works within the municipally owned right-of-way included the removal and restoration of one lane width and exclude all potential full roadway reconstruction, replacement or incorporation of curb and gutters and the potential for any utility (sanitary, watermain, hydro, gas) relocations.

Pump Station Improvements

In regards to the recommended improvements for the pump station design, it has been identified that these stations may not be constructed for a number of years, so a high level and conservative approach has been implemented with respect to the capital construction costs for each pump station.

Consideration was given to additional costs for flow control chambers, temporary pipes and pumps, decommissioning and demolishing of old stations and costing for new outfalls or improvements to the existing outfalls.

It is assumed that at this time, sufficient power is available to accommodate each storm pump station. The cost of the Starwood/Southwind pump station was estimated using recently completed 2018 pump station projects of similar sizes.

14.2 Capital Construction Costs and Implementation

The recommended surface flooding solutions outlined within this document have been designed to a level satisfying Approach 2 of the Master Planning process, in which all recommended surface flooding solutions are completed to a functional level of detail. Projects that have been determined as a "Schedule B" project have satisfied all requirements to move forward to further investigation and detail design. Cost estimates for all the proposed infrastructure upgrades have been developed and are included in Table 14.1. Figure 14.1 identifies the locations of all Schedule B Projects.

The implementation phasing outlined below has been broken down into each service area within the study. The service areas have been listed in order of need for the respective solutions to be constructed based on the existing conditions surface flooding identified through this study. The recommended improvements itemized for each service area have been listed based on the recommended phasing for each of the service areas. The recommended phasing has been determined based on both order of importance to the service area and the requirement for the improvement to be constructed for further solutions to commence with construction.

The implementation for the recommended solutions may be subject to change based on current capital works requirements, including road reconstruction and other municipal service improvements. At this time, the implementation of each solution has not been phased based on the storm improvement capital construction costs.

Table 14.1: Storm Drainage Capital Cost Estimates and Implementation Phasing

Implementation Stage ID	Improvement Details	Estimated Construction Cost and Contingency	Engineering Cost	Total
LESPERANCE PUMP STATION SERVICE AREA				
PS-1	Lesperance Pump Station Improvements	$14.30M	$2.43M	$16.73M
LE-1	Lesperance PS Storm Trunk Sewer – Riverside Drive (St. Pierre Street to PS)	$1.30M	$0.22M	$1.52M
LE-2	St. Pierre Street Trunk Sewer	$3.93M	$0.67M	$4.60M
LE-3	Clapp Street Local Sewers	$0.64M	$0.11M	$0.75M
LE-4	Meander Crescent Local Sewers	$0.90M	$0.15M	$1.05M
LE-5	Underground/Aboveground Storage (Tecumseh Centre Park)	$3.21M	$0.55M	$3.76M
LE-6	Evergreen Drive Local Sewers	$0.93M	$0.16M	$1.09M
LE-7	Gauthier Drive Local Sewers	$0.88M	$0.15M	$1.03M
	SUBTOTAL =	*$26.09M*	*$4.44M*	*$30.53M*

Implementation Stage ID	Improvement Details	Estimated Construction Cost and Contingency	Engineering Cost	Total
CONSOLIDATED SCULLY/ST. MARK'S PUMP STATION SERVICE AREA				
PS-2	New Consolidated Scully/St. Mark's Pump Station	$9.88 M	$1.68M	$11.56M
SM-1	Scully/St. Mark's PS Storm Trunk Sewer – Riverside Drive (Arlington Boulevard to PS)	$1.63M	$0.28M	$1.91M
SM-2	Grant Avenue Diversion Sewer	$0.58M	$0.10M	$0.68M
SM-3	Aboveground Storage (Tecumseh Soccer Fields)	$0.25M	$0.04M	$0.29M
SM-4	Edgewater Drive Local Sewers	$2.22M	$0.38M	$2.60M
SM-5	St. Gregory's Road Local Sewers and Diversion	$0.68M	$0.12M	$0.80M
SM-6	St. Marks Road Local Sewers	$1.83M	$0.31M	$2.14M
SM-7	Arlington Boulevard Local Sewers	$2.34M	$0.40M	$2.74M
	SUBTOTAL =	*$19.39M*	*$3.31M*	*$22.70M*
WEST ST. LOUIS PUMP STATION SERVICE AREA				
PS-3	West St. Louis Pump Station Improvements	$7.15M	$1.21M	$8.36M
WSL-1	West St. Louis PS Storm Trunk Sewer – Riverside Drive (Barry Avenue to existing 2000mm storm sewer)	$1.72M	$0.30M	$2.02M
WSL-2	Little River Boulevard Underground Storage	$2.24M	$0.38M	$2.62M
WSL-3	Coronado Dish Local Sewers*	$5.14M	$0.88M	$6.02M
WSL-4	Lacasse Boulevard Local Sewers	$0.98M	$0.17M	$1.15M
WSL-5	Kimberly Drive and Jelso Place Local Sewers	$0.73M	$0.05M	$0.78M
	SUBTOTAL =	*$17.96M*	*$2.99M*	*$20.95M*
EAST ST. LOUIS PUMP STATION SERVICE AREA				
ESL-1	St. Thomas Street Overflow Sewer to Lakewood Park & Backflow Prevention	$0.62M	$0.10M	$0.72M
	SUBTOTAL =	*$0.62M*	*$0.10M*	*$0.72M*

Implementation Stage ID	Improvement Details	Estimated Construction Cost and Contingency	Engineering Cost	Total
EAST TOWNLINE DRAIN SERVICE AREA				
ETD-1	Aboveground Storage (Buster Reaume Park) & Backflow Prevention Device	$0.18M	$0.03M	$0.21M
ETD-2	Lemire/Lanoue Street Local Sewers and Sewer Diversion	$1.46M	$0.25M	$1.71M
ETD-3	Manning Road Phase 2 Drain Enclosure	$3.70M	$0.63M	$4.33M
	SUBTOTAL =	*$5.34M*	*$0.91M*	*$6.25M*
BAILLARGEON DRAIN SERVICE AREA				
BD-1	Charlene Lane Flooding Solution	$3.00M	$0.51M	$3.51M
BD-2	St. Anne Area Local Sewers*	$3.60M	$0.62M	$4.22M
	SUBTOTAL =	*$6.60M*	*$1.13M*	*$7.73M*
PJ CECILE PUMP STATION SERVICE AREA				
PS-2	PJ Cecile Pump Station Improvements	$7.02M	$1.20M	$8.22M
PJ-1	Kensington Dish Area Local Sewers	$3.96M	$0.68M	$4.64M
	SUBTOTAL =	*$10.98 M*	*$1.88M*	*$12.86M*
SOUTHWIND/STARWOOD AREA				
PS-5/SS-1	New Starwood/Southwind Pump Station and Backflow Prevention Device	$0.90M	$0.15M	$1.05M
	SUBTOTAL =	*$0.90M*	*$0.15M*	*$1.05M*
BRIGHTON PUMP STATION SERVICE AREA				
B-1	Tecumseh Road Storm Sewer Extension	$3.25M	$0.55M	$3.80M
	SUBTOTAL =	*$3.25M*	*$0.55M*	*$3.80M*
	TOTAL =	$91.13M	$15.46M	$106.59M

Construction costs include 30% contingency, Engineering costs include 15% engineering and 2% Geotechnical Investigations
All estimated costs above exclude applicable taxes
**Lumped areas for storm sewer reconstruction have the potential to be phased to implement upstream solutions earlier*

Beyond the larger scale projects identified above, a number of smaller catch basin inlet improvements are proposed within the study area to reduce surface flooding at a localized level (As shown on Figure 11.14). These minor storm inlet improvements have not been costed out at this time and include:

- Incorporation of Inlet Control Devices (ranging in sizes of 100 mm – 200 mm in diameter with flow restrictions ranging from 28 L/s – 129 L/s) within catch basins to promote surface ponding in areas currently not susceptible to surface flooding to reduce inflows into the storm sewer system; and
- Introduction of additional catch basins along roadways to increase inlet efficiency to reduce surface flooding.

Subject to resolving any concerns that may arise following the Notice of Completion and the required 30-day public and agency review period, any of the stages outlined above may proceed with detailed design and following the necessary agency and regulatory approvals, proceed to construction.

Figure 14.2 is a summary map of the locations for all recommended surface flooding solutions throughout the study area including, pump station improvements, storm sewer infrastructure improvements, localized storage area and catch basin localized improvements.

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

SCHEDULE B PROJECT MAP
FIGURE 14.1

- PUMP STATION IMPROVEMENTS
- ABOVE GROUND STORAGE
- ABOVE / UNDERGROUND STORAGE

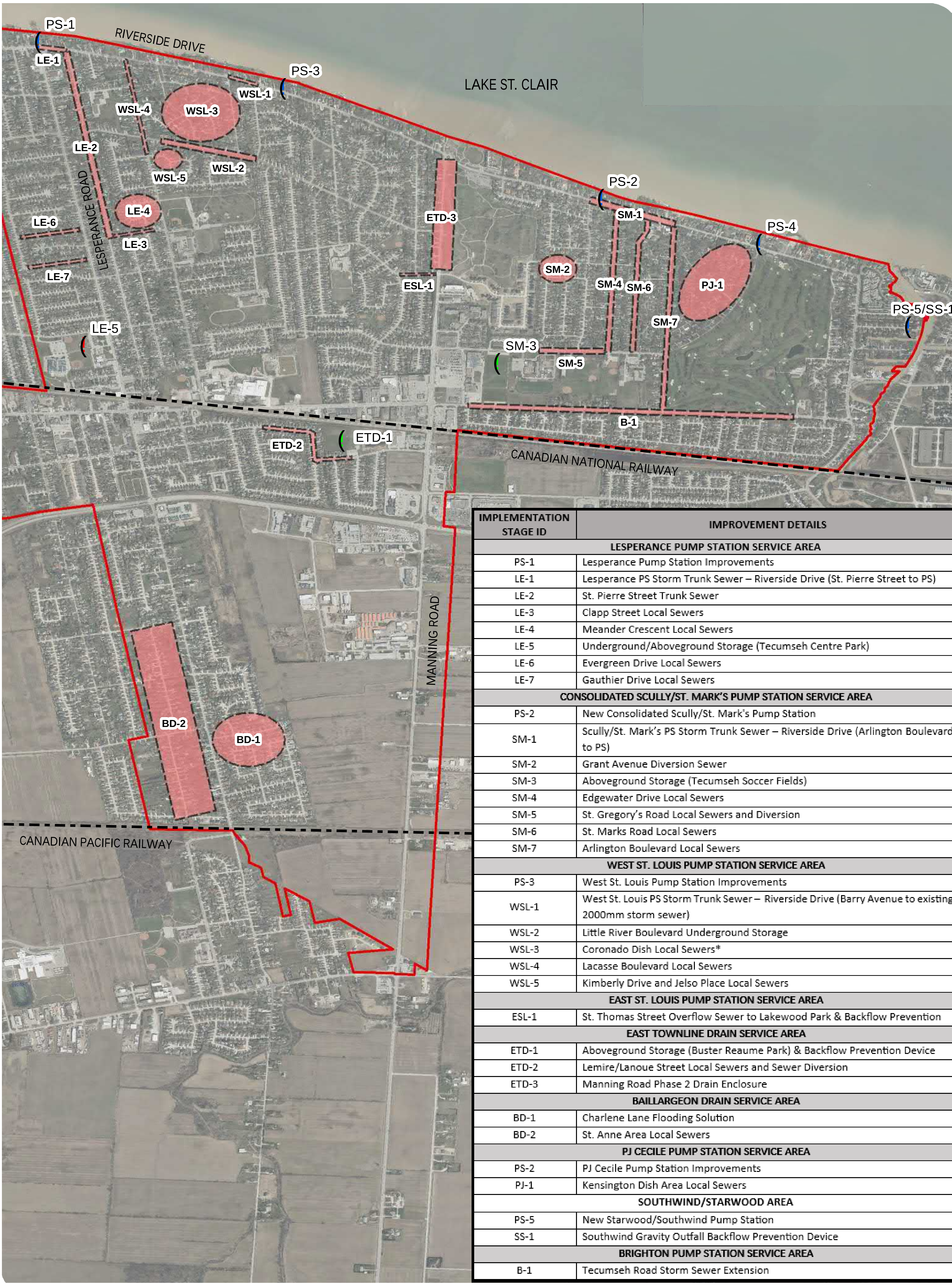
1 LESPERANCE STORM PUMP STATION IMPROVEMENTS
2 WEST ST. LOUIS STORM PUMP STATION IMPROVEMENTS
3 CONSOLIDATED SCULLY/ ST. MARKS STORM PUMP STATION
4 PJ CECILE STORM PUMP STATION IMPROVEMENTS
5 NEW SOUTHWIND CRES. STORM PUMP STATION
6 SURFACE STORAGE IN SOCCER FIELD
7 SURFACE STORAGE IN BUSTER REAUME PARK
8 SURFACE AND UNDERGROUND STORAGE (TECUMSEH CENTRE PARK)
RAILWAY

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019



IMPLEMENTATION STAGE ID	IMPROVEMENT DETAILS
LESPERANCE PUMP STATION SERVICE AREA	
PS-1	Lesperance Pump Station Improvements
LE-1	Lesperance PS Storm Trunk Sewer – Riverside Drive (St. Pierre Street to PS)
LE-2	St. Pierre Street Trunk Sewer
LE-3	Clapp Street Local Sewers
LE-4	Meander Crescent Local Sewers
LE-5	Underground/Aboveground Storage (Tecumseh Centre Park)
LE-6	Evergreen Drive Local Sewers
LE-7	Gauthier Drive Local Sewers
CONSOLIDATED SCULLY/ST. MARK'S PUMP STATION SERVICE AREA	
PS-2	New Consolidated Scully/St. Mark's Pump Station
SM-1	Scully/St. Mark's PS Storm Trunk Sewer – Riverside Drive (Arlington Boulevard to PS)
SM-2	Grant Avenue Diversion Sewer
SM-3	Aboveground Storage (Tecumseh Soccer Fields)
SM-4	Edgewater Drive Local Sewers
SM-5	St. Gregory's Road Local Sewers and Diversion
SM-6	St. Marks Road Local Sewers
SM-7	Arlington Boulevard Local Sewers
WEST ST. LOUIS PUMP STATION SERVICE AREA	
PS-3	West St. Louis Pump Station Improvements
WSL-1	West St. Louis PS Storm Trunk Sewer – Riverside Drive (Barry Avenue to existing 2000mm storm sewer)
WSL-2	Little River Boulevard Underground Storage
WSL-3	Coronado Dish Local Sewers*
WSL-4	Lacasse Boulevard Local Sewers
WSL-5	Kimberly Drive and Jelso Place Local Sewers
EAST ST. LOUIS PUMP STATION SERVICE AREA	
ESL-1	St. Thomas Street Overflow Sewer to Lakewood Park & Backflow Prevention
EAST TOWNLINE DRAIN SERVICE AREA	
ETD-1	Aboveground Storage (Buster Reaume Park) & Backflow Prevention Device
ETD-2	Lemire/Lanoue Street Local Sewers and Sewer Diversion
ETD-3	Manning Road Phase 2 Drain Enclosure
BAILLARGEON DRAIN SERVICE AREA	
BD-1	Charlene Lane Flooding Solution
BD-2	St. Anne Area Local Sewers
PJ CECILE PUMP STATION SERVICE AREA	
PS-2	PJ Cecile Pump Station Improvements
PJ-1	Kensington Dish Area Local Sewers
SOUTHWIND/STARWOOD AREA	
PS-5	New Starwood/Southwind Pump Station
SS-1	Southwind Gravity Outfall Backflow Prevention Device
BRIGHTON PUMP STATION SERVICE AREA	
B-1	Tecumseh Road Storm Sewer Extension

PUMP STATION IMPROVEMENTS
ABOVE GROUND STORAGE
ABOVE/UNDERGROUND STORAGE
STUDY AREA
STORM DRAINAGE AND INFRASTRUCTURE IMPROVEMENTS
RAILWAY

TOWN OF TECUMSEH
STORM DRAINAGE MASTER PLAN

RECOMMENDED SOLUTIONS SUMMARY MAP
FIGURE 14.2

TOWN OF Tecumseh ONTARIO · CANADA

DILLON CONSULTING

MAP CREATED BY: SZ
MAP CHECKED BY: RTL
MAP PROJECTION: NAD 1983 UTM Zone 17N

SCALE 1:NTS

2

PROJECT: 16-4880 STATUS: FINAL DATE: JUNE 2019

15.0 Implementation/Phasing and Detailed Design

15.1 Implementation Considerations

The recommended solutions outlined with this study to reduce surface flooding are generally extensive, broader-based projects that are expected to be phased over a long-term implementation plan. In particular, the majority of the storm sewer infrastructure improvements that form solutions are expected to be implemented in conjunction with the planned reconstruction of roadways and other municipal infrastructure based on schedules defined within the Town of Tecumseh's Long Term Capital Works Plan.

In order to achieve more immediate surface flood relief in certain areas, phasing of larger storm sewer infrastructure solutions may be deemed beneficial in improving the resiliency of the system to address flood vulnerability and climate change considerations. If phasing of the broader-based storm infrastructure improvements is preferred, the PCSWMM model should be further evaluated based on the interim phasing of projects to assess the impacts on the system prior to full build out of the solutions.

The findings of this study and solutions are functional in nature and provide sound direction on the nature of the designs that would be effective in addressing surface flooding concerns. A proposed construction implementation strategy has been identified for the study area in Table 14.1.

15.2 Detailed Design Considerations

During detailed design of the functional design solutions outlined within this study, it is recommended that the following design considerations be included:

- Further geotechnical assessments;
- Erosion and coastal assessment surrounding areas of pump station and outlet improvements;
- Erosion and sediment control plans;
- Water management plan during construction of in-water works at pump station outfalls;
- Further archaeological investigation, if required;
- Additional natural environment investigations, if required in support of permits/approvals from MECP;
- Potential requirements for water quality control for any of the recommended solutions involving upgrades to the storm sewer infrastructure where roadways are being reconstructed from semi-urban cross sections to fully urban roadway cross sections as directed by the MECP during the Environmental Compliance Process;
- Obtain permits from appropriate agencies as required; and
- Use of Low Impact Development techniques.

15.3 Next Steps

The following studies, design and approval requirements will influence the schedule for implementation of the solutions outlined in this report:

- Potential refinement of the recommended solutions based on any future developments (greenfield or infill) not assessed within this study that could impact the design of each solution;
- Detailed design of all recommended improvements;
- Environmental Compliance Approvals for all storm sewer and pump station infrastructure works; and
- Essex Region Conservation Authority and municipal permitting and approvals.

16.0 Conclusions and Recommendations

Dillon Consulting Limited was retained by the Town of Tecumseh to complete a Storm Drainage Master Plan (Storm Drainage MP) for the urbanized residential areas within the Town currently being serviced by the storm pump stations outletting into both Lake St. Clair and Pike Creek. The Storm Drainage MP follows the requirements of the Municipal Class Environmental Assessment (Class EA) (2000, as amended) - Approach No. 2 and the requirements of Phases 1 and 2 of the Class EA, including requirements for any Schedule B projects. The purpose of the study was to complete an existing condition modelling analysis to identify surface flooding problem areas during large storm events and develop surface flooding solutions to reduce ponding depths along the roadway during extreme rainfall events to provincially acceptable levels. Solutions were further analyzed through the surface flooding solution decision framework discussed in Section 9.0 and designed to a functional level of design for either a traditional or more adaptive and enhanced level of service.

To assess both existing and future conditions throughout the study area, a 1-Dimensional/2-Dimensional PCSWMM model was developed to dynamically analyze the study area and included the integration of linear storm infrastructure throughout the Town, including storm sewers, catch basins, pump station and municipal drains as a 1-Dimensional hydraulic model for stormwater runoff. The advanced 2-Dimensional modelling component was used to integrate the existing ground surface (roadways, overland flow routes, private property land topography) using digital elevation mapping (DEM) taken from 2017 LiDAR.

Based on the existing condition analysis, surface flooding problem areas were delineated based on areas east and west of Manning Road/County Road 42. The problem areas were determined by identifying surface ponding depths within the municipal right-of-way during more infrequent rainfall events where surface ponding exceeds +- 0.30 m during the Chicago 1:100 year 4 hour storm event. This criteria is consistent with the recently completed Windsor/Essex Region Stormwater Management Standards Manual (December 2018). Each problem area throughout the study area is illustrated in Figure 8.2, Figure 8.3 and Figure 8.5. Problem areas included both regional areas and isolated surface flooding areas.

A number of alternative regional surface flooding solutions were developed for each problem area as part of the Master Planning Process (detailed in Section 10.0). Through the Environmental Assessment (EA) process, an evaluation of each alternative regional surface flooding solution was completed to identify the most suitable recommended solution for the problem area. Based on the proposed design for each alternative discussed in Section 10.0, a recommended solution was identified for each problem area. The recommended regional surface flooding solutions for this study are as follows, further detailed in Section 11.0 and shown in Figure 14.2:

SURFACE FLOODING SOLUTIONS – WEST OF MANNING ROAD/COUNTY ROAD 19

Lesperance Pump Station Service Area
Regional Solution W-1

- Lesperance Pump Station upgrade to a firm pumping capacity of 9 m^3/s; and
- Construction of a new storm trunk sewer along St. Pierre Street from Clapp Street to Riverside Drive.

West St. Louis Pump Station Service Area
Regional Solution W-1

- West St. Louis Pump Station upgrade to a firm pumping capacity of 7 m^3/s;

East St. Louis Pump Station Service Area
Regional Solution W-2

- 1050mm diameter storm sewer overflow from the existing 1350 mm diameter storm trunk sewer along St. Thomas Street at the intersection with the Green Valley Drive storm sewer. Overflow outlet to the Lakewood Park Drainage Channel.

Baillargeon Drain Service Area
Regional Solution W-3

- Alternative 1:
  - Storm relief sewer (675 mm to 900 mm diameter) along Charlene Lane to intercept storm sewer runoff east of Lesperance Road and south of Charlene Lane from the existing residential lands;
  - Relief sewer outlet proposed through the existing 4.0 m storm easement between two residential properties and into the future development storm trunk sewer;
- Alternative 2:
  - 825 mm diameter underground storage sewer along St. Martin directly upstream of the existing 675 mm diameter storm sewer outlet from St. Martin Crescent to Charlene Lane;
  - 900 mm diameter underground storage sewer Charlene Lane directly north of Eugeni Street; and
  - 900 mm x 1800 mm underground storage rectangular box chambers along Charlene Lane and St. Agnes upstream of the existing 1200 mm diameter enclosed Baillargeon Drain.

Localized Solutions West of Manning Road

- Roadway and storm sewer improvements within the Coronado Dish area and St. Anne Street Area;
- Oversized storm sewers along Evergreen (1200mm diameter) and Gauthier Drive (1050mm diameter) with existing orifice controls to the Lesperance storm trunk sewer maintained;
- Roadway and storm sewer improvements along Meander Crescent;
- Incorporation of a backflow prevention device at the shared storm manhole between the two service areas along Riverside Drive directly east of St. Pierre Street; and
- Storm inlet modifications (ie. inlet control devices, catchbasin inlet improvements) as identified in Section 11.3.

Lesperance Road within Tecumseh Centre Park

- Depression of open space green areas for approximately 1,081 m^3 of surface storage within Tecumseh Centre Park; and
- Incorporation of approximately 2,000 m^3 of underground chamber system storage.

Little River Boulevard

- Underground storage along Little River Boulevard, from Lacasse Boulevard to Michael Drive, in the form of 1500 mm diameter storm sewers;
- Storm sewer upgrades along Jelso Place, Kimberly Drive and Lacasse Boulevard. Storm sewer sizes in the proposed conditions ranged from 525 mm diameter to 1200 mm diameter;
- 600 mm diameter overflow sewer from the Little River Boulevard storm sewer to the new storm trunk sewer along St. Pierre Street; and
- Increased catch basin inlets along Jelso Place, Kimberly Drive, Lacasse Boulevard and Little River Boulevard.

Manning Road Phase 2 Drain Enclosure

- 1.80 m x 3.0 m box structure Enclosure of the East Townline Drain (ETLD) between the existing culvert outlet north of St. Gregory's Road, to the proposed outlet at St. Thomas Street to the Lakewood Park Drainage Channel. The completion of this culvert extension will re-direct the ETLD flows from the existing ditch on the west side of Manning Road to the newly constructed open channel in the Lakewood Park; and
- Construction of a local storm sewer system (sizes ranging from 600 mm - 900 mm diameter) servicing Manning Road residential properties, between Riverside Drive and St. Thomas Street, including the backfill of the existing swale on the west side of Manning Road.

Lemire Street and Lanoue Street

- Redirection of storm drainage and upgrade of local storm sewers (675 mm – 825 mm diameter) along Lanoue and Lemire street through to Buster Reaume Park and discharge flows into the existing 900 mm diameter CN Railway Ditch outlet;

- Construction of a 0.80 m depression along the southwestern portion of Buster Reaume Park to provide approximately 4,100 m^3 of aboveground surface storage with a connection to the upgraded municipal storm sewers;
- Installation of a catch basin and a 750 mm diameter storm sewer outlet from the Buster Reaume Park aboveground storage to the existing 900 mm diameter storm sewer to the CN Railway ditch; and
- Installation of a backflow prevention device at the outlet of the 900 mm diameter sewer to the CN Railway ditch within the Buster Reaume pathway from Lemire Street.

SURFACE FLOODING SOLUTIONS – EAST OF MANNING ROAD/COUNTY ROAD 19

Scully/St. Mark's/PJ Cecile Pump Station Service Area

Regional Solution E-1

- Decommission of the St. Mark's storm pump station and consolidation of the Scully and Mark's pump station service areas;
- Consolidated Scully/St. Mark's pump station upgraded to a firm pumping capacity of 6 m^3/s;
- PJ Cecile Pump Station upgrade to a firm pumping capacity of 3 m^3/s;

Localized Solutions East of Manning Road

- Roadway and storm sewer improvements along Edgewater Boulevard, St. Mark's Road, Arlington Boulevard and the Kensington Dish area;
- Extension of the Tecumseh Road Sewer and enclosure of the Tecumseh Road Ditch from Dresden Place to the storm sewer tie-in east of Lexham Gardens to convey flows ultimately to the Brighton pump station; and
- Storm inlet modifications (ie. inlet control devices, catchbasin inlet improvements) as identified in Section 11.3.

St. Gregory's Road at Tecumseh Soccer Fields

- Incorporation of a backflow prevention device at the St. Gregory's Road storm outlet to the East Townline Drain at the intersection of St. Gregory's and Manning Road;
- Elimination of the storm sewer interconnection along St. Gregory's Road fronting L'Essor School with the East Townline Drain and Scully storm pump station service areas;
- Redirection of the storm sewer outlet for St. Gregory's Road East of L'Essor School to extend further east from the Cada Crescent storm outlet to the proposed Edgewater Drive storm sewer;
- Road grading improvements along St. Gregory's Road, from Village Grove Drive to L'Essor School to reduce surface flooding depths in this area of the road; and
- Construction of a 0.70 m depression of the existing northern soccer fields within the Tecumseh Soccer Fields Park currently owned by the L'Essor high school and Conseil Scolaire Catholique (CSC) Providence to provide approximately 3,200 m3 of aboveground surface storage with a 200mm

diameter storm sewer connection to the existing 750 mm diameter storm sewer along St. Gregory's Road.

Starwood Lane/Southwind Crescent

- Incorporation of a new pump station at the existing gravity outlet into Pike Creek with a firm pumping capacity of 0.2 m^3/s.

A number of the recommended surface flooding solutions (both regional and local) have been identified as Schedule B Projects, as shown in Figure 14.1. These include the following:

1. Lesperance Storm Pump Station Improvements;
2. West St. Louis Storm Pump Station Improvements;
3. New Consolidated Scully/St. Mark's Storm Pump Station;
4. PJ Cecile Storm Pump Station Improvements;
5. New Southwind Crescent Storm Pump Station;
6. Surface storage within the "Tecumseh Soccer Fields" Park at École Secondaire L'Essor;
7. Surface Storage within Buster Reaume Park; and
8. Surface and Underground Storage within Tecumseh Centre Park.

The findings of this study and solutions are functional in nature and provide sound direction on the nature of the designs that would be effective in addressing surface flooding concerns. A proposed construction implementation strategy has been identified for the solutions within the study area and detailed in Table 14.1.