ESSEX TOWN CENTRE STORM DRAINAGE SYSTEM STUDY

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Prepared for:
Town of Essex

Prepared by:
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1.0: INTRODUCTION

1.1: BACKGROUND

The Town of Essex (the Town) is comprised of four urban communities: Essex (Essex Town Centre / Ward 1 Area), Harrow, Colchester, and McGregor. The Town was created on January 1st, 1999, with the amalgamation of the former towns of Essex and Harrow, and townships of Colchester North and Colchester South. The population of the Town is approximately 21,200 and covers a land area of approximately 278 square kilometers. Generally speaking, the Town extends from the shore of Lake Erie on the south to County Road No. 8 on the north and from County Road 11/County Road 41 on the west and County Road 23 on the east. The topography in the region is relatively flat and slopes gently to Lake Erie (in the south). The predominant soil type in the region is impervious and/or clayey soils with scattered sandy and gravely knolls. The combination of relatively flat and impervious soil results in increased challenges to effective stormwater management.

The Essex Town Centre is located in the northern portion of the Town and lies in the centre of Essex County at the intersection of several natural drainage systems. Essex Town Centre's drainage basin consists of five (5) urban catchments: South Talbot, Maidstone, Rush, Hopgood, and Arner Townline. Stormwater runoff from Essex Town Centre is conveyed through the stormwater drainage system to receiving municipal drains in two (2) subwatersheds. The northern portion of the Town is located within the upper limits of the Puce River subwatershed and the southern portion of the Town is located within the upper limits of the Canard River subwatershed. Runoff from Puce River and Canard River subwatersheds is conveyed in watercourses through adjoining towns to Lake St. Clair (roughly 15 km downstream) and the Detroit River (roughly 35 km downstream).

In 2019, the Town of Essex retained Stantec Consulting Ltd. (Stantec) to carry out a storm sewer study of the Essex Town Centre. Upon completion of the study, the findings were summarized in a study report entitled "Essex Storm Sewer System Hydrologic-Hydraulic Model Development, Calibration and Capacity Assessment" dated January 31, 2019. As a part of this study, a hydrologic-hydraulic model was developed and calibrated for a majority of the Essex Town Centre's stormwater drainage system. Further to this study, the Town of Essex retained Stantec to carrying out a Municipal Class Environmental Assessment (Class EA) for improvements to the Essex Ward 1 Southwest Storm Sewer System which is focused on the South Talbot Catchment Area. The Class EA reports on planning-level conceptual designs to accommodate future development and improve flood protection within the South Talbot catchment area. The remaining four (4) urban catchments areas remain to be updated.

1.2: OBJECTIVE

The objective of this Essex Town Centre study is to build upon the findings of the 2019 storm sewer study of the Essex Town Centre and develop planning-level conceptual designs in the remaining four (4) urban catchment areas (Maidstone, Rush, Hopgood, and Arner Townline). This study includes updating and calibrating the hydrologic-hydraulic model to reflect ongoing and future development in the region with the purpose of performing a capacity assessment. This capacity assessment will be used to identify areas of concern in these catchments and make preliminary recommendations for improvements to the stormwater drainage system.

1.3: REFERENCES AND DATA SOURCE

The sources of information referred to in this study include construction record drawings, field investigations, previous reports, codes, standards, and guidelines. The list of references includes the following:

- Essex Storm Sewer System Hydrologic-Hydraulic Model Development, Calibration and Capacity Assessment. Stantec Consulting Ltd. 2019.
- Town of Essex Interactive Mapping Service.
- Essex Regional Conservation Authority (ERCA) Interactive Mapping Service.
- Design Guidelines for Sewage Works. Ontario Ministry of Environment, Conservation and Parks (MECP). 2008.
- Stormwater Management Planning and Design Manual. MECP. 2003.
- Windsor / Essex Region Stormwater Management Standards Manual. Stantec. 2018.
- Storm Water Management Model (SWMM) User's Manual V5.0. United States Environmental Protection Agency (USEPA). 2010.
- SWMM Reference Manual Volume I – Hydrology (Revised). USEPA. 2016.
- SWMM Reference Manual Volume II – Hydraulics. USEPA. 2017.
- Report on Storm Water Drainage in the Town of Essex. Lafontaine Cowie Buratto Ltd. 1975.

2.0: EXISTING CONDITIONS

2.1: STORMWATER SERVICING STUDIES / MASTER PLANS

2.1.1: History of Construction

The earliest available storm sewer records for the Essex Town Centre date back to the late 1960's. Construction record drawings before this period are scarce. Prior to 1960, the stormwater drainage system was constructed primarily of ditches, small diameter sewers, and clay tiles which were shallow in depth and outlet into municipal drains in adjoining municipalities. Many of the sewers and tiles were hydraulically inadequate and have since silted up and are partially plugged.

2.1.2: Stormwater Drainage Studies

Stormwater drainage studies were undertaken in 1968 (C.G. Russel Armstrong Associates Ltd.) and 1975 (LCBA Ltd.) with the purpose to undergo a systematic storm sewer construction program to improve stormwater drainage in the Town of Essex. The LCBA study proposed a conceptual layout of the storm sewer system based on 2yr return period, average 30min inlet time, and rational method design. Portions of the Town's storm sewer system have been upgraded to meet or exceed this design criteria over the course of many contracts completed since the 1975 LCBA study. Through these contracts many storm sewers were installed as open ditches and drains were filled or as clay tiles were replaced. Although a majority of the recommendations in the LCBA study have been completed by the Town to date, some have not been completed.

As noted, Stantec completed a storm sewer study of the Essex Town Centre in 2019. Upon completion of the study, the findings were summarized in a study report entitled "Essex Storm Sewer System Hydrologic-Hydraulic Model Development, Calibration and Capacity Assessment" dated January 31, 2019. As a part of this study, a hydrologic-hydraulic model was developed in PCSWMM and calibrated for a majority of the Essex Town Centre's stormwater drainage system. The report documented the approach and methodology used in model development and model calibration as a basis for future capacity assessments.

Due to the length of time since previous capacity studies and in consideration of concerns regarding flooding risk, the Town decided it was prudent to perform a capacity assessment of the South Talbot catchment, one of the most developed areas within the Township. Therefore, in 2021 the Town retained Stantec to initiate a Class EA for improvements to the Essex Ward 1 Southwest Storm Sewer System focused on South Talbot Catchment Area. The project objective was to reduce the risk and extent of flooding and property damage through storm system improvements and evaluate the storm system to ensure adequate infrastructure is in place for future development in the South Talbot catchment. This ongoing Environmental Study Report (ESR) is the documentation of the Class EA process outlined by the Municipal Engineers Association (MEA) for the South Talbot stormwater system and reports on planning-level conceptual designs.

2.2: STORMWATER CATCHMENTS AND INFRASTRUCTURE OVERVIEW

The Essex Town Centre's drainage basin consists of five (5) urban catchments: South Talbot, Maidstone, Rush, Hopgood, and Arner Townline and one (1) rural catchment: South Talbot. The Essex Town Centre stormwater drainage system is outlined in **Table 2.1** and shown in **Figure 2.1** of **Appendix B**. Additional detailed regarding the Essex Town Centre's catchments and infrastructure is outlined in **Section 3.0:**. The stormwater drainage subwatersheds are shown in **Figure 2.2** of **Appendix B**.

Stormwater runoff from Essex Town Centre is conveyed through the stormwater drainage system to receiving municipal drains in two (2) subwatersheds. The northern portion of the Town is located within the upper limits of the Puce River subwatershed and the southern portion of the Town is located within the upper limits of the Canard River subwatershed. Runoff from Puce River and Canard River subwatersheds is conveyed in watercourses through adjoining towns to Lake St. Clair (roughly 15 km downstream) and the Detroit River (roughly 35 km downstream).

Table 2.1: Essex Town Centre Subcatchments

Catchment	Area (ha)	Subwatershed	Outlet
South Talbot			
Urban	250	Canard River	Essex Outlet Drain & Canaan Drain
Rural	1,300	Canard River	Essex Outlet Drain & John's Creek Drain
Rush	115	Canard River	Rush Drain & 14th Conc. E Drain (Munch Drain)
Arner Townline	21	Canard River	West Townline Drain & East Townline Drain
Maidstone	155	Puce River	Maidstone Ave Drain
Hopgood	40	Puce River	Hopgood Drain

As a part of the 2019 study, a hydrologic-hydraulic model was developed through field investigations and field measurements, review of record drawings, and CCTV inspection of targeted storms sewers. This model represented the drainage system as of April 2016. As a part of this study update, the hydrologic-hydraulic model was updated to represent the drainage system as of May 2023. The following is a list of storm sewer improvement projects and development projects which were incorporated into the updated model based on provided record drawings:

Storm Sewer Improvement Projects:

- Victor Street Reconstruction - 2022
- Southwest Area Storm Sewer Improvements (Brien Avenue) - 2021

Historic and Ongoing Development Projects:

- Essex Town Centre Development (Phase 1 & 5) - 2022
- Woodview Estates (Phase 3) - 2022

- Jakana Subdivision - 2020
- Townsview Estates - 2017

2.3: SOIL CONDITIONS & TOPOGRAPHY

The soil conditions in the Essex Town Centre are generally described as having a topsoil stratum of approximately 0.3 m (1 foot) overlaying a predominantly clay stratum. According to the Essex County Soil Map, the soils in this area are predominantly Brookston Clay with a small portion of Fox Sandy Loam.

The Town of Essex is located in the centre of the County at the height of land between Lake St. Clair and Lake Erie. The grade is highest along Talbot Street North and slopes gently away from Talbot Street North on either side. **Figure 2.3** of **Appendix B** shows a site plan of the general topography in the Essex Town Centre.

2.4: EXISTING SEWER DESIGN CRITERIA

The historic storm sewer design criteria used to design the Town's existing storm sewer system are shown in **Table 2.2**. The storm sewer conveyance system was designed primarily using the rational method with 2-year Yarnell IDF-curve intensity (C = 0.3 for most residential land uses). Portions of the system related to the newer developments were designed using the rational method with 5-year AES Windsor Airport Station No. 6139525 IDF-curve intensity.

Table 2.2: Existing Storm Sewer Design Criteria

	Reference	Return Period	Urban Inlet Time	Method of Design
Pre- 1975	-	Unknown	Unknown	Unknown
1975 – 2003	LCBA Stormwater Drainage Study	2-year Yarnell's formula	30 min avg.	Rational
Post-2003	Town of Essex Development Standards Manual, 2003	5-year AES	Not specified (typical is 10 – 20 min)	Rational

3.0: STORMWATER CATCHMENTS AND INFRASTRUCTURE

The boundaries of subcatchments within the South Talbot Catchment, Maidstone Catchment, Rush Catchment, Hopgood Catchment, and Arner Townline Catchment are shown in **Figure 2.1**. Discussion on the individual subcatchments are provided in the following sections.

3.1: SOUTH TALBOT CATCHMENT

The South Talbot catchment is located in the southern and central section of the Town of Essex and is generally bounded by Talbot Road (northeastern limit), Gosfield Townline (southeastern limit), Highway No. 3 (southwestern limit), and the Canadian Southern Railway Line (northwestern limit). The South Talbot catchment's minor system contains trunk sewers constructed primarily along north-south streets intercepted via two interceptor sewers over seven (7) subcatchments which further drain to the Canard River Subwatershed. The Hanlan Street interceptor sewer splits flows to downstream trunk sewers, and the South Talbot Road interceptor sewer conveys flow to receiving municipal drains. Runoff in the Essex Outlet Drain flows freely downstream to Canard River with overflow to John's Creek Drain which outlets to Craig's Creek Drain and ultimately Canard River. The first flush from a portion of the catchment serviced by the Canaan Drain is designed to be diverted to the Canaan Pond for treatment and the remainder is designed to overflow to the downstream Canaan Drain (however, the Canaan diversion weir and pond do not currently function as designed). Runoff from the South Talbot catchment is conveyed via the Canard River through adjoining towns to the Detroit River (roughly 35km downstream).

The catchment's sewer system is hydraulically connected to the Maidstone catchment via an overflow sewer at Iler Avenue and Talbot Street North and connected to the Essex PCP sanitary sewer system via a normally closed sluice gate at Laird Avenue and South Talbot Road.

3.1.1: LAND USE

The South Talbot Catchment consists of 250 ha of urban land that is mostly residential with commercial lands concentrated in the downtown core, and 1,300 ha of rural agricultural land. The existing land uses in the catchment are shown in **Table 3.1** and **Figure 3.1** in **Appendix B**. Urban subcatchment boundaries were developed based on the as-constructed storm sewer system, and the rural subcatchment boundaries in the South Talbot catchment were developed from the Essex Outlet Drain drainage plan prepared by C.G. Russell Armstrong Ltd. dated August 1975.

Table 3.1: Existing Land Uses in the South Talbot Catchment

Land Location	Land Use	Area (ha)	Fraction of Total Area (%)
Urban (within Essex's urban limits)	Industrial	1.7	0.7%
	Commercial	10.6	4.0%
	Institutional	11.7	5.0%
	Residential	162.5	65.0%
	Mobile Homes	17.8	7.0%
	Apartments & Hotels	0.2	0.1%
	Parkland	9.5	4.0%
	Open Space or Agricultural	37.0	15.0%
Rural (outside Essex's urban limits)	Primarily Agricultural	1,300.0	

3.1.2: STORMWATER MANAGEMENT CONTROLS & INFRASTRUCTURE

3.1.2.1: CONVEYANCE CONTROLS

Conveyance controls in the catchment are shown in **Table 3.2**. Conveyance controls are installed in Subcatchment 11 (Woodview Estates) and Subcatchment 12 (Tulley Meadows).

Table 3.2: Conveyance Controls in the South Talbot Catchment

Hydraulic Structure	Location	Purpose
Orifice	Woodview Estates Channel Outlet	Quantity - Restrict release rate to receiving channel
Orifice – Superpipe Storage	MH_1 173 Oak Dr	Quantity - Restrict release rate to receiving channel

3.1.2.2: END-OF PIPE STORMWATER MANAGEMENT FACILITIES

There are no pumping stations and two stormwater management ponds in the South Talbot catchment: (1) Canaan Pond and (2) Woodview Estates Pond. The locations of the stormwater management ponds are shown in **Figure 2.1** in **Appendix B**.

Canaan Pond was designed as an off-site dry pond system and was sized to service the first flush (runoff = 6 mm) from Kimball & Jakana Development (14.9 ha) for basic level quality control according to the Drainage Report and ECA prepared by NJ Peralta in 2004. The first flush is designed to be diverted to the pond and runoff exceeding this quality volume overflows to the downstream receiving channel. The outlet structure was designed to restrict the pond release rate over a 24hr drawdown period at the quality water level (HGL = 192.01 m) via a 150 mm diameter orifice to the Canaan Drain with provision for spillway overflow when the hydraulic grade line reaches 193.29 m. Field investigations, undertaken by Stantec Consulting as a part of the 2019 ESR, showed currently the pond does not operate as intended in the design. The Canaan Pond has a permanent pool depth of approximately 500 mm because the outlet structure does not drain the pond. This causes standing water upstream of the pond (channel weir) at approx. the weir crest elevation (quality water level), and the first flush in practice flows primarily to the receiving channel. In its current condition, the pond provides minimal quality and quantity control to benefit the South Talbot catchment's storm sewer system. The Canaan Pond is constructed under Section 78 of the Drainage Act R.S.O. 1990 c. D17 and covered by the current MOE Certificate of Approval No. 3364-6SNQNA, issued on August 26, 2006. Access to the Canaan Pond parcel owned by the Town is through a 6 m wide easement along the west limit of 2964 County Rd 12.

The Woodview stormwater management pond design consists of a Forebay and meadow (dry open channel). The Forebay with infiltration drawdown to the outlet channel and a bypass when forebay capacity exceeded. The outlet structure at the end of the meadow (outlet channel upstream of Canaan Drain) is equipped with a 150 mm dia. orifice to restrict the release rate, principal spillway for overflowing the 10-year and auxiliary spillway for overflowing the 100yr storm. No Stormwater Management Report or ECA that Stantec is aware of is available for the Woodview Pond SWM Facility. The land where the pond was constructed is owned by the County of Essex. Stantec is not aware of a registered easement for the existing Woodview SWM Facility.

The stormwater management pond as-built design parameters are shown in **Table 3.3**.

Table 3.3: South Talbot Stormwater Management Pond Design Parameters

Pond	Service Area	Pond Design	Quantity or Quality Control	Liquid Level	Active Storage Volume (m^3)
Woodview Estates	Woodview Estates Subdivision	Dry Forebay & Meadow	Quality (first flush)	Forebay Bypass (weir crest 193.0 m designed)	600
				Outlet structure auxiliary spillway (weir crest 193.95 m)	1,550 (forebay) 1,050 (meadow) 2,600 (total)
Temporary Woodview Estates	Woodview Estates Subdivision	Dry Pond – Single Bay	Quantity	Quality level	2,200
Canaan	Regional – Subcatchments No.'s 9 – 12	Dry Pond – Single Bay	Quality (first flush)	Quality level	1,600
				Spillway level	7,100
Note: Storage volume approximated from footprint of as-constructed infrastructure.					

3.2: RUSH CATCHMENT

3.2.1: OVERVIEW

The Rush catchment is located in the in the northwestern section of the Town of Essex and is generally bounded by Maidstone Avenue (northern limit), the Canadian Southern Railway Line (southeastern limit), and Highway No. 3 (southwestern limit). The Rush catchment's minor system contains trunk sewers constructed along Talbot Street North over four (4) subcatchments which further drain to the Canard River Subwatershed. Subcatchment 1 outlets to the Essex Crossing Pond which is then pumped to the 14th Concession Drain Branch (also referred to as Munch Drain). Subcatchments 2, 3 and 4 outlets directly to the Rush Drain.

3.2.2: LAND USE

The Rush Catchment consists of 115 ha of mostly agricultural or open space land with industrial land concentrated along Forest Ave, and residential lands in the southeastern portion of the catchment. The existing land uses in the catchment are shown in **Table 3.4** and **Figure 3.1** in **Appendix B**.

Table 3.4: Existing Land Uses in the Rush Catchment

Land Use	Area (ha)	Fraction of Total Area (%)
Industrial	14.9	13%
Commercial	7.2	6%
Institutional	4.7	4%
Residential	31.4	27%
Mobile Homes	5.2	5%
Apartments & Hotels	0.0	0%
Parkland	4.5	4%
Open Space or Agricultural	47.1	41%

3.2.3: STORMWATER MANAGEMENT CONTROLS & INFRASTRUCTURE

3.2.3.1: CONVEYANCE CONTROLS

No conveyance controls are installed in this catchment.

3.2.3.2: END-OF PIPE STORMWATER MANAGEMENT FACILITIES

Two (2) ponds are constructed in this catchment: (1) Sadler's Pond and (2) Essex Crossing Pond. The locations of the ponds are shown in **Figure 2.1** in **Appendix B**.

Sadler's Pond receives runoff from Sadler's park and a portion of Viscount Estates in Subcatchment No. 3. Because the pond bottom is not flat some runoff is stored within a permanent pool while the remainder outlets to a ditch. Sadler's Pond was not designed to be used for stormwater management. In practice it provides an outlet for a portion of Viscount Estates and flood control storage when water backs up in the Rush Drain for subcatchments 2, 3, and 4.

The Essex Crossing Pond was designed as a dual bay dry pond system with one (1) bay for normal level quality control and one (1) quantity bay for servicing Subcatchment No. 1 according to the ECA. The first flush was designed to be diverted to the quality bay via a flow diversion weir. The outlet structure of the quality bay contains a 100 mm orifice restricting the release rate to the quantity bay at the quality water level 192.07 m (peak release rate, and drawdown time were not provided in SWM Report). Runoff entering the quantity bay is directed to the Essex Crossing Pumping Station.

Storage for the Essex Crossing dual bay pond system was sized based on not exceeding pre-development release rates with AES Windsor Airport Station No. 6139525 IDF curve, and 6-hour Chicago distribution design storms using PCSWMM 2002 with SWMM4 engine. Details of the quality bay sizing, pond and pump station design are not indicated in the SWM Report prepared by Glos Engineering Ltd., March 2004. The stormwater management pond design parameters are shown in **Table 3.5.**

Table 3.5: Stormwater Management Pond Design Parameters

Pond	Service Area	Pond Design	Quantity or Quality Control	Liquid Level	Active Storage Volume (m^3)
Sadler's Pond	Park Area	N/A	NIL	NIL	NIL
Essex Crossing Pond	Subcatchment No. 1	Dry Pond – dual bay system	Quality & Quantity	Quality level (in quantity bay)	1,100[1]
				Mid-bay berm level (in quantity & quality bay)	3,330[1]
Rush Pond	Town Centre Development	Wet Pond	Quantity	3.17 m	25,000[2]

Notes:
[1] Storage volume approximated from footprint of as-constructed infrastructure
[2] Storage volume based on Dillon Consulting Ltd. As-Built Drawings 2022

The Essex Crossing Pumping Station contains two (2) submersible pumps (one duty, one standby) with firm capacity of 140 L/s at 2.35 m total dynamic head (not one pump rated at 180 L/s at 2.35 m total dynamic head as specified in ECA) controlled via float switches with total maximum capacity of 280 L/s via two 300 mm diameter forcemains to the 14th Concession East Branch Drain (also referred to as Munch Drain).

3.3: ARNER TOWNLINE CATCHMENT

3.3.1: OVERVIEW

The Arner Townline catchment's minor system contains a combination of twinned roadside ditches and sewers constructed along either side of Talbot Street North that discharge to the Gosfield Townline trunk sewer which outlets to West Townline Drain in the Canard River subwatershed.

3.3.2: LAND USE

The Arner Townline Catchment consists of 21 ha of mostly residential land. The existing land uses in the catchment are shown in **Table 3.6** and **Figure 3.1** in **Appendix B**.

Table 3.6: Existing Land Uses in the Arner Townline Catchment

Land Use	Area (ha)	Fraction of Total Area (%)
Commercial	1.5	7%
Institutional	1.0	5%
Residential	15.0	71%
Open Space or Agricultural	3.6	17%

3.3.3: STORMWATER MANAGEMENT CONTROLS & INFRASTRUCTURE

No conveyance controls, stormwater management ponds or municipal pumping stations are installed in this catchment.

3.4: MAIDSTONE CATCHMENT

3.4.1: OVERVIEW

The Maidstone catchment is located in the northeastern section of the Town of Essex and is generally bounded by Maidstone Avenue (northern limit), Fairstone Avenue (southeastern limit), and Talbot Road (southwestern limit). The Maidstone catchment's minor system contains sewers and open channels that ultimately outlet to the Maidstone Ave Drain and further to the Puce River Subwatershed. The Maidstone catchment consists of two major subcatchments which are generally divided along Fairstone Avenue East. Subcatchment 15 contains trunk sewers constructed primarily along north-south streets intercepted via the Maidstone Ave interceptor sewer before its outlet to the Maidstone Ave Drain. Subcatchment 14 contains side street sewer mains that discharge to the Fairview Avenue East trunk sewer before discharging to the McInteer Drain, which outlets to the Maidstone Ave Drain. The catchment's sewer system is hydraulically connected to the South Talbot Catchment via an overflow sewer at Iler Avenue and Talbot Street North and is connected to the Northeast Lagoon sanitary sewer system via a normally closed sluice gate at sanitary Pumping Station No. 4.

3.4.2: LAND USE

The Maidstone Catchment consists of 155 ha of mostly residential land with commercial lands concentrated in the downtown core and agricultural and open space towards the extents of the catchment. The existing land uses in the catchment are shown in **Table 3.7** and **Figure 3.1** in **Appendix B**.

Table 3.7: Existing Land Uses in the Maidstone Catchment

Land Use	Area (ha)	Fraction of Total Area (%)
Industrial	2.2	1.0%
Commercial	14.7	9.0%
Institutional	5.9	4.0%
Residential	93.8	60.0%
Apartments & Hotels	0.5	0.3%
Parkland	1.1	1.0%
Open Space or Agricultural	37.4	24.0%

3.4.3: STORMWATER MANAGEMENT CONTROLS & INFRASTRUCTURE

3.4.3.1: CONVEYANCE CONTROLS & HYDRAULIC STRUCTURES

Conveyance controls and hydraulic structures installed within the catchment are shown in **Table 3.8**.

Table 3.8: Conveyance Controls in the Maidstone Catchment

Hydraulic Structure	Location	Purpose
Orifice – Superpipe Storage	Cranbrook Ct & Arthur Ave	Quantity - Restrict release rate to receiving channel
Flap gate	Galos Pumping Station Bypass Sewer	Mitigate backflow
Flap gate	322 Maidstone Ave E	Mitigate backflow

3.4.3.2: END-OF PIPE STORMWATER MANAGEMENT FACILITIES

One (1) stormwater management pond is constructed in this catchment in the Galos subdivision. The Galos Pond ultimate design consists of a dual bay dry pond system with one (1) bay for basic level quality control and one (1) quantity bay. The first flush is diverted to the quality bay. Runoff exceeding the quality level (HGL=192.80 m) is then split between the quality and quantity bays. The outlet structure of the quality bay contains approx. a 63x63 mm orifice (based on field inspection, not 75mm diameter as per the ECA) restricting the quality bay peak release rate to approx. 10L/s over a 36hr drawdown at the quality water level (192.80 m or 2.0 m water depth) to the Galos Pumping Station. Runoff entering the quantity bay is directed around the perimeter of the bay to the Galos Pumping Station.

Storage for the Galos dual bay pond system was sized based on not exceeding 2-year pre-development release rate with 100-year AES Windsor Airport Station No. 6139525 IDF curve using the Modified Rational Method (C=0.4), and quality bay sized per MECP design guidelines. The stormwater management pond design parameters are shown in **Table 3.9**. The Galos subdivision and Galos stormwater management pond system was expanded in 2017.

Table 3.9: Stormwater Management Pond Design Parameters in the Maidstone Catchment

Pond	Service Area	Pond Design	Quantity or Quality Control	Liquid Level	Active Storage Volume (m^3)
Galos	Galos Subdivision	Dry pond – dual bay system	Quality & Quantity	Quality level (2.0m)	1,250[1]
				Quantity level (2.7m)	2,960[1]
Galos Pond Expansion	Galos Subdivision	Dry pond – thru bay system with diversion berm	Quality & Quantity	Quality level (2.3m)	6,600[2]
				Quantity level (3.05m)	13,800[2]

Notes:
[1] Volume from Galos Development SWM Report prepared by Dillon Consulting Ltd.; amended June 2006 & Record Drawings
[2] Volume from Dillon Consulting Ltd. As-Built Drawings 2019

The Galos Pumping Station contains one (1) submersible pump rated with firm capacity of 47.3 L/s at 5.5 m total dynamic head controlled via level sensor capable of discharging the allowable release rate of 134 L/s (at the 100-year storm event) via a 150 mm diameter forcemain and 300 mm diameter gravity overflow pipe to the Maidstone Ave trunk sewer. The pumping station has provision for bypassing flow when the hydraulic grade line reaches 193.05m.

3.5: HOPGOOD CATCHMENT

3.5.1: OVERVIEW

The Hopgood catchment is located in the in the north and west section of the Town of Essex and is generally bounded by the property lines on either side of Hopgood Sideroad and Talbot Road (north of Maidstone Avenue). Hopgood catchment's minor system contains twin sewer mains constructed along either side of Talbot Street North that discharge to the Hopgood trunk sewer which outlets to Hopgood Drain in the Puce Subwatershed. The catchment's sewer system is hydraulically connected to the Pike Creek Subwatershed via an overflow pipe at 358 Talbot Street North.

During small storm events the Hopgood Drain serves as the primary receiver, however during significant storm events most flow outlets to the Hopgood Drain and a portion overflow to the South Talbot Rd Drain.

3.5.2: LAND USE

The Hopgood Catchment consists of 40 ha of mostly commercial lands located along Talbot Street North and residential lands along Hopgood Side Road with some agricultural and open space land that drains into the catchment's sewer system. The existing land uses in the catchment are shown in **Table 3.10** and **Figure 3.1** in **Appendix B**.

Table 3.10: Existing Land Uses in the Hopgood Catchment

Land Use	Area (ha)	Fraction of Total Area (%)
Industrial	0.0	0%
Commercial	10.0	25%
Institutional	2.1	5%
Residential	15.5	39%
Apartments & Hotels	1.0	2%
Parkland	1.6	4%
Open Space or Agricultural	10.0	25%

3.5.3: Stormwater Management Controls & Infrastructure

No conveyance controls, stormwater management ponds or municipal pumping stations are installed in this catchment.

4.0: APPROACH AND METHODOLOGY

4.1: MODEL APPROACH

The general approach for the improvement of the hydrologic-hydraulic model was to update the existing model to incorporate stormwater drainage infrastructure improvements and land use changes that occurred since April 2016. This would be done to compute responses throughout the network, calibrate portions of the system, and assess the hydraulic capacity based on the updated level of development in the Town Centre.

A full site plan of the Town of Essex's stormwater drainage system in the model is shown in **Figure 4.1(a)** and a partial site plan showing the urban portion of Essex's model is shown in **Figure 4.1 (b)** in **Appendix B**. The major and minor systems were updated based on the construction record drawings for historic and ongoing storm sewer improvement projects and development projects listed in **Section 2.2:**. The analysis was performed using PCSWMM 2022 Professional 2D software version 7.5.3406. PCSWMM utilizes the USEPA SWMM5 engine (5.1.015).

4.2: MODEL INPUT DEVELOPMENT

4.2.1: OUTFALL BOUNDARY CONDITIONS

With the relatively flat topography in Essex County, it is common that drain levels will create a backwater condition on storm sewer system's that reduce the sewer outflow to some undetermined amount and for some undetermined period of time. Near the Essex County shoreline there are records of historical lake levels and near major inland watercourses Ministry of Natural Resources and Forestry (MNRF) floodline maps can be used as reliable outfall boundary conditions to simulate a backwater effect on stormwater drainage systems. In other areas of the County that outlet to smaller watercourses, there is no available gauged data or agreed upon standard for setting backwater conditions for a drainage system hydraulic analysis.

The historical lake levels and the floodline maps, which have floodlines at lower elevations than Essex's outfall sewer inverts, were too far downstream from Essex's sewer system to be used as outfall boundary conditions for the Rush, Maidstone, Hopgood, and Arner Townline catchments. Where backwater conditions are expected, a prudent and practical approach is to assume a constant high backwater level which effectively reduces sewer system release rates and/or raises HGL. With the exception of the South Talbot catchment, the criteria for setting the fixed outfall boundary condition were to set the fixed stage 0.3m (1') below the soffit or overt of the culvert crossing nearest the sewer outfall (similar to MTO culvert clearance standard WC-7), or where no nearby open channel culvert crossing exists, the fixed outfall stage was set at the outfall sewer obvert. **Table 4-7** show the outfall boundary conditions for performing the calibration and capacity assessment.

Table 4.1: Outfall Boundary Conditions

No.	Catchment	Receiving Watercourse	Outfall Fixed Stage Boundary Condition	Outfall Fixed Stage (m)
1	Hopgood	Hopgood Drain	Outfall sewer overt	192.67
2		South Talbot Rd Drain	Culvert crossing at 478 Talbot Rd, Lakeshore	194.10
3	Maidstone	Maidstone Ave Drain	Culvert crossing over Maidstone Drain at N Talbot Rd	192.80
4	South Talbot	Essex Outlet Drain at Mole Rd culvert crossing to Canard River outlet	100yr floodline (Dwg. ER5-38)	189.20
5		John’s Creek Drain at outlet to Craig’s Creek	100yr floodline (Dwg. ER5-38)	188.00
6	Rush	14th Concession E Branch Drain (Munch Drain)	Outfall pipe (900mm dia.) overt from Essex Crossing P.S.	193.60
7		Rush Drain	Hwy 3 Culvert crossing	192.76
8	Arner	West Townline Drain	Outfall sewer overt	194.04

4.2.2: HYDROLOGICAL ANALYSIS

Runoff was computed using the SWMM method. Each subcatchment is treated as a reservoir where inflows are from rainfall and any upstream subcatchments and outflows are from runoff. Surface runoff occurs only when the depth of water in the subcatchment exceeds the rainfall losses in which case the outflow is calculated by the Manning's equation and routed as overland flow to the minor system.

4.2.2.1: RUNOFF COMPUTATIONS

Subcatchment parameters are summarized in **Table 4.2** and vary based on land use characteristics. The SWMM non-linear reservoir runoff method was used to compute runoff in urban lands. The SWMM runoff method produce hydrographs with a steeper peak and recession limb that is prevalent in urban watersheds. SWMM’s runoff method with aquifer routine produces hydrographs with a time-delayed peak and a shallower recession limb that is prevalent in rural watersheds with flat topographies and slower field tile drainage.

Table 4.2: Subcatchment Parameters

Parameter	Equation/Value
Overland flow	Manning’s equation
Subcatchment slope (%) *	0.3% - agricultural, open space, parkland 1.0% - urban developed

Parameter	Equation/Value
Overland Flow Length	As measured
Manning's roughness coefficient (n) for overland flow	0.011 – pavement 0.06 – cultivated soils 0.24 – grass
Imperviousness (%)	From 2023 Aerial Photograph
Subcatchment subarea routing * – % of runoff routed from impervious to pervious surface (accounts for nDCIA & DCIA)	Residential – 50% Institutional, Commercial, Industrial – 25% Downtown (imperviousness > 70%) – 10% Open space, Agricultural – 95% Parkland – 100% Pavement – route to outlet
Land Uses with Aquifer Computations	Rural, Parks, Open Space, Agricultural, and Areas Serviced by Small Tiles
Notes: * Indicates parameter was calibrated. Refer to final calibrated parameters in **Appendix A.2**	

4.2.2.2: RAINFALL LOSSES (INFILTRATION + INITIAL ABSTRACTION)

Infiltration capacity depends largely on soil type, initial moisture content, and surface vegetation. The Modified Green-Ampt method was utilized to compute infiltration loss, with defined soil parameters as follows: Saturated Hydraulic Conductivity (Ks) = 180 mm/hr; Capillary Suction (ψ) = 1.3 mm; and Initial Moisture Deficit (Md) = 0.21. These infiltration parameters were calibrated. Refer to final calibrated parameters in **Appendix A.2.**

Initial abstractions include depression storage and evapotranspiration are deducted from rainfall to compute the runoff entering the minor system. Depression storage is the surface storage provided by ponding, and interception. **Table 4.3** summarizes the values for depression storage. Evapotranspiration was not analyzed because it is typically negligent when analyzing single-storm events (short-term simulation).

Table 4.3: Depression Storage Parameters

Surface	Depression Storage Depth (mm)
Impervious surfaces – roads, pavement	3.5
Pervious surfaces – urban	7.5
Pervious surfaces – rural, parks, open space, ag-lands	10.0
Notes: *Depression storage parameters were calibrated. Refer to final calibrated parameters in **Appendix A.2**	

4.2.3: HYDRAULIC ANALYSIS

4.2.3.1: FLOW ROUTING

Runoff that enters the minor system flows within conduits (open channels or sewers). Flow in conduits was computed as unsteady state flow using Dynamic Wave routing, which solves the complete Saint Venant flow equations. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes. Other hydraulic methods of analysis and hydraulic parameters are shown in

Table 4.4.

Table 4.4: Hydraulic Flow Routing Parameters and Methods of Analysis

Item	Equation/Parameter
Flow routing	Dynamic wave with keep inertial terms
Open channel flow	Manning's equation
Manning's roughness coefficient (n) for pipes	0.013 – smooth interior wall 0.024 – corrugated interior wall
Manning's roughness coefficient (n) for open channels	0.013 – concrete lined 0.035 – lined with vegetation 0.1 – unmaintained
Manning's roughness coefficient (n) for overland flow	0.011 – pavement 0.06 – cultivated soils 0.24 – grass
Pressurized pipe flow	Hazen Williams equation
Hazen Williams roughness coefficient (C)	120

4.2.3.2: HYDRAULIC STRUCTURES

Table 4.5 lists parameters and methods of analysis for analyzing flow rates through existing weirs and orifices, and head loss across flap gates.

Table 4.5: Hydraulic Structure Parameters and Methods of Analysis

Hydraulic Structure	Coefficient	Equation
Orifice	discharge coefficient (C) = 0.65	Refer to SWMM User's Manual V5.0
Weir	discharge coefficient (C_w) = 1.6	Refer to SWMM User's Manual V5.0
Flap gate	minor loss coefficient ($K_{L\ exit}$) = 0.5	$h_L = K_L v^2/2g$

4.3: CAPACITY ASSESSMENT METHODOLOGY

The existing stormwater drainage system was assessed with storms ranging in return periods from 2-years to 100-years to identify capacity constraints under current conditions and land uses. **Table 4.6** summarizes the storms selected with storm duration, rainfall amounts, and rainfall distribution. The design storm total rainfall amounts are based on Atmospheric Environment Services (AES) rainfall data for Windsor Airport Station No. 6139525. The Windsor Airport Station was selected over the Harrow station because it offers a more robust historical dataset.

The design storms used to evaluate the capacity of the existing system included:

- Minor storm event hydraulic grade line: 2-year and 5-year synthetic rainfall events
- Moderate storm event hydraulic grade line: 10-year, 25-year, and 50-year synthetic and historical rainfall events
- Extreme storm event hydraulic grade line: 100-year synthetic and Urban Stress Test

There is potential for surface flooding if the surcharge level in the storm sewer exceeded ground level elevation.

Table 4.6: Design Storms Utilized for Simulating Wet Weather Capacity

Storm Frequency	Duration	Total Rainfall	Distribution	Source
2 Year	12 hours	46.2 mm	AES	Synthetic
5 Year	12 hours	60.1 mm	AES	Synthetic
10 Year	12 hours	69.2 mm	AES	Synthetic
25 Year	12 hours	80.4 mm	AES	Synthetic
25 Year	24 hours	93.7 mm	-	Historic
50 Year	12 hours	89.4 mm	AES	Synthetic
100 Year	12 hours	98.0 mm	AES	Synthetic
Urban Stress Test 100 Year + Uniform Distribution of Additional 42 mm	24 hours	150 mm	Chicago	Synthetic

5.0: FLOW MONITORING AND MODEL CALIBRATION

5.1: RAINFALL AND FLOW MONITORING

Rainfall data was collected at Essex Pollution Control Plant, 4000 Malden Road, during the period of flow monitoring. Parameters recorded at the rain gauge location include the rainfall intensity (mm/hr) and volume (mm).

Flows were monitored at a total of four (4) locations from March 2023 to August 2023. The locations of the rain gauge and flow monitors are shown in **Figure 5.1**. Rainfall and flow monitoring data were used to calibrate the hydrologic-hydraulic drainage system model. The flow monitoring locations were selected in order to calibrate the remaining areas of the Essex Town Centre stormwater drainage area that were not calibrated in the 2019 study. The calibration areas are shown in **Figure 4.1 (c)** of **Appendix B**. The Rush Catchment was not calibrated due to extensive ongoing development in the catchment which would render the calibration obsolete.

Parameters recorded at the flow monitor locations include the flow (L/s), depth (mm), and velocity (m/s). Information related to each monitoring location is presented in **Table 4.1.**

Table 4.1: Flow Monitoring Locations

Flow Monitor No.	Location	Catchment being Monitored	Drainage Area	Impervious Percentage
1	155 South Talbot Road N; CBMH3	South Talbot (West)	22.2 ha	45.7 %
2	360 Fairview Avenue W; CB542	South Talbot (East)	36.2 ha	41.1 %
3	Gosfield Townline (near Lester Drive); MH001d	Arner	19.7 ha	36.4 %
4	321 Fairview Avenue E; MH#1	Maidstone	47.3 ha	41.7 %

5.2: CALIBRATION EVENTS

Five (5) rainfall events in 2023 were selected for use in the calibration process. Rainfall events start and stop dates were selected so that flows before and after storm events returned to dry weather flow conditions.

Table 5.2 presents the rainfall statistics for the selected rainfall events.

Table 5.2: **Rainfall Events Selected for Model Calibration**

Event No.	Year	Simulated Rainfall Event (Start – End)	Duration (hr)	Total Rainfall Volume (mm)	Peak Rainfall Intensity (mm/hr)
1	2023	June 26 10:00 – June 26 21:00	11.0	24.5	36
2	2023	July 20 12:30 – July 20 19:30	7.0	25.8	33
3	2023	July 26 13:00 – July 27 00:30	11.5	24.5	57
4	2023	August 14 17:00 – August 15 05:30	12.5	28.5	48
5	2023	August 17 12:00 – August 17 22:30	10.5	17.3	21

Data collected for these rainfall events were compared against IDF curves from Environment Canada's Harrow CDA Auto rain gauge (Station No. 6133362, v3.0) to determine the return period of each rainfall event based on event volume and peak intensity in order to quantify the magnitude of these storm events. The term 'return period' is used to quantify the magnitude or rarity of a storm event. The storm return period is the reciprocal of the exceedance probability, which is the probability that a storm event is equaled or exceeded in any given year. As the storm return period increases the magnitude or rarity of the storm increases as well. For example, storms with 2-year and 25-year return periods have a probability of 50% and 4% of being equaled to or exceeded in any given year respectively.

Table 5.3 presents a summary of the return period for the selected calibration events, of which the majority are less than a 2-yr storm, except for the event July 26, 2023, that was the largest recorded intensity. From the available flow monitoring results, the events with the lowest frequency were selected for the calibration. Since a majority of the events had a return frequency of 2-yrs or less, there is increased confidence in the ability to represent frequent events. Calibrating the model with events with lower frequency (for example, a 1 in 10-yr or greater) would be ideal; however, no such storm events occurred during the monitoring period. Model response for design storms with frequency greater than the 2-yr event were extrapolated from the calibrated parameters, with less confidence in results for increasingly rare events. Nonetheless, the data from the 2019 and 2023 monitoring campaigns represent the best available information from which to base decision-making.

Table 5.3: Estimated Rainfall Event Return Periods

Event No.	Rainfall Event	Estimated Return Period (yrs)
1	June 26 10:00 – June 26 21:00	**< 2 yr**
2	July 20 12:30 – July 20 19:30	**< 2 yr**
3	July 26 13:00 – July 27 00:30	**2 yr**
4	August 14 17:00 – August 15 05:30	**< 2 yr**
5	August 17 12:00 – August 17 22:30	**< 2 yr**

5.3: MODEL CALIBRATION

During model calibration, rainfall from the selected calibration events (**Table 5.2**) were used as model input, and the observed flow and depth data at the locations listed in **Table 4.1** were compared with the model simulation results. The following calibration metrics in order of precedence were used to calibrate simulated against observed data:

- Depth Hydrograph Shape
- Peak Depth
- Flow Rate Hydrograph Shape
- Peak Flow Rates
- Volume

The following subcatchment parameters were selected for the calibration process: infiltration parameters, slope, depression storage, and subarea routing. These parameters were adjusted as required until the simulated results reasonably matched the observed flow monitor data for all events. These parameters were selected to match those of the previous model calibration work and their ability to alter the hydrograph shape and peak values.

The calibrated hydrographs are shown graphically in **Appendix A.1** and a table outlining the observed and predicted volume (m^3), peak depth (m), and peak flow (L/s) are available in **Appendix A.2**. **Table 5.4** summarizes the adjustments made during model calibration and calibrated parameters are shown in **Appendix A.2**.

Table 5.4: Summary of Adjustments Made During Model Calibration

Flow Monitor	Catchment	Initial Calibration Comments*	Action (Parameters that were adjusted)
1	South Talbot (West)	Peak flow overestimated Volume overestimated & Depth underestimated	• D_{store}-perv= 8.5 mm • D_{store}-imperv= 3.5 mm • Slope (urban/developed) = 0.3 • Infiltration - k = 3 mm/hr - M_d= 0.3 - Ψ= 110 mm • Subarea routing refer to **Appendix A.2**
2	South Talbot (East)	Peak flow overestimated & Depth underestimated	• D_{store}-perv= 7.5 mm • D_{store}-imperv= 2.5 mm • Slope (urban/developed) = 0.5 • Slope (undeveloped) = 0.3 • Infiltration - k = 3.9 mm/hr

			- M_d= 0.3 - Ψ= 270 mm • Subarea routing refer to **Appendix A.2**
3	Arner	Peak flow underestimated Volume overestimated & Depth overestimated	• D_{store}-perv= 7.5 mm • D_{store}-imperv= 3.5 mm • Slope (urban/developed) = 1 • Slope (undeveloped) = 0.3 • Infiltration - k = 2.6 mm/hr - M_d= 0.25 - Ψ= 230 mm • Subarea routing refer to **Appendix A.2**
4	Maidstone	Peak flow overestimated Volume underestimated & Depth underestimated	• D_{store}-perv (urban/developed)= 7.5 mm • D_{store}-perv (undeveloped) = 10 mm • D_{store}-imperv= 2.5 mm • Slope (urban/developed) = 0.5 • Slope (undeveloped) = 0.15 • Infiltration - M_d= 0.3 • Subarea routing refer to **Appendix A.2**

In addition, where the depth of the hydrograph could not be reasonably matched through the selected calibration parameters the boundary conditions were modified. The boundary conditions for the South Talbot (East) Catchment (near FM No. 2) and the Maidstone Catchment (near FM No. 4) were modified to simulate standing water conditions measured by the flow monitoring devices. For the FM No. 2 area, the Fairview Avenue West sewer at its outlet to the municipal drain was modified to simulate standing water of 0.375 m. For the FM No. 4 area, the Fairview Avenue sewer at its outlet to the municipal drain, near the roundabout) was modified to simulate standing water of 0.250 m.

5.4: MODEL CONFIDENCE AND LIMITATIONS

It is noted that many of the observed depth hydrographs have long-shallow recession limbs, while the calibrated depth hydrographs have steeper recession limbs. This is believed to be due to the slowly draining agricultural lands and poor outlet conditions within the subwatershed that create a backwater condition on the sewer system. These poor outlet conditions and backwater conditions are a known problem in the Town of Essex and are supported by field investigations and residents' complaints. Although the calibrated hydrographs have steeper recession limbs the predicted peak values and overall shape of the hydrographs were considered to be a reasonable representation of the observed data. A majority of the predicted peak depths falling within a ± 25% deviation of the observed data. Therefore, the model is believed to be suitable for the capacity assessment because the primary objective is computing peak HGL.

Throughout the calibration process precedence was given to matching the peak depth and depth hydrograph shape against the observed data. The precedence was given to the depth calibration metrics as this variable is the more reliable of the flow monitoring program measurements and the focus of the study was to address surcharging and flooding concerns. In general, the priority for the depth variables has resulted in an overestimation of the peak flows throughout the model. The overestimation of the peak flow values was considered acceptable for the purpose of this study as the calibration would result in conservative estimation of pipe capacity and conservative sizing of proposed pipe upgrades.

Since the model was calibrated to achieve a best fit over a variety of storm events during the 2023 monitoring campaign, it is deemed suitable for capacity assessments and capital upgrade planning. Given the events captured in 2023 only achieved a 2-yr return frequency, there is increased confidence in the ability to represent frequent events. Model response from design storms with frequencies beyond the 2-yr event were necessarily extrapolated from the calibrated parameters, with less confidence in results for increasingly rare events. Nonetheless, the data from the monitoring program represents the best available information from which to base decision-making.

6.0: CAPACITY ASSESSMENT OF EXISTING STORM SYSTEM

As discussed in **Section 1.0 – Introduction**, the objective of this study is to build upon the findings of the 2019 storm sewer study of the Essex Town Centre and develop planning level conceptual designs in the remaining four (4) catchment areas (Maidstone, Rush, Hopgood, and Arner Townline). The following sections of this report outline the findings of the capacity assessment and identify areas of concern in the Essex Town Centre which will be considered to make preliminary recommendations for improvements to the stormwater drainage system. In addition, the findings of the South Talbot Catchment capacity assessment and recommendations provided in the Ward 1 Southwest Storm Sewer System Environmental Study Report are outlined in the following sections.

The capacity assessment was conducted for the existing storm sewer system based on the potential for surcharging and surface level flooding during design storm events. For more details regarding the design and historical storm events, refer to **Section 4.5**. To visualize the hydraulic capacity of the existing storm sewer system a figure was produced for each of the storm events. A colour code was used for the junction icons to represent the depth of water in the pipe and potential flooding or surcharging. Surcharging was defined as when the water level rises above pipe crown (top of pipe) but not above the grade elevation. Flooding was defined as when the water level rises above grade elevation and ponds onto the ground surface.

The figures depicting the hydraulic capacity are available in **Appendix B** and correspond to the following storms: 1 in 2-year Design Storm – **Figure 6.1**, 1 in 5-year Design Storm – **Figure 6.2**, 1 in 10-year Design Storm – **Figure 6.3**, 1 in 25-year Design Storm – **Figure 6.4**, 1 in 25-year Historic Storm – **Figure 6.5**, 1 in 50-year Design Storm – **Figure 6.6**, 1 in 100-year Design Storm – **Figure 6.7**, and Urban Stress Test Design Storm – **Figure 6.8**.

The Town's storm sewer system was designed to convey flows from 2-year or 5-year storms which was consistent with development in other municipalities at the time of construction. However, the existing storm sewers and municipal drainage systems cannot handle extreme rainfall events that have been experienced in recent years. During heavy rainfall events, significant amounts of stormwater pool on Town streets and runoff from urban areas significantly increase the volume of stormwater entering the storm sewer system resulting in flooding.

The Windsor/Essex Region Stormwater Management Standards Manual determined that stormwater infrastructure should be design, constructed, and evaluated based on a minor storm event and a major storm event. Minor and major storm events used in this study are the 1 in 5-year AES design storm and the 1 in 100-year AES design storm, respectively. Ideally, the hydraulic grade line would always be maintained below basement elevations; however, this is impractical in most of the Windsor/Essex region due to limited gradient and the shallow sewer installations that are required to preserve fall. A typical acceptable level of service in this region requires that for the minor storm the system HGL be maintained below ground elevations (i.e., no surface storage) and for the major storm event that the HGL be maintained such that the maximum surface level ponding depth is 0.3 meters. Areas that were identified as having limited hydraulic capacity for these design storm events are discussed below.

6.1: SOUTH TALBOT CATCHMENT

As shown in **Figure 6.2** of **Appendix B**, a majority of the system meets the servicing requirements for the minor storm (i.e., no ponding). However, there are sections of the system which do not meet the servicing requirements and are at risk of surface ponding / flooding during the 1 in 5-year storm event. As shown in **Figure 6.7** of **Appendix B**, the upper portions of the system do not meet the servicing requirements for the major storm (i.e., ponding greater than 0.3 m).

The Essex Outlet Drain and Canaan Outlet Drain were identified to have limited hydraulic capacity during severe storm events. As a result of this limited hydraulic capacity surface level ponding upstream of these drains were predicted in the model.

6.2: RUSH CATCHMENT

As shown in **Figure 6.2** of **Appendix B**, a majority of the system meets the servicing requirements for the minor storm (i.e., no ponding). However, there are sections of the system which do not meet the servicing requirements and are at risk of surface ponding / flooding during the 1 in 5-year storm event. The areas of concern are focused on the following streets: Brock Street, Thomas Street, College Street and Harvey Street.

As shown in **Figure 6.7** of **Appendix B**, the farthest upstream portions of the system do not meet the servicing requirements for the major storm (i.e., ponding greater than 0.3 m). These areas are mainly located in the residential neighbourhoods located north and south of Maidstone Avenue.

The trunk sewer on Maidstone Avenue (from Talbot Street North to the connection at the Rush Drain) was identified to have limited hydraulic capacity during severe storm events. As a result of this limited hydraulic capacity surface level ponding upstream of this trunk sewer and drain was predicted in the model. In addition, the local sewers within these residential areas are primarily tile drainage that are likely in poor condition and not adequately sized for modern storm events. Solutions to address these issues will be identified and evaluated in the following sections of the report.

6.3: ARNER TOWNLINE CATCHMENT

As shown in **Figure 6.2** of **Appendix B**, the system meets the servicing requirements for the minor storm (i.e., no ponding). As shown in **Figure 6.7** of **Appendix B**, the system meets the servicing requirements for the major storm (i.e., ponding greater than 0.3 m).

6.4: MAIDSTONE CATCHMENT

As shown in **Figure 6.2** of **Appendix B**, a majority of the system meets the servicing requirements for the minor storm (i.e., no ponding). However, there are sections of the system which do not meet the servicing requirements and are at risk of surface ponding / flooding during the 1 in 5-year storm event. The areas of concern are focused on the following streets: Medora Avenue, Cameron Avenue, Brien Avenue, and the northern portion of Gosfield Townline.

As shown in **Figure 6.7** of **Appendix B**, portions of the system do not meet the servicing requirements for the major storm (i.e., ponding greater than 0.3 m).

The trunk sewer along Maidstone Avenue (from Gosfield Townline to the 6th Concession Outlet) was identified to have limited hydraulic capacity during severe storm events. As a result of this limited hydraulic capacity surface level ponding upstream of this trunk sewer and drain was predicted in the model. Solutions to address these issues will be identified and evaluated in the following sections of the report.

6.5: HOPGOOD CATCHMENT

As shown in **Figure 6.2** of **Appendix B**, a majority of the system meets the servicing requirements for the minor storm (i.e., no ponding). As shown in **Figure 6.7** of **Appendix B**, a majority of the system meets the servicing requirements for the major storm (i.e., ponding greater than 0.3 m).

There is a section at the upper reaches of the system which are at risk of surface ponding / flooding during the storm events; however, this does not pose an issue as this land is undeveloped. This flooding is due to large volumes of stormwater runoff which are contributed from the undeveloped agricultural lands. When development occurs in this area, stormwater sewers and controls should be implemented.

7.0: ALTERNATIVE CONCEPTUAL DESIGN SOLUTIONS

7.1: INTRODUCTION

Based on the capacity assessment outlined in Section 5.0, several conceptual solutions may be implemented to address the areas of concern and improve the storm sewer system to the required level of service. The following broad planning level alternative solutions have been considered for providing adequate service for the stormwater system in the Ward 1 Southwest area of the Town of Essex:

Alternative No. 1 – Runoff Reduction Through Low Impact Development (LID) and Green Infrastructure
Alternative No. 2 – Field Inspection and Maintenance of Existing Storm Sewers
Alternative No. 3 – Increasing Hydraulic Capacity of Existing Storm Sewers
Alternative No. 4 – Construction of Stormwater Management Ponds
Alternative No. 5 – Combination of the Above Alternatives

7.2: EVALUATION CRITERIA

The criteria used to develop the alternatives were based on generally accepted principles and previous experience. The criteria included the following:

- Application of current engineering practices and standards;
- Adherence to applicable laws and regulations;
- Economic considerations;
- Operation and maintenance issues;
- Health and safety;
- Acceptability to concerned stakeholders; and
- Feasibility of implementation.

The alternative solutions were evaluated based on a variety of social, natural environmental, economic, and technical criteria. These evaluation criteria were developed based on servicing needs, applicable municipal plans / commitments, design principles, and past industrial experience. The evaluation criteria are as follows:

Technical Criteria:

- Ability to meet current and future servicing needs;
- Constructability, implementation timeline, and phasing;
- Flexibility to meet future needs and/or climate change projections; and
- No adverse impacts on existing infrastructure (operations and/or maintenance).

Social Criteria:

- Impacts to archaeological sites or areas of archaeological potential;
- Impacts to known or potential built heritage resources and cultural heritage landscapes;

- Noise, vibration, odour, or air pollution emissions;
- Permanent changes or impacts to society / community; and
- Development policies and agreements.

Natural Environmental Criteria:

- Impacts to vegetation, fish and wildlife, areas of natural and scientific interest, environmentally sensitive areas, and soil / geology;
- Regulatory compliances; and
- Development and planning policies.

Economic Criteria:

- Capital, operational and maintenance (O&M) costs.

The advantages and disadvantages of each alternative together with their effects on the social, economic, and natural environment are discussed in the following sections.

7.3: DESCRIPTION AND EVALUATION OF ALTERNATIVES

7.3.1: ALTERNATIVE NO. 1 – RUNOFF REDUCTION

Under this strategy, LID and green infrastructure would be implemented in order to reduce the volume and / or rate of stormwater runoff entering the stormwater collection system. Reducing the volume and rate of stormwater entering the collection system works to mitigate the risk of sewer back ups, flooding, and property damage. These technologies mimic natural processes and manage runoff as close to the source as possible by providing storage followed by infiltration, evapotranspiration, and/or use of stormwater to runoff volume and/or runoff rates. Examples of LIDs include rain gardens, bioswales, infiltration trenches, permeable pavement, rainwater harvesting, etc. The implementation of LIDs and other green infrastructure is becoming increasingly popular worldwide.

In terms of technical suitability, these technologies rely primarily on infiltration to reduce the volume of runoff in the water cycle. The predominant soil stratum in the Town of Essex is one foot of topsoil overlying poorly draining clays. These soil conditions will result in poor infiltration and inability to reduce the volume of runoff to the stormwater collection system. The cost to implement LIDs and other green infrastructure would be significant and the systems would not provide adequate retention or storage of stormwater. Therefore, Alternative No. 1 – Runoff Reduction through Low Impact Development (LID) and Green Infrastructure is not considered a viable alternative.

7.3.2: ALTERNATIVE NO. 2 – FIELD INSPECTION AND MAINTENANCE

Under this strategy, field inspections and maintenance of storm sewers and drains would be carried out to reduce bottlenecks in the system and allow for effective drainage during storm events. Field inspections should be carried out at all outfalls to municipal drains, intersections of trunk sewers, and stormwater management controls / ponds. Maintenance at these locations would include cleaning any debris and removing accumulated sediment which may be inhibiting flow. Reducing bottlenecks in the stormwater collection system works to mitigate the risk of sewer back ups, flooding, and property damage.

In terms of technical suitability, the hydrologic-hydraulic drainage model indicated that removal or debris and sediment is not expected to significantly improve the storm systems level of service in the areas of concern. Although this alternative does not provide an all-inclusive solution to address the servicing issues, maintenance is an important part of upkeeping an efficient and effective stormwater system. Therefore, Alternative No. 2 – Field Inspection and Maintenance is not considered a viable alternative on its own but should be included as a part of the preferred solution.

7.3.3: Alternative No. 3 – Increasing Hydraulic Capacity of Sewers and Drains

Under this strategy, the hydraulic capacity of local sewers, trunk sewers, and municipal drains may be improved to increase the level of service, reduce bottlenecks in the system, and allow for effective drainage during storm events. The hydrologic-hydraulic drainage model will be used to identify sewers and drains that have limited hydraulic capacity and make recommendations for improvements. Improvements may include replacing existing sewers with larger more adequately sized piping or deepening/widening municipal drains.

In terms of technical suitability, this alternative could be implemented throughout the Town to meet current and future servicing needs. The replacement of stormwater sewers and improvement to municipal drains is considered a proven method with relatively easy constructability (dependant on the size of sewer/drain improvements). The implementation timeline and phasing of these improvements may be aligned with other municipal infrastructure works in an effort to minimize construction costs. The potential impact of the recommended improvements on existing upstream and downstream infrastructure will be evaluated in the hydrologic-hydraulic model.

In terms of the social and natural environmental activity, the design and implementation of these improvements would conform with the municipal and regional development policies, agreements, and regulatory compliances. Drain and sewer improvements would mostly be like-for-like sewer replacement projects within or adjacent to the existing roadways. These areas have recent extensive and intensive disturbances due to infrastructure developments including sanitary sewer lines, water lines, stormwater lines, gas lines, and roads. Therefore, the improvement projects are anticipated to have low potential for impact to archaeological resources, built heritage resources, and/or cultural heritage landscapes. Further, the improvement projects are anticipated to have low potential for negative impact to vegetation, fish and wildlife, areas of natural and scientific interest, environmentally sensitive areas, and soil / geology. Although sewer improvements may pose an inconvenience during construction, it is anticipated that these projects will have a positive impact on the community as they will reduce the risk of flooding.

In terms of the economic impacts, this alternative would be economically effective with a low to moderate capital cost investment and minimal O&M costs. The opinion of probable cost for these improvements are outlined in **Section 7.4**.

In consideration of the factors discussed above, Alternative No. 3 – Increasing Hydraulic Capacity of Sewers and Drains is considered a viable solution to address the stormwater management needs in the Town of Essex.

7.3.4: Alternative No. 4 – Construction of Stormwater Management Ponds

Under this strategy, stormwater management ponds would be implemented to store runoff during major storm events and release it at a controlled rate following these events preventing flooding. Stormwater management ponds are engineered infrastructure that is designed to store stormwater, provide flooding control, prevent erosion in municipal drains, and enhance water quality. Stormwater management ponds can be implemented on a local or regional scale to enhance service in large branches of the stormwater collection system.

In terms of technical suitability, this alternative could be implemented throughout the Town to meet current and future servicing needs. The implementation of stormwater management ponds is considered a proven and robust method with relatively easy constructability (dependant on site size, location, technical requirements). The implementation timeline and phasing of these improvements may be aligned with other municipal infrastructure works in an effort to minimize construction costs. The potential impact of the recommended improvements on existing upstream and downstream infrastructure will be evaluated in the hydrologic-hydraulic model.

In terms of the social and natural environmental activity, the design and implementation of stormwater management ponds would conform with the municipal and regional development policies, agreements, and regulatory compliances. The sites for the proposed ponds should be evaluated for potential for impact to archaeological resources, built heritage resources, and/or cultural heritage landscapes early in the planning or design process. The improvement projects are anticipated to have low potential for negative impact to vegetation, fish and wildlife, areas of natural and scientific interest, environmentally sensitive areas, and soil / geology. Although sewer improvements may pose an inconvenience during construction, it is anticipated that these projects will have a positive impact on the community as they will reduce the risk of flooding. Noise and vibration from associated pumping stations (if applicable) or odours from the proposed ponds during operation is expected to be minimal. Early in the planning and design process for the proposed ponds applicable noise, vibration, and/or odour studies should be carried out to mitigate off-site impacts to an acceptable level.

In terms of the economic impacts, this alternative would be economically effective with a moderate capital cost investment and low to moderate O&M costs. The opinion of probable cost for these improvements are outlined in **Section 7.4**.

In consideration of the factors discussed above, Alternative No. 4 – Construction of Stormwater Management Ponds is considered a viable solution to address the stormwater management needs in the Town of Essex.

7.3.5: Alternative No. 5 – Combination of the Above Alternatives

The combination of the above alternatives is considered the most viable solution to address the stormwater management needs in the Town of Essex. Under this strategy, a combination of Alternative No.'s 2 (Field Inspection and Maintenance), 3 (Increasing Hydraulic Capacity of Sewers and Drains), and 4 (Construction of Stormwater Management Ponds) would be evaluated to address needs in the problem areas identified in **Section 5. 0.** The combination of the above alternatives which form the preferred conceptual design are outlined in **Section 7.0**.

8.0: RECOMMENDED SOLUTIONS

8.1: IMPROVEMENTS OUTLINED IN THE ESSEX SOUTHWEST CENTRE ENVIRONMENTAL STUDY REPORT

The Town of Essex is carried out a municipal class environmental assessment for improvements to the Essex Ward 1 Southwest Storm Sewer System which is focused on South Talbot Catchment Area. This study reports on planning level conceptual designs which would accommodate future development and improve flood protection within the South Talbot catchment area. Improvements to the Ward 1 Southwest Storm Sewer system are outlined in the Environmental Study Report and generally include the following:

- Increasing Hydraulic Capacity of Storm Sewers
    - Replace storm sewer along Centre Street between Hanlan Street and South Talbot Road to a provide proper outlet for service the Centre Street drainage area;
    - Replace storm sewer in Optimist Park between Hanlan Street and Milne Street to provide a proper outlet for servicing the Iler Ave drainage area; and
    - Replace storm sewer along Brien Ave West between Kimball Dr and South Talbot Rd and along South Talbot Rd from Brien Ave West to Fairview Ave West at the outfall.
- Construction of Stormwater Management Ponds
    - Woodview SWMF expansion;
    - Canaan Pond expansion; and
    - New Essex Outlet Pond on the west side of the Essex Pollution Control Plant.

It is assumed that the Town of Essex will complete the recommended upgrades outlined in the Environmental Study Report as budget becomes available. The hydraulic capacity of the collection system with the implementation of the recommended upgrades under the 1 in 100-year Design Storm is depicted in **Figure 8.1** of **Appendix B.**

8.2: STORM SEWER IMPROVEMENTS

8.2.1: RUSH CATCHMENT

In the Rush Catchment area, the trunk sewer on Maidstone Avenue (from Talbot Street North to the connection at the Rush Drain) was identified to have limited hydraulic capacity during severe storm events. To reduce the likelihood of flooding upstream it is recommended that the hydraulic grade and capacity of this trunk sewer be improved.

Figure 8.4 of **Appendix B** shows the existing hydraulic profile and hydraulic grade line in this trunk sewer during the 1 in 100-year storm. Based on the hydrologic-hydraulic model (dynamic flow routing including backwater effect), the recommendations for this trunk sewer are outlined in **Table 8.1**. **Figure 8.5** of **Appendix B** shows the new hydraulic profile and hydraulic grade line in this trunk sewer under the 1 in 100-year storm with proposed upgrades.

Table 8.1: Recommended Sewer Improvements for Rush Catchment

Location	Description of Improvements
Bell Avenue Harvey Avenue to Maidstone Avenue	• Realign (lower invert)
Maidstone Avenue Bell Avenue to 492 Maidstone Avenue	• Realign (lower invert) • Upgrade from 750 mm and 900 mm
Maidstone Avenue 125 Maidstone Avenue to 137 Maidstone Avenue	• Realign (raise invert) • Upgrade from 750 mm and 1050 mm
Maidstone Avenue Under public trail	• Realign (raise invert) • Upgrade from 900 mm and 1050 mm
Local Sewers on Albert Street, Harvey Street, and Brock Street	• Replace in the future when other supplementary infrastructure projects occur

8.2.2: MAIDSTONE CATCHMENT

In the Maidstone Catchment area, the trunk sewer along Maidstone Avenue (from Gosfield Townline to the 6th Concession Outlet) was identified to have limited hydraulic capacity during severe storm events. To reduce the likelihood of flooding upstream it is recommended that the hydraulic grade and capacity of this trunk sewer be improved.

Figure 8.2 of **Appendix B** shows the existing hydraulic profile and hydraulic grade line in this trunk sewer during the 1 in 100-year storm. Based on the hydrologic-hydraulic model (dynamic flow routing including backwater effect), the recommendations for this trunk sewer are outlined in **Table 8.2**. **Figure 8.3** of **Appendix B** shows the new hydraulic profile and hydraulic grade line in this trunk sewer under the 1 in 100-year storm with proposed upgrades.

Table 8.2: Recommended Sewer Improvements for Maidstone Catchment

Location	Description of Improvements
Maidstone Avenue Gosfield Townline to 313 Maidstone Avenue	• Realign (lower invert) • Upgrade from 900 mm to 1350 mm
Maidstone Avenue 313 Maidstone Avenue to 363 Maidstone Avenue	• Realign (lower invert) • Upgrade from 1200 mm and 1350 mm to 1500 mm
Maidstone Avenue	• Upgrade from 1300 mm semi-diameter box culvert to 1650 mm • Upgrade from 1500 mm sewer to 1650 mm

363 Maidstone Avenue to Roundabout (Fairview Ave)	
Maidstone Avenue Roundabout (Fairview Ave) to Proposed Stormwater Management Pond	• Deepen, widen, and realign (lower) inverts of roadside ditch/drain to the proposed stormwater management pond • Deepen and realign of box culverts

8.3: STORMWATER MANAGEMENT PONDS

In addition to the storm sewer improvements listed in the previous sections, two (2) stormwater management ponds are recommended. The exact location and capacity requirements for the proposed ponds should be determined through a planning study or through a detailed design process.

One (1) stormwater management pond is recommended for the Rush Catchment area downstream of Maidstone Avenue and upstream of the 14th Concession Drain. The location of this pond should be refined based on land availability and other social, natural environmental, and economic factors.

Utilization and upgrades to the Sadler's Pond may be implemented to mitigate flooding in the upper reaches of the Rush Catchment area. This would include upgrades such as a new inlet weir and outlet control that is needed to restrict outlet flow to match existing conditions, provide attenuation during significant storm events, and reduce flood risk and property damage within the upper reached of the catchment. Preliminary design criteria for the proposed Rush Stormwater Management Pond are shown in **Table 8.4**. The storage volume available and utilization of the Sadler's Pond should be confirmed through a planning study or the detailed design process.

Table 8.3: Sadler's Stormwater Management Pond – Preliminary Design Criteria

Design Parameter	Sadler's Pond
Pond Design	Wet Pond
Bottom of Pond El.	192.9 m
Permanent Pond El.	193.0 m
Top of Bank El.	194.0 m
Water Level **100yr-12hr AES (1hr Time Step)**	HWL = 193.84 m
Min. Provided Active Storage Volume (m^3)	11,800 m^3
Proposed Dimensions	Existing Pond
Inlet Structure	Weir – 193.7 m
Outlet Structure	Orifice – restricting flow to 55 L/s

One (1) new stormwater management pond is being proposed for the Maidstone Catchment area downstream of the 6th Concession Drain Outlet. The location of this pond should be refined based on land availability and other social, natural environmental, and economic factors.

This regional pond is needed to restrict outlet flow to match existing conditions, provide attenuation during significant storm events, and reduce flood risk and property damage within the catchment. Preliminary design criteria for the proposed Maidstone Stormwater Management Pond are shown in **Table 8.4** and may be refined through a planning study or the detailed design process.

Table 8.4: Maidstone Stormwater Management Pond – Preliminary Design Criteria

Design Parameter	Maidstone Pond
Pond Design	Dry Pond
Bottom of Pond El.	191.35 m OR 189.85 m
Top of Bank El.	193.8 m
Water Level **Weir Crest** **100yr-12hr AES (1hr Time Step)**	Crest = 192.8 m HWL = 193.21 m
Min. Provided Active Storage Volume (m^3)	31,250 m^3
Proposed Dimensions	150 m x 125 m (2 m Depth) OR 120 m x 120 m (3.5 m Depth) Side Slope 1:6
Inlet Structure	Open channel
Outlet Structure	During storm – Gravity Outlet Weir
	Following storm – Pumped Outlet Dewatering Pumping Station Q, TDH, and drawdown TBD in detailed design

The preliminary design criteria presented in this section should be confirmed through a planning study or through a detailed design process.

8.4: IMPACT OF RECOMMENDED IMPROVEMENTS

It is recommended that the Town of Essex complete the recommended upgrades as budget becomes available. The hydraulic capacity of the collection system with the implementation of the recommended upgrades under the 1 in 5-year and 1 in 100-year Design Storm is depicted in **Figure 8.6** and **Figure 8.7**, respectively, of **Appendix B.** As shown in **Figure 8.6**, there is minimal surcharging and no flooding during the 1 in 5-year event which meets the design standards for the Town of Essex. As shown in **Figure 8.7**, there is some flooding, but no ponding greater than 0.3 m during the 1 in 100-year event which meets the design standards for the Town of Essex.

8.5: OPINION OF PROBABLE COST

A capital budget estimate (in 2023 dollars) is summarized in **Table 8.5**. In addition to the level of accuracy discussed, the opinion of probable cost was prepared taking into consideration the following factors.

- All estimates are 2023 Canadian dollars based on an Engineering News Record (ENR) Construction Cost Index of 1200.
- It is assumed that the Contractor will have unrestricted access to the site and will complete the work during normal working hours from 7:00 am to 6:00 pm Monday to Friday. There is no allowance for premium time included. Labour costs are based on union labour rates for the Windsor area. Bulk material and equipment rental costs used are typical for the Windsor area.
- An allowance is included for mobilization and demobilization and the Contractor's overhead and profit.
- The estimate does not include the cost of application or permit fees.
- No allowance is included for interim financing costs or legal costs.
- No allowance is included for escalation beyond the date of this report.
- No allowance is included for property acquisition.
- Allowances for engineering and contingency allowances (approximately 30% and 15%, respectively) are included in the estimate.
- It is not known whether contaminated soil conditions or presence of archaeological resources may be encountered in the areas proposed for the upgrades. The potential impact cannot reasonably be determined at this point and no allowance is included in the estimate.
- Cost for the storm sewer replacement includes stormwater infrastructure and partial road reconstruction.

Table 8.5: Opinion of Probable Capital Cost for Preferred Solution

Item	Description	Probable Cost
1	Maidstone Avenue Trunk Sewer Replacement / Realignment from Gosfield Townline to New Maidstone Pond (Maidstone Catchment)	$ 3,200,000
2	Maidstone Avenue Trunk Sewer Replacement / Realignment from Bell Avenue to Public Trail (Rush Catchment)	$ 1,600,000
3	Maidstone Pond and Dewatering Pumping Station	$ 2,500,000
4	Rush Pond (assuming new unpumped pond)	$ 1,000,000
SUBTOTAL		$ 8,300,000
Contingency Allowance (30%)		$ 2,500,000
Engineering Allowance (15%)		$ 1,250,000
TOTAL CAPITAL COST (excluding taxes)		$ 12,050,000
Note: * The opinion of probable cost for the Maidstone Pond and Dewatering Pumping Station will vary greatly depending on the location of the proposed pond.		

9.0: SUMMARY

In summary, the objective of this study was to build upon the findings of the 2019 storm sewer study of the Essex Town Centre and develop planning level conceptual designs in the remaining four (4) catchment areas (Maidstone, Rush, Hopgood, and Arner Townline). This study included updating and calibrating the hydrologic-hydraulic model to reflect ongoing and future development in the region with the purpose of performing a capacity assessment. This capacity assessment was used to identify areas of concern in the Essex Town Centre and preliminary recommendations for improvements to the stormwater drainage system are outlined in **Table 9.1.**

When capital budget funding becomes available, it is recommended that the work described in the Essex Town Centre ESR and in this report proceed to Phase 5 with final design and construction.

Table 9.1: Summary of Recommended Upgrades

Item	Description	EA Schedule
1	Maidstone Avenue Trunk Sewer Replacement / Realignment (Maidstone Catchment)	Exempt
2	Maidstone Avenue Trunk Sewer Replacement / Realignment (Rush Catchment)	Exempt
3	Maidstone Pond	Exempt OR Schedule B (if additional property is required)
4	Sadlers Pond	Exempt OR Schedule B (if additional property is required)

APPENDICES

APPENDIX A CALIBRATION RESULTS

A.1 DEPTH AND FLOW HYDROGRAPHS

A.2 SUMMARY OF OBSERVED-VS-PREDICTED VALUES

A.2.1 Flow Monitor No. 1 – South Talbot (West) Catchment

Event No.	Event Start	Volume (m^3)			Peak Depth (m)			Peak Flow (L/s)		
		Observed	Predicted	Difference	Observed	Predicted	Difference	Observed	Predicted	Difference
1	June 26 10:00	588.5	706.4	20.0%	0.5513	0.4787	-13.2%	167.9	241.0	43.5%
2	July 20 12:30	681.2	1107.0	62.5%	0.6211	0.8552	37.7%	290.8	283.4	-2.5%
3	July 26 13:00	604.2	666.5	10.3%	0.4407	0.4179	-5.2%	146.3	197.7	35.1%
4	August 14 17:00	1095.0	1219.0	11.3%	0.6978	0.8433	20.9%	277.9	383.3	37.9%
5	August 17 12:00	469.0	423.4	-9.7%	0.362	0.2957	-18.3%	89.91	80.27	-10.7%

A.2.2 Flow Monitor No. 2 – South Talbot (East) Catchment

Event No.	Event Start	Volume (m^3)			Peak Depth (m)			Peak Flow (L/s)		
		Observed	Predicted	Difference	Observed	Predicted	Difference	Observed	Predicted	Difference
1	June 26 10:00	1245.0	1307.0	5.0%	0.8257	0.6310	-23.6%	172.3	175.0	1.6%
2	July 20 12:30	1851.0	1507.0	-18.6%	1.108	0.8402	-24.2%	229.7	238.3	3.7%
3	July 26 13:00	1759.0	1274.0	-27.6%	1.012	0.5386	-46.8%	172.3	127.0	-26.3%
4	August 14 17:00	2310.0	1692.0	-26.8%	1.055	0.8795	-16.6%	234.1	249.8	6.7%
5	August 17 12:00	1438.0	825.8	-42.6%	0.6414	0.4787	-25.4%	104.6	93.07	-11.0%

A.2.3 Flow Monitor No. 3 – Arner Townline Catchment

Event No.	Event Start	Volume (m^3)			Peak Depth (m)			Peak Flow (L/s)		
		Observed	Predicted	Difference	Observed	Predicted	Difference	Observed	Predicted	Difference
1	June 26 10:00	-	-	-	-	-	-	-	-	-
2	July 20 12:30	1023.0	1167.0	14.1%	0.7793	0.5847	-25.0%	268.6	295.3	9.9%
3	July 26 13:00	1145.0	788.0	-31.2%	0.7645	0.4114	-46.2%	308.2	165.7	-46.2%
4	August 14 17:00	1701.0	1289.0	-24.2%	0.7605	0.8195	7.8%	277.1	420.9	51.9%
5	August 17 12:00	711.8	480.1	-32.6%	0.5768	0.3277	-43.2%	60.7	101.1	66.6%

A.2.4 Flow Monitor No. 4 – Maidstone Catchment

Event No.	Event Start	Volume (m^3)			Peak Depth (m)			Peak Flow (L/s)		
		Observed	Predicted	Difference	Observed	Predicted	Difference	Observed	Predicted	Difference
1	June 26 10:00	1432.0	1281	-10.5%	0.6309	0.4876	-22.7%	75.7	181.8	140.2%
2	July 20 12:30	2223.0	1330	-40.2%	0.6022	0.6061	0.6%	87.6	282.4	222.4%
3	July 26 13:00	1292.0	1105	-14.5%	0.5673	0.4552	-19.8%	54.2	154.7	185.5%
4	August 14 17:00	2507.0	3469	38.4%	0.5953	0.6022	1.2%	197.4	304.2	54.1%
5	August 17 12:00	843.5	2110	150.1%	0.5904	0.4237	-28.2%	110.6	104.7	-5.3%

A.3 CALIBRATED PARAMETERS

Table A.2.1: Hydrologic-Hydraulic Model Calibrated Parameters

Flow Monitor No.	Catchment Description	Slope		Infiltration			Depression Storage		Subarea Routing (%)						
		Developed Areas (%)	Agricultural, Open Space, and Parkland (%)	Ks (mm/hr)	Md	Ψ (mm)	Dstore-Pervious (mm)	Dstore-Impervious (mm)	Residential	Institutional, Commercial	Industrial	Downtown (Impervious >70%)	Open Space, Agricultural	Parkland	Pavement
1	South Talbot (West)	0.3	N/A	3	0.3	110	8.5	3.5	55	-	-	-	-	-	0
2	South Talbot (East)	0.5	0.3	3.9	0.3	270	7.5	2.5	75	50	-	-	100	100	0
3	Arner	1	0.3	2.6	0.25	230	7.5	3.5	60	60	-	-	90	-	0
4	Maidstone	0.5	0.15	3.9	0.3	270	7.5 OR 10	2.5	50	10 OR 25	-	-	90	-	0

APPENDIX B FIGURES

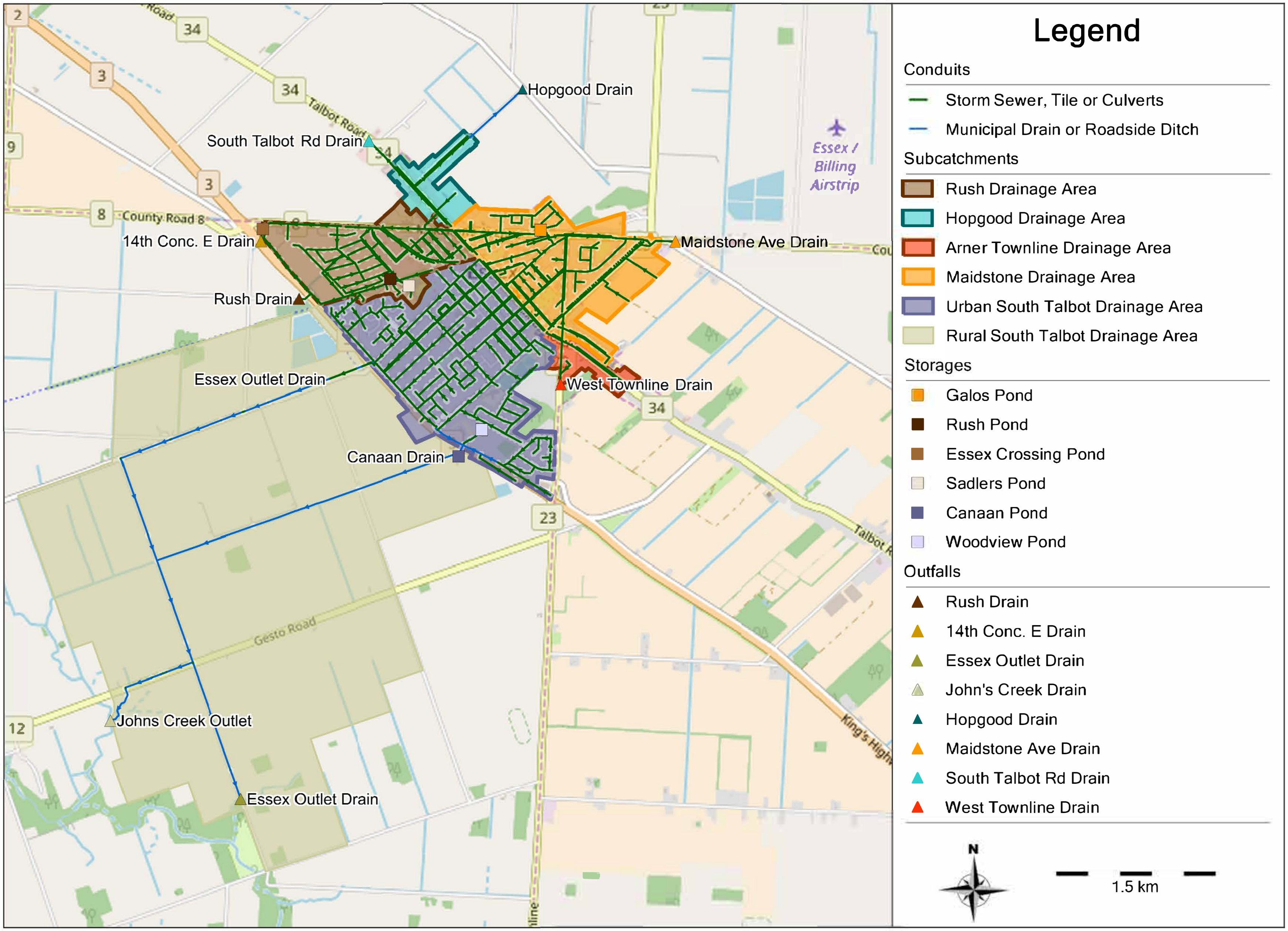

Figure 2.1: Site Plan of the Town of Essex Drainage System

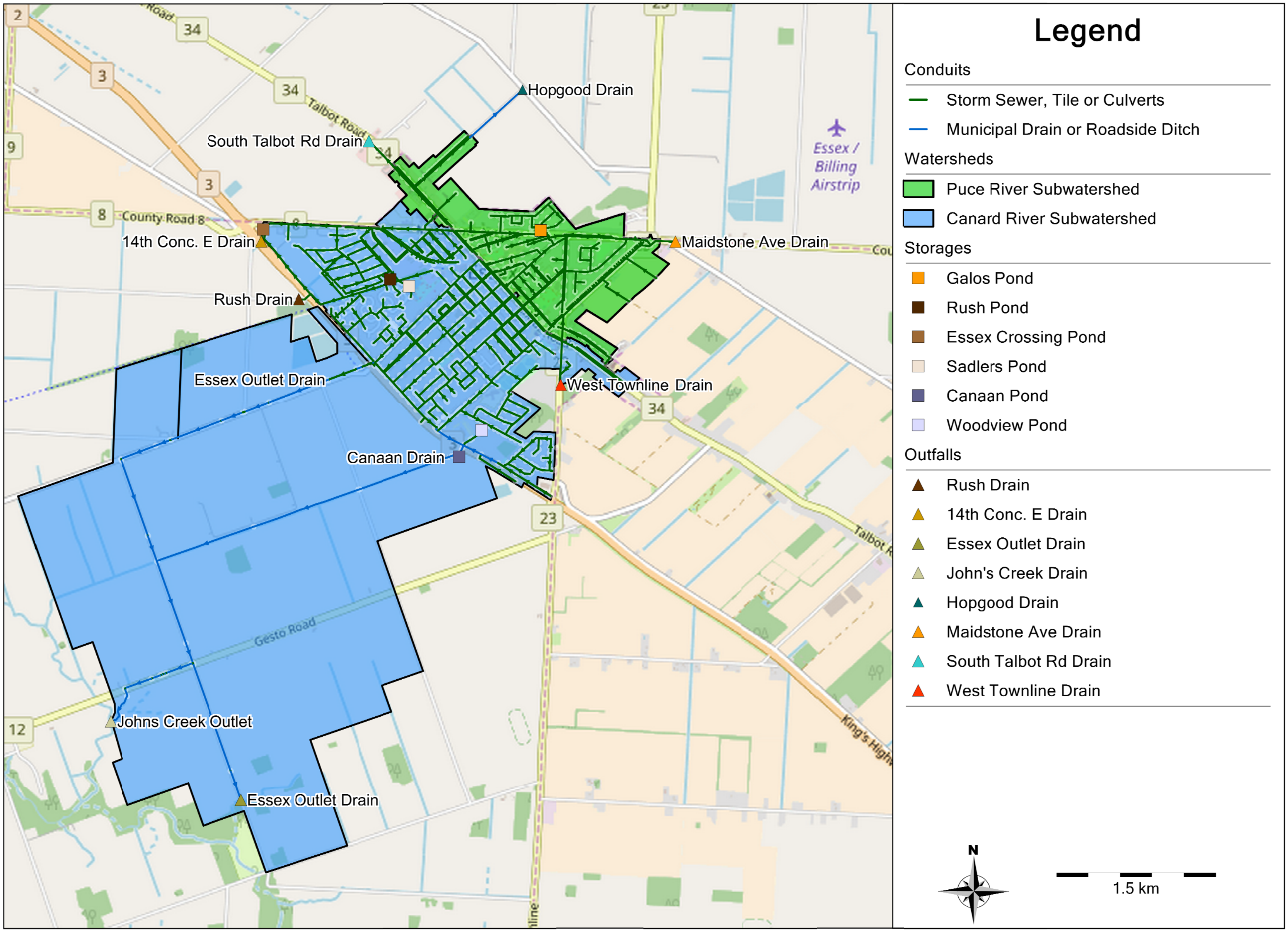

Figure 2.2: Subwatersheds in the Town of Essex Drainage System

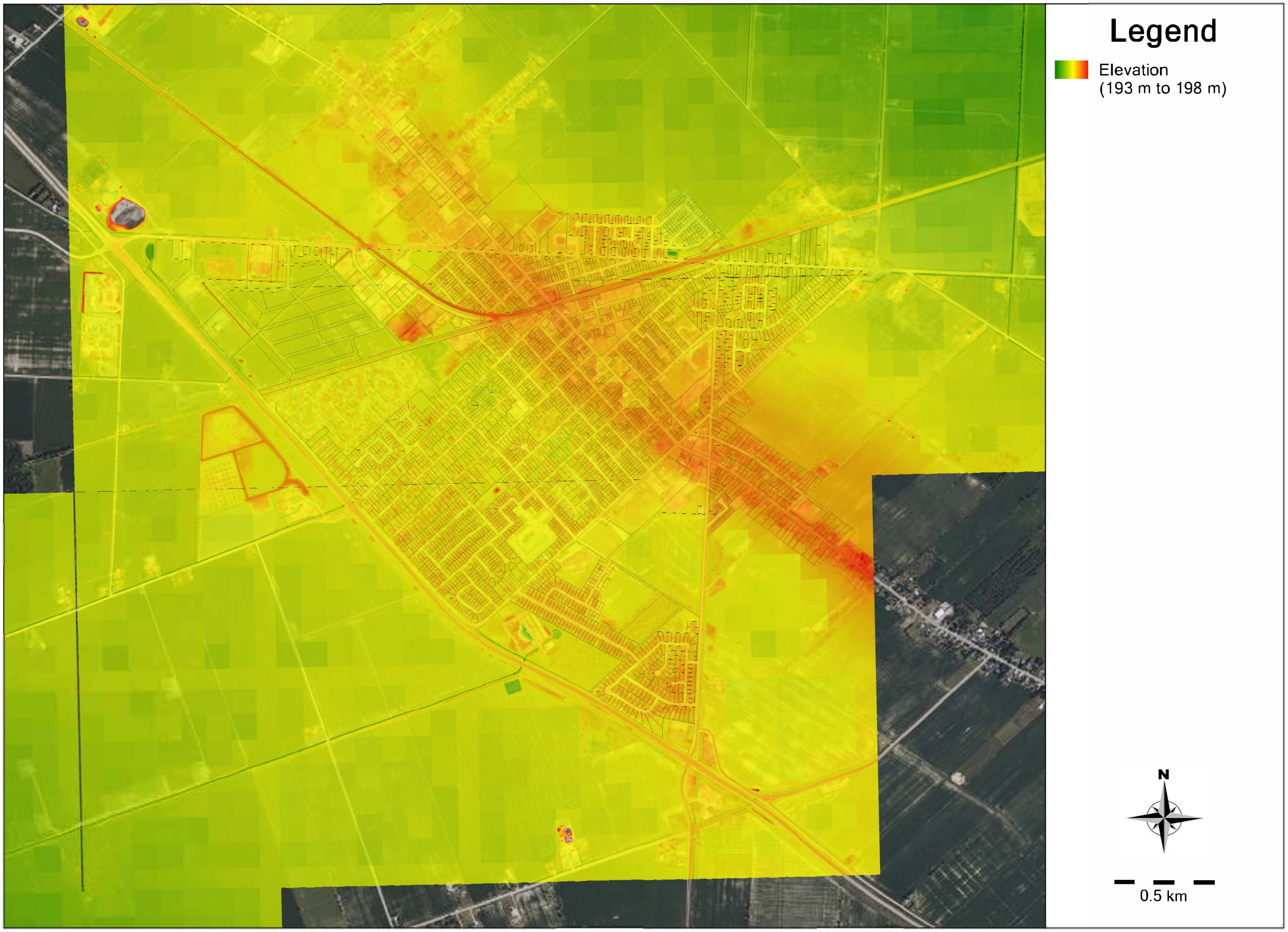

Figure 2.3: Topography in the Essex Town Centre

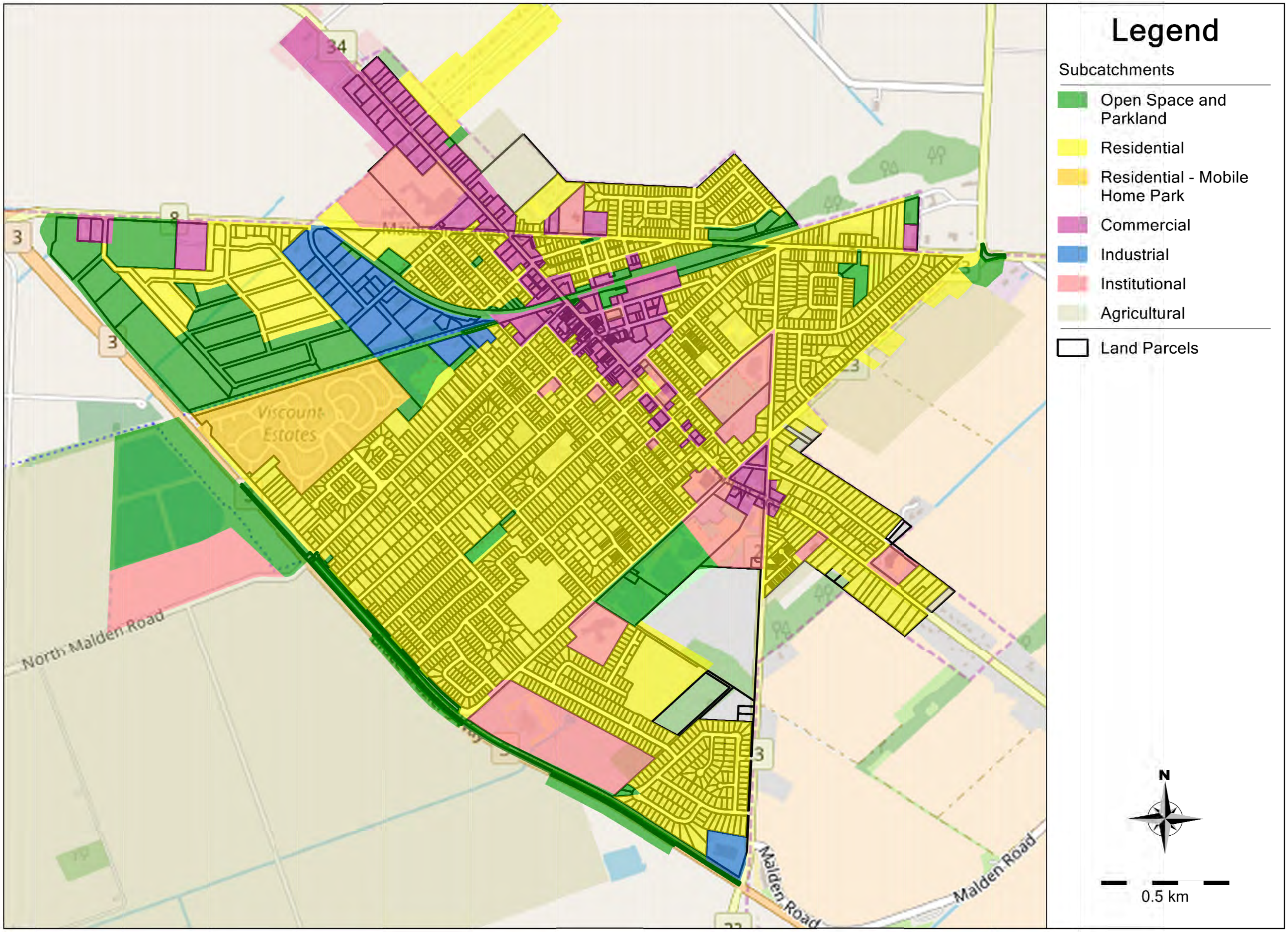

Figure 3.1: Land Use in the Essex Town Centre

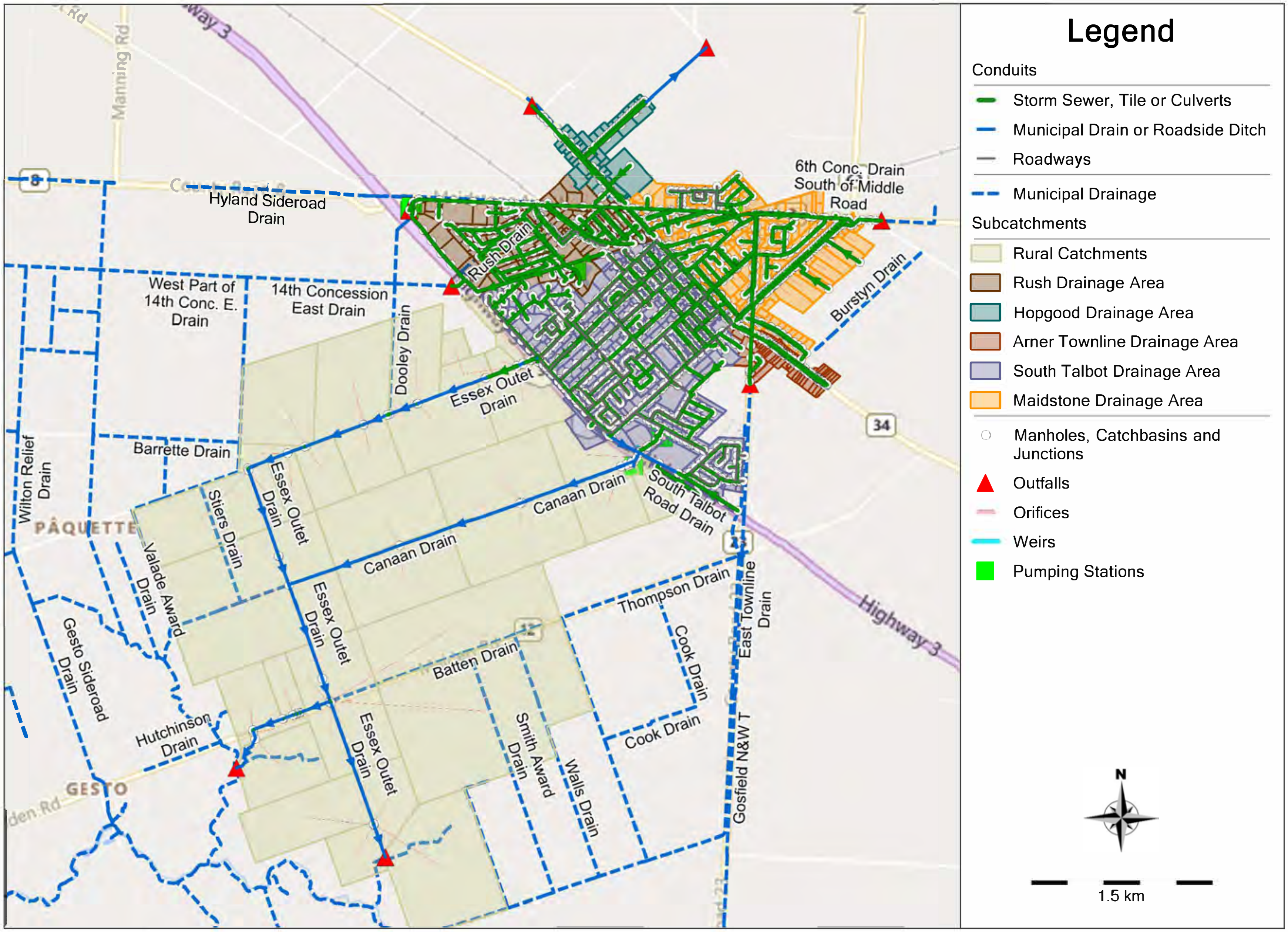

Figure 4.1 (a): Model Site Plan of the Entire Town of Essex Drainage System

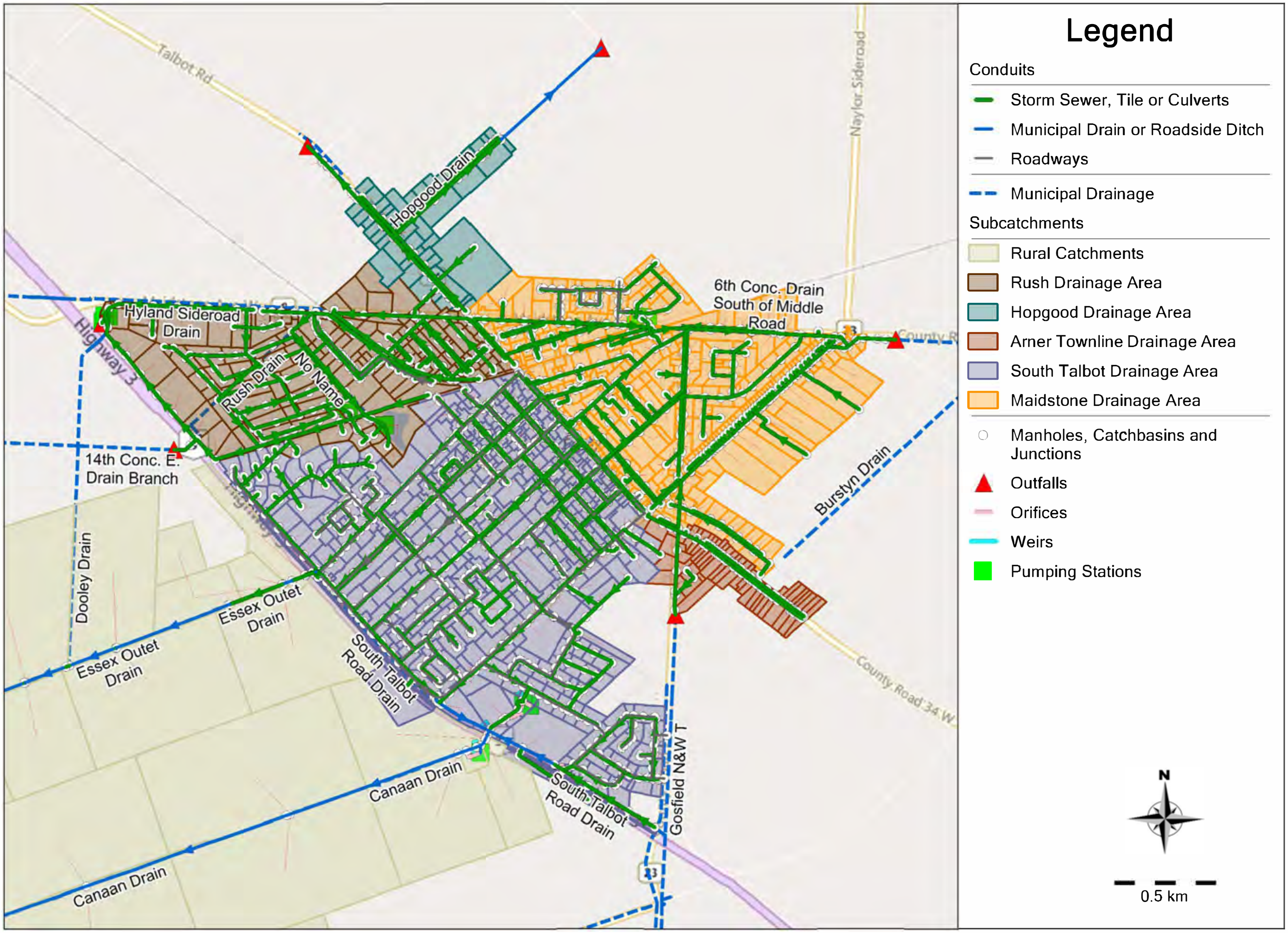

Figure 4.1 (b): Model Site Plan of the Urban Centre Drainage System

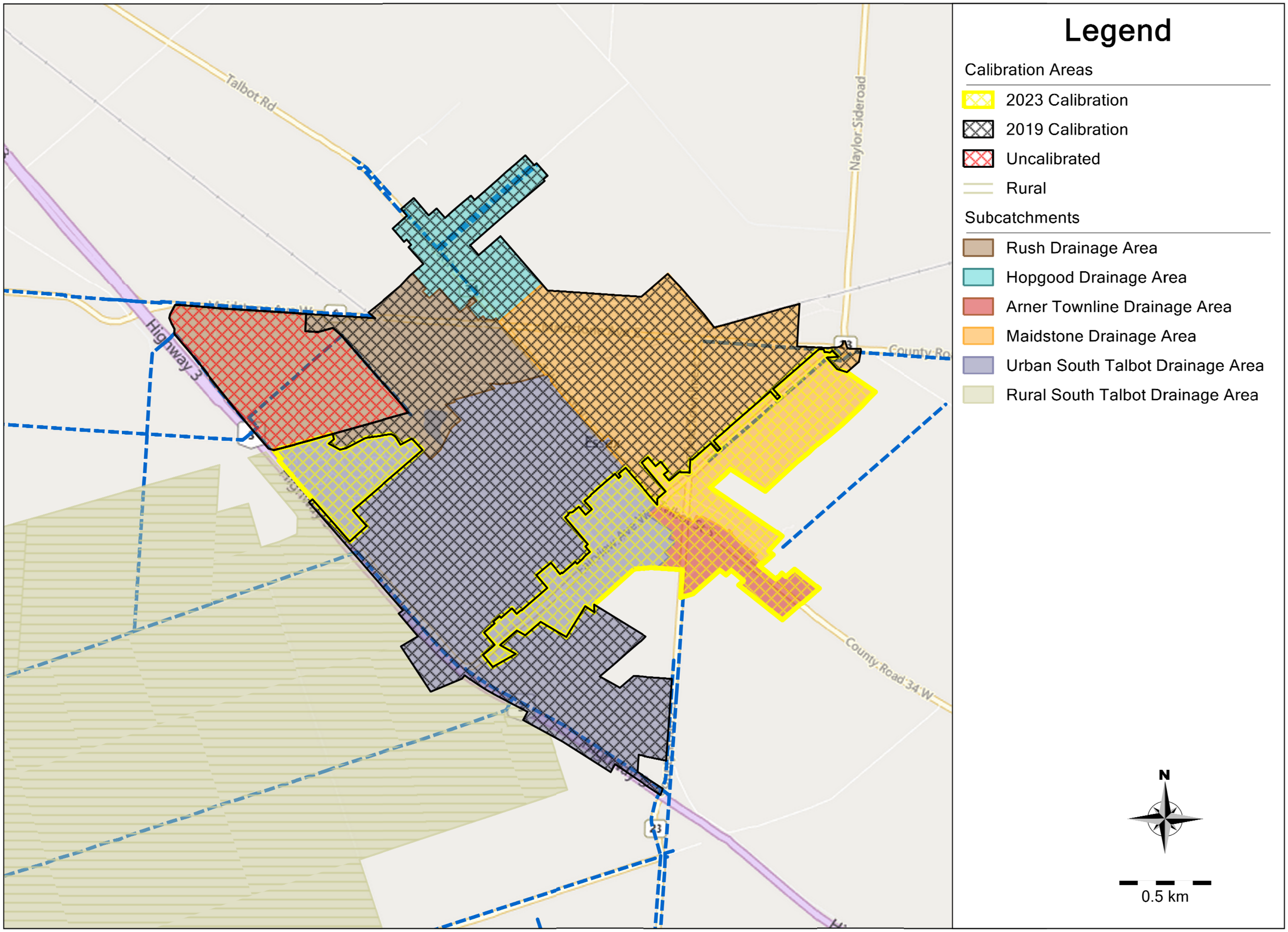

Figure 4.1 (c): Model Site Plan with Calibration Areas

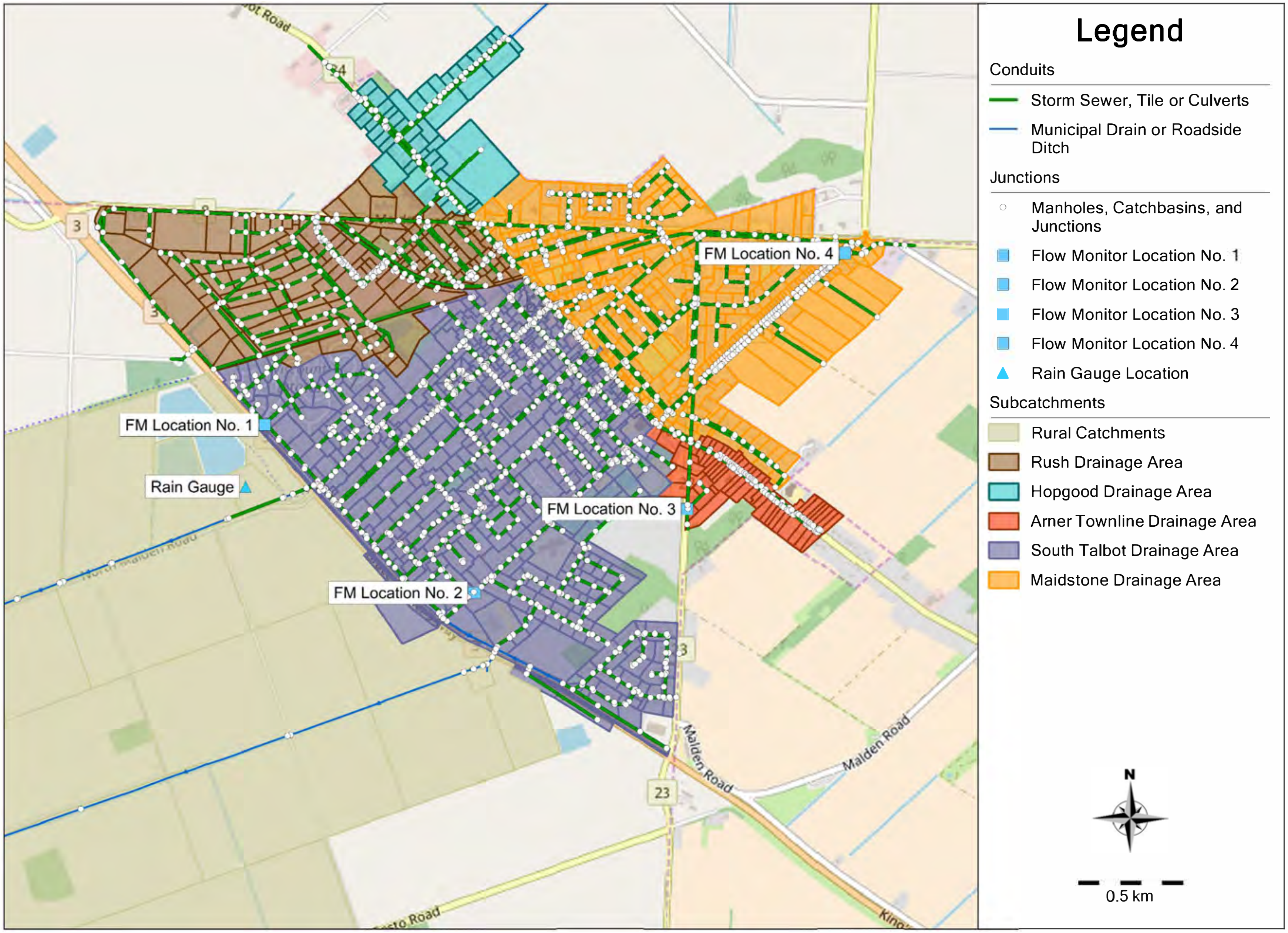

Figure 5.1: Rain Gauge and Flow Monitoring Locations

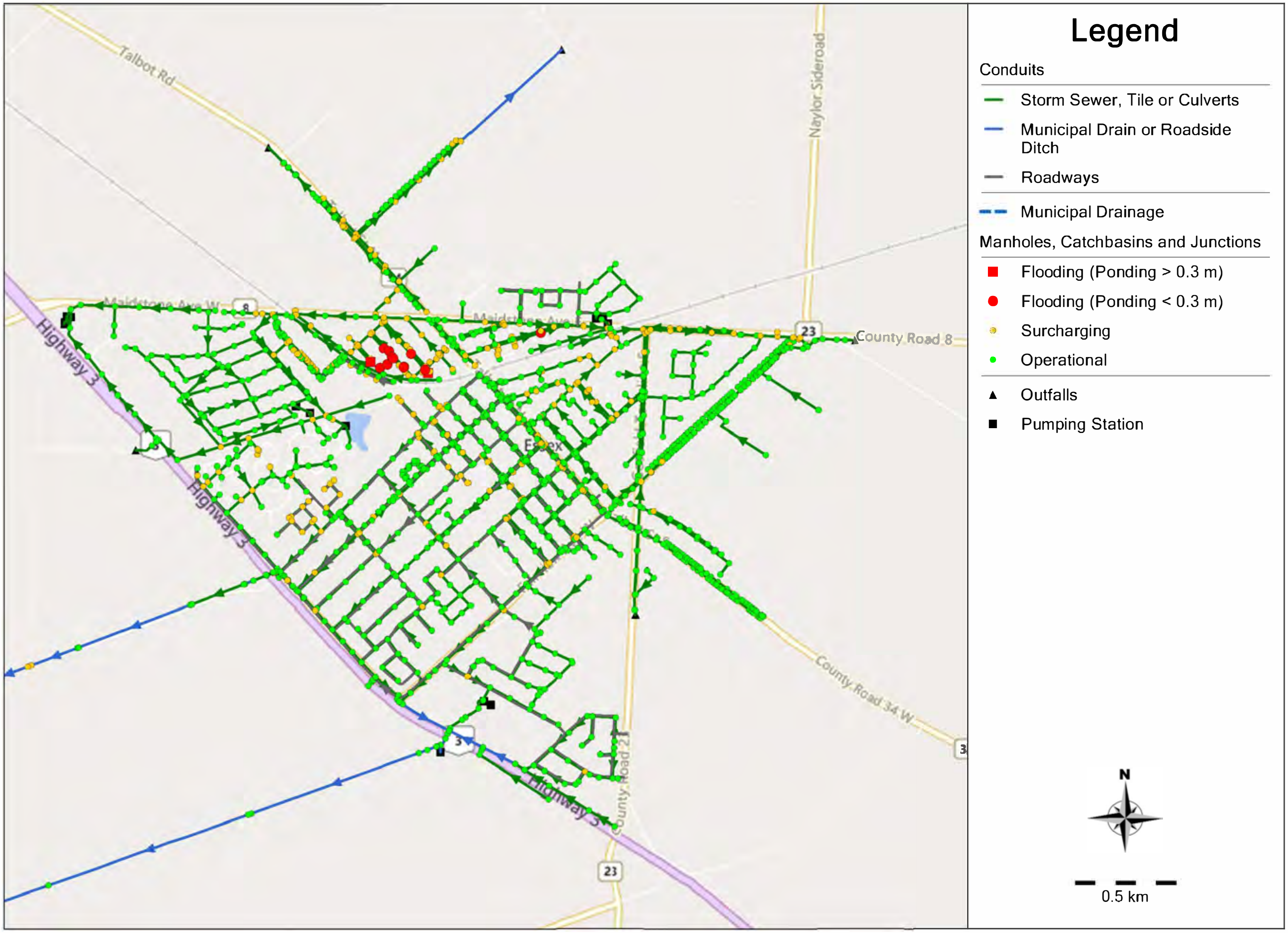

Figure 6.1: System Response to the 1 in 2-year Design Storm

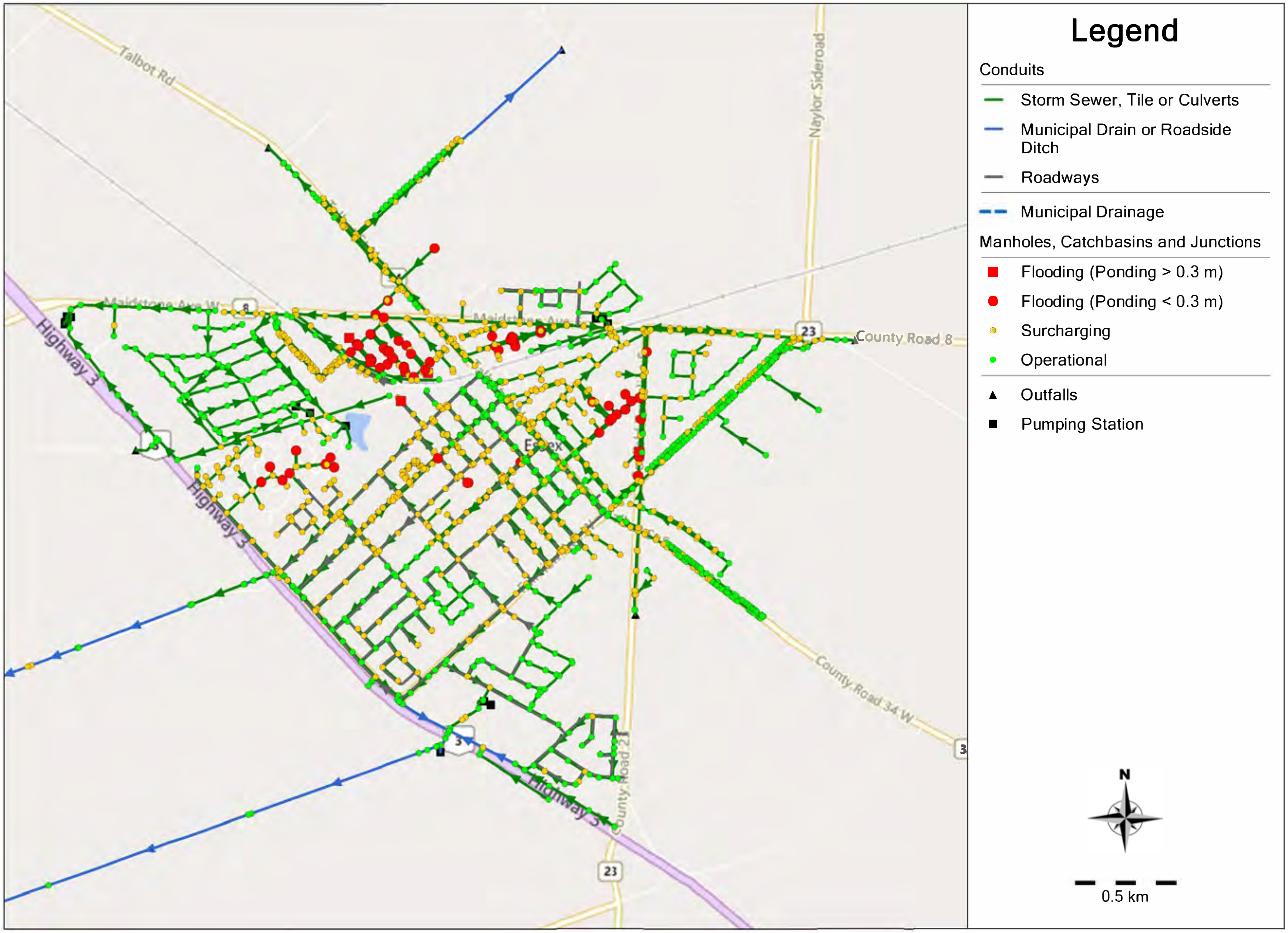

Figure 6.2: System Response to the 1 in 5-year Design Storm

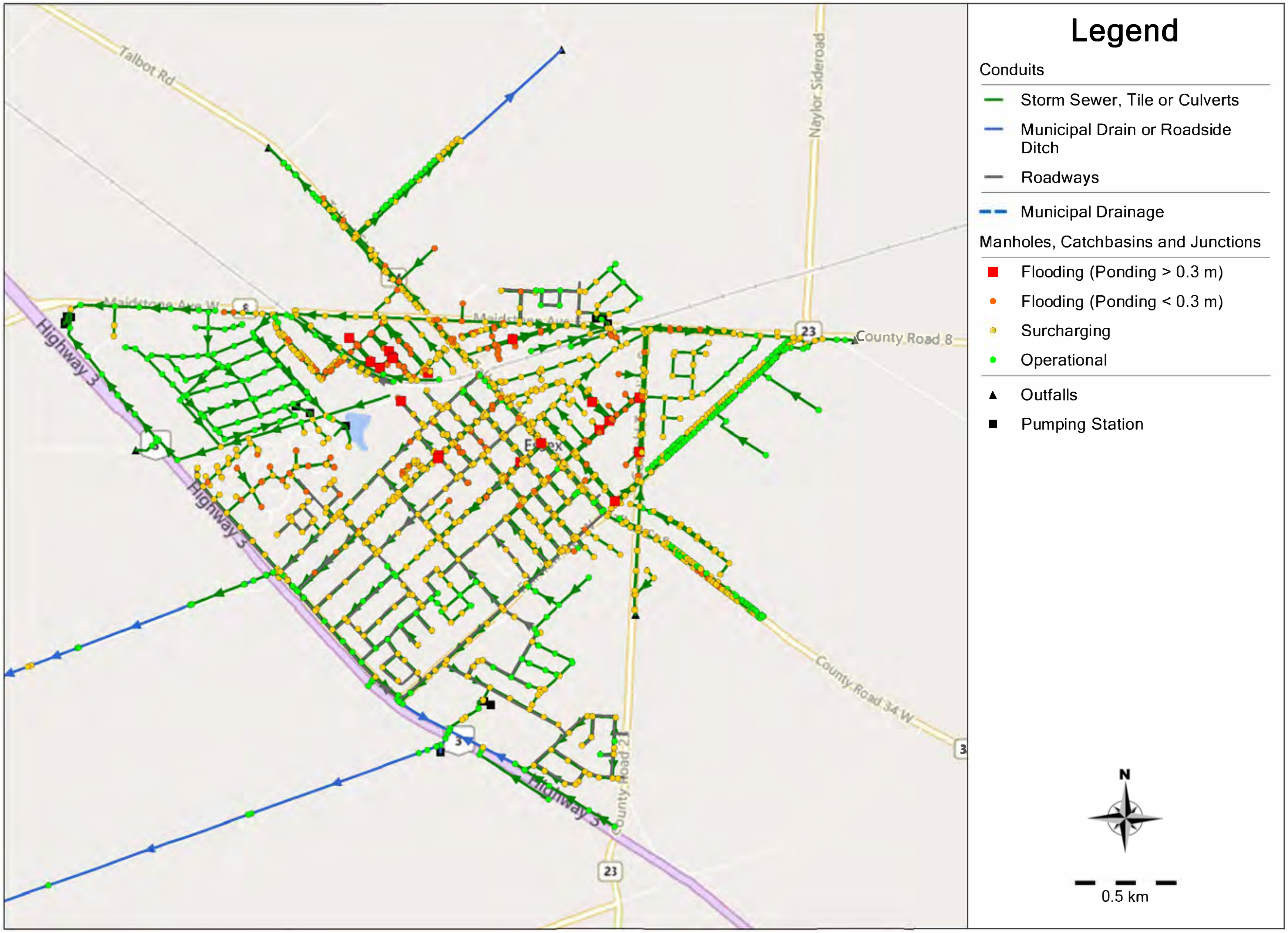

Figure 6.3: System Response to the 1 in 10-year Design Storm

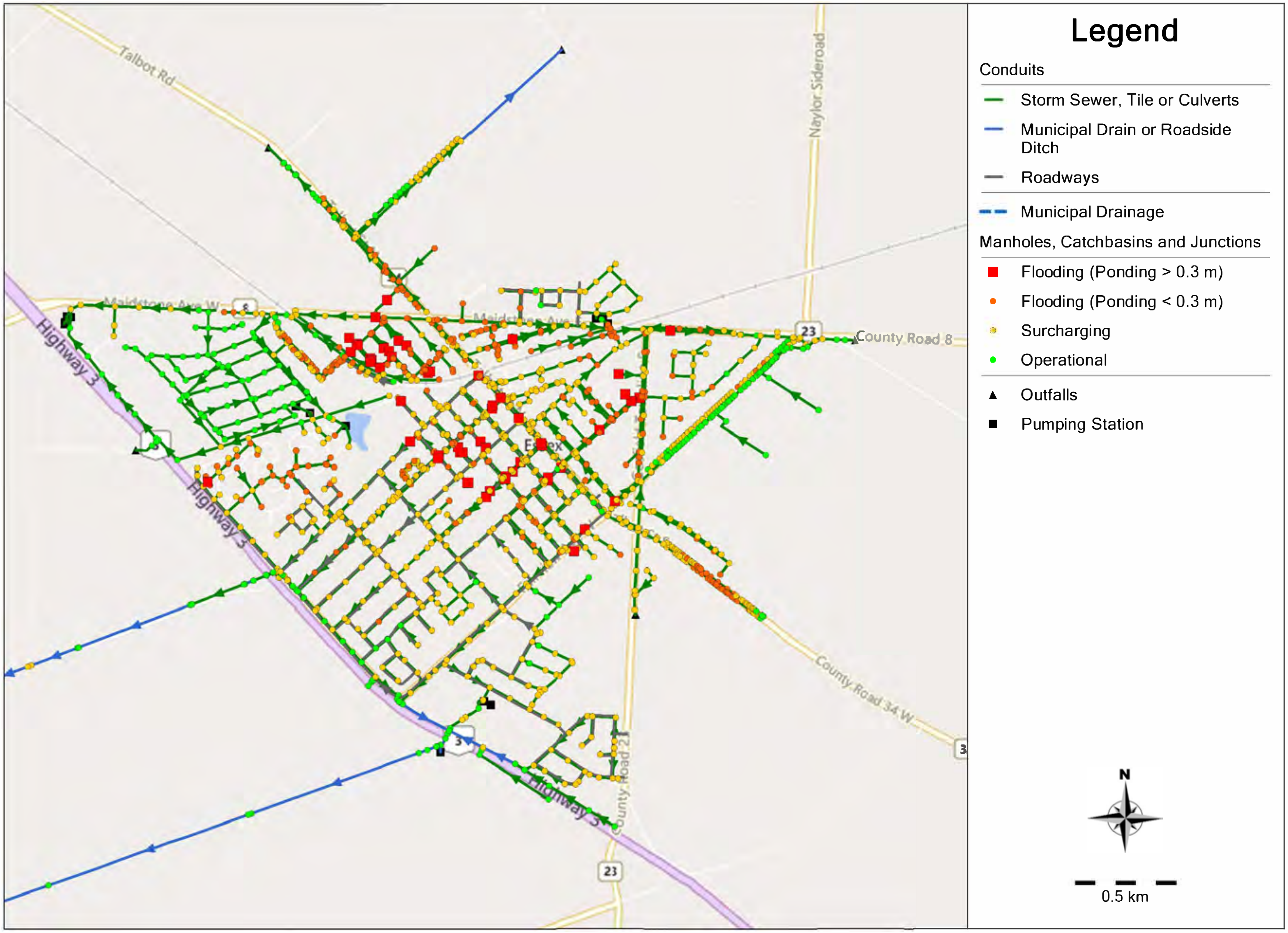

Figure 6.4: System Response to the 1 in 25-year Design Storm

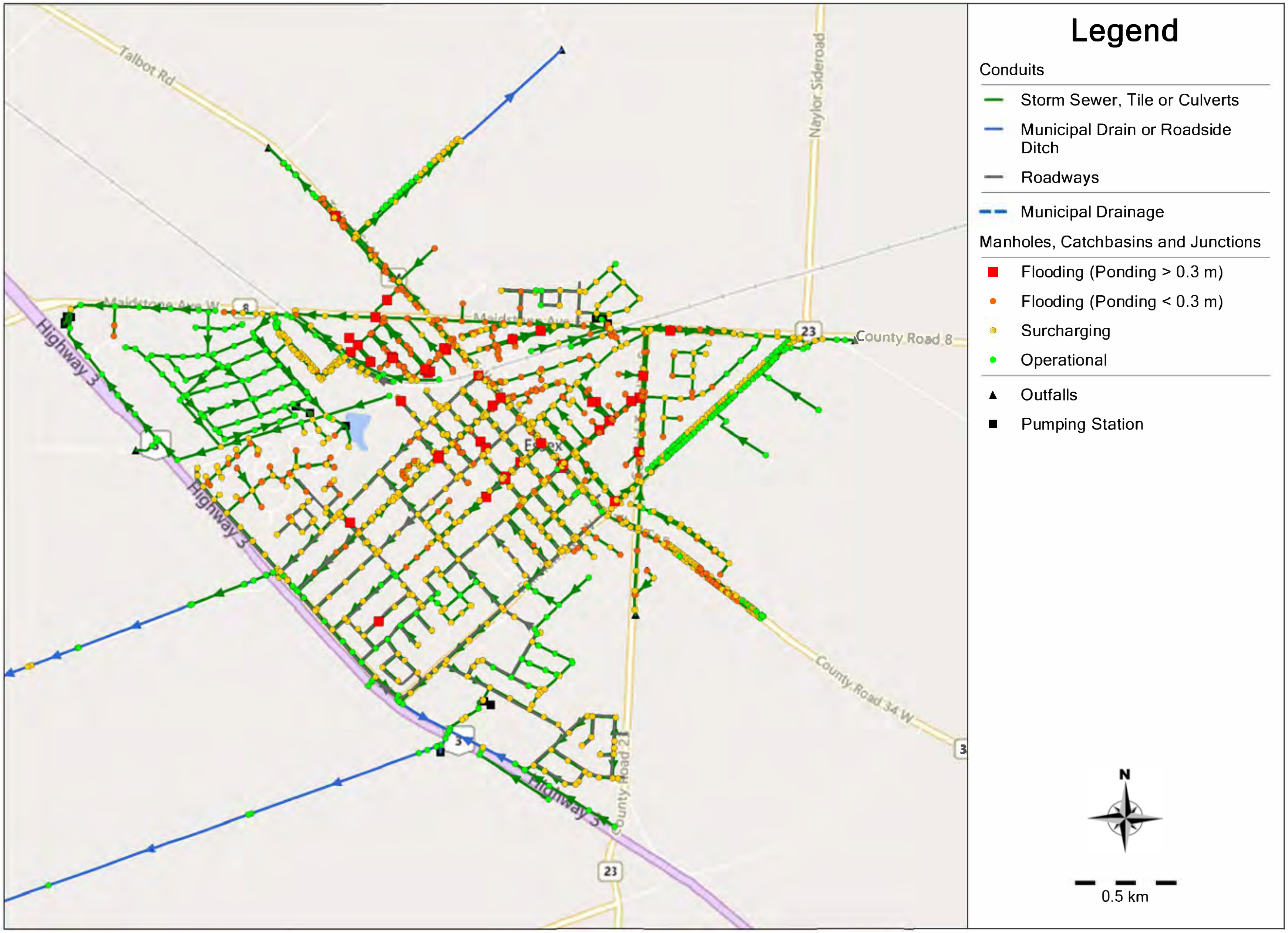

Figure 6.5: System Response to the 1 in 25-year Historic Storm

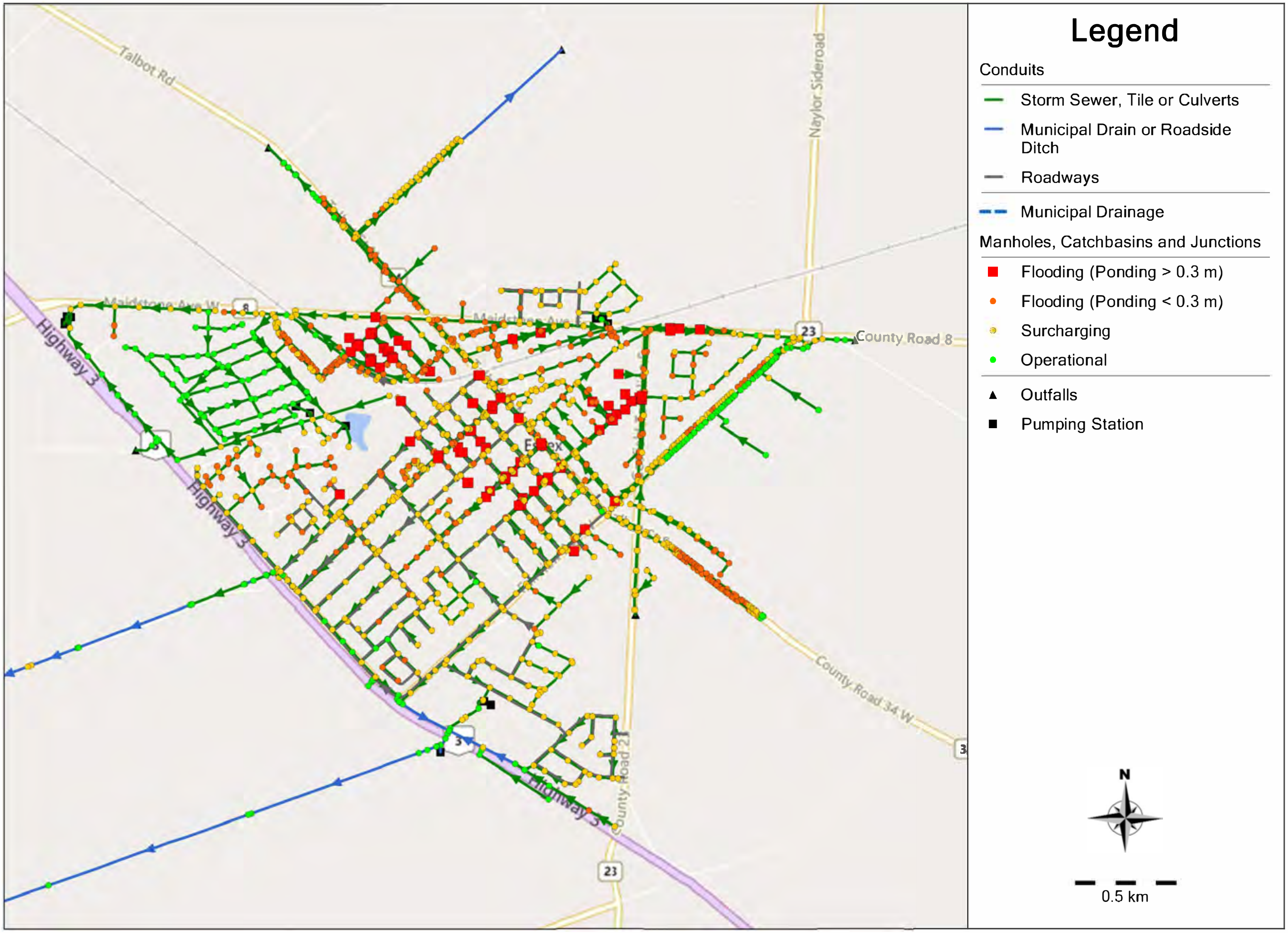

Figure 6.6: System Response to the 1 in 50-year Design Storm

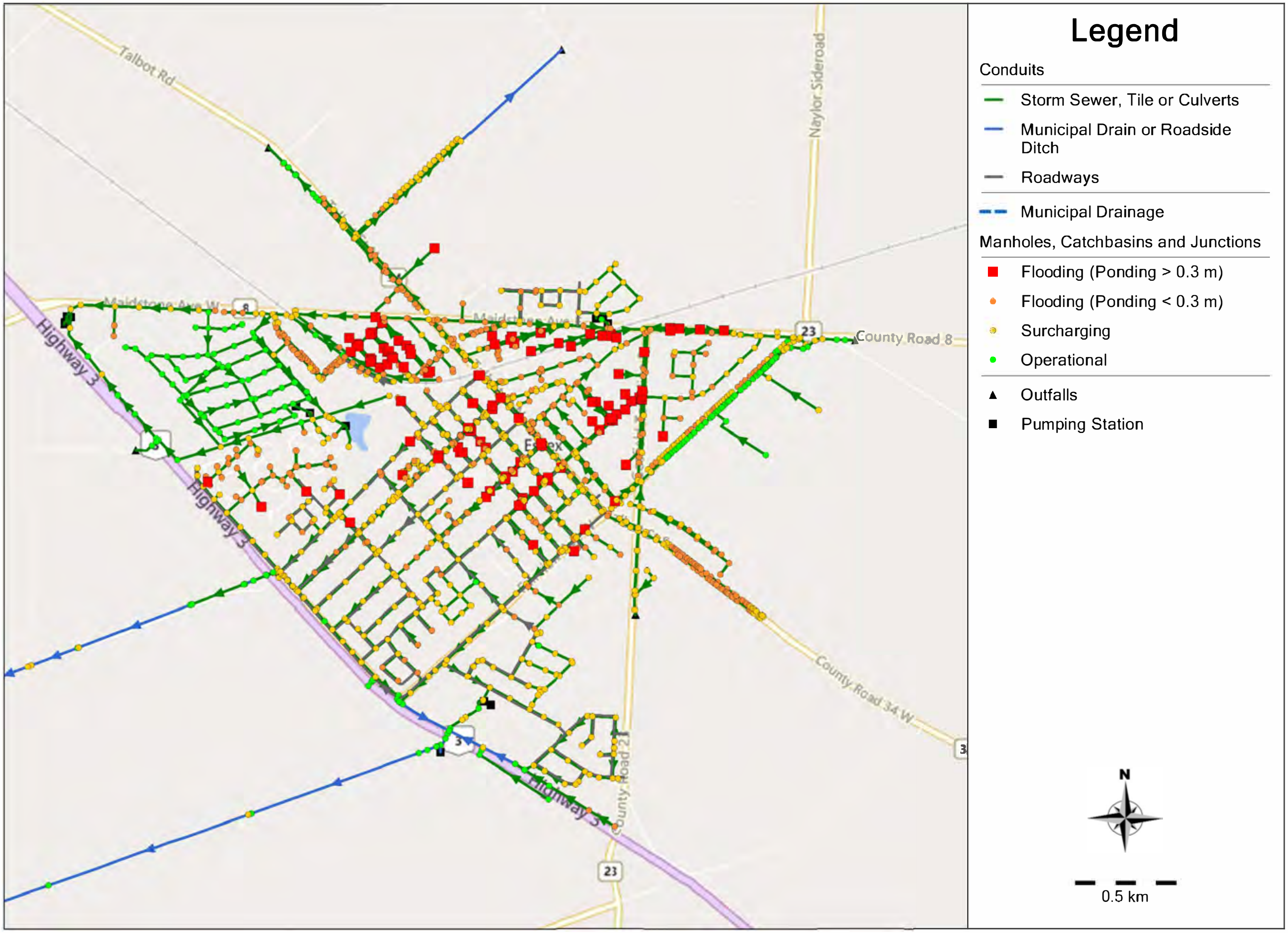

Figure 6.7: System Response to the 1 in 100-year Design Storm

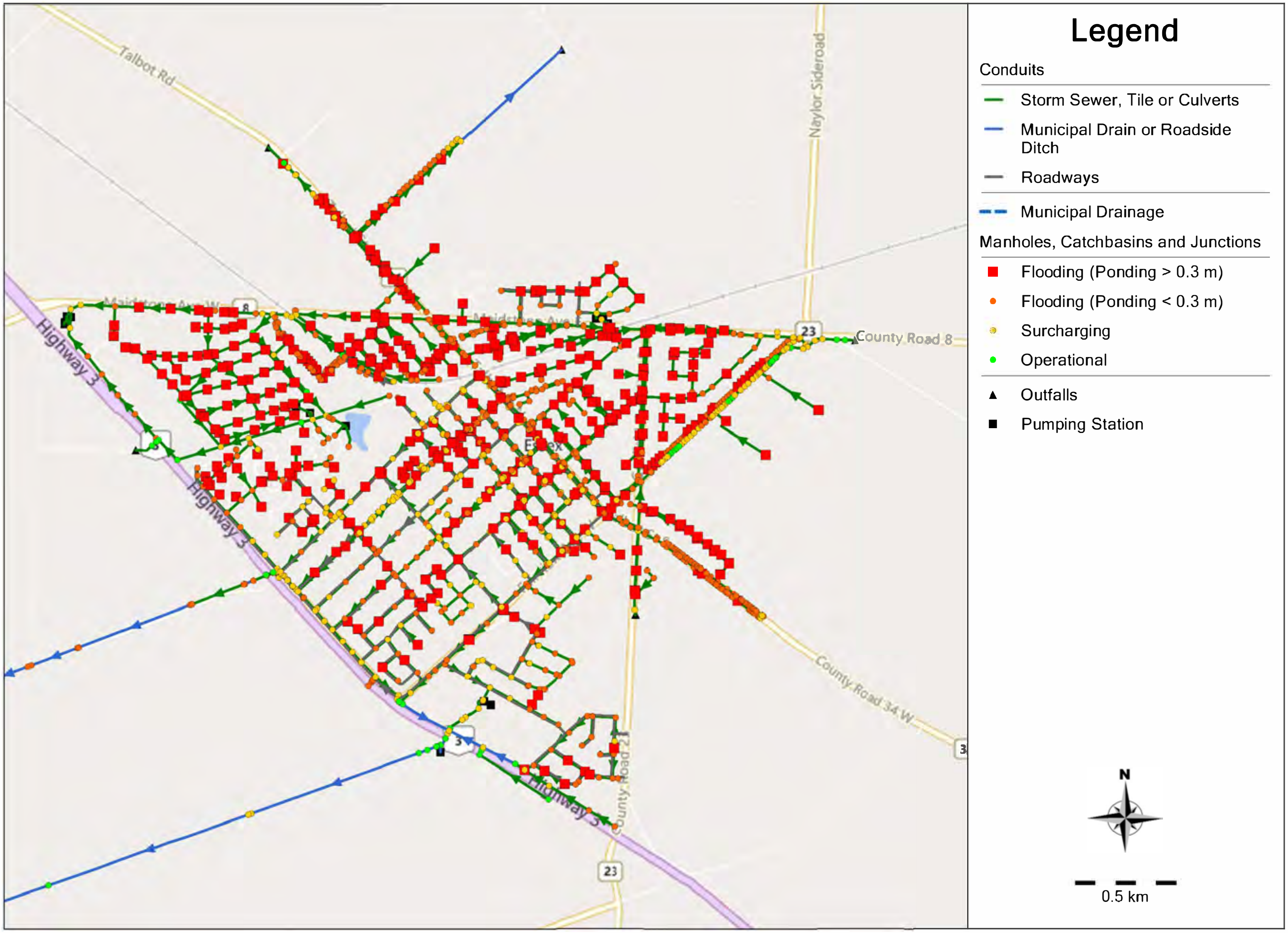

Figure 6.8: System Response to the Urban Stress Test Design Storm

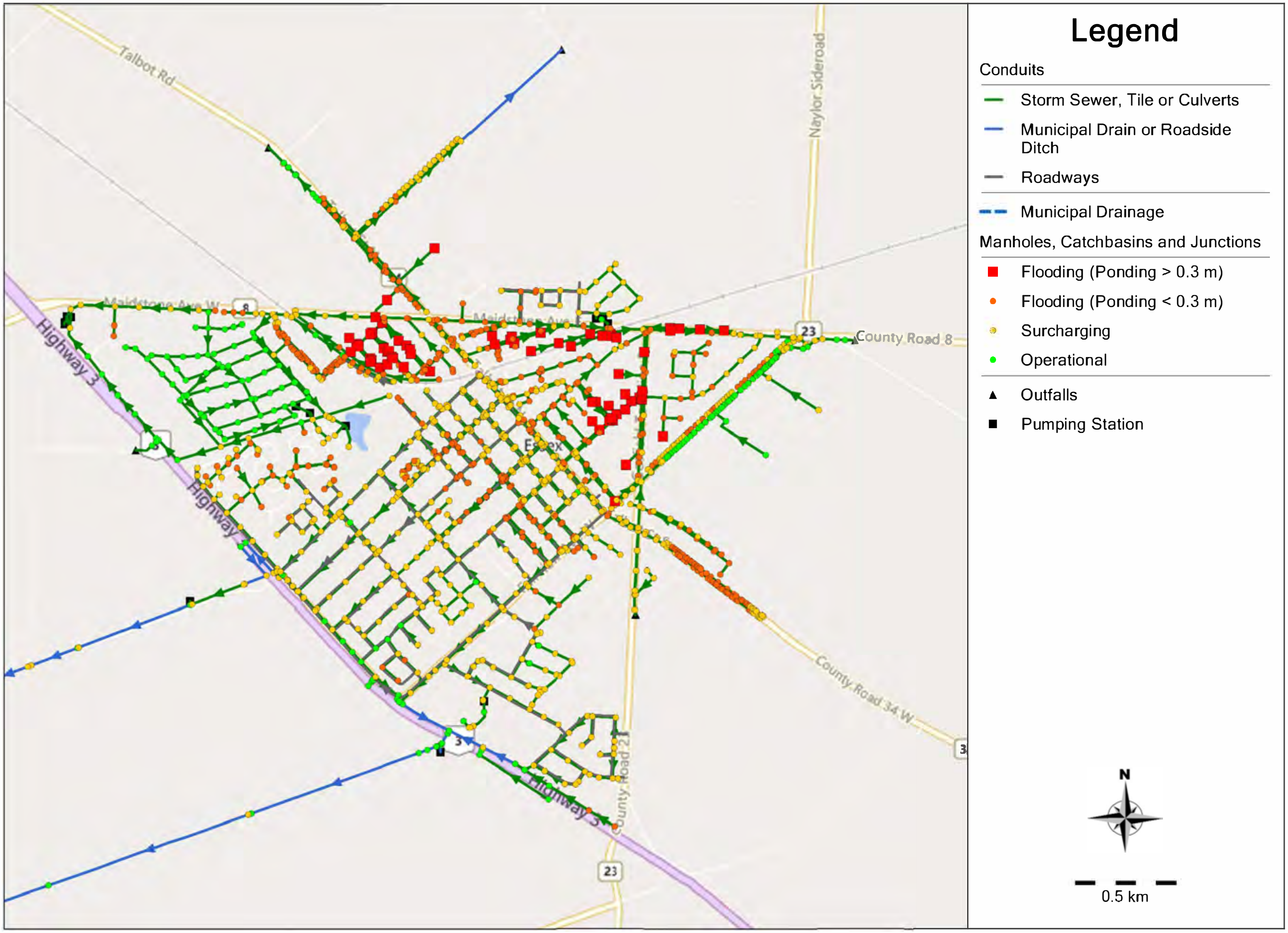

Figure 8.1: System Response after Essex Southwest Centre ESR Improvements (1 in 100-year)

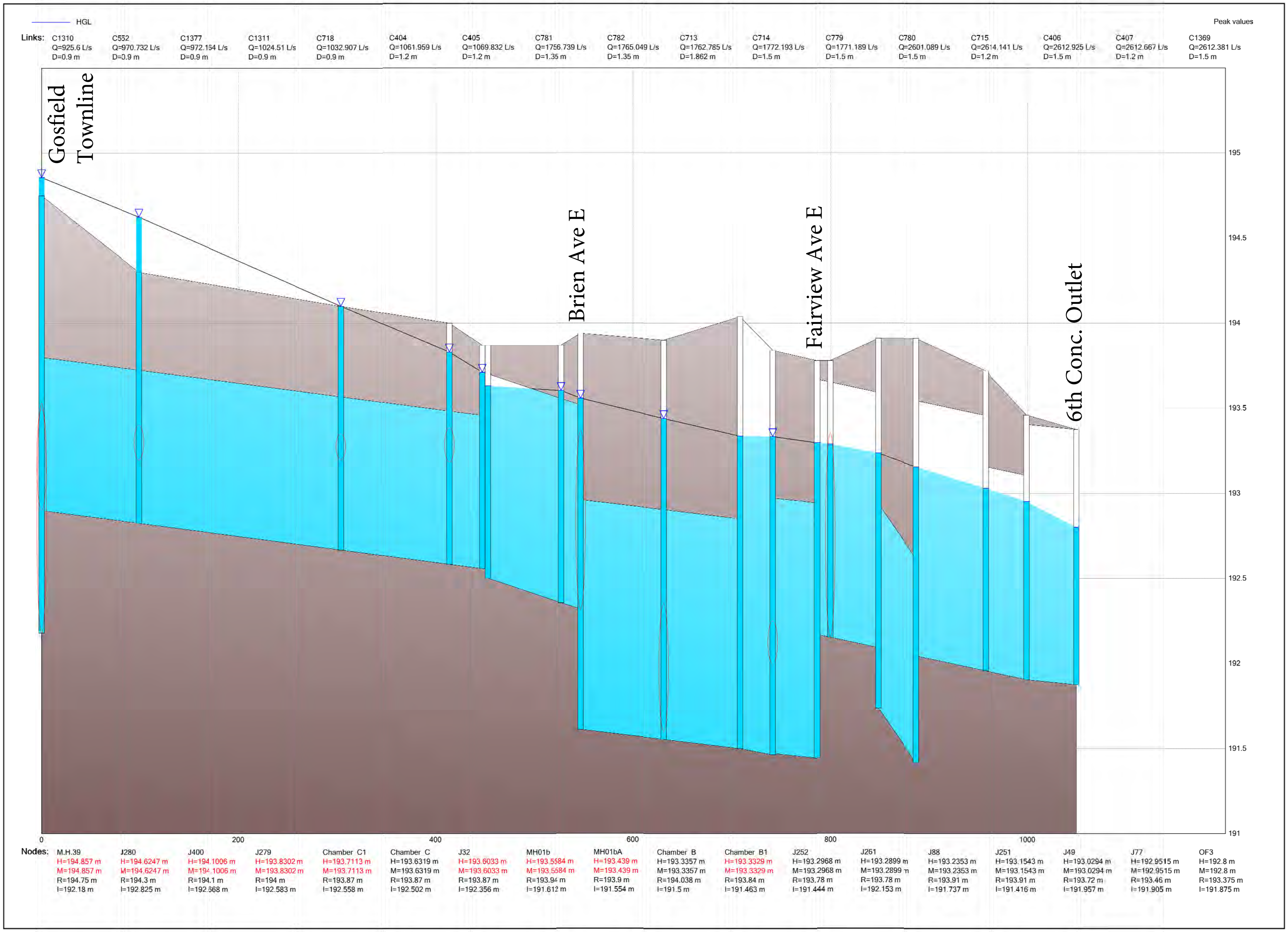

Figure 8.2: Existing Profile of the Maidstone Avenue Trunk Sewer (Maidstone Catchment)

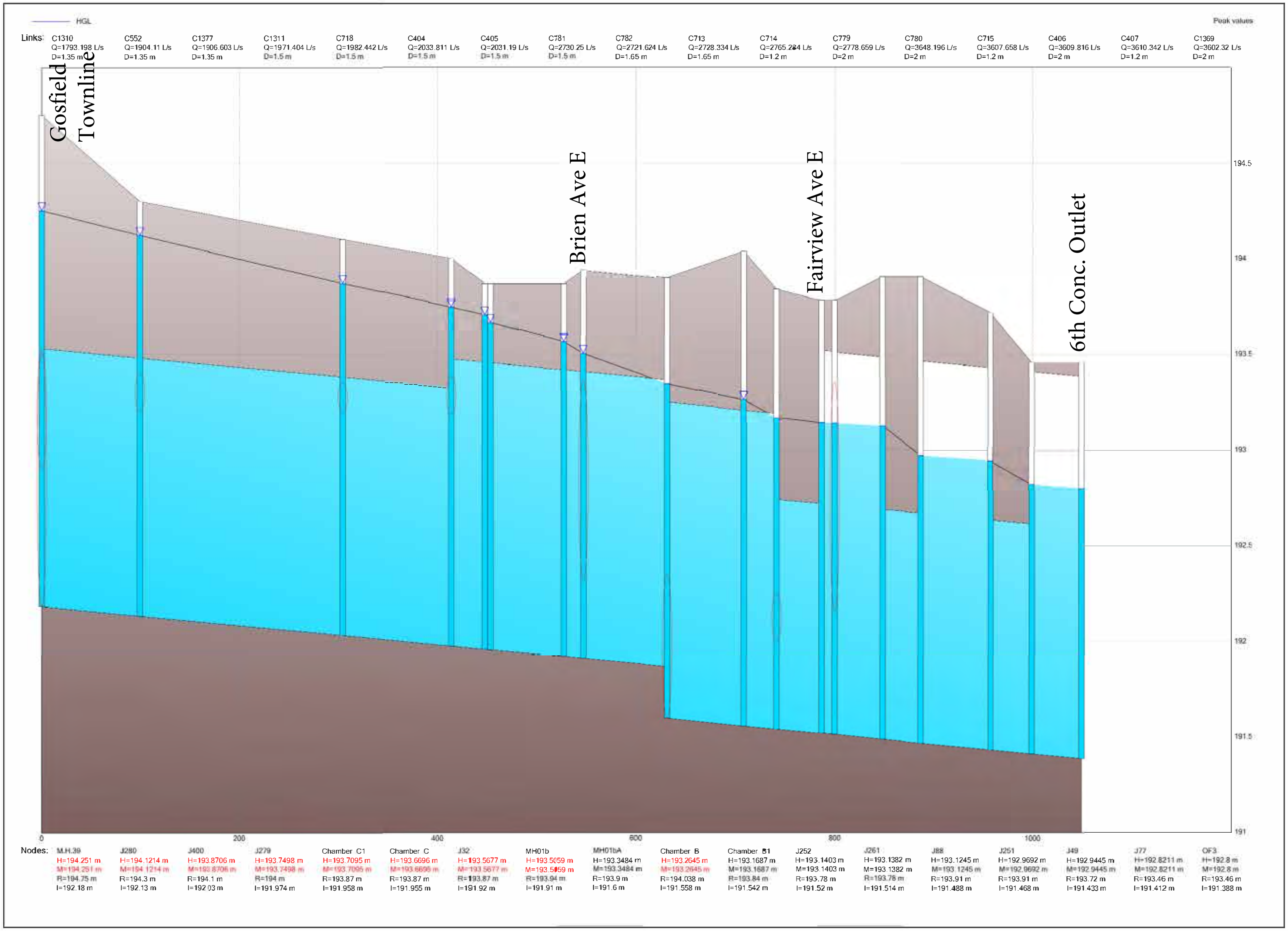

Figure 8.3: Proposed Profile of the Maidstone Avenue Trunk Sewer (Maidstone Catchment)

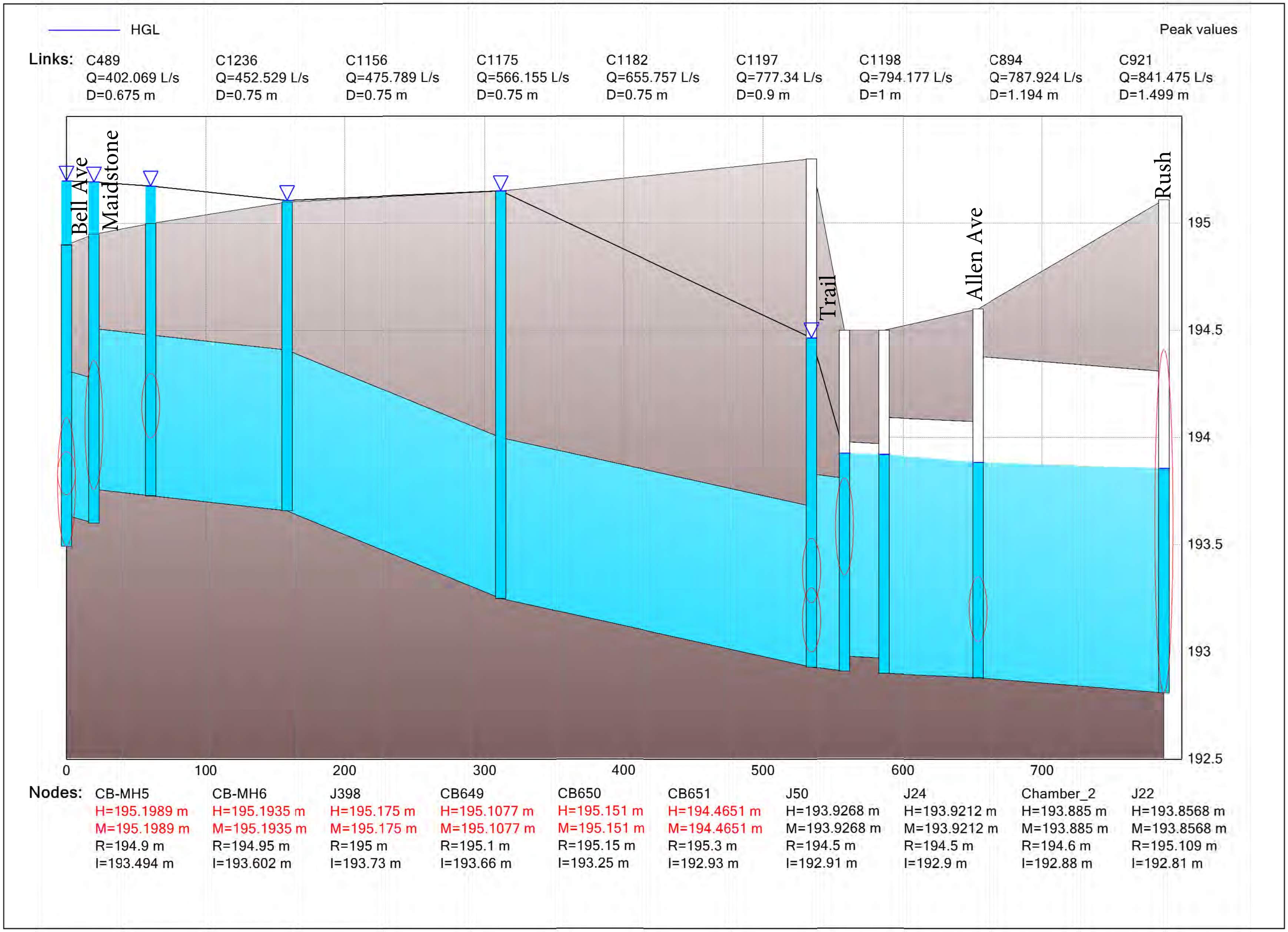

Figure 8.4: Existing Profile of the Maidstone Avenue Trunk Sewer (Rush Catchment)

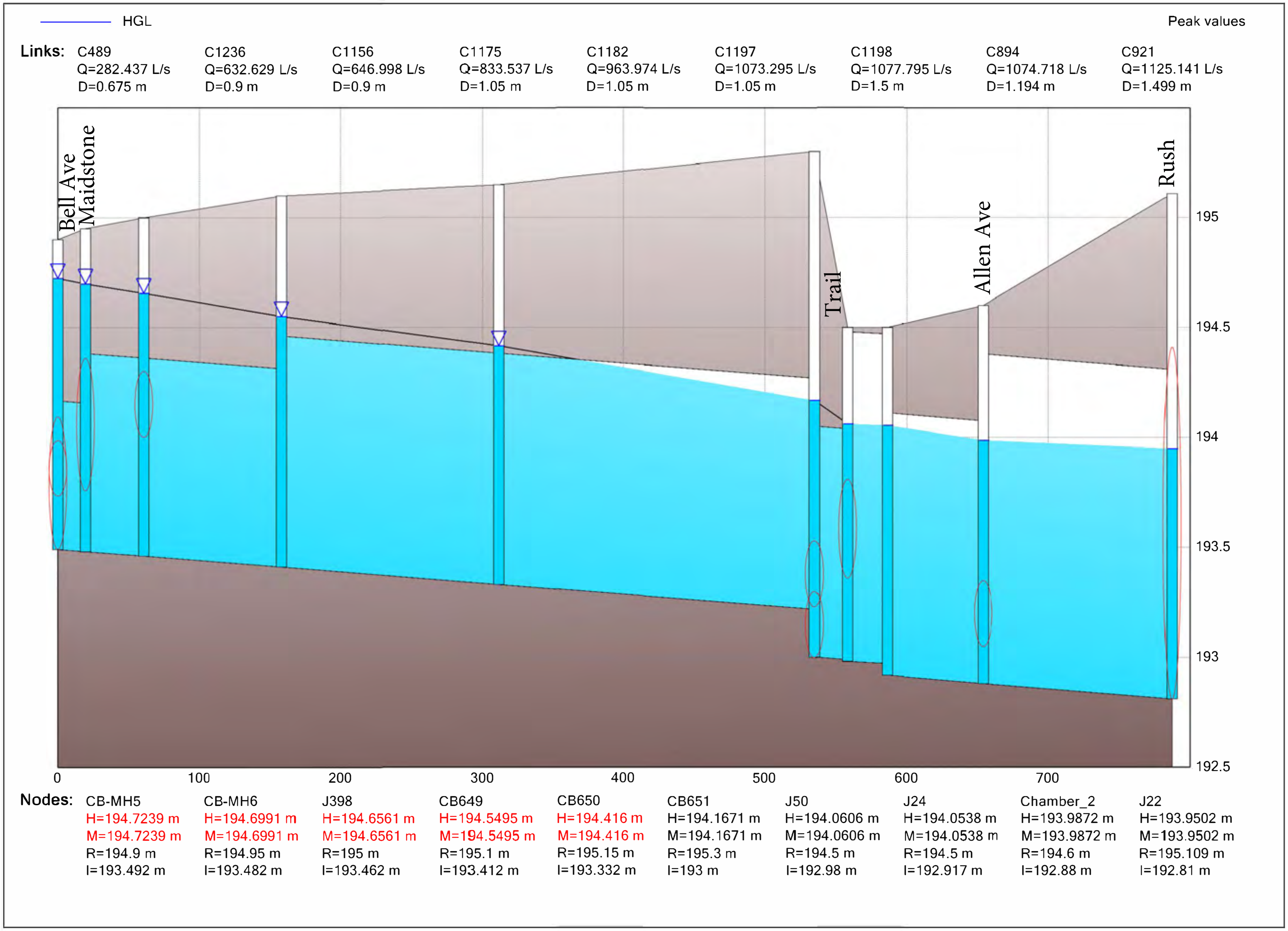

Figure 8.5: Proposed Profile of the Maidstone Avenue Trunk Sewer (Rush Catchment)

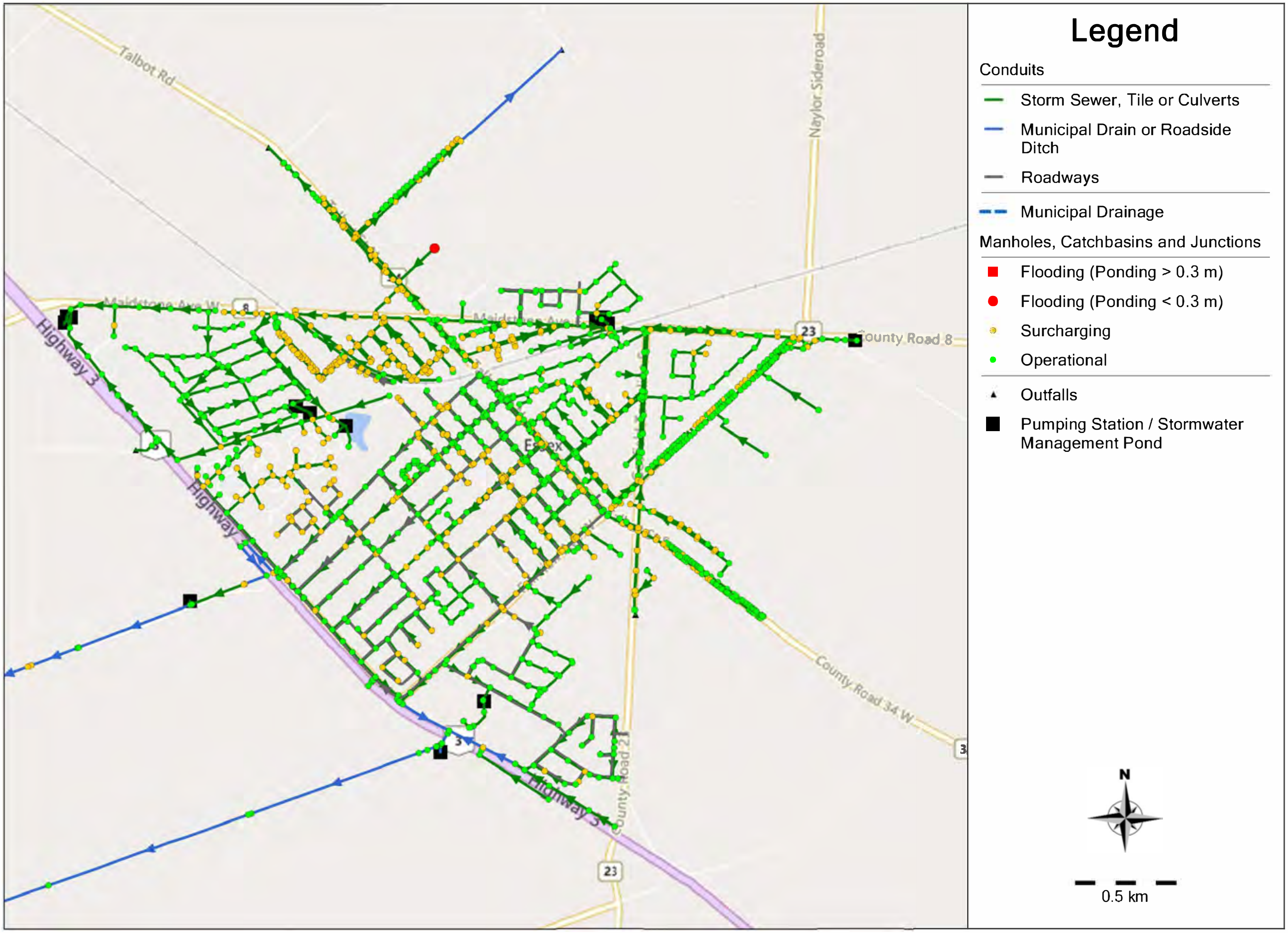

Figure 8.6: System Response after all Proposed Improvements (1 in 5-year)

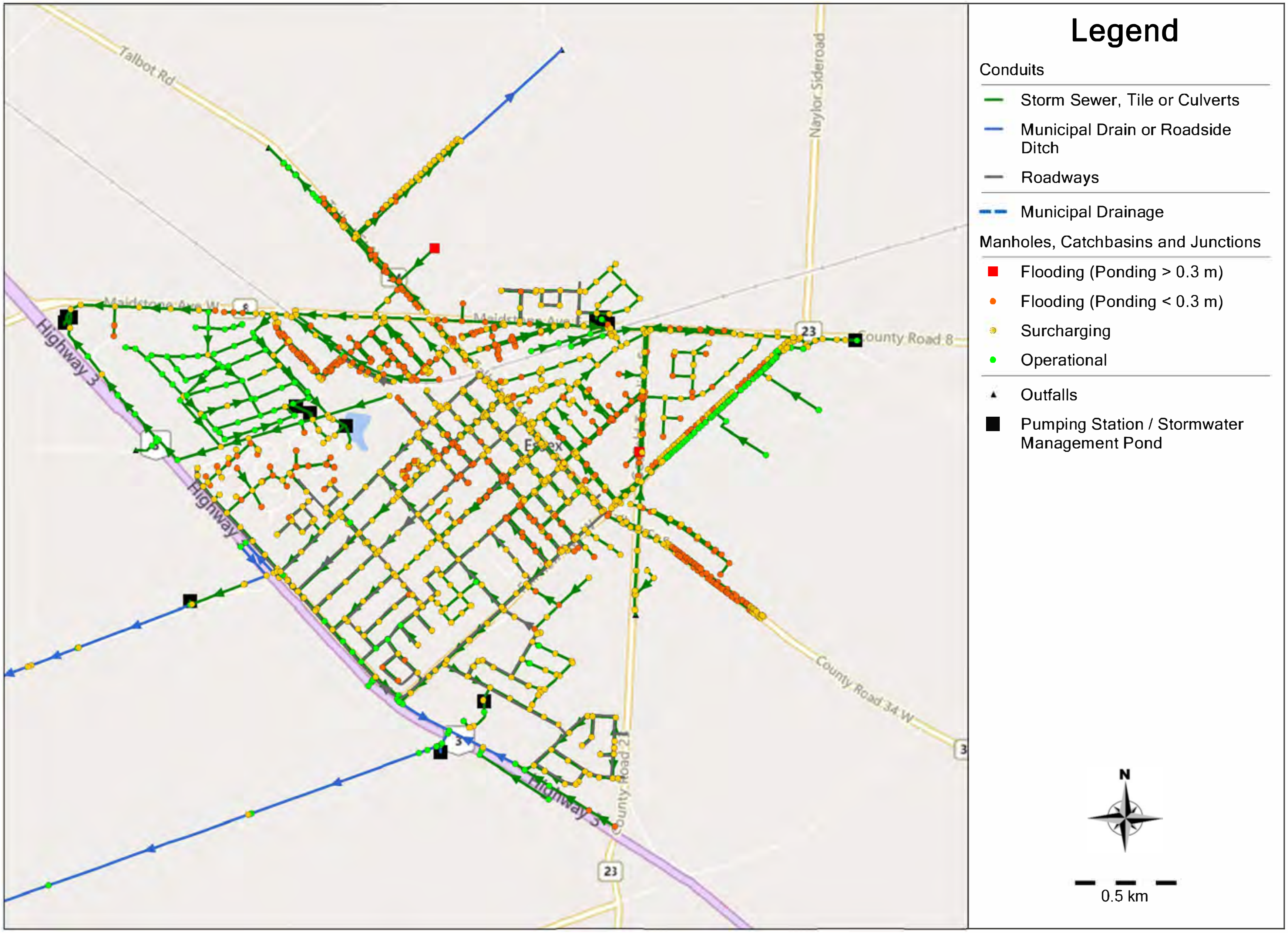

Figure 8.7: System Response after all Proposed Improvements (1 in 100-year)