JLR No.: 27931
Revision: 2

November 10, 2023

Prepared for:

THE TOWNSHIP OF STIRLING-RAWDON
2529 Stirling Marmora Rd,
Marmora, ON
K0K 2M0

Prepared by:

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DRAFT

Stirling Infrastructure Capacity Assessment

J.L.Richards
ENGINEERS · ARCHITECTS · PLANNERS

Table of Contents

List of Tables

List of Figures

List of Appendices

1.0 Introduction

1.1 General

The Township of Stirling-Rawdon retained JLR to conduct the update to the infrastructure capacity assessment, taking into account the existing servicing conditions and projected growth that it anticipates over the coming years. The Water, Wastewater and Stormwater Capacity Assessment was completed and identifies preferred options to meet the Existing, Short-Term (10 year design basis), Mid-Term (20 year design basis), and Full Build Out (beyond 20 years design basis) water, wastewater and stormwater infrastructure needs of the Municipality.

The Municipality's urban area of Stirling is located approximately 20km northwest of the Bellville Hwy 62 Exit on the 401 Expressway. It is located on the Rawdon Creek at the confluence of Highway 8 and Highway 14.

2.0 Background

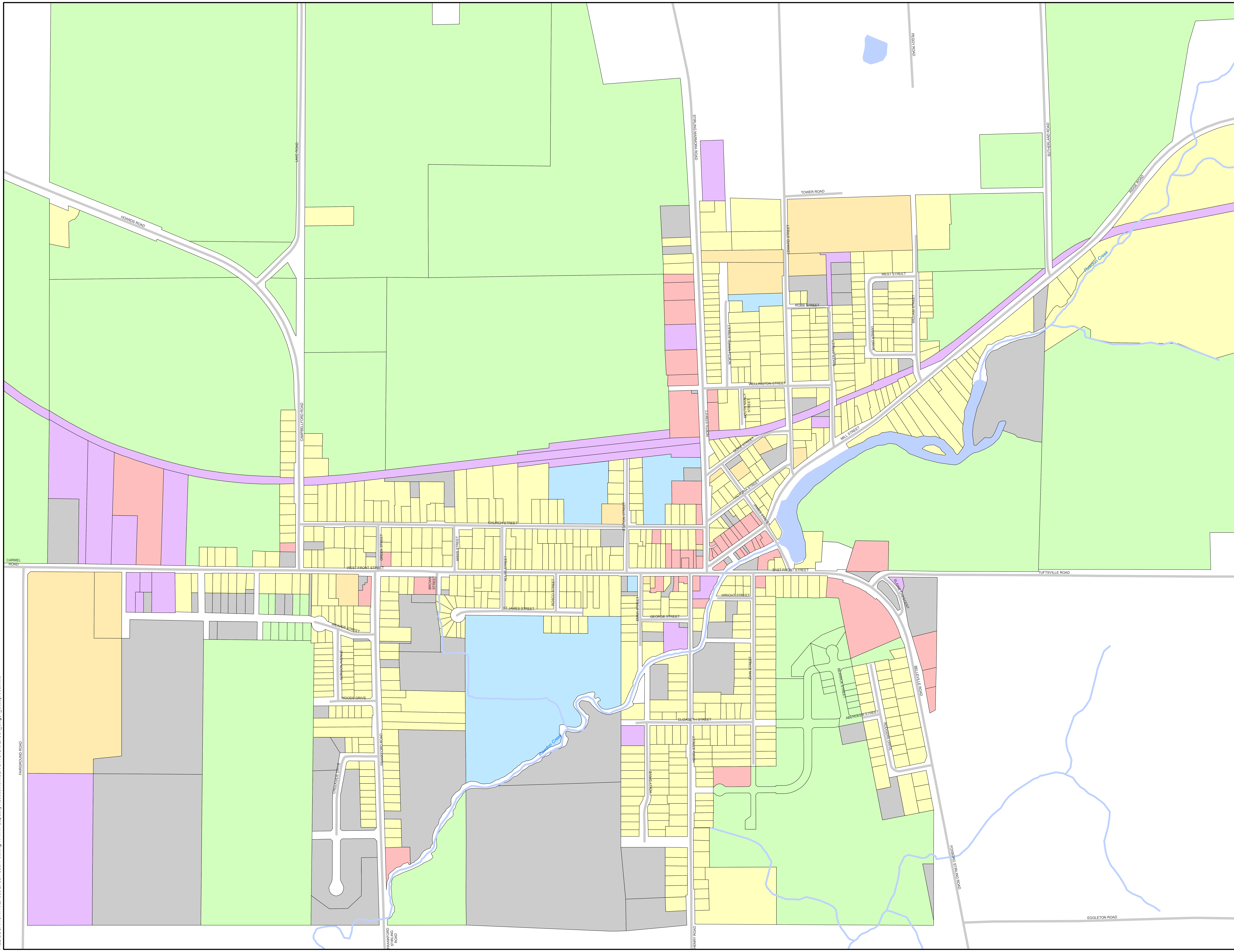
2.1 Study Area and Planning Periods

This Water & Wastewater Capacity Assessment considers the Study Area to be the entire boundary of the urban area of the Town of Stirling within the Municipality. Refer to Figure 1 for Study Area.

Future development areas are also considered as part of this report. The planning periods considered as part of this servicing assessment are as follows:

- Existing
- Short-term
- Mid-term
- Full build-out

Timeframes have been assigned to each of the three future planning periods but are approximate and should be based on the level of development actual occurring.



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MPAC Property Code

- No Information
- Vacant
- Farm
- Residential
- Commercial
- Industrial
- Institutional
- Special & Exempt

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PROJECT:

WATER AND WASTEWATER CAPACITY ASSESSMENT

STIRLING, ONTARIO

DRAWING:

STUDY AREA

DESIGN: CT

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FIGURE 01

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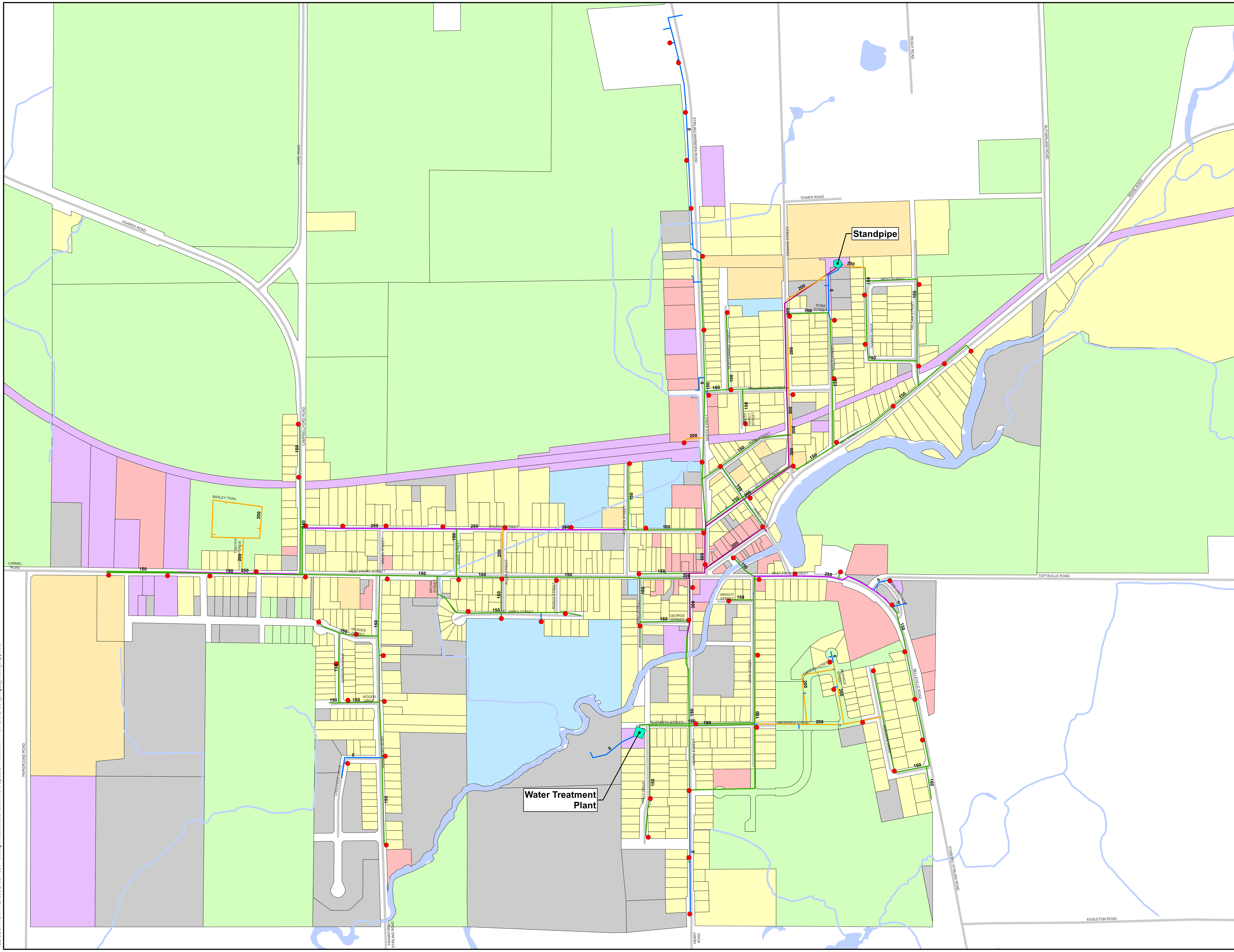
2.2 Existing Infrastructure

Wastewater
The sanitary system consists of gravity sewers conveying flow to five (5) pumping stations across the Town. Pumping station and forcemain upgrades were completed at Annis Street and Frankford Road SPS in 2011. Two lower capacity pumping stations, Henry Street SPS and Rogers Drive SPS service small areas and haven't had any major changes since their initial construction. All sewer lines terminate at the final and largest George Street SPS, which recently received capacity upgrades in 2022. George Street SPS pumps all of the town's wastewater into the town's sewage treatment lagoon.

Water
The Water Supply Plant located at the west end of Elizabeth Street, which runs four wells, provides the town's water supply. The Town resumed operation of the Water Treatment Plant in 2016 from Ontario Clean Water Agency (OCWA). In 2013, petroleum hydrocarbon odours were noted in well #5 and was subsequently taken offline. Well #4 was also taken offline as a precaution due to its close proximity to Well #5. Well #5 was returned to production in September 2017 and Well #4 returned to production in early 2018. A new well (Well #6) recently came online after its pilot well was drilled in 2018. Additionally, the Standpipe had spot repair done at the base of the structure in 2013.

Stormwater
The storm sewer system services a large portion of the town and has been upgraded to varying degrees over the years as road reconstructions have taken place. The un-serviced portion of the town relies on ditching for its stormwater management. Currently there are approximately 15 outlets into the Rawdon Creek.

Figures 2, 3 & 4 depict the existing systems, respectively.



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WATER AND WASTEWATER CAPACITY ASSESSMENT

STIRLING, ONTARIO

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EXISTING WATER SYSTEM

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FIGURE 02

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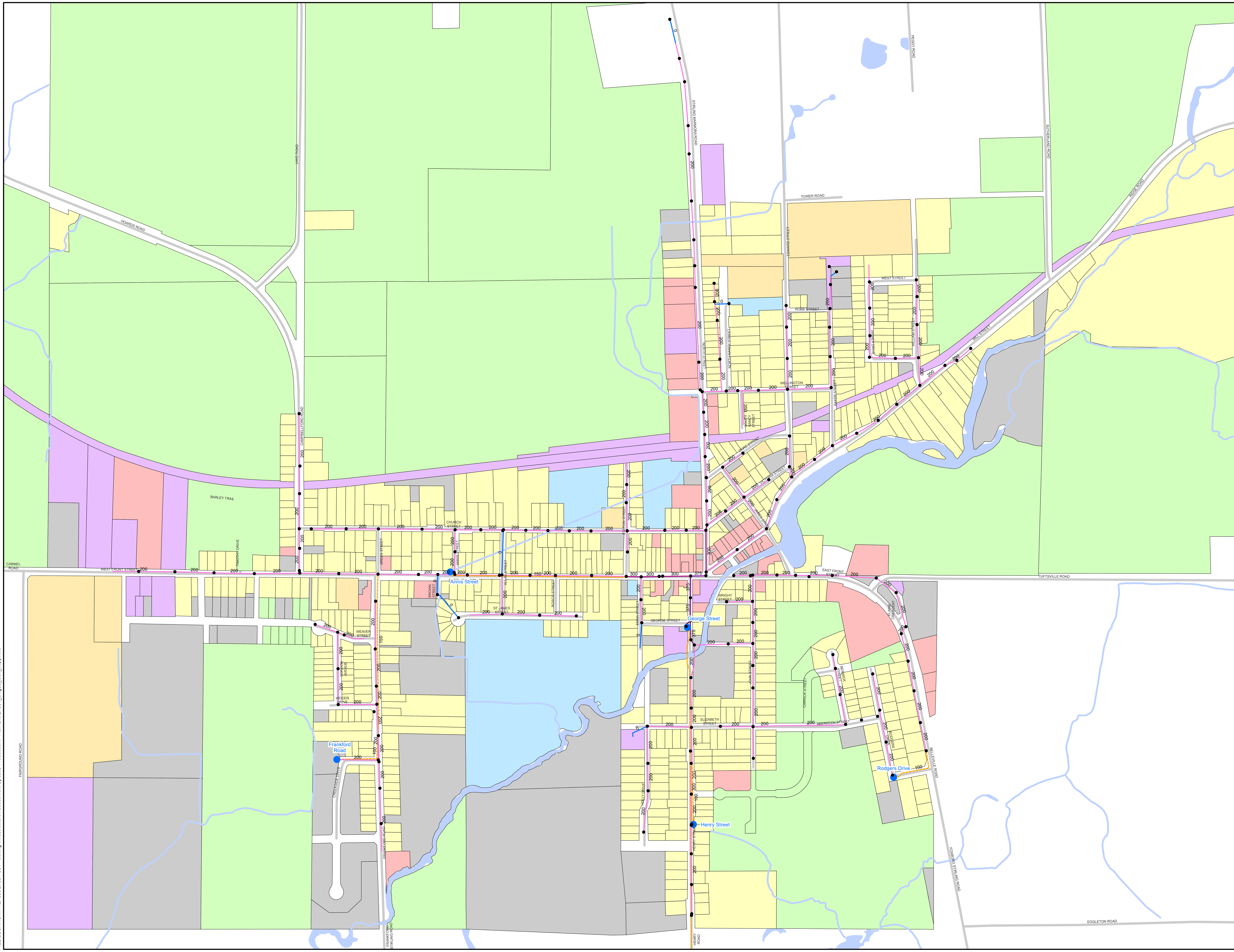

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MPAC Property Code

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- Vacant
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WATER AND WASTEWATER CAPACITY ASSESSMENT

STIRLING, ONTARIO

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EXISTING SANITARY SYSTEM

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FIGURE 03

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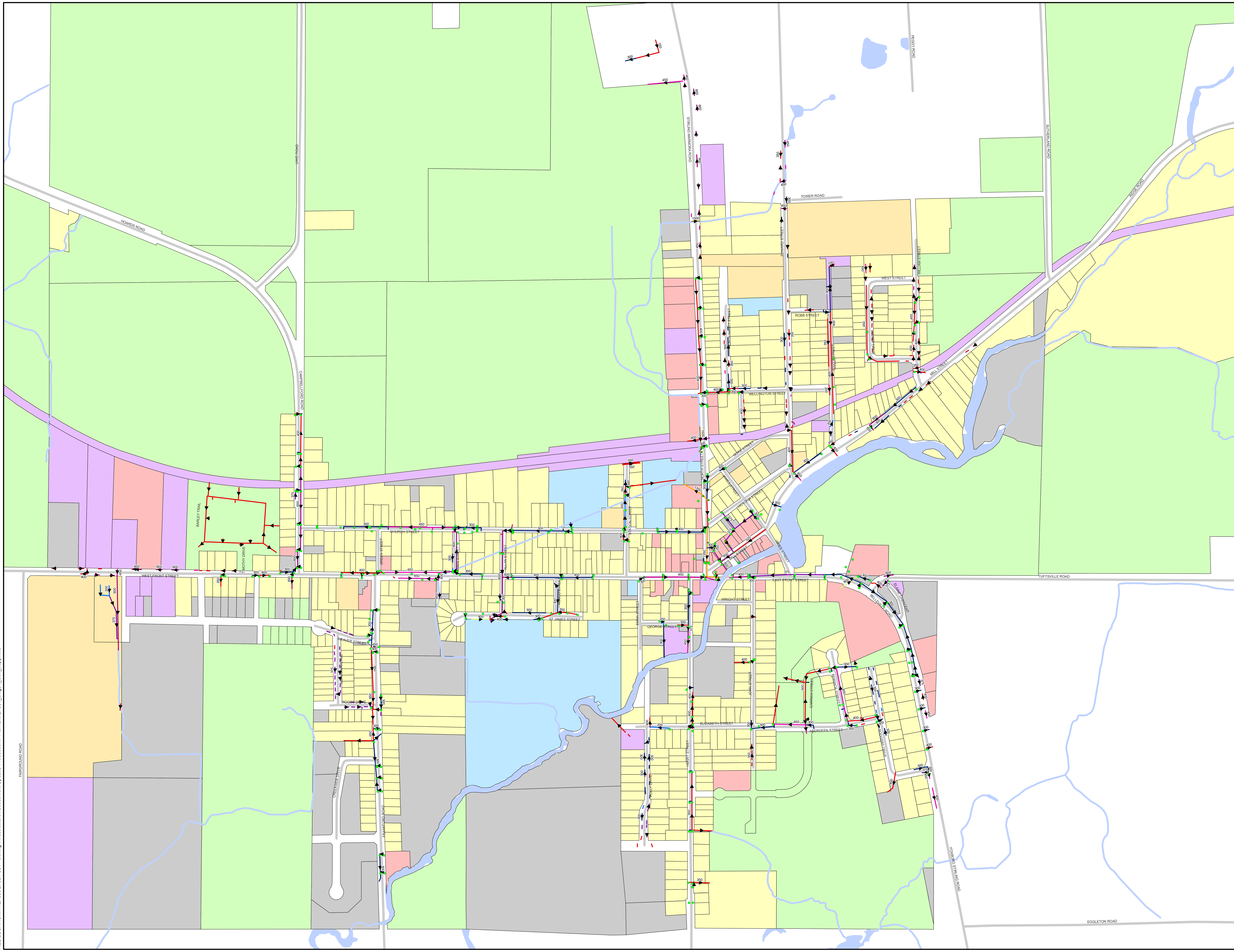

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MPAC Property Code

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WATER AND WASTEWATER CAPACITY ASSESSMENT

STIRLING, ONTARIO

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EXISTING STORMWATER SYSTEM

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FIGURE 04

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2.3 Population Projections

Based on the StatsCan 2021 Census, the Town currently has a population of 2,074 residents and from 2016 to 2021 grew by 2.2%. Discussions with the Town have confirmed that this stagnation of growth was due largely to the Town's restrictions on adding new developments. Development was restricted due to the lack of sewage capacity at the sewage lagoon. Capacity is now available at the lagoon due to an upgrade which was completed in February of 2015.

The Town has identified future land areas that are expected to be developed over the short to long term. This future growth provided by the Town was divided into three scenarios. The various areas proposed for future development are shown on Figure 5 and Table 1 illustrates their respective areas and projected populations. The Short Term Growth scenario includes areas 1, 4, 9, 10, 14 and 16 which are subdivisions currently in their final development or construction stage. The Medium Growth scenario includes areas 3, 5, 6, 8, 9, and 11 which includes developments that have been proposed to the Town or are currently delayed. The final Full Build Out scenario includes all of the proposed development lands with a full buildout of all properties.

Based on discussions with the Town for planning purposes, the short-term growth scenario would be growth anticipated over the next 10 years to 2033. The medium-term over the next 20 years to 2043 and the full build out scenario would be growth anticipated beyond the 20 year timeframe.

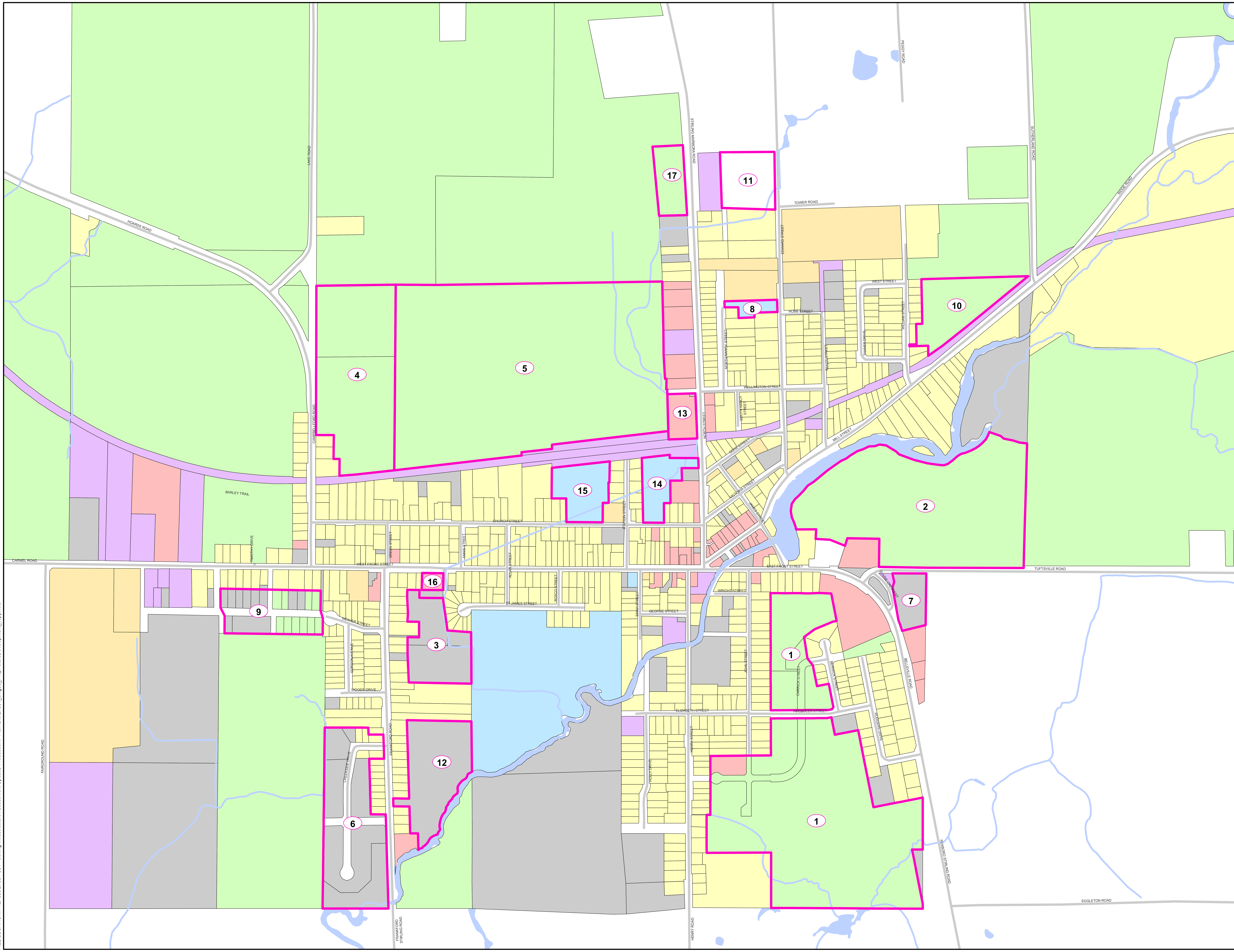
In Table 1, residential density of 13 units/ha was utilized. Although the most current development (Area 4 - Campbellford Rd.) is a subdivision with a density currently proposed at approx. 25 units/ha, applying that density to all developments would have a significant impact on the population growth for the future (approx. 245% growth rate). A population growth of this magnitude is both un-realistic and not feasible to for the Town's infrastructure. To more reasonably account for future unknown development lots, a medium residential density of 13 units/ha was utilized.

In addition, assessing the Town's building permits over the last five years, while excluding an outlier year of 2018 due to covid limited development, an average of 30 building permits were issued per year. Applying this estimation would provide roughly 720 people over the Short-Term period, while with a residential density of 13 units/ha, a population growth of 1085 is estimated for the Short-Term Period. With the additional push the Town is receiving from developers, this indicated that the residential density of 13 units/ha is reasonable.

Table 1: Proposed Development

Proposed Development				
Area	**Location**	**Area [Ha]**	**New Units**	**Projected Population(1)**
	Short-Term Growth			
1	Ryell Subdivision	11.10(2)	100(3)	240
4	Property on Campbellford Rd.	9.24	215	516
9	Weaver Street Extension (1)	2.91	5	12
10	Mill and William Street	2.96(4)	80	192
14	Old School (1)	1.38	87	209
16	Old Black Dog Restaurant	0.24	29	70
N/A	Unassigned Growth	N/A	30	71
	TOTAL	**27.83**	**546**	**1310**
	Mid-Term Growth			
3	Hilden Homes	2.92	51	122
5	Thompson Farmland(4)	29.66(4)	386	926
6	Dorann Holmes	6.77	45	108
8	Stirling Manor Expansion	N/A	N/A	56
9	Weaver Street Extension (2)	1.38	14	34
11	Edward Street	2.03(4)	19	46
N/A	Unassigned Growth	N/A	31	75
	SUBTOTAL	42.76	546	1367
	TOTAL	**70.59**	**1092**	**2676**
	Full Build Out			
2	Spry/Cleaver property	15.17(4)	197	473
7	Old Brown Shoe Property (4)	1.09(4)	14	34
12	Frankford Road	4.01(4)	52	125
13	Stirling Manor Expansion (2)	0.84	N/A	96(5)
15	Old School (2)	1.85	117	280
17	Stirling-Marmora Road	1.88	24	58
N/A	Unassigned Growth	N/A	26	61
	SUBTOTAL	24.84	429	1126
	TOTAL	**95.43**	**1521**	**3802**

(1) Population density of 2.4 ppl/unit was used
(2) Area prorated based on units already constructed
(3) Proposal for 124 units, 24 units have been built to-date
(4) A Medium Residential Density of 13 units/ha was used
(5) Included for potential relocation



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Future Development

MPAC Property Code

- No Information
- Vacant
- Farm
- Residential
- Commercial
- Industrial
- Institutional
- Special & Exempt

Future Development

Parcels	Area_Ha
1	19.22
1	3.89
2	15.17
3	2.92
4	9.24
5	29.67
6	6.77
7	1.09
8	0.42
9	2.88
10	2.94
11	2.01
12	4.01
13	0.84
14	1.41
15	1.85
16	0.24
17	1.33

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WATER AND WASTEWATER CAPACITY ASSESSMENT

STIRLING, ONTARIO

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PROPOSED FUTURE DEVELOPMENT

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FIGURE 05

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Based on the development identified and standard design criteria, Table 2 details the projected average day flow (ADF) and infiltration:

Table 2: Future Average Day Flow and Average Day Demand

Future ADF/ADD			
Area	**Location**	**ADF[(1)] [L/s]**	**WW Infiltration[(2)] [L/s]**
Short-Term Growth			
1	Ryell Subdivision	0.83	1.55
4	Property on Campbellford Rd.	1.79	1.29
9	Weaver Street Extension (1)	0.04	0.41
10	Mill and William Street	0.67	0.41
14	Old School (1)	0.73	0.19
16	Old Black Dog Restaurant	0.24	0.03
N/A	Unassigned Growth	0.25	N/A
	SUBTOTAL	4.55	3.90
Mid-Term Growth			
3	Hilden Homes	0.43	0.41
5	Thompson Farmland[(4)]	3.22	4.15
6	Dorann Holmes	0.38	0.95
8	Stirling Manor Expansion	0.19	N/A
9	Weaver Street Extension (2)	0.12	0.19
11	Edward Street	0.16	0.28
N/A	Unassigned Growth	0.26	N/A
	SUBTOTAL	4.74	5.99
	TOTAL	9.29	9.88
Full Build Out			
2	Spry/Cleaver property	1.64	2.12
7	Old Brown Shoe Property [(4)]	0.12	0.15
12	Frankford Road	0.43	0.56
13	Stirling Manor Expansion (2)	0.33	0.12
15	Old School (2)	0.97	0.26
17	Stirling-Marmora Road	0.20	0.26
N/A	Unassigned Growth	0.21	N/A
	SUBTOTAL	3.70	3.48
	TOTAL	13.20	13.36

(1) 300 L/cap/day was used to develop ADF
(2) 0.14 L/s/ha was used to develop new WW Infiltration

2.4 Land Use

Land use in the Town is comparable to other similarly sized towns in Ontario. The land use is predominantly residential with some areas of commercial, industrial and institutional land distributed across the town. Industrial lands exist along the west end of West Front Street and commercial lands are centred around the downtown core and along North Street and East Front Street.

3.0 Identification and Evaluation of Servicing Strategies

One of the objectives of this Capacity Assessment is to develop and evaluate possible servicing strategies for both water, wastewater and stormwater infrastructure. All reasonable potential solutions to the problem are typically considered. Servicing strategies are examined in sufficient detail to allow conclusions to be drawn and to move forward to the next stage of the project. Capacity Assessments for infrastructure generally result in the identification and review of a broad range of options.

3.1 Evaluation Methodology

The evaluation process for the Infrastructure Capacity Assessment consisted of a review of the potential servicing strategies in consideration of the criteria described in the table below.

Table 3: Evaluation Criteria

Criteria	Description
Natural Environment Considerations	Natural features, natural heritage areas, Areas of Natural and Significant Interest, designated natural areas, watercourses and aquatic habitat
Social and Cultural Environment Considerations	Proximity of facilities to residential, commercial and institutions, archeological and cultural features, designated heritage features, well or wellhead protection areas, land-use and planning designations
Technical Feasibility	Constructability, maintaining, or enhancing drinking water quality, maintaining or enhancing wastewater treatment, reliability and security of systems, ease of connection to existing infrastructure and operating and maintenance requirements
Financial Considerations	Capital and Operational costs

In order to qualitatively evaluate any servicing alternatives, each of the criteria presented in the following sections were assessed in a descriptive manner rather than a quantitative manner. Rather than having a numerical or weighted ranking system, the evaluation focuses instead on the strengths and weaknesses of each servicing alternative to identify the preferred alternative.

For each evaluation criterion and for each system alternative, the potential effects on the environment were identified and evaluated relative to the other alternatives as being most preferred, less preferred and least preferred. The evaluation is based on the relative advantages and disadvantages of the potential effects for each system alternative.

3.2 Cost Estimates

All opinion of probable costs associated with the preferred alternatives were completed in 2023 dollar value. These costs are based on a Class 'D' estimate class, which is generally defined as follows:

- Work Definition: A description of the intended solutions with such supporting documentation as is available (definition of project typically in the order of 1% to 5%).
- Intended Purpose: To aid in the screening of various options prior to recommending a preferred solution.
- Level of Effort: Limited and expected accuracy could range from -15% to +30%.

It is noted that a mark-up has been applied to base construction cost estimates to account for items such as engineering, permits, approvals, construction overhead, building and site works, field investigations, etc.

4.0 Potable Water System

4.1 Existing Potable Water System

The existing potable water system is comprised of five functioning groundwater wells located along the south creek-bed in the 100-year floodplain of the Rawdon Creek, an elevated water storage standpipe and an assortment of sized watermains.

The five groundwater wells in operation identified as 1, 3, 4, 5 and 6 have a total Permit to take water (PTTW) rated capacity of 2,687 m^3/day and services a population of approximately 2000 people (BluMetric 2018).

Wells 4 and 5 have had earth fill placed around them to protect against flooding. The water that is collected from the aquifer is classified as Groundwater Under the Direct Influence of surface water (GUDI) with adequate filtration in place. The aquifer is comprised of a sand and gravel filled bedrock valley. Wells 4 & 5 are located approximately 50 m from Rawdon Creek and Wells 1 & 3 are located approximately 60 m from Rawdon Creek. There was a water shortage in 2016 and there have been concerns that the current system is unsatisfactory (BluMetric 2018). Well #6 came online recently and was installed southeast of Wells 4 and 5 and is approximately 110 m from Rawdon Creek.

4.1.1 Groundwater Well Number 1

Well #1 has a depth of 6.1 m and is constructed with a 1.5 m diameter corrugated steel cased dug well that is perforated in the bottom 3 m (BluMetric 2018). It has operated since at least 1958 and there hasn't been any noted reduction in yield over that time period. The well has had a video inspection in April 2017 and showed that it was in good condition with some hardness in

the lower section. The well currently operates with a maximum flow rate of 576 m^3/day which is roughly 44% of it's rated PTTW maximum allowable of 1,305 m^3/day.
Well #1 is located in the basement level of the pump house and in peak high-water table conditions the existing sump hole system cannot control artesian flow into the building. The Township has indicated they rely heavily on Well #1.

4.1.2 Groundwater Well Number 3

Well #3 extends to the limestone bedrock and has a depth of 12 m and a 0.25 m stainless steel casing with a screen located from 9.75 m to the bottom of the well (BluMetric 2018). It has operated since at least 1986 and there has not been any noted reduction in yield over that time period. The well screen was last cleaned in February 2009. The well currently operates with a maximum flow rate of 598 m^3/day which is roughly 46% of it's rated PTTW maximum allowable of 1,305 m^3/day.

4.1.3 Groundwater Well Number 4

Well #4 is a 400 mm x 200 mm diameter drilled well with 2.8 m of 200 mm diameter well screen within a 200 mm diameter gravel pack. The bottom 2.4 m is installed within the limestone bedrock. When the well was commissioned in 1994 it rapidly experienced a decline in well yield soon after entering operation.

In November 2013, the well was taken offline after petroleum hydrocarbon odours were reported for the well water, various hydrocarbon sources have been investigated by Greer and Galloway Group in 2016 but there was no conclusion on the source of the contamination. After extensive rehabilitation and efforts to increase well yield, the well has been brought back into service and as of January 2018 has operated with a maximum flow rate of 825 m^3/day which is roughly 42% of it's rated PTTW maximum allowable of 1,944 m^3/day. Well pumping rate maximum flow rates were historically not achieved due to the flow demand on the water system. Well #4 has previously had some fowling issues plugging the screen with iron and manganese, however this was fixed due to chemical cleaning and bailing. There are currently no issues with the flow rate at this well.

4.1.4 Groundwater Well Number 5

Well #5 is a 150 mm diameter drilled well and is located 3 m from Well #4. The well depth extends to the limestone bedrock and the bottom 2.4 m is installed with a telescoping stainless steel well screen to the limestone bedrock. Well #5 has historically outperformed Well #4.

In November 2013, the well was taken offline with Well #4 after petroleum hydrocarbon odours were reported for the well water. The well was brought back into operation in September 2016. The well has operated with a maximum flow rate of 343 m^3/day which is roughly 53% of it's rated PTTW maximum allowable of 648 m^3/day. Due to Well #5's proximity to Well #4 the two well's frequently influence the yield of the other and maximum flow rates cannot be reached when both are in operation concurrently.

4.1.5 Groundwater Well Number 6

There was a study conducted by Blumetric Engineering in February 2018 that investigated the potential for installing a new groundwater well outside of the 100-year floodplain that wouldn't be as impacted by GUDI concerns. As a result of this study a pilot well was installed with an outer diameter (OD) of 168 mm and a depth of 13.57 m, this well was installed with a recommended maximum pumping rate of 817 m^3/day. This well has recently come into service.

A table summarizing the operational characteristics of the wells is shown below.

Table 4: Groundwater Well Operating Conditions

Well	Year Constructed	Depth (m)	Size (mm)	PTTW (m^3/day)	Peak Production (m^3/day)
1	1958	6.1	1500	**2687**	576
3	1986	12	250		598
4	1994	12	200		825
5	1991	12	150		343
6	2020	12	150		817
				TOTAL	**3,159**

The water distribution system includes a standpipe water storage tank (2.6 ML nominal capacity) and piping network. The standpipe is located in the northeast quadrant of the Town at the end of Baker Street. The piping network generally consists of polyvinyl chloride, ductile iron and cast iron piping ranging in size from 50 mm to 300 mm in diameter. It is understood that some of the piping is the original infrastructure dating back to 1950's and earlier.

4.2 Historic Potable Water Demands

The first aspect to understanding the water system is the current demand on the system; Average Day Demand (ADD) & Max Day Demand (MDD). Water usage in the Town is metered at the Water Treatment Plant (WTP). Results for the past five years of which can be found in Table 5:

Table 5: Potable Water Demands

Existing Water Demand															
	2022			2021			2020			2019			2018		
	MDD	ADD	MDF	MDD	ADD	MDF	MDD	ADD	MDF	MDD	ADD	MDF	MDD	ADD	MDF
Jan	901	692	1.30	902	566	1.59	813	622	1.31	840	592	1.42	932	692	1.35
Feb	933	691	1.35	795	628	1.27	733	586	1.25	792	625	1.27	938	675	1.39
Mar	849	720	1.18	730	603	1.21	934	692	1.35	785	634	1.24	781	651	1.20
Apr	975	722	1.35	741	545	1.36	1268	688	1.84	777	580	1.34	722	573	1.26
May	961	686	1.40	1186	691	1.72	789	584	1.35	730	588	1.24	858	668	1.28
Jun	1416	830	1.71	1306	786	1.66	775	638	1.21	1167	737	1.58	888	710	1.25
Jul	992	738	1.34	904	629	1.44	1290	767	1.68	1317	711	1.85	895	687	1.30
Aug	1033	749	1.38	903	654	1.38	720	576	1.25	1194	992	1.20	1003	648	1.55
Sep	1029	747	1.38	825	645	1.28	641	573	1.12	1462	1051	1.39	812	597	1.36
Oct	1057	832	1.27	879	625	1.41	850	572	1.49	1177	640	1.84	1026	609	1.68
Nov	1050	710	1.48	788	603	1.31	673	583	1.15	681	565	1.21	878	602	1.46
Dec	847	642	1.32	795	664	1.20	770	603	1.28	757	611	1.24	796	628	1.27
MAX	1416	730	1.71	1306	637	1.72	1290	624	1.84	1462	694	1.85	1026	645	1.68

The two highlighted values in each year represent seasonal high usage. Using these seasonal highs, the existing daily usages were taken and the representative Average Day Demand and Max Day Demand were determined. Based on the operational data, the typical water demand was summarized as in Table 6.

Table 6: Typical Water Demands

Typical Water Demand		
666	m3/day	ADD (5yr)
1462	m3/day	MDD (2018 - 2022)
1.85		MDD factor (Ext Data)
2.7		PHF (MOECC for MDF 1.8)

4.3 Potable Water System Design Criteria

Water pumping stations or wells are rated on their 'firm' pumping capacity. The MOECC Guidelines for Drinking Water Systems (MOECC, 2008) defines firm capacity as the "Capacity of the raw water pumping station able to supply the water treatment plant design capacity with the largest unit out of service." This allows for the continuity of service in the event that one of the pumps experiences mechanical issues. Treated water and booster pumping stations are also rated based on their firm capacity defined as the capacity of the station with the largest pump out of service. In pressure zones that do not have adequate floating storage and the treated water pumped is the sole source of water supply then firm capacity is defined as the capacity of the pumping station with the two largest units (including the fire pumps, if any) out of service. Since the Township has adequate storage and uses high lift pumps to pump raw water to the water treatment plant, the required firm capacity is only for the largest pumping unit out of service.

Pumping stations or well systems are sized based on maximum day flows for areas with sufficient water storage volume, and on peak hour flows for areas without sufficient storage. Storage capacities are based on MOECC Guidelines for Drinking Water Systems (MOECC, 2008). The total storage capacity requirements for a pressure zone are the sum of the equalization storage, fire storage, and emergency storage allowances.

The assessment used a land based approach to assess the distribution systems capacity, with industrial commercial and institutional lands having a greater demand than residential areas. The Max Day Demand that is typical for the Town of Stirling was used to assess the systems distribution capacity. The design used a conservative 25% of the MDD at any given point in the system to assess the areas ability to handle a worst case scenario.

Watermains are to be sized to carry the greater of the maximum day plus fire flow or peak hour demand. The MOECC Guidelines (MOECC, 2008) recommend the following range of acceptable pressures:

- Under normal conditions (average to peak hour flows) 275 kPa (40 psi) to 690 kPa (100 psi),
- During fire flow conditions pressures a minimum of 140 kPa (20 psi)

4.3.1 Fire Flow Demand

The process to determine the Fire Flow demands and duration on a Town scale is typically determined through one (1) of two (2) ways, either the use of the MOECC Drinking Water Guidelines (DWG) or the Fire Underwrites Survey (FUS).

The MOECC Drinking Water Guidelines (DWG) (MOECC, 2008) is a population based approach and provides a general fire flow target for an entire area. The MOECC provides typically suggested fire flows and durations for small municipalities but indicates that the latest edition of the FUS document should be consulted. Based on the Town of Stirling's populations (2000 ppl) the MOECC DWG criteria indicates at Fire Flow of 95 L/s with a duration of 2 hrs.

The FUS method is a formula based approach to calculate the target fire flows (FF) based on the specifics of a site. Construction type, floor area, building use, exposure distance and fire suppression system (i.e. sprinklers) are considered in the calculations. In order to calculate the FUS target fire flows on a Town scale, a land use based approach was developed. This approach entailed reviewing a representative sample of the different land use types to determine an average fire flow: Based on the different residential and Industrial, Commercial & Institutional (ICI) land uses the flowing fire flows were determined:

Table 7: Typical Fire Flow Demands

Typical Fire Flow	
Residential	50 L/s
ICI	100 L/s

As the FUS method provides a more Town specific demand on the system, the FUS fire flow demands were used in the analysis.

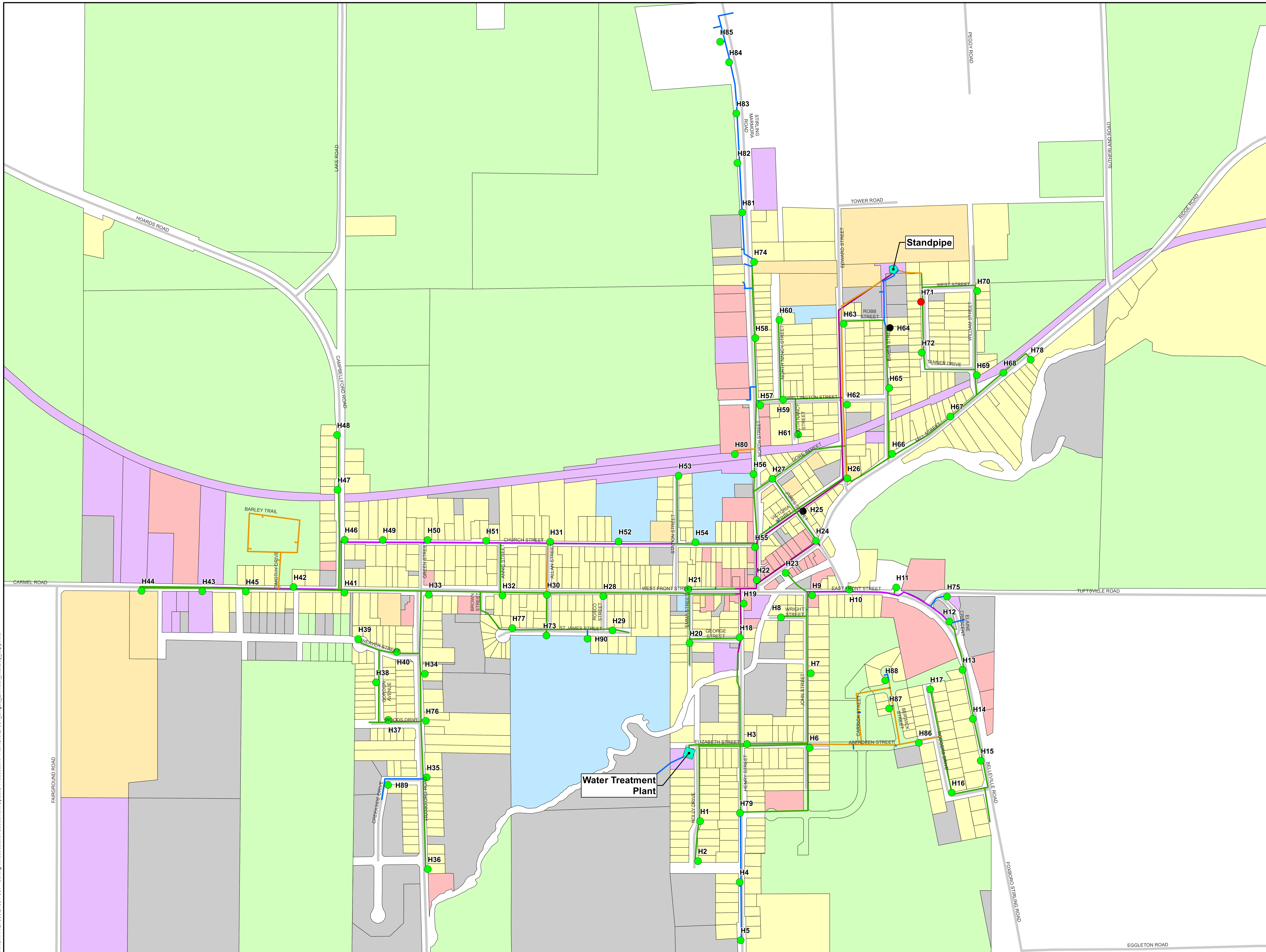
It should be noted that the above noted targets would be applied (most stringent target applied for multi-zoned areas) to determine if there is a difference or gaps between the available capacity and the above noted targets. Targets may not be achieved due to limitation of the existing system.

4.4 Water Distribution System

The existing and future requirements for the potable water system are shown in Figure 6 through 9. Figure 6 and 7 show the existing water system under Max Day Demand (MDD) and MDD + Fire Flow (FF) conditions. It can be seen from the Figure 6 that under MDD the system can meet the demand within the required pressures. The MDD + FF under the existing system does generally provide the required demand and pressure with the exception of two (2) dead end areas, North St and Belleville Rd. These conditions have recently been approved, and potentially rectified at some hydrant locations due to the recent downtown reconstruction project and looping completed through the Ryell subdivision. Updated hydrant testing is required to confirm locations that are still inadequately serviced for MDD + FF.

Figure 8 illustrates the MDD condition with all of the proposed developments. Previous hydrant testing across the Town shows that under MDD conditions, all areas in the distribution system are able to achieve the required 140KPa (40PSI) minimum operating pressure for the Full Build Out study period.

Figure 9 shows the proposed potable water system under MDD + FF conditions with all of the proposed developments. Most future developments have sufficient flow under current infrastructure conditions with the exception of areas 5, 11, and 17, the Campbellford Rd. Subdivision, Edward St. Development, and northern Stirling-Marmora Rd. development, respectively. These locations lack available pressures (minimum of 20 psi) under the required MDD + FF conditions due to elevation increases, as well as increased distances with a smaller pipe network creating too sufficient of pressure losses.



NOTE: MDD WAS ASSUMED TO BE 6 L/S AT EACH HYDRANT

Legend

Standpipe

Water Supply Plant

Fire Hydrant

Flows at 275Kpa (40psi)

- No Info
- Green - Sufficient Capacity
- Red - Over 20Kp under capacity

Watermain (Diameter)

- No Info
- 150
- 200
- 250
- 300

MPAC Property Code

- No Information
- Vacant
- Farm
- Residential
- Commercial
- Industrial
- Institutional
- Special & Exempt

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PROJECT:

WATER AND WASTEWATER CAPACITY ASSESSMENT

STIRLING, ONTARIO

DRAWING:

EXISTING WATER SYSTEM MAX DAY DEMAND

DESIGN:	CT
DRAWN:	KTK
CHECKED:	MM
JLR #:	27931

DRAWING #:

FIGURE 06

PLOT DATE: November 1, 2023 3:50:20 PM

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PROJECT:

DRAWING:

EXISTING WATER SYSTEM
MAX DAY DEMAND / FIRE FLOW

DESIGN:

DRAWN:

CHECKED:

JLR #:

DRAWING #:

FIGURE 07

FILE LOCATION: P:\27000\27931-001 - Stirling Infrastructure Assessment Update\5-Production\7-Plan\GIS\27931_dwg07_ExWater_MaxDayFF_copy.mxd

PLOT DATE: November 2, 2023 10:45:35 AM

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PROJECT:

DRAWING:

FUTURE WATER SYSTEM: FULL BUILD OUT MAX DAY DEMAND

DESIGN:

DRAWN:

CHECKED:

JLR #:

DRAWING #: **FIGURE 08**

FILE LOCATION: P:\27000\27931-001 - Stirling Infrastructure Assessment Updat\5-Production\7-Plan\GIS\27931_dwg08_FutWater_MaxDay_copy.mxd

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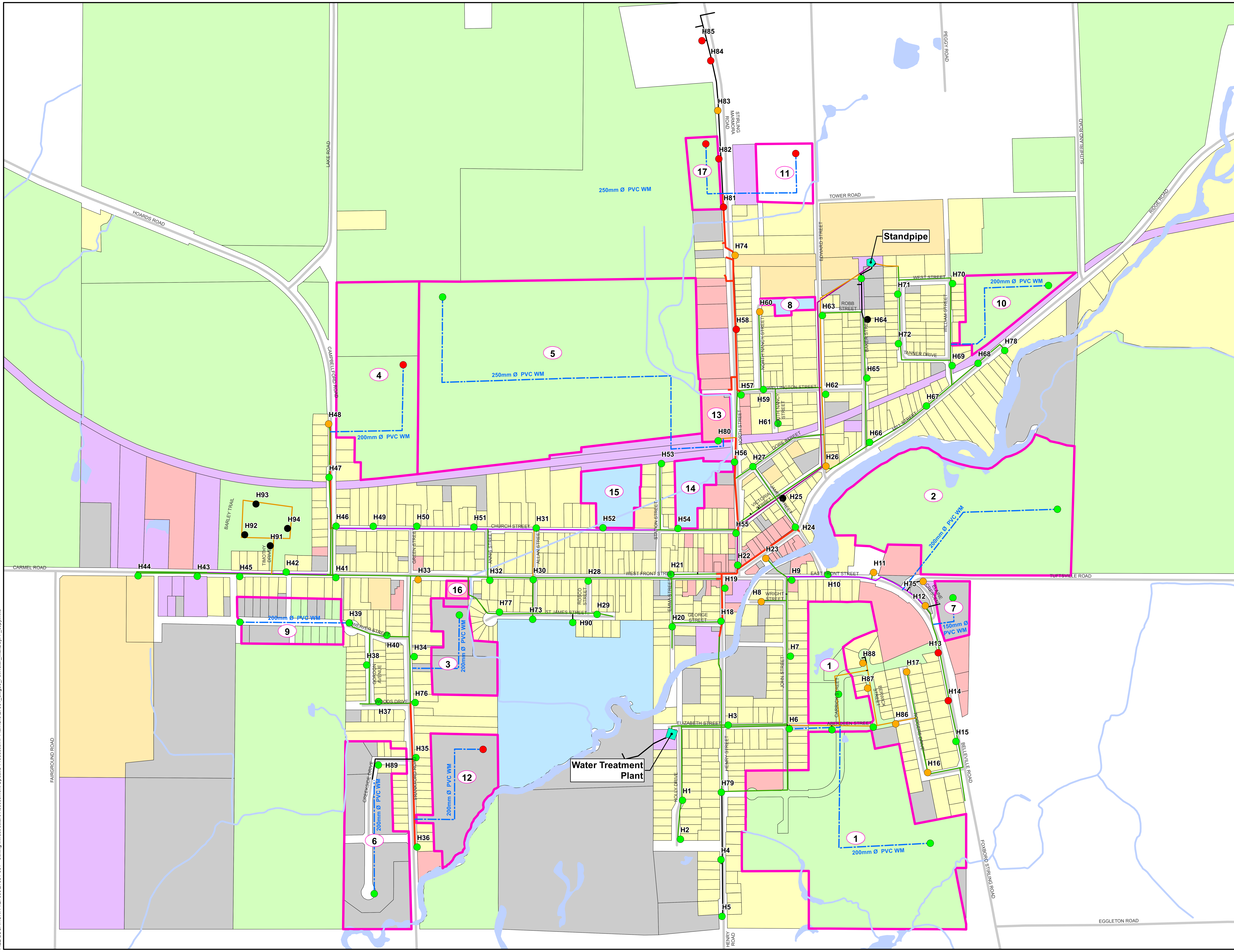

Legend

- Standpipe
- Water Supply Plant

Fire Hydrant

Flows at 140kpa (20psi)

- NoInfo
- Green
- Orange
- Red

Watermain (Diameter)

- No Info
- 150
- 200
- 250
- 300
- Undersized
- Future

Future Development

MPAC Property Code

- No Information
- Vacant
- Farm
- Residential
- Commercial
- Industrial
- Institutional
- Special & Exempt

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PROJECT:

DRAWING:

FUTURE WATER SYSTEM:
FULL BUILD OUT
MAX DAY DEMAND / FIRE FLOW

DESIGN:

DRAWN:

CHECKED:

JLR #:

DRAWING #:

FIGURE 09

FILE LOCATION: P:\27000\27931-001 - Stirling Infrastructure Assessment Update\5-Production\7-Plan\GIS\27931_dwg09_FutWater_MaxDayFF_copy.mxd

PLOT DATE: November 9, 2023 11:33:43 AM

4.5 Water Distribution Servicing Strategies

The distribution system was analyzed for sufficient Max Day Demands and Fire Flows using the results of hydrant tests. The existing system can accommodate the MDD across the Town as shown in the figures. However, when looking at the demand for MDD + FF there are some areas that do not meet the requirements.

The distribution system was also analyzed under the Full Buildout condition to evaluate the worst-case scenario. In order to improve the system and close the gap between the provided and required demand and pressure, the following distribution upgrades were identified:

- Aberdeen Street looping has now been completed by extending a 200 mm watermain from John/Elizabeth St. intersection to Aberdeen/Berwick St. intersection. Existing hydrants should be re-tested to determine if the industrial locations flows are adequate.

- Campbellford Road: by installing a 200 mm watermain on Campbellford Rd., north from Church St., the proposed development off Campbellford Rd. will be adequately serviced. With the existing watermain, the development will not achieve 20 psi under MDD + FF.

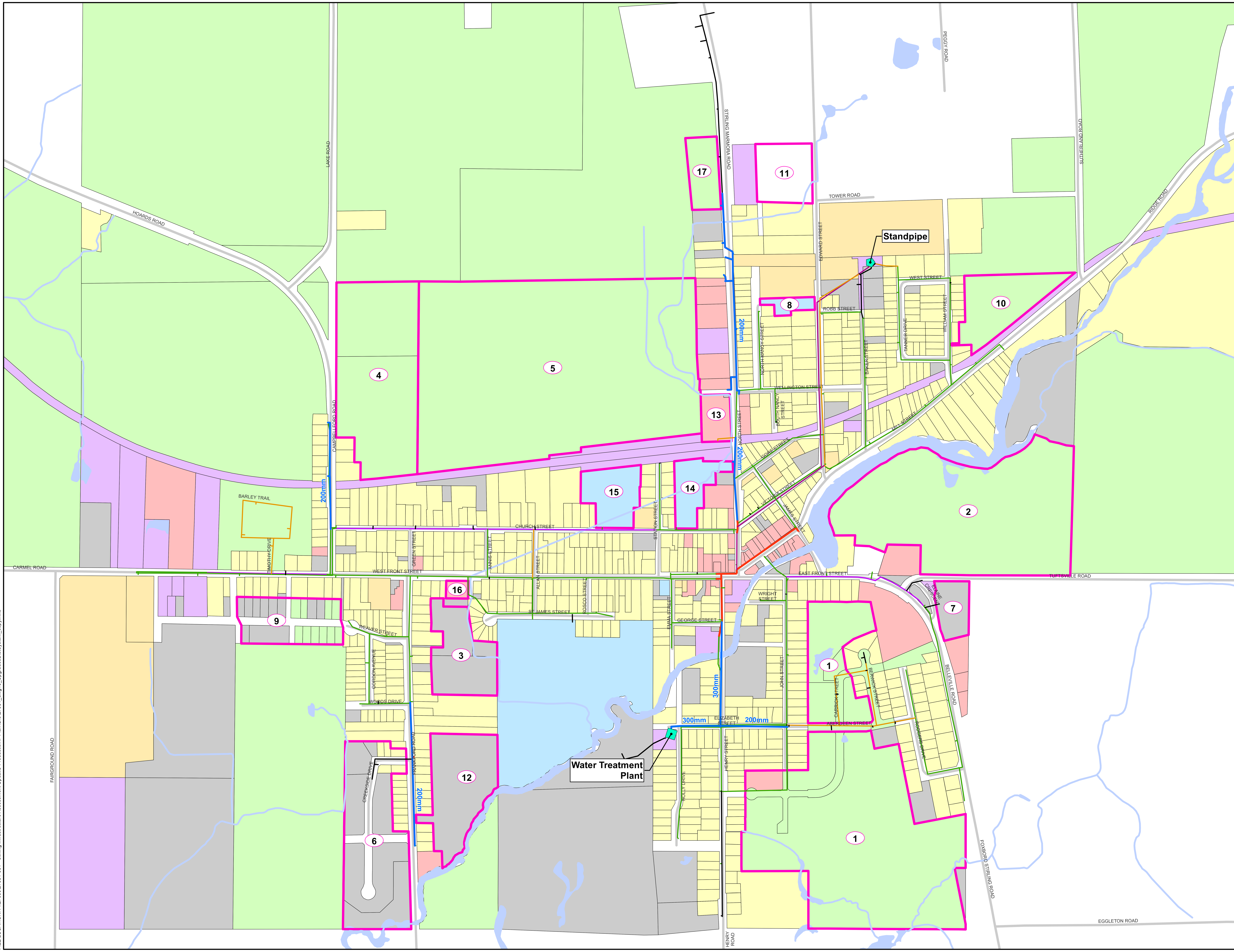
- Frankford Road: by installing a 200 mm watermain on Frankford Rd. the proposed development off Frankford Dr. will be adequately serviced. With the existing watermain, the development will not achieve 20 psi under MDD + FF.

- North Street: installing a 200 mm watermain from Victoria St. to the Thompson Farmland development connection location (Hydrant #80) should be completed to improve flows to the development as well as northern hydrants on Stirling-Marmora Rd. Additionally, a 200 mm watermain should be installed further up North St. as required in the future to service the Edward St. and northern Stirling-Marmora Rd. developments, as well as northern hydrants which currently have pressure deficits under fire flows. This may be solved with the recent upgrades on Henry, Front and North St. and should be investigated with updated hydrant tests.

- Completion of a Distribution Spine: Currently the water distribution system has a partially complete arterial water main to help facilitate flow through the system and specifically between the WTP and the standpipe. By increasing the watermain size in the remaining places, water can be distributed more evenly throughout the system.

    - Elizabeth St. Upgrades: by installing a 300 mm watermain from the WTP to Henry St. water flow will be increased to the entire water distribution network. By upgrading the watermain to 200mm from Henry St to John St in conjunction with the looping along Aberdeen St., flow will be improved to the entire south-eastern portion of the town where there are current deficits.

    - Henry St. Upgrades: the remainder of the installation of an upgraded 300 mm watermain from Elizabeth St. to Front St., when done in conjunction with the 300mm upgrades on Elizabeth St. will improve hydraulic flow to all of the town north of the Rawdon Creek. Additionally, since this line forms the spine of the system and feeds into the Standpipe, it will improve flow to the entire town.



Legend

- Standpipe
- Water Supply Plant

Proposed Watermain

Watermain (Diameter)

- No Info
- 150
- 200
- 250
- 300
- 999

Future Development

- 2

MPAC Property Code

- No Information
- Vacant
- Farm
- Residential
- Commercial
- Industrial
- Institutional
- Special & Exempt

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PROJECT:

DRAWING:

PROPOSED WATER SYSTEM UPGRADES DISTRIBUTION SYSTEM

DESIGN:

DRAWN:

CHECKED:

JLR #:

DRAWING #:

FIGURE 10

FILE LOCATION: P:\27000\27931-001 - Stirling Infrastructure Assessment Update\5-Production\7-Plan\GIS\27931_dwg10_UpgradesDistSystem_copy.mxd

PLOT DATE: November 9, 2023 11:49:27 AM

4.6 Water Supply and Treatment Servicing Strategies

The MOECC DWG (MOECC, 2008) indicates that plant capacity should be greater or equal to the Maximum Day Demand (MDD) with an allowance for water need for plant use. The current rated capacity of the Water Supply Plant is 2687 m^3/day. Figure 11 shows the MDD for the past 5 years in comparison to the Water Supply Plants rated capacity.

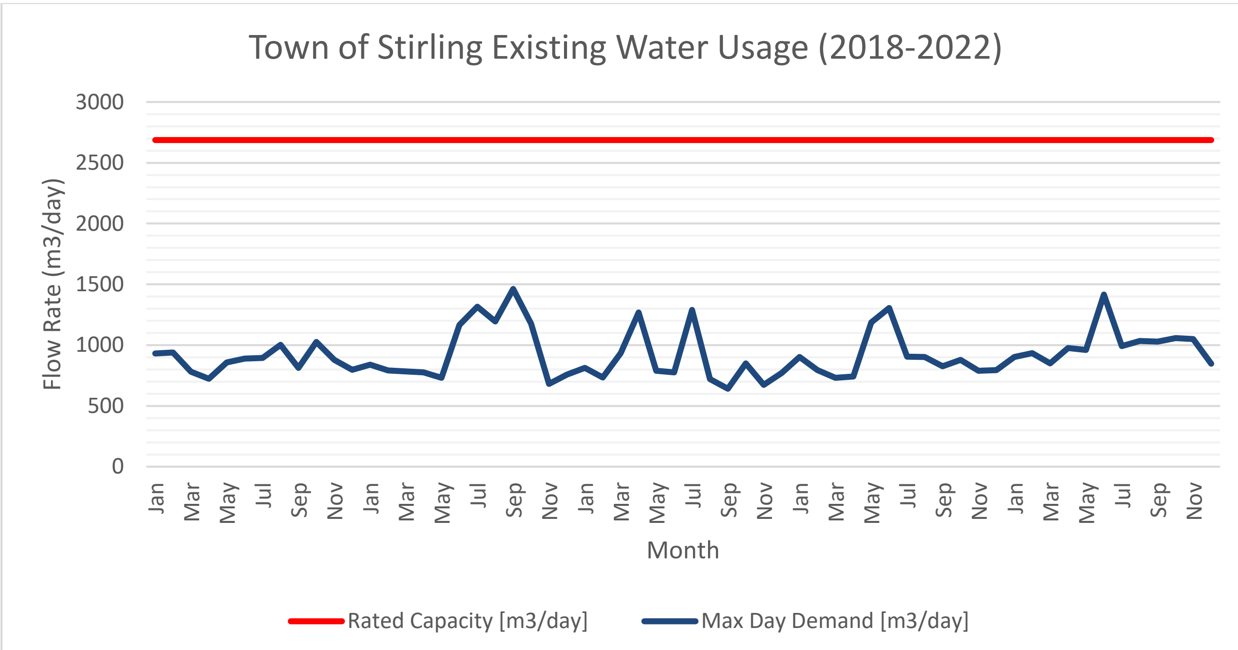


Figure 11: Existing Water Usage (2018-2022)

As shown above, the Town's water demand over the last five years did not surpass a MDD of 1462 m^3/day and typically has more than sufficient capacity.

Figure 12 presents the MDD vs. the plant capacities for each of the different future scenarios.

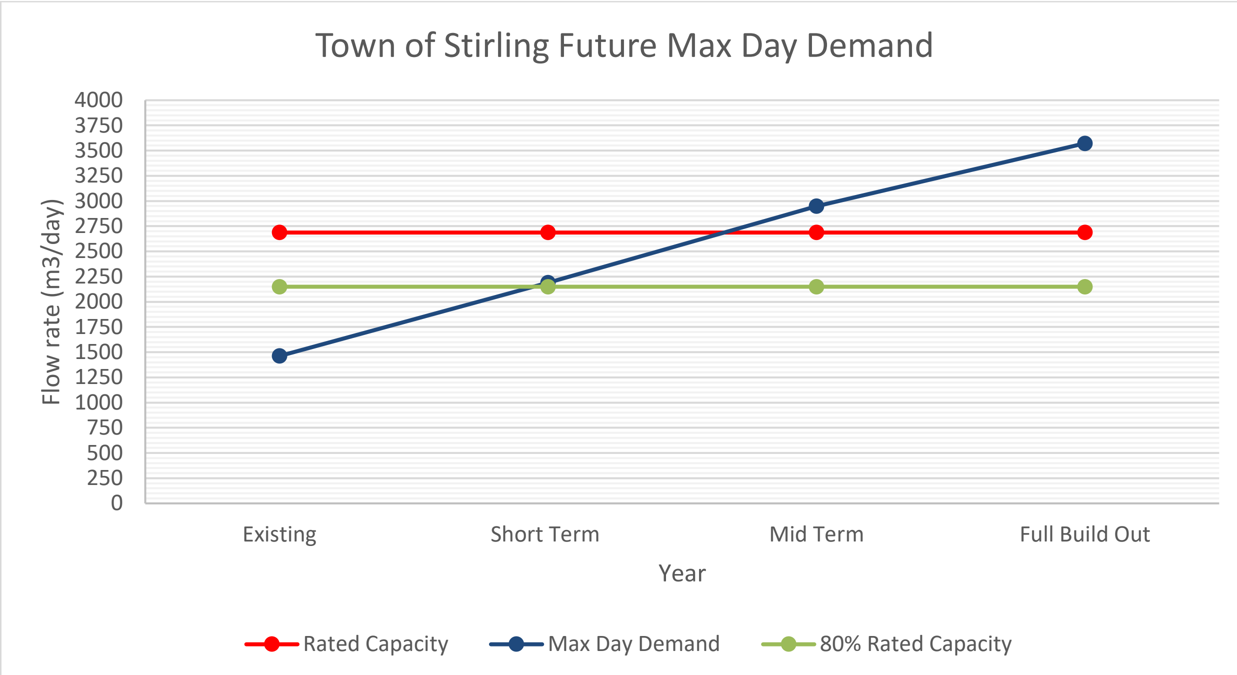


Figure 12: Future Max Day Demand

The figure above illustrates that the WTP should have sufficient capacity for the short-term development, though will be surpassing the 80% capacity mark at the completion and therefore would trigger the need to develop a plan to increase capacity prior to the plant being over capacity, which is indicated to occur part way through the mid-term development. The plant is shown to require additional capacity beyond the Mid-Term Scenario. Increasing capacity of the WTP would require a Schedule ‘C’ Municipal Class Environmental Assessment.

4.7 Water Storage Servicing Strategies

Water Storage in a drinking water system is intended to provide continuity of supply, maintain system pressure and to meet critical water demands during fire flow and emergency conditions. The MOECC DWG (MOECC, 2008) outlines the requirements to determine the quantity of storage needed. The formula for calculating the required treated water storage in the distribution system is as follows:

$$\textit{Total Treated Water Storage Requirements} = A + B + C$$

Where: A = Fire Storage
B = Equalization Storage (25% of Max Day Demand)
C = Emergency Storage (25% of A+B)

Fire storage is the product of the maximum fire flow required in the system and the corresponding fire duration.

The current standpipe system has a rated total capacity of 2.6 ML. The standpipe typically operates between 60% and 80% of the total capacity. However, the standpipe needs to be capable of supplying minimum required pressures for fire flow scenarios (20 psi). This means the volume considered as capacity inside the tank is limited based on the highest elevation area to be serviced in the Town. The elevation of liquid required to reach 20 psi (approx. 14 m) is required in the reservoir at all times, and therefore the volume up to this level is subtracted from the total capacity of 2.6 ML for the functional capacity. In this circumstance, the nearest homes are approximately at the elevation of the base of the tank; however, there are approximately 5 homes that were constructed after the storage tank, while the next highest homes are approx. 4 m lower. Since fire flow requirements assume worst case scenarios (industrial flows at 100 L/s), and the highest elevation areas are residential (with fire flow requirements of only 50 L/s), the considered functional capacity remains conservative. The total storage requirements for the existing system and each growth scenario are shown in Figure 13.

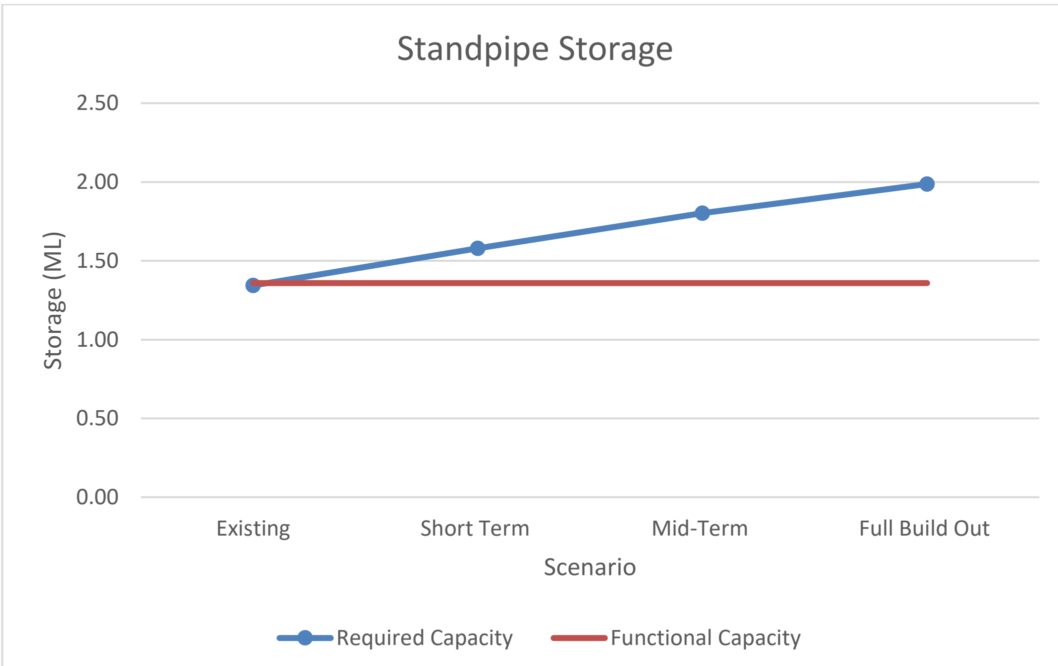


Figure 13: Existing and Future Storage Requirements

As indicated in the MOECC DWG, storage facilities should be sized for a 20 year projection as this is a reasonable lifespan and within growth predictability. Based on the projected growth and criteria indicated above, the standpipe functional capacity (capacity where fire flow pressures are available) is currently at capacity. The functional capacity (approx. 1.36 ML based on an elevation of 166.300 m (20 m of water in the tank)) would need to increase approx. 50% in functional capacity (to 1.99 ML) to meet the full buildout demand.

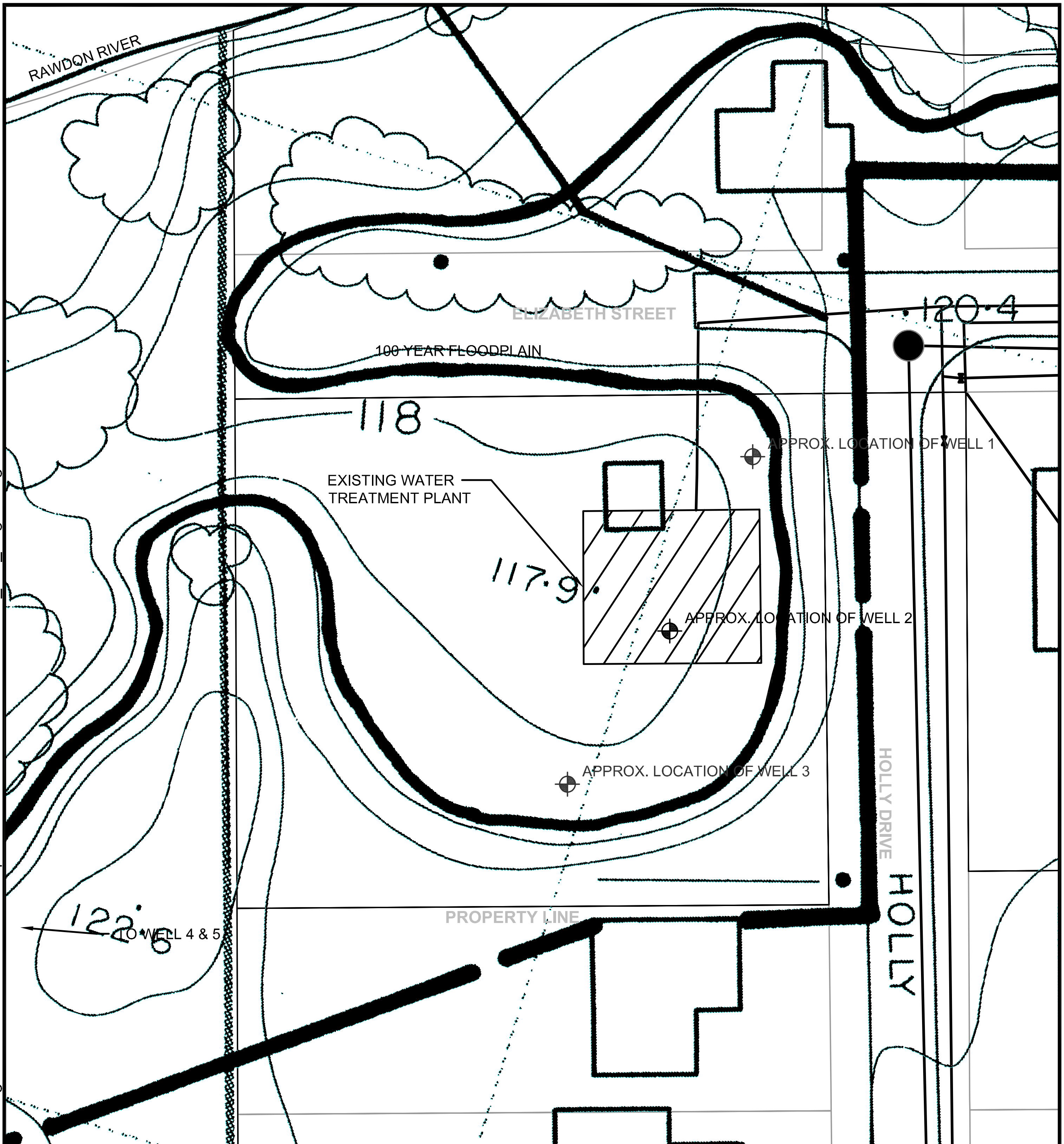
The Town has recently completed a rehabilitation project for the standpipe as it was nearing the end of its lifespan and has had noted leakage. As such, JLR would recommend that the Town look at alternative water storage locations and type to facilitate additional storage. Water storage reliability within the system should also be considered (see section below).

4.7.1 Water Storage Reliability

The Town has indicated that during any maintenance of the standpipe there is concern that there is no available storage within the system. In addition, there are limited watermain crossings of the Rawdon Creek and additional storage on the south side of the Creek would provide additional reliability in case of watermain breaks. In order to increase reliability, consideration should given to provide storage in an alternate location, including the WTP. If water storage was provided at the WTP, the process to distribute potable water would need to be revised including:

- Installation of a tank
- Installation of high lift pumps
- New well pumps with lower head (i.e. pump to clear only, not for distribution)

The new storage tank would be placed outside of the 100yr flood plain to ensure integration of the system. Figures 14 and 15 illustrate this alternative to increase reliability in the system.



PROJECT:

STIRLING STANDPIPE REHABILITATION

2529 STIRLING-MAMORA ROAD, STIRLING, ONTARIO

DRAWING:

WATER TREATMENT PLANT SITE PLAN

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CHECKED:	MM
JLR #:	27931

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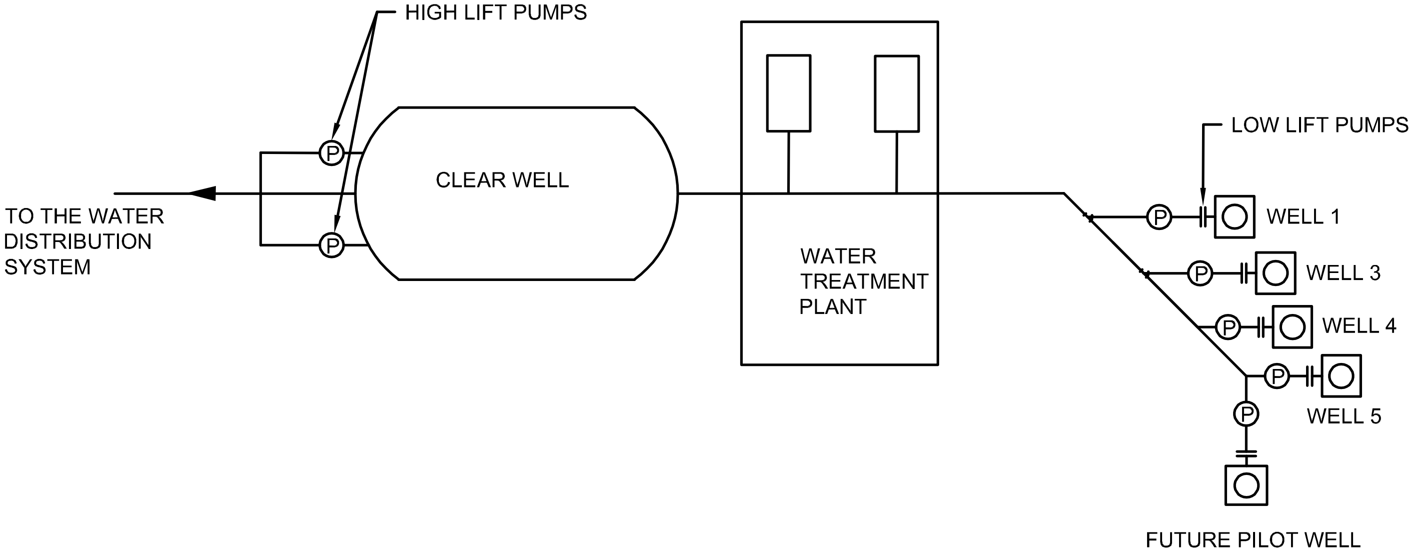
FIGURE 14

File Location: P:\27000\27931-001 - Stirling Infrastructure Assessment Updat\5-Production\7-Plan\GIS\27931_civil_dwg14.dwg

PLOT DATE: November 3, 2023 3:35:48 PM



PROJECT:

STIRLING STANDPIPE REHABILITATION

2529 STIRLING-MAMORA ROAD, STIRLING, ONTARIO

DRAWING:

WATER TREATMENT PLANT PROCESS DIAGRAM

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JLR #:	27931

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FIGURE 15

PLOT DATE: November 6, 2023 8:56:15 AM

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4.8 Summary of Potable Water Servicing Strategies

In summary, the Town should consider the following servicing strategies for the water system as they move forwards;

- Prioritize the completion of the network spine from the Water Treatment Plant to the Standpipe as the Town proceeds with road reconstructions. This process has begun with the 2022/2023 reconstruction of Henry, West Front, North and Mill Street Project.
- During the second phase of the Ryell subdivision development, continue to ensure that the subdivision loops the watermain from Rogers Dr to Elizabeth St. This looping was completed, though only to John St., while increasing the size between Henry and John St. to a 200mm would further improve flows.
- Install a 200mm watermain on Campbellford Rd. from the existing 200mm watermain on Church St. to provide adequate fire flows to the Campbellford Development.
- Install a 200mm watermain on Frankford Rd. south from Front St. to provide adequate fire flows to the southern Frankford Dr. development.
- Update hydrant test results with improved flows from various upgrades.
- Install a 200 mm watermain further up North St. as required to service the Edward St. and northern Stirling-Marmora Rd. developments, as well as northern hydrants which currently have pressure deficits under fire flows. This may be solved with the recent upgrades on Henry, Front and North St. and should be investigated with updated hydrant tests.
- Complete a Schedule 'C' MCEA to increase the Town's water supply and treatment capacity.
- Complete a Schedule 'B' MCEA to increase the Town's available water storage.

5.0 Wastewater System

5.1 Existing Wastewater System

The Town of Stirling is serviced by a communal wastewater system. The existing communal wastewater system was established in the 1950s and generally consists of 30 km gravity sewers/forcemains, four sub-area pumping stations, a main pumping station, and a relatively new extended aeration wastewater treatment lagoon. The sewage collection system is owned and operated by the Municipality.

5.1.1 Existing Wastewater Collection System

The wastewater collection system generally consists of polyvinyl chloride, ductile iron, asbestos cement, and vitrified clay piping ranging from 100 mm to 450 mm in diameter. It is understood that some of the piping is the original infrastructure dating back to the 1950s.

5.1.2 Existing Sewage Pump Stations

All sewage generated in the Town of Stirling service area is ultimately conveyed to the George Street Sewage Pumping Station (SPS), which houses three VFD pumps (each rated for 74 L/s at 36.2 m TDH, providing a firm capacity of 145 L/s) in a wet well configuration and conveys wastewater to the wastewater treatment lagoon via one 1300 m long 300 mm forcemain. The George Street Sewage Pumping Station was upgraded in 2022.

The Annis Street SPS was reconstructed in 2011 to house two submersible pumps (each rated for 48.89 L/s at 12.35 m TDH) in a 3 m diameter wet well configuration. The SPS drainage area includes the area of the Town west of Station St. (existing and future). There have been reported issues with flooding near the SPS, however this has been directly attributed to the mechanical failure of equipment components in the station or due to lightening strikes that caused damage. This station receives high flows during the spring thaw and it is assumed that multiple illegal sump pumps are interconnected into the sanitary sewer system in this collection area.

The Frankford Road SPS was reconstructed in 2011 to house two submersible pumps (each rated for 17.5 L/s at 25.57 m TDH) in a 2.4 m diameter wet well configuration. The SPS drainage area includes the area of the Town along Frankford Road (existing and future).

The Henry Street SPS houses two submersible pumps (each rated for 6.5 L/s at 4.9 m TDH) in a wet well configuration. The SPS drainage area includes the area of the Town along Henry St south of Elizabeth St and will eventually also include the phase 4 of the Ryell Subdivision development (existing and future).

The Rogers Drive SPS houses two submersible pumps (each rated for 6.5 L/s at 5 m TDH) in a wet well configuration. The SPS drainage area includes the area of the Town along Rogers Drive (existing and future). Operators have noted that this wet well has relatively small storage and will frequently run intermittently for short periods of time.

5.1.3 Existing Wastewater Treatment System

The Stirling Lagoons and Wetlands are located south of Stirling, at the end of Henry St. The wastewater treatment facility has a rated capacity of 1,500 cubic meters/day. The facility is operated in accordance with Environmental Compliance Approval (ECA) number 9487-9GFSJS issued May 27, 2014. The wastewater enters the treatment system from the George Street Sewage Pumping Station where aluminum sulfate (alum) is added for coagulation purposes. From the George Street Pumping Station the wastewater enters the North Lagoon through a horizontal discharge forcemain in the Northeast corner of the lagoon berm edge. The North Lagoon has an operating volume of 119,670 cubic meters. The water is aerated and the settling of suspended solids and facultative processes provides passive treatment. Water is then gravity fed to the South Lagoon.

The South Lagoon continues treatment though retention time for the chemical and biological breakdown of the organic matter and the settling of the suspended solids. The South Lagoon outlet controls flow to the wetland polishing system on the West bank of the cell and discharges via Agri-Drain control. Levels are controlled through the addition or removal of weir plates.

The effluent from the South Lagoon is gravity fed to the engineered wetland cells. The engineered wetland consists of a total of 15 operational trains for the full average day capacity of 1,500 cubic meters/day, plus an additional three trains for redundancy. At the pumping station, 25% hydrogen peroxide is dosed to control hydrogen sulphide levels to conform to the plants ECA. The plant effluent can either flow gravity to Mud Creek or can be discharged to Mud

Creek via two submersible pumps rated at 50 L/s at a 25 m TDH. Effluent can also be pumped back into the South Lagoon to be held during maintenance or if discharging is not permitted.

5.2 Historic Wastewater Flows

Design sheets of the existing wastewater collection system have been developed using land use designation, as discussed below, which are in included in the appendix. The capacities of the collection system shows that there are few cases where there is potential surcharging. Most of the pipe infrastructure has significant room to accommodate additional growth.

5.2.1 Historic Sanitary Collection System Flows

There are currently no sewers estimated to be surcharging in the existing collection system. The sewers nearest their capacities at approx. 60 – 80% are MH 65 – 61 on North St. from Wellington to Gore St. This can be seen on the attached Figure 17.

5.2.2 Historic Sewage Pumping Station Flows

George St. SPS underwent recent upgrades in 2022 per recommendations in the 2017 Capacity Assessment Report. With these upgrades complete, all existing sewage pumping stations have sufficient capacity to convey peak flow rates from the sanitary collection system. This can be seen in Table 8.

Table 8: Historic Sewage Pumping Station Flows

Pumping station	Number of Pumps	Rated firm capacity [L/s]	Existing Flows [L/s]
Annis Street	2	**48.9**	38.20
Frankford Road	2	**17.5**	7.07
Henry Street	2	**6.5**	2.54
Rodgers Drive	2	**6.5**	2.58
George Street	2	**145**	110.86
1. Calculated flows assume sewage pump stations outlet at their firm capacity (or at the estimated flow rate when exceeding existing firm capacity) to provide worst case scenarios			

The Municipality advised there is a history of sewer system surcharging at the Annis St SPS resulting in basement flooding during peak weather events. This was thought to be caused by mechanical issues with the pumps and heavy flows generated from illegal sump pump connections, though it was discovered additionally that approximately 40% of Annis St. SPS flows were directed back toward the station as the manhole the SPS outlets into has a sewer that also heads back towards the SPS. It is noted that there have been no bypasses to the wastewater system from 2015 to 2023.

5.2.3 Historic Wastewater Treatment Flows

Figure 16 provides a summary of historic wastewater flows recorded at the Wastewater Treatment Lagoon in 2022 for the Town of Stirling.

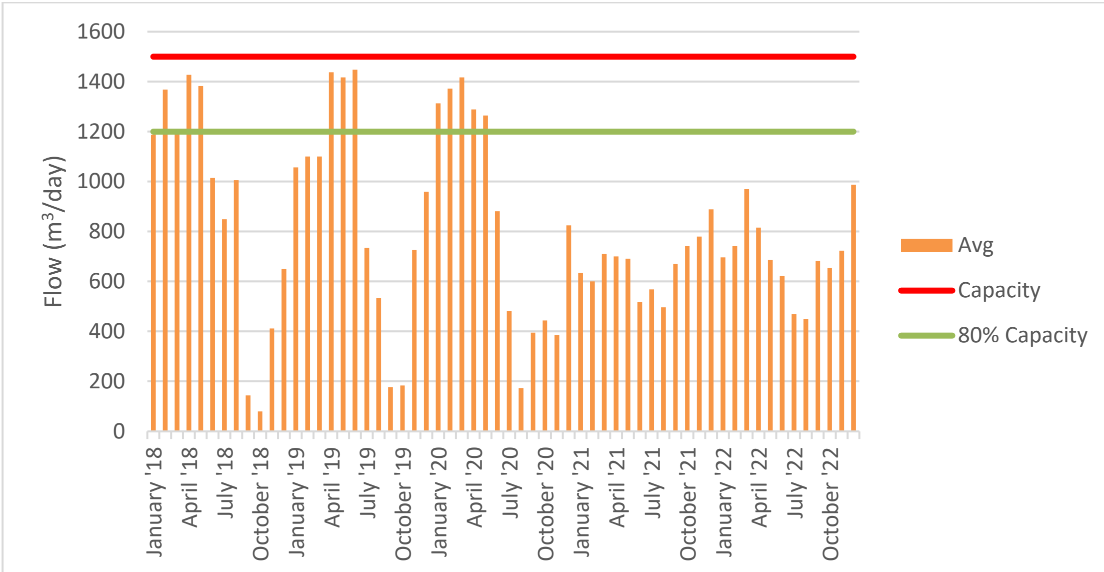


Figure 16: Wastewater Treatment Plant Operational Flows

Seen above, ADF for certain months is near capacity during 2018 – 2020 years; however, the ECA for the WWTP indicates the rated capacity is for an annual ADF. Additionally, data from these three (3) years are effluent flow, with highs and lows driven by release periods, whereas 2021 and 2022 data are influent flow which provides a better representation of the actual ADF incoming to the WWTP. The 2018 – 2020 data is however equally relevant for a 5-year average as influent will match effluent flows over a longer averaged time period. Based on the 2018 – 2022 data, the ADF was 806 m^3/day which is below the rated capacity of 1500 m^3/day.

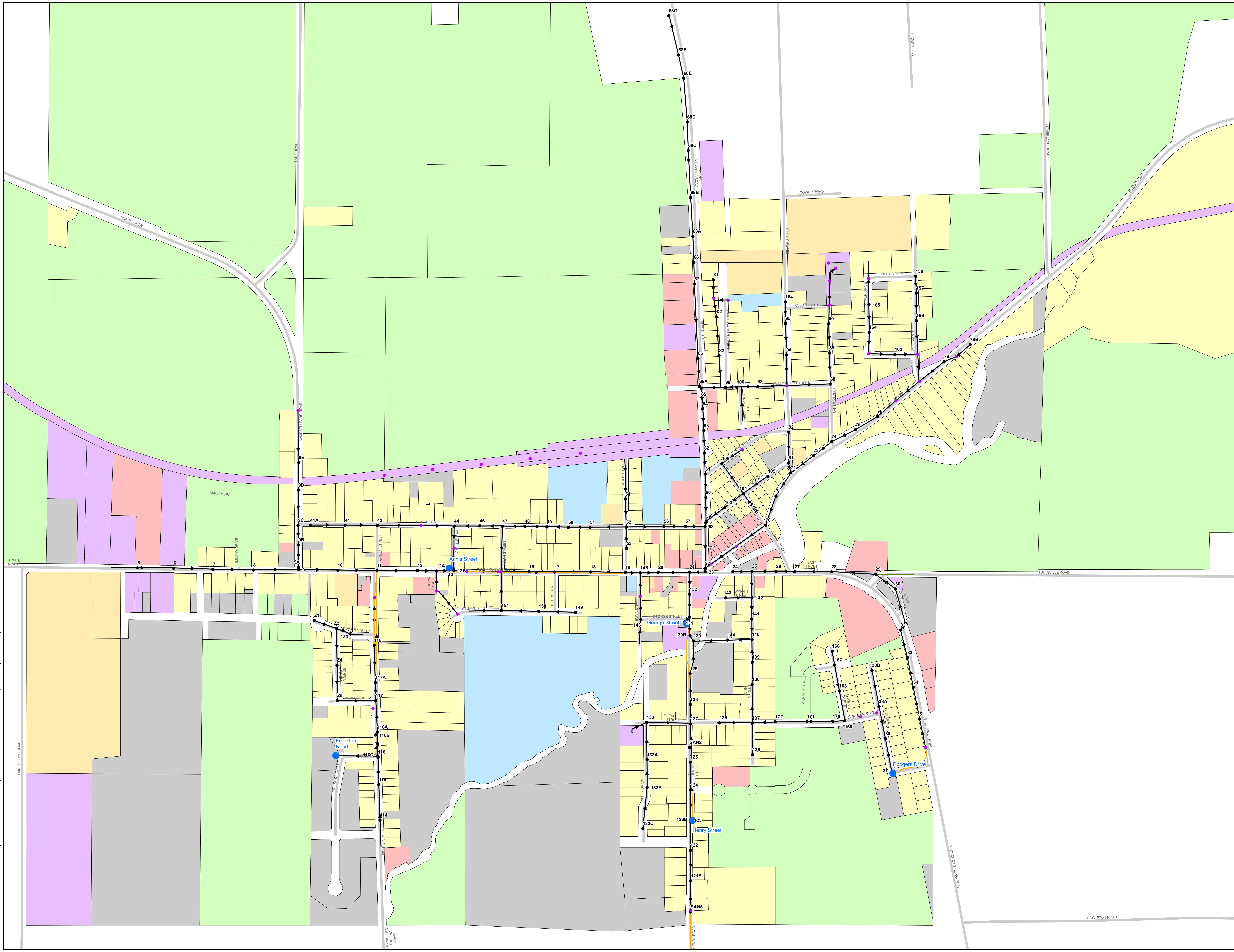
5.3 Wastewater System Design Criteria

Table 9 provides a summary of the residential wastewater generation rates to be used to assess and size the Municipality's wastewater system. These values are based on the MOECC Design Guidelines for Sewage Works.

The design used a land based approach to estimate population distribution across the Town. Rather than a separate Average Day Flow (ADF) from the typical residential value of 300 L/cap/day, the Industrial, Institutional and Commercial lands were converted to an equivalent population. The population per hectare (ppha) estimates of population densities are in Table 9.

Table 9: Land Use Equivalent Population

Land Use Designation	Equivalent Population Density (ppha)	Comments
Low Density Residential	15.6	Detached, Semi-detached
Medium Density Residential	31.2	Multi-unit, Low Rise Apartment
High Density Residential	60	Large Apartment buildings
Industrial	12.9	Warehouses, Factories etc.
Commercial	23.3	Stores, offices, etc.
Institutional	6.4	Schools, Community centres, Public services, Churches, etc.



Legend

- MH Missing IDs
- PS

Sewers

- Missing Information
- Gravity Sanitary Sewer
- Sanitary Forcemain
- Sewers

MPAC Property Code

- No Information
- Vacant
- Farm
- Residential
- Commercial
- Industrial
- Institutional
- Special & Exempt

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PROJECT:

WATER AND WASTEWATER CAPACITY ASSESSMENT

STIRLING, ONTARIO

DRAWING:

EXISTING SANITARY SEWERS

DESIGN: CT
DRAWN: KTK
CHECKED: MM
JLR #: 27931

DRAWING #:

FIGURE 17

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The sewage flows were separated into 5 different drainage areas by the pumping stations that service them. In order to get a representative analysis of the design flows the extraneous flow varied from drainage area to drainage area. Some areas with newer piping networks are noted as having smaller extraneous flows while other areas with older piping are noted as having higher extraneous flows.

Table 10: Drainage Areas Design Criteria

Drainage Area	Average Day Flow	Extraneous Flow	Peaking Factor
Frankford Rd. SPS	300 L/cap/day	0.40 L/s/ha	Varies based on Harmon Peaking Factor
Annis St. SPS	300 L/cap/day	0.28 L/s/ha	Varies based on Harmon Peaking Factor
Henry St. SPS	300 L/cap/day	0.35 L/s/ha	Varies based on Harmon Peaking Factor
Rogers Dr. SPS	300 L/cap/day	0.50 L/s/ha	Varies based on Harmon Peaking Factor
George St. SPS	300 L/cap/day	0.45 L/s/ha	Varies based on Harmon Peaking Factor

Wastewater pumping facilities are rated on their ‘firm’ pumping capacity. Firm capacity is based on the capacity of the station with the largest pump out of service. Pumping stations are sized based on peak flows. Wastewater treatment facilities are designed based on the average and peak flows, depending on the treatment process (e.g., aeration tanks are sized for average day flows, whereas settling tanks are sized for peak flows).

5.4 Future Requirements: Wastewater System

5.4.1 Wastewater Collection

Design sheets of the future wastewater collection system have been developed and are included in the appendix. The capacities of the collection system shows that there are few cases where there is potential surcharging. Most of the pipe infrastructure has significant room to accommodate additional growth. The following infrastructure capacity gaps were noted in the different scenarios.

Short Term Scenario: Due to the development on Campbellford Rd., there is surcharging on Front Street sewers between MH 9 to MH 11, from Campbellford Rd. to Green St.

Medium-Term Scenario: Due to the development within the Thompson Farmland, there is surcharging on North Street between MH 63 to MH 59, from the old rail bed (where Thompson Farmland development is proposed to connect to the existing network) to Victoria Street.

Full Build Out Scenario: Due to northern developments on Edward St. and Stirling-Marmora Rd., the North Street sewers between MH 65 to MH 63, from Wellington St. to the old rail bed are near capacity. This exacerbates the surcharging immediately south to Victoria St (MH 59). Additionally, due to the Spry/Cleaver Property development, there is surcharging on John St. sewers between MH 25 and 142, down to MH 140, from East Front St. to Robert St.

The capacity gaps in the gravity sewer system are identified in Figures 18 through 20.

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PROJECT:

WATER AND WASTEWATER CAPACITY ASSESSMENT

STIRLING, ONTARIO

DRAWING:

FUTURE SANITARY SYSTEM SHORT-TERM GROWTH

DESIGN: CT
DRAWN: KTK
CHECKED: MM
JLR #: 27931

DRAWING #:

FIGURE 18

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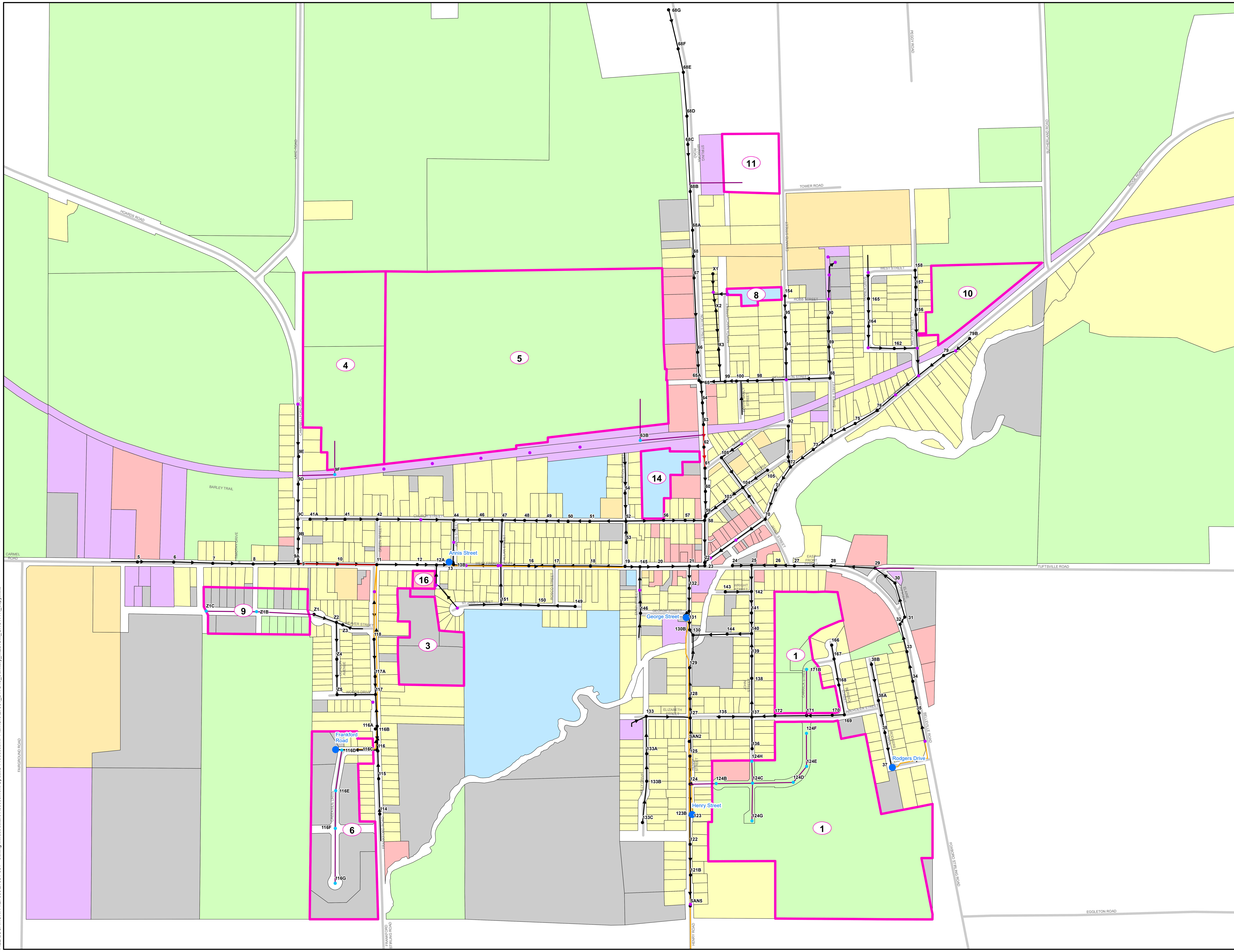

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- MH
- PS
- Future Development

Sewers

- MISSING INFORMATION
- Gravity Sanitary Sewer
- Sanitary Forcemain
- Surcharging on Sanitary Sewers
- Proposed Sanitary Sewers

MPAC Property Code

- No Information
- Vacant
- Farm
- Residential
- Commercial
- Industrial
- Institutional
- Special & Exempt

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WATER AND WASTEWATER CAPACITY ASSESSMENT

STIRLING, ONTARIO

DRAWING:

FUTURE SANITARY SYSTEM MID-TERM GROWTH

DESIGN: CT
DRAWN: KTK
CHECKED: MM
JLR #: 27931

DRAWING #:

FIGURE 19

FILE LOCATION: P:\27000\27931-001 - Stirling Infrastructure Assessment Update\5-Production\7-Plan\GIS\27931_DWG19_Sanitary_MEDIUM_GROWTH_copy.mxd

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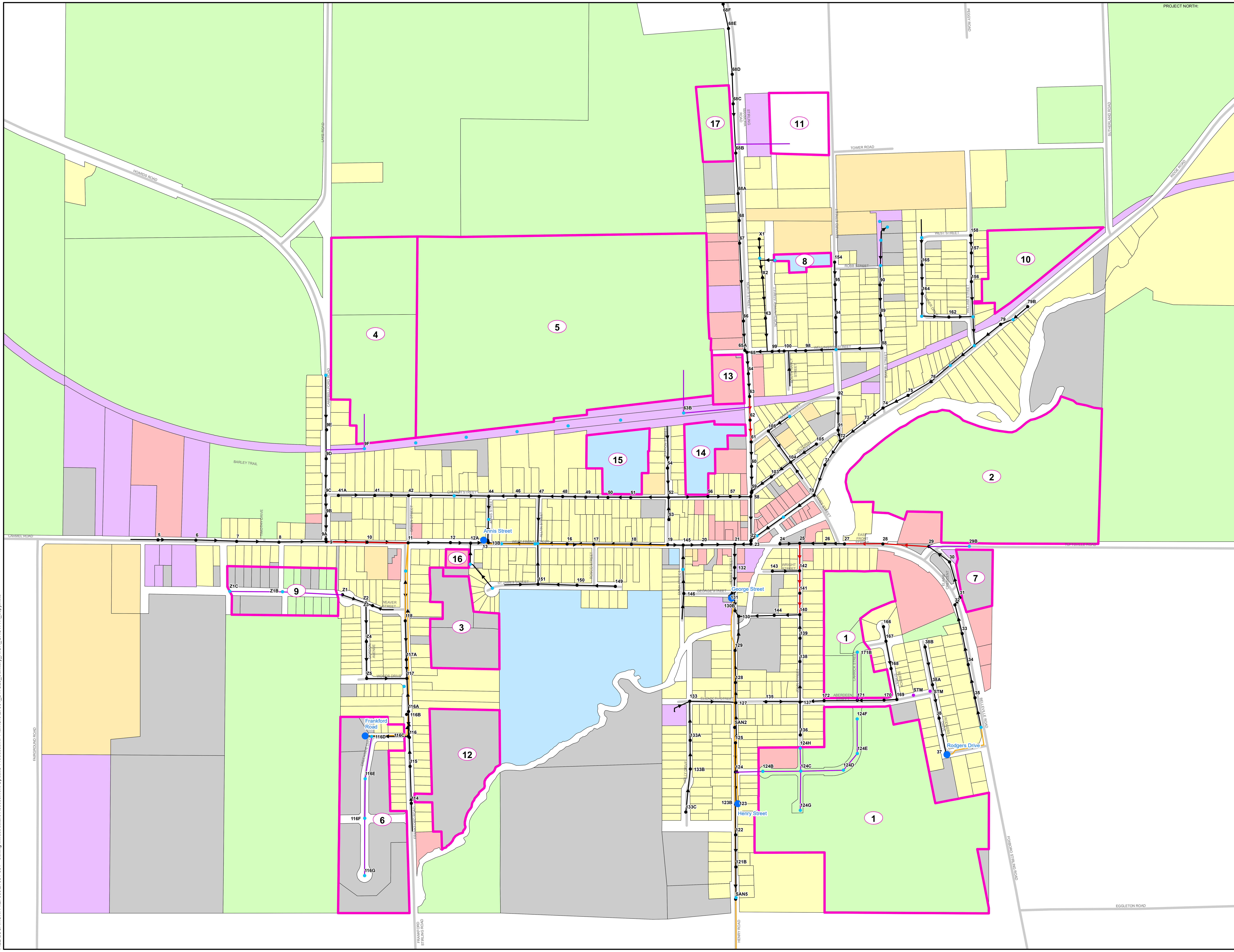

Legend

- Proposed MH
- MH Missing IDs
- MH
- PS
- Future Development

Sewers

- Missing Information
- Gravity Sanitary Sewer
- Sanitary Forcemain
- Surcharging Sewers
- Proposed Sanitary Sewers

MPAC Property Code

- No Information
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- Special & Exempt

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PROJECT:

WATER AND WASTEWATER CAPACITY ASSESSMENT

STIRLING, ONTARIO

DRAWING:

FUTURE SANITARY SYSTEM FULL BUILD OUT

DESIGN: CT
DRAWN: KTK
CHECKED: MM
JLR #: 27931

DRAWING #:

FIGURE 20

FILE LOCATION: P:\27000\27931-001 - Stirling Infrastructure Assessment Update\5-Production\7-Plan\GIS\27931_DWG20_Sanitary_HIGH_GROWTH_copy.mxd

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5.4.2 Wastewater Pumping Stations

Sewage pumping stations provide pumping of wastewater from low points in the sewage collection system while maintaining certain flow rates. Sewage pumping stations are rated based on their firm capacity. The MOECC Design Guidelines For Sewage Works (DGSW) defines "firm capacity" as the capacity of the station with the largest pump out of service. (MOECC, 2008). These rated firm capacities are shown below in the table below with the potential flows generated in each growth scenario.

Table 11: Sewage Pumping Station Operating Conditions

Pumping station	Number of Pumps	Rated firm capacity [L/s]	Existing Flows [L/s]	Short Term Projection [L/s]	Mid Term Projection [L/s]	Full Buildout Projection [L/s]
Annis Street	2	**48.9**	38.20	49.16	49.35	54.91
Frankford Road	2	**17.5**	7.07	7.17	13.18	18.09
Henry Street	2	**6.5**	2.54	4.73	4.76	4.79
Rodgers Drive	2	**6.5**	2.58	2.61	2.64	2.66
George Street	2	**145**	110.86	119.34	134.21	149.38

1. Calculated flows assume sewage pump stations outlet at their firm capacity (or at the estimated flow rate when exceeding existing firm capacity) to provide worst case scenarios

As shown above, Annis St. SPS is of primary concern, exceeding the firm capacity in the short term due to the Campbellford Rd. development. The issue is exacerbated in the Full Buildout with development of the 2nd Old School on Church St, though would remain adequate without the Campbellford Rd. development.

The Frankford Rd. SPS is shown to exceed firm capacity in the Full Buildout Scenario. This is due to additional flows assumed from Hilden and Dorann Homes developments in the Mid-Term, as well as the southeastern Frankford Rd. development in the Full Buildout. It should be noted that because the projected flows are only slightly above the rated capacity, it is recommended that these flows be monitoring over time as the actual flows maybe be slight lower than the theoretical values and Frankford St SPS would have sufficient capacity.

Additionally, the George St. SPS is shown to minorly exceed the firm capacity at the completion of the Full Buildout. Though George St. was recently upgraded, population projections far exceed those assumed in the previous 2017 Infrastructure Assessment, while all flows, including all new development flows, increase the demand of George St. SPS. It should also be noted that because the projected flows are only slightly above the rated capacity, it is recommended that these flows be monitoring as the actual flow maybe be slight lower than the theoretical values and George St SPS would have sufficient capacity.

From an operational and maintenance standpoint, Annis St. SPS had multiple reports of the station running continuously in wet weather events and flooding of local resident's basements, which was contrary to what the theoretical SPS capacity can take.

Further investigation of the Annis St. SPS showed that the outlet structure of the forcemain from the SPS in Manhole-19 was not directed towards the outlet of the manhole. The manhole with the forcemain outlet has two inverts, the eastern invert at 122.675 m directs all flow towards the George St SPS and the western invert at 122.705 m directs sewer flows west on Front St W back to the Annis St. SPS. With a difference of only 30 mm between the two directions and the forcemain outlet being undirected toward the outlet allowed a significant portion of the flow to the west invert, back toward the Annis St PS that was being continually re-pumped. Particularly in wet weather events when the flows increased this would have significantly inhibited the SPS stations ability to pump flows into the George St SPS drainage area.
With the installation of a temporary barrier on the western invert of Manhole-19, flows have decreased to the pumping station by 20-25% during dry weather conditions. This increase in efficiency will be even greater in wet weather events, when the flows are larger and more turbulent.

5.4.3 Wastewater Lagoons and Wetlands

The MOECC indicates a sewage treatment plant should be able to treat the flows of sewage generated within buildings serviced by the sewer system exclusive of any extraneous flows (i.e. the average daily flow).

Figure 21 illustrates the anticipated daily flow as a result of the various flow and development scenarios:

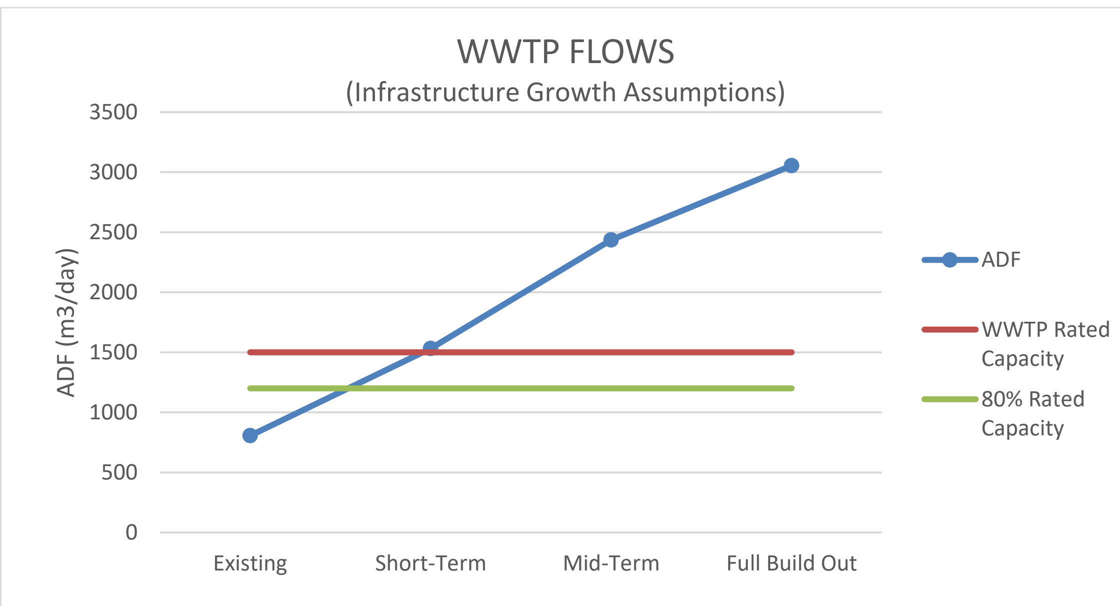


Figure 21: Wastewater Treatment Plant Existing and Future ADF

It can be seen from the figure above that the WWTP will exceed 80% capacity during the Short Term and exceed the rated capacity by the end. Exceedance will be heavily increased within the Mid Term and Buildout growth scenarios. Generally, capacity upgrades are triggered when a treatment facility reaches approximately 80% of the current functional or production capacity. This early identification allows time to accommodate the required planning and design between the anticipated need and the implementation of the upgrades. Based on projections, the WWTP will reach 80% capacity with the development of approx. 327 units, and reach capacity with approx. 576 units.

5.5 Wastewater Collection and Pumping Servicing Strategies

As previously noted, the Annis St. SPS will require additional capacity over the Short-Term growth-planning period. Additionally relevant to the Campbellford Rd. development, the Front St. sewers from Campbellford Rd. to Green St. will become surcharged. Since the alternatives to both the conveyance system and pumping station capacity issues are inter-connected, servicing strategies were combined.

5.5.1 Annis Street SPS and Front Street Sewer Design Alternatives

Various options to address the noted future capacity issues anticipated at the Annis St SPS and North St sewer were reviewed. The alternatives included:

(1) Redirect the Campbellford Rd. Development sanitary sewer flows to a new sewer installed along the old railbed which outlets to North St. and bypasses Annis St. SPS.
(2) Upgrading the Annis St. SPS and the Front St. Sewers

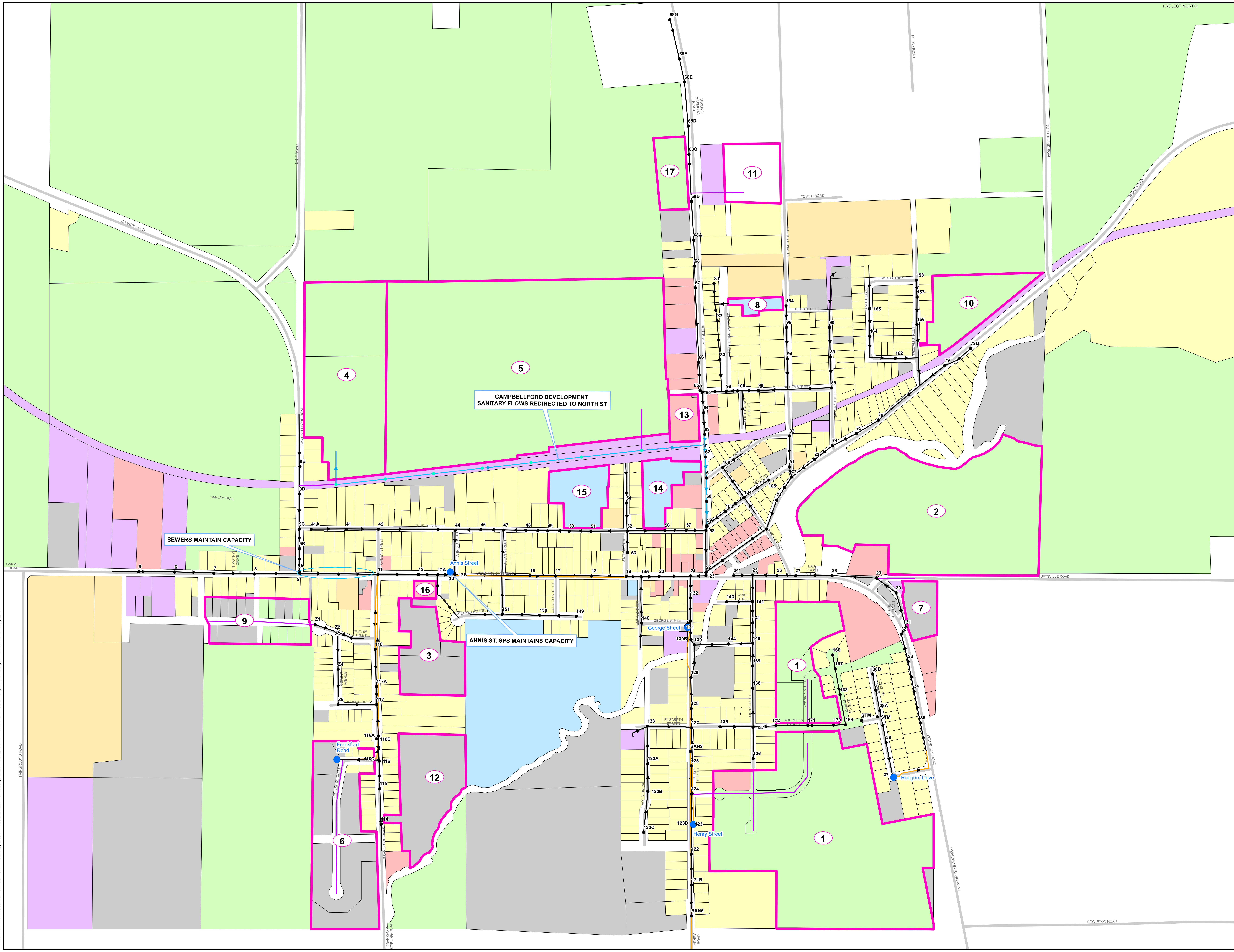
5.5.1.1 Alternative 1: Redirecting Campbellford Rd. Development Sewer Flows to North St.

This alternative would involve directing the Campbellford Rd. development sewer flows to a new sewer installed along the old rail bed (immediately south of the development), directing flows east approx. 900 m to North St. Refer to Figure 22 for details.

Since flows will head immediately to North St. from the development, Campbellford Rd. will be bypassed, therefore bypassing both Front St. sewers and the Annis St. SPS. This solution then reduces Front St. sewers on concern to existing flows, and Annis St. SPS to approx. 46.45 L/s for the Full Buildout scenario (below the firm capacity of 48.9 L/s).

5.5.1.2 Alternative 2: Upgrading Annis St. SPS and Front St. Sewers

This alternative involves upgrading the Annis St. SPS and the Front St. sewers to accommodate for the increased flows from the Campbellford development and additional Full Buildout flows. Refer to Figure 23 for details.



Legend

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- Proposed MH
- MH
- PS
- Over Capacity PS
- 2 Future Development

Sewers

- Missing Information
- Gravity Sanitary Sewer
- Sanitary Forcemain
- Proposed Sanitary Sewer

MPAC Property Code

- No Information
- Vacant
- Farm
- Residential
- Commercial
- Industrial
- Institutional
- Special & Exempt

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DRAWING:

PROPOSED SANITARY SYSTEM ALTERNATIVE 1 CAMPBELLFORD RD DEVELOPMENT REDIRECTION TO NORTH ST

DESIGN: CT
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JLR #: 27931

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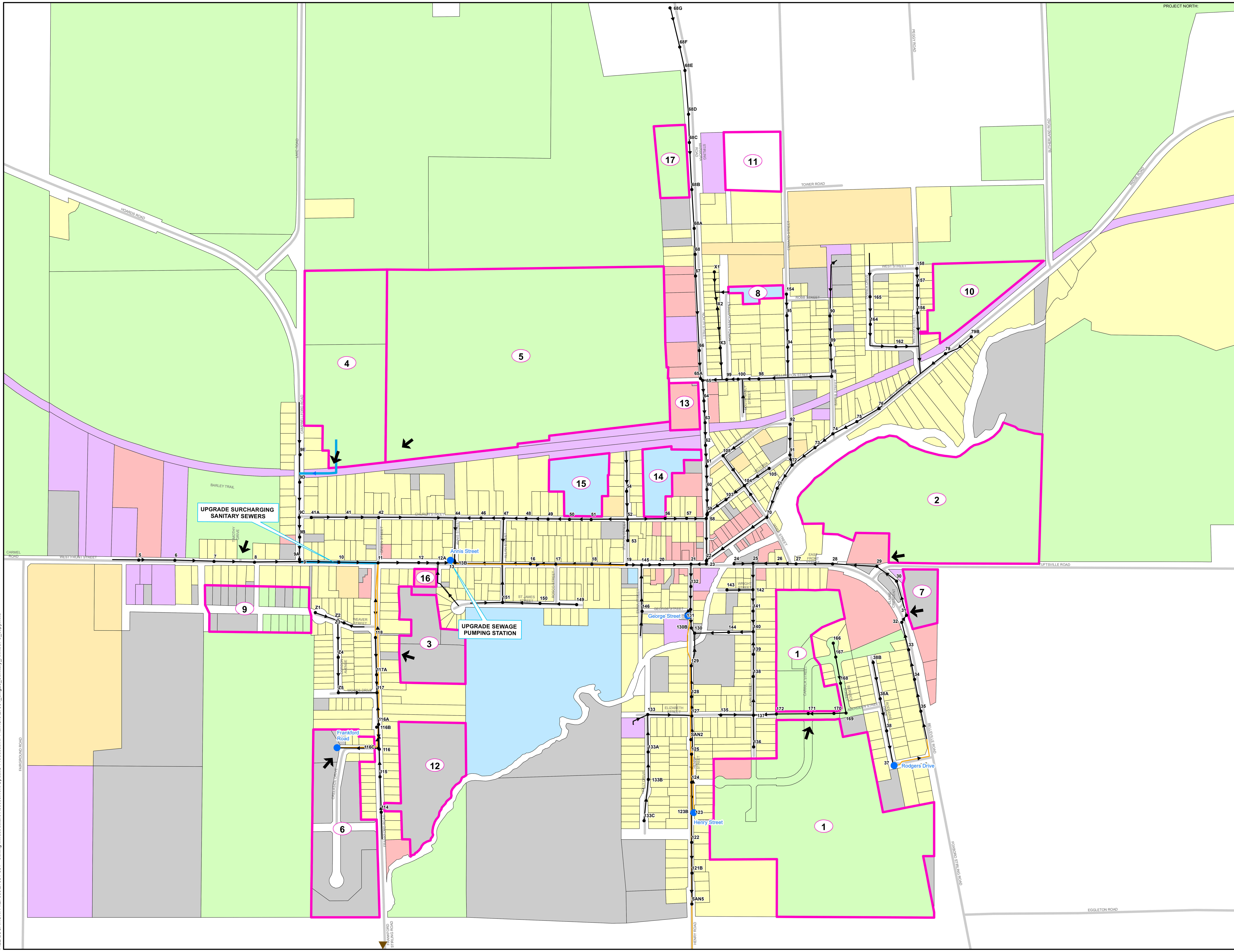

Legend

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- MH
- PS
- Over Capacity PS
- Future Development

Sewer

- Proposed Sanitary Sewer
- Surcharging on Sanitary Sewer
- Proposed Forcemain
- Missing Information
- Gravity Sanitary Sewer
- Sanitary Forcemain

MPAC Property Code

- No Information
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DRAWING:

**PROPOSED SANITARY SYSTEM
ALTERNATIVE 2
ANNIS ST SPS UPGRADES**

DESIGN: CT
DRAWN: KTK
CHECKED: MM
JLR #: 27931

DRAWING #:

FIGURE 23

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5.5.2 Evaluation of Alternatives and Recommendations

The evaluation of key aspects of the two alternatives for the wastewater collection system are shown below.

Table 12: Wastewater Evaluation of Alternatives

Criteria		Description	
		Alt 1: Campbellford Rd. Dev. Redirection to North St.	Alt 2: Annis St. SPS and Front St. Sewer Upgrade
Financial Considerations	**Capital Costs**	$1,700,000	$2,000,000
	Operational Costs	$0	$200,000
	Total	**$1,700,000**	**$2,200,000**
Natural Environment Considerations: Natural features, natural heritage areas, Areas of Natural and Significant Interest, designated natural areas, watercourses and aquatic habitat		• Minimal impact to existing natural features, watercourses and aquatic habitat	• Minimal impact to existing natural features, watercourses and aquatic habitat
Social and Cultural Environment Considerations: Proximity of facilities to residential, commercial and institutions, archeological and cultural features, designated heritage features, well or wellhead protection areas, land-use and planning designations		• Minimal impact on residents with increased noise, minimal impact to traffic with trucks entering/existing site area, minimal interruption to residents	• Interruption to the busy roadway along West Front St, Interruption of local traffic, Interruption to residents • Increased noise at night due to extended bypass requirement
Technical Feasibility: Constructability, maintaining, or enhancing drinking water quality, maintaining or enhancing wastewater treatment, reliability and security of systems, ease of connection to existing infrastructure and operating and maintenance requirements		• Installation of approx. 900 m of sanitary sewers at standard depths • Straightforward installation requirements and staging with minimal anticipated obstacles	• Installation of new pumps and potential wet well upsizing • Constant SPS bypass operation • Minimal available workspace at station • Upsizing of approx. 200 m of existing sanitary sewers
Overall Preference Rating		Has the least overall impact to the Town with the lower financial burden	More impact to the Town and large financial burden.

As indicated above, altering the Campbellford development sanitary flows to outlet easterly to North St. with approx. 900 m of additional sanitary sewers along the abandoned railbed is favourable in all accounts compared to the typical design approach of utilizing the nearest existing maintenance hole (Campbellford Rd.). Redirecting flows allows the increased demand to bypass Annis St. SPS, therefore enabling Annis St. SPS to avoid requiring upgrades. This is the primary benefit to Alternative 1 which creates large capital cost savings. Additionally, Annis St. SPS is located in a busy area on West Front St and therefore Alternative 1 drastically reduces the social and cultural impacts of residents and heavily simplifies technical requirements and constructability.

While the Annis St SPS is noted for it`s history of surcharging and basement flooding, this can be heavily attributed to the forcemain outlet directing flows back towards itself. Additionally, during one rainfall event, the pumping station experienced a mechanical malfunction, which lead to the surcharging and flooding. In another instance, a lighting strike caused issues within the station.

The Town has also expressed a desire for additional redundancy to prevent any flooding in the future. Ideally, this could be accomplished with an overflow being installed from the pumping station. However, investigations showed that with current grades around the pumping station, an overflow would have to be piped approx. 500m before overflowing to the Rawdon Creek. Other mitigation measures would include Inflow and Infiltration reduction programs, but these programs don't always yield measurable concrete reductions in flows.

Since Annis St SPS theoretically has more than enough capacity over the entire study period, it is recommended that the Town complete the control upgrades currently schedule and to monitor the flows at the station now that the outlet has been directed downstream.

Alternative 1 to redirect the Campbellford Rd. Development flows to North St. is the preferred option. Installation of these sewers would need to be completed in conjunction with the Campbellford Rd. Development. Increased flows will not immediately surcharge sewers on North St., though from the connection location (MH 63) to Gore St. (MH 61) will be near capacity, and these sewers, along with those extending to Victoria St. (MH 59) will be surcharged with the addition of the Thompson Farmland Development in the Mid-Term and will require upgrades.

5.5.3 Reliability and O&M Design Alternatives

Rogers Dr. is constantly running for short periods of time and the Town has expressed a desire to try and eliminate the SPS by redirecting flows through the new Ryell Subdivision. See Figure 24 for more detail.

5.5.3.1 Alternative 1: Rogers Drive SPS Redirection

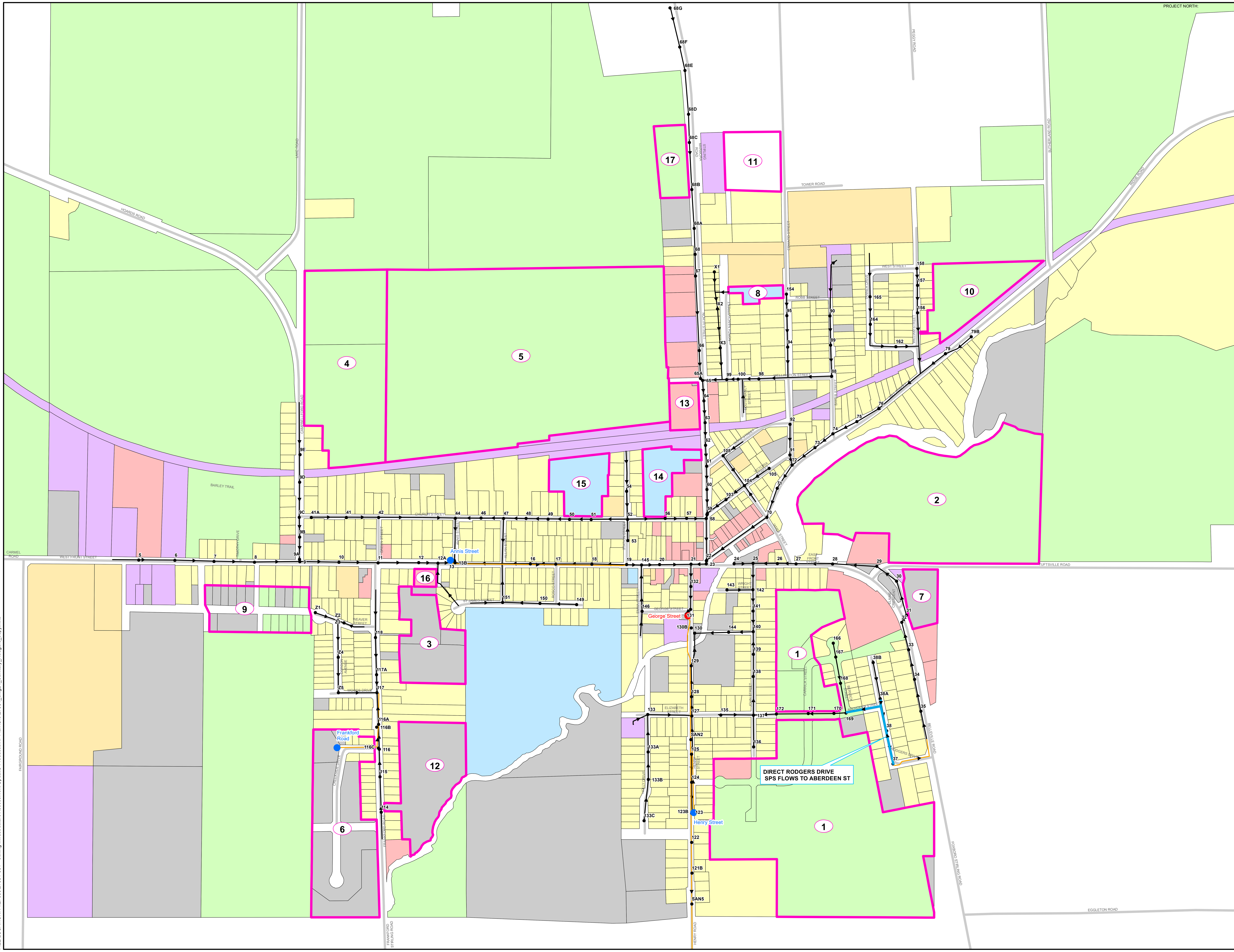
This option would involve redirecting the sewer flows from the Rogers Dr SPS to Aberdeen St and then draining by gravity through the new subdivision to the George St SPS. A small portion of Aberdeen St will have to be excavated to reach the nearest manhole to connect into.

5.5.3.2 Alternative 2: Do Nothing

Do not update sewer line and continue to run the Rogers Dr SPS. This option has no capital costs however there will be operational maintenance costs over time.

5.5.4 Evaluation of Alternatives and Recommendations

The existing Rogers Drive roadway is in a poor state of repair, as such since it is feasible to redirect the flows to Aberdeen Street, It is recommended that the Town redirect the sewer flows and remove the Rogers Dr. SPS when the Town chooses to proceed with the road reconstruction. This will mitigate the capital costs associated with the redirection and save the Town annually in operational and maintenance costs.



Legend

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- MH
- PS
- Over Capacity PS
- Future Development

Sewers

- Missing Information
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- Sanitary Forcemain
- Surcharging on Sanitary Sewer

MPAC Property Code

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PROJECT:

DRAWING:

PROPOSED SANITARY SYSTEM
O&M UPGRADES
ROGERS DR SPS REDIRECTION

DESIGN: CT

DRAWN: KTK

CHECKED: MM

JLR #: 27931

DRAWING #:

FIGURE 24

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5.6 Wastewater Treatment Servicing Strategies

It can be seen from Figure 21 the WWTP will be near its rated capacity by the end of the Short-Term Scenario, exceeding 80% capacity part way through. Starting the upgrade process will need to commence before the short term growth is achieved and will allow time to accommodate the required planning and design between the anticipated need and the implementation of the upgrades. A detailed analysis of the treatment options through a Municipal Class Environmental Assessment would need to be completed but it is assumed that the current treatment strategy would be maintained.

5.7 Summary of Wastewater Servicing Strategies

In summary, the recommend upgrades are as follows:

Short Term:

- Install new sanitary sewers along the abandoned railbed from the Campbellford Road development to North St., avoiding the requirement to upgrade Annis St. SPS and Front St. sewers between MH 9 and MH 11.
- When the Wastewater Treatment Lagoon reaches it's 80% capacity the Town should look into the viability of various treatment options and increasing capacity through the Municipal Class Environmental Assessment process.

Mid Term:

- Upgrade North St. sewers from MH 63 to MH 59 (between Gore and Wellington St. to Victoria St.). This can be done in conjunction with watermain upgrades on North St.
- Redirect the Rogers Drive SPS to the nearest Aberdeen St. manhole, and take the pumping station out of service

Long Term:

- Upgrade North St. sewers from MH 65 to MH63 (from Gore St. heading south). These upgrades could be completed during the medium term in conjunction with upgrades required for the Thompson Farmland to save on long-term capital costs.
- Upgrade John St. sewers from MH 25 – MH 140 (From Front St. to Robert St.).

6.0 Stormwater System

6.1 Existing Stormwater System

The existing storm sewer system in Stirling is a combination of storm sewers and ditches. Figure 4 illustrates all the known storm infrastructure in Stirling. As noted in the drawing, there are significant gaps of information in the storm sewer system. Current understanding of the Stormwater systems capacity is only in certain areas.

6.2 Historic Stormwater Flows

A large portion of downtown Stirling is located in the 100-year floodplain of Rawdon Creek. There is a history of flooding in the downtown core, particularly in the block north of Mill St. A report by Jewell Engineering (2017) looked into the condition of the storm sewer that passed under the downtown storefronts on Mill St. The report showed CCTV inspections that identified a collapsed portion of the storm sewer and recommended that the line be taken out of service and the building that sits above the collapsed pipe be assessed for its structural integrity. The recent downtown reconstruction project has undertaken the required structural repairs under the building prior to sealing each end for abandonment and rerouting the existing sewer system for alternate outlets.

6.3 Stormwater System Design Criteria

Stormwater conditions and design criteria were used to create a representative look at the stormwater flows generated for a typical 5-year storm for the town of Stirling. The rainfall conditions were assessed from data taken from the Belleville weather station (The nearest reporting weather station). The short duration rainfall intensity was then taken from the IDF curve.

Rainfall intensity is assessed based on the following formula:

$$i = A * T^{B}$$

Where;
i = Rainfall intensity
A = Storm Return Period Variable
T = Time of Concentration
B = -0.677

For the town of Stirling, the following design criteria were used;

Table 13: Stormwater Design Criteria

Parameter	Value	Comment
Runoff Coefficient	0.4	
Storm Return Period Variable	28.2 – 38.5	Dependent on return period
Time of Concentration	20 minutes	

6.4 Future Requirements: Stormwater System

As identified in the population growth projections, there is significant growth for the Town of Stirling. There are a variety of concerns for the storm sewer system that should be addressed. As identified in the existing storm sewer system. There are various streets in the Town that lack any storm sewers for residents to connect. Many streets in the town have ditches that have deteriorated over time and will need to be re-trenched or replaced with storm sewers. Additionally, the standpipe has an outlet for draining in emergencies and the storm sewer along Baker St needs to be assessed for its ability to drain the standpipe.

6.5 Stormwater Distribution Servicing Strategies

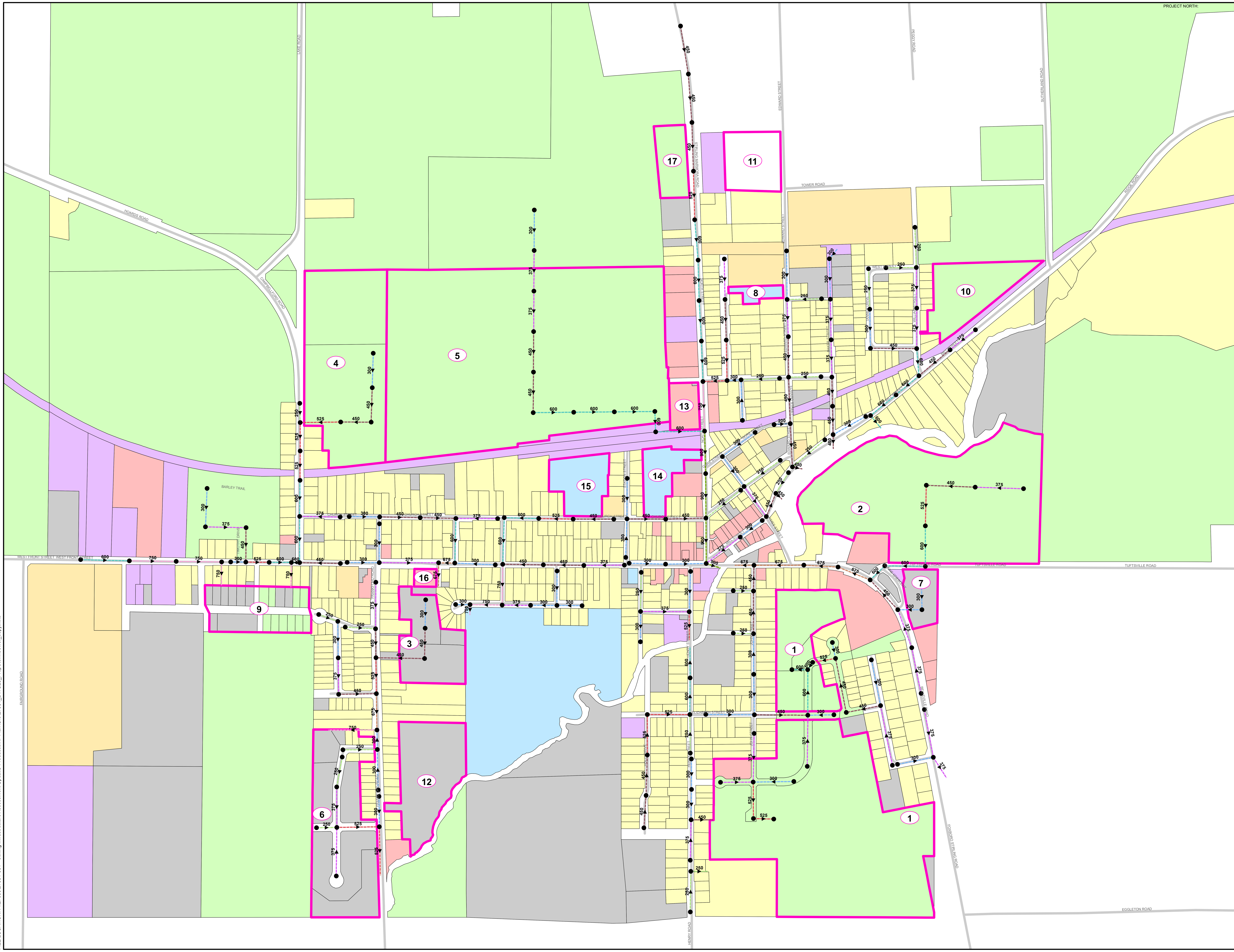
For future road reconstructions, it is recommended that the Town of Stirling repair and replace existing infrastructure holistically and look at all the infrastructure within the road easement needing to be replaced or upgraded over the entire lifespan of the roadway. In future road reconstructions, the Town should consider installing storm sewers along streets that are currently lacking them, and look into the adequacy and condition of existing storm sewers when they already exist.

The proposed storm sewers for each roadway in Stirling were assessed based on the design criteria for a 5-year storm. Each roadway was then sized for what the storm sewer would need to be able to accommodate all the stormwater runoff in the Towns existing catchment areas. The future sizing of all the proposed storm sewers is shown in Figure 25. This can be regarded as a preliminary assessment of the storm sewer system, prior to any development a more detailed analysis should take place to confirm design criteria used in this assessment. The proposed storm sewer does not generally modify the Towns existing outlet infrastructure and maintains the current drainage schematic. Additionally, there are hypothetical storm sewers added to the design for each of the future developments. These storm sewers are not accurate predictions of the storm sewers required for each of the proposed developments but they act as placeholders and help to predict the effect the new developments could have on the downstream storm sewer network.

6.6 Stormwater Retention Servicing Strategies

There is a noted need to stop the drainage from Thompson Farmlands into the downtown area, the current system drains all the fields with tile drainage into a vegetated drainage ditch south of the railway easement on North St. Discussions with the Town investigated the possibility of redirecting this runoff either east or west along the railway easement. Due to the railway easements high elevations on Campbellford Rd and Tanner Dr, it would be difficult to redirect the surface overland flow from the farmlands away from the downtown core. As such, the storm sewers have been adequately sized along North St to include the runoff from the Thompson Farmlands.

There are several future developments proposed for the Town. Typically any new developments are required to include stormwater retention systems so that they don’t add significant new flows to the system and cause surcharging downstream in the system (i.e. pre vs. post). Additionally, there are new regulations coming into place that require the application of LID design principles and the use of perforated pipes. Any new developments will need to consider how they will comply with new regulations.



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PROPOSED STORMWATER SYSTEM

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DRAWN:

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DRAWING #:

FIGURE 25

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6.7 Summary of Stormwater Servicing Strategies

In summary, the following strategies should be implemented for the storm sewer system as the Towns roadways are gradually upgraded.

- Clean and unplug existing storm sewers specifically at areas that are noted for having poor drainage.
- Reinstate adequate ditching along roads where they have eroded, sedimentation has occurred and there is no storm-sewer.
- Ensure that storm-sewer installation is considered whenever there is a new road or road reconstruction and design holistically to account for future system requirements.
- Look into mapping the existing storm sewer system to identity the current systems capacity and areas of concern.
- New developments will need to comply with new storm-water retention regulations when they come into force.
- New developments will need to incorporate stormwater retention systems to mitigate new stresses on the existing system.

7.0 Recommended Servicing Strategies

7.1 Implementation

It is recommended that the Town coordinate all of the different servicing recommendations together to optimize efficiency and cost effectiveness. The Town should consider the full lifespan of infrastructure and when it will need replacement, then it should proceed with making upgrades to the water, wastewater and storm in tandem. The Town should also consider which infrastructure upgrades are a priority and stagger the implementation of these recommendations so that new infrastructure will come online when its needed.

7.2 Capital Improvement Plan

A capital Improvement Plan has been developed to summarize the infrastructure upgrades that need throughout the Town. This section provides an implementation timing of recommended servicing strategies as well as the Opinion of Probably Cost (OPC) in 2023 dollars and the anticipated Municipal Class Environmental Assessment requirements. It should be noted that the timing on some of the recommended upgrades may have been altered from the original recommended timing in order to coordinate projects or group upgrades together in a logical manner. These aspects are included to assist the Town in planning for the required upgrades.

Municipal Class Environmental Assessments (MCEA) are categorized into Schedules A, A+, B and C with reference to the magnitude of their anticipated environmental impact. The divisions between schedules also have varying levels of complexity. Schedule A activities are Pre-approved. The proponent may proceed without following the procedures set out in any other parts of the MCEA requirements. Schedule A+ activities are Pre-approved, however, the public has to be advised prior to the projects implementation. Schedule B activities involve completion of Phases 1 and 2 of the MCEA planning process and are then approved subject to screening. If the screening process through Phases 1 and 2 results in other requirements of the Class EA being applicable, then those requirements must also be fulfilled. Finally, a Schedule C MCEA pertains to activities subject to the full planning process of the Municipal Class Environmental Assessment.

<u>Overall Potable Water System Improvements:</u>
The potable water system currently has some difficulty distributing potable water efficiently; it is recommended that the Town finish installation of the network spine from the WTP to the downtown area to improve water distribution, as well as upgrades identified on Aberdeen St., Campbellford Rd., and Frankford Rd. The Town will need to undertake a Class C MCEA to address the requirement for increasing their water supply capacity. The Town will also need to undertake a Class B MCEA to address the requirement for increased potable water storage capacity.

<u>Overall Wastewater System Improvements:</u>
With the recent upgrades part of the George St. SPS Upgrades and the Downtown Reconstruction Project, the current wastewater system is working well, with no immediate risks of reaching capacity for any sewers or pumping stations. However, partway through the Short-term Scenario, the WWTP demand will exceed 80% capacity and trigger the need to develop a plan for increasing capacity. The Town will then need to begin the process of undertaking a Class C MCEA an ensure the determined solution is implemented prior to exceeding capacity (shortly within the Mid-Term Scenario). Furthermore, sewer installation is required in the short term for the Campbellford Rd. development redirection to North St. Additionally, the Town will require a few future upgrades to sanitary sewers on North St. and John St., as well as Rodgers Dr. and Annis St. to accommodate abandonment of Rodgers Dr. SPS.

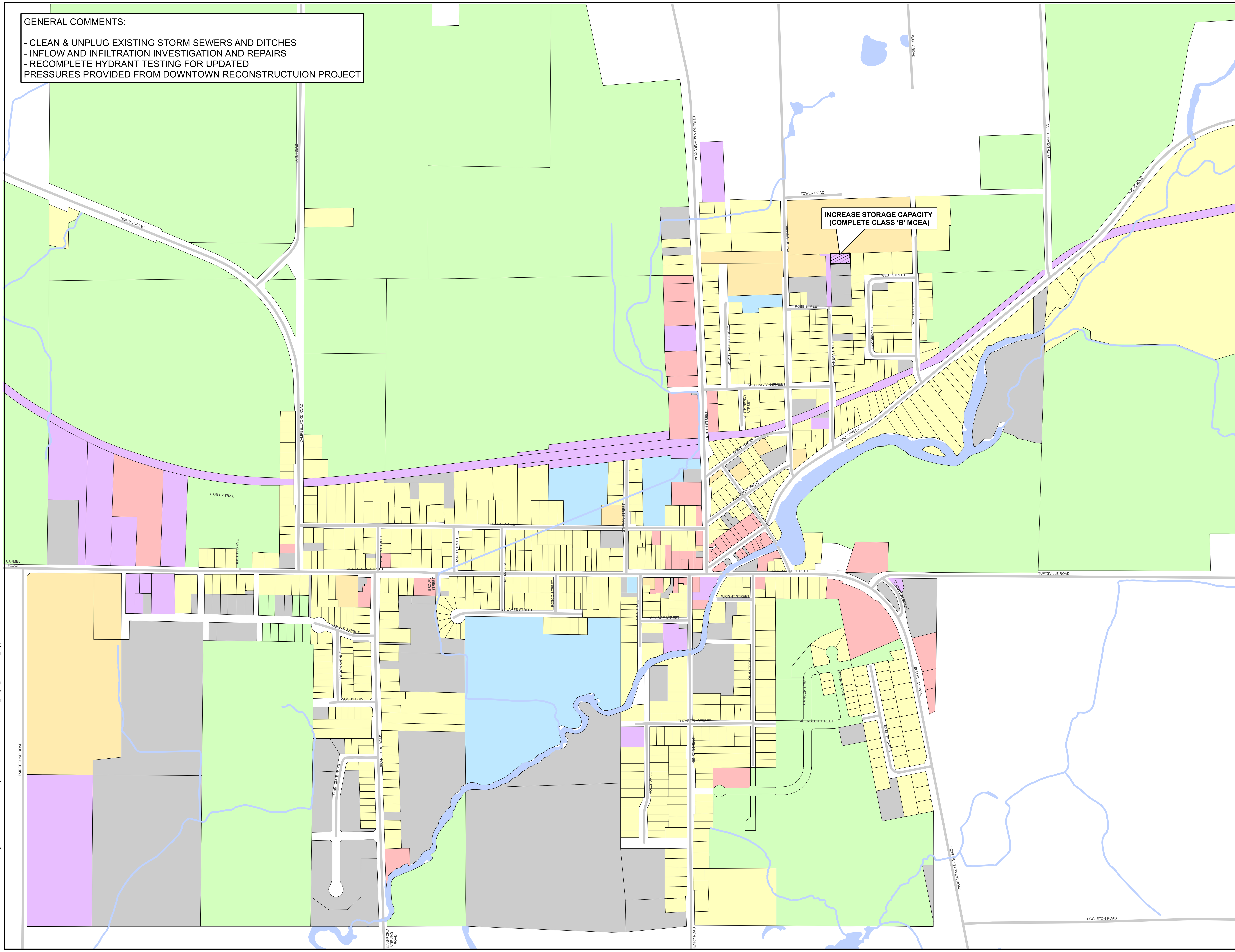
<u>Overall Stormwater System Improvements:</u>
Due to a lack of knowledge of the existing storm sewer system, existing system recommendations are limited. The theoretical sizing of the storm sewer system for any future road reconstructions has been developed. Downstream effects of these upgrades should be determined during design to determine the effects on existing infrastructure. It should be noted that overall costs for the storm sewer upgrades have not been included but we have included typical unit pricing of any future storm sewer work. We would recommend that the Town continue to map and examine the existing storm sewer network, clean and unplug existing storm sewers where possible and reinstate any eroded ditches in areas of the Town that lack any storm sewers.

Table 14: Implementation and Timing for Recommended Servicing Strategies

Timing	Area	Classification	EA Schedule	Cost 2023$ (2)	Description (1)	Notes
Existing	Water	Storage	B	150k	Class 'B' MCEA for Water Storage Upgrades	• Complete a Class 'B' Municipal Class Environmental Assessment to determine the most appropriate means to increase the Town's available water storage.
	Water	Storage	A/A+	2.0M	Water Storage Upgrades	• Complete water storage upgrades per Class 'B' MCEA
	Water	Distribution	A/A+	50K	Hydrant Pressure Testing	• Complete updated hydrant testing to inform on updated available hydrant pressures from the Downtown Reconstruction Project.
	Wastewater	Collection	A/A+	50k	Inflow & Infiltration Investigation & Repairs	• Incremental inspection of existing manholes and CCTV of sewers. Gradual repair of all recommended areas.
	Stormwater	Collection	A/A+	50k	Clean and Unplug Existing Storm Sewers & Ditches	• Cleaning and Ditching program to fix poorly drained areas of the Town.
Short-Term Growth	Water	Distribution	A/A+	800k	Aberdeen St. Watermain Looping	• Replacement and upsizing of watermain on Elizabeth St. from Henry St. to John St. with next phase of subdivision development
	Water	Distribution	A/A+	1.1M	Elizabeth & Henry St. Watermain Upgrades	• Completion of the network spine from the WTP to Front St. tie in.
	Water	Distribution	A/A+	600K	Campbellford Rd. Watermain Upgrades	• Replacement and upsizing of watermain on Campbellford Rd. from Church St. to the Campbellford Rd. development connection
	Wastewater	Treatment	C	350k	Class 'C' MCEA for WWTP Capacity Upgrades	• Complete a Class 'C' Municipal Class Environmental Assessment in preparation to upgrade WWTP Capacity.
	Wastewater	Treatment	C	15 - 20M(4)	WWTP Capacity Upgrades	• Complete WWTP capacity upgrades per Class 'C' MCEA
	Wastewater	Treatment	A/A+	1.7M	Campbellford Rd. Sanitary Sewers to North St.	• Install new sanitary sewers along the abandoned railbed from the Campbellford Rd. development to North St.
	Wastewater	Collection	A/A+	50k	Inflow & Infiltration Investigation & Repairs	• Incremental inspection of existing manholes and CCTV of sewers. Gradual repair of all recommended areas.
	Stormwater	Collection	A/A+	50k	Clean and Unplug Existing Storm Sewers & Ditches	• Cleaning and Ditching program to fix poorly drained areas of the Town.
Medium-Term Growth	Water	Treatment	C	350K	Class 'C' MCEA for WTP Capacity Upgrades	• Complete a Class 'C' Municipal Class Environmental Assessment in preparation to upgrade WTP Capacity.
	Water	Treatment	C	5.25M(3)	Class 'C' MCEA for WTP Capacity Upgrades	• Complete WTP capacity upgrades per Class 'C' MCEA. To provide cost estimations, it has been assumed there is additional groundwater sources available approx. 1 km east of the Belleville Rd. and Tuftsville Rd. intersection.
	Water	Distribution	A/A+	700K	North St. Watermain Upgrades	• Replacement and upsizing to 200mm watermain on North St. from Victoria St. to the Thompson Farmland connection location.
	Wastewater	Collection	A/A+	600K	North St. Sanitary Sewer Upgrades	• Replacement and upsizing of sanitary sewers on North St. from Victoria St. to the Thompson Farmland connection location.
	Wastewater	Pumping	A/A+	1.0M	Rogers Dr SPS redirection and decommissioning	• Redirect sewers to new section of Aberdeen St and decommission PS
	Wastewater	Collection	A/A+	50k	Inflow & Infiltration Investigation & Repairs	• Incremental inspection of existing manholes and CCTV of stormsewers. Gradual repair of all recommended areas.
	Stormwater	Collection	A/A+	50k	Clean and Unplug Existing Storm Sewers & Ditches	• Cleaning and Ditching program to fix poorly drained areas of the Town.

Timing	Area	Classification	EA Schedule	Cost 2023$(2)	Description(1)	Area
Full Build Out	Water	Distribution	A/A+	800k	Frankford Rd. Watermain Upgrades	• Replacement and upsizing to 200mm watermain on Frankford Rd. from Woods Dr. south to eastern Frankford Rd. development connection
	Wastewater	Collection	A/A+	500k	Upgrade North St. sewers	• Replacement and upsizing of sewer on North St. from Wellington St. to the Thompson Farmland tie in location. This work could be completed in conjunction with the Medium-Term North St. upgrades to reduce the long-term capital costs.
	Wastewater	Collection	A/A+	700k	Upgrade John St. sewers	• Replacement and upsizing of sewer on John St. from Front St. to Robert St.

(1) This table provides a summary of all of the proposed works, for a detailed description of each of the proposed servicing strategies refer to sections 4.8, 5.8 and 6.8
(2) All costs indicated include full reinstatement of asphalt, granulars, curbs & sidewalks (if currently present) unless otherwise noted
(3) Assumes that: land acquisition is not included; additional treatment is not required; suitable subsurface conditions (unknown); excludes dewatering, bedrock excavation, electrical servicing; historic construction data accounting for inflation, but does not include impacts on labour, material, equipment, manufacturing, supply, and transportation in relation to COVID-19.
(4) Assumes primary treatment can be accommodated through existing system.



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- Vacant
- Farm
- Residential
- Commercial
- Industrial
- Institutional
- Special & Exempt

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EXISTING RECOMMENDED SERVICING STRATEGIES

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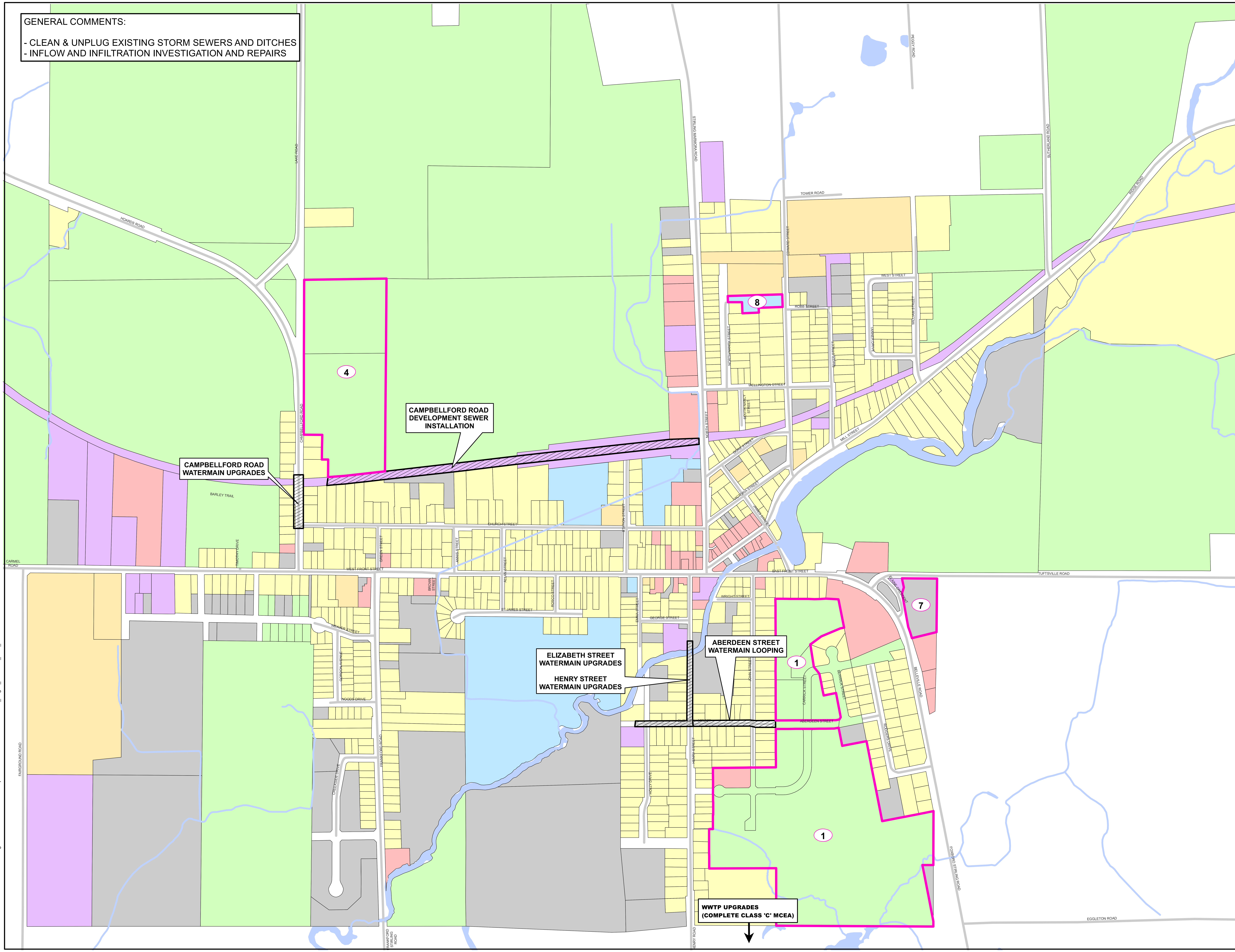

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- 2 Future Development

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SHORT-TERM GROWTH RECOMMENDED SERVICING STRATEGIES

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FIGURE 27

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MID-TERM GROWTH RECOMMENDED SERVICING STRATEGIES

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FIGURE 28

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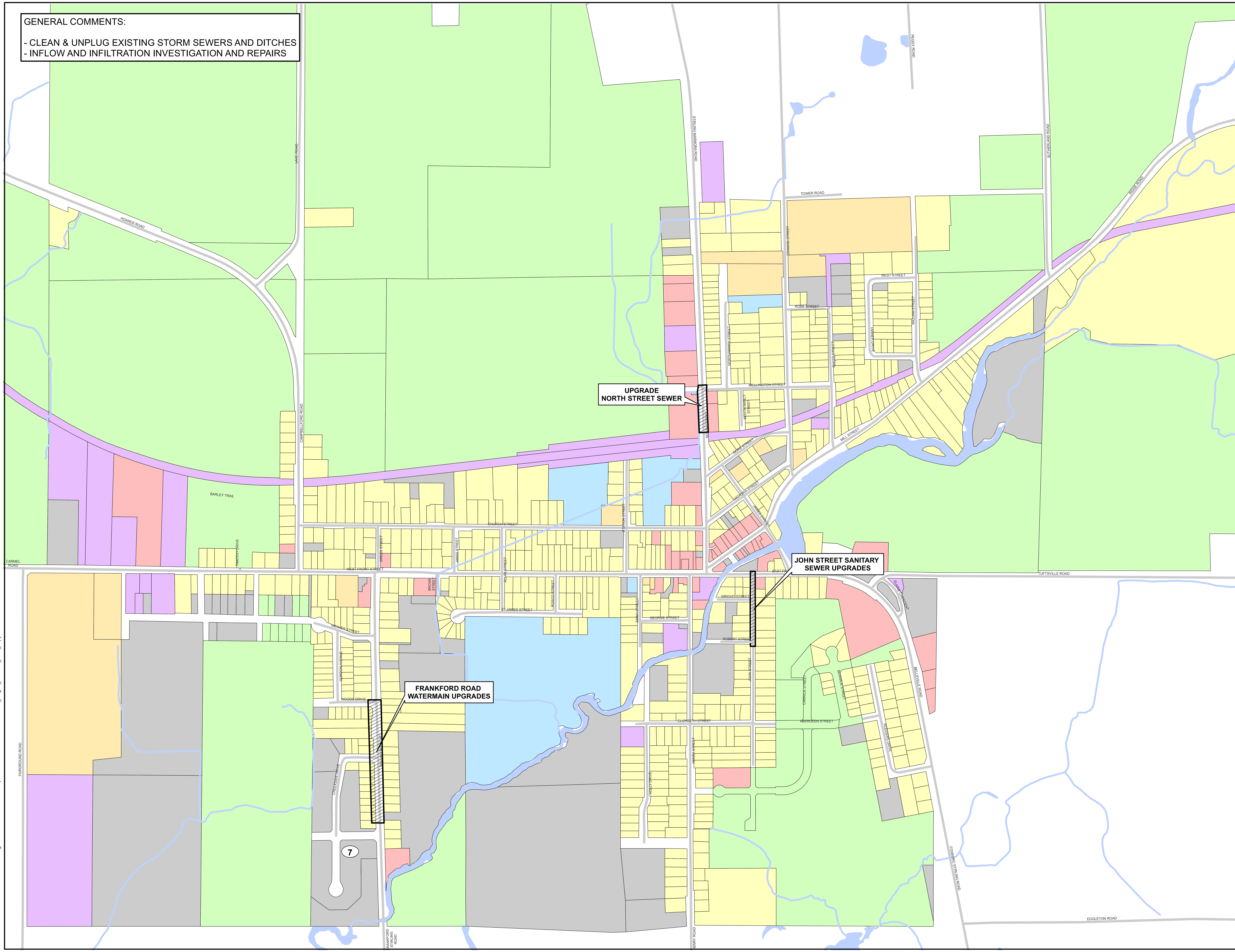

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MPAC Property Code

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- Vacant
- Farm
- Residential
- Commercial
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- Institutional
- Special & Exempt

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FULL BUILD-OUT RECOMMENDED SERVICING STRATEGIES

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7.3 Unit Price Costing

In order to assist the Town in estimating costs of infrastructure upgrades, below is a list of cost for typical linear infrastructure in 2023 dollars:

Watermains

Table 15 - Watermain Costs

Watermain	
Size (mm)	**$/m**
150	600
200	700
300	775
400	850
500	925

- Watermain costs included all appurtenance (valves, bends etc.)
- Costs do not include roadway reinstatement
- Costs do not include rock removal

Sanitary & Storm Sewers

Table 16 - Sanitary & Storm Sewer Costs

Sanitary Sewer	
Size(mm)	**$/m**
200	1120
250	1240
300	1380
375	1600
450	1860
525	2140
600	2460
675	2800
750	3170

- Sanitary Costs include all appurtenances (manholes, temporary pumping etc.)
- Costs do not include roadway reinstatement
- Costs do not include rock removal

Roadway Reinstatement

Table 17 - Roadway Reinstatement Costs

Roadway	
Width (m)	**$/m**
7	1200
8	1400

- Cost includes sidewalk and curb on both sides
- Cost do not include lighting or traffic signals
- Cost includes standard roadway cross section
- Cost are not for high traffic roadways requiring more than a standard roadway cross section (e.g. 40mm of HL3, 50mm of HL8, 150mm of Gran A & 300mm of Gran B)

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Prepared by:

Owen Perrett, EIT
Civil Engineering Intern

Prepared by:

Matthew Morkem, P.Eng., Senior Associate
Environmental Market Chief

8.0 References

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APPENDIX A: POPULATION GROWTH ESTIMATES

Proposed Development		Area [Ha]	New Units	Projected Population	ADF	Peaking Factor	Peak Flow Rate (SAN)		WW Infiltration		Water MDD	
Lot (Figure 5)	Location				[l/s]		[l/s]	[m3/day]	[l/s]	[m3/day]	[m^3/day]	[L/s]
1	Ryell Subdivision*	11.10	100	240	0.83	4.12	3.43	296.50	1.55	134.27	133.20	1.54
4	Property on Campbellford Rd.	9.24	215	516	1.79	3.97	7.11	614.11	1.29	111.75	286.38	3.31
9	Weaver Street Extension (1)	2.91	5	12	0.04	4.41	0.18	15.86	0.41	35.20	6.66	0.08
10	Mill and William Street	2.96	80	192	0.67	4.15	2.77	239.30	0.41	35.80	106.56	1.23
14	Old School (1)	1.38	87	209	0.73	4.14	3.00	259.40	0.19	16.69	115.88	1.34
16	Old Black Dog Restaurant	0.24	29	70	0.24	4.28	1.04	89.44	0.03	2.90	38.63	0.45
-	Unassigned Growth	-	30	71	0.25	4.28	1.06	91.72			39.64	0.46
3	Hilden Homes	2.92	51	122	0.43	4.22	1.79	154.90	0.41	35.36	67.93	0.79
5	Thompson farm land	29.66	386	926	3.22	3.82	12.29	1061.98	4.15	358.79	514.15	5.95
6	Dorann Holmes	6.77	45	108	0.38	4.23	1.59	137.19	0.95	81.83	59.94	0.69
8	Stirling Manor Expansion**	N/A**	N/A**	56	0.19	4.30	0.84	72.32	0.00	0.00	31.08	0.36
9	Weaver Street Extension (2)	1.38	14	34	0.12	4.35	0.51	43.81	0.19	16.69	18.65	0.22
11	Edward Street	2.03	19	46	0.16	4.32	0.68	59.13	0.28	24.55	25.31	0.29
-	Unassigned Growth	-	31	75	0.26	4.28	1.11	95.62			41.37	0.48
2	Spry/Cleaver property	15.1678	197	473	1.64	3.99	6.54	565.46	2.12	183.47	262.40	3.04
7	Old Brown Shoe Property	1.090	14	34	0.12	4.35	0.51	43.81	0.15	13.18	18.65	0.22
12	Frankford Road	4.010	52	125	0.43	4.22	1.83	157.85	0.56	48.50	69.26	0.80
13	Stirling Manor Expansion(2)	0.84	N/A**	96	0.33	4.25	1.42	122.35	0.12	10.16	53.28	0.62
15	Old School (2)	1.850	117	280	0.97	4.09	3.98	343.55	0.26	22.38	155.35	1.80
17	Stirling-Marmora Road	1.880	24	58	0.20	4.30	0.86	74.34	0.26	22.74	31.97	0.37
-	Unassigned Growth	-	26	61	0.21	4.30	0.92	79.11			34.07	0.39
	TOTAL	**95**	**1495**	**3741**	**13**		**53**	**4539**	**13**	**1154**	**2110**	**24**
	Low Growth	**27.83**	**545.76**	**1309.82**	**4.55**		**18.59**	**1606.34**	**3.90**	**336.62**	**726.95**	**8.41**
	Medium Growth	**70.59**	**1091.82**	**2676.36**	**9.29**		**37.40**	**3231.30**	**9.88**	**853.85**	**1485.38**	**17.19**
	Full Buildout	**95.43**	**1521.02**	**3802.46**	**13.20**		**53.45**	**4617.77**	**13.36**	**1154.28**	**2110.36**	**24.43**

Avg Household size	2.4	cap/unit
Low Residential Density	6.5	unit/Ha
Med Density	13	unit/Ha
High Density	25	unit/Ha
Daily Flow/Cap (q)	300	L/cap/D
Infiltration Rate	0.14	L/s/Ha

APPENDIX B: WATER PRESSURE HYDRANT FLOW ANALYSIS

APPENDIX C: STANDPIPE STORAGE CAPACITY ASSESSMENT

Storage Requirements - Existing			
A) Fire Storage	Fire Flow	Duration	Required
	100 L/sec	2 hrs	0.72 ML
B) Equalization Storage	Max Day	Multiplier	Required
	1.42 ML/day	0.25	0.36 ML
C) Emergency Storage	A+B	Multiplier	Required
	1.08 ML	0.25	0.27 ML
		Total	**1.34 ML**

Storage Requirements - 10 year			
A) Fire Storage	Fire Flow	Duration	Required
	100 L/sec	2 hrs	0.72 ML
B) Equalization Storage	Max Day	Multiplier	Required
	2.17 ML/day	0.25	0.54 ML
C) Emergency Storage	A+B	Multiplier	Required
	1.26 ML	0.25	0.32 ML
		Total	**1.58 ML**

Storage Requirements - 20 year			
A) Fire Storage	Fire Flow	Duration	Required
	100 L/sec	2 hrs	0.72 ML
B) Equalization Storage	Max Day	Multiplier	Required
	2.89 ML/day	0.25	0.72 ML
C) Emergency Storage	A+B	Multiplier	Required
	1.44 ML	0.25	0.36 ML
		Total	**1.80 L**

Storage Requirements - Build Out			
A) Fire Storage	Fire Flow	Duration	Required
	100 L/sec	2 hrs	0.72 ML
B) Equalization Storage	Max Day	Multiplier	Required
	3.48 ML/day	0.25	0.87 ML
C) Emergency Storage	A+B	Multiplier	Required
	1.59 ML	0.25	0.40 ML
		Total	**1.99 ML**

APPENDIX D: SANITARY SEWER DESIGNSHEETS

APPENDIX E: STORMSEWER DESIGNSHEETS

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