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April 1, 2024

Prepared for:

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and

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Phase 1 Report

Madoc Water, Wastewater, and Stormwater Master Plan

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1.0 Introduction

1.1 Background

The Municipality of Centre Hastings (the Municipality) and Ontario Clean Water Agency (OCWA) initiated a Class Environmental Assessment (Class EA) Master Plan exercise to identify existing conditions, residual capacity in the current system, and future upgrades to the water, wastewater and stormwater infrastructure to accommodate future growth in Madoc. This Master Plan is being completed in accordance with the Municipal Engineers Association (MEA) Class EA Approach 1 master planning process. The ultimate objective of the Master Plan is to develop a strategy to accommodate future growth within Madoc for the next 20-years and beyond that can be implemented in a prioritized fashion to improve the overall performance and reliability of the water, wastewater and stormwater systems.

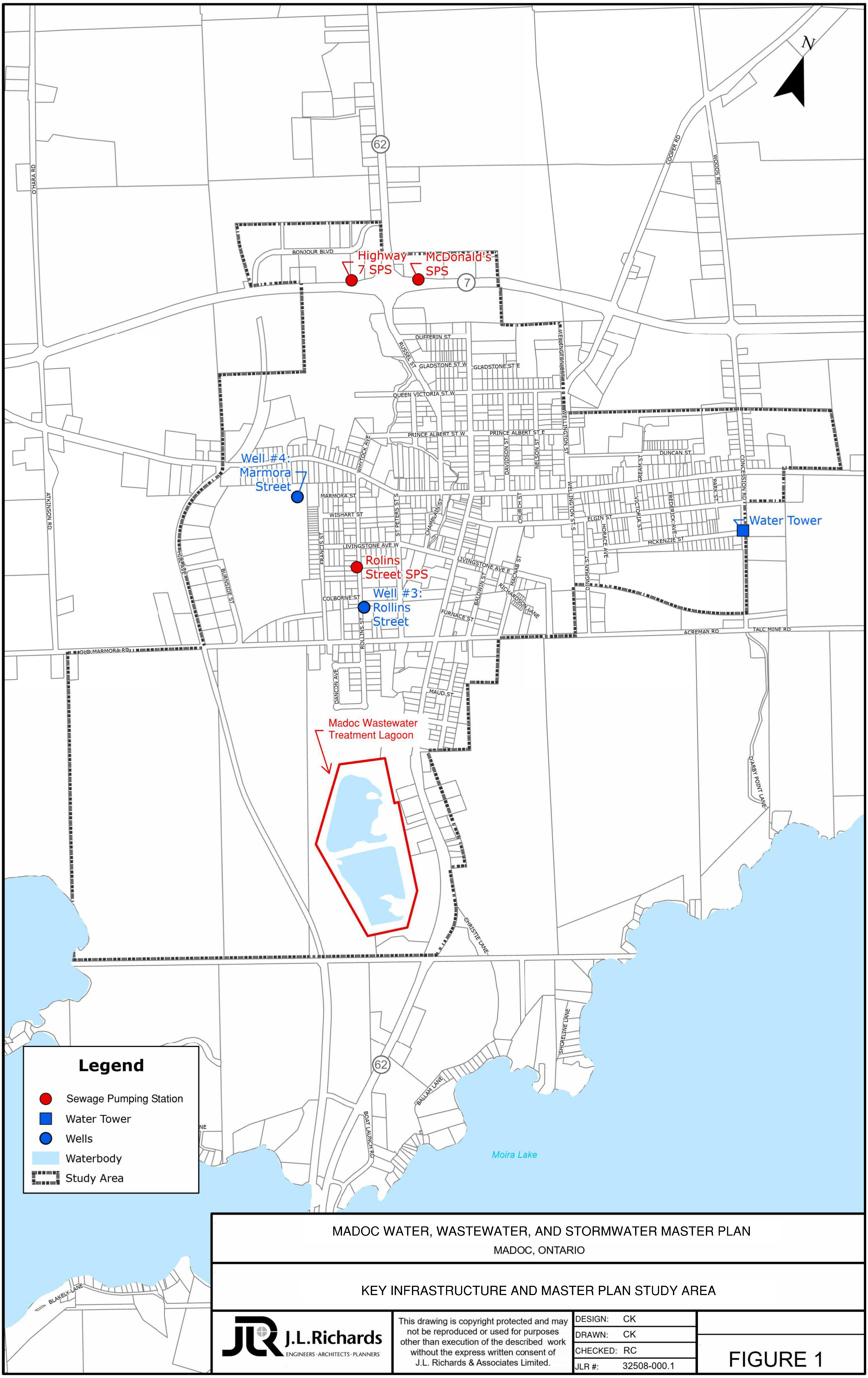
The Village of Madoc is located within the Municipality of Centre Hastings, at the intersection of Trans-Canada Highway 7 and Provincial Highway 62 and is bordered by the rural Township of Madoc. The water and wastewater infrastructure in Madoc is owned by the Municipality and operated by OCWA. The stormwater infrastructure is owned and operated by the Municipality. The Study Area includes the urban boundary of the Village of Madoc and potential future developments located within the Township of Madoc and the Municipality of Centre Hastings, as shown in Figure 1.

Madoc's water supply and distribution system consists of two groundwater wells and pumphouses, one elevated storage tank, and over 16 km of watermains. Well #3, located on Rollins Street, has a maximum daily rated capacity of 1,150 m^3/day and includes filtration and disinfection. Well #4 located on Marmora Street, has a maximum daily rated capacity of 1,470 m^3/day and includes an ion-exchange arsenic removal system in addition to filtration and disinfection. Both wells are defined as groundwater under the direct influence of surface water (GUDI). There is an elevated water storage tank with a total volume of 1,250 m^3 that maintains the hydraulic grade line and required water storage within the distribution system. The Madoc Drinking Water System is operated under the Ministry of Environment, Conservation and Parks (MECP) Municipal Drinking Water License (MDWL) Number 153-101 and Drinking Water Works Permit (DWWP) Number 153-201.

The wastewater collection and treatment system consist of over 16 km of sanitary mains, three sewage pumping stations, one aluminum sulfate storage tank, and one wastewater treatment lagoon. The wastewater treatment lagoon is a two-celled facultative lagoon, operating in series, with an average daily rated capacity of 1,008 m^3/day and a total volume of 184,000 m^3. The lagoon is used to treat municipal sanitary sewage collected from Madoc's sewer system and hauled sewage. The final effluent is discharged seasonally from the lagoon to Deer Creek, which leads to Moira Lake. The lagoon is operated under the Environmental Compliance Approval Number 1652-BRKT58.

Main road corridors in Madoc, including St. Lawrence Street West, St. Lawrence Street East, Durham Street, Elgin Street, Russel Street, and Wellington Street are serviced by minor storm sewers. Roadside ditches are routed to catch basins in low-lying areas in the road system to protect residential properties.

J.L. Richards & Associates Limited (JLR) was retained by OCWA on behalf of the Municipality to assist in the preparation of the Master Plan. The purpose of this report is to summarize the findings from Phase 1 of the Master Plan process. This includes a comprehensive description of the existing water distribution and treatment, wastewater collection and treatment, and stormwater systems, an understanding of the residual capacity of each system under current conditions, and existing and future servicing constraints. This information has been used to develop the Problem and Opportunity Statement that will form the basis of undertaking Phase 2 of the Master Plan process.



MADOC WATER, WASTEWATER, AND STORMWATER MASTER PLAN

MADOC, ONTARIO

KEY INFRASTRUCTURE AND MASTER PLAN STUDY AREA

J.L.Richards
ENGINEERS · ARCHITECTS · PLANNERS

DESIGN:	CK
DRAWN:	CK
CHECKED:	RC
JLR #:	32508-000.1

FIGURE 1

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1.2 Summary of Previous and Ongoing Work

The following is a list of recent, previously completed, and ongoing, water, wastewater, and stormwater infrastructure works in Madoc.

- Water Annual Reports (Ongoing)
- Wastewater Annual Reports (Ongoing)
- St. Lawrence Street East Design and Rehabilitation (Ongoing)
- Hydrant fire flow test (Nicol Water Services, 2023)
- Sanitary and storm manhole survey (JLR, 2023)
- Construction and commissioning of Marmora Street Drinking Water Well #4 (Ongoing)
- Centre Hastings Development Charges Background Study (Hemson, 2020)
- Madoc Sewage Lagoon Capacity Re-Rating Study (OCWA, 2021)

1.3 Class Environmental Assessment and Master Planning

The *Ontario Environmental Assessment Act* (EA Act), enacted in 1976, formally recognizes the Municipal Class Environmental Assessment (Class EA) process and outlines requirements for EA approval. The Municipal Class EA process and Master Planning process applies to municipal infrastructure projects, including roads, water, and wastewater projects. To ensure that environmental impacts and effects are considered for each project as per the EA Act, proponents are required to generally follow the planning process set out in the Municipal Class EA Guidelines, prepared by the Municipal Engineers Association (MEA), as amended in 2015 and 2023 (www.municipalclassea.ca). The Class EA process includes the following stages:

- Phase 1: Problem and/or opportunity identification.

- Phase 2: Identification and evaluation of alternative solutions to determine a preferred solution to the problem or opportunity. This Phase also compiles an environmental ‘inventory’, identifies impacts, and outlines mitigation measures.

- Phase 3: Identification and evaluation of design concepts for the preferred solution. A detailed evaluation of the environmental effects and mitigation measures are addressed during this project Phase.

- Phase 4: Complete and place Environmental Study Report on Public Record. The Report will document Phases 1 through 3 and summarize the consultation undertaken throughout the planning process and is considered valid for a 10-year period.

- Phase 5: Implementation and monitoring.

Since projects may vary in their environmental impact, they are classified in terms of the following schedules:

- Schedule ‘A’ projects usually have minimal environmental effects and generally include normal or emergency operational and maintenance activities. These projects are pre-approved under the Class EA planning process. Projects within this category are subject to Phases 1 and 5.

- Schedule ‘A+’ projects are pre-approved projects similar to Schedule ‘A’, however, the public is to be advised prior to project implementation.

- Schedule ‘B’ projects have the potential for some adverse environmental impacts and, therefore, the proponent is required to proceed through a screening process, including consultation with affected parties. Generally, these projects include improvements and minor expansions to existing facilities. Projects within this category are subject to Phases 1, 2 and 5.

- Schedule ‘C’ projects have the potential for greater environmental impacts and are subject to all five Class EA Phases. Generally, these projects include the construction of new facilities and major expansions to existing facilities.

A Master Plan is conducted under the framework of the MEA Class EA Process. It is a planning tool that identifies infrastructure requirements for existing and future land use, through the application of environmental assessment principles, and is intended to satisfy Phases 1 and 2 of the Class EA process. The Municipal Class EA guideline identifies four (4) basic approaches of the Master Planning process, including:

- Approach No. 1: This approach concludes at the end of Phases 1 and 2 of the Municipal Class EA Process. With this approach, the Master Plan is being completed at a broad level of assessment and may require further detailed assessment at the project-specific level depending on the nature of the project.

- Approach No. 2: This approach also concludes at the end of Phases 1 and 2 of the Municipal Class EA Process. However, the level of detail (i.e., investigation, consultation and documentation) fulfills the requirements for Schedule ‘B’ projects.

- Approach No. 3: This approach involves the preparation of a Master Plan document at the conclusion of Phase 4 of the Municipal Class EA Process. The level of detail of the Master Plan document can fulfill requirements for Schedule ‘B’ and/or Schedule ‘C’ projects.

- Approach No. 4: This approach involves integration with the approvals under the Planning Act.

The Madoc Water, Wastewater, and Stormwater Master Plan has followed Approach No. 1, which involves the preparation of a Report at the conclusion of Phases 1 and 2. In this case, the Master Plan has been completed at a broad level of assessment thereby requiring more detailed investigations at a project-specific level in order to fulfill the Municipal Class EA documentation requirements for any specific Schedule ‘B’ and ‘C’ projects identified within the Master Plan.

This Master Plan should be reviewed every five years to determine the need for detailed formal review and/or updates. Potential changes, which may trigger the need for an update, include:

- Major changes to the original assumptions
- Major changes to components of the Master Plan
- Significant new environmental effects
- Major changes in the proposed timing of projects within the Master Plan based on changed conditions relative to the original projections/predictions.

2.0 Phase 1 Methodology

2.1 Project Initiation Meeting and Site Visits

A project initiation meeting was held on September 19th, 2023, with representatives from the Municipality, OCWA, and JLR to confirm roles and responsibilities, project understanding, proposed work plan and schedule and to review current and historical issues associated with the Municipality's water, wastewater, and stormwater systems.

A site visit was undertaken on September 19th, 2023, to understand conditions of the drinking water pump houses and equipment and wastewater treatment lagoon. The site visit was limited to visual observations and discussion with OCWA operators.

JLR staff had undertaken extensive field work to collect sanitary sewer and manhole inverts, as well as the storm sewer, manhole, and catch basin inverts.

2.2 Compilation and Review of Existing Documentation

A comprehensive inventory of available historical reports, permits/approvals, studies, drawings, and GIS data related to the current water, wastewater, and storm infrastructure was developed. The documentation provided was publicly available or provided by the Municipality and OCWA. Several key documents are referenced herein. The data collected was reviewed and analyzed to establish current operating conditions for each system.

2.3 Consultation Planning and Contact with Stakeholders

A Public Consultation Plan was developed and submitted to the Municipality and OCWA for review, taking into consideration mandatory requirements and objectives of effective consultation with the public and other potential stakeholders, as outlined in the MEA Class EA document (refer to Appendix A for a copy of the Stakeholder Consultation Plan, dated November 13th, 2023). The Plan identifies potential stakeholders, defines the level of consultation, establishes appropriate means of contact, and provides a schedule highlighting the general timing of contact. A comprehensive stakeholder contact list, consisting of the MECP's Government Review Team and Agency Contacts, and the Municipality's local stakeholders to ensure all interested agencies and stakeholders are involved in the consultation process.

2.4 Phase 1 Report

This Phase 1 Report was prepared to summarize the findings from the first phase of the Master Plan process and to use as a basis for the identification and evaluation of alternative options during Phase 2.

The objectives of this Report are:

- To establish 30-year future growth projections.
- To provide a description of existing conditions and constraints associated with the water, wastewater, and stormwater infrastructure within Madoc, including a summary of historical water/wastewater flows and water/wastewater quality, and findings from models of each system.

- To determine the residual capacity for water supply/treatment and sewage lagoon.
- To provide anticipated timing for when rated capacities of each system will be reached.
- To establish proposed design basis for future servicing needs.
- To identify land use and planning constraints, and natural environment constraints.
- To establish a Problem/Opportunity Statement.

3.0 Design Basis

3.1 Existing Service Connections

The number of existing units or service connections was approximated as the number of water meters serviced within Madoc. Water consumption data provided by the Municipality was used to confirm that there are 673 water meters serviced in Madoc.

The existing service population of 1,489 was collected from 2021 Census data. The Census population and assumed number of service connections generally aligns with the total number of dwellings (1,975) and population (4,858) from the 2020 Development Charges Study completed for the Municipality of Centre Hastings, which includes the Study Area.

3.2 Growth Projections

3.2.1 Planning Periods

For the purposes of this Master Plan, population and flow projections and servicing recommendations have been categorized for the short-term (0-5 Years; 2024 to 2029), mid-term (5-10 Years; 2029 to 2034), long-term (10-20 Years; 2034 to 2044), and build-out (20-30 Years; 2044 to 2054) planning periods.

3.2.2 Future Growth

The Municipality provided a list of planned residential developments which were categorized into residential development types listed in Table 1. The population densities, expressed in persons per unit (PPU), for typical residential development types were taken from the Hemson Development Charges Study (2020). Population densities used for retirement residence developments were provided by the Municipality. The population densities were multiplied by the number of units (provided by the Municipality) in order to create population projections for future planning periods.

Table 1: Population Density by Development Type

Development Type	Population Density (Persons per Unit, PPU)
Singles & Semis (1)	3.26
Rows & Other Multiples (1)	2.25
Apartments (1)	1.50
Independent and Supported Living Retirement Residence (2)	1.20
Independent Living Retirement Residence (2)	2.00

(1) Population density obtained from Hemson Development Charges Study (2020).
(2) Population density provided by the Municipality.

The Municipality's planned residential developments, anticipated number of units, and projected population growth for the short, mid, long, and build-out term planning periods are summarized from Table 2 to Table 5. The residential growth projection was used as the design basis for this Master Plan. It was assumed that there are 673 existing units which aligns with the existing number of water service connections in Madoc. A 5% intensification factor was applied to the existing units to provide a contingency for factors such as additional developments and the effects of Ontario Bill 23 – More Homes Faster Act. Bill 23 allows homeowners to build up to three additional residential units on their property. The Municipality noted that they have not received any permit applications to date for additional units due to Bill 23.

Table 2: Short-Term (0-5 Years; 2024-2029) Residential Development Units and Population Projections

ID		Development	Type	Timeframe (1)	Units (2)	Population
1	A	Danford's - Phase 1	Semis and Singles	0-5 Years	15	49
1	B	Danford's - Phase 1	Rows & Other Multiples	0-5 Years	5	11
2	A	Bonter's	Semis and Singles	0-5 Years	17	55
2	B	Bonter's	Rows & Other Multiples	0-5 Years	15	34
3		Moira Meadows	Semis and Singles	0-5 Years	24	78
4		95 Rollins St.	Apartments	0-5 Years	29	44
5		108 Russel St.	Apartments	0-5 Years	9	14
6		Seymour St. W #2	Semis and Singles	0-5 Years	1	3
7		29 Rollins St.	Semis and Singles	0-5 Years	2	7
8		Champlain St.	Semis and Singles	0-5 Years	1	3
9		75 Baldwin St.	Semis and Singles	0-5 Years	1	3

(Table 2 Continued)

ID		Development	Type	Timeframe (1)	Units (2)	Population
10		Elgin and McKenzie	Semis and Singles	0-5 Years	7	23
11	A	Morey	Semis and Singles	0-5 Years	8	26
11	B	Morey	Rows & Other Multiples	0-5 Years	12	27
		Intensification of Existing Units (3)(4)		0-5 Years	9	23
Total Short-Term Growth (0-5 Years; 2024-2029) (5)					155	400

(1) Timeframes presented herein indicate the anticipated timeline when the development will be connected to the water/wastewater services. This is not an indication of when the development can begin. The actual timing of upgrades will be contingent on the rate of development in each of the contributing areas.
(2) Unit counts provided by the Municipality.
(3) Assume 5% intensification for existing dwelling units divided evenly into each timeline term.
(4) Number of existing dwellings approximated by number of water meters serviced by Madoc.

(5) Total units and population are not cumulative and do not include existing service population and units.

Table 3: Mid-Term (5-10 Years; 2029-2034) Residential Development Units and Population Projections

ID		Development	Type	Timeframe (2)	Units (3)	Population
12		Duncan St. (1)	Semis and Singles	5-10 Years	21	68
13		Former Becker's	Apartments	5-10 Years	9	14
14		Danford's - Phase 2	Semis and Singles	5-10 Years	22	72
15		Durham St. S Development #2	Semis and Singles	5-10 Years	4	13
16		McKenzie St. (1)	Semis and Singles	5-10 Years	142	463
17	A	Bonjour Blvd.	Independent and Supported Living Retirement Residence	5-10 Years	60	72
17	B	Bonjour Blvd.	Independent Living Retirement Residence	5-10 Years	60	120
18		Marmora St. (1)	Semis and Singles	5-10 Years	7	23
19		Elgin and St. Lawrence (1)	Semis and Singles	5-10 Years	5	16
20		Seymour St. W	Semis and Singles	5-10 Years	2	7
		Intensification of Existing Units (4)(5)		5-10 Years	9	23
Total Mid-Term Growth (5-10 Years; 2029-2034) (6)					341	891

(1) Number of Units estimated based on 2002-10 Municipality of Centre Hastings Zoning By-law.
(2) Timeframes presented herein indicate the anticipated timeline when the development will be connected to the water/wastewater services. This is not an indication of when the development can

begin. The actual timing of upgrades will be contingent on the rate of development in each of the contributing areas.
(3) Unit counts provided by the Municipality.
(4) Assume 5% intensification for existing dwelling units divided evenly into each timeline term.
(5) Number of existing dwellings approximated by number of water meters serviced by Madoc.
(6) Total units and population are not cumulative and do not include existing service population and units.

Table 4: Long-Term (10-20 Years; 2034-2044) Residential Development Units and Population Projections

ID		Development	Type	Timeframe (2)	Units (3)	Population
21		Durham St. S Development #1	Semis and Singles	10-20 Years	4	13
22		Gladstone St E	Semis and Singles	10-20 Years	45	147
23		Marmora St.	Rows & Other Multiples	10-20 Years	400	900
24		Concession Rd (1)	Semis and Singles	10-20 Years	1	3
25		St. Lawrence E.	Semis and Singles	10-20 Years	100	326
26		35 Seymour St E (1)	Semis and Singles	10-20 Years	1	3
27		Maud St.	Semis and Singles	10-20 Years	24	78
28		McNab St.	Semis and Singles	10-20 Years	2	7
29		Richardson Lane (7)	Rows & Other Multiples	10-20 Years	16	36
		Intensification of Existing Units (4)(5)	Historical Population Density	10-20 Years	18	46
Total Long-Term Growth (10-20 Years; 2034 to 2044) (6)					611	1,559

(1) Number of Units estimated based on 2002-10 Municipality of Centre Hastings Zoning By-law.
(2) Timeframes presented herein indicate the anticipated timeline when the development will be connected to the water/wastewater services. This is not an indication of when the development can begin. The actual timing of upgrades will be contingent on the rate of development in each of the contributing areas.
(3) Unit counts provided by the Municipality.
(4) Assume 5% intensification for existing dwelling units divided evenly into each timeline term.
(5) Number of existing dwellings approximated by number of water meters serviced by Madoc.
(6) Total units and population are not cumulative and do not include existing service population and units.
(7) Number of units based on existing Richardson Lane development.

Table 5: Build-Out (20-30 Years; 2034-2044) Residential Development Units and Population Projections

ID		Development	Type	Timeframe (2)	Units (3)	Population
30		Whytock Park Property	Semis and Singles	20-30 Years	150	489
31		Wellington St.	Semis and Singles	20-30 Years	22	72
32		231 Seymour St. W	Semis and Singles	20-30 Years	549	1,790
33		105 Seymour St. W	Semis and Singles	20-30 Years	291	949
34		Rollins St. (1)	Semis and Singles	20-30 Years	2	7
		Intensification of Existing Units (4)(5)	Historical Population Density	20-30 Years	18	46
Total Build-Out Growth (20 - 30 Years; 2044 to 2054) (6)					1,032	3,353

(1) Number of Units estimated based on 2002-10 Municipality of Centre Hastings Zoning By-law.
(2) Timeframes presented herein indicate the anticipated timeline when the development will be connected to the water/wastewater services. This is not an indication of when the development can begin. The actual timing of upgrades will be contingent on the rate of development in each of the contributing areas.
(3) Unit counts provided by the Municipality.
(4) Assume 5% intensification for existing dwelling units divided evenly into each timeline term.
(5) Number of existing dwellings approximated by number of water meters serviced by Madoc.
(6) Total units and population are not cumulative and do not include existing service population and units.

The Municipality provided the following Institutional, Commercial, and Industrial (ICI) planned developments, which were categorized into hospital, commercial, and typical industrial development types and summarized from Table 6 to Table 9. The ICI growth presented in these tables served as the design basis for the master plan.

Table 6: Short-Term (0-5 Years; 2024-2029) ICI Development Growth

ID	Development	Type	Timeframe (2)	Beds (1)
35	Long Term Care	Hospital	0-5 Years	128
Total Short-Term Growth (0-5 Years; 2024-2029)				128

(1) Number of beds provided by Municipality.
(2) Timeframes presented herein indicate the anticipated timeline when the development will be connected to the water/wastewater services. This is not an indication of when the development can begin. The actual timing of upgrades will be contingent on the rate of development in each of the contributing areas.

Table 7: Mid-Term (20-30 Years; 2034-2044) ICI Development Growth

ID	Development	Type	Timeframe (2)	Hectares (1)
36	Hwy 7 Commercial Development #1	Commercial	5-10 Years	1.5
37	Hwy 7 Commercial Development #2	Commercial	5-10 Years	2.3
Total Mid-Term Growth (5 to 10 Years; 2029 to 2034) (3)				**3.8**

(1) Number of hectares provided by Municipality.
(2) Timeframes presented herein indicate the anticipated timeline when the development will be connected to the water/wastewater services. This is not an indication of when the development can begin. The actual timing of upgrades will be contingent on the rate of development in each of the contributing areas.
(3) Total number of beds are not cumulative and do not include existing units.

Table 8: Long-Term (10 to 20 Years; 2034 to 2044) ICI Development Growth

ID	Development	Type	Timeframe (2)	Hectares (1)
38	Hwy 62 Commercial Properties	Commercial	10-20 Years	2.8
39	Downtown Core Commercial Lots	Commercial	10-20 Years	0.1
40	Hill Ave/Burnside St. Commercial Lots	Commercial	10-20 Years	5.0
41	Hill Ave/Burnside St. Industrial Lots	Typical Industrial	10-20 Years	2.3
Total Long-Term Growth (10 to 20 Years; 2034 to 2044)(3)				**10.3**

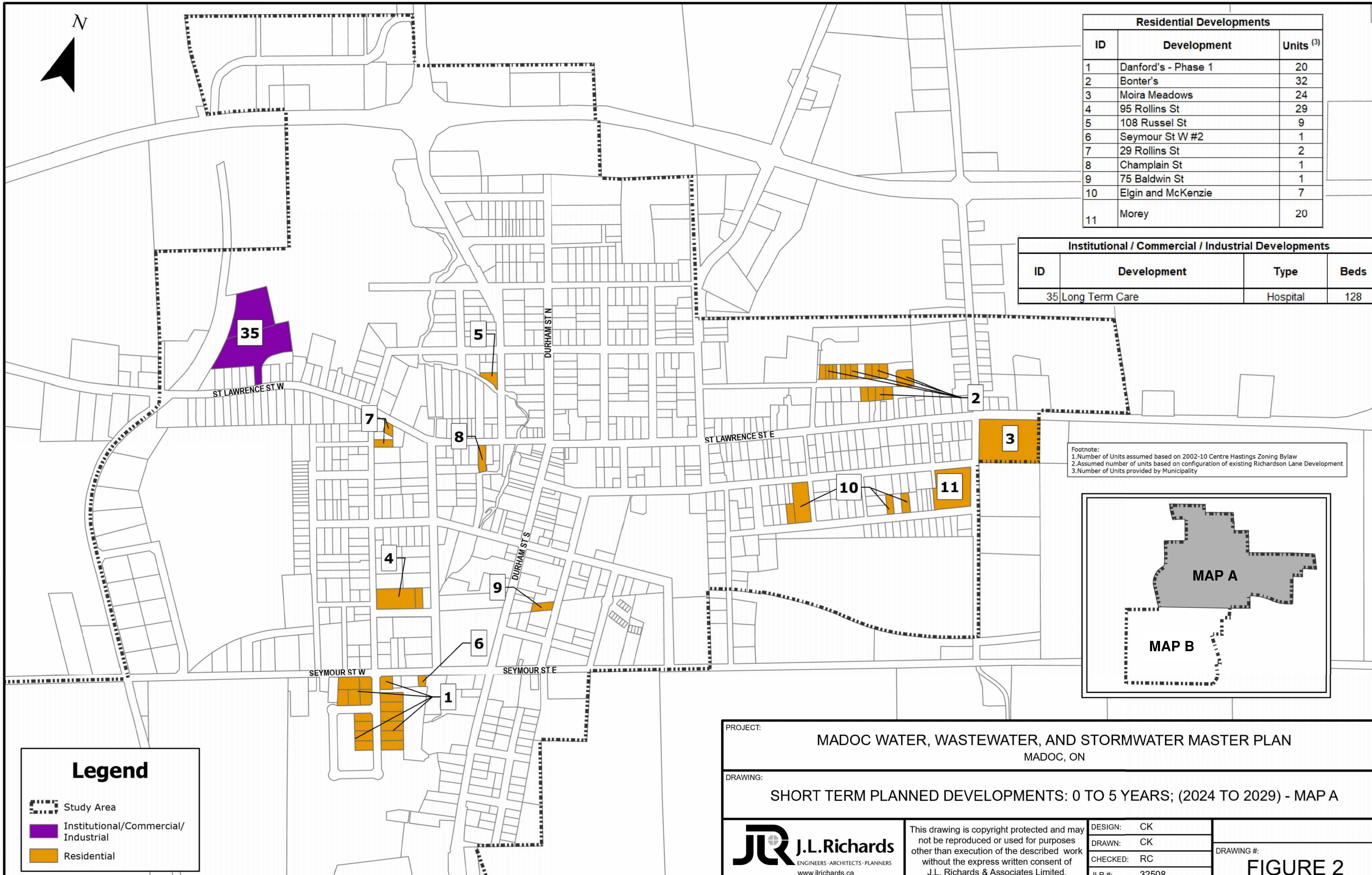
(1) Number of hectares provided by Municipality.
(2) Timeframes presented herein indicate the anticipated timeline when the development will be connected to the water/wastewater services. This is not an indication of when the development can begin. The actual timing of upgrades will be contingent on the rate of development in each of the contributing areas.
(3) Total number of beds are not cumulative and do not include existing units.

Table 9: Build-Out (20 to 30 Years; 2044 to 2054) ICI Development Growth

ID	Development	Type	Timeframe (2)	Hectares (1)
42	Hwy 7 Commercial Development #3	Commercial	20-30 Years	2.5
Total Build-Out Growth (20 to 30 Years; 2044 to 2054) (3)				**2.5**

(1) Number of hectares provided by Municipality.
(2) Timeframes presented herein indicate the anticipated timeline when the development will be connected to the water/wastewater services. This is not an indication of when the development can begin. The actual timing of upgrades will be contingent on the rate of development in each of the contributing areas.
(3) Total number of beds are not cumulative and do not include existing units.

Locations of residential and institutional developments were obtained from roll numbers provided by the Municipality and overlaid on parcel fabric obtained from County of Hastings GIS to create Figure 2 to Figure 9. The following maps identify all residential and ICI developments presented for each planning period as listed in the preceding tables. The development locations will serve as the design basis for future water distribution, sanitary sewer, and storm sewer network capacity modelling in Phase 2 of the Master Plan.



Residential Developments

ID	Development	Units [3]
1	Danford's - Phase 1	20
2	Bonter's	32
3	Moira Meadows	24
4	95 Rollins St	29
5	108 Russel St	9
6	Seymour St W #2	1
7	29 Rollins St	2
8	Champlain St	1
9	75 Baldwin St	1
10	Elgin and McKenzie	7
11	Morey	20

Institutional / Commercial / Industrial Developments

ID	Development	Type	Beds
35	Long Term Care	Hospital	128

Footnote:
1. Number of Units assumed based on 2002-10 Centre Hastings Zoning Bylaw
2. Assumed number of units based on configuration of existing Richardson Lane Development
3. Number of Units provided by Municipality

Legend

- Study Area
- Institutional/Commercial/ Industrial
- Residential

PROJECT:

MADOC WATER, WASTEWATER, AND STORMWATER MASTER PLAN
MADOC, ON

DRAWING:

SHORT TERM PLANNED DEVELOPMENTS: 0 TO 5 YEARS; (2024 TO 2029) - MAP A

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DESIGN:	CK
DRAWN:	CK
CHECKED:	RC
JLR #:	32508

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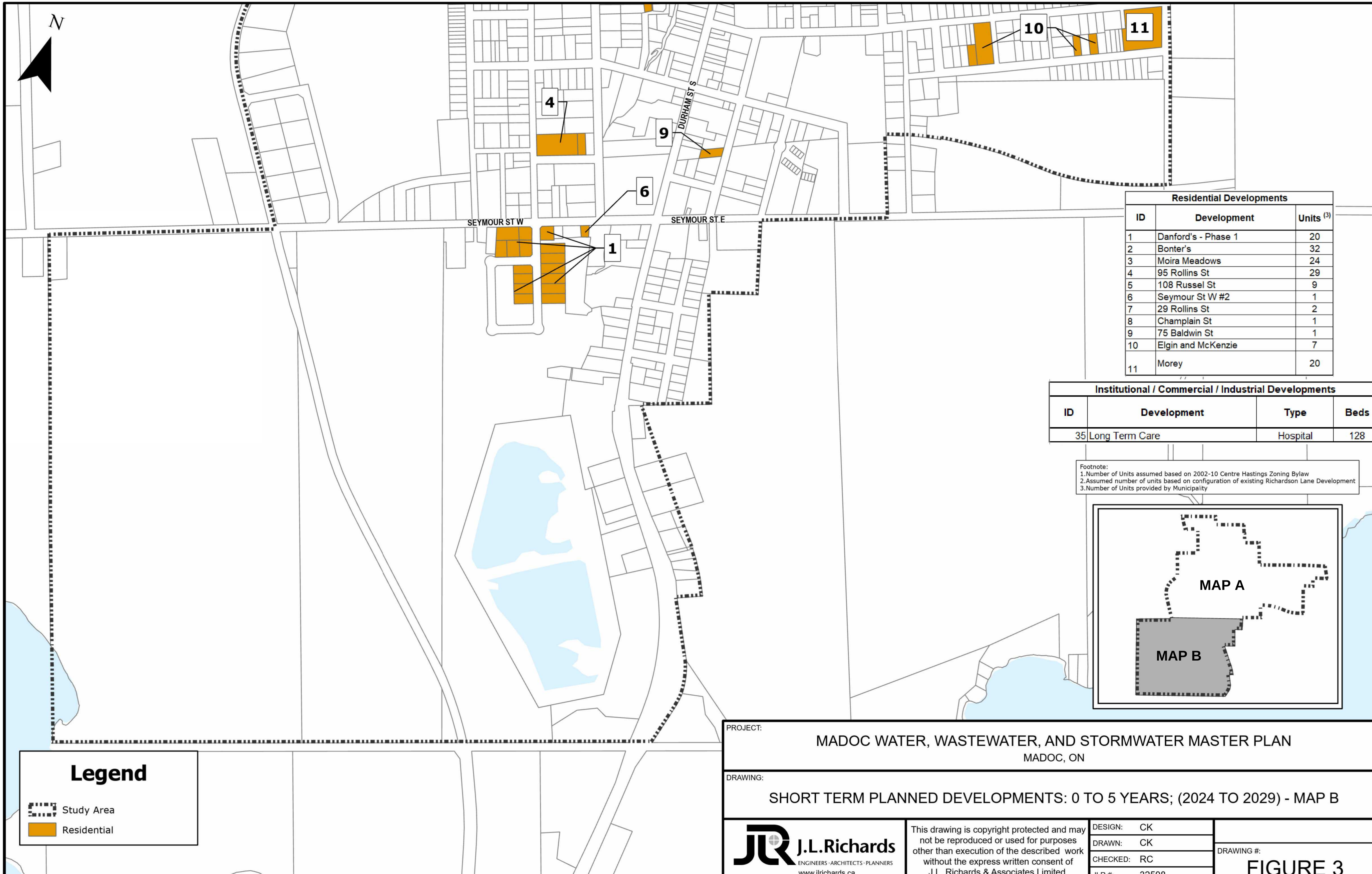

Residential Developments

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9	75 Baldwin St	1
10	Elgin and McKenzie	7
11	Morey	20

Institutional / Commercial / Industrial Developments

ID	Development	Type	Beds
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PROJECT:
MADOC WATER, WASTEWATER, AND STORMWATER MASTER PLAN
MADOC, ON

DRAWING:
SHORT TERM PLANNED DEVELOPMENTS: 0 TO 5 YEARS; (2024 TO 2029) - MAP B

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CHECKED: RC
JLR #: 32508

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FIGURE 3

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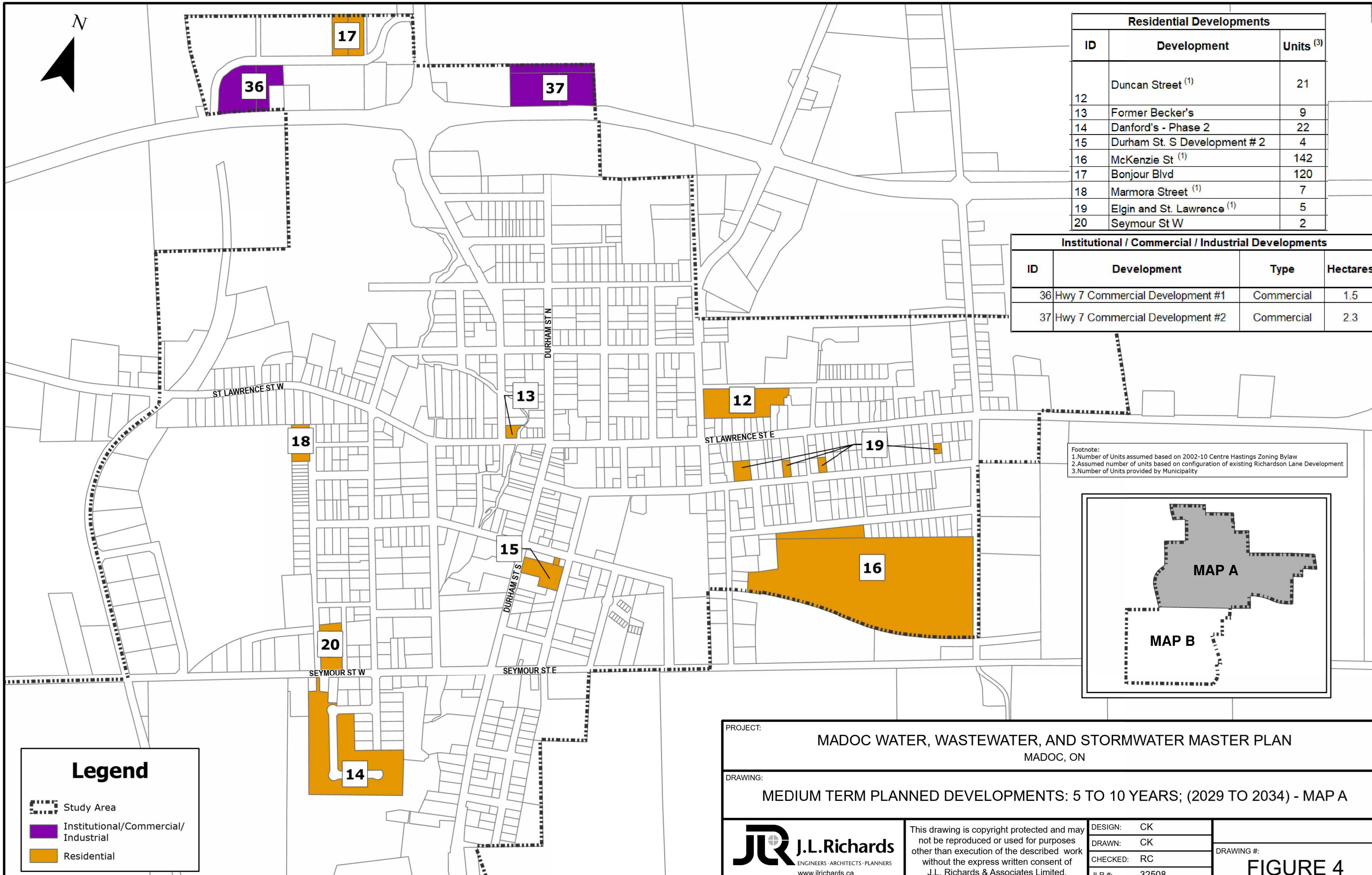

Residential Developments		
ID	Development	Units [3]
12	Duncan Street [1]	21
13	Former Becker's	9
14	Danford's - Phase 2	22
15	Durham St. S Development # 2	4
16	McKenzie St [1]	142
17	Bonjour Blvd	120
18	Marmora Street [1]	7
19	Elgin and St. Lawrence [1]	5
20	Seymour St W	2

Institutional / Commercial / Industrial Developments			
ID	Development	Type	Hectares
36	Hwy 7 Commercial Development #1	Commercial	1.5
37	Hwy 7 Commercial Development #2	Commercial	2.3

Footnote:
1.Number of Units assumed based on 2002-10 Centre Hastings Zoning Bylaw
2.Assumed number of units based on configuration of existing Richardson Lane Development
3.Number of Units provided by Municipality

Legend

- Study Area
- Institutional/Commercial/Industrial
- Residential

PROJECT:
MADOC WATER, WASTEWATER, AND STORMWATER MASTER PLAN
MADOC, ON

DRAWING:
MEDIUM TERM PLANNED DEVELOPMENTS: 5 TO 10 YEARS; (2029 TO 2034) - MAP A

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FIGURE 4

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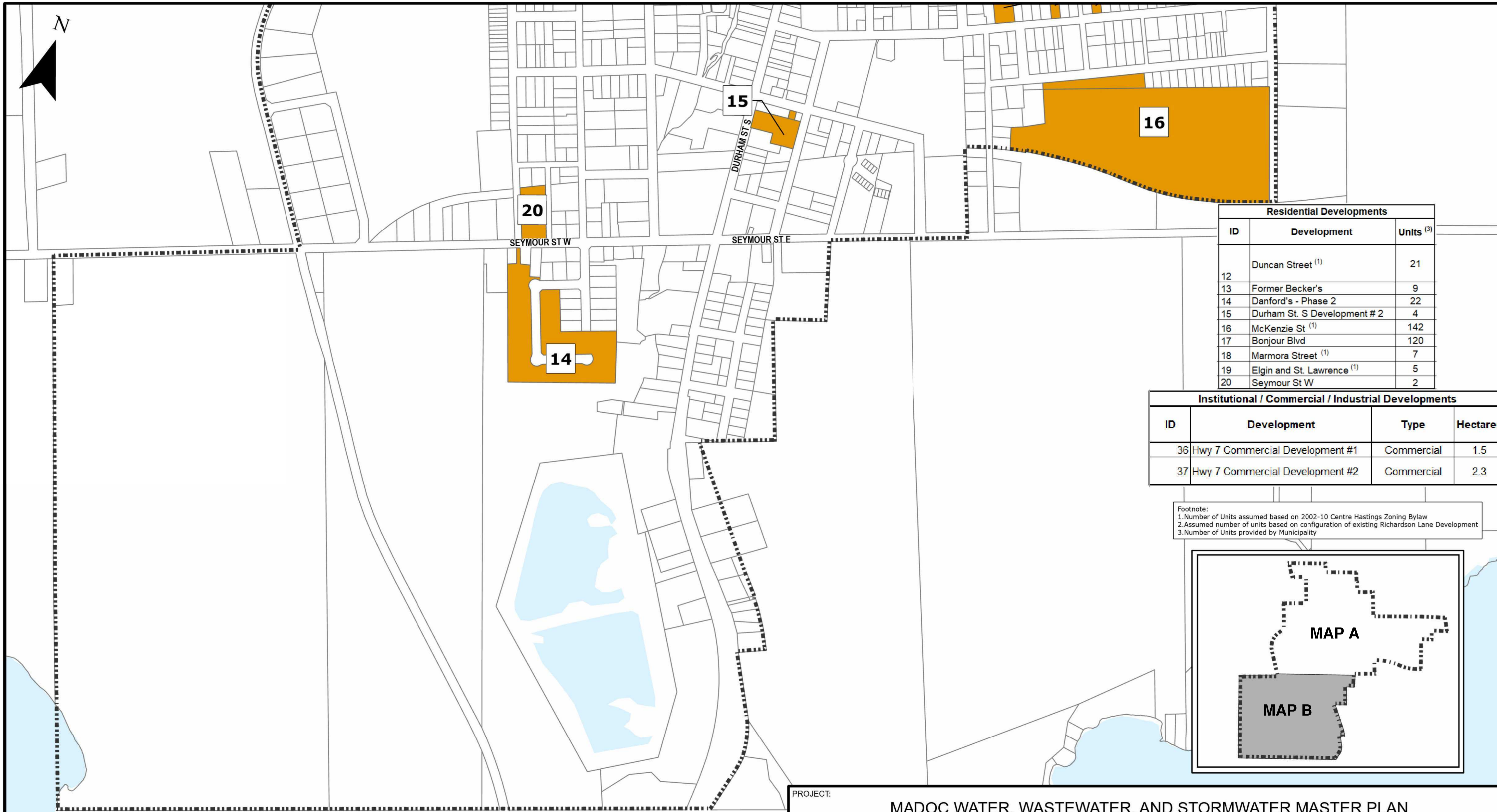

Residential Developments		
ID	Development	Units (3)
12	Duncan Street (1)	21
13	Former Becker's	9
14	Danford's - Phase 2	22
15	Durham St. S Development # 2	4
16	McKenzie St (1)	142
17	Bonjour Blvd	120
18	Marmora Street (1)	7
19	Elgin and St. Lawrence (1)	5
20	Seymour St W	2

Institutional / Commercial / Industrial Developments			
ID	Development	Type	Hectares
36	Hwy 7 Commercial Development #1	Commercial	1.5
37	Hwy 7 Commercial Development #2	Commercial	2.3

Footnote:
1.Number of Units assumed based on 2002-10 Centre Hastings Zoning Bylaw
2.Assumed number of units based on configuration of existing Richardson Lane Development
3.Number of Units provided by Municipality

Legend

- Study Area
- Residential

PROJECT:
MADOC WATER, WASTEWATER, AND STORMWATER MASTER PLAN
MADOC, ON

DRAWING:
MEDIUM TERM PLANNED DEVELOPMENTS: 5 TO 10 YEARS; (2029 TO 2034) - MAP B

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FIGURE 5

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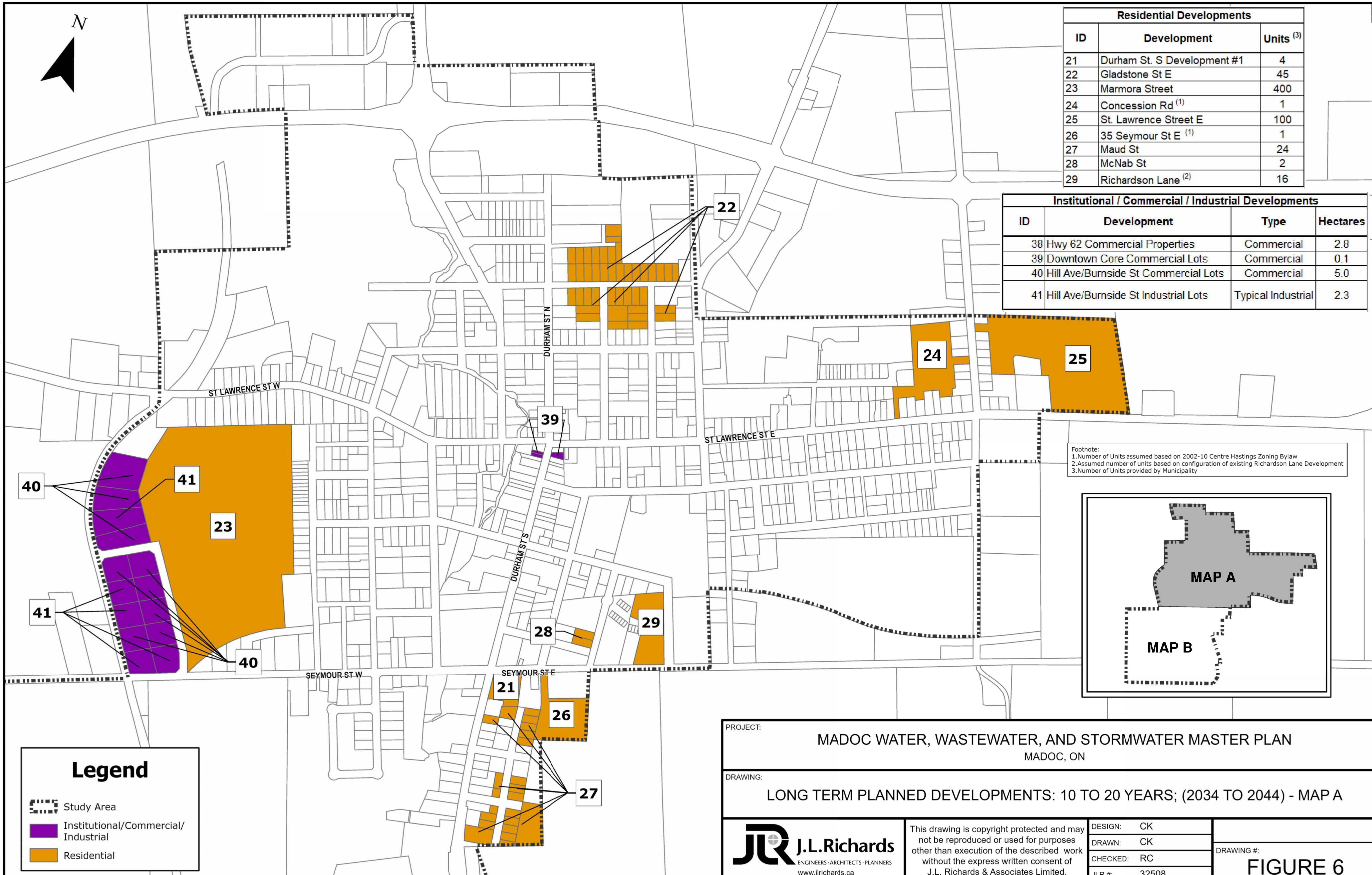

Residential Developments		
ID	Development	Units [3]
21	Durham St. S Development #1	4
22	Gladstone St E	45
23	Marmora Street	400
24	Concession Rd [1]	1
25	St. Lawrence Street E	100
26	35 Seymour St E [1]	1
27	Maud St	24
28	McNab St	2
29	Richardson Lane [2]	16

Institutional / Commercial / Industrial Developments			
ID	Development	Type	Hectares
38	Hwy 62 Commercial Properties	Commercial	2.8
39	Downtown Core Commercial Lots	Commercial	0.1
40	Hill Ave/Burnside St Commercial Lots	Commercial	5.0
41	Hill Ave/Burnside St Industrial Lots	Typical Industrial	2.3

Footnote:
1.Number of Units assumed based on 2002-10 Centre Hastings Zoning Bylaw
2.Assumed number of units based on configuration of existing Richardson Lane Development
3.Number of Units provided by Municipality

PROJECT:

MADOC WATER, WASTEWATER, AND STORMWATER MASTER PLAN
MADOC, ON

DRAWING:

LONG TERM PLANNED DEVELOPMENTS: 10 TO 20 YEARS; (2034 TO 2044) - MAP A

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FIGURE 6

File Location: P:\32000\32508-000 - Madoc W-WW-SW MP\05-Production\01-Civil\GIS\32508_ArcGIS_Pro\32508_ArcGIS_Pro.aprx

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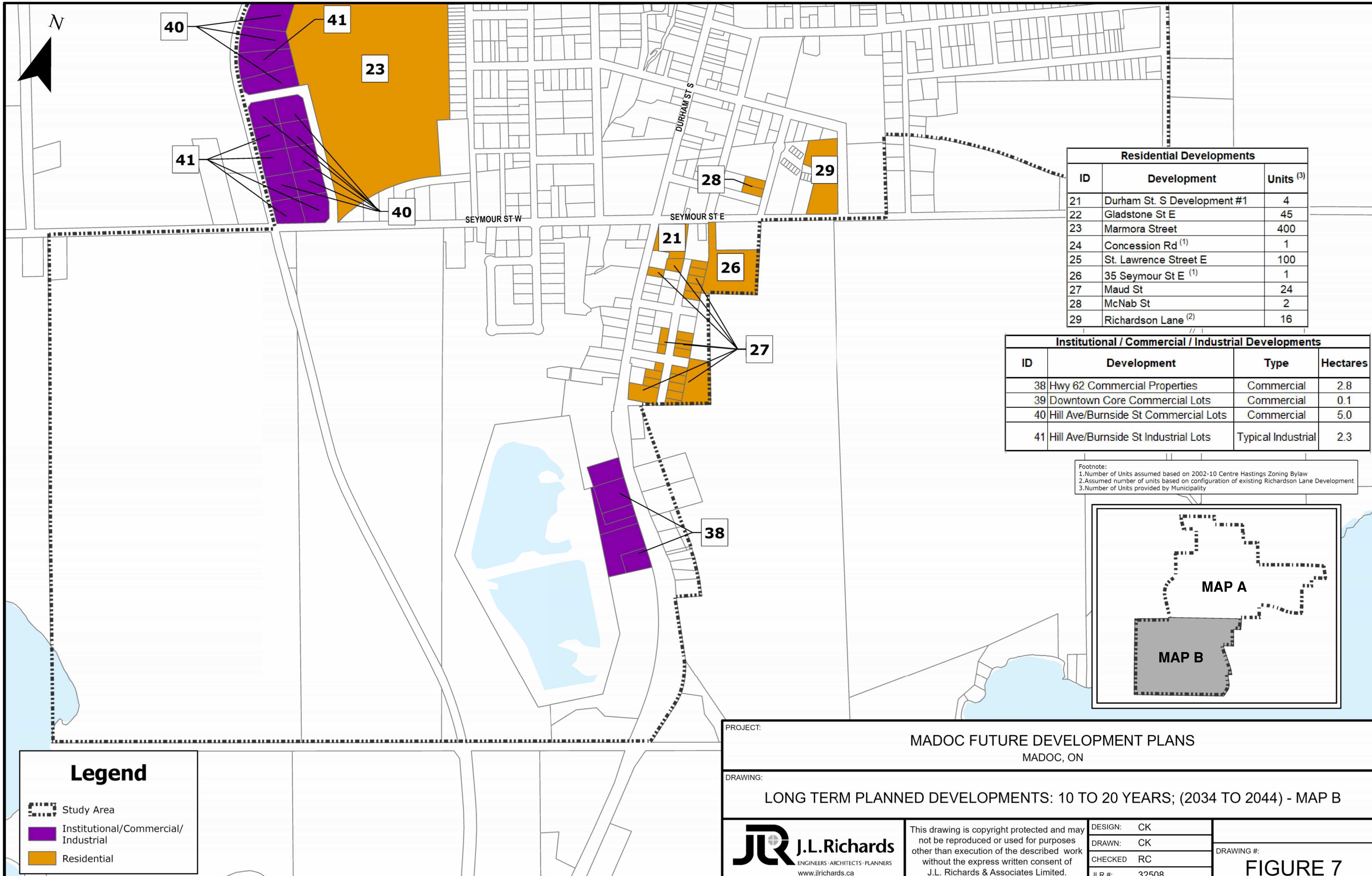

Residential Developments

ID	Development	Units (3)
21	Durham St. S Development #1	4
22	Gladstone St E	45
23	Marmora Street	400
24	Concession Rd (1)	1
25	St. Lawrence Street E	100
26	35 Seymour St E (1)	1
27	Maud St	24
28	McNab St	2
29	Richardson Lane (2)	16

Institutional / Commercial / Industrial Developments

ID	Development	Type	Hectares
38	Hwy 62 Commercial Properties	Commercial	2.8
39	Downtown Core Commercial Lots	Commercial	0.1
40	Hill Ave/Burnside St Commercial Lots	Commercial	5.0
41	Hill Ave/Burnside St Industrial Lots	Typical Industrial	2.3

Footnote:
1. Number of Units assumed based on 2002-10 Centre Hastings Zoning Bylaw
2. Assumed number of units based on configuration of existing Richardson Lane Development
3. Number of Units provided by Municipality

PROJECT:
MADOC FUTURE DEVELOPMENT PLANS
MADOC, ON

DRAWING:
LONG TERM PLANNED DEVELOPMENTS: 10 TO 20 YEARS; (2034 TO 2044) - MAP B

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DRAWING #: FIGURE 7

File Location: P:\32000\32508-000 - Madoc W-WW-SW MP\05-Production\01-Civil\GIS\32508_ArcGIS_Pro\32508_ArcGIS_Pro.aprx

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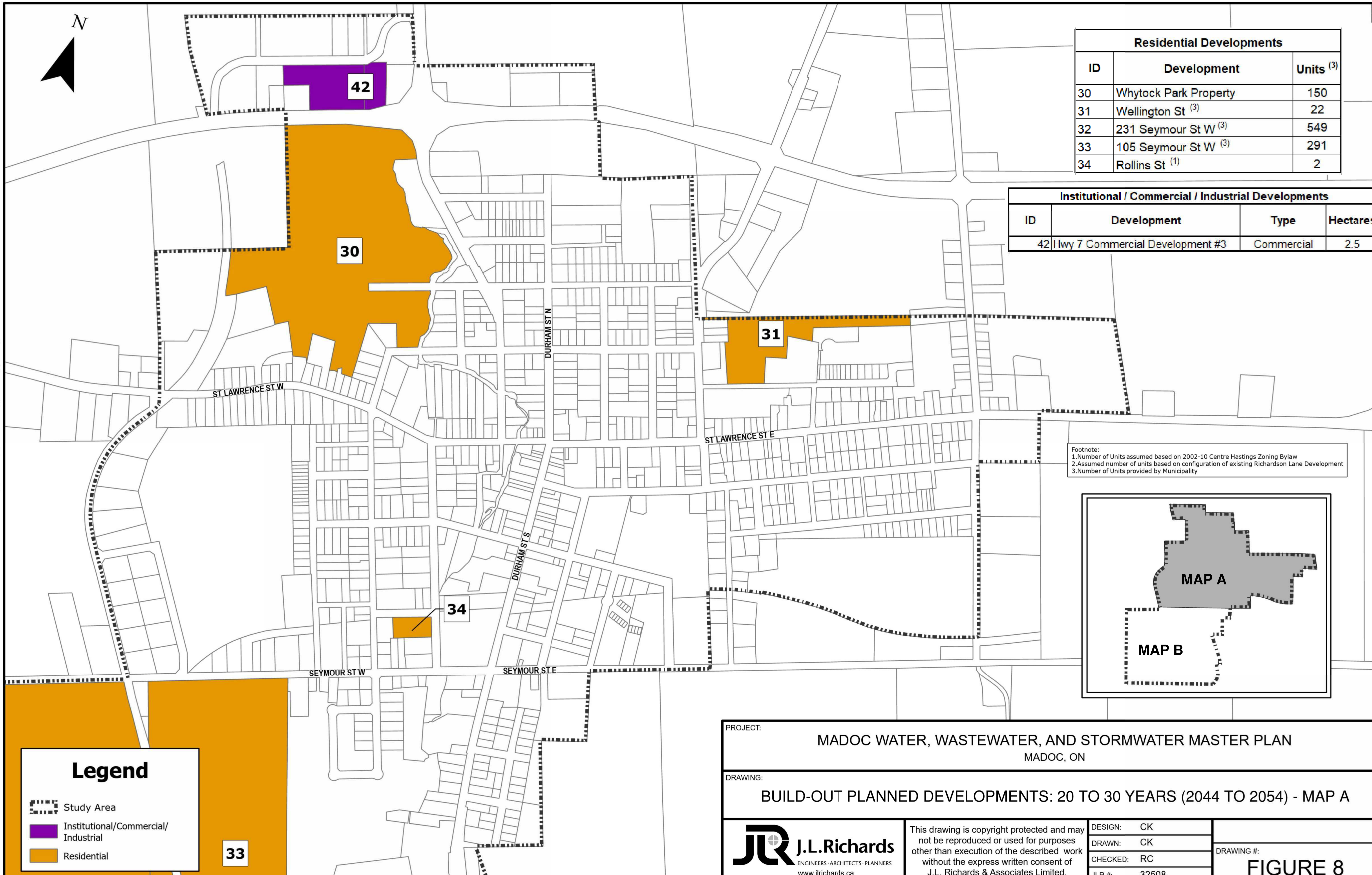

Residential Developments		
ID	Development	Units [3]
30	Whytock Park Property	150
31	Wellington St [3]	22
32	231 Seymour St W [3]	549
33	105 Seymour St W [3]	291
34	Rollins St [1]	2

Institutional / Commercial / Industrial Developments			
ID	Development	Type	Hectares
42	Hwy 7 Commercial Development #3	Commercial	2.5

Footnote:
1.Number of Units assumed based on 2002-10 Centre Hastings Zoning Bylaw
2.Assumed number of units based on configuration of existing Richardson Lane Development
3.Number of Units provided by Municipality

PROJECT:
MADOC WATER, WASTEWATER, AND STORMWATER MASTER PLAN
MADOC, ON

DRAWING:
BUILD-OUT PLANNED DEVELOPMENTS: 20 TO 30 YEARS (2044 TO 2054) - MAP A

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FIGURE 8

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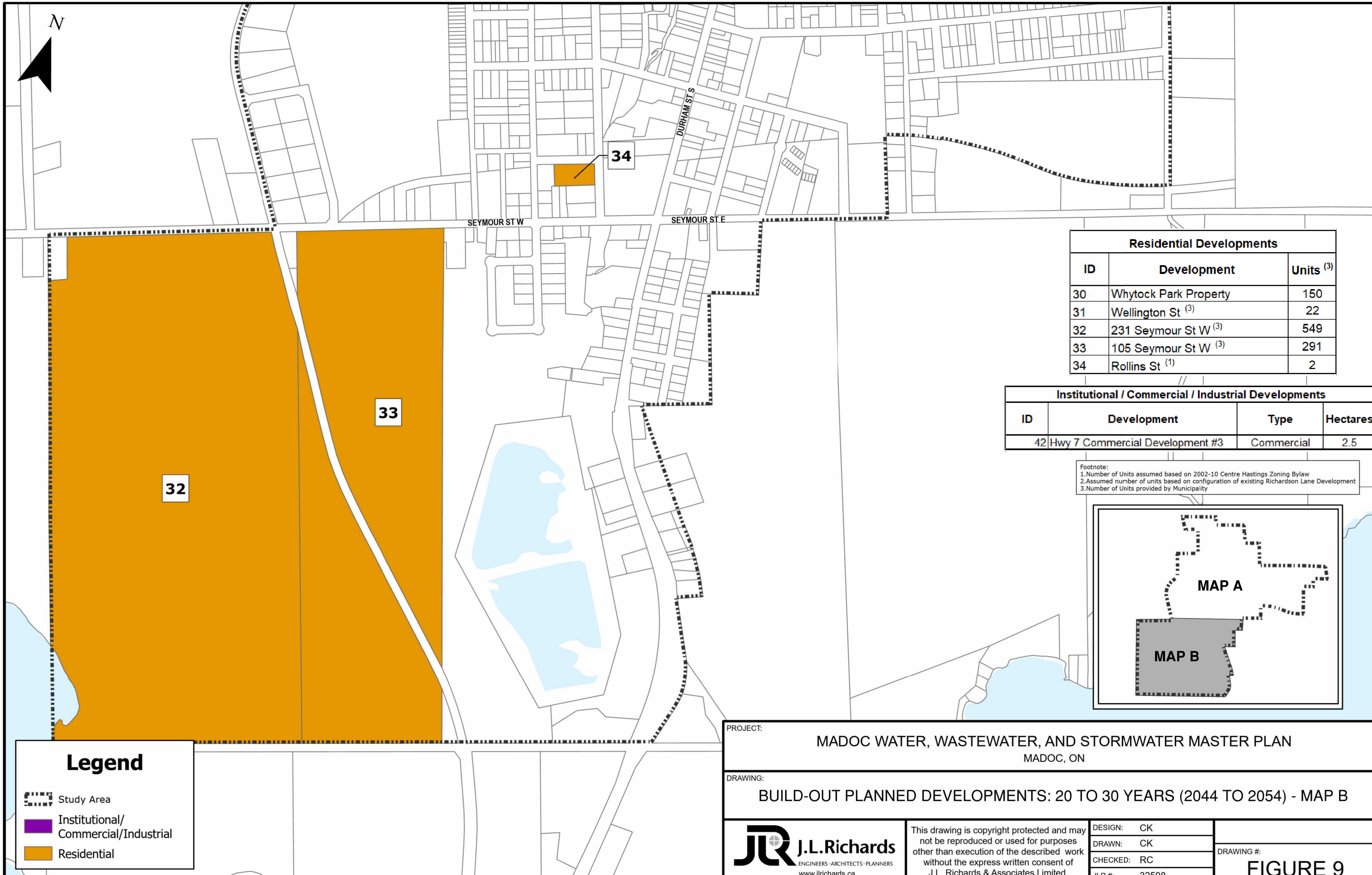

Residential Developments

ID	Development	Units [3]
30	Whytock Park Property	150
31	Wellington St [3]	22
32	231 Seymour St W [3]	549
33	105 Seymour St W [3]	291
34	Rollins St [1]	2

Institutional / Commercial / Industrial Developments

ID	Development	Type	Hectares
42	Hwy 7 Commercial Development #3	Commercial	2.5

Footnote:
1.Number of Units assumed based on 2002-10 Centre Hastings Zoning Bylaw
2.Assumed number of units based on configuration of existing Richardson Lane Development
3.Number of Units provided by Municipality

Legend

- Study Area
- Institutional/ Commercial/Industrial
- Residential

PROJECT:
MADOC WATER, WASTEWATER, AND STORMWATER MASTER PLAN
MADOC, ON

DRAWING:
BUILD-OUT PLANNED DEVELOPMENTS: 20 TO 30 YEARS (2044 TO 2054) - MAP B

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FIGURE 9

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3.3 Natural Environment

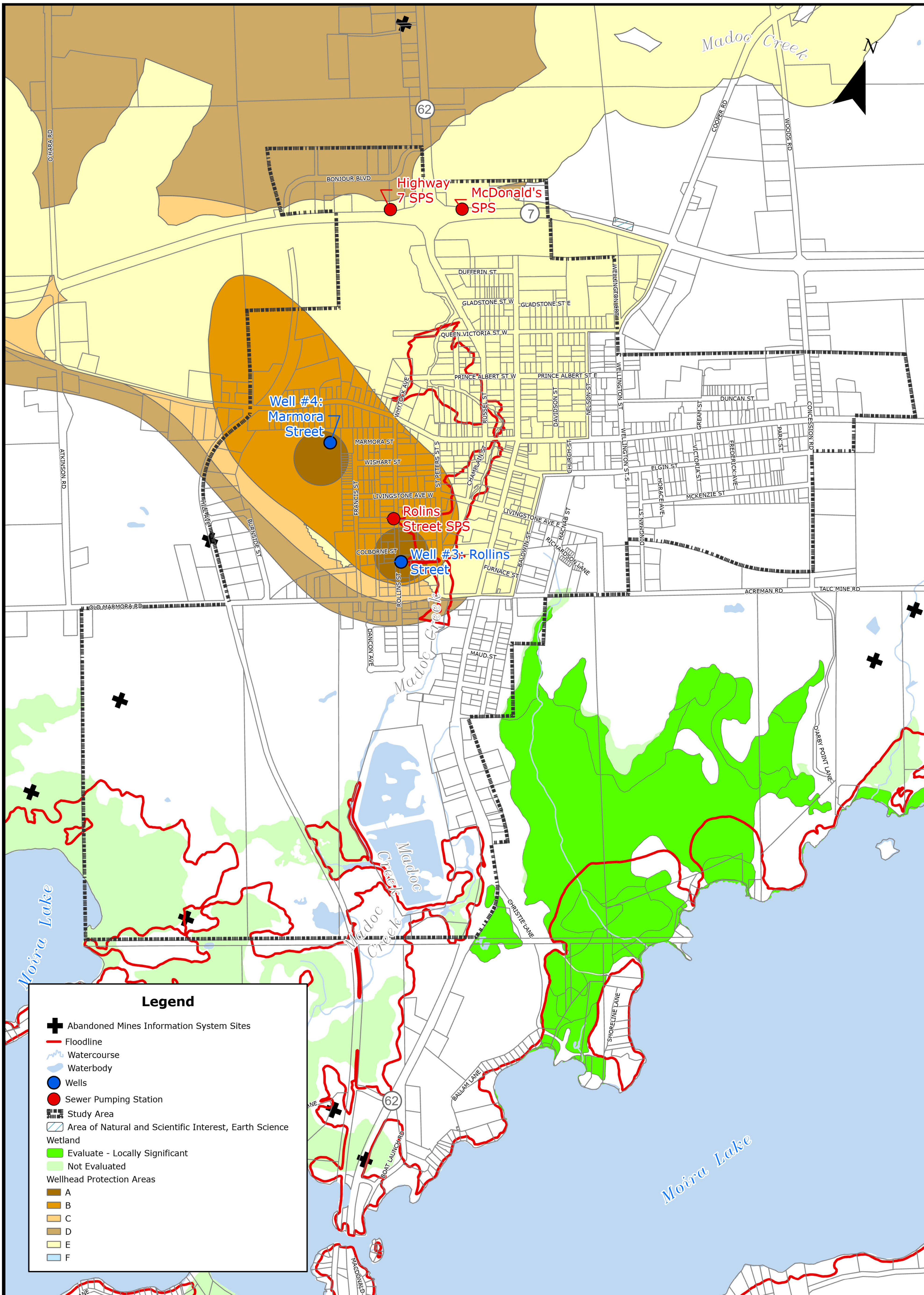
Refer to Figure 10, which identifies significant landforms, groundwater and floodplain areas, and source water protection zones within the study area.

Well #3 and Well #4 are both categorized as groundwater under the direct influence of surface water of the well head protection areas (WHPA) shown in Figure 8.

Three abandoned mines were identified within the study area. Two abandoned mines are located within the 231 Seymour Street development area (ID No. 30) and one abandoned mine is located on the border of the Hill Avenue/Burnside St Commercial Lots (ID No. 41). There are no abandoned mines within the WHPAs for Well #3 and Well #4.

There are areas of locally significant and unevaluated wetlands located near the south and north borders of the study. The Quinte Conservation Authority regulates developments within and adjacent to Deer (Madoc) Creek, unevaluated wetlands, and Madoc Wetland Provincially Significant Wetlands.

Limitations associated with natural environment constraints will be further explored in Phase 2.



VILLAGE OF MADOC WATER AND WASTEWATER MASTER PLAN

MADOC, ONTARIO

NATURAL ENVIRONMENTAL CONSTRAINTS

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FIGURE 10

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4.0 Description of Existing Conditions – Water Facilities

4.1 Water Supply, Treatment and Pump Houses

Madoc's drinking water system is owned by the Municipality and operated by Ontario Clean Water Association (OCWA) under Drinking Water Works Permit (DWWP) No. 153-201, Issue #4 and Municipal Drinking Water License (MDWL) 153-101, Issue #5. The system is supplied by two groundwater wells, Well #3 and Well #4. Both wells are categorized as groundwater under the direct influence of surface water (GUDI).

Table 10 summarizes the rated capacity of the Water Treatment Plant (WTP) as indicated in the DWWP, MDWL, and Permit to Take Water (PTTW). Well #3 is located at 109 Rollins Street. The permitted daily water taking limit at Well #3 is lower than the pump capacity. Therefore, the PTTW and DWWP governs the rated capacity at Well #3. Well #3 is treated using a cartridge filtration system, followed by ultraviolet (UV) and chlorine (sodium hypochlorite) addition for disinfection.

Well #4 is accessible via Marmora Street. The Marmora Street Well was commissioned as a back-up well in 2020. A rated capacity of 1,470 m^3/day was used for Well #4 as the daily water taking limit is equal to the pump capacity. Treatment at Well #4 consists of an arsenic removal system using Adsorbia media, a cartridge filtration system, UV, and chlorine disinfection.

There is currently no on-site backup power generator at the wells. Connections to a portable backup power generator are available.

Table 10 summarizes the allowed water taking quantity under the PTTW. The PTTW allows a combined total daily water taking of 2,620 m^3/day for Well #3 and Well #4. However, it should be noted that Well #4 is currently designated as the standby well. It has been assumed that the administrative changes can be made to the PTTW to remove the standby well designation for Well #4 and that the total combined water taking limit of 2,620 m^3/d remains.

Table 10: Water Supply Capacity

	Well Type	Daily Water Taking Limit (m^3/day)
Well #3 - Rollins Street	Duty	1,150
Well #4 - Marmora Street	Stand-by	1,470
TOTAL Water Supply Capacity		**2,620**

Table 11 summarizes the WTP treated water pumphouse capacity for each well as listed in the MDWL. The pumphouses and treatment for Well #3 and Well #4 are designed to operate in rotation.

Table 11: Water Treatment Plant Pump Capacity

	Well Pump Capacity (m^3/day)
Well #3 - Rollins Street	1,470
Well #4 - Marmora Street	1,470
TOTAL Well Pump Capacity	**2,940**

4.1.1 Historical Flow Rates

Annual Drinking Water System Reports over five (5) years, from 2017 to 2021, was provided by the Municipality. The reports were used to determine the current water demands for the water distribution system. Table 12 summarizes the average day, maximum day, and peak hour demands for Madoc.

Table 12: Madoc Water Demands (2017-2021)

Years	Average Day (m^3/day)	Maximum Day (m^3/day)	Peak Hour (m^3/day)
2017	437	791	Not available
2018	448	906	Not available
2019	440	909	Not available
2020	487	862	Not available
2021	423	922	Not available
5-Year Demand (m^3/day)	443	922	1,383
5-Year Demand (L/s)	5.2	10.7	16.0
Rated Capacity(2)	Not applicable	2,620	Not applicable
Percent (%) of Rated Capacity (2)	Not applicable	35%	Not applicable

(1) Peak hour demand calculated using a theoretical peaking factor of 1.5 times the maximum day demand, MECP Design Guidelines for Drinking Water Systems (2008).
(2) Based on PTTW maximum allowable capacity from Table 10.

The 5-year average day demand was taken as the average treated water flow reported every day between 2017 and 2021, which was calculated to be 443 m^3/day (5.2 L/s). The maximum day demand, 922 m^3/day (10.7 L/s), taken from the maximum flow reported from 2017 to 2021. As the peak hourly data was not specifically recorded, the peak hour demand was estimated using a theoretical peaking factor of 1.5 times the maximum day demand, as recommended in Ministry of the Environment, Conservation, and Parks (MECP) Design Guidelines for Drinking Water Systems (2008) for a community of this size, which resulted in a peak hour demand of 1,383 m^3/d (16.0 L/s). 35% of the total WTP rated capacity is utilized under existing maximum day demand conditions.

4.1.2 Water Quality

A review of Madoc's Annual Drinking Water System Reports over five (5) years, from 2017 to 2021, was completed. Notable adverse water quality events include exceedance of total coliforms (reported once in 2018) and exceedance of sodium (reported once in 2018) at Well #3. With exception to these water quality events, it was reported that the system complies with all other regulations for microbiological, chlorine residual, organic and inorganic parameter concentrations in the distribution system.

4.1.3 Future Water Demands

The design parameters used to calculate the future water demands of the water distribution system are summarized in Table 13.

Table 13: Design Parameters – Future Water Demand

Future Water Flow Projection – Design Parameters		
Parameter	**Residential**	**Industrial / Commercial / Institutional (ICI)**
Average Day Flow	300 L/cap/day(3)	35,000 L/ha/day (Light Industrial)(1) 45,000 L/ha/day (Typical Industrial)(1) 28,000 L/ha/day (Commercial)(1) 1,400 L/bed/day (Long Term Care / Hospital)(1)
Maximum Day Flow (2)	2.08 x Average Day	2.08 x Average Day
Peak Hour Flow (1)	1.5 x Maximum Day	1.5 x Maximum Day

(1) MECP Design Guidelines for Drinking Water Systems Table 3-1.
(2) Peak factor determined from average and maximum day demand data provided in Table 12.
(3) Residential average day flow determined from 5-year average day demand (Table 12) divided by the total service population.

The design parameters, presented in Table 13, were used to calculate future water demands presented in Table 14. The rated capacity of the WTP can accommodate maximum day water demand for existing, short-term, and mid-term. WTP rated capacity will be exceeded in the long-term.

Table 14: Future Water Demands

Demand Scenario	Existing Conditions (2023)	Short-Term (2024-2029)	Mid-Term (2029-2034)	Long-Term (2034-2044)	Build-Out (2044-2054)
Population Growth		400	891	1,559	3,353
Total Serviced Population (1)	1,489	1,889	2,780	4,339	7,692
Hospital Development (beds)		128			
ICI Development Area (ha)			3.8	10	3
Average Day (m^3/day) - Residential		120	267	468	1,006
Average Day (m^3/day) - ICI		179	108	328	70
Average Day (m^3/day) Non-Cumulative	443	299	375	795	1,076
Average Day (m^3/day) Cumulative		742	1,117	1,913	2,989
Maximum Day (m^3/day) Non-Cumulative	922	622	780	1,654	2,238
Maximum Day (m^3/day) Cumulative		1,544	2,324	3,979	6,217
Peak Hour (m^3/day)	1,383	2,316	3,485	5,967	9,325
Rated Capacity (m^3/day)(2)	2,620				

(1) The total serviced population represents residential population only and excludes equivalent institutional households and populations.
(2) WTP Rated Capacity presented in Table 10.

4.1.4 Projected Timing for WTP Expansion

Figure 11 represents the projected maximum day water demand from WTP and anticipated timing to reach 80%, 90% and 100% of the WTP rated capacity. 80% WTP rated capacity will be reached in 2032, 90% WTP rated capacity will be reached in 2034, and 100% WTP rated capacity will be reached in 2036.

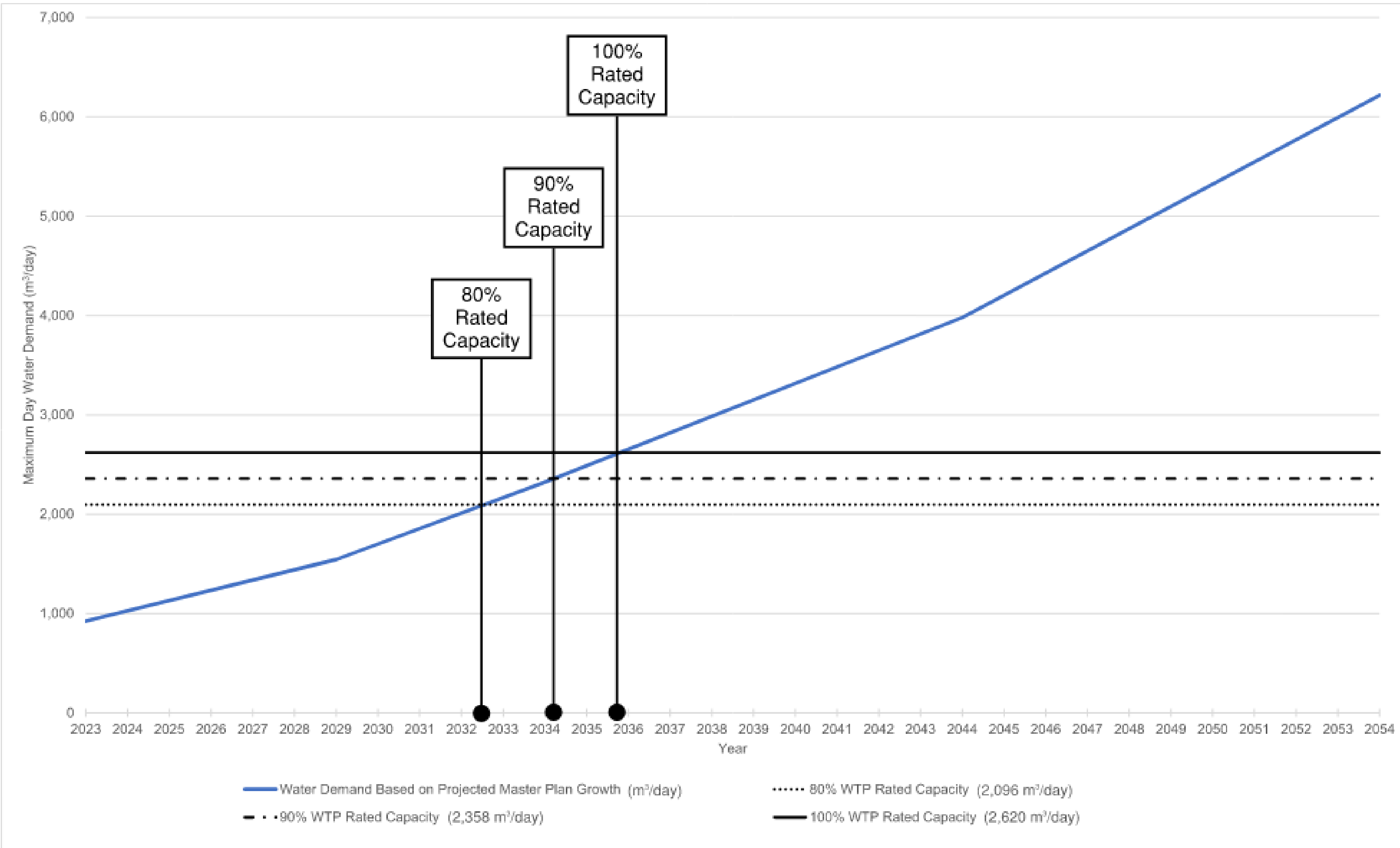


Figure 11: Projected Timing for WTP Expansion

4.2 Water Storage (Elevated Water Tank)

The Madoc elevated water tower is located at 119 McKenzie Street, Madoc and was originally constructed in 1981. The following table summarizes the key parameters of the elevated water tower.

Table 15: Madoc Water Tower Parameters

Parameter	Value
Physical Characteristics of the Water Tower	
Internal Tank Diameter	11.6 m (1)
Total Tank Height	12.85 m (1)
Operating Characteristics of the Water Tower	
Operating Level – High	219.86 m (1)(2)
Operating Level – Low	218.76 m (1)(2)
Top Water Level (Max)	220.83 m (1)
Low Water Level (Min)	208.66 m (1)
Existing Available Storage	1,250 m^3 (1)

(1) Obtained from Elevated Water Tank As-Built Drawings (1981).
(2) Operating level calculated from OCWA's Start and Stop setpoints of 83% to 92%.

Per MECP Design Guidelines for Drinking-Water Systems (2008), total available treated water storage within the system should at least amount to the sum of the required equalization storage (B), fire storage (A), and emergency storage (C) allowances, as depicted in Figure 12. The total water storage requirement was compared against the existing available storage in Table 15. The WaterCAD® model confirmed that a 200.4 m hydraulic grade line (HGL) at the water tank will yield a minimum system pressure of 140 kPa (20psi) under maximum day demand. The required HGL was lower than the low-water level provided, therefore 100% of the tank volume is available for treated water storage.

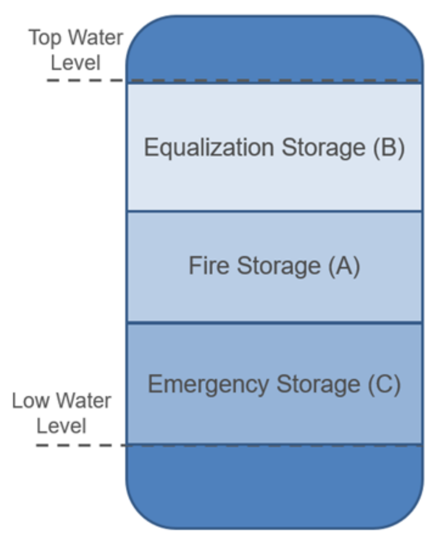


Figure 12: Total Required Treated Water Storage

Based on these guidelines, Table 16 provides a summary of the estimated existing, short, mid, and long-term and build-out total storage requirements for Madoc. Note that the equivalent population in Table 7 is not equal to the service population as used in previous sections of this report. The service population is the number of residents living in Madoc, obtained from the 2021 Census. The equivalent population considers contributions from residential and ICI water demand and was calculated using the following equation:

$$\text{Equivalent Population} = \frac{\text{Average Day Demand}}{\text{Average Per Capita Water Consumption}}$$

Where,

Average Day Demand is in m^3/day and presented in Table 14

Average Per Capita Water Consumption is in m^3/cap/day and was calculated using the following equation:

$$\begin{aligned}\text{Average Per Capita Water Consumption} &= \frac{\text{Existing Average Day Demand (from Table 13)}}{\text{Existing Service Population (from Table 13)}} \\ &= \frac{443\,\frac{\text{m}^3}{\text{day}}}{1{,}489\text{ Population}} \\ &= 0.298 \\ &\cong 0.30\,\text{m}^3/\text{cap}/\text{day}\end{aligned}$$

Based on the available information, the existing treated water storage volume is sufficient for the existing demand. It is anticipated that the storage capacity will be insufficient for water demand in the short-term. Additional modelling will be completed in Phase 2 of the Master Plan to incorporate future development growth and investigate storage pressure constraints.

Table 16: Future Water Storage Requirements

Parameter	Existing (2023)	Short-Term (2024-2029)	Mid-Term (2029-2034)	Long-Term (2034-2044)	Build-out (2044-2054)
Non-Cumulative Equivalent Population (1)	1,477	997	1,250	2,651	3,587
Cumulative Equivalent Population (1)	1,477	2,474	3,724	6,375	9,962
Fire Flow (2) (L/s)	78	102	120	162	189
Duration (2) (Hours)	2	2	2	3	3
A – Fire Storage (3) (m^3)	564	735	862	1,748	2,037
B – Equalization Storage (4) (m^3)	231	386	581	995	1,554
C – Emergency Storage (5) (m^3)	199	280	361	686	898
Total Storage Requirement (m^3)	993	1,401	1,804	3,428	4,489
Existing Available Storage (m^3)	1,250	1,250	1,250	1,250	1,250
Surplus (m^3)	257	-151	-554	-2,178	-3,239

(1) Estimated to be equal to average day demand / per capita usage of 300 L/cap/d. The equivalent population also includes ICI flow contribution.
(2) Values interpolated from Table 8-1 of the MECP Design Guidelines (2008) based on equivalent service population. Fire flow is described as the largest expected fire flow requirement in L/s and duration is length of time fire flow shall be sustained.
(3) Largest expected fire volume = fire flow x duration.
(4) 25% of Maximum Day Demand.
(5) 25% of the sum of A and B.

4.3 Level of Service

Based on available information, the Madoc Water Treatment Plant and Water Tower have sufficient capacity to service existing water demand. Water quality generally complies with regulations for residual, organic, and inorganic concentration parameters.

It is anticipated that the WTP will reach 80% capacity in 2032, during the mid-term demand scenario. Exceedance of the water tower capacity is more imminent with insufficient storage requirements for the short-term planning period. Phase 2 of the Master Plan will identify and evaluate alternatives for the WTP and water tower to accommodate water demand for long-term and short-term growth, respectively.

5.0 Description of Existing Conditions – Wastewater Facilities

5.1 Sewage Pumping Stations (SPS)

There are three (3) SPSs in Madoc, owned by the Municipality and operated by OCWA. Table 17 summarizes information on each SPS.

Table 17: Sewage Pumping Station Inventory

Pumping Station	Pump	Rated Capacity (1) (L/s)	Address	Construction Year (Major Upgrades)	Operated By
Highway 7 SPS (2)	1 (Duty)	10.2	East of 105953 Highway 7 Madoc, ON	2002	OCWA
	2 (Stand-by)	*10.2*			
McDonald's SPS	1 (Duty)	7.5	14118 ON-62 Madoc, ON	2012	OCWA
	2 (Stand-by)	*7.5*			
Rollins St. SPS	1 (Duty)	13	North of 88 Rollins Street Madoc, ON	N/A(3)	OCWA
	2 (Stand-by)	*13*			

(1) ECA No. 7572-BQXR8E and ECA No. 5744-BF4RBB.
(2) Per OPP Detachment Building As-Builts.
(3) Original construction year unknown.

5.2 Wastewater Treatment Lagoon

5.2.1 Historical Flow Rates and Storage

Madoc's Sewage Treatment System (STS) consists of a two-celled facultative wastewater treatment lagoon (the 'Lagoon'), operating in series, with a total volume of 184,000 m^3 and a rated average daily capacity of 1,008 m^3/day. Under ECA No. 1652-BRKT58, the lagoon is discharged seasonally for a minimum of 21 and maximum of 45 days in the spring, between April 1st and May 20th, and in the fall, between November 1st and December 15th. Under the ECA, spring discharge should occur when the liquid surface of the lagoon is substantially free of ice cover and should coincide with spring freshet and elevated flows in Deer Creek. It was noted by OCWA staff that there have been no recent overflow or early discharge events. There was one occurrence of discharge under ice cover in 2021. The north cell of the Lagoon was recently dredged in 2018 to remove the sludge. It is documented that the Lagoon has an average operating depth between 2 to 2.3 metres (6.8 to 7.8 ft).

The Lagoon is permitted to receive hauled waste from local septage haulers. The Municipality currently is in an agreement with a local hauler to receive up to 90,920 L of septage per year. The Municipality has indicated that this hauler has deposited septage into the Lagoon only a handful of times in the past 15 years. As such, septage is not considered a significant load to the Lagoon.

The annual wastewater flows into the Lagoon from 2018 to 2022 was obtained from Monthly Quality Reports provided by OCWA and summarized in Table 18. The average day flow of

734 m^3/day was calculated as the average annual daily flows from 2018 to 2022. Note that the Maximum Day column in Table 18 is the maximum wastewater flow for the reporting year and generally coincided with the spring freshet. The maximum day flow of 2,999 m^3/day was calculated as the maximum flow reported from 2018 to 2022.

Table 18: Madoc STS Raw Wastewater Flows (2018-2022)

Years	Average Day (m^3/day)	Maximum Day (m^3/day)
2018	873	2,999
2019	704	2,488
2020	748	2,818
2021	686	1,834
2022	659	2,873
5-Year Flow (m^3/d)	734	2,999
5-Year Flow (L/s)	8.5	34.7
ECA Rated Capacity	1,008	Not Applicable
Percent (%) of Operating Capacity Used	73%	Not Applicable

5.2.2 Influent and Effluent Wastewater Quality

Monthly Quality Reports from 2018 to 2022 were reviewed to summarize the average influent and effluent wastewater quality parameters, as shown in Table 19 and Table 20.

Table 19: Average Influent Wastewater Quality

Parameter	2018	2019	2020	2021	2022	Average
$CBOD_5$ (mg/L)	183	210	187	222	201	201
Total Suspended Solids (mg/L)	193	204	208	265	225	219
Total Phosphorous (mg/L)	3.7	4.3	4.0	5.1	4.9	4.4
Total Kjeldahl Nitrogen (mg/L)	N/A	39	47	52	48	46

There were no ECA Compliance Limit exceedance events between 2018 and 2022. The treated effluent quality also meets the design objectives. The lagoon treatment performance findings presented herein are consistent with the Madoc Sewage Lagoon Capacity Re-Rating Study by OCWA in 2021. In summary, the Lagoon has historically provided treatment levels above and beyond the required level of treatment by the ECA and are considered to have been providing secondary level of treatment to the sewage and hauled sewage.

Table 20: Effluent Wastewater Quality

Parameter	Discharge Window(2)	ECA Compliance Limit (1)	2018	2019	2020	2021	2022
$CBOD_5$ (mg/L)	Fall	30	6.6	11.0	8.0	10.9	2.9
	Spring	30	14.5	10.0	8.7	7.8	6.6
Total Suspended Solids (mg/L)	Fall	30	6.0	4.6	11.6	9.9	6.9
	Spring	30	7.9	11.6	8.6	8.9	10.0
Total Phosphorus (mg/L)	Fall	0.5	0.06	0.03	0.09	0.22	0.03
	Spring	0.5	0.09	0.12	0.05	0.08	0.07
pH	Fall	6.0 to 9.5	7.4 to 7.8	7.7 to 8.6	7.3 to 8.5	7.6 to 7.8	8.0 to 8.4
	Spring	6.0 to 9.5	7.1 to 7.5	7.6 to 7.8	7.5 to 8.1	7.5 to 8.1	7.5 to 8.1
Total Ammonia Nitrogen (mg/L)	Fall	No compliance limit	0.2	5.3	11.8	9.2	1.7
	Spring		0.2	10.9	9.2	15.1	9.5

(1) Per ECA No. 1652-BKRT58
(2) Spring season allows discharges from April 1st to May 20th and must be substantially free of ice cover. The Fall season allows discharges from November 1st to December 15th. Under the ECA, the lagoon must be discharged for a minimum of 21 days and maximum of 45 days.

5.2.3 Future Wastewater Flow

The design parameters used to calculate the future wastewater flows for the wastewater collection system are summarized in Table 21.

Table 21: Design Parameters – Future Wastewater Demand

Future Wastewater Flow Projection – Design Parameters		
Parameter	**Residential**	**Industrial / Commercial / Institutional (ICI)**
Average Day Flow	350 L/cap/day (3)	35,000 L/ha/day (Light Industrial) (1) 45,000 L/ha/day (Typical Industrial) (1) 28,000 L/ha/day (Commercial) (1) 1,400 L/bed/day (Long Term Care / Hospital) (1)
Maximum Day Flow (2)	2.08 x Average Day	2.08 x Average Day

(1) The design parameters presented in Table 12 were used to calculate future wastewater flows presented in MECP Design Guidelines for Sewage Works Table 5-3.
(2) Peak factor determined from average and maximum day demand data provided in the water demand section. For newer development, the sewer system will be new and less influenced by inflow and infiltration. As such it becomes too conservative to use a max day peaking factor of more than 4 times (according to historic flows).
(3) Residential average day flow for future is based on typical residential flow for a similar system.

Based on conversation with the Municipality, it is assumed that additional septage due to growth in the surrounding areas, hauled from outside the study area, will not be significant as discussed in Section 5.2.1.

The STS has sufficient rated capacity to meet existing average day wastewater flows. The STS rated capacity will be exceeded in the short-term demand scenario.

Table 22: Future Wastewater Flow

Demand Scenario	Existing Conditions (2023)	Short-Term (2024-2029)	Mid-Term (2029-2034)	Long-Term (2034-2044)	Build-Out (2044-2054)
Population Growth		400	891	1,559	3,353
Total Serviced Population (1)	1,489	1,889	2,780	4,339	7,692
Hospital Development (beds)		128			
ICI Development Area (ha)			3.8	10.3	2.5
Average Day (m^3/day) - Residential		140	312	546	1,174
Average Day (m^3/day) - ICI		179	108	328	70
Average Day (m^3/day) Non-Cumulative	734	319	419	873	1,244
Average Day (m^3/day) Cumulative	734	1,053	1,473	2,346	3,590
Rated Capacity (m^3/day)	1,008				

(1) The total serviced population represents residential population only and excludes equivalent institutional households and populations.

5.2.4 Projected Timing for Expansion

Figure 13 represents the projected average day wastewater flows and the anticipated timing to reach 80%, 90%, and 100% of the lagoon rated capacity. The graph indicates that 80% rated capacity will be reached in 2024, 90% rated capacity will be reached in 2026, and 100% rated capacity will be reached in 2028.

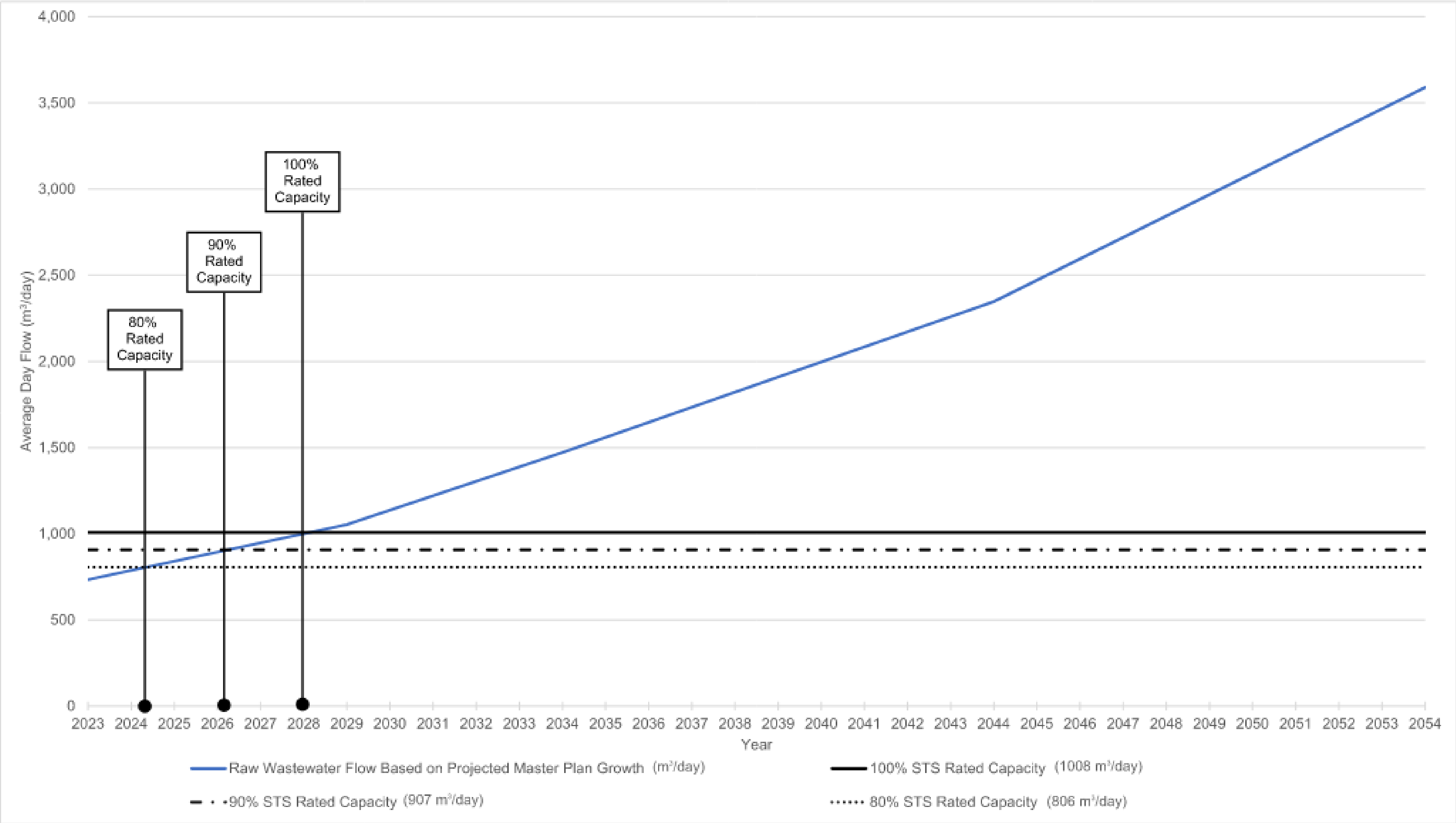


Figure 13: Projected Timing for STS Expansion

5.2.5 Level of Service

The Madoc STS treatment quality is meeting effluent quality design objectives. Further, the STS has historically provided treatment levels above and beyond the required level of treatment as specified in the ECA.

The Lagoon rated capacity is able to accommodate existing average day wastewater flows for existing conditions. The Lagoon rated capacity will be exceeded in the next 0-5 Years. Phase 2 of the Master Plan will identify and evaluate alternatives to accommodate future wastewater flows for the next 20 Years and beyond.

6.0 Existing Level of Service Conditions and Linear Infrastructure Model Updates

6.1 Water Distribution Model

The purpose of the following Water section of the Master Plan is to confirm long-term security of supply, ensure adequate distribution to existing developments, identify residual capacity in the current system, and identify areas of deficiencies. The methodology associated with this study comprises of developing a new water model based on the existing GIS data and record drawings provided by the Municipality.

Refer to Appendix D for the complete water modelling memorandum.

6.1.1 Watermain Distribution Network

Madoc's hydraulic water model was built using Bentley's WaterCAD® software platform. The scaled water distribution network was imported from GIS data consisting of pipes, junctions, and hydrants. The information within the GIS data included pipe diameters, materials, and lengths. In accordance with the Ministry of the Environment, Conservation and Parks (MECP) design guidelines, the actual inside pipe diameters were modelled as follows:

Table 23: Pipe Diameters

Nominal Diameter (mm)	Inside Diameter (mm) (PVC, Ductile Iron)
50	50
100	108
150	155
200	204
250	250
300	297

Roughness coefficients or Hazen-Williams C-Factors were developed based on past experience and from the work done by Peter A. Lamont, entitled "Common pipe flow formulas compared with the theory of roughness" published in the American Water Works Association (AWWA) Journal in May 1981. Based on available information, consideration was given to pipe material and approximate pipe age. The modelled C-Factors are presented in the table below.

Table 24: C-Factors

Material	Installation Year	C-Factor for Nominal Diameter (mm)					
		50	100	150	200	250	300
Polyvinyl Chloride (PVC)	All	100	100	100	110	110	120
Cast Iron (C/I)	All	100	105	105	106	107	107
Ductile Iron (D/I)	All	100	105	105	106	107	107
Polyethylene	All	100	100	100	110	110	120
Copper	All	100	100	100	110	110	120
High Density Polyethylene (HDPE)	All	100	100	100	110	110	120
Unknown	All	100	100	100	110	110	120

Junction and hydrant topographical elevations were obtained from LiDAR-based DEM (digital elevation model) data. Junction and hydrant locations are as shown in Figure 14 and Figure 15, respectively.

Figure 14: Madoc Water Model Schematic - Junctions

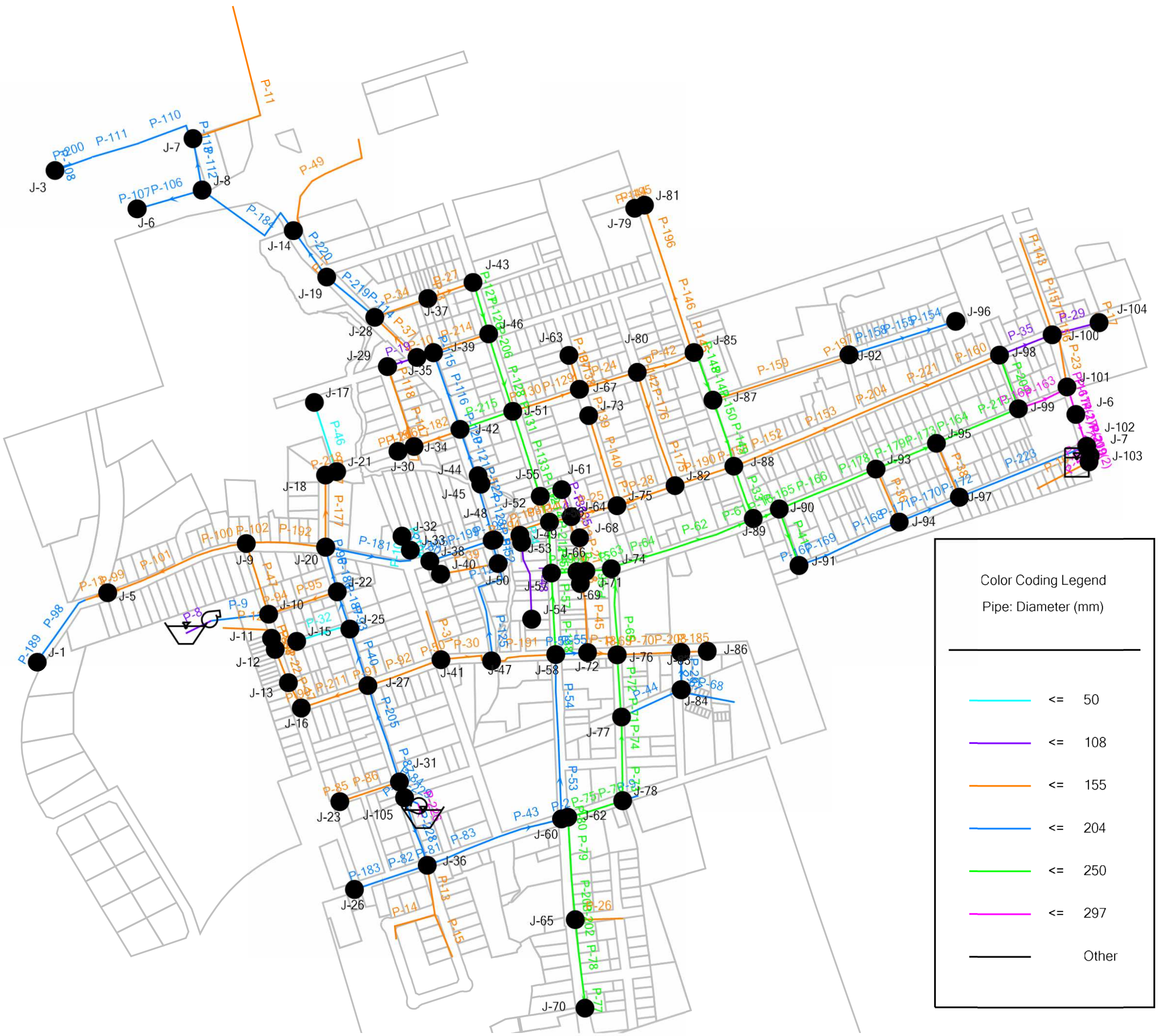
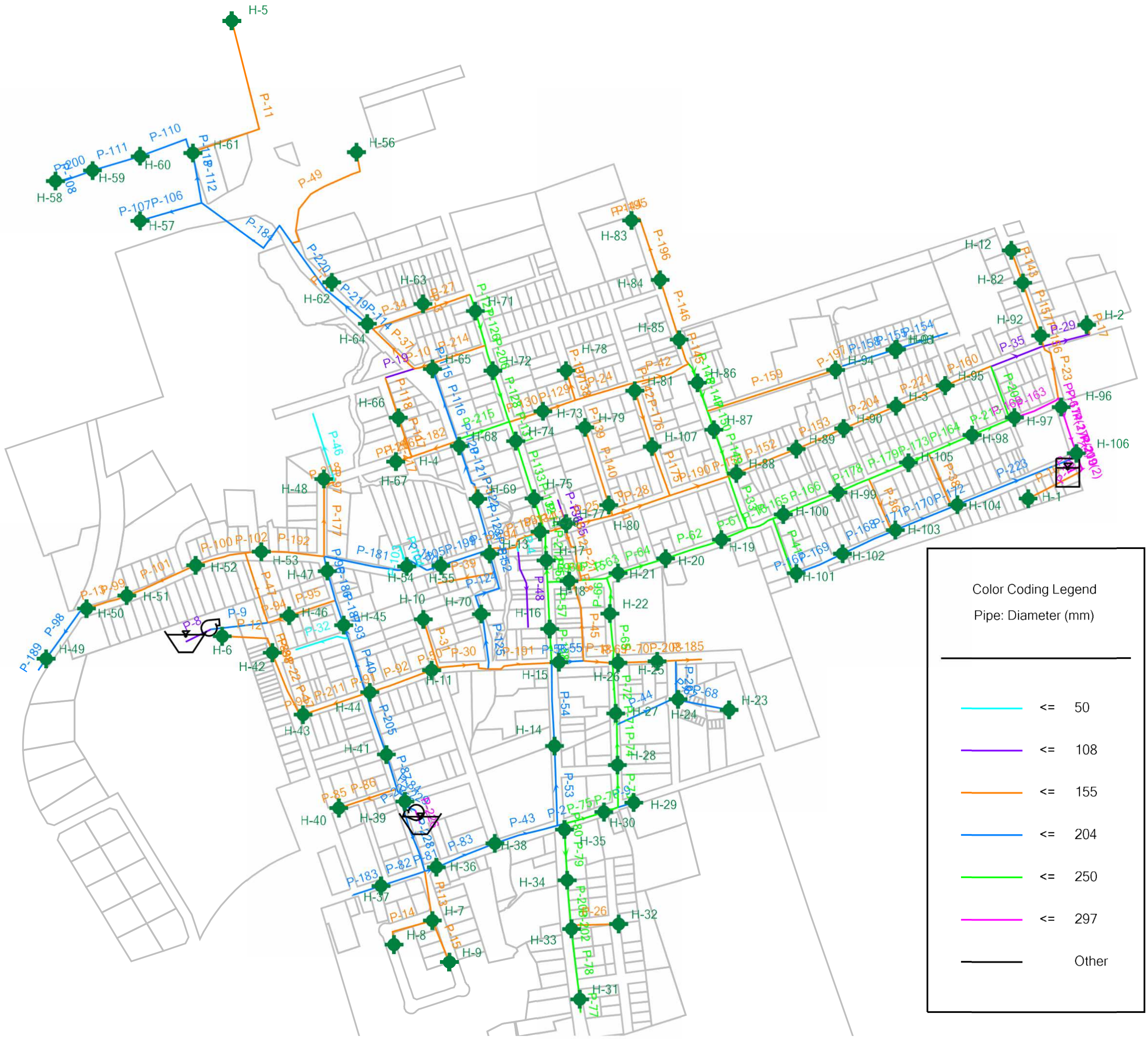


Figure 15: Madoc Water Model Schematic - Hydrants



6.1.2 Tanks, Wells and Pump Houses

LiDAR-based DEM (digital elevation model) data was not provided for the pumps and wells; therefore, they were approximated using the same elevations as the nearest junction node. The well stations and pump houses were modelled based on information shown on the following drawings:

- Madoc 2 Rollins St Plan and Profile, Drawing No. 2 dated June 1987 prepared by Totten Sims Hubicki Associates
- Madoc Well 4 IFC Mech. Set, Drawing SP1, Rev. 3 dated April 12, 2018, prepared by Greer Galloway Consulting Engineers

Pump curves were obtained from the pump manufacturer's (GrundFos) online website given the make and model of the pumps. The same pump curve was applied to both pumps in the system as the Municipality's background information specified the same make and model for both pumps. Please refer to Appendix D, Attachment 1.1 for the pump curve.

Watermain lengths were scaled in the model, however the following watermains were manually input based on record drawings mentioned above as no GIS data was provided:

- Watermain length and diameter between pump at Well #4 and connection to system.
- Watermain length and diameter between pump at Well #3 and connection to system.

The water tower elevations were provided from a drawing received from the Municipality (refer to Appendix D, Attachment 2 for water tower elevations). The normal operating level was calculated from OCWA's Start and Stop setpoints of 83% to 92%. The low water level (LWL) of 83% full (Hydraulic grade line, HGL= 218.76 m) is the initial water elevation for the tank used in the model. The watermain length and size between the tank and street watermain was manually input in the model based on the information provided in the drawing (refer to Appendix D, Attachment 3).

The following table summarizes the water tower operating levels input in the model:

Table 25: Water Tower Operating Levels

Description	Tower Elevation (m)
Base Elevation	181.82
Low Water Level	218.76
High Water Level	219.86
Maximum Water Level	220.83

6.1.3 Water Demands

The modelled water demands were based on Annual Reports which consisted of monthly average day and maximum day demand data over five (5) years (2017 – 2021) provided by the Municipality. Flow data from only Well #3 was considered in the years between 2017 – 2020. From 2020 to 2021, flow data from both wells, Well #3 and Well #4 were considered. The MECP peaking factor in accordance with Table 3-1 of their design guidelines was used to estimate the total peak hour demand. Based on the estimated existing service population of 1,489 people, a peaking factor of 1.5 x maximum day demand was used for the peak hour demand.

The water demands for the high-water users were deducted from the total Municipality demands and assigned to the nearest nodes (refer to Appendix D, Attachment 4). The remaining water demands were calculated by multiplying the remaining average day demand per unit with the number of units assigned to each junction.

Ten (10) high water users were also accounted for in Madoc. The Municipality provided multiple consumption readings (average day demand) for these ten users. The average of these readings for each user is listed in Appendix D, Attachment 5. The maximum day peaking factor of 2.08 was calculated by taking the ratio of the maximum day demand over the average day demand from the Annual Reports. The maximum day demand for the top ten high water users was calculated using a peaking factor of 2.08 x average day demand. The peak hour peaking factor of 1.5 was based on the MECP design guidelines (Table 3-1) for an estimated existing service population of 1,500 people. The peak hour demand for the top ten water users was calculated as 1.5 x maximum day demand. Please refer to Appendix D, Attachment 5 for a detailed list of the top ten water users and to Table 12 for Madoc's total water demand.

6.1.4 Model Scenarios and Design Criteria

The newly constructed hydraulic water model was used to simulate the performance of the current system under existing steady-state flow conditions. The following operating conditions were assumed for these simulations:

- The existing average day, maximum day plus fire flow, and peak hour scenarios assume that the pump (PMP-3 in WaterCAD) is operating at Well #3 and the other pump (PMP-4 in WaterCAD) is offline at the standby Well #4, while the water tower level is at 218.76 m (normal low operating level provided from OCWA).

Note that under the average day, maximum day and peak hour scenarios, the following MECP Design Guidelines are applicable:

- The maximum pressure at any point in the distribution system in unoccupied areas shall not exceed 689 kPa (100 psi), and in occupied areas shall not exceed 552 kPa (80 psi).
- Maximum Day: Pressure is to be within the range of 345 kPa (50 psi) and 480 kPa (70 psi).
- Maximum Day + Fire Flow: Residual pressure at any point in the distribution system shall not be less than 140 kPa (20 psi).
- Peak Hour: Pressure is to be above 275 kPa (40 psi).

A summary of the simulation results is provided in Table 26.

Table 26: Hydraulic Water Model Results – Existing Conditions

Demand Scenario	General Results	Notes
Average Day	Good. Pressure Range: 276- 569 kPa	Most junction nodes experience pressures between 350 kPa and 552 kPa.
Maximum Day + Fire Flow	Good. Fire Flow Availability: 22-500 L/s	Most hydrant nodes experience fire flows above 45 L/s, which is the minimum required fire flow per the Ontario Building Code (OBC) for a typical 2-storey home.
Peak Hour	Good. Pressure Range: 276-566 kPa	Most junction nodes experience pressures between 350 kPa and 552 kPa.

6.1.5 Hydrant Testing

The hydrant testing data provided by the Municipality (refer to Appendix D, Attachment 7) was compared to the model results for various locations across the system. Although it was found that there is a minor discrepancy in the static pressures between the hydrant testing and the water model, the results were generally found to be representative of real-world conditions. It was also found that there is a larger discrepancy between the dynamic pressures under fire flow from the hydrant testing and the water model. The findings are summarized in the table below.

Table 27: Comparison of Hydrant Testing Field Data with Water Model

Static Pressure				
Hydrant Testing ID	Hydrant Testing Pressure (psi)	WaterCAD ID	WaterCAD Pressure (psi)	Pressure Discrepency (psi)
H-96	56	H-61	64	8
H-49	61	H-84	65	4
H-52	68	H-87	72	4
H-85	77	H-36	80	3
H-38	47	H-106	50	3
H-45	59	H-105	62	3
H-21	68	H-76	73	5
H-87	77	H-30	81	4
Dynamic Pressure				
H-35	54	H-103	58	4
H-55	40	H-73	47	7
H-04	57	H-35	68	11

Overall, the model is expected to be a useful tool in assessing Madoc's water distribution system. The Municipality could consider implementing a water model calibration to ensure that the results from the field data and water model align more closely.

6.1.6 Water Model Simulation Results

The following tables summarize the model results under existing conditions for Madoc based on the percentage of junctions in the model within each stated pressure range or available fire flow range.

6.1.7 Average Day Demand

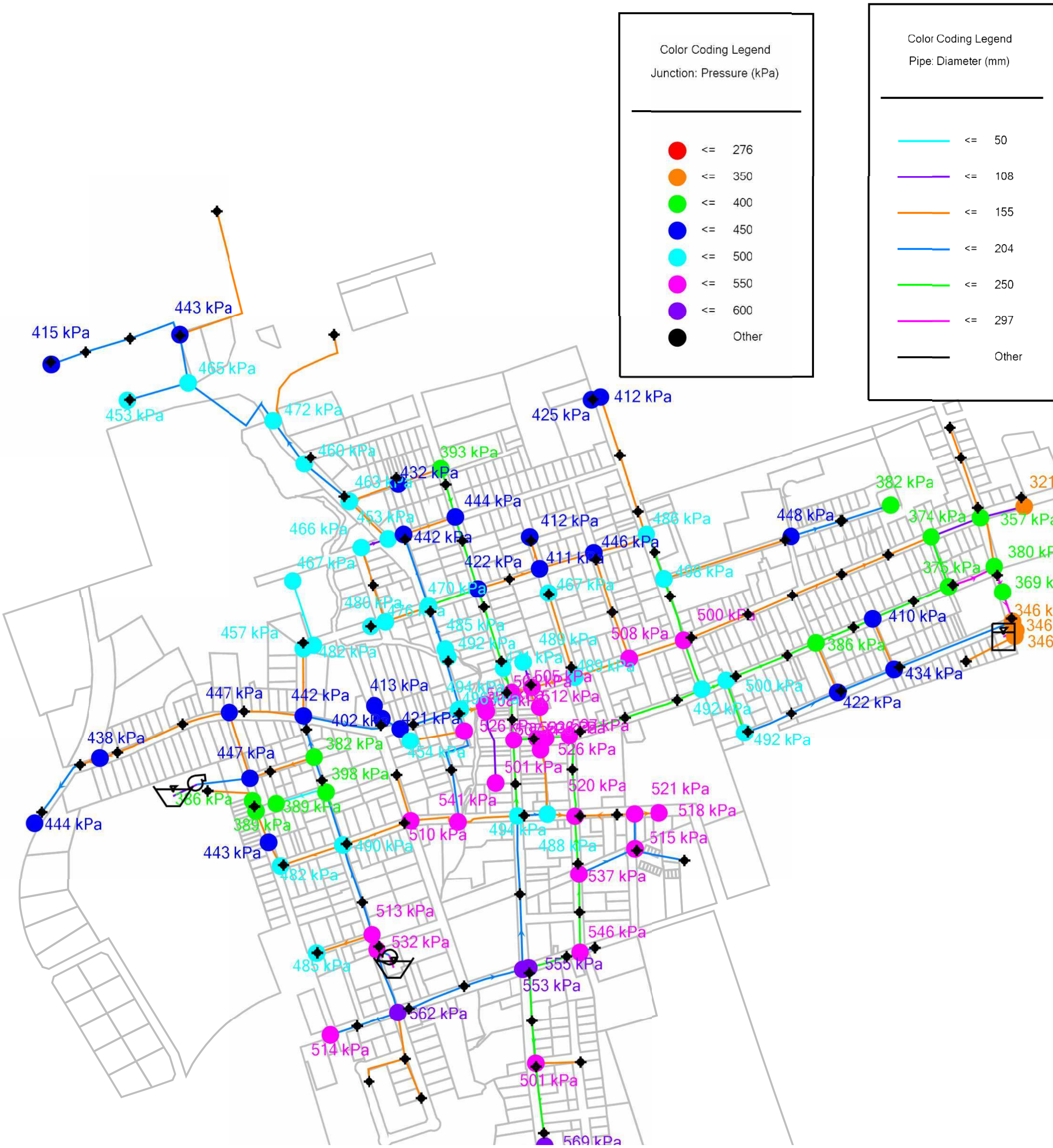
Table 28 presents the average day simulation results.

Table 28: Hydraulic Water Model Results - Average Day Demand

Average Day Demand			Percentage of Junctions
Pressure Range (kPa)			**Existing Conditions**
	Less than	276	0.0%
276	up to	350	3.9%
350	up to	400	12.6%
400	up to	450	23.3%
450	up to	500	30.1%
500	up to and incl.	552	26.2%
	Greater than	552	3.9%

Under average day demand, the table above shows that most junction nodes experience pressures between 350 kPa and 552 kPa, and a smaller percentage of the junction nodes experience pressures above 552 kPa. System pressures under existing conditions are found to be above the minimum recommended pressure of 275 kPa (40 psi), in accordance with the MECP Design Guidelines. Four (4) junction nodes located on Durham Street South (J-60, J-62, and J-70) and Seymour Street West (J-36) experience pressures above 552 kPa due to their low topographic elevations.

Figure 16: Madoc Water Model
Existing Conditions - Average Day Demand
1 Pump On, Water Tower HGL = 218.76m



6.1.8 Maximum Day Plus Fire Flow

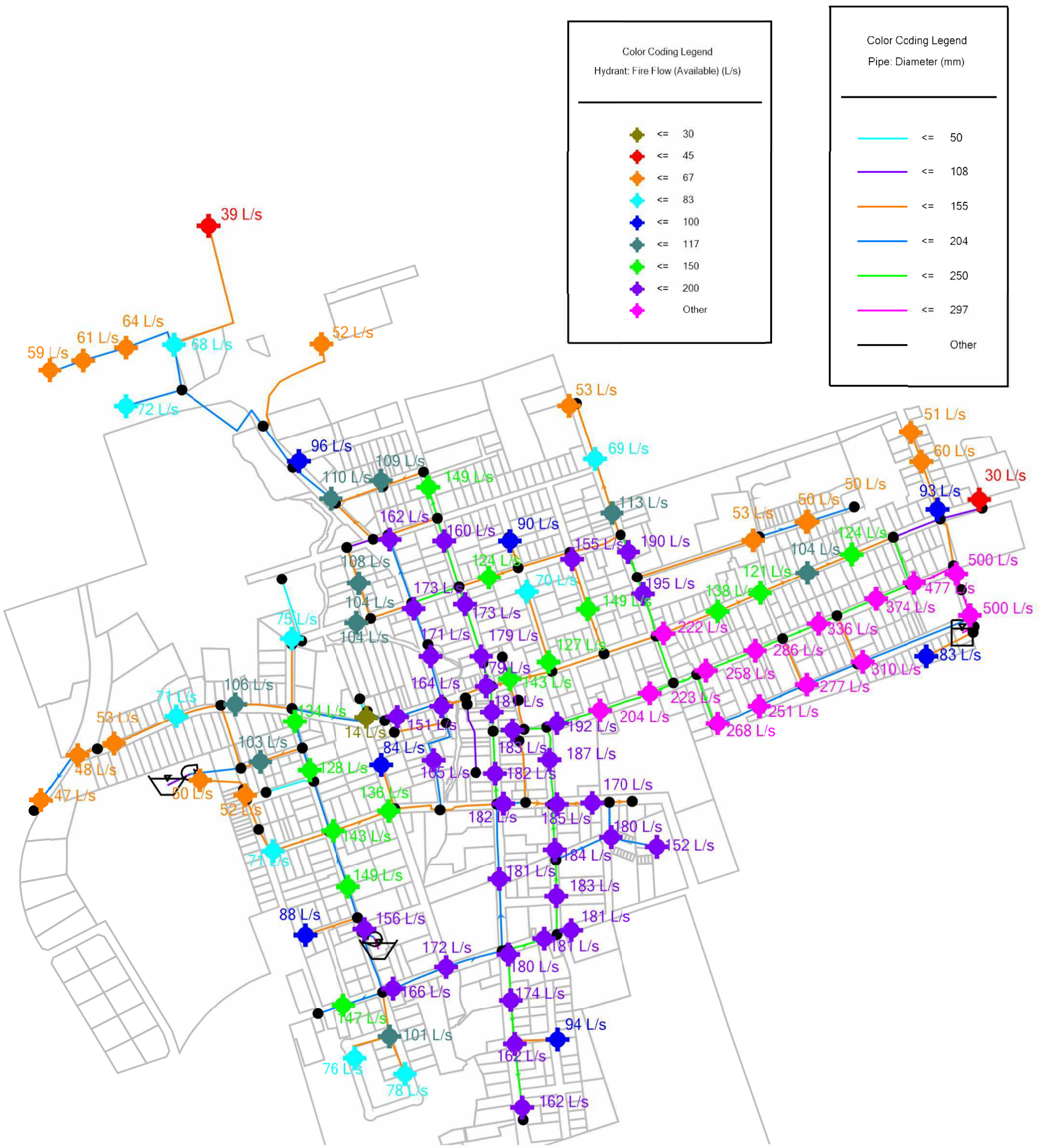
Table 29 presents the maximum day plus fire flow simulation results.

Table 29: Hydraulic Water Model Results – Maximum Day Demand + Fire Flow

Maximum Day Demand + Fire Flow			Percentage of Hydrants
Fire Flow Range (L/s)			**Existing Conditions**
	Less than	30	0.9%
30	up to	45	1.9%
45	up to	67	14.0%
67	up to	83	8.4%
83	up to	100	6.5%
100	up to	117	9.3%
117	up to	150	13.1%
150	up to and incl.	200	32.7%
	Greater than or equal to	200	13.1%

Under maximum day demand plus fire flow, the table above shows that most hydrant nodes experience fire flows above 45 L/s, which is the minimum required fire flow per the Ontario Building Code (OBC) for a typical 2-storey home. A smaller percentage of the hydrant nodes experience fire flows below the minimum OBC requirement. These hydrant nodes have low fire flow availability as they are located at dead end watermains on Russel Street (H-5), St. Lawrence Street East (H-2), and St. Peters Street North (H-54).

Figure 17: Madoc Water Model
Existing Conditions - Maximum Day Demand + Fire Flow
1 Pump On, Water Tower HGL = 218.76m



6.1.9 Peak Hour Demand

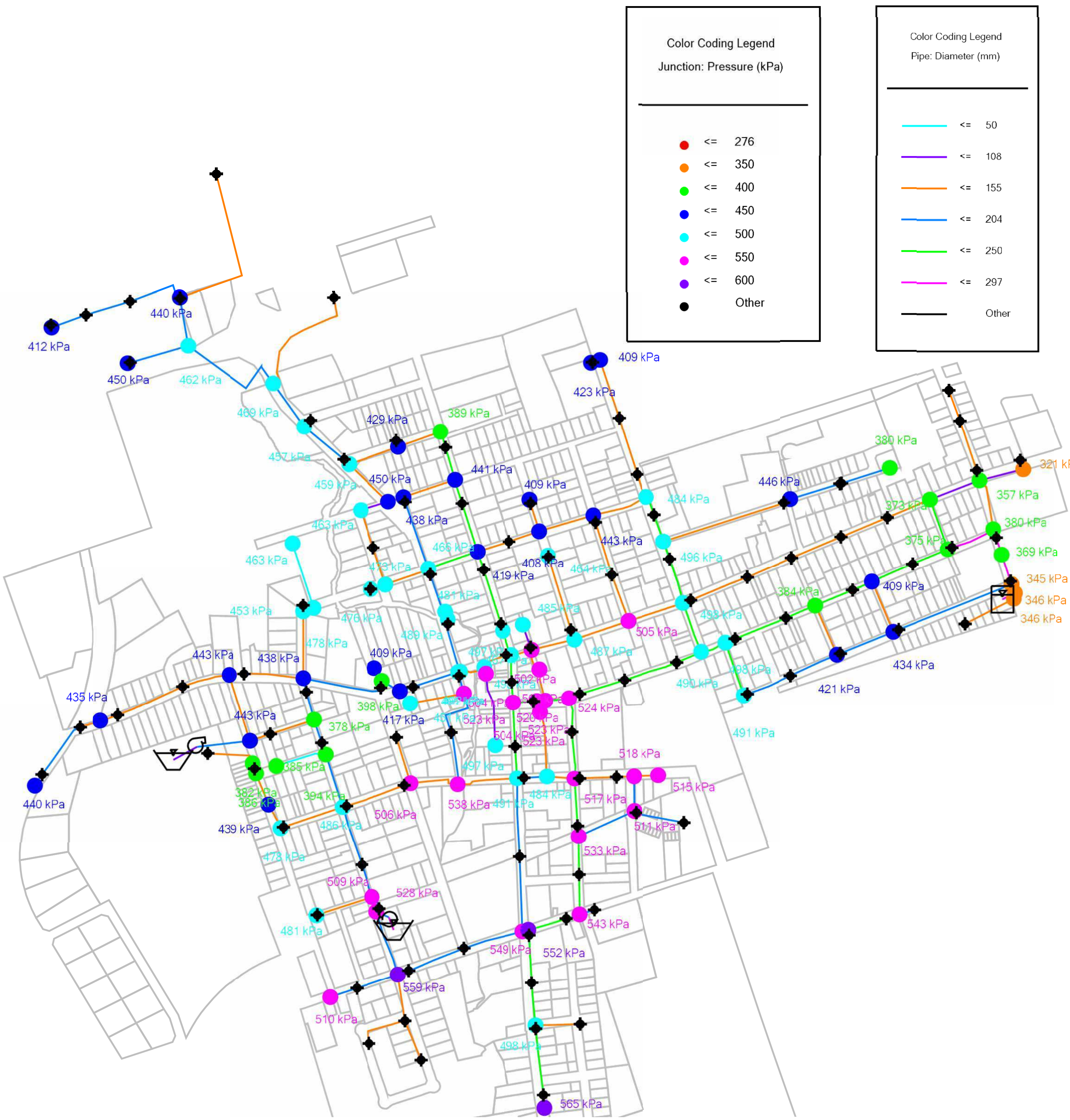
Table 30 presents the peak hour simulation results.

Table 30: Hydraulic Water Model Results – Peak Hour Demand

Peak Hour Demand			Percentage of Junctions
Pressure Range (kPa)			**Existing Conditions**
	Less than	276	0.0%
276	up to	350	3.9%
350	up to	400	13.6%
400	up to	450	22.3%
450	up to	500	35.9%
500	up to and incl.	552	21.4%
	Greater than	552	2.9%

Under peak hour demand, the table above shows that most junction nodes experience pressures between 350 kPa and 552 kPa, and a smaller percentage of the junction nodes experience pressures below 350 kPa or above 552 kPa. System pressures under existing conditions are found to be above the minimum recommended pressure of 275 kPa (40 psi), in accordance with the MECP Design Guidelines. Two (2) junction nodes located on Durham Street S (J-70) and Seymour Street W (J-36) experience pressures above 552 kPa due to their low topographic elevations.

Figure 18: Madoc Water Model
Existing Conditions - Peak Hour Demand
1 Pump On, Water Tower HGL = 218.76m



6.1.10 Level of Service

Based on the model results, the overall existing water distribution system is operating in general accordance with the pressure and flow recommendations of the current MECP Water Design Guidelines.

It is recommended that the Municipality update Madoc's water model periodically as new and better information becomes available over time regarding watermain rehabilitation or extensions and system operation. A pressure and flow monitoring field testing program in support of a model validation exercise would be beneficial in further refining the model's ability to accurately simulate real world conditions. Development of an extended period simulation (EPS) scenario within the model would also be beneficial in assessing water quality aspects of the distribution system.

6.2 Sanitary Sewer Model

6.2.1 Wastewater Collection System Design Criteria

Madoc's sanitary sewer system was analyzed by completing a pipe-by-pipe sanitary sewer design spreadsheet. Refer to Appendix E for the completed sanitary design spreadsheet. Standard design parameters selected were used to develop peak sanitary sewage flows and may or may not be reflective of the actual flow being generated within a given sewershed. The actual flow will vary with population, demographic, land use, ground conditions, groundwater table elevation, construction practices, and other factors.

6.2.2 Standard Design Parameters

The following design parameters were used to model existing sanitary network conditions.

Table 31: Sanitary Design Parameters

Sewage Generation Type	Design Parameter
Residential Average Flow:	350 L/Cap/day
Commercial Average Flow:	28,000 L/ha/day
Institutional Average Flow:	100 L/Cap/day (School)
Industrial Average Flow:	35,000 L/ha/day
Peaking Factor:	Harmon's Equation (2.0<P.F.<4.0)

6.2.3 Extraneous Flows

Sanitary sewers must be designed to convey waste discharges (the consumption flow), as well as extraneous, non-waste flow components, such as groundwater infiltration and inflow of surface runoff. Excessive extraneous flows can limit the capacity of existing sewer systems to serve expanding growth. They can also result in sewer backups, basement flooding, and increased operation and maintenance costs for pumping and treatment facilities. Conversely, successful control of extraneous flows can increase or maintain the life expectancy of the infrastructure and free available capacity for expansion and development.

The extraneous flow design allowance is added to the peak theoretical consumption flow, described earlier, to yield the total theoretical peak flow that the sewer must be designed to convey. A general allowance of 0.14 L/s/ha was used for Madoc to calculate the extraneous flow component of the total flow, irrespective of land use classifications, sewer construction, or soil type.

6.2.4 Theoretical Sewage Generation

Madoc's sanitary sewer system services existing residential, commercial, institutional and industrial developments.

1) Residential Developments

To confirm the extent of existing residential development on Madoc's sewer system zoning GIS data obtained from the County of Hastings was used in conjunction with the total population data of Madoc obtained from Statistics Canada Census Profile 2021.

An average population density of 14 people per hectare (Ppha) was calculated using the total existing residential area of 109.65 ha and total population of 1,489 as of 2021, obtained from the zoning GIS data and Statistics Canada, respectively. All the institutional, industrial, commercial and future development areas were excluded from the calculation.

The total existing residential area included right-of-way areas (ROW) and flat sewer rate properties. The Municipality provided a list of properties charged with Flat Sewer Rates. These are properties with sanitary sewers running along the property frontage but are not directly connected to sanitary services. Therefore, a population density of 13 Ppha was used for ROW and flat sewer areas in the sanitary model sheets in Appendix E.

2) Industrial, Commercial, and Institutional (ICI) Lands

The revised peak flow generation parameters specify a value of 35,000 L/ha/day to estimate flow generation rates for industrial type developments. This value is generally reserved for new or existing type developments at a Master Plan level of detail rather than specifics depending on the development type, number of employees, number of fixtures, etc. The total area of industrial lands serviced by the municipal sewer system was calculated as 0.28 ha using the zoning information in GIS. The peak flow generation value of 35,000 L/ha/day was converted into a relative population density of 100 Ppha using the residential average daily flow value of 350 L/cap/day.

The revised peak flow generation parameters specify a value of 28,000 L/ha/day to estimate flow generation rates for commercial type developments. This value is generally reserved for new or existing type developments at a Master Plan level of detail rather than specifics depending on the development type, number of employees, number of fixtures, etc. The total area of commercial lands serviced by the municipal sewer system was calculated as 21.74 ha using the zoning information in GIS. The peak flow generation value of 28,000 L/ha/day was converted into a relative population density of 80 Ppha using the residential average daily flow value of 350 L/cap/day.

The average daily flow generation parameters specify a value of 100 L/student/day to estimate flow generation rates for schools obtained by taking the average of the common sewage flow rates in Table 5-3 of the MECP Sewer Design Guideline. The total area of school land serviced by the municipal sewer system was calculated as 5.05 ha using the zoning information in GIS. The average daily flow generation of 100 L/student/day was converted into a relative population density of 46 Ppha using the residential average daily flow value of 350 L/cap/day, area of the school land, and a total number of 820 students (data obtained from Ministry of Education).

6.2.5 Total Theoretical Sewage Generation Rates

The total consumption flow was calculated using the design parameters in Table 31. Total consumption flow consists of the combined residential and commercial/institutional sewage generation rates, as summarized in the following table.

Table 32: Theoretical Existing Sewage Generation

Sewage Generation Type	Units	Average Day Flow (L/s)
Residential(1)	1,489 people	6.03
Industrial(1)	0.28 hectares	0.11
Commercial(1)	21.74 hectares	7.05
School(1)	820 students	0.95
Inflow and Infiltration(2)	143.61 hectares	20.10
Total Peak Sewage Generation		**48.56**

(1) Harmon Peaking Factor of 3.68 applied to Residential and ICI sewage generation.
(2) Inflow and infiltration multiplied by 0.14 L/s/ha to represent extraneous flows.

Below are the calculations leading to the Harmon peaking factor, using the equation found in MECP Sewer Design Guidelines:

$$P.F. = 1 + \frac{14}{4 + \left(\frac{P}{1000}\right)^{\frac{1}{2}}}$$

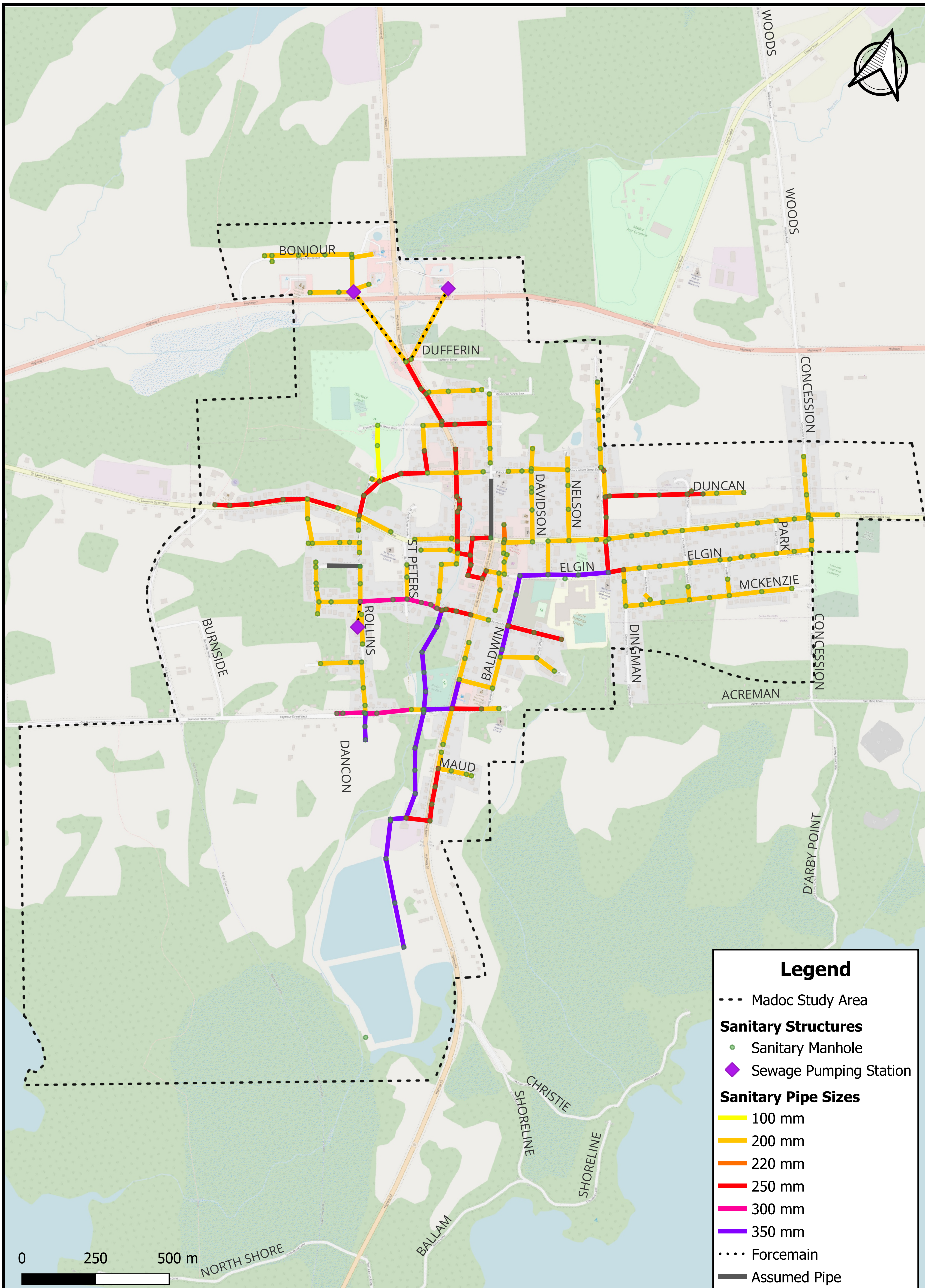
where:

- P.F.: Peaking Factor
- K.: Correction Factor = 1.0
- P: Population

6.2.6 The Sanitary Sewer Model

The Municipality provided GIS data and drawings of approximate pipe and manhole locations, material, diameter, and length of Madoc's sanitary sewer infrastructure. Data gaps were identified in the physical attributes of the system, such as invert elevation, GPS location, diameter, and slope, which are key parameters in calculating the theoretical capacity of each sewer segment. A site survey was conducted to confirm pipe direction and size and collect the manhole locations, invert elevations and top of cover elevation using GPS and laser level equipment. The survey was beneficial in identifying existing and missing assets within the provided GIS data. The sanitary sewer network is shown in Figure 19, including assumed sanitary pipes on Wishart Street and Durham Street North (Between Prince Albert Street East and St. Lawrence Street East). Based on anecdotal information, sanitary pipe was assumed to be exist on Wishart Street. Based on anecdotal information, sanitary sewer on Durham Street North, between Prince Albert Street East and St. Lawrence Street East, is suspected to be located on private property. An assumed pipe was drawn to represent the sanitary sewer from Durham Street North.

Sanitary sewersheds were delineated based on lot parcel data and proximity to sanitary manholes. Parcel data was provided by the Municipality and overlaid with the sanitary sewer system. Each sewershed area was assigned to the upstream sewer run within the area. This practice is conservative to ensure that all pipes within the drainage area are of sufficient size to collect wastewater from the area for which it is responsible. Sanitary sewersheds are shown in Figure 20 to Figure 23.



PROJECT:

MADOC WATER, WASTEWATER, AND STORMWATER MASTER PLAN

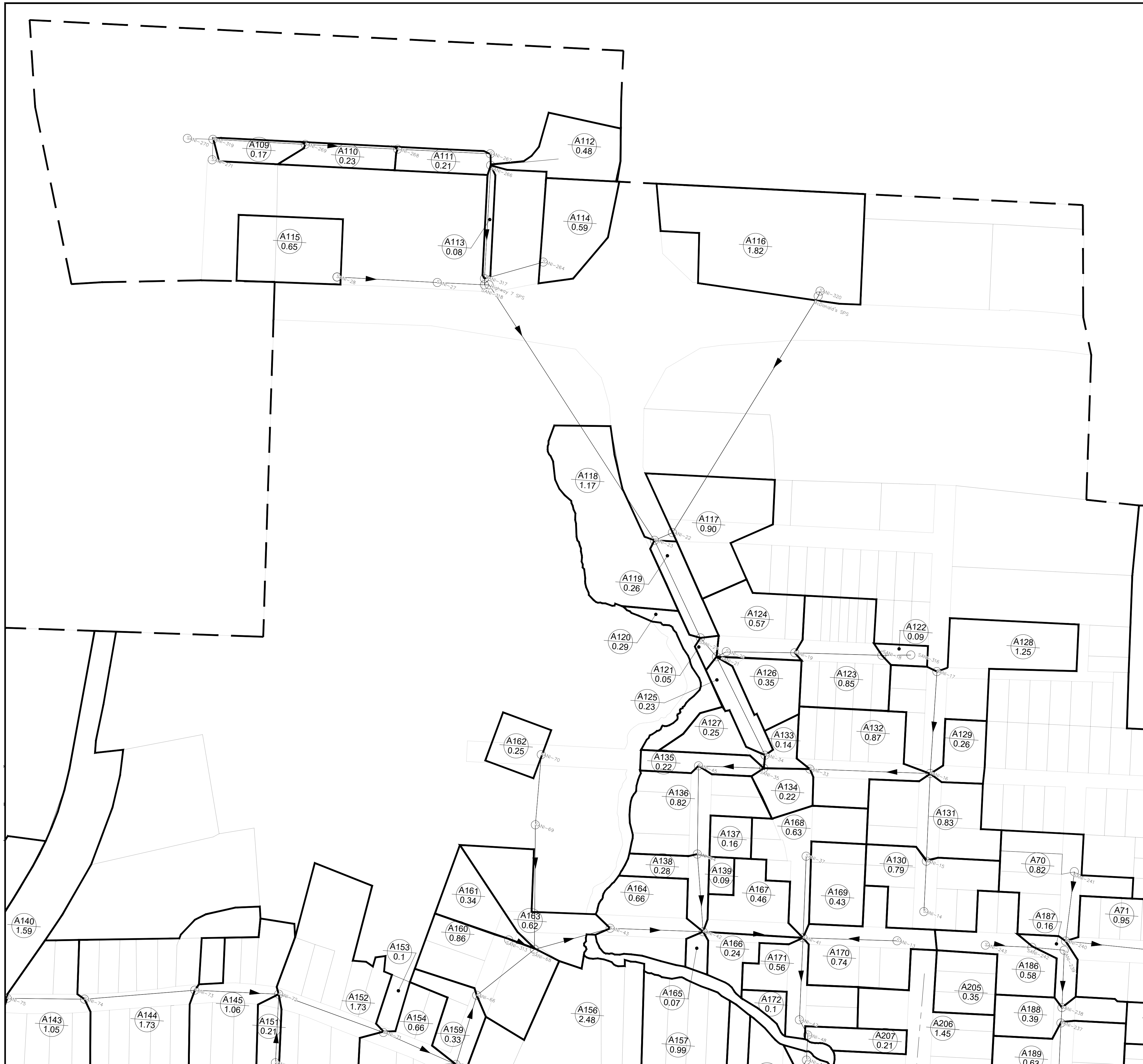
Madoc, ON

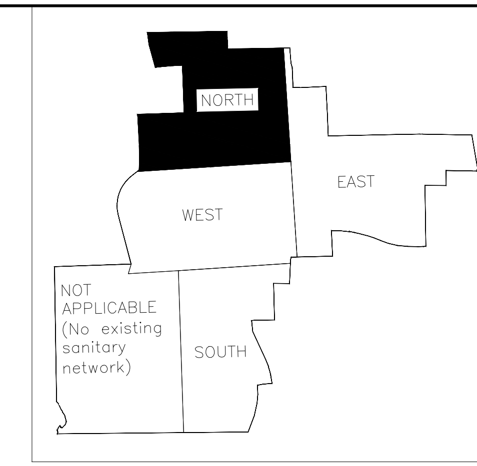
DRAWING:

Existing Sanitary Pipe Sizes

J.L.Richards
ENGINEERS · ARCHITECTS · PLANNERS

DESIGN:	PSC	JLR NO:	32508-000
DRAWN:	RC	DRAWING NO:	**Figure 19**
CHECKED:	SJS		





LEGEND

A## AREA LABEL
AREA IN HECTARES
SEWERSHED AREA
PROPERTY LINE
SANITARY FLOW DIRECTION
EXISTING SANITARY SEWER
EXISTING SANITARY SEWER (ASSUMED)
SANI-### SANITARY MANHOLE AND I.D. LABEL
STUDY AREA BOUNDARY

No.	ISSUE / REVISION	DD/MM/YY
01	Client Review	07/02/24

VERIFY SHEET SIZE AND SCALES. THE BAR TO THE RIGHT IS 25mm IF THIS IS A FULL SIZE DRAWING. 0 25mm

SCALE:

CLIENT: HASTINGS Centre Hastings COUNTY

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PROJECT: Madoc Water, Wastewater and Stormwater Master Plan

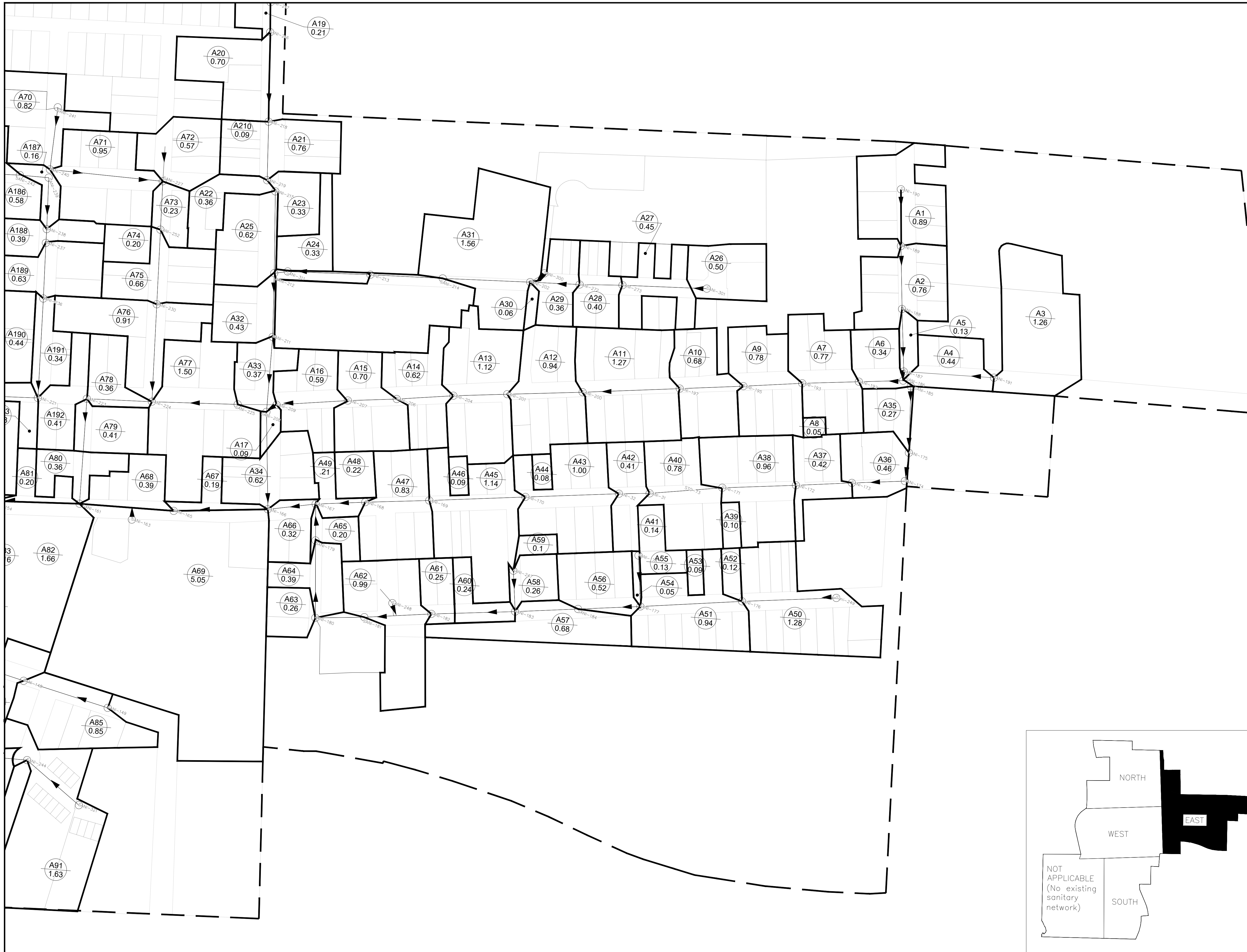
Madoc, ON

DRAWING: EXISTING SANITARY SEWER SHEDS - NORTH

DESIGN: PSC
DRAWN: RC
CHECKED: MM
JLR #: 32508-000

DRAWING #: Figure 20

PLOT DATE: February 22, 2024 2:24:22 PM



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PROJECT:

Madoc Water, Wastewater and Stormwater Master Plan

Madoc, ON

DRAWING:

EXISTING SANITARY SEWER SHEDS - EAST

DESIGN: PSC
DRAWN: RC
CHECKED: MM
JLR #: 32508-000

DRAWING #:
Figure 21

PLOT DATE: February 22, 2024 2:23:41 PM

LEGEND

A## / ####	AREA LABEL / AREA IN HECTARES
	SEWERSHED AREA
	PROPERTY LINE
	SANITARY FLOW DIRECTION
	EXISTING SANITARY SEWER
	EXISTING SANITARY SEWER (ASSUMED)
SANI-###	SANITARY MANHOLE AND I.D. LABEL
	STUDY AREA BOUNDARY

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PROJECT:

Madoc Water, Wastewater and Stormwater Master Plan

Madoc, ON

DRAWING:

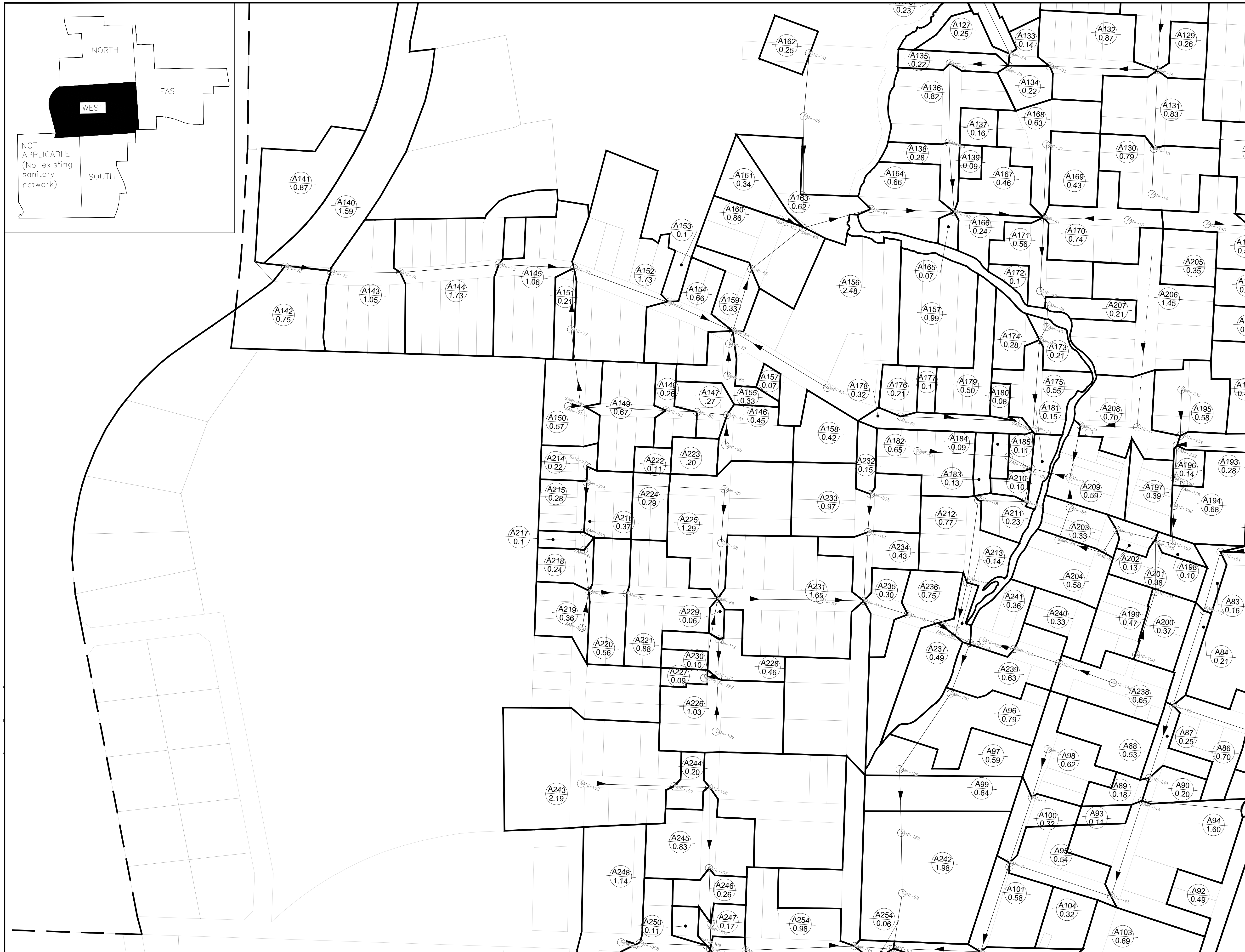
EXISTING SANITARY SEWER SHEDS - SOUTH

DESIGN: PSC
DRAWN: RC
CHECKED: MM
JLR #: 32508-000

DRAWING #:

Figure 22

PLOT DATE: February 22, 2024 2:22:52 PM



LEGEND

A## / ####	AREA LABEL / AREA IN HECTARES
	SEWERSHED AREA
	PROPERTY LINE
	SANITARY FLOW DIRECTION
	EXISTING SANITARY SEWER
	EXISTING SANITARY SEWER (ASSUMED)
SANI-###	SANITARY MANHOLE AND I.D. LABEL
	STUDY AREA BOUNDARY

No.	ISSUE / REVISION	DD/MM/YY
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Madoc Water, Wastewater and Stormwater Master Plan

Madoc, ON

DRAWING:

EXISTING SANITARY SEWER SHEDS - WEST

DESIGN: PSC
DRAWN: RC
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JLR #: 32508-000

DRAWING #: Figure 23

PLOT DATE: February 22, 2024 2:22:06 PM

6.2.7 Sanitary Sewer Model Results

Based on the sanitary sewer spreadsheet model the highest peak design flow with pump flow for the Municipality sanitary sewer system is 96.5 L/s. The spreadsheet model indicates that most sewers in the system operate under 20% capacity. The spreadsheet model indicates that 27 sewer segments have insufficient capacity to convey the peak design flow as shown in Table 33.

Table 33: Sanitary Sewers Over 100% Capacity

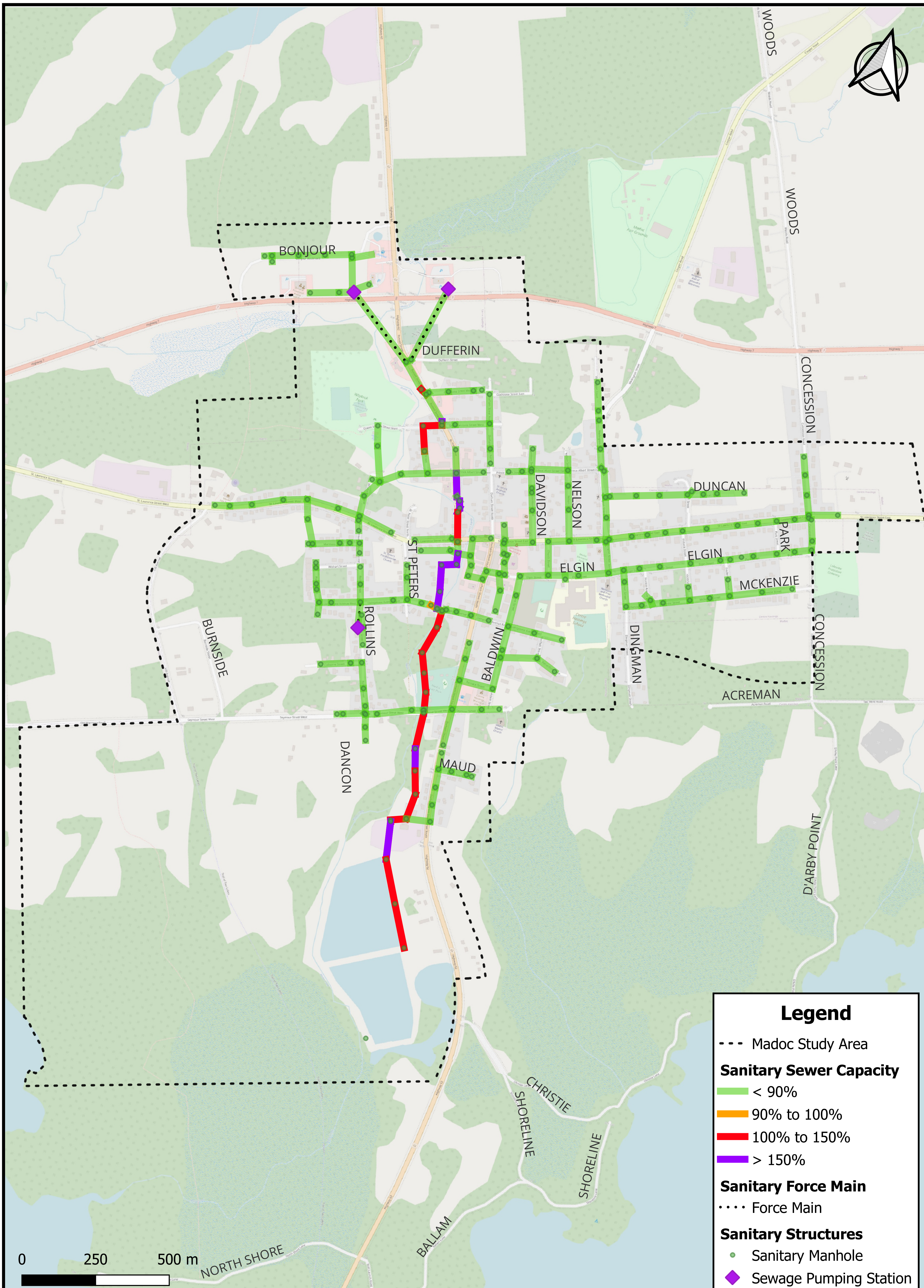
Segment	Corresponding Area / Street	Q_d/Q_{full} (%)	Length of pipe (m)
SANI-24 to SANI-21	Russel St. between Dufferin St. & Gladstone St.	127%	24.18
SANI-34 to SANI-35	Intersection of Russel St. & Queen Victoria St. E	244%	12.78
SANI-41 to SANI-47	Russel St. between Prince Albert St. W & St. Lawrence St. W	184%	80
SANI-48 to SANI-49	Russel St. between Prince Albert St. W & St. Lawrence St. W	206%	24.44
SANI-49 to SANI-50	Russel St. between Prince Albert St. W & St. Lawrence St. W	172%	15.89
SANI-50 to SANI-53	Russel St. between Prince Albert St. W & St. Lawrence St. W	106%	99.8
SANI-120 to SANI-119	Champlain St.	161%	36.61
SANI-119 to SANI-118	Champlain St.	246%	50.67
SANI-118 to SANI-117	Champlain St.	191%	90.44
SANI-117 to SANI-122	Champlain St.	191%	59.05
SANI-95 to SANI-100	Seymour St. W	196%	10.03
SANI-100 to SANI-142	ROW South of Seymour St. W	110%	124.18
SANI-142 to SANI-141	ROW South of Seymour St. W	154%	75.32
SANI-141 to SANI-140	ROW South of Seymour St. W	115%	79.3
SANI-140 to SANI-138	ROW South of Seymour St. W	117%	88.82
SANI-138 to SANI-139	ROW	116%	53.21
SANI-139 to SANI-263	ROW	106%	5.15
SANI-263 to SANI-254	ROW	173%	129.85
SANI-254 to SANI-255	ROW	142%	153.48
SANI-255 to SANI-256	ROW	141%	152.89
SANI-35 to SANI-45	Queen Victoria St. W	112%	63.66
SANI-125 to SANI-261	ROW	118%	59.8
SANI-261 to SANI-126	ROW	106%	99.22
SANI-126 to SANI-262	ROW	117%	69.16
SANI-262 to SANI-99	ROW	111%	65.61
SANI-99 to SANI-95	ROW	134%	60.98
SANI-45 to SANI-44	Madawaska St.	110%	86.01

The Pipe-by-Pipe Design sheet indicates that an additional 1 segments of the sanitary sewer are functioning between 90% to 100% capacity as summarized in Table 34.

Table 34: Sanitary Sewers functioning at 90% to 100% Capacity.

Segment	Corresponding Area / Street	Q_d/Q_{full} (%)	Residual Capacity (L/s)
SANI-116 to SANI-122	Livingstone Ave. W between St. Peters St. & Champlain St.	95%	0.89

Figure 24 illustrates the location of sanitary sewers modelled to be 90% full or more within the sanitary network.



PROJECT:

MADOC WATER, WASTEWATER & STORMWATER MASTER PLAN

Madoc, ON

DRAWING:

Existing Sanitary Network Capacity

J.L.Richards
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DESIGN:	PSC	JLR NO:	32508-000
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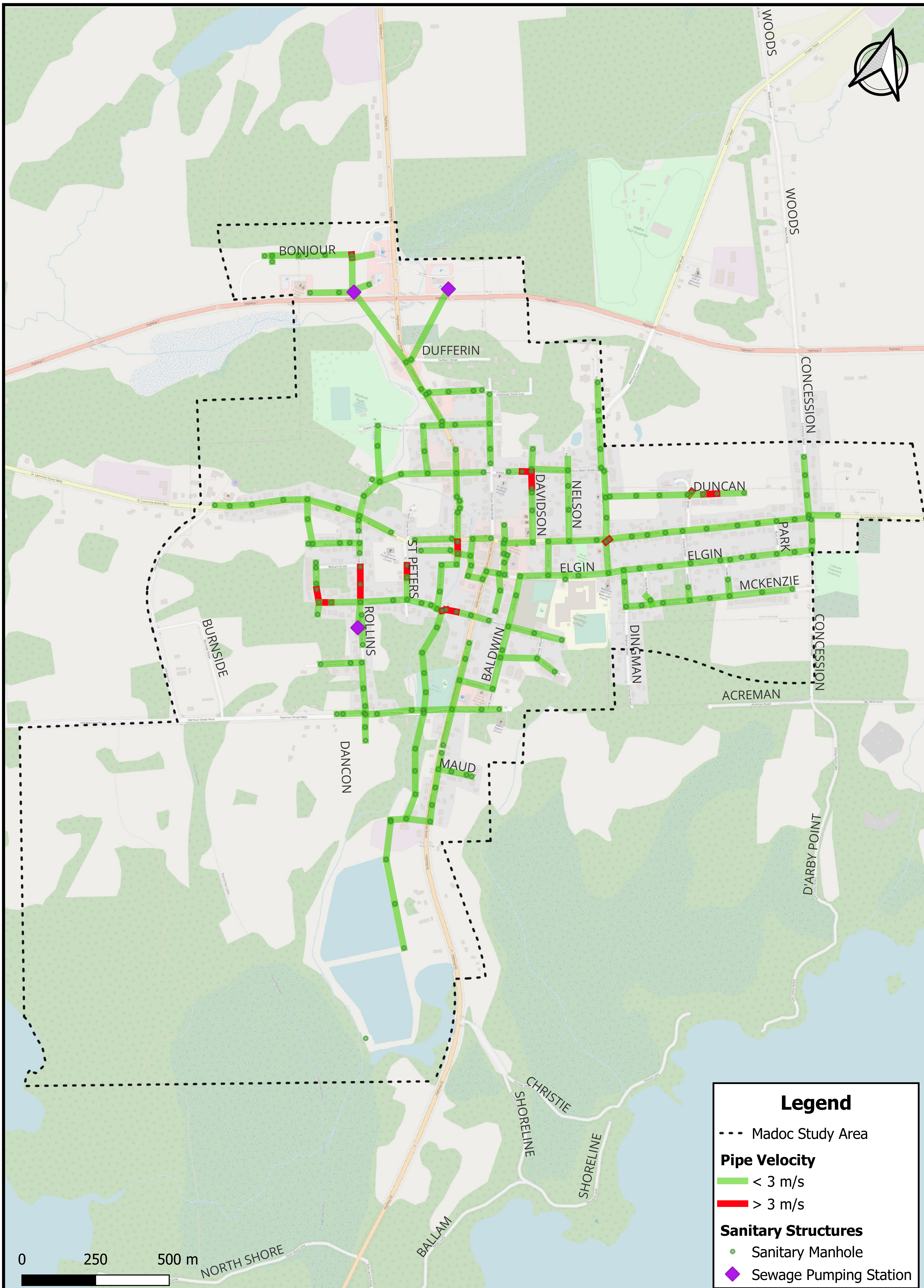
Figure 25 shows the existing sanitary network segments with velocities exceeding 3 m/s. The Pipe-by-Pipe Design sheet indicates that there are segments of the sanitary sewer system, with velocities exceeding 3m/s (Table 35).

Table 35: Sanitary Sewers with Velocities Greater than 3.00m/s

Segment	Corresponding Area / Street	Velocity (m/s)
SANI-209 to SANI-208	St. Lawrence Street	3.14
SANI-273 to SANI-272	Duncan Street	3.02
SANI-300 to SANI-202	Duncan Street	3.19
SANI-267 to SANI-267	Bonjour Blvd.	3.63
SANI-53 to SANI-120	Champlain Street	3.40
SANI-242 to SANI-239	Prince Albert Street E	3.07
SANI-239 to SANI-238	Davidson Street	3.73
SANI-275 to SANI-315	Francis Street	3.10
SANI-92 to SANI-91	Francis Street	3.10
SANI-87 to SANI-88	Rollins Street	3.18
SANI-303 to SANI-114	St. Peters Street S	3.23
SANI-124 to SANI-123	Livingstone Ave. W	3.03
SANI-123 to SANI-125	Livingstone Ave. W	3.57
SANI-88 to SANI-89	Rollins Street	3.11



PROJECT:

MADOC WATER, WASTEWATER & STORMWATER MASTER PLAN

Madoc, ON

DRAWING:

Existing Sanitary Sewers with Velocity Greater than 3 m/s

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DESIGN:	PSC	JLR NO:	32508-000
DRAWN:	RC	DRAWING NO:	**Figure 25**
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The Pipe-by-Pipe Design sheet indicates that there are segments of the sanitary sewer system, with slopes less than the MECP Guidelines.

The MECP Guidelines recommend the minimum slope for 200 mm pipe to be 0.4%. The table below shows the 200 mm segments with less than 0.4% slope.

Table 36: 200mm Sanitary Sewers with Slope less than 0.4%

Segment	Corresponding Area / Street	Slope (%)
SANI-193 to SANI-195	St. Lawrence Street	0.30%
SANI-195 to SANI-197	St. Lawrence Street	0.30%
SANI-207 to SANI-209	St. Lawrence Street	0.26%
SANI-172 to SANI-171	Elgin Street	0.30%
SANI-32 to SANI-170	Elgin Street	0.37%
SANI-170 to SANI-169	Elgin Street	0.30%
SANI-177 to SANI-184	Mckenzie Street	0.39%
SANI-184 to SANI-183	Mckenzie Street	0.37%
SANI-180 to SANI-179	Dingman Street	0.19%
SANI-230 to SANI-224	Nelson Street	0.37%
SANI-224 To sani-223	St. Lawrence Street	0.23%
SANI-244 to SANI-144	Richardson Lane	0.39%
SANI-15 to SANI-16	Durham Street N	0.26%
SANI-81 to SANI-82	Marmora Street	0.14%
SANI-84 to SANI-77	Marmora Street	0.19%
SANI-77 to SANI-72	Marmora Street	0.01%
SANI-236 to SANI-221	Davidson Street	0.37%
SANI-158 To SANI-157	ROW	0.16%
SANI-151 To SANI-155	ROW	0.16%
SANI-118 to SANI-119	Champlain Street	0.36%
SANI-110 To SPS-2	Rollins Street	0.08%
SANI-305 to SANI-104	Rollins Street	0.31%
SANI-302 to SANI-104	Rollins Street	0.34%
SANI-132 to SANI-131	Maud Street	0.02%

The MECP Guidelines recommend the minimum slope for 250mm pipe to be 0.28%. The table below shows the 250mm segments with less than 0.28% slope.

Table 37: 250 mm Sanitary Sewers with Slope less than 0.28%

Segment	Corresponding Area / Street	Slope (%)
SANI-219 to SANI-215	Wellington Street	0.27%
SANI-24 to SANI-21	Russel Street	0.07%
SANI-34 to SANI-35	Russel Street	0.02%
SANI-43 to SANI-42	Prince Albert Street W	0.12%
SANI-41 to SANI-47	Russel Street	0.11%
SANI-48 to SANI-49	Russel Street	0.09%
SANI-49 to SANI-50	Russel Street	0.13%
SANI-155 to SANI-10	Elgin Street	0.14%
SANI-10 to SANI-8	Durham Street S	0.09%
SANI-116 To SANI-122	Livingstone Ave. W	0.08%
SANI-136 to SANI-138	ROW	0.22%

The MECP Guidelines recommend the minimum slope for 300mm pipe to be 0.22%. The table below shows the 300mm segments with less than 0.22% slope.

Table 38: 300mm Sanitary Sewers with Slope less than 0.22%

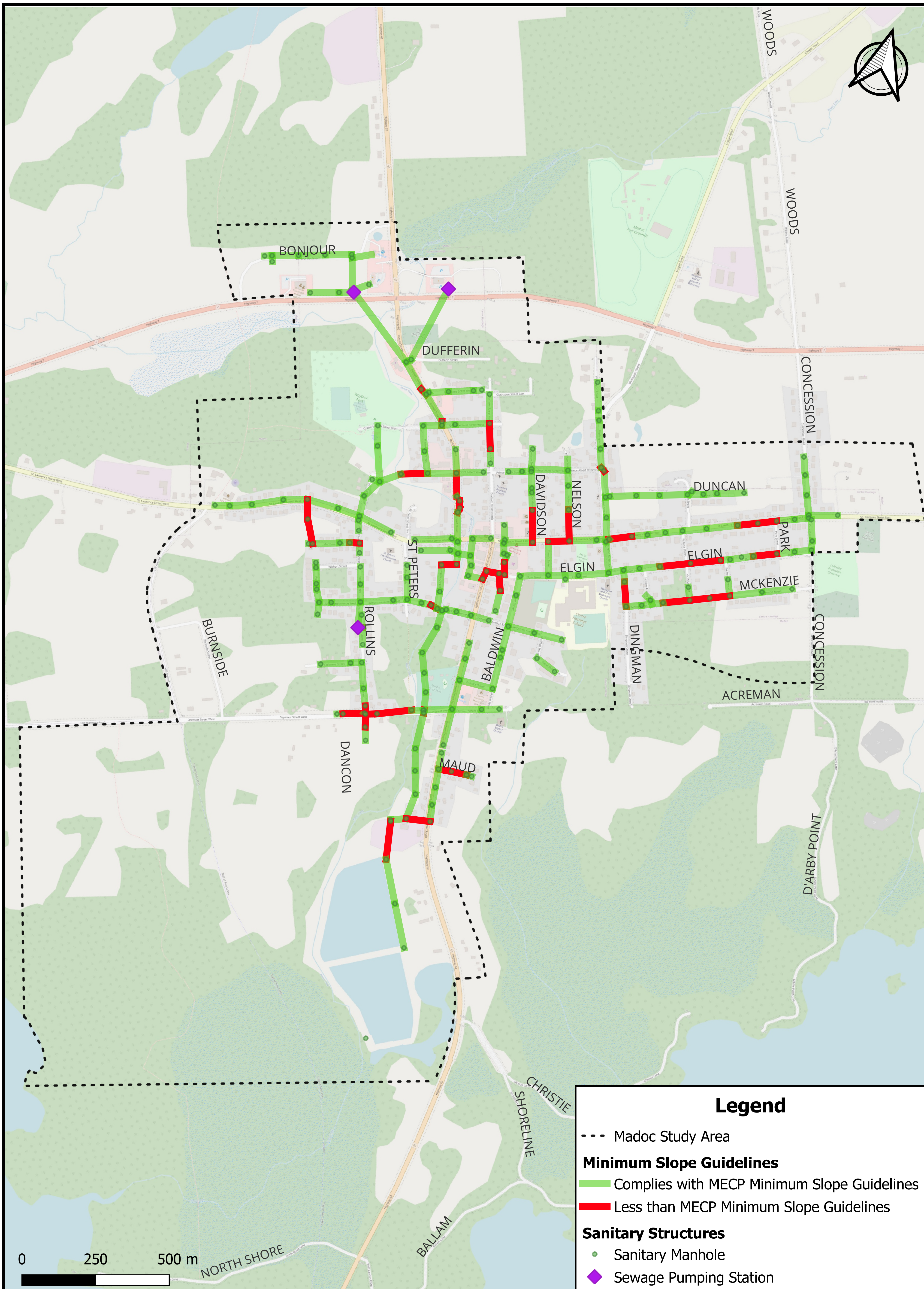
Segment	Corresponding Area / Street	Slope (%)
SANI-308 to SANI-309	Seymour Street W	0.20%
SANI-104 to SANI-103	Seymour Street W	0.12%
SANI-103 to SANI-101	Seymour Street W	0.215%

The MECP Guidelines recommend the minimum slope for 350mm pipe to be 0.17%. The table below shows the 350mm segments with less than 0.17% slope.

Table 39: 350mm Sanitary Sewers with Slope less than 0.17%

Segment	Corresponding Area / Street	Slope (%)
SANI-95 to SANI-100	ROW	0.11%
SANI-263 to SANI-254	ROW	0.15%

Figure 26 shows the existing sanitary network segments with slope less than the MECP Guidelines.



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MADOC WATER, WASTEWATER & STORMWATER MASTER PLAN

Madoc, ON

DRAWING:

Existing Sanitary Sewers with Slope Less than MECP Guidelines

J.L.Richards ENGINEERS · ARCHITECTS · PLANNERS

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DRAWN:	RC	DRAWING NO:	**Figure 26**
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6.2.8 Pump Capacity Assessment

Flows in the design sheet were assessed against the rated and peak capacities at each pump station in the system. Table 40 summarizes the rated capacity and modelled flow at each sewage pumping station.

Table 40: SPS Capacity Assessment

Pumping Station	SPS Rated Capacity (L/s)	Modelled Peak Flow (L/s)
Highway 7 SPS	10.2	2.61
McDonald's SPS	7.5	2.66
Rollins St. SPS	13.0	9.33

As shown in the above table, all pump stations receive flows below their rated capacity.

6.2.9 Level of Service

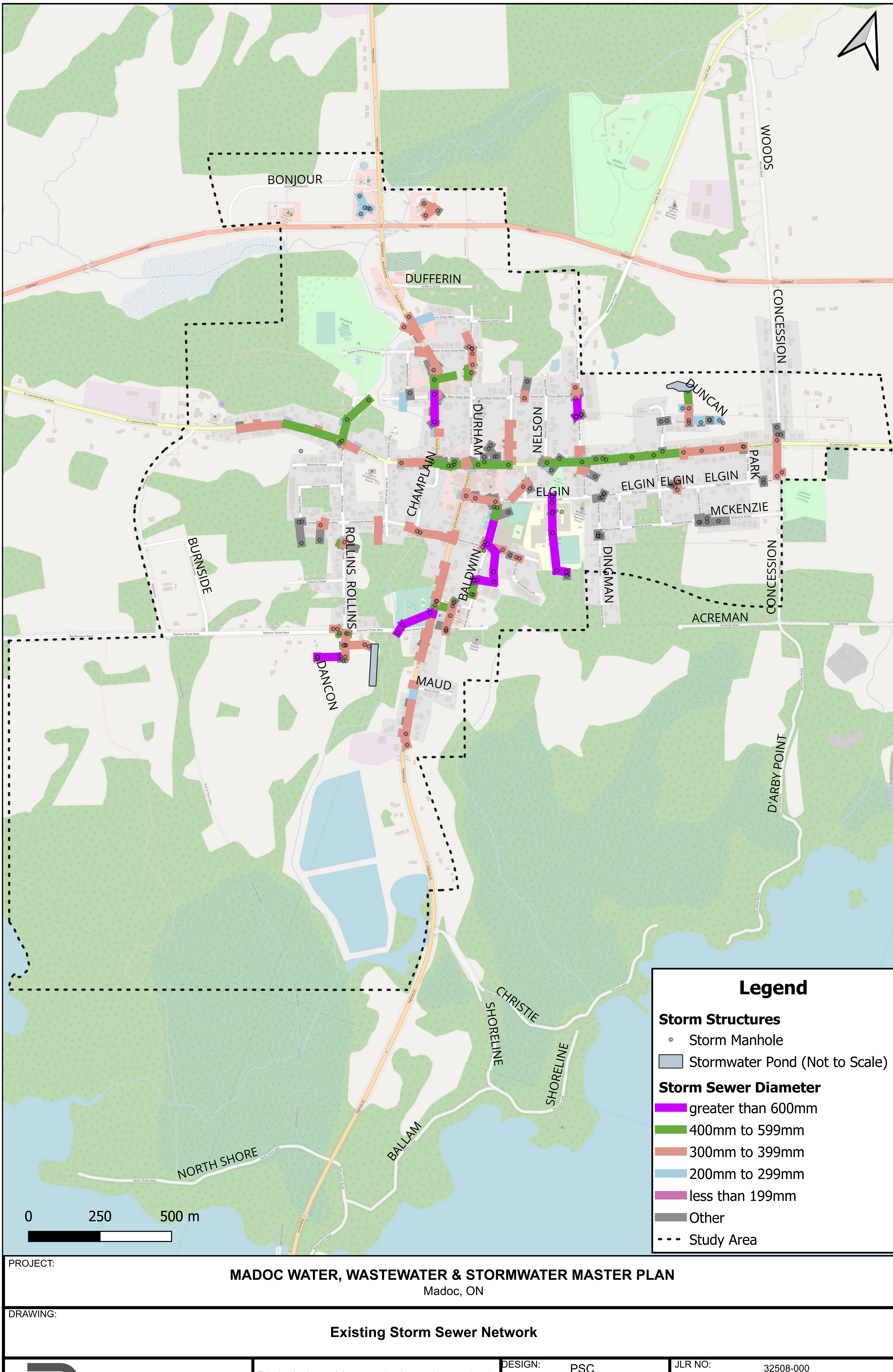
Based on the sanitary model results, the existing sanitary sewer network consists of pipes operating at 90% capacity or above. The pipe network also consists of several pipe sections which do not meet MECP Sewer Design Guidelines for velocity and slope. Based on anecdotal information from OCWA and Municipal Staff, sanitary sewer backups have not been observed within the past 5 years. The Municipality conducts cleanouts of the sanitary pipe network twice per year to prevent sediment build-up.

Additional modelling will be completed in Phase 2 of the Master Plan to incorporate sanitary flows from future development areas. Alternatives will be discussed in Phase 2 of the Master Plan to accommodate sanitary flows for the upcoming planning period of 20 years and beyond.

6.3 Stormwater Sewer Model

6.3.1 Existing Stormwater System

The existing storm sewer system in Madoc is a combination of storm sewers and ditches. Figure 27 illustrates all the known storm infrastructure in Madoc. The stormwater design sheet assesses the minor stormwater infrastructure (i.e., piped storm network) in the Madoc.



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MADOC WATER, WASTEWATER & STORMWATER MASTER PLAN

Madoc, ON

DRAWING:

Existing Storm Sewer Network

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CHECKED:	SS		

6.3.2 Historic Stormwater Flows

There are 10 separate minor stormwater systems in Madoc, which outlet to different areas in and around the Village of Madoc, as shown in Figure 28. Most of the systems outlet into Deer Creek, which runs through the Village. Based on anecdotal information provided by the Municipal/OCWA staff, there is no history of flooding in the past 30 years. No overtopping of Deer Creek has been observed in the past 30 years as well. According to the anecdotal information, there has been no flooding in the surrounding areas of intersection of Wellington St. N and Prince Albert St. E due to the unnamed creek that flows into the village from the northeast corner on Wellington St. N. However, there is not enough information on the effects of flow through the above-mentioned creek due to the new development on Duncan St. in the recent years.

6.3.3 Stormwater System Design Criteria

Stormwater conditions and design criteria were used to create a representative look at the stormwater flows generated for a typical 2-year storm for Madoc. The short duration rainfall intensity was taken from the IDF curve provided by MTO.

Rainfall intensity is assessed based on the following formula:

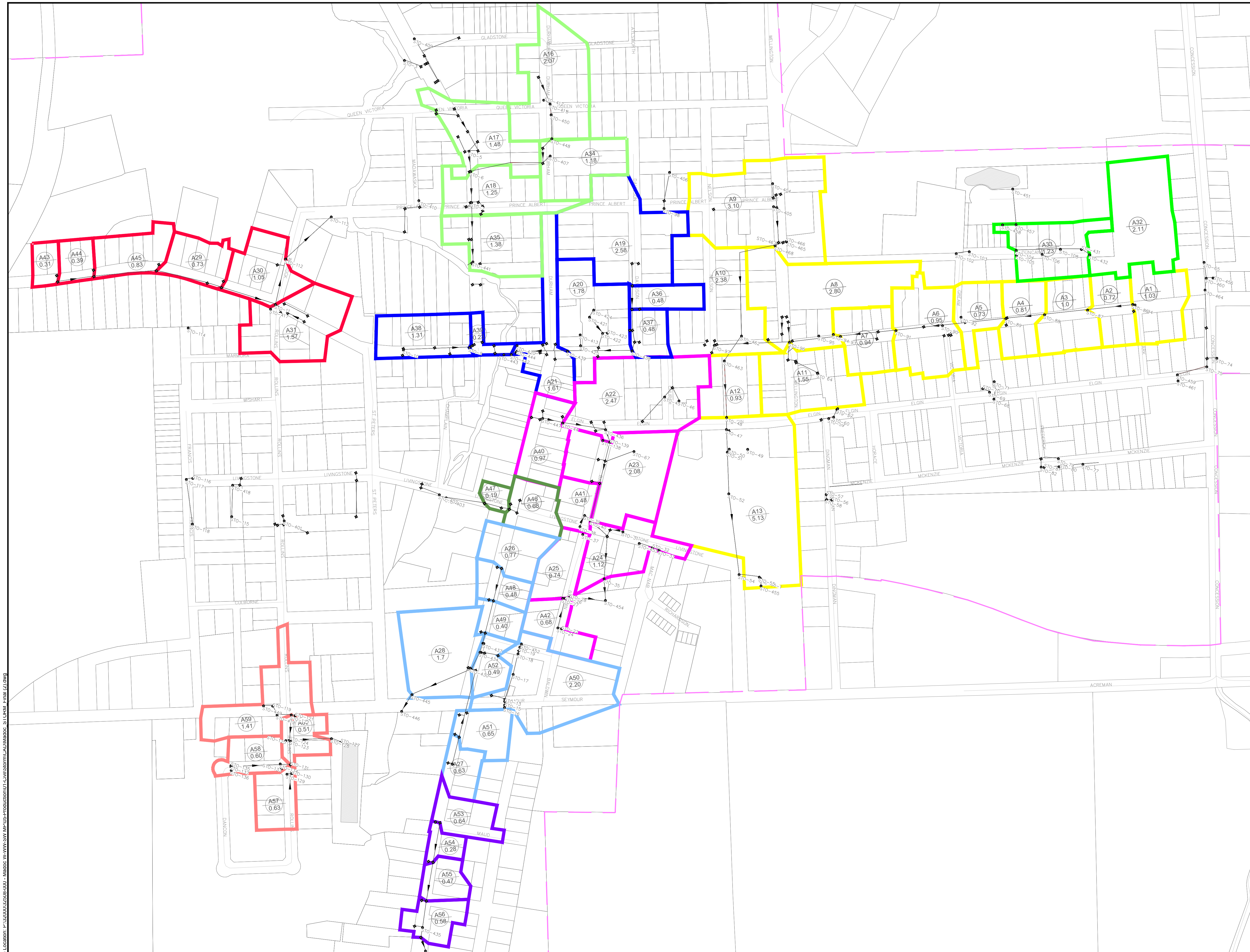
$$i = A*T^{B}$$

Where,
i = Rainfall intensity
A = Storm Return Period Variable
T = Time of Concentration
B = -0.699

For Madoc, the following design criteria were used.

Table 41: Stormwater Design Criteria

Parameter	Value	Comment
Runoff Coefficient • Grass areas • Paved areas	 0.25 0.9	Weighted average was calculated based on the catchment areas.
Storm Return Period Variable	21.1-46.6	Dependent on return period
Time of Concentration	20 minutes	



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- A## / #### — AREA LABEL / AREA IN HECTARES
- PROPERTY LINE
- STORM FLOW DIRECTION
- EXISTING STORM SEWER
- SANI-### — SANITARY MANHOLE AND I.D. LABEL
- CATCHBASIN
- STUDY AREA BOUNDARY
- DRAINAGE AREAS - STO-451 OUTLET
- DRAINAGE AREAS - STO-55 OUTLET
- DRAINAGE AREAS - STO-443 OUTLET
- DRAINAGE AREAS - STO-403 OUTLET
- DRAINAGE AREAS - STO-113 OUTLET
- DRAINAGE AREAS - STO-441 OUTLET
- DRAINAGE AREAS - STO-454 OUTLET
- DRAINAGE AREAS - STO-456 OUTLET
- DRAINAGE AREAS - STO-127 OUTLET
- DRAINAGE AREAS - STO-12 OUTLET
- EXISTING STORM WATER POND

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PROJECT:
Madoc Water, Wastewater and Stormwater Master Plan
Madoc, ON

DRAWING:
EXISTING STORMWATER DRAINAGE AREAS

DESIGN: PSC
DRAWN: RC
CHECKED: MM
JLR #: 32508-000

DRAWING #: Figure 28

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6.3.4 Stormwater Model Results

The spreadsheet model indicates that the flow from the unnamed creek on the northeast corner on Wellington St. N runs through the stormwater system that passes underneath Central Hastings School. The flow quantities for the unnamed creek are based on the open channel calculations using a high roughness coefficient value of 0.08 due to the tall grass and weeds in the creek.

The spreadsheet model indicates that 32 sewer segments have insufficient capacity to convey the design flow. Figure 29 shows the existing storm network capacities.

It shall be noted that the stormwater modelling results are not representative of the historic performance of the storm system. The following considerations have been provided as consideration in the difference between model results and real-life situation:

- All storm catchment areas have been assumed to flow to a single inlet, i.e., all stormwater free flowing to the inlet location. In reality, this does not always happen due to topography. Stormwater can flow in different directions and discharges to multiple locations within a catchment area.
- Stormwater pipes are assumed to have free flowing conditions. If there is a build-up of water level upstream of the pipe (i.e., pipe fully submerged), more flow can be pushed through the piped section. The model does not predict the additional flow under pressurized conditions, therefore potentially underestimating the actual flow that can be pushed through.

Table 42: Storm Sewers Over 100% Capacity

Segment	Corresponding Area / Street	Q_d/Q_{full} (%)	Length of pipe (m)
STO-87 to STO-88	St. Lawrence St. E	110%	70.7
STO-91 to STO-93	St. Lawrence St. E	162%	91.9
STO-93 to STO-96	St. Lawrence St. E	216%	83.9
STO-96 to Ditch	St. Lawrence St. E	266%	87.9
STO-48 to STO-47	Elgin St.	903%	19.8
STO-47 to STO-52	ROW	243%	68.2
STO-52 to STO-54	ROW	340%	133.2
STO-54 to STO-55	ROW	631%	33.3
STO-431 to STO-104	Duncun St.	198%	106.8
STO-104 to STO-457	ROW	102%	58.5
STO-450 to STO-448	Durham St. N	156%	39.2
STO-448 to CB-33	Durham St. N	128%	21.5
STO-5 to STO-6	Russel St.	102%	33.5
STO-6 to STO-7	Russel St.	173%	50.6
STO-209 to STO-208	Davidson St.	234%	36.4
STO-208 to STO-300	Davidson St.	612%	38.6
STO-300 to STO-204	Davidson St.	519%	34.1
STO-204 to STO-202	Davidson St.	769%	19.8
STO-438 to STO-439	St. Lawrence St. E	271%	105.8
STO-439 to STO-444	St. Lawrence St. E	251%	83.3
STO-444 to STO-442	St. Lawrence St. E	225%	9.74
STO-442 to STO-443	ROW	107%	12.1
STO-44 to STO-436	Elgin St.	112%	70.8
STO-436 to STO-139	Baldwin St.	166%	21.7
STO-40 to CB-133	Baldwin St.	204%	37.3
STO-112 to STO-113	St. Lawrence St. W	104%	101.5
CB-3 to CB-4	ROW	109%	22.4
STO-434 to CB6	Durham St. S	169%	30.4
CB-6 to CB-8	Durham St. S	228%	7.25
CB-8 to STO-445	ROW	168%	105.6
STO-435 to CB-129	Durham St. S	130%	16.6
STO-122 to STO-124	Rollins St.	104%	41.3

The Pipe-by-Pipe Design sheet indicates that an additional 7 segments of the storm sewer system are functioning close to capacity, at between 90% to 100% capacity.

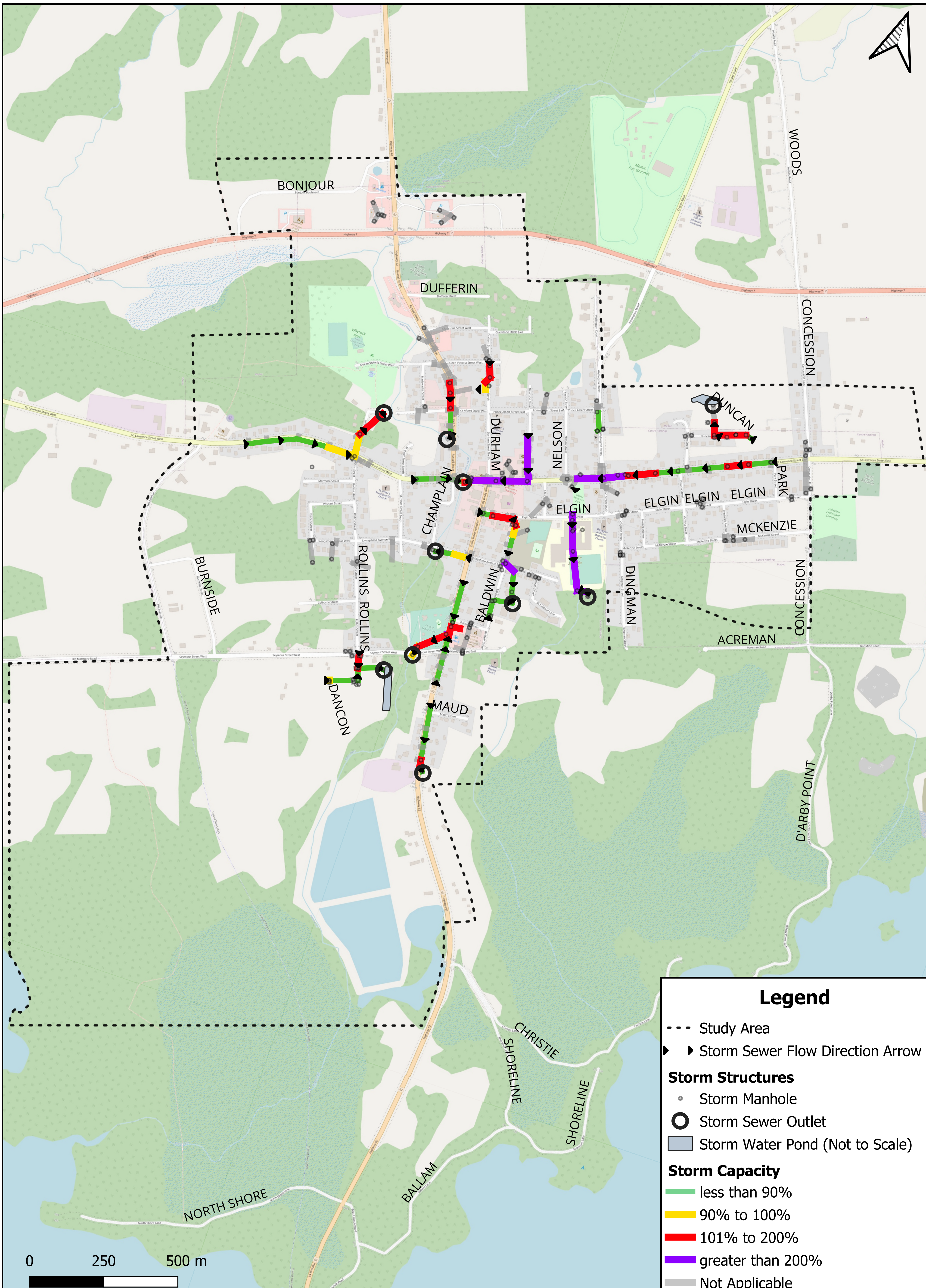
Table 43: Storm Sewers Functioning at 90% to 100% Capacity

Segment	Corresponding Area / Street	Q_d/Q_{full} (%)	Residual Capacity (L/s)
CB-33 to CB-31	Durham St. N	94%	11.2
STO-139 to STO-137	Baldwin St.	91%	30.83
STO-135 to STO-40	Baldwin St.	97%	12.21
CB-96 to STO-11	St. Lawrence St. W	94%	9.05
STO-11 to STO-112	St. Lawrence St. W	91%	22.04
CB-116 to CB-115	Livingstone Ave. W	98%	1.5
STO-445 to STO-446	ROW	96%	23.05

It is noted that based on the available GIS data, and the field survey conducted to get the information related to the physical attributes of sewer pipes such as invert elevations and pipe sizes, the sewer segments listed in Table 44 have negative slope. Sewer invert elevations are often difficult to measure in the field. For pipe sections with relatively flat slopes, compounded errors in invert measurement at the upstream and downstream maintenance holes can result in an inaccurate assessment of the slope, especially for short pipe lengths.

Table 44: Storm Sewers with Negative Slopes

Segment	Corresponding Area / Street	Slope (%)
STO-432 to STO-431	Duncun St.	-2.05%
STO-40 to CB-133	Baldwin St.	-0.05%
CB-4 to CB-5	ROW	-1.13%
STO-124 to STO-123	ROW	-1.46%



PROJECT:

MADOC WATER, WASTEWATER & STORMWATER MASTER PLAN

Madoc, ON

DRAWING:

Existing Storm Sewer Network Capacity

J.L.Richards
ENGINEERS · ARCHITECTS · PLANNERS

DESIGN:	PSC	JLR NO:	32508-000
DRAWN:	RC	DRAWING NO:	**Figure 29**
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6.3.5 Level of Service

The stormwater model reveals that the system comprises of pipe segments operating at 90% capacity and above, with a total of 32 pipe segments exceeding their capacity according to the stormwater design model. Additionally, there are pipe segments with negative slope within the system. Despite these challenges, anecdotal data spanning the last 30 years indicate no observed instances of flooding. Phase 2 of the Master Plan will identify and evaluate stormwater sewer capacity requirements to service future developments.

7.0 Problem and Opportunity Statement

Based on the work completed during the Phase 1 Master Plan Process, the following Problem/Opportunity Statement has been developed:

Madoc is serviced by communal water and wastewater systems consisting of Well #3 and Well #4, a water tower, over 16km of watermains, a sewage treatment system, three sewage pumping stations, over 16km of sanitary sewers, and minor storm systems on main road corridors. Water supply, treatment, treated water storage and lagoon treatment systems will not be sufficient to support projected growth within the Madoc servicing area for the next 20 years and beyond. In addition, there are various locations within the sanitary sewer and storm sewer systems that currently experience capacity constraints.

There is an opportunity through the Master Planning process to review the water, wastewater, and stormwater systems holistically and develop a strategic plan that can be prioritized and implemented logically with the intended goal of addressing future servicing needs and ensuring appropriate performance and reliability of Madoc's water, wastewater, and stormwater systems for the upcoming planning period of 20 years and beyond.

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Appendix A

Stakeholder Consultation Plan

Appendix B

Notice of Commencement

Appendix C

Stakeholder Responses and Mailing List

Appendix D

Water Model

Appendix E

Wastewater Model

Appendix F

Stormwater Model

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