

Prepared for:

THE MUNICIPALITY OF GREY HIGHLANDS
206 Toronto Street South, Unit 1,
Markdale, Ontario
N0C 1H0

Prepared by:

J.L. RICHARDS & ASSOCIATES LIMITED
107-450 Speedvale Avenue West
Guelph, ON
N1H 7Y6
Tel: 519-763-0713
Fax: 519-763-9261

Project File Report

Municipality of Grey Highlands Markdale Wastewater Municipal Class Environmental Assessment



Revision History

Rev.	Date	Description	Distribution
00	2026-03-20	Project File Report 30-Day Review Period	Posted for public review

Table of Contents

1.0	INTRODUCTION	1
1.1	Background.....	1
1.2	Study Area Overview	1
1.3	Project File Report Objectives	2
1.4	Class Environmental Assessment Process	5
2.0	PLANNING CONSIDERATIONS AND GROWTH PROJECTIONS	7
2.1	Planning and Policy Considerations	7
2.1.1	Provincial Policy Statement	7
2.1.2	Special Provincial Planning Policies	7
2.1.3	Grey County Official Plan.....	7
2.1.4	Municipality of Grey Highlands Official Plan	8
2.2	Population and Employment Growth Forecast	8
3.0	OVERVIEW OF EXISTING INFRASTRUCTURE	10
3.1	Description of Study Area and Existing Physical Infrastructure	10
3.2	Markdale Wastewater Treatment System.....	13
3.2.1	Pumping Stations Overview	13
3.2.2	Treatment Plant Overview	14
3.2.3	Water Quality Monitoring	16
3.2.4	Certificate of Approval Requirements	18
4.0	ENVIRONMENTAL AND LAND USE CONSIDERATIONS.....	19
4.1	Adjacent Property Uses	19
4.2	Source Water Protection.....	19
4.3	Surface Water.....	20
4.4	Groundwater	20
4.5	Geology and Hydrogeology	21
4.6	Natural Heritage and Species at Risk	21
4.7	Cultural Environment and Archaeological Resources	27
4.8	Climate Change	27
4.8.1	Climate Change Mitigation	28
4.8.2	Climate Change Adaptation	28
4.8.3	Grey County Climate Change Action Plan	28
5.0	EXISTING WASTEWATER TREATMENT PLANT PERFORMANCE	29
5.1	Historical WWTP Flow Data	29
5.2	Existing Per Capita Flows	29
5.2.1	Review of 2022 Water Billing Records.....	29
5.2.2	Review of Industrial Water and Wastewater Flow Contribution	30
5.2.3	Equivalent Populations	32
5.2.4	Review of Design Per Capita Wastewater Flows	34
5.3	Historical Influent Quality	35
5.4	Historical Treated Effluent Quality	36
5.5	Imported Sewage and Septage	38
5.6	Biosolids and De-sludging	38
6.0	WWTP CAPACITY ASSESSMENT	39
6.1	WWTP Overview	39
6.2	Hydraulics	40
6.2.1	Current Design and Operation	40
6.2.2	Potential for Increased Rated Capacity.....	41
6.3	Organics Treatment and Removal.....	41

6.3.1	Current Design and Operation	41
6.3.2	Potential for Increased Rated Capacity.....	42
6.4	Suspended Solids Treatment and Removal	43
6.4.1	Current Design and Operation	43
6.4.2	Potential for Increased Rated Capacity.....	44
6.5	Nitrogen Treatment and Removal.....	45
6.5.1	Current Design and Operation	45
6.5.2	Potential for Increased Rated Capacity.....	45
6.6	Phosphorus Treatment and Removal	46
6.6.1	Potential for Increased Rated Capacity.....	46
6.6.2	Current Design and Operation	46
6.7	RIB Wastewater Discharge.....	47
6.7.1	Current Design and Operation	47
6.7.2	Potential for Increased Rated Capacity.....	47
6.7.3	WSP Hydrogeological Baseline Assessment (Desktop), 2023	48
6.8	Overview of Capacity Constraints.....	49
7.0	PROBLEM AND OPPORTUNITY STATEMENT	51
8.0	DESIGN BASIS	52
8.1	Design Influent Volume.....	52
8.1.1	Average Day Flow.....	52
8.1.2	Maximum Day Flow	53
8.2	Design Influent Concentrations.....	53
8.2.1	Domestic Wastewater Quality and Loading	53
8.2.2	Imported Sewage Quality and Loading	54
8.2.3	Chapman's Wastewater Quality and Loading	55
8.3	Design Influent Flow and Loading Summary	55
8.4	Effluent Regulations and Requirements	55
9.0	PHASE 2 - EVALUATION OF TREATMENT EXPANSION OPTIONS	57
9.1	Alternative Solutions	58
9.2	Evaluation of Alternative Solutions	59
9.3	Description of Preferred Alternative.....	59
10.0	PHASE 3 - POTENTIAL DESIGN ALTERNATIVES	61
10.1	Headworks and Septage Receiving.....	61
10.1.1	Manual Bar Screen and Package Septage Receiving Station	61
10.1.2	Mechanical Screen System with Integrated Septage Receiving.....	62
10.1.3	Cost Estimate.....	62
10.2	Treatment Process	63
10.2.1	Aeration Upgrades	63
10.2.2	Addition of Advanced Treatment.....	63
10.2.3	Cost Estimate.....	64
10.3	Rapid Infiltration Basins (RIBs).....	64
10.3.1	WSP Hydrogeological Baseline Assessment (Field), 2026.....	65
10.3.2	Cost Estimate.....	66
10.4	Development of Alternative Design Concepts	68
10.4.1	OPTION 1: Manual Headworks + Upgraded Aeration + RIB Expansion	69
10.4.2	OPTION 2: Mechanical Headworks + Upgraded Aeration + RIB Expansion	69
10.4.3	OPTION 3: Mechanical Headworks + Advanced Treatment + RIB Expansion	70
10.5	Evaluation of Alternatives	70

11.0	DESCRIPTION OF THE PREFERRED ALTERNATIVE	73
11.1	Class D Opinion of Probable Cost	73
11.2	Land Use and Environmental Considerations	74
12.0	CONSULTATION	78
12.1	Notices	78
12.2	Public Information Centres	78
12.3	Consultation with Indigenous Communities	78
12.4	Comments	80
13.0	LIMITATIONS	83
14.0	REFERENCES	84

List of Tables

Table 2-1: Allocation of Residential Growth (2022)	9
Table 3-1: Markdale WWTP System History	10
Table 3-2: Sewage Pumping Station Physical Parameters	13
Table 3-3: Cell and Rapid Infiltration Basin Characteristics	14
Table 3-4: Effluent Design Objectives and Limits (ECA No. 9653-B5GLKM)	18
Table 3-5: Effluent Compliance Descriptions (ECA No. 9653-B5GLKM)	18
Table 5-1: WWTP Annual Flow Summary Table (2017-2022)	29
Table 5-2: Summary of 2022 Water Billing Records	30
Table 5-3: Available Domestic and Industrial Flow Data	31
Table 5-4: Historical Water Demand from the Chapman's facility	31
Table 5-5: Process Wastewater Flows from the Chapman's Facility	32
Table 5-6: Equivalent Population based on 2022 Water Consumption	33
Table 5-7: Daily Wastewater Flows at the Markdale WWTP (2017 – 2022)	34
Table 5-8: Per Capita Design Wastewater Flows used in Design.	35
Table 5-9: Summary of Wastewater Influent Quality Parameters ⁽¹⁾	36
Table 5-10: Summary of Effluent Water Quality Parameters	37
Table 5-11: Summary of Effluent Water Quality Percent Removal	37
Table 5-12: Imported Sewage Characterization (2021-2022)	38
Table 5-13: Imported Sewage Volumes (2021-2022)	38
Table 6-1: Flow and Concentration Data reported for the Markdale WWTP (2021)	40
Table 6-2: Lagoon Residence Times at Various Flow Rates	41
Table 6-3 WWTP Flow Rate Increase Scenarios	49
Table 6-4: Summary of WWTP Capacity Constraints	50
Table 8-1 Design Average Day Flow by Land Use	52
Table 8-2 Design Concentration and Loading for Domestic Wastewater Influent	54
Table 8-3 Design Concentration and Loading for Imported Sewage	54
Table 8-4 Design Influent Flows and Loading	55
Table 8-5: Anticipated Effluent Objectives and Limits	56
Table 9-1 Evaluation Impact Levels and Indicators	57
Table 9-2 Evaluation of Alternative Solutions	60
Table 10-1: Summary of Screening Technologies Cost Estimate	63
Table 10-2: Summary of Process Treatment Technologies Cost Estimate	64
Table 10-3: Summary of RIB Expansion Cost Estimate	66
Table 10-4 Evaluation of Design Concept Alternatives	72
Table 11-1 Class D OPCC Summary for the Preferred Alternative	73
Table 11-2: Mitigation Measures and Next Steps	75
Table 12-1: Consultation with Saugeen Ojibway Nation (SFN and CNUFN)	78

Table 12-2: Consultation with Métis Nation of Ontario.....	79
Table 12-3: Consultation with CGI, CRFN, GLMC, and HSM.....	79
Table 12-4: Comments from the Public	80
Table 12-5: Comments from Review Agencies.....	80
Table 12-6: Comments from Developers	82

List of Figures

Figure 1: Study Area Boundary.....	3
Figure 2: Overview of Key Wastewater Infrastructure	4
Figure 3: Wastewater Treatment Plant Site Plan	17
Figure 4: Adjacent Land Use Surrounding the WWTP	23
Figure 5: Natural Features Surrounding the WWTP	24
Figure 6: Source Water Protection Areas in Markdale.....	25
Figure 7: Land Classifications/Environmental Constraints (Pinchin, 2024).....	26
Figure 8: RIB Expansion Locations and Boreholes (WSP, 2026).....	67

List of Appendices

Appendix A	Terrestrial Ecology Report (Pinchin, 2024)
Appendix B	MCM Screening Forms and Archaeological Assessment Reports (ARA)
Appendix C	Hydrogeological Baseline Assessments (WSP, 2023 & 2026)
Appendix D	Consultation Records

1.0 INTRODUCTION

1.1 Background

J.L. Richards & Associates Limited (JLR) has been retained by the Municipality of Grey Highlands (the Municipality) to complete a Schedule B Municipal Class Environmental Assessment (MCEA) for the Markdale Wastewater System Capacity Assessment. The MCEA will assess the ability of the Markdale Wastewater Treatment Plant (WWTP) to accommodate increased demands from future development in the serviced region. During the original assignment, a capacity increase was found to be required and the project was upgraded to a Schedule C Class EA as per the MCEA process. Operational and infrastructure improvements will be identified to improve treatment and capacity.

The Environmental Assessment is being conducted in accordance with Phases 1, 2 and 3 of the Municipal Engineers Association Municipal Class Environmental Assessment (updated March 2023) to fulfill the requirements for Schedule C projects.

This Assessment will consider the following key issues:

- Project future sewage flow and loadings from the Chapman's Ice Cream facility.
- Determine the adequacy of the existing WWTP at the projected flows and recommend improvements to allow the WWTP to meet future flow requirements. Review the suitability of the current WWTP system and biosolids management strategies against the future organics loading.
- Evaluate whether further re-rating of the facility can be accommodated, or if long term and robust solutions are needed.
- Determine the feasibility of a headworks being incorporated into the WWTP.

Following the MCEA process will enable the Municipality to determine the most practical, workable, and affordable solution in the long term.

1.2 Study Area Overview

The Village of Markdale is situated within the Municipality of Grey Highlands, located in Grey County, southeast of Owen Sound. The Municipality has a population of approximately 10,000 people. The Markdale Wastewater system services approximately 1,200 people and a major area employer, Chapman's Ice Cream, which is a significant contributor of wastewater flows to the system.

The wastewater system consists of a gravity collection system, conveying flows to the Markdale WWTP with the assistance of two pumping stations at County Road 12 and Grayview Drive. The WWTP consists of a two-celled facultative lagoon operating in series and eight rapid infiltration basins (RIBs). Treated effluent is discharged to the granular subsoil, emerging as groundwater discharge at or slightly above the Rocky Saugeen River level, avoiding direct discharge into the river.

The WWTP is operated under Environmental Compliance Approval (ECA) No. 9653-B5GLKM, amended October 2018. It was initially designed with an average rated capacity of 1,122 m³/day when two RIBs were operated at a total flow rate of 1,000 m³/day for seven days. The WWTP was re-rated in 2017 for an average rated capacity of 1,722 m³/day where four RIBs are operated for seven days. The flow rate is constant at 500 m³/day per basin. Figure 1 illustrates the study area boundary. An overview of key wastewater infrastructure is shown in Figure 2.

1.3 Project File Report Objectives

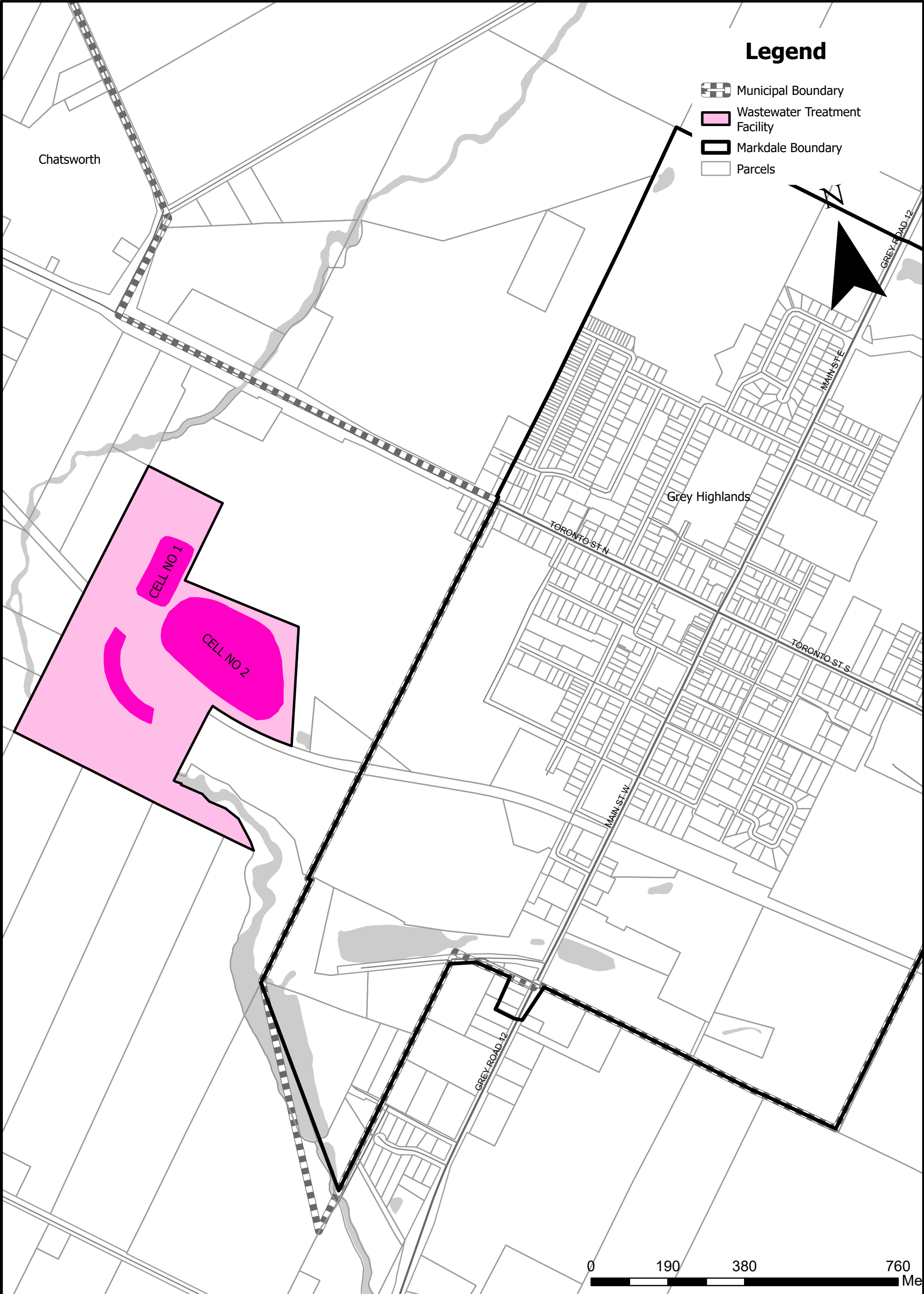
The main objective of a Schedule 'C' Class EA is to identify and evaluate alternative design concepts for the preferred solution identified in Phase 2.

Reasonable design concepts to implement the preferred solution are considered based upon the existing environment; input from the public, Indigenous communities, and review agencies; anticipated environmental effects; and methods of minimizing negative effects and maximizing positive effects.

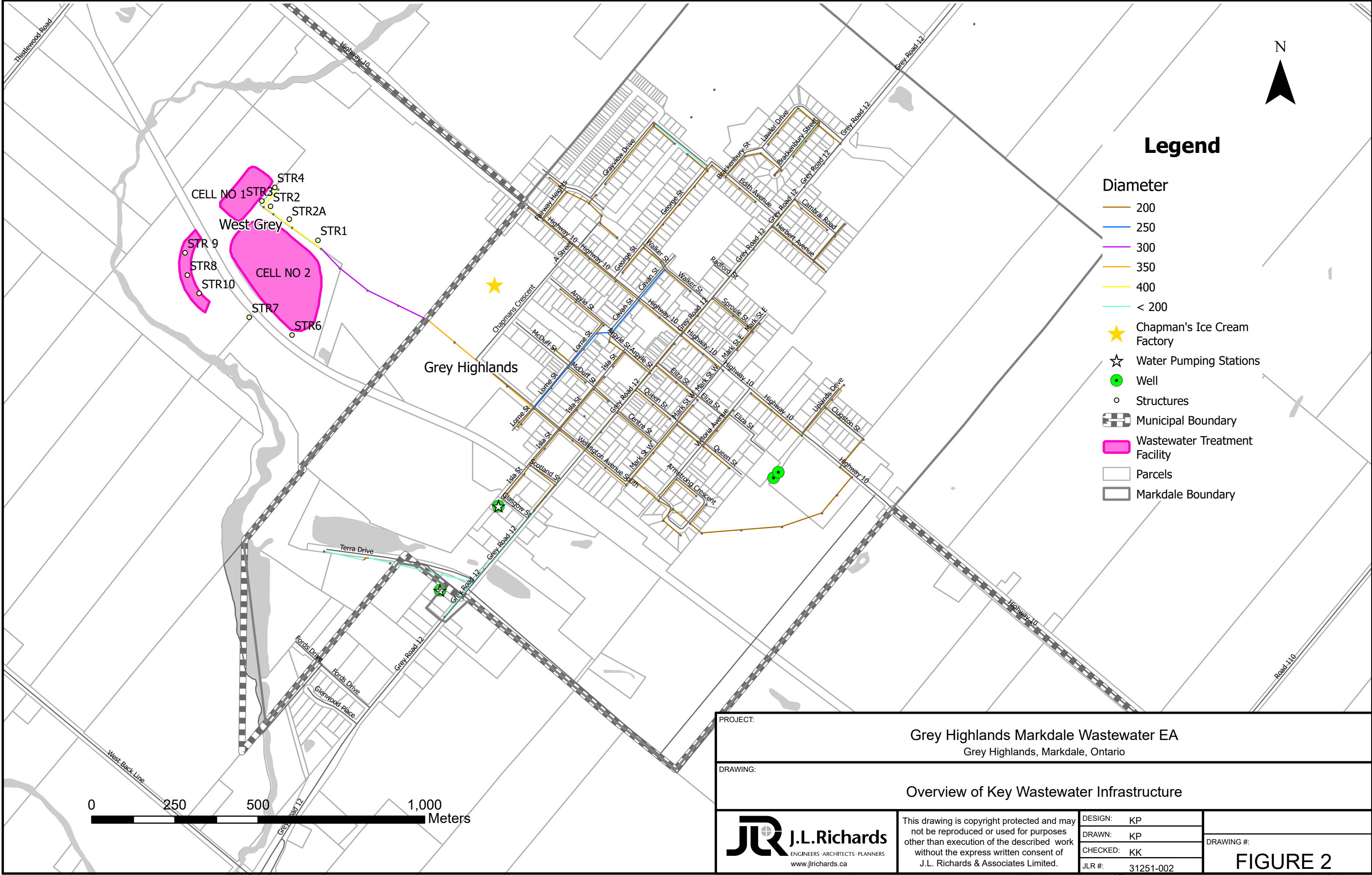
The objectives of the Project File Report are to:

- Identify alternative technologies and design concepts for the preferred solution.
- Identify the potential impact of the alternative design concepts, including environmental effects and mitigation measures.
- Perform an evaluation of the alternative design concepts.
- Confirm and finalize the preferred design concept after consulting with review agencies, Indigenous Communities, and the public.

File Location: \\jrichards\Corp\Projects\31000\31251-002 - Grey Highlands Markdale Wastewater EA.aprx



PROJECT:			
Grey Highlands Markdale Wastewater EA Grey Highlands, Markdale, Ontario			
DRAWING:			
Study Area Boundary			
 J.L. Richards ENGINEERS · ARCHITECTS · PLANNERS www.jlrichards.ca	This drawing is copyright protected and may not be reproduced or used for purposes other than execution of the described work without the express written consent of J.L. Richards & Associates Limited.		DESIGN: KP
			DRAWN: KP
			CHECKED: KK
			JLR #: 31251-002
			DRAWING #: Figure 1



PROJECT:				Grey Highlands Markdale Wastewater EA Grey Highlands, Markdale, Ontario	
DRAWING:				Overview of Key Wastewater Infrastructure	
 J.L.Richards <small>ENGINEERS · ARCHITECTS · PLANNERS www.jlrichards.ca</small>	This drawing is copyright protected and may not be reproduced or used for purposes other than execution of the described work without the express written consent of J.L. Richards & Associates Limited.		DESIGN: KP	DRAWING #: FIGURE 2	
			DRAWN: KP		
			CHECKED: KK		
			JLR #: 31251-002		

1.4 Class Environmental Assessment Process

The Ontario Environmental Assessment Act (Act) sets out a planning and decision-making process to consider potential environmental effects before a project begins. The purpose of the Act is to provide for the protection and conservation of the natural environment (R.S.O. 1990, c.E.18, s.2).

The Municipal Class EA (MCEA) process is followed for common types of projects to streamline the review process while ensuring that the project meets the requirements of the Act. In 1987, the first Class EA document prepared by the Municipal Engineers Association (MEA) on behalf of Ontario Municipalities was approved under the Act. Amendments were subsequently made in 1993, 2000, 2007, 2011, 2015, 2023, and 2024.

The MCEA process includes the following stages:

- Phase 1: Problem and/or opportunity identification.
- Phase 2: Identification and evaluation of alternative solutions.
- Phase 3: Preparation of alternative design concepts to support a preferred solution.
- Phase 4: Preparation of an Environmental Study Report (ESR) for posting and review on the public record.
- Phase 5: Project implementation and monitoring.

Since projects may vary in their environmental impact, they are now classified in terms of the following schedules, pursuant to the most recent amendment to the MCEA process in 2024:

- ‘Exempt’ projects, most of which were formerly classified as Schedule A and A+ projects, include various municipal maintenance, operational activities, rehabilitation works, minor reconstruction or replacement of existing facilities, and new facilities that are limited in scale and have minimal environmental effects. While these projects are exempt from the MCEA process, proponents should consider whether notice about the project should be given or consultation on the project should be carried out. Furthermore, proponents are also responsible for obtaining any other applicable permits, approvals, and authorizations for the project.
- ‘Eligible for Screening to Exempt’ projects may be eligible for exemption based on the results of a screening process. Proponents may choose to complete the applicable screening process to determine whether the project is eligible for exemption or proceed with the applicable Schedule ‘B’ or Schedule ‘C’ process, as noted below.
- Schedule ‘B’ projects have the potential for some adverse environmental impacts and therefore, the proponent is required to undertake the first two phases of the MCEA process. This includes mandatory consultation with Indigenous Communities, the public and other affected stakeholders as well as relevant review

agencies; and the preparation of a Project File which documents the Class EA process and is placed on the public record for review and comment. If there are no outstanding concerns and the regulatory process has been completed, then the proponent may proceed to implement the project. Generally, these projects include improvements and minor expansions to existing facilities or smaller new projects.

- Schedule 'C' projects have the potential for greater environmental impacts and are subject to the full MCEA process. This includes mandatory consultation with Indigenous Communities, the public and other affected stakeholders as well as relevant review agencies; identifying, assessing, and refining alternative solutions to determine a preferred solution; and preparing the ESR which documents the Class EA process and is placed on the public record for review and comment. If there are no outstanding concerns and the regulatory process has been completed, then the proponent may proceed to implement the project. Generally, these projects include the construction of new facilities and major expansions to existing facilities.

This MCEA was initiated with sufficient detail to fulfill the requirements for a Schedule 'B' project. It has since evolved into a Schedule 'C' project and as such, will be subject to the full MCEA process, as introduced above.

The ESR will be prepared during Phase 4 and be placed on the public record. A Notice of Completion will be issued, indicating that the ESR will be available for review by Indigenous Communities, the public, other stakeholders, and relevant review agencies for a mandatory 30-day comment period.

Proponents must then wait 30 days following the end of the comment period to determine whether a 'Section 16 Order' (formerly known as a 'Part II Order') or a 'Notice of Proposed Order' has been issued by the Ministry i.e., the 'Section 16 Order' requires the proponent to undertake an individual EA; a 'Notice of Proposed Order' imposes conditions on the project which the proponent must satisfy.

Under either of these orders, the project cannot proceed to the Phase 5 implementation phase unless and until all the associated requirements, processes and conditions are fulfilled and satisfied by the proponent.

2.0 PLANNING CONSIDERATIONS AND GROWTH PROJECTIONS

2.1 Planning and Policy Considerations

2.1.1 Provincial Policy Statement

The 2024 Provincial Policy Statement (PPS) provides general policy guidance on matters of provincial interest related to land use planning and development. The PPS also provides policy direction for appropriate development while protecting resources of provincial interest, public health and safety, and the quality of the natural environment.

All local planning matters must be consistent with the 2024 PPS, which is issued under Section 3 of the Planning Act. Policies related to infrastructure, servicing (sewer and water), climate change, natural heritage wetlands and water, and Indigenous interests may have implications at the Class EA level. Alternatives in this Class EA will be assessed based on conformance with the 2024 PPS.

2.1.2 Special Provincial Planning Policies

The study area is not subject to the Oak Ridges Moraine Conservation Plan, Niagara Escarpment Plan, Greenbelt Plan, Lake Simcoe Protection Plan, and the Growth Plan for the Greater Golden Horseshoe. The site is approximately 10 km west of the Niagara Escarpment Plan boundary.

2.1.3 Grey County Official Plan

The Grey County Council adopted its Official Plan in October 2018. It was approved by the Province of Ontario in June 2019 and consolidated in 2025. All new development must confirm to the County's Official Plan in addition to the Municipality's Official Plan. As outlined in the Official Plan, the purpose of the document is to manage and direct physical change and monitor its effects on the cultural, social, economic, and natural environment within the regional community until 2038. It incorporates the policy and regulatory framework outlined in the Provincial Policy Statement and the Niagara Escarpment Plan.

Markdale is located within the Municipality of Grey Highlands. The Markdale WWTP site is located within the Municipality of West Grey. Both Municipalities are within Grey County. The Markdale WWTP is located on lots 94, 95, and 96, Concession 1. The Grey County plan states that all new development on Lots 95, 96, 97, and Concession 1 must adhere to the Municipality of Grey Highlands Official Plan requirements and/or a secondary plan. The Grey County plan also states that prior to any development occurring on these lots, a secondary plan or updated Municipality of Grey Highlands Official Plan shall be completed.

2.1.4 Municipality of Grey Highlands Official Plan

The Municipality adopted the Official Plan in December 2015, which was approved by the County of Grey in September 2017. The Appeals Consolidated Version of the Official Plan was published in February 2022. All new development must conform to the County Official Plan in addition to the Municipality's Official Plan. As outlined in the Official Plan, the purpose of the document is to function as a tool to manage growth and protect environmental resources, with growth projections to the year 2026.

2.2 Population and Employment Growth Forecast

This Class EA was developed based on the County of Grey Official Plan (2019), which as noted above was subsequently consolidated in 2025. The Plan contains population growth projections allocated to the County and its municipalities until 2038. Markdale is included as a primary settlement area but did not have separate population allocations.

The Municipality of Grey Highlands Official Plan (2017) contains population growth projections allocated to the Municipality until 2026 based on a previous County of Grey Official Plan. Markdale is included as a primary settlement area but did not have separate population allocations. The Municipality's Official Plan set a goal for an overall average residential development density of 20 units per net hectare for the settlement area.

The Development Charges Background Study for the Municipality of Grey Highlands was prepared by DFA Infrastructure International Inc. in 2017. It presents the development changes for the period of 2018 to 2022, with a study period to 2027 and a build out period to 2041. Markdale is included as a primary settlement area but did not have separate population allocations.

In 2021, Hemson prepared a Growth Management Study for the County of Grey for use as input to the County of Grey Official Plan update. It contained population forecasts for the County and its municipalities to 2046.

The available data listed above does not allocate population projections to the Town of Markdale. The staff of the Municipality compiled data regarding future development areas based on information related to committed, approved, and potential developments in 2022, at the start of this Class EA. This data is summarized in Table 2-1.

There are three committed developments. The Markdale Hospital has 15 units with servicing complete. Phases 1, 2, and 3 of Stonebrook Developments have a total of 202 units with servicing commenced or completed. In addition to these projects there are five phases of the Devonleigh Homes Centre Point South project that have conditional approval, totaling 379 units. In addition to the committed and approved developments, there are two potential developments named Grey Gables and Loon Call Golf Course.

There is also potential for a significant expansion of the Chapman's Ice Cream facility. These potential units have not yet been identified and are not included in the table below.

Table 2-1: Allocation of Residential Growth (2022)

Area No.	Name	Committed	Approval	Potential	Total Units
1	Devonleigh Homes Centre Point South - Phase 1	172			172
2	Devonleigh Homes Centre Point South - Phase 2		41		41
3	Devonleigh Homes Centre Point South - Phase 3		51		51
4	Devonleigh Homes Centre Point South - Phase 4		65		65
5	Devonleigh Homes Centre Point South - Phase 5		50		50
6	Grey Gables			25	25
7	Loon Call Golf Course Lands			469	469
8	Markdale Hospital	15			15
9	Stonebrook Developments - Phase 1 and 2	133			133
10	Stonebrook - Phase 3	54			54
11	Chapman's Expansion			Unknown ⁽¹⁾	
				Total	1,075

Table 2-1 Notes:

- (1) Increased flows from a potential factory expansion are not currently being considered by the Municipality.
- (2) Allocation data is from the start of this Class EA, in 2022.

3.0 OVERVIEW OF EXISTING INFRASTRUCTURE

3.1 Description of Study Area and Existing Physical Infrastructure

The current wastewater treatment system consists of two aerated lagoons cells operating in series, and eight rapid infiltration basins that discharge into the groundwater table, indirectly discharging to the Rocky Saugeen River.

A chronological summary of the history of the Markdale wastewater treatment system is provided in Table 3-1.

Table 3-1: Markdale WWTP System History

Year	Activity Description
Circa 1960	<ul style="list-style-type: none"> The Markdale Wastewater Treatment Plant is constructed with a one cell (later renamed Cell No. 2) facultative lagoon. MECP Certificate of Approval No. 3-1290-81-846. Design population for facility estimated at 1,977 persons with an average daily flow of 1,122 m³/day.
Circa 1970	<ul style="list-style-type: none"> Grayview Drive SPS is constructed.
Circa 1974	<ul style="list-style-type: none"> Lawler Dive SPS is constructed.
Circa 1984	<ul style="list-style-type: none"> Cell No. 1 is constructed. Rapid infiltration basins are installed directly into the riverbank and filled with fine brick sand to slow effluent flow.
1984	<ul style="list-style-type: none"> System upgraded to increase capacity, seasonal storage, and implement treatment through aeration. Extended old influent trunk sewer to carry raw sewage into Cell No. 1, provided bypass piping for maintenance, built blower building, built alum system, and installed lagoon control structures.
1997	<ul style="list-style-type: none"> MECP requests monthly summary sheets on sampling results and operational data due to concerns about odour and whether effluent discharge criteria were being met. Two additional surface aerators are added to Cell No. 1 in July in response to an increase in organic loading and low dissolved oxygen (DO) levels, resulting in a total of five aerators. A study by Henderson Paddon & Associates (now WSP Canada Inc.) found that the Chapman's Ice Cream facility produced 52% of the total flow and 87% of the BOD loading observed at the WWTP. This flow was significantly higher than the water consumption and was attributed to flow meter inaccuracies and milk imported to the site. Loading was 2.2 times the biological oxygen demand (BOD) capacity of the WWTP.

Year	Activity Description
1999	<ul style="list-style-type: none"> Cell No 1 is drained. A new concrete wall is constructed that splits the cell such that raw sewage enters the first third for settling before entering the remainder for aeration. After sludge removal operations, a fine bubble membrane style subsurface EDI FlexAir™ aeration system was installed by Environmental Dynamics due to increased waste loads from the Chapman's facility from 1997 to 2000. The Chapman's facility installs a sequencing batch reactor pre-treatment system. Cell No. 1 was commissioned in December. Most recent agreement between Municipality and Chapman's Ice Cream is made.
2002	<ul style="list-style-type: none"> Study conducted by Henderson Paddon & Associates to address the issue of hauled septage disposal in the Municipality of Grey Highlands sewage treatment facilities. Confirmed the hydraulic loading to the sewage works was below the average daily design capacity. MECP waived special requirements. Amended Certificate of Approval issued in November. Required sampling of raw sewage and effluent continue on a bi-monthly basis. Normal operating time of the RIBs was increased to March 1 to November 30.
2009	<ul style="list-style-type: none"> The Chapman's facility catches fire, resulting in lower sewage production for 2009 and 2010.
2010	<ul style="list-style-type: none"> Gamsby and Mannerow Ltd. Prepared a report on water servicing for the new Chapman's facility.
2011	<ul style="list-style-type: none"> Grayview Drive SPS is decommissioned, removed, and replaced. New SPS services 25 residences on Grayview drive and potential adjacent service areas. Lawler Drive SPS is decommissioned and removed. County Road 12 SPS constructed. New, larger Chapman's Ice Cream facility is constructed after the fire and begins operation in the spring. Flows and loadings higher than the WWTP's rated capacity in the late spring result in odour complaints in Cell No. 1. Municipality installs three floating aerators to temporarily increase the treatment capacity of Cell No. 1. GENIVAR Inc. (now WSP Canada Inc.) prepares a report analyzing the options to permanently mitigate this issue. The report focuses on the installation of in-situ floating aerators (Blue, Gold, and/or Yellow Frogs). The Chapman's facility pre-treatment system is upgraded, including a tank to increase capacity. In November, six Yellow Frogs are installed in Cell No. 1.

Year	Activity Description
2012	<ul style="list-style-type: none"> GENIVAR prepares Draft Master Plan for the Municipality of Grey Highlands. The GENIVAR Optimization report analyzed operations and the benefits of introducing Yellow and Gold Frogs to the system. They proposed increasing the hydraulic capacity from 1,122 m³/day to 1,412 m³/day. Two Gold Frogs and an ultrasonic device are purchased for installation in Cell No. 2 to control algae production.
2014	<ul style="list-style-type: none"> ECA No 9378-9M2RLX issued in August. Permitted the modification of two of the RIBs to enable year-round operation. Sand dug out of RIBs and replaced with stone. Blue Frog aeration system and floating islands added to Cell No. 1 by C&M Aquatics. Sludge survey conducted.
2015	<ul style="list-style-type: none"> Existing RIBs 6 and 7 modified for winter operation by GSS Engineering Consultants Ltd. Winterized operation begins in November.
2016	<ul style="list-style-type: none"> Year-round operation of RIBs begins in January. Terrapin Water analyzes the impact of Blue Frogs on the system. It concludes that the system layout is sub-optimal, and aeration should move into Cell No. 2 so settling can occur in Cell No. 1. Attached growth media is also recommended as nitrification reduces in the summer due to Blue Frogs digesting the last of the winter solids, which serve as habitat for nitrifiers.
2017	<ul style="list-style-type: none"> WSP Canada Inc. submits final Water and Wastewater Servicing Master Plan to the Municipality of Grey Highlands. Determined the feasibility of re-rating the Markdale WWTP alongside optimization of the existing plant and construction of a new water storage facility in Markdale. Aeration tube in Cell No. 2 is damaged during sludge removal by GeoDredge and no longer functions at full capacity. Geobags are filled with sludge and remain on site. Current configuration of Blue Frogs and islands is finalized.
2018	<ul style="list-style-type: none"> Amended ECA No 9653-B5GLKM issued in October. Re-rated plant capacity from 1,122 to 1,722 m³/day.
2019	<ul style="list-style-type: none"> Milne Technologies conducts sonar mapping survey of sludge accumulation in Cell No. 1 in September.
2020	<ul style="list-style-type: none"> Milne Technologies conducts sonar mapping survey of sludge accumulation in Cell No. 2 in September.

3.2 Markdale Wastewater Treatment System

Figure 2 in Section 1.0 shows an overview of the existing wastewater servicing system. Gravity sewers (200 mm) are installed throughout the Village to collect wastewater. A general description of the Grayview Drive and Country Road 12 Sewage Pumping Stations (SPSs) are provided in Table 3-2.

A 100 mm diameter DR 25 PVC forcemain conveys collected sewage from the Grayview Drive SPS for discharge at Manhole 196 near the Grayview Drive and George Street intersection.

A 125 mm diameter DR-26 PE forcemain conveys collected sewage from the County Road 12 SPS for discharge at Manhole 147 near the Cambrai Road and County Road 12 intersection.

All wastewater subsequently flows by gravity to the Markdale WWTP.

3.2.1 Pumping Stations Overview

This system has two SPSs that are local collectors which discharge to the gravity system.

Table 3-2: Sewage Pumping Station Physical Parameters

Parameter	Grayview Drive SPS	County Road 12 SPS
Wet Well Diameter (m)	3	3
Approximate Wet Well Depth (m)	11	8
Pumps	2 vertical centrifugal, non clog, wet well sewage pumps. One duty, one standby.	2 vertical centrifugal, non clog, wet well sewage pumps. One duty, one standby.
Firm PS Capacity (Imperial Gallon/minute)	80 IGPM (6.06 L/s) at 12.4 m TDH.	110 IGPM (8.33 L/s) at 18.7 m TDH.
Standby Power	Diesel generator at generator building south of Lawler Drive and County Road 12	Diesel generator at generator building adjacent to the SPS at Lawler Drive and County Road 12
Flow Measurement	n/a	n/a

3.2.2 Treatment Plant Overview

The Markdale WWTP generally consists of two (2) facultative lagoon cells, a Parshall flume for flow monitoring, a blower building, an alum dosing system for phosphorus removal, and eight (8) rapid infiltration basins (RIBs). Refer to Figure 3 for an overview of the existing WWTP site plan and key infrastructure.

Table 3-3: Cell and Rapid Infiltration Basin Characteristics

Cell	Function	Operating Depth (m)	Surface Area (m ²)	Theoretical Volume (m ³)
No. 1	Treatment (settling, sludge accumulation, and aeration) Primary Cell	3.575	13,980	38,908 ⁽¹⁾
No. 2	Polishing (alum and aeration), storage Secondary Cell	3.9	62,700	220,570 ⁽¹⁾
RIBs		0.6	336 ⁽²⁾	

Table 3-3 Notes:

(1) From ECA No. 9653-B5GLKM

(2) Calculated from dimensions provided in ECA No. 9653-B5GLKM

The current system of cells and lagoons was constructed by the 1980s. Notable updates were made in 1984, 1999, and 2014. The current system is described below, referencing discussion with operators and the Markdale WTP and SPS Operations Manual (WSP, 2022).

Raw sewage enters Cell No. 1 through an influent discharge structure fed by a 400 mm diameter gravity sewer. A Parshall flume on the raw sewage line measures influent flow. Half of the alum required for phosphorus removal is injected into the flume to aid with settling. Septage is added into the system by direct entry through a cement pad with a screen. There is no headworks system which can cause clogging issues in Cell No. 1.

Wastewater produced by the Chapman's Ice Cream facility is collected by a 200 mm diameter sanitary sewer running South to North on Wellington St and a 250 mm diameter sanitary sewer runs West to East on Isla Street.

The first third of Cell No. 1 is partitioned from the rest by a submerged concrete wall. The first third of the cell houses six three horsepower hybrid Blue Frog aerators/mixers. Three are positioned in the septage discharge area while the remaining three are positioned over the inlet. After settling and sludge accumulation, water flows over the wall into the remaining two thirds of the cell which contains a fine bubble membrane style subsurface aeration system that was installed in 1999. Aeration runs constantly except for during

power outages. There are 150, 1 m by 1 m floating vegetated islands installed in this section of Cell No 1.

The treated sewage flows by gravity out of Cell No. 1 into Cell No. 2 through structure 5. There are three influent pipes from Cell No. 1 in structure 5, however only the 300 mm diameter mid-height pipe is used during normal operation. The remaining alum is added by metering pumps in structure 5 for phosphorus removal. Water quality data for Cell No. 1 is also measured at this manhole.

A blower building and 22,000 L fibreglass reinforced plastic chemical storage tank are located between the two cells. A site tube records the alum tank level daily to crosscheck proper alum dosage to the system for phosphorus removal. The feed rate varies from 30 to 200 mg/L and is usually close to 100 mg/L.

Cell No. 2 is aerated by the original diffused air aeration system (circa 1960) with air provided by a positive displacement rotary blower in the blower building to ensure anaerobic conditions do not occur. One 3 HP surface aerator/mixer is located at the inlet of Cell No. 2 to assist with algae settling and aeration. Two 15 HP aerators can be utilized at the outlet of Cell No. 2 through a 3 HP fine bubble surface aerator/mixer as needed. Cell No. 2 also contains an original overflow structure for emergency discharge to the Rocky Saugeen River. Reportedly, due to the limited capacity of the Cell No. 2 aeration system, a significant portion of aeration treatment occurs in Cell No.1.

The treated effluent flows by gravity or pumping out of Cell No. 2, through a 200 mm sewer, into the rapid infiltration basins (RIBs). A control valve in structure 6 controls the flow in the system based on the water level in Cell No. 2. Operators maintain the water level at approximately 2.5 m, measured from the lid of structure 6 to the water surface. Structure 6 contains the flow controls, instrumentation, and pumps for operation of the RIBs, as well as three pipes to receive effluent from Cell No. 2. During normal operation only the mid-height pipe is used. When high density algae are present, the inlet height is manually adjusted.

Flow is conveyed to structure 7, where flow can pass into the RIBs or bypass to the river. Structure 8 contains mud valves to direct flow to RIBs 1 through 4, 5 through 8, or all basins. Structures 9 and 10, both fed by structure 8, can feed any individual basin or a combination. Typically, one or two RIBs are active during normal operation, depending on flow levels.

RIBs 6 and 7 were winterized in 2015 and are both active from December to the spring, between mid-March and the end of April. They are covered when not in operation to avoid algae build up. The remaining six RIBs can receive effluent between March 1 and November 30. The location for effluent sampling is dependent on which RIBs are active and which structure is conveying effluent into the RIBs. Flow out of the RIBs is not measured directly.

During normal operation, two RIBs are loaded daily for seven days, then rested for seven days. If needed, near the end of the seven-day resting period the bottoms of the two rested RIBs are re-tilled to restore hydraulic capacity. The WWTP was re-rated in 2017 for an average rated capacity of 1,722 m³/day where four RIBs are operated for seven days.

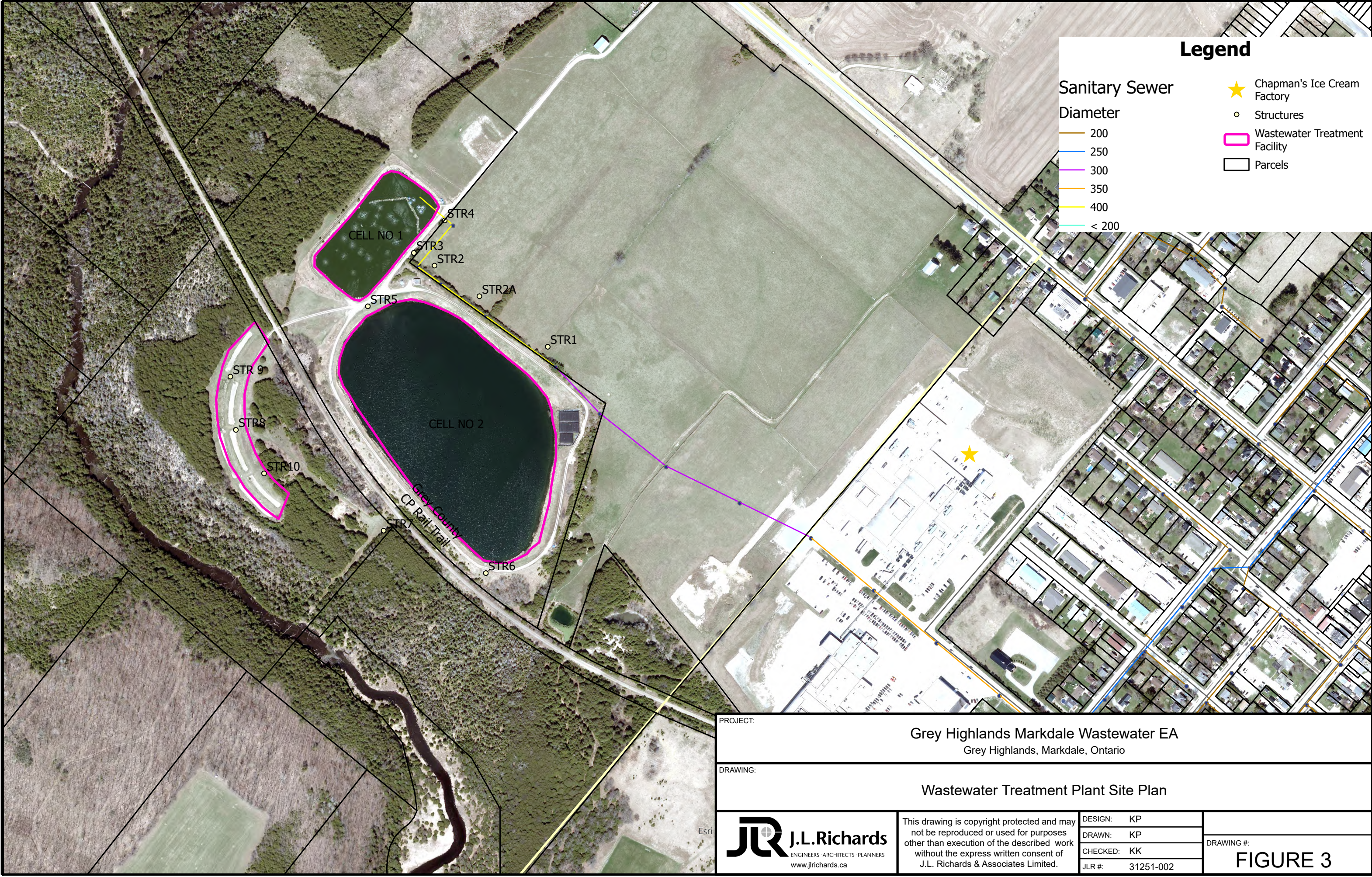
Typically, one to two RIBs are in operation throughout the warm months. Operation is dependent on flows. Two RIBs were commonly used for most of the warm months until 2020 when the predominant mode of operation utilized only one RIB. Two RIBs tend to be in use during the transitional periods in April and November. Exceptional RIB usage included having four RIBs in operation in April 2016 and 2019, no RIBs in operation in late June and early July 2017, and fluctuating RIB usage in November 2021.

Effluent from the RIBs travels downward to the groundwater table where it is treated in the aerobic zone and subsurface geologic environment.

3.2.3 Water Quality Monitoring

There are a series of groundwater and surface water quality monitoring stations where data is collected quarterly. Groundwater test wells OW1-86, OW2-86, OW6-86, OW6A-86, OW6B-86, and OW13-86 are located along the predicted flow path of effluent from the RIBs. Some effluent reappears at the ground surface in the form of seeps monitored by TP3-79 and CW2-86. The effluent moves laterally through organic soil in the cedar wetland monitored by CW9-86 where it is further attenuated. Data is also collected from wells OW 16-79 located between the RIBs and Cell No. 2 and CW 10-86 located on the banks of the Rocky Saugeen River, east of CW9-86. There are many more observation wells in the area that are not monitored.

All the effluent eventually enters the Rocky Saugeen River. Three surface water quality monitoring stations are located on the Rocky Saugeen River: upstream of the effluent infiltration area (SW1-86), in the river facing the infiltration system area (SW2-86), and downstream of the effluent infiltration area at the footbridge (SW3-86).



Legend

- Sanitary Sewer Diameter**

 - 200
 - 250
 - 300
 - 350
 - 400
 - < 200
- ★ Chapman's Ice Cream Factory
 - Structures
 - Wastewater Treatment Facility
 - Parcels

PROJECT:			Grey Highlands Markdale Wastewater EA Grey Highlands, Markdale, Ontario	
DRAWING:			Wastewater Treatment Plant Site Plan	
 <p>J.L. Richards ENGINEERS · ARCHITECTS · PLANNERS www.jlrichards.ca</p>	This drawing is copyright protected and may not be reproduced or used for purposes other than execution of the described work without the express written consent of J.L. Richards & Associates Limited.		DESIGN: KP	DRAWING #: FIGURE 3
			DRAWN: KP	
			CHECKED: KK	
			JLR #: 31251-002	

3.2.4 Certificate of Approval Requirements

The Markdale WWTP is operated in accordance with the Environmental Compliance Approval (ECA) Number 9653-B5GLKM amended October 2018. The WWTP is rated for an average day flow of 1,722 m³/day. There are limits set on concentration in effluent and effluent loading. The key objective and compliance requirements for the treatment system after re-rating are outlined in Table 3-4 and Table 3-5 below.

Table 3-4: Effluent Design Objectives and Limits (ECA No. 9653-B5GLKM)

Parameter	Concentration Objective	Concentration Limit	Waste Loading Limit
Five-day Carbonaceous Biochemical Oxygen Demand (CBOD ₅)	20 mg/L	25 mg/L	43.0 kg/d
Total Suspended Solids	25 mg/L	30 mg/L	51.6 kg/d
Total Phosphorus	0.5 mg/L	0.65 mg/L	1.12 kg/d
pH	6.5 – 8.5 (inclusive)	6.0 – 9.5 (inclusive)	-

Table 3-5: Effluent Compliance Descriptions (ECA No. 9653-B5GLKM)

Parameter	Non-compliance
CBOD ₅	Monthly average effluent concentration is the arithmetic mean of all Single Sample Results of the concentration of a contaminant in the Final Effluent sampled or measured during a calendar month. (24-hour composite, weekly)
Total Suspended Solids	Monthly average effluent concentration is the arithmetic mean of all Single Sample Results of the concentration of a contaminant in the Final Effluent sampled or measured during a calendar month. (24-hour composite, weekly)
Total Phosphorus	Monthly average effluent concentration is the arithmetic mean of all Single Sample Results of the concentration of a contaminant in the Final Effluent sampled or measured during a calendar month. (24-hour composite, weekly)
pH	Single sample result means the test result of a parameter in the effluent discharged on any day, as measured by a probe, or grab sample.

4.0 ENVIRONMENTAL AND LAND USE CONSIDERATIONS

4.1 Adjacent Property Uses

The Markdale Wastewater Treatment Plant is owned and operated by the Municipality of Grey Highlands. Located at 775342 Highway 10, Markdale, Ontario (Lots 94 to 96, Concession 1), the land use surrounding the WWTP is categorized mostly as rural, agricultural, and hazard lands according to Schedule A of the Grey County Official Plan. The Municipality of West Grey zones the site as institutional use. The property borders lands designated as natural environment, due to the Rocky Saugeen River, and rural. The latter rural land has been purchased for development.

The Grey County CP Rail trail runs through the facility, southeast of the cells, and northwest of the RIBs. The 77 km long gravel trail is used by pedestrians, bikes, ATVs, and equestrians. Provincial Highway 10 is located within 500 m of the facility, to the Northeast. Figure 4 shows the adjacent land use and Figure 5 shows natural environmental features surrounding the WWTP.

The nearest development to the WWTP is the Chapman's Ice Cream facility. The remaining adjacent property is undeveloped. However, the farmland Northeast of the facility has been purchased by the factory.

The Markdale WWTP has received no odour complaints documented in the Annual Reports after 2014. In 2011, after the opening of the newly expanded Chapman's facility, municipal staff report that there were odour complaints at the factory and in Cell No. 1 of the lagoon. This led to the expansion of the Chapman's facility's pre-treatment system and the implementation of in-situ floating aerators (Blue Frogs) in the lagoons.

4.2 Source Water Protection

Ontario's Clean Water Act provides the mandate for a provincial drinking water source protection program in Ontario. Its focus is on the protection of water sources for municipal drinking water systems, with additional attention to surface water and groundwater sources on the broader landscape.

The Saugeen Valley Conservation Authority (SVCA) adopted the Saugeen, Grey Sauble, Northern Bruce Peninsula Source Protection Plan in July 2016, and it was approved by the Ministry of the Environment and Climate Change in October 2015.

According to Map 5.1 of the Source Protection Plan, the Markdale WWTP lies approximately 270 m west of the Town's well head protection areas (WHPAs) which include most of the Town and the Chapman's Ice Cream facility.

Figure 6 shows the Well Head Protection Areas in the town of Markdale.

The Markdale Water System is a Class 1 Treatment and Class 1 Distribution System consisting of three drilled wells and two pumping stations which pump directly into the distribution system and elevated standpipe. Water is supplied from a bedrock aquifer.

The location of vulnerable areas, to protect the source water for municipal residential drinking water systems, will need to be considered in subsequent stages of this MCEA and during project implementation.

4.3 Surface Water

All effluent from the RIBs eventually enters the Rocky Saugeen River after travelling in the subsurface. As noted in Section 3.2.3, three surface water quality monitoring stations are located on the Rocky Saugeen River: upstream of the effluent infiltration area, in the river facing the infiltration system area, and downstream of the effluent infiltration area at the footbridge. These stations measure Total Phosphorus, Dissolved Reactive Phosphorus, Total Kjeldahl Nitrogen, Ammonia Nitrogen, Nitrite, Nitrate, Chloride, and Dissolved Organic Carbon. They are sampled on a quarterly basis.

The Rocky Saugeen River is located East of the WWTP. Its closest approach is approximately 50 m Southeast of the RIBs. Impacts to the Rocky Saugeen and mitigation measure to protect watercourse from construction or operational activities (e.g., spills, erosion, pollution) are identified as part of the proposed undertaking.

The WWTP is not listed in Table 1 of OWRA O Reg 60/08.

After consulting the Fisheries and Oceans Canada guide it was determined that this project does not need a review. The project will implement the measures to protect fish and fish habitat in the case construction work is needed, such as the installation of an outfall.

After consultation with the MECP, it was determined that no surface water impact assessment will be required, and the existing effluent limits and objectives do not need to be revisited. Refer to Section 8.4 for more details.

4.4 Groundwater

As noted in Section 3.2.3, six groundwater observations wells are located along the predicted flow path of effluent from the RIBs. Two monitoring stations monitor effluent seepage at the surface. Data is also collected from a well located between the RIBs and Cell No. 2 and a well located on the banks of the Rocky Saugeen River.

These stations measure Total Phosphorus, Dissolved Reactive Phosphorus, Total Kjeldahl Nitrogen, Ammonia Nitrogen, Nitrite, Nitrate, Chloride, and Dissolved Organic Carbon. They are sampled on a quarterly basis. Additionally, groundwater measurements and water table elevations are measured once a month.

There are three Permits to Take Water (PTTW) from ground sources, for communal water use owned by the Municipality, within two kilometres of the WWTP.

There are many more observation wells in the area that are not monitored.

Refer to Section 4.2 above for a discussion of Source Water Protection considerations.

During construction, a Permit to Take Water (PTTW) under the OWRA will be required for any water takings that exceed 50,000 L/day, except for activities prescribed by the Water Taking EASR Regulation O. Reg. 63/16. The need for this will be confirmed by studies in the design phase.

4.5 Geology and Hydrogeology

A 2003 Groundwater study of the Grey Highlands area indicated that bedrock in the Markdale site is dolostone, part of the Guelph Formation. Markdale's municipal wells extend into the Guelph, Amabel, Fossil Hill, Cabbot Head, and Queenston formations, which comprise of dolostone and shale. These wells are supplied from fractured bedrock aquifers. Quaternary geology at the site is drumlinized till, specifically the Saugeen Kames, which consist of glaciofluvial ice-contact sand and gravel.

A desktop hydrogeological assessment was completed by WSP in July 2023 and is summarized in Section 6.7.3. Field work was subsequently completed by WSP in 2025, with a hydrogeological assessment report issued in 2026. This report is summarized in Section 10.3.1.

The MECP stated that future detailed hydrogeological assessment should confirm that effluent entering shallow groundwater from RIBs will not pass underneath the river. Investigations should be completed prior to any construction to verify site specific conditions.

4.6 Natural Heritage and Species at Risk

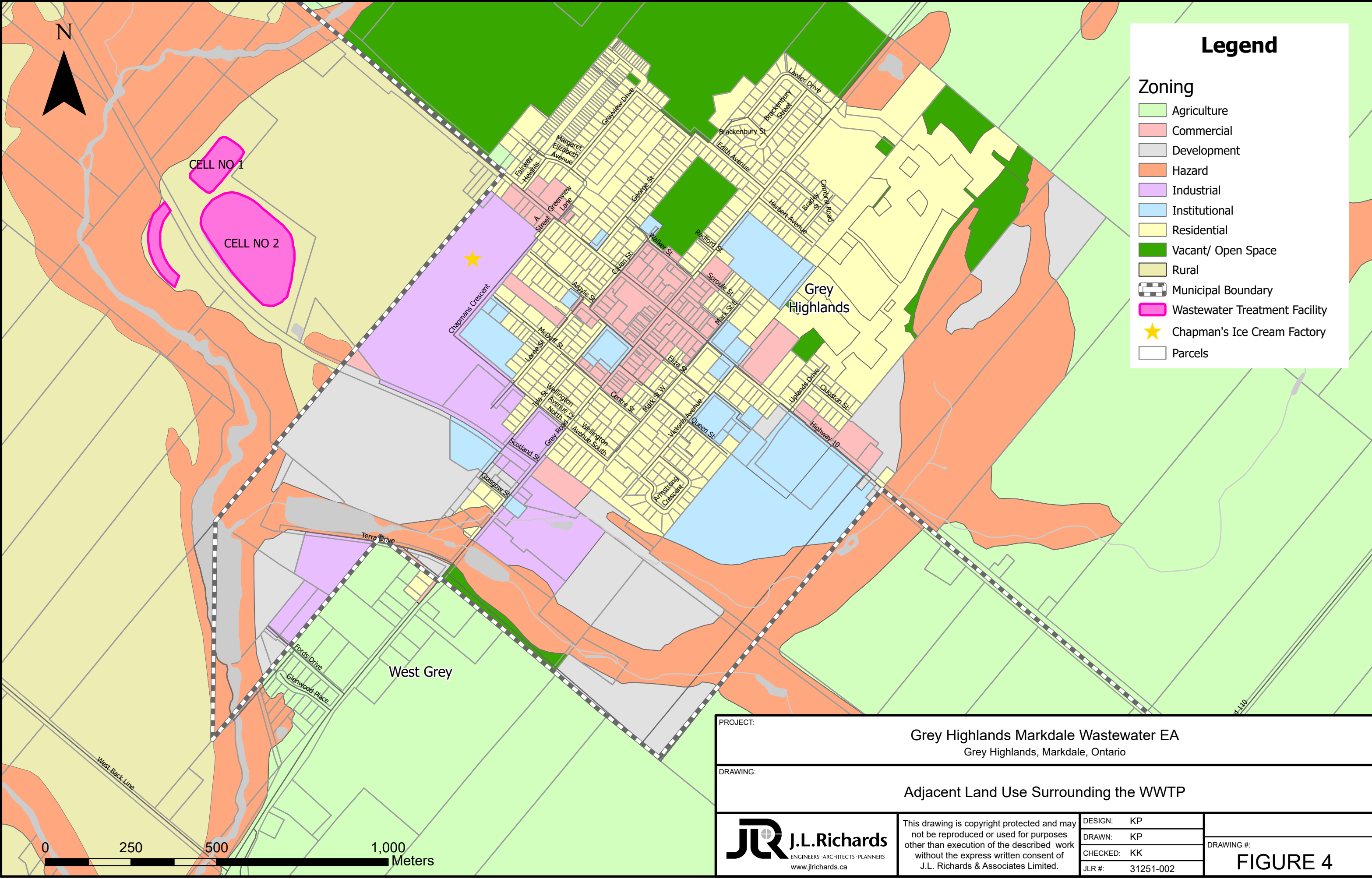
WSP (2017) completed a Species at Risk search on the Environment Canada SARA website and the Ontario SARO website for the proposed servicing phases in the Municipality of Grey Highlands Water & Wastewater Servicing Master Plan published in 2017. These searches revealed a total of 26 species at risk found in the proposed servicing phases. They include 12 birds, seven reptiles, two fish, three mammals, two arthropods, and five vascular plants. Of these, 12 species found in Grey County are listed as Endangered.

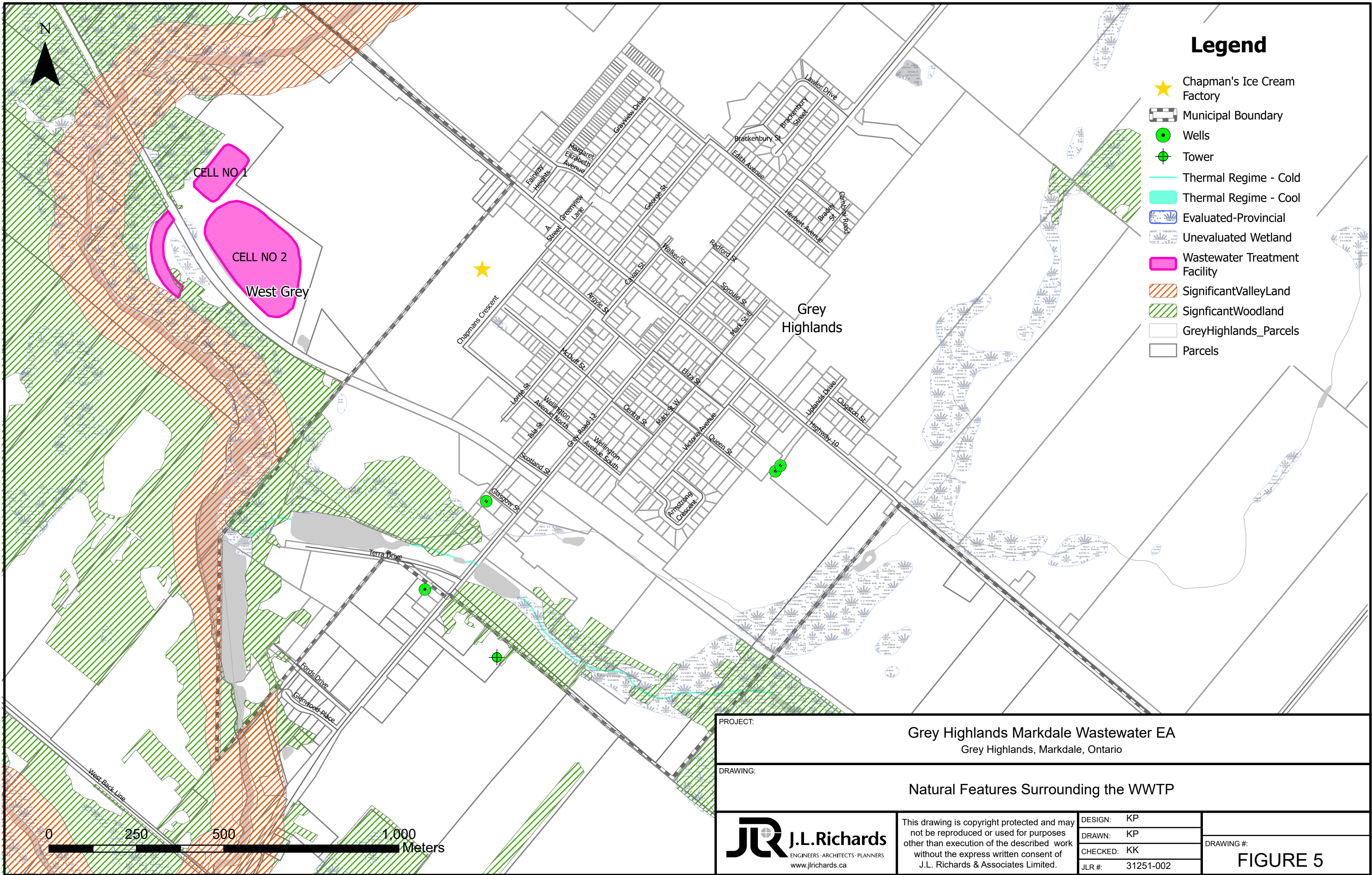
According to Schedule C of the Grey County Official Plan, the study area does not contain any Natural Heritage System Core Areas or Linkages. Appendix B Map 3 of the Plan shows the WWTP is not located in areas of natural and scientific interest (ANSIs) related to Earth or Life. The study area does not include Significant Valleylands. However, the RIBs border and have slight overlap over Significant Woodlands.

In 2024, Pinchin Ltd. (Pinchin) was retained to complete field studies and prepare a Terrestrial Ecology Report (TER) for the study area. This report is provided in Appendix A of this report. Land classifications of the study area (environmental constraints) are illustrated in Figure 7 (Figure 4 of the TER).

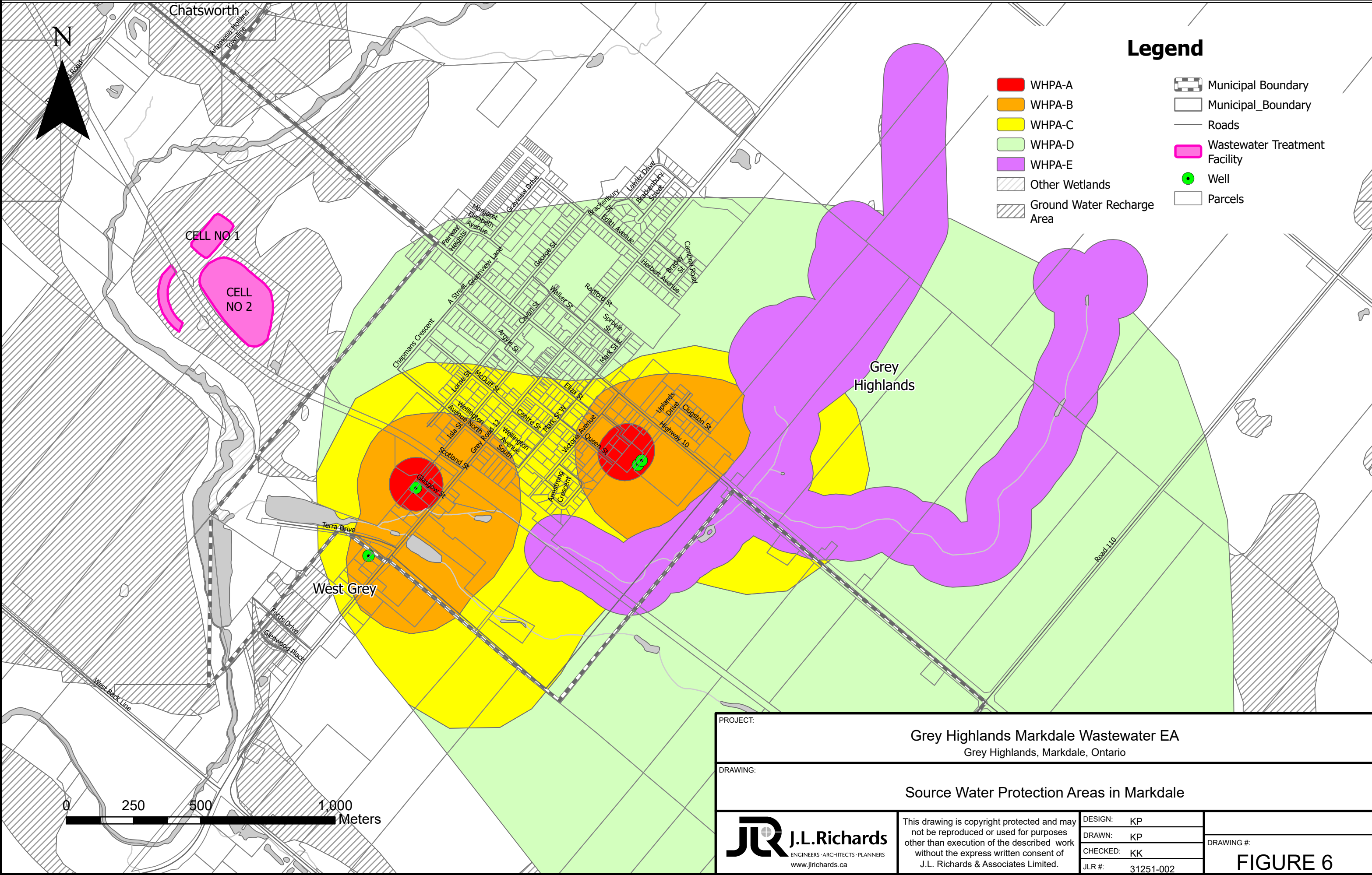
Significant Wildlife Habitat (SWH) was identified on site. Chimney Swift, Bobolinks, and Eastern Meadowlarks were heard and observed foraging. Candidate SWH was identified on site for Bat Maternity Colonies, Amphibian Breeding Habitat, and Turtle Nesting Areas. The TER's recommended mitigation measures to address all potential impacts include the following:

- Conduct breeding amphibian surveys, snag surveys, and turtle basking surveys prior to development
- A leaf off/on survey in areas of tree removal, followed by a Chimney Swift survey if snag trees are to be removed.
- Consultation with the MECP if the meadow on the site will be developed, to assess impacts on Eastern Meadowlarks and Bobolinks
- Implement a 15 m setback with exclusion fencing for the wetlands, where possible, to eliminate impacts.
- Implement a 5 m setback with exclusion fencing for the woodlands, where possible, to eliminate impacts.
- Restoration planting at an off-site location to compensate for encroachment as identified in a Landscape Plan. Consult with the Conservation Authority prior to development if the wetland is encroached.
- Remove vegetation within the site outside of the breeding periods for bird and bat species (April 1 to September 30), or deploy a qualified Biologist to conduct bird nest and bat roost surveys prior to tree removal.
- Develop an Erosion and Sediment Control Plan, establish erosion and sediment control measures
- Implement wildlife and Species at Risk encounter protocols during construction
- Develop and implement a monitoring plan for restored areas.





File Location: \\jrichards\Corp\Projects\31000\31251-002 - Grey Highlands Markdale Wastewater EA\3-Production\1-Civil\GIS\31251-002 - Grey Highlands Markdale Wastewater EA.aprx



Plot Date: August 23, 2023 9:11 AM



Ecological Land Classification	
1. White Cedar Mineral Coniferous Swamp (SWCM1)	
2. Dry – Fresh White Cedar Coniferous Forest (FOCM2-2)	
3. Willow Mineral Deciduous Swamp (SWDM4-1)	
4. Balsam Poplar Mineral Deciduous Swamp (SWDM4)	
5. Grey County CP Rail Trail (CVI_1)	
6. Dry - Fresh Mixed Meadow (MEMM3)	
7. Sewage and Water Treatment (CVI_3)	

Service Layer Credits: MECP, MNRF, NHIC, 2024.
World Imagery: Maxar



PROJECT NAME:	Terrestrial Ecology Report
CLIENT NAME:	J.L. Richards & Associated Limited
PROJECT LOCATION:	Vacant Lot West of Highway 10, Markdale, Ontario
FIGURE NAME:	Environmental Constraints

PROJECT NO.	339761.000
DATE:	Nov. 2024
SCALE:	1:4,800
FIGURE NO.	4

LEGEND	
Site Boundary	Recommended Setback Area
Roadway	- Woodland #2 (5 m)
Watercourse	- Wetlands #1, 3, 4 (15 m)
NOTE: Imagery was taken in 2014 and may not reflect current conditions.	

NOTES
1. All features and measurements are approximate and subject to field verification. This map is for planning purposes only.
2. Use dimensions as shown, do not scale drawing.
3. This map is not to be used for legal purposes.

Coordinate System: NAD 1983 CSRS UTM Zone 17N Projection: Transverse Mercator Datum: North American 1983 CSRS		
DRAWN BY: MH	REVIEWED BY: KH	REVISION: 1

4.7 Cultural Environment and Archaeological Resources

Markdale was founded in 1846, and a water works system was in place by 1905. The village serves the needs of a large agricultural community as well as artists and tourists. The land falls under the 1836 Treaty 45 1/2 with the Saugeen Ojibway Nations.

WSP previously completed a Ministry of Culture, Tourism, and Sport (MCTS) Criteria for Evaluating Potential for Built Heritage Resources and Cultural Heritage Landscapes checklist for the existing Markdale WWTP, during the Municipality of Grey Highlands Water & Wastewater Servicing Master Plan in 2017. The 2017 report found that the Heritage Impact Assessment was not required as the property had low potential for built heritage or cultural heritage landscape. WSP completed a MCTS Criteria for Evaluating Archaeological Potential checklist for the existing Markdale WWTP during the 2017 Master Plan. It found that the associated evaluation checklist did not need to be completed as the property had been previously disturbed. The 2017 Master Plan noted that future projects require consultation with the Saugeen Ojibway Nation regarding Stage 1 Archaeological Assessments.

For this Class EA, JLR completed the screening forms provided by the Ministry of Citizenship and Multiculturalism (MCM), in consultation with Archaeology Ontario:

- Criteria for Evaluating Archaeological Potential
- Criteria for Evaluating Potential for Built Heritage Resources and Cultural Heritage Landscapes
- Criteria for Evaluating Marine Archaeological Potential

The screening process determined that a Stage 1 Assessment was required on the undisturbed portions of the property.

Consequently, in 2024 Archaeological Research Associates Ltd. (ARA) was retained to complete a Stage 1 Archaeological Assessment (AA) of the study area. This study identified that portions of the study area had archaeological potential, and these areas should be subject to a Stage 2 AA.

Per this recommendation, ARA was retained to complete the Stage 2 AA for the identified areas. The Stage 2 AA did not identify any archaeological materials, and the project area was determined to be clear. Indigenous communities including the Saugeen Ojibway Nation were consulted by ARA as part of the AA process, as documented in their reports.

Refer to Appendix B of this report for the completed screening forms, Stage 1 AA report, and Stage 2 AA report.

4.8 Climate Change

The Considering Climate Change in the Environmental Assessment Process Guide (2017) sets out the MECP's expectation for considering climate change in the preparation, execution and documentation of environmental assessment studies and processes.

Methods to reduce the impact of the WWTP on the climate and increase the resiliency of the WWTP, including existing features, will be considered in the evaluation (Section 11).

4.8.1 Climate Change Mitigation

Climate change mitigation refers to the project's expected production of greenhouse gas emissions and impacts on carbon sinks. These include any greenhouse gas emission from a project, or landscape change that affects the removal or storage of carbon dioxide from the atmosphere.

Wastewater treatment in lagoons is driven by natural physical and biological processes. These may have lesser climate impacts compared to facilities that utilize higher inputs of chemicals and energy. Upgrades and expansions of the WWTP may contribute to climate change through the production of greenhouse gas emissions such as GHG emissions from heavy vehicles during construction or biological waste produced by the facility. Other impacts of the WWTP on climate change include impacts on carbon sinks from construction impacting the landscape's ability to remove and store atmospheric carbon.

4.8.2 Climate Change Adaptation

Climate change adaptation refers to the resilience or vulnerability of the WWTP to changing climatic conditions. Impacts of climate change on projects include property-specific concerns to system-wide impacts.

This system of lagoons may be more vulnerable to climate change compared to facilities sheltered from the local climate. The WWTP may experience negative impacts to functionality and reliability due to changing climatic conditions. These include drought, flooding, sewer overflow, and ice storm damage.

4.8.3 Grey County Climate Change Action Plan

The Grey County Climate Change Action Plan was published in 2022 following the update of the 2019 County of Grey Official Plan. It notes that 1% of the GHGs emitted by biogenic sources came from the decomposition of waste in wastewater. These GHGs include CH₄ and NO₂. Energy consumption by wastewater treatment facilities is not calculated separately from other municipal energy use in buildings. The Plan also lists strategies for improving energy efficiency at wastewater facilities, including operator training on best practices, resource sharing between municipalities, and installing new biodigesters at WWTPs.

5.0 EXISTING WASTEWATER TREATMENT PLANT PERFORMANCE

5.1 Historical WWTP Flow Data

Annual reports and daily flow data for the WWTP were reviewed for 2017 to 2022. Influent flow is measured at the Parshall flume on the raw sewage line. As summarized in Table 5-1, the WWTP is currently operating at approximately half of its rated average day flow capacity.

Table 5-1: WWTP Annual Flow Summary Table (2017-2022)

Year	Annual Average Day Flow (m ³ /day)	Annual Maximum Day Flow (m ³ /day)	Max Day Factor
2017	855	2,328	2.7
2018	882	4,119	4.7
2019	917	1,957	2.1
2020	755	2,055	2.7
2021	766	2,907	3.8
2022	857	1,932	2.3
Average (m ³ /day)	839	2,550	3.0
Average (L/s)	9.71	29.51	
Rated Capacity	1,722 m ³ /day		

The average day flow to the WWTP is 839 m³/day, which is approximately 49% of the plant's rated capacity of 1,722 m³/day. The maximum flow seen at the WWTP was in February of 2018 due to a snow melt, which led to stop boards being pulled between Cell No. 1 and Cell No. 2.

The average maximum day flow is 2,550 m³/day. The maximum day peaking factor was calculated using the average maximum day flow, and the five-year average of the average day flows. The factor was determined to be 3.04.

5.2 Existing Per Capita Flows

5.2.1 Review of 2022 Water Billing Records

Table 5-2 summarizes the water consumption data provided by the Municipality for the year 2022.

Table 5-2: Summary of 2022 Water Billing Records

Demand Type	Total Consumption in 2022 (m³)
Commercial	11,269
Industrial ⁽¹⁾	22,351
Institutional	2,846
Residential	90,334
Chapman's Ice Cream ⁽²⁾	224,654
Total	351,454

Table 5-2 Notes:

- (1) Industrial flows excluding the Chapman's Ice Cream facility.
- (2) Only industrial flows. Residential flows from lands owned by Chapman's Ice Cream (i.e., lands purchased for expansion but still house residents) are included in the residential consumption.

The Chapman's Ice Cream facility comprises over 60% of Markdale's water demand. If this contribution is excluded, residential demand makes up over 70% of Markdale's remaining water demand.

5.2.2 Review of Industrial Water and Wastewater Flow Contribution

Chapman's Ice Cream was founded in 1973 and established in an old creamery. During construction work in 2009, a fire started and destroyed the production facility. The new Phoenix facility was rebuilt in 2010 and has 160,000 sq ft of production space. This is in addition to a distribution centre that has 150,000 sq ft of storage space (*The Inside Scoop on Our History*, Chapman's Ice Cream).

In 2000, Chapman's built a wastewater treatment plant on site that purifies the 800,000 L of water it uses in the cleaning and production process (*Chapman's Commitment to a Greener Future*, Chapman's Ice Cream).

Chapman's utilized a Fluidyne Integrated Surge Anoxic Mix (ISAM) anaerobic/aerobic batch reactor. The ISAM reactor consists of a surge/anoxic mix tank, jet aerators, and a PLC in the control panel that adjusts aeration and mixing to incoming loading rates. The reactor does not allow flow in during effluent discharge periods to maintain effluent quality. The design effluent for the ISAM reactor 50 mg/L for both BOD₅ and TSS, which were more stringent than the regulatory requirements at the time. In 2002 the effluent BOD₅ concentration was consistently under 10 mg/L (Mandt & Graves, 2002).

A reverse osmosis system was added to the wastewater treatment system in the 2010s. The facility can reuse water, has rooftop containers, and heats water with excess heat from the engine rooms. The facility is exploring avenues for co-generation to reduce energy consumption (*Chapman's Commitment to a Greener Future*, Chapman's Ice Cream).

Production at the factory fluctuates on a daily and seasonal basis. The Municipality has reported that typically the facility has multiple shifts throughout the week and has limited production on the weekends. Production is highest in the months between April and September and decreases in the colder months. As of 2019, 700 full time and 150 seasonal workers were employed at the facility (Chapman's Ice Cream, 2019).

There is limited data available to confirm industrial flow contribution. A summary of available data is in Table 5-3 to Table 5-5.

Table 5-3: Available Domestic and Industrial Flow Data

Data Type	Collection Frequency	Parameters
Chapman's water meter	Monthly	Water consumption (m ³)
Chapman's production wastewater flow	Daily (collected with weekly samples)	Volume produced (m ³)
Wastewater produced domestically by Chapman's (i.e., employee contribution)	Not available	
Markdale WWTP operational data	Daily (manually collected by operators)	Blower pressures, alum tank level, flows (m ³)
Markdale WWTP sampling	Monthly	BOD ₅ , CBOD ₅ , TSS, Total Phosphorus, TKN, ammonia, ammonium, field pH and temperature for unionized ammonia calculations

Table 5-4: Historical Water Demand from the Chapman's facility

Year	Consumption (Isla St.)	Consumption (Chapman's Crescent)	Total "Customers" (1)	Total Water Consumption (m ³)
2018	1,400	202,095	8	203,495
2019	512	213,111	8	213,623
2020	686	188,327	10	189,013
2021	131	208,747	8	208,878
2022	116	224,535	11	224,654
Average				209,042

Table 5-4 Notes:

- (1) Industrial water meter data for the Chapman's facility is measured at the Isla St and Chapman's Crescent watermains. Six of these points are located at Chapman's Crescent.
- (2) Residential water demand from lands owned by Chapman's is not included.

Table 5-5 summarizes the process wastewater flows produced by the Chapman's facility from 2017 to 2022. Wastewater flows from Chapman's are measured daily by the Municipality when they collect a sample every week. This flow is comprised of the wastewater from the factory's production processes ("process flow"). It does not include flows produced domestically by employees at the plant ("employee flow").

Table 5-5: Process Wastewater Flows from the Chapman's Facility

Month	Total Flow (m ³)						
	2017	2018	2019	2020	2021	2022	Average
January	6,814	6,648	9,572	7,444	8,085	9,176	7,957
February	6,931	7,952	8,452	8,400	9,220	10,185	8,523
March	9,247	8,238	7,653	6,112	10,702	10,362	8,719
April	8,471	7,225	9,035	5,828	9,649	9,106	8,219
May	8,935	8,569	11,638	9,078	10,034	8,701	9,493
June	9,561	8,692	9,922	9,195	8,927	9,292	9,265
July	8,028	9,944	13,608	9,931	8,826	8,081	9,736
August	7,171	11,530	11,555	8,785	8,971	9,371	9,564
September	5,720	7,491	8,152	8,022	8,404	9,316	7,851
October	5,977	8,945	9,494	8,376	7,548	8,751	8,182
November	6,031	9,351	9,571	8,334	8,639	9,240	8,528
December	5,545	6,240	7,135	6,855	5,395	8,618	6,631
Total Annual Flow	88,431	100,823	115,788	96,361	104,399	110,200	102,667

The Chapman's facility produces an average of 102,667 m³ of wastewater per year. The lowest production year was the COVID-19 pandemic shutdown year of 2020. 2022 flows indicate the factory is returning to pre-pandemic levels of consumption.

On average, the Chapman's produces the highest flows in the summer months of May to August. Peaks are centered in March during 2017, 2021, and 2022, while they are centered in July/August during 2018, 2019, and 2020. The lowest flows are produced in December, excluding the spring pandemic shutdown in 2020.

5.2.3 Equivalent Populations

The Markdale WWTP services a population of 1200 residents. An equivalent population is needed to determine future commercial, industrial, and institutional flow contributions.

To determine equivalent populations for Industrial, Commercial, and Institutional (ICI) contributors, the 2022 residential water demand was used. Water data was used instead of wastewater data as flows were available for each meter, where each meter has an assigned land use.

Thus, the 2022 residential water demand, as seen in Table 5-2, was divided by the Markdale service population of 1200 users, to find the residential per capita water demand of 76.2 m³/year or 209 L/d.

Then, the water demand from ICI users, excluding Chapman's, was totaled. This total ICI water demand was divided by the per capita water demand of 209 L/d to determine the equivalent ICI populations listed in Table 5-6.

As noted in the previous section, the Chapman's facility employs 700 full time and 150 seasonal workers. Assuming a 40-hour work week and approximately 100 waking hours per week, a full-time employee is equivalent to 40% of a resident. Using the conservative assumption that this 40% equivalent also applies to part-time employees, the 850 Chapman's employees are equivalent to 340 full time residents.

Table 5-6 summarizes the equivalent populations calculated above. The total equivalent population is 824 people. When combined with the serviced residential population of 1,200 means this results in a total serviced population equivalent to 2,024 people.

Table 5-6: Equivalent Population based on 2022 Water Consumption

Demand Type	People
Industrial ⁽¹⁾	297
Commercial	150
Institutional	38
Chapman's ⁽²⁾	340
Equivalent Population Subtotal	824
Serviced Residents	1,200
Equivalent Serviced Population	2,024

Table 5-6 Notes:

- (1) Industrial flows excluding the Chapman's Ice Cream facility.
- (2) Only industrial flows.

The equivalent serviced population of 2,024 people was carried forward to calculate a per capita wastewater flow and future design flows.

5.2.4 Review of Design Per Capita Wastewater Flows

5.2.4.1 Historical Per Capita Wastewater Flows

To determine the per capita wastewater flow, the total annual industrial wastewater flows produced by Chapman's operations (refer to Table 5-5) were subtracted from the total annual wastewater flows received by the WWTP (refer to Table 5-1). Note that wastewater produced domestically by Chapman's employees, i.e., "employee flow", is not metered and the flow used in this calculation is generated purely by production processes, i.e., "process flow". The remaining flow was divided by the equivalent service population of 2,024 to determine the per capita wastewater flow for each year. The results are listed in Table 5-7.

Table 5-7: Daily Wastewater Flows at the Markdale WWTP (2017 – 2022)

Wastewater Flow (m³/day)	2017	2018	2019	2020	2021	2022	Average
Average Day	855	882	917	755	766	857	839
Chapman's	242	276	317	264	286	302	281
Remaining	613	606	600	491	480	555	557
Per Capita ⁽¹⁾	0.303	0.299	0.296	0.243	0.237	0.274	0.275

Table 5-7 Notes:

- (1) Uses a service population of 2,024, including 1,200 residents and the equivalent populations listed in Table 5-6.

The pandemic years of 2020 and 2021 have the lowest per capita wastewater flow. The flow in 2022 indicates an upward trend returning to higher pre-pandemic levels, which average 299 L/d per capita (2017-2019). These calculations assume the number of employees at Chapman's has remained the same since 2019, which is unlikely due to the pandemic and is shown by the low flows in 2020 seen in Table 5-5. Therefore, the per capita wastewater flow is likely higher than the reported average of 275 L/day and the pre-pandemic average of 299 L/d.

Therefore, the per capita historical wastewater flow is estimated to be 300 L/capita/d.

5.2.4.2 Previous Per Capita Design Wastewater Flows

In the 2017 Master Plan, WSP assumed a wastewater flow of 275 L/day per capita, assuming 2.2 persons per unit, for residential and Chapman's facility design flows. A wastewater flow of 450 L/day per capita was used for remaining usage types such as hospitals, schools, and businesses. The per capita daily wastewater flows were multiplied by the estimated number of equivalent residential units, to calculate the estimated sewage flow. For future projections they assumed a 1% increase in equivalent residential units per year for 21-50 years.

The Municipality is currently working to update growth projections using a density of 2.5 persons per unit. Their wastewater projections are 1.1 m³/day of wastewater produced per unit, which comes from multiplying the density by a per capita flow of 450 L/day and rounding. Due to rounding, the per unit wastewater flow is equivalent to a per capita flow of 440 L/day.

The Ontario Design Guidelines for Sewage Works recommend an average domestic wastewater flow rate between 225 to 450 L/day per capita.

Table 5-8: Per Capita Design Wastewater Flows used in Design.

Design Document	Year	Design Per Capita Wastewater Flow (L/day)
Master Plan, WSP	2017	275 (residential and Chapman's) 450 (other ICI)
Growth projections, Municipality	2022	440
Ontario Design Guidelines for Sewage Works	2008	225 to 450

Table 5-8 summarizes the historical design wastewater flows described above. As seen in this table, design flows range from 225 to 450 L/d per capita.

The design rates exceeding 400 L/d per capita may overestimate residential wastewater production. Residential flows account for most of the equivalent serviced population, excluding Chapman's process flows, as seen in Table 5-6. The WSP rate also assumes a lower population density per unit, which inflates their per capita flow.

The design rates below the historical rate of 300 L/d per capita are specifically for residential flows, while the historical rate includes ICI contributions.

Therefore, the design flow rate should be within the historical rate of 300 L/d per capita and 400 L/d per capita. To maintain a conservative estimate due to the uncertainty of the Chapman's expansion and the impact of the pandemic, a design wastewater flow rate of 375 L/d per capita is recommended.

5.3 Historical Influent Quality

At the WWTP, 24-hour composite samples are analyzed weekly for Biochemical Oxygen Demand (BOD₅), Total Phosphorus (TP), Total Suspended Solids (TSS), Total Kjeldahl Nitrogen (TKN), and Ammonia. pH and temperature are also measured weekly. The influent quality from these sampling events is summarized in Table 5-9.

Table 5-9: Summary of Wastewater Influent Quality Parameters ⁽¹⁾

	BOD₅ (mg/L)	Total Phosphorus (mg/L)	Total Suspended Solids (mg/L)	Total Kjeldahl Nitrogen (mg/L)	Ammonia (mg/L)
2017	143	3.66	154	24.5	19.6
2018	143	3.16	138	25.5	19.9
2019	119	3.34	138	24.8	21.4
2020	143	3.29	161	27.1	7.2
2021	134	4.99	162	28.3	19.3
2022	124	5.42	140	26.5	22.3
Average	134.3	4.0	148.8	26.1	18.3
Typical Wastewater Strength ⁽²⁾	133 (Low) 200 (Med) 400 (High)	3.7 (Low) 5.6 (Med) 11.0 (High)	130 (Low) 195 (Med) 389 (High)	23 (Low) 35 (Med) 69 (High)	--
Wastewater Strength	Low Medium	Low Medium	Low Medium	Low Medium	--

Table 5-9 Notes:

(1) Data from Markdale Wastewater Annual Reports (WSP, 2017 – 2020) (Douglas Water Services, 2021 – 2022).

(2) Typical composition of untreated domestic wastewater (Metcalf and Eddy, 2014).

The results indicate that the average concentration of BOD₅, TP, TSS, and TKN in the influent wastewater are in the lower end of literature values for medium strength untreated domestic wastewater.

5.4 Historical Treated Effluent Quality

Samples of the final effluent are taken at the end of Cell No. 2 before entering the RIBs. The 24-hr composite samples are analyzed weekly for CBOD₅, TSS, TP, and TKN. Samples for pH and temperature were measured in house. The effluent quality is summarized in Table 5-10. A summary of the percent removal of BOD₅, total phosphorus and total suspended solids for final effluent is included in Table 5-11.

Table 5-10: Summary of Effluent Water Quality Parameters

	CBOD ₅ (mg/L)	TP (mg/L)	TSS (mg/L)	pH
2017	4.2	0.16	3.3	7.91
2018	4.4	0.09	2.8	7.98
2019	5.6	0.1	6.5	8.11
2020	5.4	0.09	5.4	7.72
2021	4.9	0.07	5.3	8.06
2022	5.3	0.17	7.2	8.01
Average	5.0	0.11	5.1	7.96
<i>Objective</i>	20	0.5	25	6.5 to 8.5
<i>Limit</i>	25	0.65	30	6 to 9.5

Table 5-10 Notes:

- (1) Data from Markdale Wastewater Annual Reports (WSP, 2017 – 2020) (Douglas Water Services, 2021 – 2022).

Table 5-11: Summary of Effluent Water Quality Percent Removal

	CBOD ₅	TP	TSS
2017	97	96	98
2018	97	97	98
2019	95	97	95
2020	96	97	97
2021	96	99	97
2022	96	97	95
Average	96	97	97

The effluent water quality has been well within ECA objectives for CBOD₅, TP, TSS, and pH over the five years since the facility was re-rated in 2017.

5.5 Imported Sewage and Septage

The WWTP receives sewage from septage, portable toilets, and holding tanks. Imported sewage was tracked starting in 2018 after the new ECA was issued. The ECA requires a grab sample every month for BOD₅, TSS, TP, and TKN.

Imported sewage data is available only for 2021 and 2022, which are summarized in Table 5-12 and Table 5-13.

Table 5-12: Imported Sewage Characterization (2021-2022)

	Concentration (mg/L)				Loading (kg/day)		
	2021	2022	Average	EPA Average	2021	2022	Average
BOD₅	4,632.5	3,819.0	4,226	6,480	381.5	418.0	400
TSS	6,696.7	6,176.3	6,437	12,862	586.3	855.8	721
TP	134.6	173.8	154	210	11.3	20.6	16

The average concentration of the quality parameters in the imported sewage were compared to the typical values taken from the EPA Guide to Septage Treatment and Disposal (1994) and found to be below the average values.

Table 5-13: Imported Sewage Volumes (2021-2022)

Sewage Volume (m ³)	2021	2022	Average
Total Imported	1,209.3	1,458.4	1,333.9
Daily Average	3.3	4.0	3.7
Maximum Monthly Total	225.0 (Dec)	298.7 (Mar)	261.9

Most of the imported sewage came from emptied toilets. In 2021, their contribution was 70% of imported sewage and in 2022, it was 61%.

5.6 Biosolids and De-sludging

Biosolids are typically contained within the lagoons until they are de-sludged. In the period between 2017 and 2021, sludge was removed once during June 26 – July 20, 2017. At that time, 200 bone dry tonnes of sludge were removed from Cell No. 2 by GeoDredge. At the time the sludge remained onsite where dried in GeoTubes located along the Lagoon Bank.

Milne Technologies conducted a sonar mapping survey of sludge accumulation in Cell No. 1 in September 2019 and in Cell No. 2 in September 2020. Both were previously surveyed in 2014.

6.0 WWTP CAPACITY ASSESSMENT

6.1 WWTP Overview

The Markdale WWTP consists of two aerated facultative lagoons in series. Cell No. 1 has been further partitioned with a submerged concrete wall into two distinct treatment areas. The upstream third of the volume acts as a sedimentation basin to decrease the overall load. The remaining two thirds of the cell is aerated. The upstream area has six Blue Frog systems in place for mixing to maintain the facultative conditions within this cell. Floating vegetative cells, added for nutrient management, cover 150 m² of Cell No. 1, or 1% of the area. The downstream area of Cell No. 1 is aerated using fine bubble diffusers.

Cell No. 2 is an aerated facultative lagoon that provides settling for the chemical precipitation of phosphorus (alum dosing), nitrification, and storage prior to discharge from the WWTP.

The discharge from Cell No. 2 is directed into eight Rapid Infiltration Basins (RIBs) which allow the effluent wastewater to infiltrate the subsurface unsaturated zone. The unsaturated zone provides additional polishing. The polished effluent enters the groundwater, where it is diluted and either seeps at the surface where it flows over land to the Rocky Saugeen River or remains in the groundwater and discharges as a diffuse seep into the Rocky Saugeen River.

Table 6-1 provides the current flow and concentration data as measured at the WWTP in 2021.

Table 6-1: Flow and Concentration Data reported for the Markdale WWTP (2021)

Parameter		Raw Wastewater	Cell No. 1 Effluent (Cell No. 2 Influent)	Cell No. 2 Effluent (Discharge)	Discharge Objective
Flow (m ³ /d)	Maximum Average Minimum	1213 766 482		2423 ⁽¹⁾ 719 0	
BOD ₅ ⁽²⁾ (mg/L)	Maximum Average Minimum	288 134 54	38 16 5	18 ⁽³⁾ 7 4	20
TSS (mg/L)	Maximum Average Minimum	391 159 32	70 33 4	38 ⁽⁴⁾ 6 2	30
TKN (mg/L)	Maximum Average Minimum	52.5 28.3 14.0	22.8 13.2 1.5	14.5 4.9 0.6	
TP (mg/L)	Maximum Average Minimum	9.1 5.0 1.0	2.0 1.1 0.5	0.2 0.1 0.0	0.65

Table 6-1 Notes:

- (1) Discharge controlled by RIBs, not always consistent.
- (2) Influent and Cell No. 1 Effluent reported as BOD₅, Cell No. 2 Effluent reported as CBOD₅, it is assumed due to the nature of the wastewater being sampled that limited nitrification will occur during the 5-day BOD tests and therefore all analyses can be assumed to be CBOD₅.
- (3) Single anomalous reading. Removing this outlier reduced maximum effluent concentration to 10 mg/L
- (4) Single anomalous reading. Removing this outlier reduced maximum effluent concentration to 11 mg/L

6.2 Hydraulics

6.2.1 Current Design and Operation

The minimum residence time listed in the Ontario Design Guidelines for Sewage Works for aerated facultative lagoons is 4 to 5 days. The minimum residence time for a WWTP consisting of an aerated facultative lagoon followed by a conventional lagoon is a total of 30 days. Average wastewater flow rates in 2021 were 766 m³/day. The residence times at both the current average daily flow (766 m³/day) and the design daily flow (1722 m³/day) are presented in Table 6-2. Both the 2021 and rated flow rates maintain residence times conforming to the Ontario Guidelines.

Table 6-2: Lagoon Residence Times at Various Flow Rates

Location	Minimum Residence Time (d) ⁽¹⁾	Cell Volume (m ³)	Residence Time (d)		Maximum Allowable Average Daily Flow (m ³ /d)
			2021 Average Daily Flow	Rated Daily Flow	
Cell No. 1	4-5	38,908	50	23	9,727
Cell No. 2		220,570	128	282	
Overall WWTP	30	259,478	333	151	8,649

Table 6-2 Notes:

(1) Ontario Sewage Works Design Guidelines

6.2.2 Potential for Increased Rated Capacity

As shown in the previous section, the currently constructed WWTP lagoon cell volumes exceed the minimum residence time requirements outlined by the Ontario Design Guidelines for Sewage Works. Therefore, there is potentially capacity to increase flows based solely on the hydraulic capacity in the system. As is evident in Table 6-2, the average daily flow could be increased as high as 8,500 m³/day without exceeding the residence time requirements.

6.3 Organics Treatment and Removal

6.3.1 Current Design and Operation

The organic component of the wastewater is measured as BOD₅, or CBOD₅, an amalgam measurement based on the amount of oxygen required to degrade the organic matter, rather than a concentration of components of the organic matter. This BOD₅ is removed from the wastewater through settling of the suspended component or the aerobic degradation of the dissolved component. Degradation of BOD₅ requires 1 kg of dissolved oxygen (kgO₂) per kg of BOD₅.

At the current flow rates, Cell No. 1 is removing 88% of the organic component of the raw wastewater (measured as BOD₅ or CBOD₅) and is achieving, on average, the required effluent quality with respect to BOD₅. The settling occurring in the upstream section of Cell No. 1 allows the suspended organic matter to settle and be degraded anaerobically in the sediment. The aeration system provides oxygen for the biological degradation of the dissolved organic matter. The floating vegetated islands provide further removal of organics through filtration by the submerged root system and the uptake of carbon by the vegetation. Overall, Cell No. 1 is close to capable of achieving the effluent requirements for BOD₅ with only four instances in 2021 of weekly effluent concentrations above the

discharge limits. This elevated extent of organics removal in Cell No. 1 allows for Cell No. 2 to focus on nitrogen and phosphorus removal without competition from the microorganisms associated with organics removal.

Cell No. 2 accounts for the remaining 6% removal of organics. This low removal in Cell No. 2 should not be attributed to an inability of the cell to remove organics, but rather the low organic load into the Cell. Cell No. 2 is removing 56% of the BOD that discharged into the Cell, and if there were a higher influent concentration, the cell would most likely be capable of achieving additional removal. This increased treatment would be dependent on the oxygen transfer rate to Cell No. 2 as the organic load increases. The total WWTP is achieving effluent BOD₅ concentrations far below the effluent limits and is removing 95% of the organics from the raw wastewater influent.

Aeration is provided by fine bubble diffusers in the downstream section of the cell, after the submerged concrete dividing wall, as well as through the surface mixing of the Blue Frogs. The Blue Frog systems are regularly removed from the cell for repairs and therefore do not represent a consistent supply of oxygen and will be excluded in the calculations. The aeration is provided by three 15 HP blowers, each capable of providing 188 cfm (67 m³/h). The Ontario Guidelines for sewage works provides a typical oxygen transfer efficiency of 6-15% for fine bubble diffusers. Assuming a 10% efficiency, the current system can provide 641 kgO₂ per day to the lagoon. Based on the 2021 average flow rate (766 m³/day) and the average BOD₅ concentration (134 mg/L), Cell No. 1 received an average of 102 kg of BOD₅ per day, which requires 102 kgO₂ to degrade. This oxygen requirement is well below the 641 kgO₂/d range that can be provided by the aeration system (16% of the capacity). If this oxygen demand were extrapolated to the rated capacity of the WWTP (1,722 m³/day) while maintaining the same influent BOD₅ concentration (134 mg/L), the Cell would require 231 kgO₂/day (36% of the capacity). It can therefore be assumed that the Cell No. 1 in its current layout is sufficient to meet the rated capacity of the WWTP.

6.3.2 Potential for Increased Rated Capacity

The design organic loading rate for aerated facultative lagoons in the Ontario Design Guidelines for Sewage Works is 0.031-0.048 kgBOD₅/m³/day. The current loading rate in this system is 102 kgBOD₅/day as reported in Section 6.3.1. As the Cell No. 1 volume is 38,908 m³, the resulting organics loading rate is 0.002 kgBOD₅/m³/day (8% of the lower limit to BOD₅ loading). At the current rated capacity of the WWTP (1722 m³/day) this loading rate increases to 0.004 kgBOD₅/m³/day (15% of the lower limit to BOD₅ loading). Therefore, there is additional capacity beyond the current rated capacity in Cell No. 1. Should the Municipality wish to increase the flows to the WWTP beyond the currently rated capacity with no modifications to current operations, Cell No. 1 has capacity to manage these flows.

The aeration system, as discussed in Section 6.3.1 can provide 641 kgO₂/day. Applying a safety factor of 1.5 times, this system could be considered capable of providing

427 kgO₂/day. This system would, therefore, be capable of degrading 427 kgBOD₅/day, which would amount to a flow rate of 3,180 m³/day into Cell No. 1 with the current aeration system.

Aerated facultative lagoons are typically modelled kinetically using a tanks-in-series model to account for the non-idealities of flow in the cells. This model needs to be applied to the overall WWTP (combining Cell No. 1 and Cell No. 2) instead of just Cell No. 1, to identify total organic treatment capacity. When the WWTP is analyzed using this model, it is best described as a single tank with an average 70% mixing efficiency. When described as two distinct tanks the mixing is 80% in Cell No. 1 and 20% in Cell No. 2. This mixing is mainly achieved in Cell No. 1 with the aeration system and the Blue Frogs. After the number of tanks in series that best describes the WWTP was estimated from the 2021 data, the model was used to determine the availability for additional flow in the WWTP. Assuming organic concentrations remained at the same range, this flow is 6,356 m³/d.

The WWTP has an effluent requirement based on concentration as well as daily mass loading. For BOD₅ this effluent mass loading limit is 43 kg/d. Currently (2021) the WWTP is discharging on average 5 kg/d, or 12% of the loading limit. Extrapolating this limit to the current rated capacity of the WWTP and assuming the current effluent BOD₅ of 7 mg/L is maintained, the effluent loading at the rated capacity is 12 kg/day which is 28% of the effluent load limit. Therefore, there is capacity for additional effluent discharge within the ECA. Assuming the current average discharge BOD₅ the WWTP would be capable of receiving 6,143 m³/day. Should the effluent BOD₅ increase to the treatment objective of 20 mg/L this maximum annual flow rate would decrease to 2,150 m³/day. Both effluent flow rates are above the current rated capacity of the WWTP.

Using the four methods described above, biological loading as compared to the Ontario Sewage Guidelines, aeration system capacity, and kinetic modelling all demonstrate that in terms of organic reduction (BOD₅ removal) the WWTP could receive a higher flow rate than the current rating.

6.4 Suspended Solids Treatment and Removal

6.4.1 Current Design and Operation

The removal of suspended solids in the WWTP is governed by settling, which requires calm conditions in the cells. In conventional WWTPs, this occurs in specifically designed system in the plant that achieve quiescent conditions. Lagoons are typically oversized to ensure that there can be efficient mixing for biological activity while having sufficient calm areas for settling of solids. However, high flow rates can result in resuspension of solids, which causes elevated discharge concentrations that are not indicative of the actual performance of the lagoons. Algal growth in the vicinity of the lagoon effluent can also present as TSS in the effluent samples.

In the Markdale WWTP, Cell No. 1 has an upstream volume that is separated from the remainder of the cell. This section is to collect the bulk of the larger solids from the raw wastewater and allow degradation in the sediments of the cell, as well as the precipitated phosphorus from the initial alum addition. Cell No. 1 is currently removing an average of 79% of the TSS (73.7 kg/d), with the remaining 17% being removed in Cell No. 2. The total removal of solids in the WWTP is 96% (88 kg/day). The average effluent from Cell No. 1 did not meet the discharge objectives of the WWTP. After polishing in Cell No. 2, the effluent was well below treatment objectives.

Solids that are being removed, and consequently accumulating as sludge, include organic and inorganic material in the raw wastewater, biomass growth in the WWTP and precipitated phosphorus. Degradation occurring in the sludge will reduce the volume of the organic material, however the inorganic material and precipitated phosphorus will accumulate in the cells.

6.4.2 Potential for Increased Rated Capacity

The evaluation of the settling characteristics in lagoons is very difficult, due to aeration and mixing in the cells. It is evident that the current operation of the WWTP is sufficient for the removal of TSS from the wastewater. If the Ontario Sewage Works Guideline Residence Time (30 days) is maintained, there should be sufficient removal of TSS in the WWTP. The major risk will be the aeration system that is required for BOD₅ removal affecting the conditions in Cell No. 2 and not allowing the solids to settle. This risk can be mitigated by ensuring that any additions to the aeration system do not extend to the effluent location, allowing mixing in the areas with active aeration and allowing an area of no aeration to allow settling.

Increases in influent to the WWTP will increase the mass of solids settled, the frequency of sludge removal activities and the volumes of removed sludge that requires disposal. The exact effect depends on the increased flow rate and how that affects the degradation of organic sludge accumulated in Cell No. 1.

The WWTP has an effluent requirement based on concentration as well as daily mass loading. For TSS this effluent mass loading limit is 51.6 kg/d. Currently (2021) the WWTP is discharging on average 4.6 kg/d (9% of the loading limit). Extrapolating this limit to the current rated capacity of the WWTP and assuming the current effluent TSS concentration (6 mg/L) is maintained, the effluent loading at the rated capacity is 10.3 kg/day (20% of the effluent load limit). Therefore, there is capacity for additional effluent discharge within the ECA. Assuming the current average discharge TSS concentration (6 mg/L) the WWTP would be capable of receiving 8,600 m³/day. Should the effluent TSS concentration increase to the treatment objective of 25 mg/L this maximum annual flow rate would decrease to 2,064 m³/day. Both effluent flow rates are above the current rated capacity of the WWTP.

The WWTP currently allows all components of the raw wastewater to enter Cell No. 1. This includes inorganic waste materials such as bags, rags, flushable material, etc. This material does not degrade in the WWTP and can clog flow pathways or interfere with mechanical components like the Blue Frogs. As the flow to the WWTP increases, the volumes of inorganic material will also increase, heightening the likelihood of adverse effects. The inclusion of a headworks for the WWTP would remove this inorganic component from the raw wastewater. Options include a simple bar screen that is cleaned manually during daily operator rounds, automatically cleaned screens, or a full headworks with an aerated grit chamber and screen. The inclusion of inorganic removal at the WWTP will allow for less inorganics in the removed sludge and increase end uses for the sludge, decrease the overall risk of equipment failures due to inorganics, and decrease the overall volumes of accumulated sludge.

6.5 Nitrogen Treatment and Removal

6.5.1 Current Design and Operation

The TKN in the wastewater is an amalgam measurement of organic nitrogen and ammoniacal nitrogen. Removal of TKN in the WWTP is governed by both the settling of solids that contain organic nitrogen (included with a component of the BOD₅ that is settled) and the aerobic degradation of soluble TKN, through hydrolysis of the organic nitrogen to ammoniacal nitrogen, and the aerobic nitrification of the ammoniacal nitrogen. The settling of suspended TKN occurs in the upstream volume of Cell No. 1 with the removal of suspended BOD₅, the remained of the TKN removal occurs in the aerobic zones of the WWTP. The degradation of TKN requires 4.6 kgO₂ per kilogram of TKN (kgTKN).

Based on the 2021 concentration data, Cell No. 1 is currently removing 53% of the influent TKN. This amounts to an average daily removal of 12 mg/L (9.2 kgTKN/day) which requires 42 kgO₂/day from the blowers. Cell No. 2 currently removes an additional 30% of the influent TKN (5.1 kgTKN/day) requiring an additional 23 kgO₂/day. This increases the overall oxygen requirements of the WWTP as outlined in Section 6.3.1 to 168 kgO₂/day at the 2021 flows and 379 kgO₂/day at the rated capacity of the WWTP. The oxygen demand for both BOD₅ removal and TKN removal remains below overall blower capacity of 641 kgO₂/day (Section 3.2.2).

6.5.2 Potential for Increased Rated Capacity

The removal of TKN is not mandated at the WWTP but should be maintained to ensure adequate source water protection. The requirements for TKN removal are sufficient with aeration and adequate residence time to reduce the BOD₅ and allow the nitrifying bacteria to remove the TKN. The 30 days residence time as required by the guidelines for sewage works will be adequate to achieve TKN removal. The aeration system, with a safety factor of 1.5 times, can provide 427 kgO₂/day (Section 6.3.1) which would allow the treatment of a maximum of 1,940 m³/d at the current estimates. The limiting factor in the treatment

is aeration. This capacity could be increased by installing additional aeration capacity at the WWTP.

6.6 Phosphorus Treatment and Removal

6.6.1 Potential for Increased Rated Capacity

Increasing WWTP flows will increase the overall load of phosphorus to the cells. As discussed in the suspended solids removal section (Section 6.4.2) residence times are enough to allow for the settling of additional solids introduced by the increased load. The settling of the precipitated phosphorus will also be achieved in the minimum residence time of 30 days.

As flows increase, the alum dosing system will need to be reviewed to ensure that sufficient chemical addition can be achieved. The operations manual suggests that the dosing rate should typically be “close to 100 mg/L”. This concentration can be refined using jar tests as desired by the Municipality, but these values will be used for the purposes of this evaluation. The current dosing system includes two pumps capable of providing 18 L/hr of the alum solution each, or 36 L/hr combined. These pumps could be capable of providing 864 L/day of 48% alum solution to the WWTP. Providing 100 mg/L of dry alum requires 208 mg/L of 48% alum solution. 0.156 L of alum solution is required per cubic metre of wastewater to achieve the desired 100 mg/L dry alum dosing rate. The current alum dosing system could provide sufficient alum to treat 5,500 m³ of raw wastewater, which exceeds the current rated capacity of the WWTP.

The WWTP has an effluent requirement based on concentration and daily mass loading. For TP, this effluent mass loading limit is 1.12 kg/d. Currently (2021) the WWTP is discharging on average 0.08 kg/d or 7% of the loading limit. Extrapolating this limit to the current rated capacity of the WWTP, and assuming the current effluent TP concentration of 0.6 mg/L is maintained, the effluent loading at the rated capacity is 0.17 kg/day (15% of the effluent load limit). Therefore, there is capacity for additional effluent discharge within the ECA. Assuming the current average discharge TP concentration (0.1 mg/L) the WWTP would be capable of receiving 11,200 m³/day, above the current rated capacity of the WWTP.

6.6.2 Current Design and Operation

Phosphorus removal in the WWTP is controlled mainly through the addition of alum (aluminum sulphate), which in turn reacts with the phosphate to produce aluminum phosphate. This precipitates from the wastewater solution and is settled along with the suspended solids in the WWTP. The floating vegetated islands will utilize a small portion of the phosphorus as a nutrient; however, this phosphate will be released upon decomposition of the vegetation and should not be considered a removal mechanism for phosphorus.

The addition of alum and the settling of solids are the driving factors in the removal of phosphorus. Alum is dosed at both the influent of Cell No. 1 and the influent of Cell No. 2 and so both cells should be achieving phosphorus removal. Cell No. 1 removed an average of 76% of the influent phosphorus (2.7 kg/day). Cell No. 2 removed an additional 22% of the phosphorus (1 kg/day) resulting in a total removal in the WWTP of 98% (3.7 kg/day).

These phosphorus removal rates align well with the results from TSS removal (79% and 17% TSS removal in Cell No. 1 and Cell No. 2, respectively), which indicates that the settling is the limiting factor in the removal of phosphorus, rather than the amount of alum.

6.7 RIB Wastewater Discharge

6.7.1 Current Design and Operation

Currently the WWTP is discharging to eight RIBs, with two available in the winter and the remaining six available from April to October. These RIBs are designed to allow the treated effluent to infiltrate into the ground and percolate to the water tables. Treatment is occurring in the unsaturated zones as well as in the saturated water table, and there will be dilution occurring in the water table. The RIBs are designed to operate at a rate of 2,871 m³/day, with a dosing time of one week followed by a rest period of two weeks (WSP, 2022).

The actual maximum loading rate of the RIBs is reliant on the infiltration rate and capacity of the subsoil as well as the non-native soil installed at the surface of the RIBs. The ECA requires the unsaturated space between the groundwater mound and the base of the RIB to be maintained at a minimum depth of 60 cm. In 2021, the unsaturated zone at any monitored well did not decrease past 1.02 m, which indicates that the RIBs are not overloaded.

Furthermore, the concentrations of wastewater constituents in the Rocky Saugeen River measured at the upstream, central, and downstream sampling locations did not detect any TP or TKN. This indicates that the infiltration system is providing adequate treatment, or dilution of the effluent prior to eventual discharge in the river.

6.7.2 Potential for Increased Rated Capacity

The potential for increasing the flow at the RIBs is highly dependent on the hydrogeological conditions under the RIBs. There is currently limited information on the subsurface and therefore it is difficult to fully determine the effects of additional flow on the local groundwater.

At the current flow rates, the RIBs are not creating a ponding situation where the RIB require dosing, followed by infiltration time. This is largely due to the coarse subsurface material allowing for high infiltration rates. This infiltration rate will need to be monitored

as the effluent flow rate to the WWTP is increased to ensure that there is no major increase in ponding on the RIBs as an effect of the increased flow.

The RIBs are currently designed to receive 1.5 times the 1986 design flow rate of the WWTP, i.e., 2,871 m³/day, to account for surges due to the increased discharge after winter storage, or high rainfall (Henderson Paddon, 1986). If the WWTP is discharged continuously at a rate equivalent to the inflow, this additional design capacity can be used to discharge treated wastewater. However, there will be no additional capacity to discharge surges at the WWTP. The rating of the RIBs could be further increased with geotechnical study to ensure the infiltration rates and subsurface flow rates are sufficient.

The discharge requirements for the RIBs are a 0.6 m unsaturated zone under the RIBs to allow aerobic treatment in the subsurface zone below the RIBs. Well OW16-79, upstream of the RIBs, has the highest water table readings. The water level decreases from this well, to well OW13-86 which is upstream of the RIBs. The water table decreases further at OW1-86 which is located at the edge of RIB #6. This high-water level at OW01-79 is likely a localized mound that is affected by infiltration upstream of the RIBs but should be monitored to ensure that the local groundwater level does not increase high enough to halt RIB discharge.

There is a higher groundwater elevation under the RIBs (OW1-86) than the downstream area. There is evidence of groundwater mounding under RIB #6 when discharge is occurring in the winter, at the current flow rates. This mound increased rapidly in November of 2021 as the dosing moved to the winterized RIBs but also lowered quickly. At the maximum height, the mound still provided 2.3 m of unsaturated space which is still greater than the 0.6 m requirement.

To properly determine if there is additional capacity at the RIBs investigations into the local hydrogeology are needed. This analysis would look at the effect of the effluent constituents from the wastewater, the local assimilative capacity, and the effect of the volume of water increasing in the groundwater at this location.

6.7.3 WSP Hydrogeological Baseline Assessment (Desktop), 2023

WSP conducted a hydrogeological baseline assessment of the WWTP in July 2023 that consisted of reviewing historical data, developing a model, and identifying gaps. The results are included in the report attached in Appendix C.

The WWTP is located on high elevation relative to the valley coincident to the Rocky Saugeen River. The RIBs are located on coarse-grained glaciofluvial material: sand, gravel, with silt, cobbles, and some boulders. This is underlain by sandy silt, followed by silt till, then bedrock. The depth of the RIBs to the water table varies from 1.1 to 3.9 m, and the average depth has decreased since 2014 in some wells, in addition to expected fluctuations due to climatic patterns.

Mounding calculations made during pre-construction by Henderson Paddon in 1981 were re-run using recent discharge and groundwater level data in the vicinity of one RIB. The results found that an average loading rate of 2,350 m³/day will cause the groundwater level to rise within the minimum separation distance of 0.6 m from the ground surface, meaning a 50% increase in rating would not be achievable.

This groundwater level within 50 m of the RIB is directly affected by the rate of loading to the RIB. WSP found there was minimal data and recommended the inclusion of additional wells in the monthly monitoring program to evaluate the water level changes in the other RIBs, and the inclusion of more detailed water quality monitoring. WSP reran mounding calculations conducted by Henderson in 1981 with the most recent discharge data and found they were broadly consistent.

6.8 Overview of Capacity Constraints

The capacity constraints described in the previous sections are summarized in Table 6-4. Hydraulic loading associated with pending development approvals will exceed the current rated capacity by 13%. This development will also result in the WWTP's aeration system operating slightly above 100% of its rated capacity.

The pending development approvals do not include the future expansion of the Chapman's facility. Therefore, the Markdale WWTP will require upgrades to accommodate both future proposed growth and anticipated future developments.

As the 113% increase in rating capacity does not include flows from the expanded Chapman's facility, the Municipality considered various WWTP capacity increase scenarios greater than 13%. They included increasing the WWTP rating by 25%, 50%, and 100%. The associated flow rates are summarized in Table 6-3 below.

Table 6-3 WWTP Flow Rate Increase Scenarios

Proposed Capacity Increase	Flow (m ³ /day)
Current	1,722
25% Increase	2,150
50% Increase	2,580
100% Increase	3,450

The Municipality reviewed these proposed capacity increase scenarios in a meeting and decided to pursue a 50% increase in rating. A 50% rating increase is equivalent to a WWTP flow rating of 2,580 m³/d. This flow rate would accommodate the current flow, the 13% increase from pending development approvals, and a 37% buffer for future flow increases including contributions from the expansion of the Chapman's facility.

Table 6-4: Summary of WWTP Capacity Constraints

Parameter	Criteria	Existing Population	With Committed Development	With Committed and Conditional Approval Growth	With all Proposed Growth ⁽¹⁾
Hydraulic Loading	Current Rated Capacity (1722 m ³ /day)	44% of rated capacity (766 m ³ /d)	57% of rated capacity (988 m ³ /d)	82% of rated capacity (1,405 m ³ /d)	113% of rated capacity (1,948 m ³ /d)
	Minimum 4-day hydraulic residence time in the Cell No. 1	51 days	39 days	28 days	20 days
	Minimum 30-day hydraulic residence time in the two lagoon cells	339 days	263 days	185 days	133 days
Organics	Loading rate (0.031 – 0.048 kgBOD ₅ /m ³ /d)	0.002 kgBOD ₅ /m ³ /d	0.003 kgBOD ₅ /m ³ /d	0.005 kgBOD ₅ /m ³ /d	0.007 kgBOD ₅ /m ³ /d
	Discharge Limit (43 kg/d)	5 kg/d	7 kg/d	10 kg/d)	14 kg/d
TSS	Discharge Limit (51.6 kg/d)	4.6 kg/d	5.9 kg/d	8.4 kg/d	11.7 kg/d
TP	Alum Supply (864 L/day maximum)	119 L/d	154 L/d	219 L/d	304 L/d
	Discharge Limit (1.12 kg/d)	0.08 kg/d	0.10 kg/d	0.14 kg/d	0.19 kg/d
Aeration System	Oxygen Supply (427 kgO ₂ /d maximum)	169 kgO ₂ /d	217 kgO ₂ /d	72% of capacity (309 kgO ₂ /d)	100% of capacity (429 kgO ₂ /d)
RIBs	Depth to groundwater (Minimum of 0.6 m)	Can treat up to 125% (2,350 m ³ /d) of the rated capacity.			

Table Note:

(1) Based on current effluent loading limits, assuming treatment is maintained at current average effluent concentrations.

7.0 PROBLEM AND OPPORTUNITY STATEMENT

Based on the information developed and analyzed during Phase 1 of this Class EA, the following problem/opportunity statement was developed:

“A review of the Markdale Wastewater Treatment System indicates operational constraints limiting the ability of the lagoon to treat increased flows from future development. The Municipality of Grey Highlands is therefore undertaking a Schedule ‘C’ Class Environmental Assessment (Class EA) to evaluate options to upgrade the Markdale WWTP to accommodate a 50% increase in rating. The Class EA will consider the level of adequacy of wastewater treatment at the lagoon and will recommend a solution to address the findings in accordance with the Municipal Class EA process.”

8.0 DESIGN BASIS

8.1 Design Influent Volume

8.1.1 Average Day Flow

Based on growth projections presented in Table 2-1, an anticipated 1,075 units will be added to the serviced area through new developments. These developments are all residential, apart from 15 units allocated to the Markdale Hospital.

As mentioned in Section 5.0, the Municipality is working to update its population projections with a density of 2.5 persons per unit. Using this density, the population is anticipated to increase by 2,688 people from new developments. As outlined in Section 5.2, future flows use a design flow of 375 L/d per capita.

The flows associated with the future development is then 1,008 m³/d, where 14 m³/d of these flows is associated with the Markdale Hospital expansion, and the remaining 994 m³/d is residential.

The Municipality does not anticipate an expansion in the septage receiving area. Therefore, no additional future septage volumes were included.

Table 8-1 Design Average Day Flow by Land Use

Flows by Contributor	Design Average Day Flow (m ³ /day)
Existing Average Daily Flow ⁽¹⁾	839
Existing Septage ⁽²⁾	4
Future Residential	994
Future ICI (excluding Chapman's)	14
Total flows excluding Chapman's	1,850
Proposed Average Day Flow	2,580
Buffer allowed by 50% re-rating	<730

Table Notes:

(1) Five-year average flow taken from Table 5-1.

(2) Two-year average daily volume rounded from Table 5-13.

As shown in Table 8-1, no flow rate was provided for the future development at the Chapman's facility, the following calculations assess the adequacy of the 725 m³/d buffer provided by a 50% rating increase to 2,580 m³/d to accommodate future flows from Chapman's factory.

The average annual metered wastewater flows from the Chapman's facility, i.e., "process" generated flows, are 102,667 m³/year as determined in Table 5-5, or 281 m³/day. The

total area of the existing factory, determined from the Grey County GIS data available online, is 36 acres or 14.6 hectares. Therefore, the average process generated flow per hectare is 7,047 m³/year or 19.3 m³/day.

Wastewater produced domestically by Chapman's, i.e., the "employee" generated flows, is not metered. As discussed in Section 5.2, both full time and part time employees are considered to produce 40% of a resident's flows, meaning the current factory has an equivalent population of 340 people, or 9.4 people per hectare. When multiplied by the design flow of 375 L/d per capita, this results in an employee generated flow of 3.5 m³/d per hectare.

Therefore, each hectare generates an estimated 19.3 m³/day of process wastewater and 3.5 m³/d employee wastewater as established above. This results in a combined wastewater flow of 23 m³/d per hectare.

Therefore, to stay within the 725 m³/d buffer provided by the 50% rating increase, as much as 31 hectares of the land purchased by Chapman's can be developed, assuming the per hectare process and domestic wastewater flows remain approximately 19.3 m³/day and 3.5 m³/d respectively. The parcel of land purchased by Chapman's for further development is approximately 88 acres or 35.6 hectares in area, according to the Grey County GIS data. A 31-hectare expansion is nearly 2.5 times the existing factory area and comprises of almost 90% of the land purchased by Chapman's for expansion.

Therefore, the 725 m³/d buffer provided by a 50% rating increase is anticipated to accommodate future flows which could be reasonably expected from Chapman's factory. Future flows will need to be confirmed with Chapman's when their development is finalized.

8.1.2 Maximum Day Flow

Based on a review of historical data (refer to Section 5.1) the maximum day factor between 2017 and 2022 ranged from 4.7 to 2.1 with an average of 3.0. This maximum day factor is relatively high and snowmelt in the early spring appears to be contributing in some years. It is expected that over time, as inflow and infiltration are addressed this number may decrease. However, for the purposes of reviewing alternative treatment technologies, utilizing the historical maximum day factor of 3 times average day is recommended.

8.2 Design Influent Concentrations

8.2.1 Domestic Wastewater Quality and Loading

The historical influent quality was summarized in Table 5-9. It was also compared to typical untreated domestic wastewater strength as listed in Metcalf and Eddy (2014). The analysis determined that the wastewater influent has an average concentration of BOD₅, TP, TSS, and TKN below medium strength for typical untreated domestic wastewater.

The design influent concentrations for these parameters will be assumed as the rounded average between low and medium strength domestic wastewater influent as listed in Metcalf and Eddy (2014).

Table 8-2 Design Concentration and Loading for Domestic Wastewater Influent

Parameter	Average (2017-2022)	Design Concentration (mg/L) ⁽¹⁾	Design Loading (kg/d)
BOD ₅	134.3	170	428
TSS	148.8	165	418
TP	4.0	5.0	12
TKN	26.1	30	75

Table Notes:

- (1) Rounded average of the values for low and medium strength domestic wastewater taken from Metcalf and Eddy (2014).

The design loading was determined by multiplying the design concentrations by 2571.4 m³/d, which is the design rating of 2,580 m³/d, reduced by the imported sewage contribution of 3.7 m³/d. The contribution of imported sewage on influent quality is discussed below.

The contribution of Chapman's wastewater flows on influent quality is discussed in Section 8.2.3.

8.2.2 Imported Sewage Quality and Loading

The Markdale WWTP also receives imported sewage from septage, portable toilets, and holding tanks. The historical imported sewage quality is summarized in Table 5-12 and Table 5-13. In comparison with the 1994 EPA values, they are below average. Therefore, the design concentration is the rounded average of the historical and EPA concentrations.

Table 8-3 Design Concentration and Loading for Imported Sewage

Parameter	Rounded Historical Avg. (mg/L)	Rounded EPA Avg. (mg/L)	Design Concentration (mg/L)	Design Loading (kg/d)
BOD ₅	4,300	6,500	5,000	18.3
TSS	6,500	12,900	9,000	32.9
TP	160	210	180	0.7
TKN	N/A	590	600	2.2

The design loading was determined by multiplying the design concentrations by the average imported sewage volume of 3.7 m³/d or 3,650 L/d.

8.2.3 Chapman's Wastewater Quality and Loading

Data for the strength of industrial wastewater was not available to the Municipality. However, as discussed in Section 5.2, an available case study from the supplier noted the ISAM reactor installed by Chapman's in 2002 had a design effluent concentration of 50 mg/L for BOD₅ and TSS. These values are lower than the anticipated domestic wastewater influent strength. Therefore, it was assumed the quality of wastewater produced by ICI is the same as domestic flows.

8.3 Design Influent Flow and Loading Summary

The overall design loading for the Markdale WWTP is the sum of the domestic wastewater flows, which includes Chapman's and future ICI, and the imported sewage. These values and the flow rates are summarized in Table 8-4 below.

Table 8-4 Design Influent Flows and Loading

Parameter	Daily Flow (m ³ /d)	BOD ₅ (kg/d)	TSS (kg/d)	TP (kg/d)	TKN (kg/d)
Existing Domestic and ICI	839	112	124	3.0	22.0
Existing Septage	4	18	33	0.7	2.2
Future Domestic	994	428	418	12	75
Future ICI (Excluding Chapman's)	14				
Chapman's Allocation	730				
Total Design Influent	2,580	558	575	16	99

8.4 Effluent Regulations and Requirements

During the 2018 re-rating of the Markdale WWTP, only the Total Phosphorus design objectives and limits were changed to be more stringent. The objective decreased from 0.8 to 0.5 mg/L, and the limit decreased from 1.0 to 0.65 mg/L. The objectives and limits for CBOD₅, Total Suspended Solids, and pH remained the same after the re-rating.

Design effluent objectives and limits for the expanded plant are determined in consultation with the MECP, in accordance with guidelines and procedures such as Section 53 of the Ontario Water Resources Act, Guideline B-7 Incorporation of the Reasonable Use Concept into MOEE Groundwater Management Activities, Procedure B-7-1 Determination of Contaminant Limits and Attenuation Zones, Wells Regulation O Reg. 903 (Revised Regulations of Ontario 1990), and Clean Water Act, 2006. If further studies consider surface water discharge, relevant guidelines include MECP's Guideline F-5,

Levels of Treatment for Municipal and Private Sewage Treatment Works Discharging to Surface Waters, and B-1, Water Management Policies, Guidelines and Provincial Water Quality Objectives.

A consultation meeting was held with the MECP on September 19, 2023, the minutes of which are available in Appendix D. Based on discussions at the meeting and subsequent email correspondence, MECP provided the following guidance:

- No surface water impact assessment will be required.
- Given the existing phosphorus limit and the fact that, once on soil, ammonia should convert to nitrate rapidly, it is the opinion of the Ministry's Surface Water Specialist that there is no need to revisit the limits and objectives, provided that the Municipality remains confident that the exfiltration basins continue to function as designed.
- Future detailed hydrogeological assessment should confirm that effluent entering shallow groundwater from RIBs will not pass underneath the river.

Therefore, subject to the future detailed hydrogeological assessment, anticipated effluent objectives and limits for the expanded plant are summarized in Table 8-5.

Table 8-5: Anticipated Effluent Objectives and Limits

Parameter	Concentration Objective	Concentration Limit	Loading Limit
Five-day Carbonaceous Biochemical Oxygen Demand (CBOD ₅)	20 mg/L	25 mg/L	64.5 kg/d
Total Suspended Solids	25 mg/L	30 mg/L	77.4 kg/d
Total Phosphorus	0.5 mg/L	0.65 mg/L	1.68 kg/d
pH	6.5 – 8.5 (inclusive)	6.0 – 9.5 (inclusive)	-

9.0 PHASE 2 - EVALUATION OF TREATMENT EXPANSION OPTIONS

The main objective of Phase 2 of an MCEA is to identify and evaluate possible alternative solutions to the problems and/or opportunities identified in Phase 1. Reasonable potential solutions to the problem(s), including the 'do nothing' alternative, are considered. To facilitate the evaluation and selection of the preferred solutions during Phase 2, a transparent assessment process was established. Evaluation criteria were developed based on a review of the background information, experience on similar assessments, and consultation with Municipality staff.

The evaluation was conducted using criteria in the following categories:

- Archaeology and Cultural Heritage
- Ecology and Natural Environment
- Technical and Operational Considerations
- Economic Impacts

Each alternative was assigned an evaluation impact level (refer to Table 9-1) for each criterion. This method provides an overall assessment of the positive and negative impacts of each alternative. This method was used as it is recognized that there could be more than one (1) alternative or technology that can address a problem and that additional consideration of these technologies could be undertaken. Further consideration of design alternatives are included in Phase 3 of this MCEA.

Table 9-1 Evaluation Impact Levels and Indicators

Evaluation Impact Level	Indicator
High Positive Impact	▲▲
Low/Moderate Positive Impact	▲
No Anticipated Impact	●
Low/Moderate Negative Impact	▼
High Negative Impact	▼▼

Once the evaluation was completed, a recommended preferred alternative or alternatives was identified for presentation to stakeholders and to solicit input prior to finalizing a preferred alternative.

9.1 Alternative Solutions

The alternatives considered are to do nothing, upgrade the existing lagoon WWTP, construct a new mechanical WWTP at the current site, and construct a new mechanical WWTP at a new site.

Costing was developed based on an assessment of previously constructed Canadian lagoons. The available costs were divided by the WWTP flow capacity and plotted to determine a relationship between WWTP size and cost.

1. **Do nothing:** this alternative consists of continuing to use the existing lagoon at the existing site with no upgrades to accommodate higher wastewater flows. This requires no additional work or expenditure related to construction. However, this study has established that the current lagoon is unable to accommodate flows from future development. Doing nothing will mean the WWTP will be unable to treat increased flows from future growth, thereby limiting development in Markdale. This would result in negative social and economic impacts.
2. **Upgrade the existing WWTP:** this alternative involves using the existing lagoon at the existing site while installing upgrades to accommodate higher wastewater flows. Upgrades could include the installation of a septage receiving system, headworks system, improved aeration system or lagoon-based advanced treatment system and expanded RIBs. These upgrades would be designed to accommodate a 50% increase in rating to satisfy future demands and implement operational improvements. This alternative will be able to accommodate increased flows from future development, allowing for economic and social growth while preserving the natural environment. Installing upgrades at the existing site prevents the development of new land through the utilization of existing infrastructure, minimizing monetary costs and the impact on the natural environment.
3. **Construct a new mechanical WWTP at the same site:** this alternative requires the construction of a brand-new mechanical treatment facility on the current WWTP site. The existing lagoon may be decommissioned or repurposed for tertiary treatment or managing excess flows. The new mechanical plant would house preliminary treatment, primary sedimentation, biological treatment, secondary sedimentation, and/or disinfection systems. The design could accommodate re-ratings higher than a 50% increase on the existing site at a high level of effluent quality. The cost is approximated at 40 million for the 50% rating increase (2024 dollars).
4. **Construct a brand new WWTP at a new site:** this alternative is to construct a brand new mechanical WWTP at a new site. The existing lagoon may be decommissioned. Land for the new site would be selected based on the assessment of various criteria through various studies. The new mechanical plant would house settling, sludge removal, filtration, and/or disinfection systems. The cost is approximated at 40 million for the 50% rating increase (2024 dollars). This excludes the cost of land acquisition,

studies to determine the suitability of the new site, and any mitigation measures required at the new site.

9.2 Evaluation of Alternative Solutions

The alternative solutions described in Section 9.1 were evaluated using the criteria described previously. A summary of this evaluation is included in Table 9-2. Based on this analysis and discussion with the Municipality, the recommended preferred alternative is to upgrade the existing lagoon WWTP to satisfy a 50% increase in rating.

9.3 Description of Preferred Alternative

As shown in Table 9-2, the preferred alternative is to upgrade the existing lagoon. This alternative involves using the existing lagoon at the existing site while installing a new headworks and aeration system to accommodate a 50% higher flow rating. Design alternatives were reviewed in Phase 3 of the Class EA process.

Table 9-2 Evaluation of Alternative Solutions

	1 – Do Nothing		2 – Upgrade existing lagoon WWTP		3 – New mechanical WWTP at current site		4 – New mechanical WWTP at new site	
Archaeology and Cultural Heritage	▲▲	<ul style="list-style-type: none"> No potential to disrupt archaeologic and cultural heritage features. 	▲▲	<ul style="list-style-type: none"> Minimal potential to disrupt archaeologic and cultural heritage features. 	●	<ul style="list-style-type: none"> Some potential to disrupt archaeologic and cultural heritage features. 	▼	<ul style="list-style-type: none"> Highest potential to disrupt archaeologic and cultural heritage features relative to other options.
Ecology and Natural Environment	▼▼	<ul style="list-style-type: none"> No potential to disrupt vegetation and wildlife. Will not be able to meet effluent criteria in the future. 	▲▲	<ul style="list-style-type: none"> Minimal potential to disrupt vegetation and wildlife. Will be able to meet effluent criteria in the future. 	●	<ul style="list-style-type: none"> Some potential to disrupt vegetation and wildlife. Will be able to meet effluent criteria in the future. 	▼▼	<ul style="list-style-type: none"> Significant potential to disrupt vegetation and wildlife. Will be able to meet effluent criteria in the future.
Technical and Operational Considerations	▼	<ul style="list-style-type: none"> No changes to existing system and operations. Does not address future capacity constraints 	●	<ul style="list-style-type: none"> Installation of headworks and improvement of aeration system Addresses future capacity constraints 	▼	<ul style="list-style-type: none"> Construction of brand new mechanical WWTP on site Exceeds future capacity constraints 	▼	<ul style="list-style-type: none"> Find new site. Construction of brand new mechanical WWTP Exceeds future capacity constraints
Economic Impacts	▲▲	<ul style="list-style-type: none"> Lowest capital costs relative to other options. 	▲	<ul style="list-style-type: none"> Low capital costs relative to other options. 	▼	<ul style="list-style-type: none"> Construction of new WWTP will cost around 35 million (2021 dollars) 	▼▼	<ul style="list-style-type: none"> Construction of new WWTP will cost around 35 million (2021 dollars) Land acquisition and studies
Review and Recommendation	▼▼	<p>This option will have the lowest costs as no changes would be made to the existing system. However, it does not address future capacity constraints at the WWTP.</p> <p>Recommendation: Do not carry forward.</p>	▲▲	<p>This option would involve investing in upgrades to the existing lagoon to accommodate a 50% in rating.</p> <p>Recommendation: Carry forward, recommended alternative.</p>	●	<p>This option will meet future demands and regulatory requirements. However, the costs of constructing a new mechanical WWTP will be higher than the other options.</p> <p>Recommendation: Do not carry forward.</p>	▼	<p>This option will meet future demands and regulatory requirements. However, the costs of constructing a brand new mechanical WWTP on a new site will be much higher than the other options.</p> <p>Recommendation: Do not carry forward.</p>

10.0 PHASE 3 - POTENTIAL DESIGN ALTERNATIVES

A review of available technologies was undertaken to develop a list of potential alternative design concepts to facilitate the preferred treatment expansion option identified in Phase 2. For the lagoon to be able to accommodate a 50% increase in capacity to 2,580 m³/d, a headworks system, improved treatment process, and expansion of the RIBs will be required.

10.1 Headworks and Septage Receiving

There are currently no headworks or formal septage receiving infrastructure installed in the WWTP. This has resulted in clogging issues within Cell No. 1 and interference with the aeration system, such as inorganic debris becoming tangled in the Blue Frog aerators. High volumes of solids extend the settling process and result in the production of more sludge with a high inorganic content that inhibits its use in many applications. Based on the preferred alternative from Phase 2, each of the design alternative options discussed in Phase 3 will include the addition of a headworks screening system.

The Municipality requested the installation of a headworks system upstream of Cell No. 1, which would remove most inorganic debris, reduce clogging, reduce the risk of interference with equipment, and generally increase the efficiency of wastewater treatment at the facility. This would improve the existing lagoon's ability to treat the higher flows and associated solids from future development.

Two general technology options are considered for a headworks system: a manual bar screen, or a mechanical screen system. Provision of appropriate septage receiving infrastructure has been considered for both options.

10.1.1 Manual Bar Screen and Package Septage Receiving Station

Bar screens are coarse filters installed to remove large solids. They can either be manual or mechanical. Manual bar screens have no moving parts and need to be cleaned through manual raking by operators. They can be used in conjunction with other treatment methods and are often used as a back up system. They require minimal maintenance beyond raking but are labour intensive. A bypass weir should be provided to ensure that a sewage spill does not occur if rags and debris clog the screen during high-flow periods; however, debris could be washed into the lagoons during bypass events.

Septage and hauled waste can introduce additional risk related to screen clogging, as they may contain large quantities of wipes or other inorganic debris introduced over a short period of time. Therefore, a package septage receiving station with an integrated pre-treatment system is recommended for the manual bar screen option.

10.1.2 Mechanical Screen System with Integrated Septage Receiving

A mechanical screen system automates the cleaning process using motorized devices, and typically includes means for washing and conveying the collected screenings to a disposal bin. Typically, units are installed in a building for ease of maintenance and to protect against freezing. There are several styles of mechanical screen systems. For example:

- A multi-rake mechanical bar screen can have a chain-driven scraper that cleans the inclined bar screen and lifts screenings into a discharge chute that automatically washes, dewateres, and compacts collected screenings.
- A step screen is a finer screen that separates out solids by lifting out debris from wastewater. This perforated plate screen consists of a continuous chain of panels, a cleaning brush, a sparge system and discharge chute. The sparge system dislodges solids from the screen panels, and the rotating brush sweeps screenings into the discharge chute. The unit is typically installed with a washer/compactor unit and a building is typically required to house the channel, the screens and the washer/compactor systems.
- The inclined auger screen system consists of a perforated steel screen, an auger with wiper/brush affixed to the outer edge to provide cleaning and screenings transport, and a spray wash system to separate organics from the captured shredded inorganic material. A transport segment lifts and dewateres the washed inorganic material. A discharge segment compacts the washed and dewatered inorganic material for disposal. Screened materials are discharge into a bagger system.

The installation of one of the above mechanical screen systems will allow any septage to be co-treated with raw sewage, negating the need for a dedicated septage pre-treatment system. Septage receiving infrastructure could be integrated into the headworks to receive and monitor the incoming septage collected from rural properties in the Municipality prior to combining with the raw sewage upstream of the screen system.

10.1.3 Cost Estimate

Table 10-1 summarizes cost estimates for the screening options listed above. Access road improvements have been included in the cost for both technology options. The cost of the mechanical screen system option also includes a new headworks building to house screening equipment and a septage receiving station.

Table 10-1: Summary of Screening Technologies Cost Estimate

Screening Technology	Total Cost ⁽¹⁾ (excl. HST, 2024 Dollars)	Annual Operating Cost ⁽²⁾
<u>Manual Bar Screen and Package Septage Receiving</u>		
Bar screen, concrete structure, package septage receiving station with pre-treatment and road improvements	\$2,976,000 - \$3,959,000	\$58,000 - \$141,000
<u>Mechanical Screen and Integrated Septage Receiving</u>		
Screen, washer/compactor, building with integrated septage receiving and road improvements	\$3,167,000 - \$4,254,000	\$25,000 – \$42,000

Table 10-1 Notes:

- 1) OPC includes costs for installation, channel and building construction, Contractor markup, contingency (30%), and engineering costs (15%).
- 2) Operating costs include operator labour, electrical consumption and average annual spare parts replacement over 20 years.
- 3) Inflation rate of 3.1% assumed to convert capital cost from 2023 to 2024-dollar values.

10.2 Treatment Process

The existing treatment process has been successful in achieving current effluent objectives and limits; however, as noted above, there have been some operational challenges (e.g., surface aeration systems regularly removed for repairs, damaged equipment in Cell No. 2) and aeration capacity limitations. Two general technology approaches have been considered: aeration upgrades, and addition of advanced treatment.

10.2.1 Aeration Upgrades

The existing aeration system includes two 15 HP blowers connected to a fine bubble diffuser grid in Cell No. 1, one 5 HP blower connected to a fine bubble diffuser system in Cell No. 2, and several surface aeration units. Replacing aging components of the existing system with higher capacity equipment will enhance removal of BOD and promote nitrification (i.e., TKN and ammonia removal), particularly during warm temperatures. Mixing provided by aeration systems can also reduce the probability of algae blooms which can increase effluent suspended solids and phosphorus.

10.2.2 Addition of Advanced Treatment

There are several advanced treatment technologies designed to improve effluent quality and/or increase treatment capacity of existing lagoons. These include:

- Aerated attached growth biological treatment units installed within lagoons, e.g., BioCord units. These units are designed to promote nitrification (i.e., TKN and ammonia removal) during cold temperatures.
- Aerated attached growth biological treatment units installed downstream of lagoons, e.g., Moving Bed Bioreactor, Submerged Attached Growth Reactor. These units are designed to promote nitrification (i.e., TKN and ammonia removal) during cold temperatures.
- Filtration systems installed downstream of lagoons, e.g., disk filters, sand filters. These units are designed to remove suspended solids and associated phosphorus.

Given that enhanced removal of suspended solids, phosphorus, and cold-temperature nitrification are not required, this option has been considered at a high level only.

10.2.3 Cost Estimate

Table 10-2 summarizes cost estimates for the options listed above.

Table 10-2: Summary of Process Treatment Technologies Cost Estimate

Treatment Process Technology	Total Cost ⁽¹⁾ (excl. HST, 2024 Dollars)	Annual Operating Cost ⁽²⁾
<u>Aeration Upgrades</u>		
Replace blowers and diffuser system	\$1,747,000 - \$2,732,000	\$56,000 - \$95,000
<u>Advanced Treatment</u>		
Aerated attached growth system	\$11,549,000 - \$21,043,000	\$164,100 – 349,600

Table 10-2 Notes:

- 1) OPC includes costs for supply, installation, contractor markup, contingency (30%), and engineering costs (15%).
- 2) Operating costs include operator labour, electrical consumption and average annual spare parts replacement over 20 years.
- 3) Inflation rate of 3.1% assumed to convert capital cost from 2023 to 2024-dollar values.

10.3 Rapid Infiltration Basins (RIBs)

There are eight (8) rapid infiltration basins existing in the Markdale WWTP, with two previously modified to allow for winter operation. The beds were constructed to be long and narrow, each approximately 56 m long by 6 m wide and 0.6 m deep.

Current standard operation is for up to four RIBs to be loaded a rate of 500 m³/day each for a period of 7 days. This is an overall treatment capacity of up to 2,000 m³/day in the existing RIBs. The incoming effluent from lagoon Cell No. 2 is distributed to the selected basins, with the sides of each sloped upward to allow for large flow rates. The effluent then percolates through the beds to the sand and gravel below, mixing with the natural

groundwater. After the 7 days, the RIBs in operation are then rested and re-tilled if required while up to four other RIBs are loaded.

To meet the increased flow rating from the Phase 2 preferred solution, up to four additional RIBs may need to be added as part of each of the alternative design options to be considered in Phase 3. The new units would be integrated with the existing system. For operational flexibility, these new RIBs should be designed for year-round operation and include effluent level monitoring.

10.3.1 WSP Hydrogeological Baseline Assessment (Field), 2026

Following the desktop hydrogeological baseline assessment of the WWTP WSP conducted in 2023 (summarized in Section 6.7.3) WSP was retained to undertake field work to support this Class EA. The field work included the drilling of five boreholes within three potential RIB expansion locations (called Areas 1, 2, and 3), after the locations were cleared by the Stage 2 Archaeological Assessment and consultation with review agencies regarding the drilling work. The potential expansion locations, including the boreholes, are shown in Figure 8 (Figure 1 from the Hydrogeological Baseline Report (WSP, 2026)). The report documenting this work and the results is included in Appendix C.

WSP used three of the boreholes as monitoring wells, which found that the water level ranged from 3.01 to 3.2 m below the ground surface (bgs) in Area 1, 2.15 to 2.68 m bgs in Area 2, and 0.85 to 0.89 m bgs in Area 3. Area 3's water level is noticeably closer to the ground surface, attributed to the lower ground elevation in the area. This elevated water level, the existing tree cover, and the steep slope led WSP to recommend that the future RIBs not be constructed within Area 3.

Hydraulic conductivity testing of samples from the monitoring wells was also undertaken. WSP determined the geometric mean hydraulic conductivity for the cobbly sand/sand and gravel unit found as 4.1×10^{-7} m/s, lower than what may be expected based on the coarse-grained material on site. WSP theorized that the unit may be naturally cemented, reducing soil porosity and ultimately reducing the rate of transmissivity in the soil. Infiltration testing was also completed by WSP, where the estimated design infiltration rate was found to be 14-15.6 mm/hr. These values led WSP to recommend that native material may not be suitable for use in constructing the new RIBs. They noted that this is consistent with historical studies, and that the existing RIBs were built with engineered design receiving soils.

WSP reviewed groundwater level data to date and found the effluent discharge from the existing RIBs produced a maximum of 0.7 m of change in groundwater level adjacent to them. Seasonal groundwater fluctuations in the shallow overburden were around 1.5 m. Data loggers were installed at the new monitoring wells by WSP, and they recommended that they be left to monitor the data from Spring 2026 to inform the RIB design.

Ultimately, WSP recommended the future RIBs be built in either Area 1 or Area 2, with a preference for Area 1. This is because Area 2 is more sloped, which would increase the effort required to maintain a horizontal bed surface.

Both Areas 1 and 2 will be carried forward as viable in this Class EA. Selection of the preferred area and design of the RIBs will need to be confirmed following a more detailed hydrogeological investigation.

10.3.2 Cost Estimate

Table 10-3 summarizes cost estimates for the RIB expansion described above.

Table 10-3: Summary of RIB Expansion Cost Estimate

	Total Cost ⁽¹⁾ (excl. HST, 2024 Dollars)	Annual Operating Cost
RIB Expansion	\$1,799,000 - \$3,591,000	Same for all options

Table 10-3 Notes:

- 1) OPC includes costs for installation, construction, Contractor markup, contingency (30%), and engineering costs (15%).
- 2) Inflation rate of 3.1% assumed to convert capital cost from 2023 to 2024-dollar values.

10.4 Development of Alternative Design Concepts

Section 10.2 summarized the different types of treatment technologies that will be evaluated to determine the preferred design concept. The following three (3) alternative design concepts were developed:

- OPTION 1: Manual Headworks + Upgraded Aeration + RIB Expansion
- OPTION 2: Mechanical Headworks + Upgraded Aeration + RIB Expansion
- OPTION 3: Mechanical Headworks + Advanced Treatment + RIB Expansion

Each alternative design concept is summarized in the following sub-sections. A preliminary conceptual design has been developed for each of these options. Note that, for the purpose of evaluating these options, it was assumed that each specialized treatment system and aerated lagoon would need to be designed to be able to achieve a minimum 50% increase in rated flow of 2,580 m³/d as determined in Phase 2. It is noted that for all options, there are various WWTP configurations that can be explored to optimize the design flow of the specialized treatment systems.

A Class D opinion of probable cost (OPC) capital cost estimate was prepared for each option based on available information, experience on similar projects and professional judgement.

Note that a 30% contingency was added to the cost estimates based on the below definition of a Class D cost estimate:

- Definition of Work: A description of the option with such supporting documentation as is available (definition of project typically in the order of 1 to 5 percent).
- Intended Purpose: To aid in the screening of alternative potential design concepts prior to recommending a preferred design concept (not intended to establish or confirm budgets).
- Level of Effort: Is limited and expected accuracy could range from -30% to +30%.
- Dollar Value: 2024.

Specific technologies for headworks and lagoon upgrades will be confirmed during detailed design. These OPCs have been prepared for guidance in project evaluation and implementation from the information available at the time of the estimate. The final project cost will depend on actual labor and material costs, competitive market conditions, final project scope, implementation schedule and other variable factors. As a result, the final project cost will vary from the OPC presented herein. Because of this, project feasibility and funding needs must be carefully reviewed prior to making specific financial decisions to help ensure proper project evaluation and adequate funding.

An approximate annual cost was also developed for the projected annual energy consumption and projected annual chemical consumption of each option. This cost will be included in the evaluation table.

10.4.1 OPTION 1: Manual Headworks + Upgraded Aeration + RIB Expansion

Option 1 presents a potential conceptual design wherein the existing aeration system is refurbished, and the RIB system is expanded to accommodate the increase in flow. A new manual screening area, package septage receiving station and access road improvements are included.

The following items are assumed to be included as part of Option 1:

- Installation of a manual bar screen on the inlet to Lagoon Cell No. 1.
- Installation of a package septage receiving station with integrated pre-treatment discharging to manual bar screen area.
- Replacement of existing fine bubble aeration in Lagoon Cells No. 1 and No. 2 with 52 and 28 diffusers respectively.
- Replacement of existing Lagoon blowers with two common 60 HP blowers (one duty and one standby), supplying air to both Lagoons.
- Construction of four additional winterized RIBs
- Miscellaneous access road improvements.

Based on the above, a Class 'D' cost estimate of \$6,522,000 to \$9,836,000 (excluding HST) was developed for this option. This option is projected to also require annual operating and maintenance costs of approximately \$114,000 to \$236,000/year, excluding the RIB units (common to all options).

10.4.2 OPTION 2: Mechanical Headworks + Upgraded Aeration + RIB Expansion

Option 2 presents a potential conceptual design wherein the existing aeration system is refurbished, and the RIB system can be re-rated to accommodate the increase in flow. A new headworks with mechanical screening, septage receiving station, and miscellaneous site improvements are included.

The following items are assumed to be included as part of Option 2:

- Installation of mechanical screening system on the inlet to Lagoon Cell No. 1.
- Construction of a headworks building with integrated septage receiving equipment, odour control and electrical room.
- Replacement of existing fine bubble aeration in Lagoon Cells No. 1 and No. 2 with 52 and 28 diffusers respectively.
- Replacement of existing Lagoon blowers with two common 60 HP blowers (one duty and one standby), supplying air to both Lagoons.
- Construction of four additional winterized RIBs.
- Miscellaneous access road improvements.

Based on the above, a Class 'D' cost estimate of \$6,713,000 to \$10,577,000 (excluding HST) was developed for this option. This option is projected to also require annual operating and maintenance costs of approximately \$81,000 to 137,000/year, excluding the RIB units (common to all options).

10.4.3 OPTION 3: Mechanical Headworks + Advanced Treatment + RIB Expansion

Option 3 presents a potential conceptual design wherein an advanced treatment system is installed, and the RIB system is expanded to accommodate the increase in flow. A new headworks with mechanical screening, septage receiving station, and miscellaneous site improvements are included.

The following items are assumed to be included in the advanced treatment system as part of Option 3:

- Addition of an attached-growth biological treatment system with new blowers to meet aeration requirements of the attached-growth system.
- Addition of a new post-lagoon filtration system for polishing of TSS and TP.
- Addition of a new building to house the blowers, filtration system and associated electrical equipment.
- Addition of an intermediate pumping station and other miscellaneous piping, chambers, and valves.

In addition, the following items will be a part of Option 3:

- Installation of mechanical screening system on the inlet to Lagoon Cell No. 1.
- Construction of a headworks building with integrated septage receiving equipment, odour control and electrical room.
- Construction of four additional winterized RIBs.
- Miscellaneous access road improvements.

Based on the above, a Class 'D' cost estimate of \$16,515,000 to \$28,888,000 (excluding HST) was developed for this option. This option is projected to also require operating and maintenance costs of approximately \$199,000 to \$401,000/year, excluding the RIB units (common to all options).

10.5 Evaluation of Alternatives

The alternative design concepts were evaluated based on the following financial and technical criteria:

- Financial – Capital Cost
- Financial – Operation and Maintenance Cost
- Technical – Ability to Reliably Achieve Effluent Requirements
- Technical – Constructability and Complexity of Construction

Each criterion was assigned a weighting from 1 to 5 to reflect its level of importance relative to other criteria. For each alternative design concept, scores from 0 to 4 were then assigned for each criterion. The following scoring system was followed when evaluating the options:

- 4 – Highly favorable design concept or exceeds criterium requirement.
- 3 – Favorable design concept or meets criterium requirement.
- 2 – Neither favorable or unfavorable design concept or partially meets criterium requirement.
- 1 – Less favorable design concept or barely meets criterium requirement.
- 0 – Unfavorable design concept or does not meet criterium requirement.

Refer to Table 10-4 for the full summary and final scores/ranks from the evaluation of the alternative design concepts/options. Based on the evaluation of the options with the Municipality, the preferred design concept for the Markdale WWTP upgrade is Concept Option No. 2.

Table 10-4 Evaluation of Design Concept Alternatives

Major Criteria	Minor Criteria	Weight (1-5)	Design Concept Option No.1 Manual Headworks + Upgraded Aeration + RIB Expansion		Design Concept Option No.2 Mechanical Headworks + Upgraded Aeration + RIB Expansion		Design Concept Option No.3 Mechanical Headworks + Advanced Treatment + RIB Expansion	
Financial	Capital Cost	4	Score: 4 Class 'D' capital cost estimate of \$6,522,000 to \$9,836,000.	16	Score: 3 Class 'D' capital cost estimate of \$6,713,000 to \$10,577,000.	12	Score: 1 Class 'D' capital cost estimate of \$16,515,000 to \$28,888,000.	4
	Operation and Maintenance Cost	5	Score: 2 O&M cost estimate of \$114,000 to \$236,000. Cost of operator labour is higher due to frequent screen cleaning and higher likelihood of aeration issues due to poorer headworks performance.	10	Score: 4 O&M cost estimate of \$81,000 to \$137,000. Cost of operator labour is lower than other alternatives given automated headworks and simple aeration technology.	20	Score: 1 O&M cost estimate of \$199,000 to \$401,000. Cost of operator labour is higher than Option No. 2 due to more complex treatment technology.	5
Technical	Ability to Reliably Achieve Effluent Requirements	5	Score: 2 Reduced reliability due to poorer anticipated performance of headworks.	10	Score: 3 Expected to reliably meet effluent requirements.	15	Score: 4 Expected to reliably exceed effluent requirements.	20
	Constructability and Complexity of Construction	3	Score: 4 Least complex; no dedicated headworks building construction required.	12	Score: 3 Construction of a headworks building for mechanical screen system is required in addition to fine bubble diffuser and blower installation.	9	Score: 2 Advanced treatment system requires additional land area and excavation compared to Options 1 and 2.	6
Total Score and Rank:			Rank # 2	48	Rank # 1	58	Rank # 3	35

11.0 DESCRIPTION OF THE PREFERRED ALTERNATIVE

The preferred alternative was Option 2, where the existing aeration system is refurbished, and the RIB system can be re-rated to accommodate the increase in flow. A new headworks with mechanical screening, septage receiving station, and miscellaneous site improvements are included, as follows:

- Installation of mechanical screening system on the inlet to Lagoon Cell No. 1.
- Construction of a headworks building with integrated septage receiving equipment, odour control and electrical room.
- Replacement of existing fine bubble aeration in Lagoon Cells No. 1 and No. 2 with 52 and 28 diffusers respectively.
- Replacement of existing Lagoon blowers with two common 60 HP blowers (one duty and one standby), supplying air to both Lagoons.
- Construction of four additional winterized RIBs.
- Miscellaneous access road improvements.

11.1 Class D Opinion of Probable Cost

The OPCC for the preferred alternative is summarized below in Table 11-1.

Table 11-1 Class D OPCC Summary for the Preferred Alternative

ITEM	OPCC (2024 Dollars)	
	Low	High
Headworks		
Mechanical screening	\$623,000	\$836,000
Headworks building with integrated septage receiving	\$870,000	\$1,269,000
Access road improvements	\$689,000	\$827,000
Engineering and Project Management	\$330,000	\$442,000
Contingency Allowance	\$655,000	\$880,000
Subtotal - Headworks	\$3,167,000	\$4,254,000
Process Upgrades		
Aeration diffusers and blowers	\$1,153,000	\$1,729,000
Misc. electrical, mechanical, structural	\$52,000	\$155,000
Engineering and Project Management	\$181,000	\$283,000
Contingency Allowance	\$361,000	\$565,000
Subtotal - Process Upgrades	\$1,747,000	\$2,732,000

ITEM	OPCC (2024 Dollars)	
	Low	High
Rapid Infiltration Basins (RIBs)		
RIB units	\$1,230,000	\$2,460,000
Misc. electrical and instrumentation (level monitoring)	\$10,000	\$15,000
Engineering and Project Management	\$187,000	\$373,000
Contingency Allowance	\$372,000	\$743,000
Subtotal - RIBs	\$1,799,000	\$3,591,000
Total Project Cost (2024 Dollars)	\$6,713,000	\$10,577,000

Class 'D' OPCCs developed for this assignment are expected to be within +/- 30%. The OPCCs were developed based on experience on similar projects, professional judgment, and equipment costs provided by suppliers. The final project cost will depend on actual labor and material costs, competitive market conditions, final project scope, implementation schedule and other variable factors. As a result, the final project cost will vary from the OPC presented herein. Because of this, project feasibility and funding needs must be carefully reviewed prior to making specific financial decisions to help ensure proper project evaluation and adequate funding.

This cost estimate is provided in 2024 dollars. It is not possible to ascertain future price escalations; however, by industry best practices escalation should be considered likely between a baseline date of January 2024 and the implementation of this project. A cost escalation rate should be applied once implementation timing is known.

11.2 Land Use and Environmental Considerations

The work to implement the proposed WWTP improvements will take place within the existing site, except for new RIB construction in Municipality owned land at the southwest of the existing site. Therefore, no land acquisition is anticipated.

Table 11-2 summarizes mitigation measures and next steps recommended by the studies and reports completed for this Class EA.

Table 11-2: Mitigation Measures and Next Steps

Category	Recommendations
Planning & Policy	The study area is not subject to the Oak Ridges Moraine Conservation Plan, Niagara Escarpment Plan, Greenbelt Plan, Lake Simcoe Protection Plan, and the Growth Plan for the Greater Golden Horseshoe.
Source Water Protection	The location of vulnerable areas, to protect the source water for municipal residential drinking water systems, will need to be considered during project implementation. The Markdale WWTP lies approximately 270 m west of the Town's well head protection areas (WHPAs).
Climate Change	<p>Upgrades and expansions of the WWTP may contribute to climate change through the production of greenhouse gas emissions such as GHG emissions from heavy vehicles during construction or biological waste produced by the facility. Other impacts of the WWTP on climate change include impacts on carbon sinks from construction impacting the landscape's ability to remove and store atmospheric carbon.</p> <p>This system of lagoons may be more vulnerable to climate change compared to facilities sheltered from the local climate. The WWTP may experience negative impacts to functionality and reliability due to changing climatic conditions. These include drought, flooding, sewer overflow, and ice storm damage.</p>
Air Quality, Dust, and Noise - During Construction	Air Quality, Dust, and Noise impacts related to construction will be assessed during the design phase and the appropriate studies will be undertaken if required. Dust and noise control measures should be addressed and included in the construction plans.
Air Quality, Dust, and Noise - During Operation	No significant changes to the WWTP operation is proposed, so no air, dust, or noise impacts are expected. It is recommended that an air quality impact assessment be conducted during design.
Ecosystem Protection & Restoration	Pinchin's Terrestrial Ecology Report recommended field studies to be completed prior to development. These will be completed to inform the design process. Once design is finalized, the relevant mitigation measures will be implemented. Refer to Section 4.6 for the recommended studies and mitigation measures.
Wildlife & Species at Risk	<p>Significant Wildlife Habitat (SWH) was identified on site. Chimney Swift, Bobolinks, and Eastern Meadowlarks were heard and observed foraging. Candidate SWH was identified on site for Bat Maternity Colonies, Amphibian Breeding Habitat, and Turtle Nesting Areas.</p> <p>Pinchin's Terrestrial Ecology Report recommended field studies to be completed prior to development. These will be completed to inform the design process. Once design is finalized, the relevant mitigation measures</p>

	will be implemented. Refer to Section 4.6 for the recommended studies and mitigation measures.
Natural Environment	<p>The study area does not contain any Natural Heritage System Core Areas or Linkages, any areas of natural and scientific interest (ANSIs) related to Earth or Life, or any Significant Valleylands. The RIBs border and have slight overlap over Significant Woodlands.</p> <p>Pinchin's Terrestrial Ecology Report recommended field studies to be completed prior to development. These will be completed to inform the design process. Once design is finalized, the relevant mitigation measures will be implemented. Refer to Section 4.6 for the recommended studies and mitigation measures.</p>
Surface Water	<p>Impacts to the Rocky Saugeen River and mitigation measures to protect watercourse from construction or operational activities (e.g., spills, erosion, pollution) are identified as part of the proposed undertaking.</p> <p>After consulting the Fisheries and Oceans Canada guide it was determined that this project does not need DFO review. Consultation with the MECP determined that no surface water impact assessment will be required, and the existing effluent limits and objectives continue to apply.</p>
Groundwater	<p>During construction, a Permit to Take Water (PTTW) under the OWRA will be required for any water takings that exceed 50,000 L/day. The need for this will be confirmed by studies in the design phase.</p> <p>There are three PTTWs from ground sources for communal water use, owned by the Municipality, within two km of the WWTP.</p>
Stormwater	To mitigate impacts from construction, quality and quantity control measures to treat stormwater runoff should be considered during design.
Excess Materials Management	All waste generated during construction must be disposed of in accordance with MECP requirements. Activities involving the management of excess soil should be completed in accordance with O. Reg. 406/19 and the MECP's current guidance document titled "Management of Excess Soil – A Guide for Best Management Practices" (2014).
Contaminated Sites	No known contaminated sites are in the study area. During construction, if the removal or movement of soils is required, appropriate tests to determine contaminant levels from previous land uses or dumping will be undertaken. Soil disposal, if required, will be consistent with Part XV.1 of the Environmental Protection Act (EPA) and Ontario Regulation 153/04, Records of Site Condition.
Servicing, Utilities, and Facilities	Hydro One confirmed that they have no existing distribution assets within the study area. Servicing, utilities, and facilities on the site will be identified prior to the commencement of design. The existing Environmental Compliance Approval (ECA) for the WWTP will be updated as part of the design process.

Geology and Hydrogeology	<p>A desktop hydrogeological assessment was completed by WSP in 2023, followed by fieldwork in 2025. WSP determined that of the three potential expansion areas, one was not suitable for RIB expansion. Of the remaining two, one is preferred, but the water level monitoring data from Spring 2026 should be reviewed prior to furthering the design. Refer to Section 10.3.1 for further details.</p> <p>Future detailed hydrogeological assessment should confirm design of the RIB expansion and new RIBs. This study should further confirm that effluent entering shallow groundwater from RIBs will not pass underneath the river.</p>
Archaeological Resources	<p>A Stage 1 and Stage 2 AA were completed by ARA and identified no archaeological sites in the study area.</p>
Traffic	<p>No traffic impacts are expected.</p>

Based on the work completed as part of this Class EA, studies of the expansion area to be undertaken prior to the construction of the WWTP expansion include:

- Hydrogeological studies to support design
- Geotechnical studies to support design
- Amphibian, Snag & Stick Nests, Turtle Basking & Nesting, and other surveys as identified in the Terrestrial Ecology Report
- Landscape Plan (for habitat compensation off site), Landscape Planting Plan (for restoration on site), and Monitoring Plan

12.0 CONSULTATION

12.1 Notices

A Notice of Study Commencement, provided in Appendix D, was prepared by the consulting team, and issued on September 7, 2023. It was sent directly to the mailing list, posted on the Municipality's website, and shared on their social media on the 7th, as well as printed in the local newspaper on the 13th and 20th. It notified the public and other stakeholders of both the project commencement and the first Public Information Centre (PIC).

A second notice was issued on November 20, 2025 to announce the date of the second PIC, provided in Appendix D. It was sent to the mailing list, posted on the Municipality's website, and shared on their social media, as well as printed in the local newspaper.

A project mailing list was developed identifying Indigenous communities, review agencies, and stakeholders requiring consultation. A copy of the mailing list is provided in Appendix D.

12.2 Public Information Centres

The first PIC was held in Markdale on September 28, 2023 and was attended by six visitors. One agency comment sheet was submitted by the Municipality of West Grey after the PIC, which is summarized in Table 12-5.

The second PIC was held in Markdale on December 11, 2025 and was attended by four visitors. No comment sheets were completed by visitors, however their discussions with the project team at the PIC are summarised in Table 12-5.

12.3 Consultation with Indigenous Communities

The MECP identified that the Métis Nation of Ontario (MNO) and the Saugeen Ojibway Nation (SON), including Saugeen First Nation (SFN) and the Chippewas of Nawash Unceded First Nation (CNUFN), required consultation during this Class EA.

Table 12-1 and Table 12-2 provide a summary of consultation with SON and MNO regarding this Class EA.

Table 12-1: Consultation with Saugeen Ojibway Nation (SFN and CNUFN)

Date and Method	Communication	Response
September 7, 2023 email to SFN and CNUFN	JLR provided Notice of Commencement	None.

Date and Method	Communication	Response
November 20, 2025 email to SFN and CNUFN	JLR provided Notice of PIC No. 2	None.

Table 12-2: Consultation with Métis Nation of Ontario

Date and Method	Communication	Response
September 7, 2023 email to MNO	JLR provided Notice of Commencement	None.
November 20, 2025 email to MNO	JLR provided Notice of PIC No. 2	None.

The following Indigenous communities were consulted as part of the previous Master Plan and therefore continued to be consulted during this Class EA:

- Chippewas of Georgina Island (CGI)
- Chippewas of Rama First Nation (CRFN)
- Great Lakes Métis Council (GLMC)
- Historic Saugeen Métis (HSM)

Table 12-3 provides a summary of consultation with these Indigenous communities regarding this Class EA. Refer to Appendix D for a copy of written correspondence from these groups.

Table 12-3: Consultation with CGI, CRFN, GLMC, and HSM

Community	Date and Method	Communication
CGI, CRFN, GLMC, and HSM	September 7, 2023 email from JLR	JLR provided Notice of Commencement
CGI, CRFN, GLMC, and HSM	November 20, 2025 email from JLR	JLR provided Notice of PIC No. 2
Historic Saugeen Métis (HSM)	December 11, 2025 email from HSM	HSM acknowledged receipt of PIC No. 2 notice and that they have no concerns at this time.

Indigenous communities were also consulted separately by ARA as part of the Archaeological Assessments.

12.4 Comments

Table 12-4 below provides a summary of public comments received regarding this Class EA. Refer to Appendix D for written correspondence received from the public.

Table 12-4: Comments from the Public

Stakeholder	Comments	Response
No formal comments were submitted by the public as of the present date.		
Attendees of second PIC on December 11, 2025.	<p>Informal comments made during the PIC by the public include:</p> <ol style="list-style-type: none"> 1. Requested more frequent communication with the public. 2. Concerns include the possibility of encroachment on surrounding properties by the WWTP expansion and the project cost, particularly cost recouperation and impact on residents. 3. Questions regarding whether the additional WWTP capacity is designed for short or long term demands. 	<ol style="list-style-type: none"> 1. Public was invited to be added to the mailing list. Project team will confirm consultation strategy. 2. Expansion of the WWTP is planned to be within the lands currently owned by the WWTP. The WWTP expansion is required for increased development, it is anticipated new development will offset the costs on residents. 3. Expansion is based on 1,075 units of known or expected future development.

Table 12-5 provides a summary of agency comments received regarding this Class EA. Refer to Appendix D for written correspondence from these groups.

Table 12-5: Comments from Review Agencies

Agency	Date and Method	Comment	Response
Ministry of the Environment, Conservation, and Parks (MECP)	September 19, 2023 pre-consultation meeting followed by emails dating to January 9, 2024.	Refer to Section 8.4	
	November 28, 2024, letter by email	Acknowledgement of Notice of Commencement	Noted.
	March 10, 2026 by email	Submitted draft Project File Report for MECP Pre-review	N/A
Ministry of Natural	Sept 14, 2023, letter by email	Acknowledged receipt of notices. Provided guidance on identifying and	Reviewed.

Agency	Date and Method	Comment	Response
Resources (MNR)	Dec 16, 2025, letter by email	assessing natural features and resources as required by applicable policies and legislation.	
Grey County	Sept 25, 2023, letter by email.	Acknowledged receipt of notices. Forestry and Trails Services raised concerns about drainage impacts to the Rail Trail and requested an opportunity to comment on further studies.	Acknowledged . Will be addressed in future hydrogeological studies and reporting.
	Dec 5, 2025, email.		
Hydro One	Sept 28, 2023 letter by email.	Confirmed that Hydro One has existing distribution assets within the study area. Requested to continue to be notified.	Noted.
	January 30, 2026 letter by email.	Confirmed that Hydro One has no existing distribution assets within the study area. Requested to continue to be notified.	Noted.
Municipality of West Grey	Sept 29, 2023 comment form.	Requested to continue to be notified.	Noted.
Ministry of Citizenship and Multiculturalism (MCM)	October 4, 2023	Provided guidance regarding identifying archaeological, cultural, and heritage resources, including screening forms that need to be completed.	Completed forms with Archaeology Ontario input. Refer to Appendix B.
Saugeen Valley Conservation Authority (SVCA)	May 6, 2024 letter by email.	Construction of any additional RIBs would require written permission from the SVCA under Ontario Regulation 169/06, as amended. The presence and boundary of any wetland will have to be confirmed in the field, therefore, a SVCA visit to the site should be arranged.	Noted.

Agency	Date and Method	Comment	Response
Ministry of Transportation (MTO)	November 21, 2023	Confirmed that the project is in accordance with the requirements of the Public Transportation and Highway Improvement Act, MTO's Highway Access Management Policy and all related policies. Noted that if any proposed work is within the MTO highway property limits, or within MTO's Permit Control Area, then MTO review, approval and permits will be required before any alteration commences. Requested to continue to be notified.	Noted.

Table 12-6 provides a summary of comments from developers regarding this Class EA.

Table 12-6: Comments from Developers

Developer	Comments	Response
No comments from developers on the Class EA as of present date.		

13.0 LIMITATIONS

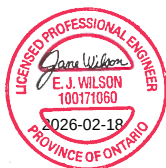
This Report has been prepared for the exclusive use of The Municipality of Grey Highlands, for the stated purpose. Its discussions and conclusions are summary in nature and cannot be properly used, interpreted, or extended to other purposes without a detailed understanding and discussions with the client as to its mandated purpose, scope, and limitations. This report was prepared for the sole benefit and use of the Municipality of Grey Highlands and may not be used or relied on by any other party without the express written consent of J.L. Richards & Associates Limited.

Cost estimates are based on historic construction data and does not include labour, material, equipment, manufacturing, supply, transportation, or any other cost impacts in relation to COVID-19. JLR shall not be responsible for any variation in the estimate caused by the foregoing factors but will notify the Client of any conditions which JLR believes may cause such variation upon delivery of the estimate.

This report is copyright protected and may not be reproduced or used, other than by the Municipality of Grey Highlands for the stated purpose, without the express written consent of J.L. Richards & Associates Limited.

J.L. RICHARDS & ASSOCIATES LIMITED

Prepared by:



Jane Wilson, P.Eng.
Senior Environmental Engineer

Prepared by:



Carolyn Chan, P.Eng.
Senior Environmental Engineer

14.0 REFERENCES

Chapman's Ice Cream. *10 Years after the fire at Chapman's: From ashes to ice cream*, May 2019.

Chapman's Ice Cream. *Chapman's commitment to a greener future*, N.D.

Chapman's Ice Cream. *The inside scoop on our history*, N.D.

GENIVAR Inc. *Background Information and Action Plan for Provincial Prevention Order*, September 2011.

GENIVAR Inc. *Optimization of the Markdale Wastewater Treatment Plant*, April 2012.

GSS Engineering Consultants Ltd. *Evaluation of Markdale Water & Sewage Use for New Residential Development and Chapman's Ice Cream*, September 2017.

Henderson Paddon & Associates. *Assessment of Chapman's Ice Cream Plant Sanitary System Loading*, June 1997

Terrapin Water. *Markdale Blue Frog System Analysis*, 2016.

WSP Canada Inc. *Water & Wastewater Servicing Master Plan*, December 2017.

WSP Canada Inc. *Operations Manual – Markdale Wastewater Treatment Plant and Sewage Pumping Stations*, May 2022.

McPherson A. *Microplastics Entering Severn Sound Watershed by Wastewater Effluent*, July 2019.

MECP. *Design Guidelines for Sewage Works*, 2008.

MECP. *Environmental Compliance Approval Number 5708-A72SPG*, 20 July 2016.

MECP. *Procedure F-5-5 Determination of Treatment Requirement for Municipal and Private Combined*, 9 March 2016.

Ministry of Municipal Affairs and Housing. *Growth Plan for the Greater Golden Horseshoe*, May 2017.

United States Environmental Protection Agency. *Guide to Septage Treatment and Disposal*, September 1994.

Appendix A

Terrestrial Ecology Report (Pinchin,
2024)

Appendix B

MCM Screening Forms and
Archaeological Assessment Reports
(ARA)

Appendix C

Hydrogeological Baseline
Assessments (WSP, 2023 & 2026)

Appendix D

Consultation Records

www.jlrichards.ca

Ottawa

864 Lady Ellen Place
Ottawa ON Canada
K1Z 5M2
Tel: 613 728-3571

ottawa@jlrichards.ca

Kingston

203-863 Princess Street
Kingston ON Canada
K7L 5N4
Tel: 613 544-1424

kingston@jlrichards.ca

Sudbury

314 Countryside Drive
Sudbury ON Canada
P3E 6G2
Tel: 705 522-8174

sudbury@jlrichards.ca

Timmins

834 Mountjoy Street South
Timmins ON Canada
P4N 7C5
Tel: 705 360-1899
timmins@jlrichards.ca

North Bay

501-555 Oak Street East
North Bay ON Canada
P1B 8L3
Tel: 705 495-7597

northbay@jlrichards.ca

Hawkesbury

326 Bertha Street
Hawkesbury ON Canada
K6A 2A8
Tel: 613 632-0287

hawkesbury@jlrichards.ca

Guelph

107-450 Speedvale Ave. West
Guelph ON Canada
N1H 7Y6
Tel: 519 763-0713

guelph@jlrichards.ca

