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STORMWATER MANAGEMENT REPORT

FOR

TIFFANY ROX NORTH HILL LTD.

14337 HIGHWAY 50

TOWN OF CALEDON
REGIONAL MUNICPALITY OF PEEL

PROJECT NO. 25-1433

MARCH 2026

FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT FOR TIFFANY ROX NORTH HILL LTD.

TOWN OF CALEDON
REGIONAL MUNICPALITY OF PEEL

MARCH 2026

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FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT FOR TIFFANY ROX NORTH HILL LTD.

TOWN OF CALEDON
REGIONAL MUNICPALITY OF PEEL

MARCH 2026

1.0 INTRODUCTION

David Schaeffer Engineering Ltd. (DSEL) has been retained by Tiffany Rox North Hill Ltd. to prepare a Functional Servicing and Stormwater Management Report in support of the Draft Plan of Subdivision (Draft Plan) submission for the Tiffany Rox North Hill Ltd. property, herein referred to as the "Subject Property". The Subject Property is within the Bolton North Hill Secondary Plan Area, located in the Town of Caledon and is approximately 19.05 hectares. As illustrated in ***Figure 1***, the lands are bounded by existing agricultural lands the north, Greenbelt lands to the east, existing agricultural lands to the south, and Highway 50 to the west. The Draft Plan for the Subject Property is illustrated on ***Appendix A*** and ***Figure 2***. The Subject Property and surrounding Landowners' properties are illustrated on ***Figure 3***.

The Subject Property is located within the Regional Official Plan Amendment 30 Area (ROPA 30). The Subject Property is proposed to be developed for residential and related purposes. The proposed draft plan includes single detached homes, townhouses, a neighborhood park, municipal rights-of-ways, and a (partial) stormwater management block.

The following report is submitted in support of the Draft Plan approval application and is intended to demonstrate the availability of existing and proposed municipal services to service the Subject Property in general conformance with the precedent Secondary Plan studies. The report will further demonstrate general conformance with Town of Caledon and Regional Municipality of Peel design criteria, the requirements of Toronto and Region Conservation Authority (TRCA), and general industry best practice.

2.0 PREVIOUS STUDIES AND REPORTS

The following background reports and design guidelines have been reviewed in preparation of this report, and are listed in ***Table 2-1.***

Table 2-1: Background Studies and Guidelines

RESOURCES
Town of Caledon Development Standards Manual Town of Caledon, 2019 ***(Town Standards)***
Toronto and Region Conservation Authority Stormwater Management Criteria Toronto and Region Conservation Authority, August 2012 **(TRCA SWM Guidelines)**
Region of Peel Public Works Design, Specifications & Procedures Manual, Linear Infrastructure, Watermain Design Criteria Region of Peel, June 2010 ***(Watermain Design Criteria)***
Region of Peel Linear Wastewater Standards Region of Peel, December 2025 ***(Wastewater Design Criteria)***
Stormwater Management Planning and Design Manual Ministry of Environment, March 2003 ***(SWMP Manual)***
Low Impact Development Stormwater Management Planning and Design Guide Toronto and Region Conservation Authority, 2011 ***(LID Design Manual)***
Functional Servicing and Stormwater Management Report – Bolton North Hill Option 1 Lands C.F. Crozier & Associates Inc., March 2026 ***(CFCA* FSSWMR*)***
Stormwater Management Planning and Design Manual Ministry of the Environment, Conservation & Parks, March 2003 ***(SWMP Design Manual)***

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3.0 LAND USE & DEVELOPMENT PHASING

3.1 Existing Land Use

The existing property is currently being used primarily for agricultural purposes. There is an existing nursery (Johns Nursery Garden) located in the southwest corner of the property limit.

3.2 Proposed Land Use

The Subject Property is proposed to be developed for residential and related purposes. The Draft Plan, illustrated in ***Appendix A***, includes the following land uses listed in ***Table 3-1*** separated between proposed land uses.

Table 3-1: Summary of Proposed Land Uses

LAND USE	AREA (Ha)	UNITS
Single Detached	5.72	180
Townhouse	2.99	129.5
Dual Frontage Towns	0.27	6
Stormwater Management Block	0.18	
Parkland	0.53	
Open Space	4.94	
Future Development	0.02	
0.3m Road Widening	0.015	
22.0m Right of Way	0.57	
20.0m Right of Way	0.44	
18.0m Right of Way	3.30	
16.0m Right of Way	0.07	
TOTAL	**19.05**	**315.5**

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4.0 STORM DRAINAGE

The Subject Property will be developed using the strategies and guidelines set in the ***CFCA FSSWMR*** for addressing stormwater runoff generated by the proposed development, consisting of:

- the application of source control and LID techniques as appropriate, preferentially located in public areas with appropriate soil conditions and groundwater to provide stormwater quality control and enhance the water balance;
- conveyance techniques as appropriate; and
- end of pipe stormwater management ponds to provide water quantity, quality, and erosion volume control.

The following sections describe the pre-development drainage pattern, the proposed drainage boundaries and strategy, and approach to stormwater management for the Subject Property.

4.1 Existing Drainage Conditions

The Subject Property is located within two tributaries of the Humber River watershed; Tributary D located west of the Subject Property, and Tributary E located east of the Subject Property. The Subject Property generally consists of agricultural row crops with a large open aquatic pond and wetland associated within Tributary E. There is also an existing residential property and John's Nursery Garden located in the southern property limit. Several Head Water Drainage Features (HDFs) were identified by Dillon Consulting which generally consist of small swale corridors throughout the Subject Property.

Approximately one third of the storm runoff from the Subject Property drains westerly towards Highway 50 via overland flow and is captured by the Highway 50 roadside ditch. The roadside ditch conveys the flows to a 1220 mm x 960 mm concrete box culvert, running east to west beneath Highway 50, and ultimately to Tributary D of the Humber River watershed. The remaining two thirds of the Subject Property drains east via overland flow towards the Greenbelt area and ultimately to Tributary E of the Humber River Sub-watershed.

The pre-development storm drainage patterns for the Subject Property are illustrated in ***Figure 4***.

4.2 Proposed Drainage Conditions

Post-development drainage areas have been regularized to reflect the draft plan and with due consideration for property ownership, community phasing, and considerations for existing drainage patterns entering the subject property.

A conventional storm sewer system and major system conveyance network is designed to direct storm flows safely through the catchment to designated stormwater management facilities for treatment where appropriate. The proposed conditions drainage strategy is illustrated in ***Figure 5.***

There are two proposed stormwater management ponds proposed for the subject site. The post development drainage divide is proposed to split near the center of the Subject Property.

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Major and minor system flows from 34.45 ha, is conveyed towards a proposed stormwater management (SWM) pond 7 located at the northwest corner of the subject lands. Flows are conveyed via storm sewers and right-of-ways to the SWM pond, which ultimately discharges to Tributary D, west of Highway 50 via a new storm outfall.

Major and minor system flows from 17.66 ha, is conveyed towards a proposed stormwater management (SWM) pond 9 located just outside of the subject lands, in the southeast corner. Flows are conveyed via storm sewers and right-of-ways to the SWM pond, which ultimately outlets to the Natural Heritage System / Greenbelt, and Tributary E.

4.2.1 Drainage Area Diversion

As noted in ***Section 4.1***, the subject property is located within two tributaries of the Humber River. Proposed drainage patterns were established while trying to maintain existing drainage patterns. The proposed drainage boundaries remain consistent with the ***CFCA FSSWMR***.

4.2.2 Boundary Road Drainage and External Drainage

The Subject Property is bounded by existing agricultural lands to the north, Greenbelt lands to the east, existing agricultural lands to the south and Highway 50 to the west. Drainage from the boundary roads and external lands are summarized below:

- **Highway 50** – Road drainage from Highway 50 is contained within the existing roadside ditches located on either side of Highway 50 and does not enter the Subject Property. Flows are directed via existing culverts towards Tributary D.

- **Greenbelt Lands** – Stormwater runoff is conveyed southeast via overland flow towards Tributary E. Flows do not enter the Subject Property.

- **Agricultural Lands (North of Subject Property)** – External drainage from lands to the north is conveyed via overland flow into the Subject Property. These lands are part of the Bolton North Hill Secondary Plan Area and are expected to be developed and constructed in a similar timeframe to the Subject Property. As a result, the lands have been assumed to be fully developed, to ensure the proposed infrastructure and stormwater ponds are sized appropriately. Minor system flows will be captured via the proposed internal storm system and conveyed to one of the two ponds located within the Subject Lands. Major system flows will be conveyed through the proposed right-of-ways to one of the proposed ponds. External drainage boundaries are consistent with the ***CFCA FSSWMR.***

- **Agricultural Lands (South of Subject Property**) – External drainage from lands to the south does not enter the Subject Property and is conveyed south via overland flow to either Tributary D or E of the Humber River. It is important to note that these lands are also part of the Bolton North Hill Secondary Plan Area and are expected to be developed and constructed in a similar timeframe to the Subject Property. Once developed, flows will be conveyed and captured via the proposed internal storm system and conveyed to proposed SWM pond 9. For design purposes, it has been assumed that the lands to the south have been fully developed, to ensure the proposed infrastructure and stormwater ponds are sized appropriately.

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- **14289/14291 Highway 50 (Holdout Properties)** – These two properties located in the southwest corner of the Subject Property are hold out properties and the timeframe of development is unknown at this point. The proposed infrastructure and ponds have been sized to handle these properties as if they were fully developed.

4.3 Conveyance of Minor System Flows

The Subject Property will be serviced by a conventional storm sewer system designed in accordance with ***Town Standards***. Per ***Town*** Standards, systems where foundation drains are connected to the storm sewer, the Town requires the system to be designed to accommodate a 10-year storm event and has been applied for this site.

Sump pumps will be required in areas of shallow storm sewers or where 0.3 m of freeboard is not achieved from the 100 year / Regional hydraulic grade line to the underside of footing elevation. The sump pump outlet locations will be established through detailed design and will require back flow preventers or gooseneck connections to discharge to the storm sewer.

Storm flows will be directed to one of the two proposed stormwater management facilities in the study are, where the runoff will be treated for water quality, erosion, and quantity control.

The conceptual storm servicing plan is provided in ***Drawing 3*** and the overall drainage plan identifying catchment areas to the respective ponds is provided in ***Figure 5.*** Preliminary storm design sheets are included in ***Appendix B***.

4.4 Conveyance of Major Storm Flows

A continuous overland flow route will be provided through the Subject Property to safely convey major system flows in excess of the minor system and up to the 100-year event. Overland flow routes will be directed to one of two stormwater management ponds located within the Subject Property.

The major system flow will be contained within the road right-of-way and will not cross private property. The product of depth of water (m) at the gutter times the velocity of flow (m/s) shall not exceed 0.65 m^2/s in accordance with ***Town Standards***. Should the major system flow exceed the conveyance capacity of any given road, the storm sewer will be sized to accommodate the flows in excess of the road capacity. The major system flows will be attenuated in the stormwater management ponds to achieve the allowable release rates as defined in ***Section 5.1.*** The conceptual major storm system is illustrated on ***Figure 5***.

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5.0 STORMWATER MANAGEMENT

Stormwater management for the subject lands will be accommodated by two (2) stormwater management ponds as described in ***Section 4.2***. Each pond services a distinct development area and all facilities are proposed as wet ponds as further described below.

Pond 7 is located northwest of the Subject Property. The pond is situated near the low point (and existing Highway 50 culvert) of the area and is bounded by the future residential development and by Highway 50. Monitoring well MW8 was drilled and installed by Crozier in November 2021 within the footprint of Pond 7, refer to details in ***Appendix C***.

Pond 9 is located southeast of the Subject Property. The pond is situated near the existing low point of the Subject Lands that drain towards Tributary E. Monitoring Well MW4 was installed in close proximity of Pond 9 by Crozier in November 2021, refer to details in ***Appendix C***.

The following sections outline the preliminary details for Ponds 7 and 9 within the subject property.

5.1 Design Criteria

In accordance with the ***Town Standards***, ***TRCA SWM Guidelines***, and the ***SWMP Design Manual***, stormwater management must be practiced as follows:

Water Quality Control
- Sized in accordance with the ***SWMP Design Manual*** for enhanced water quality protection, 80% TSS Removal from 90% of the total annual runoff

Water Quantity Control
- Post development flow rates must be controlled to the target release rates described in the Functional Servicing and Stormwater Management Report (**CFCA FSSWMR).**

Erosion Control
- Runoff volume generated from a 4-hour, 25 mm storm event to be discharged over a period of 48 hours.

All facilities will be designed to meet the criteria in Section 4.2.1 of the TRCA's Approaches to Manage Regulatory Event Flow Increases resulting from Urban Development (TRCA, 2016), where applicable. Facilities will also be designed to generally meet the criteria in Appendix E of the TRCA's Stormwater Management Criteria (TRCA, 2012), where applicable.

Stormwater treatment for the Subject Property will be provided by the proposed SWM ponds. Target release rates for the proposed SWM ponds were established in the ***CFCA FSSWMR***, refer to details in ***Appendix D*.**

Target release rates to SWM ponds 7 and 9 are summarized in ***Table 5-1*** and ***Table 5-2*** respectively. It is noted in the ***CFCA FSRSWMR*** that the Pond 9 controls are based on the TRCA Humber River Watershed Sub-Basin 10 unit flow rates however are also required to provide Regional controls.

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Table 5-1: Quantity Control - Target Release Rates Pond 7

Storm Event	Target Release Rates (L/s)
2-year	38.0
5-year	80.0
100-year	979.0
Regional	2,567.0

Table 5-2: Quantity Control – Target Release Rates – Pond 9

Storm Event	Target Release rate (L/s)
2-yr	16.7
5-yr	37.1
10-yr	94.1
25-yr	124.7
50-yr	151.0
100-yr	178.9
Regional	910.3

5.2 Operating Characteristics

The SWM ponds will be designed in accordance with the requirements of the ***Town Standards***, ***TRCA SWM Guidelines***, the ***SWMP Design Manual***, and include the following features:

Sediment Forebay ➢ to improve sediment removal prior to entering the pond

Permanent Pool ➢ to buffer storm flows and trap pollutants

Extended Detention Storage ➢ to provide water quality and erosion control

Quantity Control Storage ➢ to attenuate post development flows to the allowable release rates

All ponds are designed as wet pond facilities. A summary of the ponds is detailed in ***Table 5-3***.

Table 5-3: Summary of Required Pond Storage Characteristics

SWM Type	Minor/ Major System Drainage Area (ha)	Minor/ Major System Imp. Coverage (%)	Permanent Pool Volume[1] (m^3)	Erosion Control Volume [2] (m^3)	100 Year Flood Control Volume [3] (m^3)	Regional Year Flood Control Volume [3] (m^3)
Pond 7	34.45	74.3	6,603	6,690	20,994	28,938
Pond 9	17.66	74.3	3,385	3,480	13,094	21,925

[1] Interpolated from ***SWMP Design Manual***, Table 3.2
[2] Based on 25mm storm runoff volume with a 48-hour drawdown time
[3] As modeled in PCSWMM

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The impervious coverage has been estimated based on the various land uses and their respective sizes in the Draft Plan. The drainage area and imperviousness to the SWM Ponds have been refined as part of this site-specific analysis. Please note that the impervious coverage will be updated through detailed design based on the characteristics of the final plan, and the pond sizing adjusted accordingly. Details of the pond components are discussed in ***Section 5.3***

5.3 Pond Components

5.3.1 Sediment Forebay

A sediment forebay is provided in each pond to improve the pollutant removal by trapping larger particles near the inlet of the pond. The forebays are designed with a length to width ratio of approximately 3:1 and do not exceed one third of the permanent pool surface area, as required in the ***SWMP Design Manual***. Detailed sediment forebay sizing calculations will be provided at detailed design.

5.3.2 Permanent Pool

The permanent pool has been sized to provide an enhanced level of protection in accordance with the ***SWMP Design Manual***. Based on the imperviousness of the drainage area to the pond, the required and provided permanent pool volumes are summarized in ***Table 5-4***.

Table 5-4: Summary of Permanent Pool Storage

Pond I.D.	Drainage Area (ha)	Imp. Coverage (%)	Volume Required[1] (m^3)
Pond 7	34.45	74.3	6,603
Pond 9	17.66	74.3	3,385

[1]Permanent pool and quality control provided for MOE enhanced protection (Wet Pond) SWMP Design Manual, Table 3.2.

5.3.3 Extended Detention

Stormwater erosion criteria for proposed SWM facilities were established based on the TRCA SWM Criteria (2012) and MOE (2003) requirement for extended detention volume based on detention of the 25mm storm event over a period of 48 hours. This level of design was sufficient to develop preliminary sizing of stormwater facilities in support of the draft plan.

The resulting pond characteristics are summarized in Table 5-5:

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Table 5-5: Extended Detention / Erosion Control

Pond I.D.	Drainage Area (ha)	Imp. Coverage (%)	Volume Required[1] (m3)
Pond 7	34.45	74.3	6,690
Pond 9	17.66	74.3	3,480

[1] Based on 24-48-hour drawdown time for the 25 mm storm runoff volume.

The extended detention volume within the pond will outlet through a reverse graded pipe at the permanent pool elevations. An orifice will be provided to discharge the extended detention volume at the allowable release rate. When used in connection with a perforated pipe outlet configuration, the minimum orifice size as per the ***SWMP Design Manual*** is 50mm. If this is not possible, an alternative option such as using a custom inlet control device (e.g. Hydrovex) may be reviewed at the detailed design stage.

5.3.4 Quantity Control

Flood control for the subject site is to be provided for 2-to-100 year and Regional Storm events based on the target release rates in ***Section 0***.

The ponds have been modelled in PCSWMM to determine the storage volumes required to achieve the target peak outflow rates for each storm event. A summary of the required volume is presented in ***Table 5-6***, with further details provided in ***Appendix E.***

Table 5-6: Quantity Control - Target Storage Volumes

Storm Event	Post-Development Area	
	Required Storage Volume (m^3)	
	Pond 7	Pond 9
2-year	9,171	4,848
5-year	13,831	7,297
10-year	-	8,349
25-year	-	10,342
50-year	-	11,652
100-year	20,994	13,094
Regional	28,938	21,925

A drop inlet structure will be provided at the pond outlets, which restricts flows to the required rates for 2- to 100-year and Regional storm events by a combination of orifices and/or weirs. The outlet pipe will be sized such that the full flow capacity of the pipe exceeds the maximum regional pond outflow for each pond.

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5.3.5 Emergency Overflows

In the event of a blockage or a storm greater than the design horizon, an emergency overflow weir will be provided in both ponds. An overflow weir will be provided through detailed design to convey flows in excess of the Regional storm to the SWM Ponds respective tributaries.

Pond 7 will be provided with an overflow pipe to convey the emergency overflow under Highway 50 to avoid flooding the road in accordance with Regional standards.

5.3.6 Access Road

A 5.0 m wide access road will be provided for the to allow access for maintenance of the SWM ponds. The access road entrance will be provided off Street '4' for Pond 7, and Street '11' for Pond 9, respectively.

The pond access roads will connect to municipal right-of-ways, and will allow for access to the outlet structure, inlet structures, forebay and main cell. The maintenance access road will be configured such that two points of entry are provided where possible. The access road shall be situated in a manner that allows trucks to drive around the pond without having to turn around; alternatively, the access road may incorporate a turning circle (minimum radius 12.0m) where two access points are not provided.

5.4 BMP Pollutant Removal Efficiencies

As noted in the ***Town Standards***, Best Management Practices (BMP) are used to provide water quality control and should be in accordance with the ***MECP guidelines*** and CLI-ECA requirements. The Subject Property will be provided with a treatment train approach to provide water quality treatment. Such treatments include the following:

- Disconnected roof leaders
- Increased topsoil depths

These types of surface LID techniques promote natural infiltration simply by providing additional water volumes in the pervious areas (i.e., these areas would receive precipitation as well as extra water from roof runoff). While these techniques are not credited through the CLI-ECA program, they can provide a water quantity benefit.

In addition, LID techniques that would be credited through the CLI-ECA program involve publicly owned infrastructure that may also provide water quality benefits as part of BMP.

For more details on LIDs, please refer to ***Section 6.0***. Further discussion on the CLI-ECA program can be read under ***Section 6.3***.

5.5 TSS Removal

The proposed SWM Ponds adheres to the ***SWMP Design Manual***, and designed to provide 80% total Suspended Solids (TSS) removal. Additional TSS removal can be achieved through

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infiltration trenches, increased topsoil depth, and disconnected rooftop leaders as mentioned in ***Section 5.5***.

5.6 Thermal Mitigation

Effluent from ponds can experience increased temperatures due to solar exposure prior to discharging from the facilities. The stormwater management concept is to consider opportunities to reduce thermal inputs to the receiving watercourses. General guidance on opportunities and implementation of thermal mitigation practices can be found in the Thermal Impacts of Urbanization including Practices and Mitigation Techniques (CVC, January 2011). A combination of some of the following acceptable techniques, in order of implementation priority, should be implemented when developing detailed subdivision designs:

- LID infiltration BMPs;
- Deep pool and bottom-draw from SWM facility;
- Urban terrestrial canopy;
- Facility shading, orientation and length to width ratio; and,
- Concrete sewer system.

The CVC Thermal Impacts report identified five zones where thermal mitigation measures can be implemented. These include:

- Zone 1 - Pond catchment area
- Zone 2 - Stormwater Management Facility Inlet
- Zone 3 - Stormwater Management Facility
- Zone 4 - Stormwater Management Facility Outlet
- Zone 5 - Riparian Corridor

The typical outlet structure for all SWM facilities will consist of a deep outlet pool, reverse-slope extended detention pipe, and a sub-surface outlet pipe. The thermal mitigation strategy including planting/landscaping details will be further refined during the detailed SWM facility design stage.

The potential thermal mitigation measures that were considered for each facility is summarized in ***Table 5-7***. The recommendations below include subsurface storm sewers, LID measures, downspout disconnection, buried inlet pipes, reversed slope submerged pond outlets and extra permanent pool depth that should be implemented at detailed design.

Table 5-7: Potential Thermal Mitigation Measures

Thermal Mitigation Techniques per Development "Zone"				
Thermal Mitigation Techniques	**Zone**	**Pond**		
		7	**9**	**Notes**
Energy transfer between warm storm runoff and cool sub-surface storm sewers	1	Yes	Yes	
LID measures	1	Yes	Yes	See ***Section 6.1*** of this report
Roof Colour	1	No	No	Development characteristics not yet known

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Thermal Mitigation Techniques per Development "Zone"				
Thermal Mitigation Techniques	**Zone**	**Pond**		
		7	**9**	**Notes**
Downspout Disconnection	1	Yes	Yes	
Up-Gradient Plantings	1	TBD	TBD	
Buried Inlet Pipe	2	Yes	Yes	
Inlet Cooling Trench	2	No	No	Not recommended due to additional infrastructure/ maintenance, grading impacts
Inlet Plantings	2	TBD	TBD	
Shading of open water areas by maximizing canopy	3	Yes	Yes	
Artificial shade system	3	No	No	Not recommended due to additional infrastructure/ maintenance
Floating island	3	No	No	Not recommended due to additional infrastructure/ maintenance
Reduce Open water area	3	Yes	Yes	
Increased L:W Ratio	3	No	No	Not recommended due to location constraints
Pond orientation to increase exposure to prevailing wind	3	No	No	The orientation of the ponds are based on topographic and boundary constraints and to meet other development objectives.
Landscaped jetties for shading	3	TBD	TBD	
Sub-surface SWM Facility	3	No	No	Not practical for large drainage areas
Outlet sub-surface cooling trench and shading	4	No	No	Not recommended – historically MECP has indicated cooling trenches have marginal effect on effluent water temperatures
Concrete outlet pipe	4	Yes	Yes	
Introduce cool water at SWM Pond outlets such as foundation drain collectors, where feasible	4	No	No	Not recommended due to additional infrastructure/ maintenance
Reversed slope submerged pond outlet and extra permanent pool depth at outlet	4	Yes	Yes	
Distributed outlets along the NHS/Greenbelt to take advantage of the NHS/Greenbelt shading	4	Yes	Yes	
Night time release of discharge	4	No	No	Not recommended due to additional infrastructure/ maintenance of resulting complex system
Watercourse shading	5	Yes	Yes	

5.7 Pond Outfalls

The SWM ponds will discharge to tributaries of the Humber River Sub-watershed. All outfalls will be designed to generally meet the criteria indicated in the ***TRCA SWM Guidelines***, where applicable. Stormwater outfalls are proposed to discharge to the valley floor, given the bank steepness; generally located as close to the toe of valley slope as possible. Exact outfall locations will be refined through draft plan approval process.

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Efforts will be made to limit disturbance to the wetland resulting from the outfall and plunge pool installations. Disturbance to the wetland will be restored following installation to the extent feasible and/or compensated for, as required, through detailed design. The outfalls will provide energy dissipation at the headwall to mitigate localized erosive forces in the floodplain and valley floor. Construction of the outfalls should minimize disturbance to the Natural Heritage System.

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6.0 LOW IMPACT DEVELOPMENT STRATEGIES

6.1 LID Measures and Ownership

Low Impact Development Techniques (LIDs) can help mitigate the effects of development on the water budget, promote at-source retention, and maintain groundwater infiltration volume in the post-development condition. The use of LIDs forms part of a “treatment-train” approach to achieve overall stormwater management benefits, such as reducing thermal impacts, achieving water balance targets, promoting retention on the up-gradient pond catchment to minimize runoff and more closely match pre-development watershed characteristics along with other benefits.

A preliminary LID design has been completed for Subject Property. The location of the proposed infiltration measures are provided on ***Figure 6***. The location and size of the LIDs have had consideration for private vs public property, existing soil conditions, and depth to groundwater. The following LID options were considered as part of our analysis:

- Increased topsoil depths (300 mm) on all conventional detached product and conventional townhouse products;
- Infiltration galleries in park blocks
- Rear yard infiltration trenches
- Sub-Surface Infiltration LIDs
    - Infiltration trenches in environmental buffer areas, subject to agency approval.

The list above is based on a combination of LIDs that are applicable in public property. It should be noted that the MECP and Town of Caledon have entered into a Consolidated Linear Infrastructure Environmental Compliance Approval (CLI-ECA) agreement, that allows for expansion of the storm sewer network subject to meeting criteria outlined by the province. Information on the CLI-ECA requirements is further described under ***Section 6.3***.

6.2 Site Wide Water Balance

A Site Wide Water Balance Study will be completed to evaluate the changes in annual infiltration volumes for the Subject Property. The water balance completion should satisfy the new CLI-ECA requirements related to retention of runoff between the pre- to post-development infiltration for the Subject Property, done to the pre-development rainfall rate or 90th Percentile Storm event on the property (25 mm), as well as meeting the TRCA requirement of the first 5 mm rainfall event to be retained on site. ***Table 6-1*** outlines the target annual infiltration volume.

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Table 6-1: Target for LID Annual Infiltration Volumes

Pre-Development Annual Infiltration Volume [1] (m^3/year)	Post-Development Annual Infiltration Volume Without Mitigation[2] (m^3/year)	Target Annual Infiltration Volume (m^3/year)
13,416	16,397	2,981

1 Refer to ***Appendix C*** of the Hydrogeological Investigation, Patterson Group, March 2026 for further details
2 Mitigation measures is without disconnected roof leaders and increased topsoil depth

6.3 CLI-ECA Requirements

The Town of Caledon has entered into an agreement with the Ministry of the Environment, Conservation and Parks (MECP) to participate in the CLI-ECA program for stormwater infrastructure. The CLI-ECA program requires that the proposed design consider the Water Balance and Water Quality for development scenarios. A pre- and post-development water balance has been completed to determine the post-development infiltration target for LIDs within the subject lands. Please refer to ***Appendix C*** for the hydrogeological assessment and water balance assessment prepared by Patterson Group. A summary of the post-development infiltration target is provided in ***Table 6-1***.

As shown in the table above, without LID measures in place, post-development conditions will provide less infiltration than pre-development conditions. Where feasible, LID measures for SWM will be incorporated into the development design to provide additional water balance and control runoff measures. As outlined in the SWMP Design Manual (2003), Low Impact Development Stormwater Management Planning and Design Guide published by the CVC and TRCA (2010), and Sustainable Technologies Evaluation Program LID Stormwater Management webpage, there are a suite of LID techniques that can be considered to match pre-development infiltration volumes.

There are number of techniques that maximize the water availability in pervious areas that are not credited through the CLI-ECA program. This includes measures located on public property, environmental buffer areas, right-of-ways and increased topsoil depth within private lots. These types of surface LID techniques promote natural infiltration simply by providing additional water volumes in the pervious areas (i.e., these areas would receive precipitation as well as extra water from roof runoff). While these techniques are not credited through the CLI-ECA program, they can provide a water quantity benefit.

In addition to the techniques mentioned above, there are a number of LID measures that can be implemented as part of the CLI-ECA program. These include measures within areas owned or maintained by the municipality. To determine the optimal LID strategy, the proposed design must first consider site constraints. As part of the CLI-ECA program, the MECP provides a checklist of stormwater management site constraints that should be considered when evaluating if LID measures are suitable to meet the water balance and/ or quality control targets. Table A2 from the Environmental Compliance Approvals for a Stormwater Management System (MECP, 2022) can be used for reference of site constraints.

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A 1.0 m clearance should be provided between the high groundwater elevation and the bottom of the proposed LID to ensure that the LID is able to function properly. LIDs have therefore been proposed in areas where 1.0 m separation can be provided between groundwater and the bottom of the LID. Groundwater elevations should be confirmed through additional testing at detailed design.

Further details on LID evaluations and compliance with CLI-ECA requirements shall be provided at detailed design stage.

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7.0 WATER SUPPLY SEVICING

7.1 Water Supply Servicing Design Criteria

The water supply servicing the Subject Property will be designed according to the ***Town Standards***, **Region Guidelines**, and MECP Design Guidelines for Drinking Water Systems (2008), by taking into consideration watermain sizing, depth, crossings, valves, hydrants, and service connections such that adequate pressures and fire flows can be achieved. Water design flows will be designed with the following criteria shown in ***Table 7-1*** and ***Table 7-2*** below.

Table 7-1: Water Design Criteria

DEMAND TYPE	CRITERIA
Average Daily Demand - Residential (L/c/d)	280
Maximum Daily Demand - Residential (L/c/d)	2.0 x avg. day
Peak Hour Demand - Residential (L/c/d)	3.0 x avg. day

Table 7-2: Town of Caledon Population Densities

DEVELOPMENT TYPE	EQUIVALENT DESIGN POPULATION
Single Family	4.2 persons/unit
Townhouse / Row Dwellings	3.4 persons/unit

7.2 Existing Water Services

There are no existing water services within the vicinity of the Subject Property. Pressure Zone 6 is located south of Columbia Way, however the Subject Property is located outside of the range of elevations associated with the water distribution infrastructure for Pressure Zone 6, as noted in the Water and Wastewater Master Plan (Region of Peel, 2020). Per the ***CFCA*** **FSSWMR**, the proposed development will need to be serviced by a new Pressure Zone, Pressure Zone 7.

7.3 External Water Supply Requirements

The Subject Property will be connected to a proposed external 400 mm watermain on Highway 50 via one connection. Per the ***CFCA*** **FSSWMR**, the existing Macville Booster Pump Station (BPS) can service an interim population of 2,960 persons within the Option 1 lands (Subject Property). Beyond this threshold, additional water storage is required to satisfy and service the remaining North Bolton Hill Secondary plan Area.

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A full build-out watermain modelling of the whole North Bolton Hill Secondary Plan Area was completed by HydraTek and is included in the ***CFCA FSSWMR***. The proposed external infrastructure required to service the Subject Property includes:

- A 600 mm diameter feedermain along Emil Kolb Parkway to Regional Road 50
- A 400 mm diameter watermain along Regional Road 50 and Columbia Way
- Two Elevated Tanks
    - One new elevated tank is proposed outside of the Subject Property, within the Option 1 lands.

Per the ***CFCA FSSWMR,*** the above noted infrastructure is sufficient to service the full Pressure Zone 7 build-out.

7.4 Proposed Water Supply

The site will be serviced by one connection on Street '2' to the proposed external 400 mm watermain within Highway 50. The ***CFCA* FSSWMR** confirms the Subject Property lands can be adequately serviced by the proposed external watermains noted above in ***Section 7.3***, refer to details in ***Appendix D***.

No watermain crossings of the natural heritage system within the subject lands are required. The water distribution system within the development will be sized to meet the pressures and flows in accordance with the municipal requirements. The system will be looped internally in order to provide system security.

Sizing of the internal watermains was not provided in the ***CFCA FSSWMR***. The internal infrastructure will be further evaluated through detailed design applications using the current water distribution model and associated water demand / fire flow criteria. A layout of the proposed watermain network is illustrated in ***Figure 7.***

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8.0 SANITARY SERVICING

8.1 Sanitary Design Criteria

The sanitary mains will be designed with the following Region of Peel Engineering Standards:

Table 8-1: Sanitary Design Criteria

DEMAND TYPE	CRITERIA
Average Dry Weather Flow (residential)	290 L/cap/day
Infiltration	0.26 L/s/ha
Peaking Factor	Peak Flow Factor – Harmon Formula

Table 8-2: Town of Caledon Population Densities

DEVELOPMENT TYPE	EQUIVALENT DEISGN POPULATION
Single Family	4.2 persons/unit
Townhouse / Row Dwellings	3.4 persons/unit

8.2 Existing Sanitary Services

There are no existing wastewater services within the vicinity of the Subject Property.

8.3 External Sanitary Services

Based on the **W&WWFS**, the Subject property is to be conveyed to the McVean Sewage Pumping station and ultimately to the G.E. Booth Wastewater treatment plant. The Subject Property will be connected to a proposed external 1200 mm diameter sanitary trunk on Highway 50 via one connection off Street 2.

The proposed sanitary trunk on Highway 50 is to discharge into a proposed sanitary system on Emil Kolb Parkway and ultimately to the proposed trunk sewer on Humber Station Road. Flows are then pumped via twin 450 mm diameter forcemains to a 525 mm diameter trunk sewer flowing west via gravity on King Street, and ultimately to a 1200 mm diameter trunk sewer flowing south on Humber Station Road. Flows will then proceed to the McVean Sewage Pumping Station and ultimately to the G.E. Booth Wastewater treatment plant.

8.4 Proposed Sanitary Servicing

The Subject Property will be serviced by a network of local gravity sewers designed in accordance with Town of Caledon design standards. The sanitary sewers will connect into a local connection stub provided to the proposed 1200 mm diameter sanitary trunk on Highway 50.

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Please refer to ***Figure 8*** for the sanitary drainage figure and ***Appendix F*** for the Sanitary Design sheets.

Sanitary calculations were completed in the ***CFCA*** **FSSWMR,** confirms the Subject Property lands can be adequately serviced by the proposed sanitary sewers.

8.5 Servicing Blocks & Easements

There are no servicing blocks or easements proposed for the sanitary distribution system.

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9.0 ROADS

Access to the subject property is available from the boundary roads located on Street 2.

The community will include a series of 18 m, 20 m and 22 m wide municipal roads in accordance with ***Town Standards***. Sidewalks will be constructed in accordance with the standard municipal cross sections. The 22 m wide municipal road has been modified from the Town Standards, and is included in ***Appendix D***.

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10.0 GRADING

A preliminary grading plan has been prepared for the study area based on the environmental and engineering constraints identified. The proposed road grades range from 0.5% to 3% in accordance with the ***Town Standards***. Overland flow routes will be contained within the internal rights-of-way and directed towards the SWM ponds.

Grading in the NHS/Greenbelt area and the associated buffers is minimized but may be required at the following locations:

- Pond outfalls
- 3:1 grade transition behind rear lots within the buffer
- Grading may be required to provide emergency spillways for the ponds adjacent to the NHS/Greenbelt area.

The anticipated grading within the NHS/Greenbelt buffers is illustrated on ***Drawing 1***.

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11.0 POND MAINTENANCE

The following section provides general guidance on pond maintenance for the Subject Property community. Further refinement and details can be provided at functional and/or detailed design of the individual draft plans

11.1 Inspections

As recommended in the ***SWMP Manual***, inspections should be made after every significant storm (e.g., >10 mm) during the first two years of operation to ensure that the facility is functioning properly. It is anticipated that four inspections will be required per year. After the initial period, and proper operation has been confirmed, an inspection schedule can be established based on the observed operation of the ponds. Although four inspections per year are recommended, the ponds should be inspected annually as a minimum requirement.

11.2 Regular Operation and Maintenance Activities

Grass Cutting

Grass cutting is not recommended for the ponds. Allowing grass to grow enhances the water quality and provides other benefits. It is understood though, that grass cutting enhances the aesthetics of the facility for nearby residents and therefore, should be done as infrequently as possible.

Grass should not be cut to the edge of the permanent pool and should be done parallel to the shoreline. Grass clippings should be ejected away from the pond.

Weed Control

If weed control is required to remove a specific species, the weeds should be removed by hand.

Plantings

Different parts of the SWM ponds will require different vegetative treatments for upland, flood, shoreline and aquatic conditions. Planting methods and any replanting should be carried out in accordance with an approved Landscape Design and the recommendations of the ***SWMP Manual***, or as modified by the operating authority.

Trash Removal

Trash and debris should be removed by hand, and performed as required based on inspections.

Sediment Removal

In accordance with the ***SWMP Manual,*** it is recommended that the frequency of sediment removal be determined based on a 5% reduction in the total suspended solids (TSS) removal efficiency. The frequency of pond maintenance will be determined at the detailed design stage. It should be noted that routine cleaning of the sediment forebay should allow for less frequent cleaning of the

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main cell than indicated in the ***SWMP Manual.*** However, the extension of service life prior to cleaning cannot be quantified.

Safety

The ponds should be provided with appropriate signage which warns the public of the presence of deep water and slopes. Landscape drawings are to be prepared with strategic plantings around the perimeter of the pond to discourage direct access to the facility. All inlets, outlets, structures, and headwalls will be provided with the appropriate grates, covers, and safety features to prevent public entry or tampering.

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12.0 EROSION AND SEDIMENT CONTROL

An erosion and sediment control strategy should be implemented prior to construction of site services. The following measures are recommended:

Erosion

- regular inspection and monitoring of the erosion and sediment control devices.
- ensure stabilized soil and slope stability to prevent runoff and stability hazards.
- cover non-active work faces of stockpiles, excavation areas, all other exposed surfaces of concern with tarps or other appropriate materials as practicable.
- Monitoring the weather forecast daily and plan/schedule work activities with specific consideration of impending weather conditions to prevent events leading towards further erosion.
- Minimize to the extent possible staging and temporary stockpile of soil and other materials.

Sediment

- silt control fencing and double row silt control fencing with straw bales where required.
- stone mud mat at the construction entrance.
- interceptor swale with rock check dam where appropriate.
- use of the permanent ponds as a temporary silt basin during site construction activities.
- removal and disposal of the erosion and sediment control devices after the site has been stabilized.

Details will be provided at the detailed design stage.

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13.0 UTILITIES

Utilities will follow the proposed right-of-way sections as shown in ***Appendix D***. Utility locations and crossing locations will be coordinated with local providers and refined under detailed design.

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14.0 CONCLUSIONS

The Functional Servicing, Stormwater Management and Preliminary Design Report provides an overview of the servicing and stormwater management plan for the Tiffany Rox North Hill Ltd. lands. This report demonstrates the availability of water, wastewater, and storm services for the proposed subdivision in general conformance with the Bolton North Hill Functional Servicing and Stormwater Management report and in accordance with ***Town*** and ***Region Standards***, the requirements of Toronto and Region Conservation Authority, and general industry practice.

We trust you will find the contents of this report satisfactory.

Prepared by,
David Schaeffer Engineering Ltd.

Christine McCarthy, P.Eng.

Reviewed by,
David Schaeffer Engineering Ltd.



Tim Edmondson, P.Eng.

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1417_North Bolton Hill FSR.docx



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LEGEND

- BOLTON NORTH HILL SECONDARY PLAN BOUNDARY
- SUBJECT LANDS

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TOWN OF CALEDON

SITE LOCATION PLAN

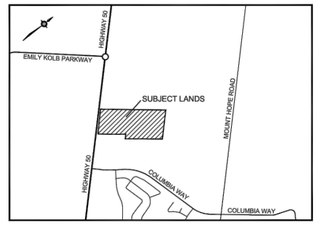
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DATE:	MARCH 2026	FIGURE:	1

DRAFT PLAN OF SUBDIVISION
PART OF LOT 12, CONCESSION 7
(GEOGRAPHIC TOWNSHIP OF ALBION)
TOWN OF CALEDON
REGIONAL MUNICIPALITY OF PEEL

SCALE: 1:1250



DRAFT PLAN T-



KEY PLAN NTS

SECTION 51(17), PLANNING ACT, ADDITIONAL INFORMATION

A. AS SHOWN ON DRAFT PLAN
B. AS SHOWN ON DRAFT PLAN
C. AS SHOWN ON DRAFT PLAN
D. SEE SCHEDULE OF LAND USE
E. AS SHOWN ON DRAFT PLAN
F. AS SHOWN ON DRAFT PLAN
G. AS SHOWN ON DRAFT PLAN
H. MUNICIPAL PIPED WATER AVAILABLE AT TIME OF DEVELOPMENT
I. CLAY-LOAM
J. AS SHOWN ON DRAFT PLAN
K. SANITARY AND STORM SEWERS, GARBAGE COLLECTION, FIRE PROTECTION
L. AS SHOWN ON DRAFT PLAN

SURVEYOR'S CERTIFICATE

I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LAND TO BE SUBDIVIDED AS SHOWN ON THIS PLAN, AND THEIR RELATIONSHIP TO THE ADJACENT LAND ARE ACCURATELY AND CORRECTLY SHOWN.

DATE _____, 2025

_____
Ophir N. Dzaldov B.Sc, O.L.S, O.L.I.P.
SCHAEFFER DZALDOV PURCELL LTD.

OWNER'S CERTIFICATE

I AUTHORIZE KLM PLANNING PARTNERS INC. TO PREPARE AND SUBMIT THIS DRAFT PLAN OF SUBDIVISION TO THE CITY OF TOWN OF CALEDON FOR APPROVAL.

OWNER

TIFFANY ROX
C/O BRETT SAITOWITZ
601 MAGNETIC DRIVE
NORTH YORK, ON
M3J3J2

_____
Brett Saitowitz A.S.O

SCHEDULE OF LAND USE

TOTAL AREA OF LAND OWNED BY APPLICANT = 19.047 ± Ha. (47.066 ± Acs.)

	BLOCKS	LOTS	UNITS	± Ha.	± Acs.
SINGLE DETACHED DWELLING					
♦ LOTS 39-54, 64-72, 75-90, 93-106, 108-143, 148-158, 168-195, AND 205-213 MIN. LOT FRONTAGE - 11.0m MIN. AREA - 298.6m²		139	139	4.581	11.320
◊ PLUS BLOCKS 144-147	4		3 *	0.096	0.237
● LOTS 33-35, 55-63, 73, 74, 91, 92, 107, 159-167, AND 196-204 MIN. LOT FRONTAGE - 9.15m MIN. AREA - 247.0m²		35	35	0.967	2.390
○ PLUS BLOCKS 29-32, AND 36-38	7		3 *	0.080	0.198
STANDARD TOWNHOUSE DWELLING					
BLOCK 1-5, 8, 9, 11, 12, 15-25 AND 28 MIN. UNIT FRONTAGE - 7.0m	21		125	2.853	7.050
PLUS BLOCKS 10, 13, 14, 26 AND 27	5		4.5 *	0.134	0.331
DUAL FRONTAGE TOWNHOUSE DWELLING					
BLOCK 6 AND 7 MIN. UNIT FRONTAGE - 7.0m	2		6	0.269	0.665
SUBTOTAL	39	174	315.5	8.980	22.190
BLOCK 214 - PARK	1			0.528	1.305
BLOCK 215 - NATURAL HERITAGE SYSTEM	1			4.937	12.200
BLOCK 216 - STORMWATER MANAGEMENT	1			0.175	0.432
BLOCK 217-222 - 0.3m RESERVE	6			0.015	0.037
BLOCK 223 & 224 - FUTURE DEVELOPMENT	2			0.022	0.054
STREETS				4.390	10.848
22.0m WIDE TOTAL LENGTH = 258m. AREA = 0.569±Ha.					
20.0m WIDE TOTAL LENGTH = 221m. AREA = 0.443±Ha.					
18.0m WIDE TOTAL LENGTH = 1834m. AREA = 3.307±Ha.					
16.0m WIDE TOTAL LENGTH = 43m. AREA = 0.071±Ha.					
TOTAL LENGTH = 2356m. AREA = 4.390±Ha.					
TOTAL	**50**	**174**	**315.5**	**19.047**	**47.066**

NOTES:
- ELEVATIONS RELATED TO CANADIAN GEODETIC DATUM
- 0.3M RESERVES HAVE BEEN EXAGGERATED TO 1.5M FOR VISUAL PURPOSES
- (*) UNITS SUBJECT TO FINAL CALCULATION

KLM PLANNING

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SCALE 1:1250 5 March 2026
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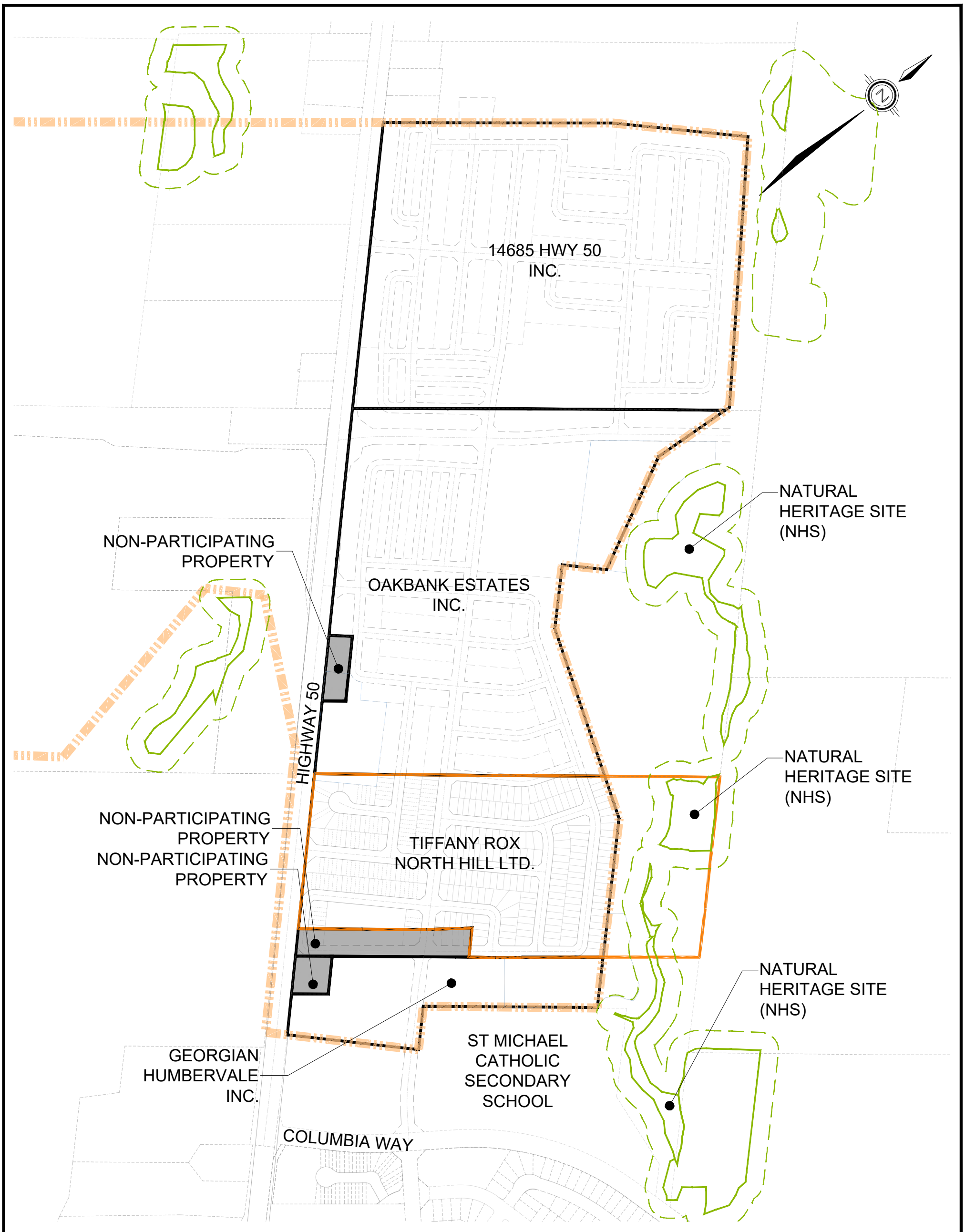
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DRAFT PLAN

SCALE:	NTS	PROJECT No.:	25-1433
DATE:	MARCH 2026	FIGURE:	2



LEGEND

- BOLTON NORTH HILL SECONDARY PLAN BOUNDARY
- SUBJECT LANDS
- OWNERSHIP BOUNDARY (PARTICIPATING PROPERTY)
- OWNERSHIP BOUNDARY (NON-PARTICIPATING PROPERTY)
- NATURAL HERITAGE SITE (NHS)
- NHS 30m SETBACK

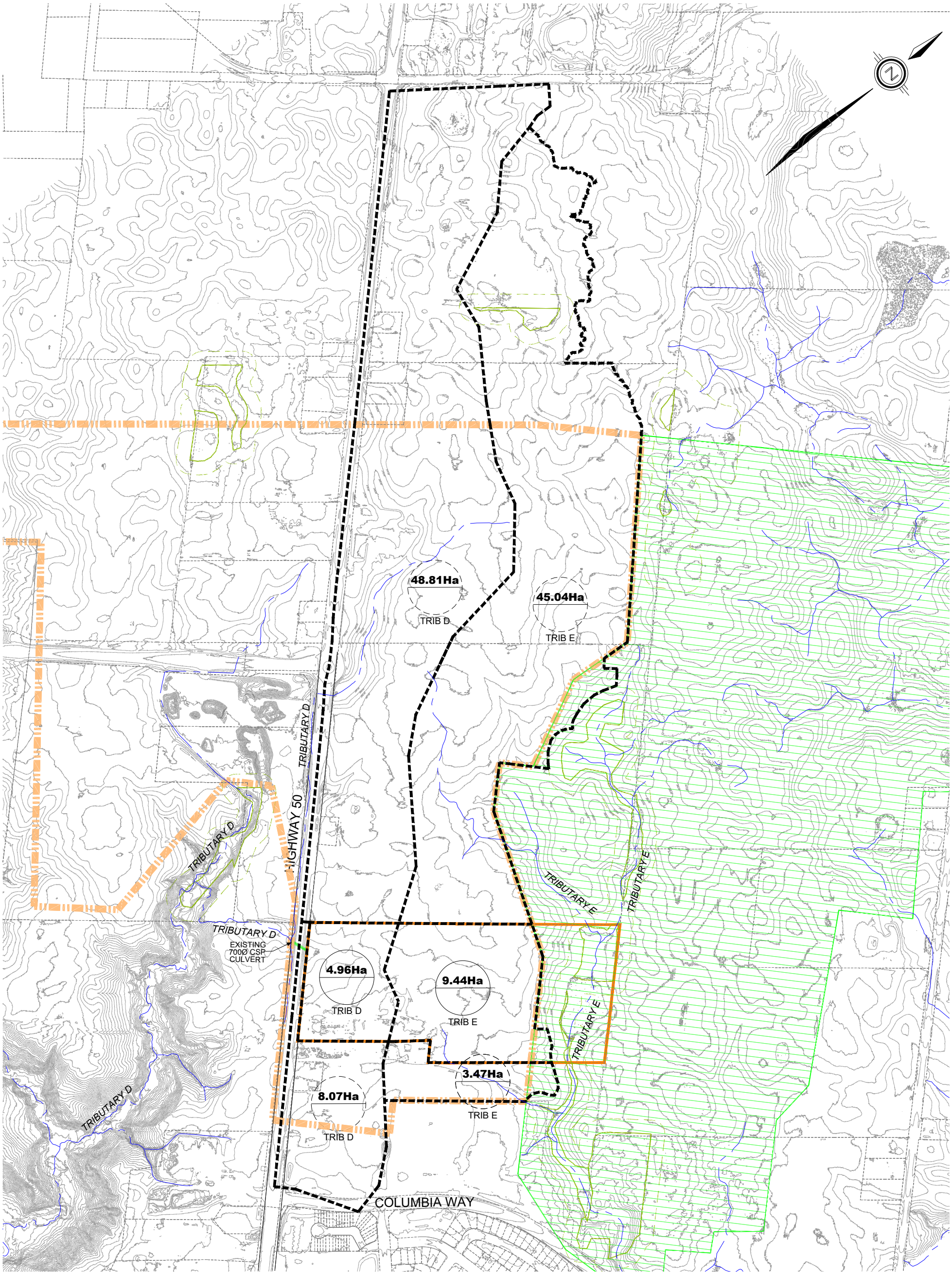
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TOWN OF CALEDON

LANDOWNERS MAP

SCALE:	1:6000	PROJECT No.:	25-1433
DATE:	MARCH 2026	FIGURE:	3



LEGEND

- SUBJECT LANDS
- PRE-DEVELOPMENT DRAINAGE BOUNDARY
- BOLTON NORTH HILL SECONDARY PLAN BOUNDARY
- GREENBELT LANDS
- NATURAL HERITAGE SITE (NHS)
- NHS 30m SETBACK
- WATERCOURSE
- 22.0Ha — INTERNAL DRAINAGE AREA
- 0.9Ha — EXTERNAL DRAINAGE AREA

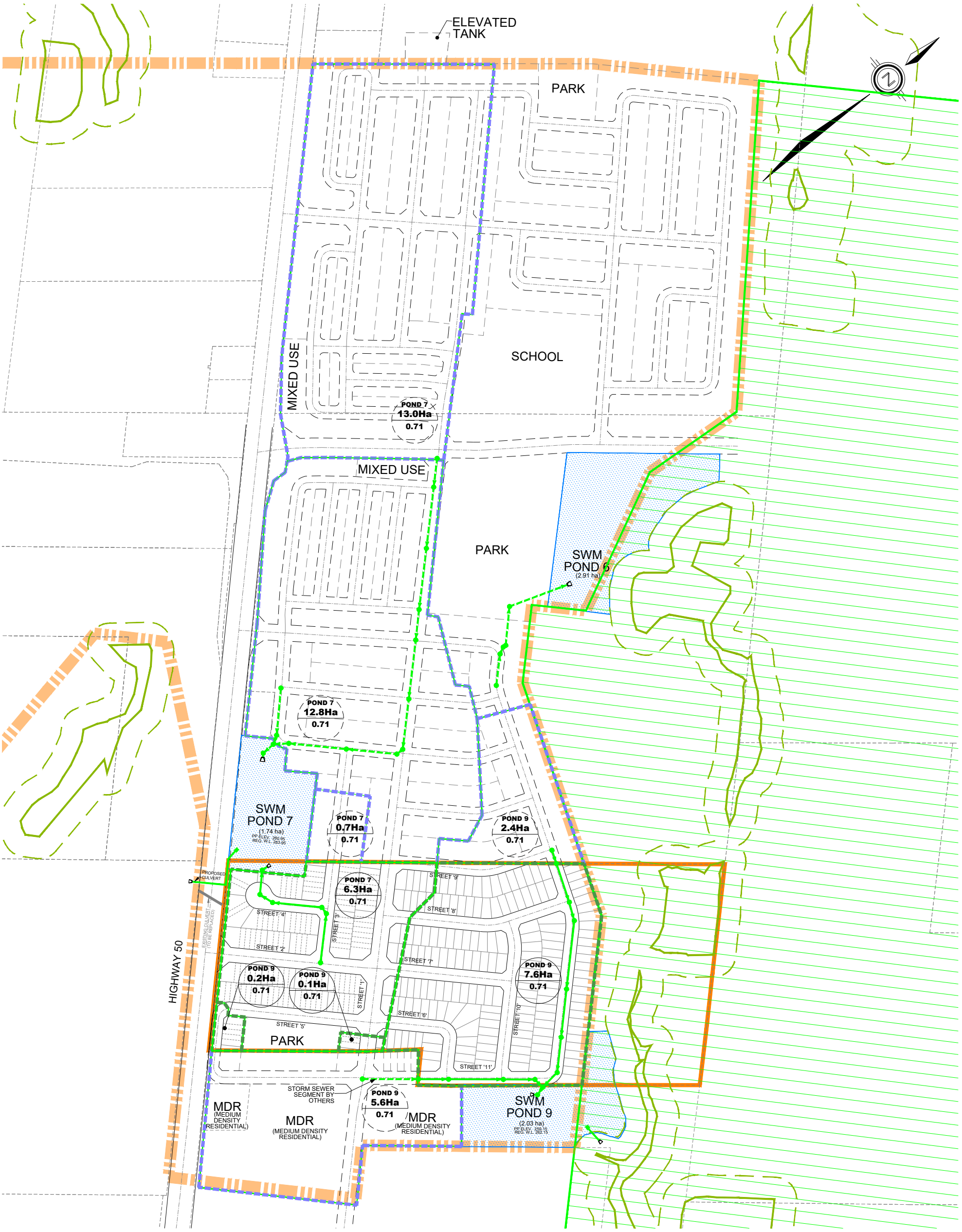
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TOWN OF CALEDON

EXISTING STORM DRAINAGE PLAN

SCALE:	1:8000	PROJECT No.:	25-1433
DATE:	MARCH 2026	FIGURE:	4



LEGEND

- SUBJECT LANDS
- INTERNAL POND DRAINAGE AREA
- EXTERNAL POND DRAINAGE AREA
- PROPOSED STORM SEWER
- PROPOSED STORM SEWER BY OTHERS
- STORM MANHOLE
- GREENBELT LANDS
- NATURAL HERITAGE SITE (NHS)
- NHS 30m SETBACK
- SWM POND BLOCK
- BOLTON NORTH HILL SECONDARY PLAN BOUNDARY
- POND 7 / 0.2 / 0.5 — DRAINAGE DESTINATION, TOTAL EXTERNAL AREA, RUN-OFF COEFFICIENT
- POND 7 / 0.5 / 0.75 — DRAINAGE DESTINATION, TOTAL INTERNAL AREA, RUN-OFF COEFFICIENT

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TOWN OF CALEDON

PROPOSED STORM DRAINAGE PLAN

SCALE:	1:5000	PROJECT No.:	25-1433
DATE:	MARCH 2026	FIGURE:	5

PARK

SWM POND 6

SWM POND 7

STREET '1'

STREET '2'

STREET '3'

STREET '4'

STREET '5'

PARK

STREET '6'

STREET '7'

STREET '8'

STREET '9'

STREET '10'

STREET '11'

HIGHWAY 50

MDR

MDR

MDR

SWM POND 9

LEGEND

SUBJECT LANDS

POTENTIAL RLCB INFILTRATION TRENCH

POTENTIAL RLCB INFILTRATION TRENCH WITHIN NHS BUFFER

POTENTIAL INFILTRATION GALLERY IN PARK BLOCK

NATURAL HERITAGE SITE (NHS)

NHS 30m SETBACK

ESTIMATED GROUND WATER DEPTH FROM PROPOSED GROUND

Depth (m)

0.00~1.00:

1.00~2.00:

2.00~3.00:

3.00~4.00:

>4.00

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TOWN OF CALEDON

POTENTIAL LID OPPORTUNITIES

SCALE:	1:3000	PROJECT No.:	25-1433
DATE:	MARCH 2026	FIGURE:	6

PARK

SWM
POND 6

SWM
POND 7

STREET '9'

STREET '8'

STREET '4'

STREET '3'

STREET '2'

STREET '7'

HIGHWAY 50

STREET '1'

STREET '6'

STREET '10'

STREET '5'

PARK

STREET '11'

MDR

MDR

MDR

SWM
POND 9

LEGEND

- SUBJECT LANDS
- PROPOSED 400Ø EXTERNAL WATERMAIN (B.O.)
- PROPOSED SUB-TRUNK WATERMAIN
- PROPOSED LOCAL WATERMAIN
- BOLTON NORTH HILL SECONDARY PLAN BOUNDARY
- PROPOSED SUB-TRUNK WATERMAIN EXTERNAL
- PROPOSED LOCAL WATERMAIN EXTERNAL

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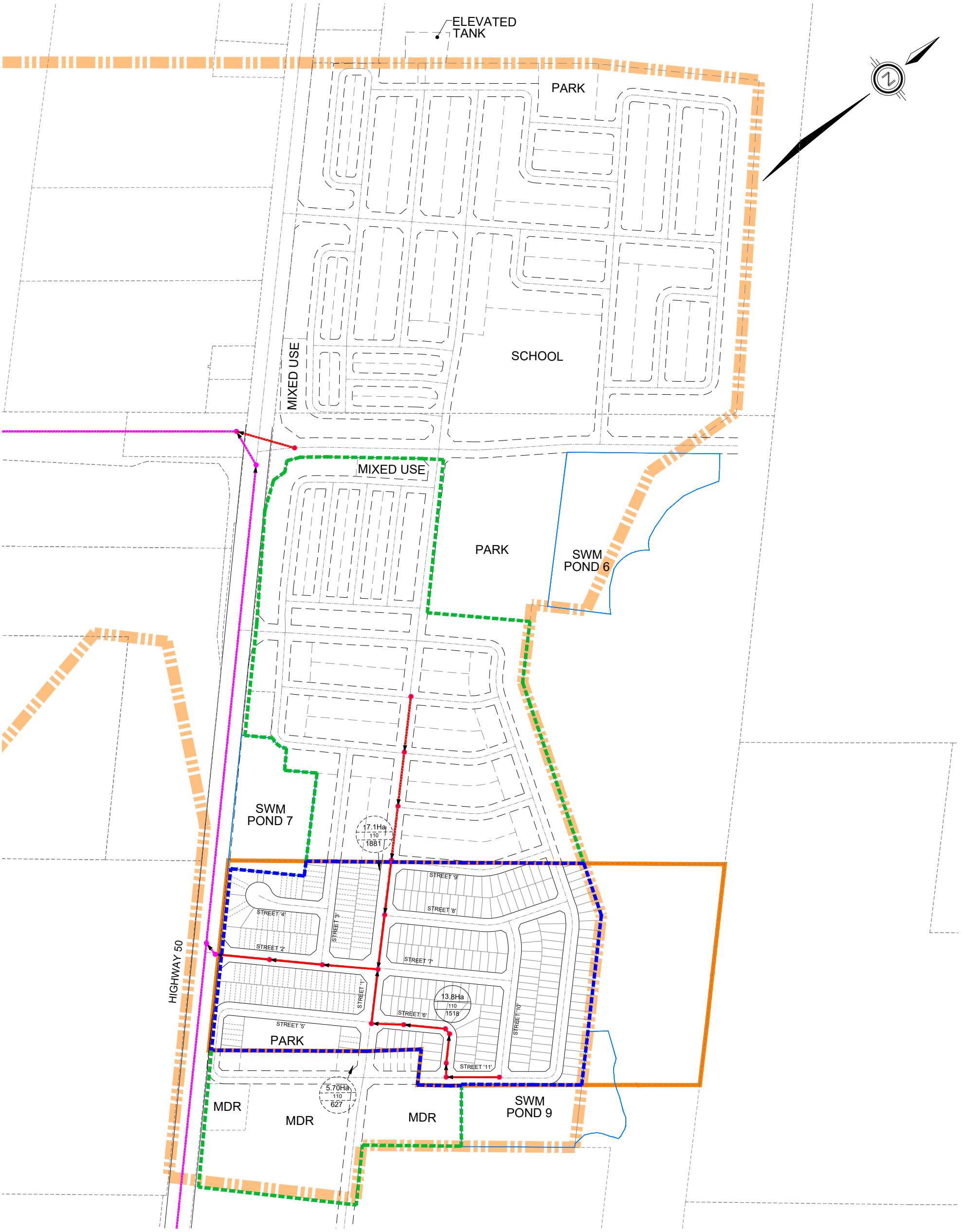
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TOWN OF CALEDON

PRELIMINARY WATERMAIN LAYOUT

SCALE:	1:3000	PROJECT No.:	25-1433
DATE:	MARCH 2026	FIGURE:	7



LEGEND

- SUBJECT LANDS
- INTERNAL DRAINAGE AREA
- EXTERNAL DRAINAGE AREA
- PROPOSED SANITARY SEWER
- PROPOSED EXTERNAL SANITARY SEWER
- PROPOSED SANITARY MAINTENANCE HOLE
- PROPOSED SANITARY MAINTENANCE HOLE BY OTHERS
- PROPOSED SANITARY TRUNK SEWER BY OTHERS
- BOLTON NORTH HILL SECONDARY PLAN BOUNDARY
- POND 7 / 0.2 / 0.5 — DRAINAGE DESTINATION, TOTAL EXTERNAL AREA, RUN-OFF COEFFICIENT
- POND 7 / 0.5 / 0.75 — DRAINAGE DESTINATION, TOTAL INTERNAL AREA, RUN-OFF COEFFICIENT

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TOWN OF CALEDON

PRELIMINARY SANITARY LAYOUT

SCALE:	1:5000	PROJECT No.:	25-1433
DATE:	MARCH 2026	FIGURE:	8

PARK

SWM POND 6

SWM POND 7

PP ELEV. 260.95
REG. W.L. 263.95

PARK

MDR

MDR

MDR

SWM POND 9

PP ELEV. 259.15
REG. W.L. 262.15

MATCH EXISTING

LEGEND

- SUBJECT LANDS
- 187.60 — PROPOSED CENTERLINE ELEVATION
- [187.60] — PROPOSED CENTERLINE ELEVATION BY OTHERS
- 193.50 / 193.50 — PROPOSED ELEVATION / EXISTING ELEVATION
- 244.50 — EXISTING CONTOUR ELEVATION
- STORM OVERLAND FLOW ARROW
- EMERGENCY OVERLAND FLOW ARROW
- 265.55 — EXISTING ROAD ELEVATION
- —0.63m→ CL DELTA - SPLIT DRAINER LOTS (0-0.80m)
- —1.16m→ CL DELTA - LOOK OUT LOTS (0.80-2.0m)
- GREENBELT LANDS
- NATURAL HERITAGE SITE (NHS)
- NHS 30m SETBACK

DISTANCE FROM CENTRELINE
ELEVATION TO EXISTING GROUND

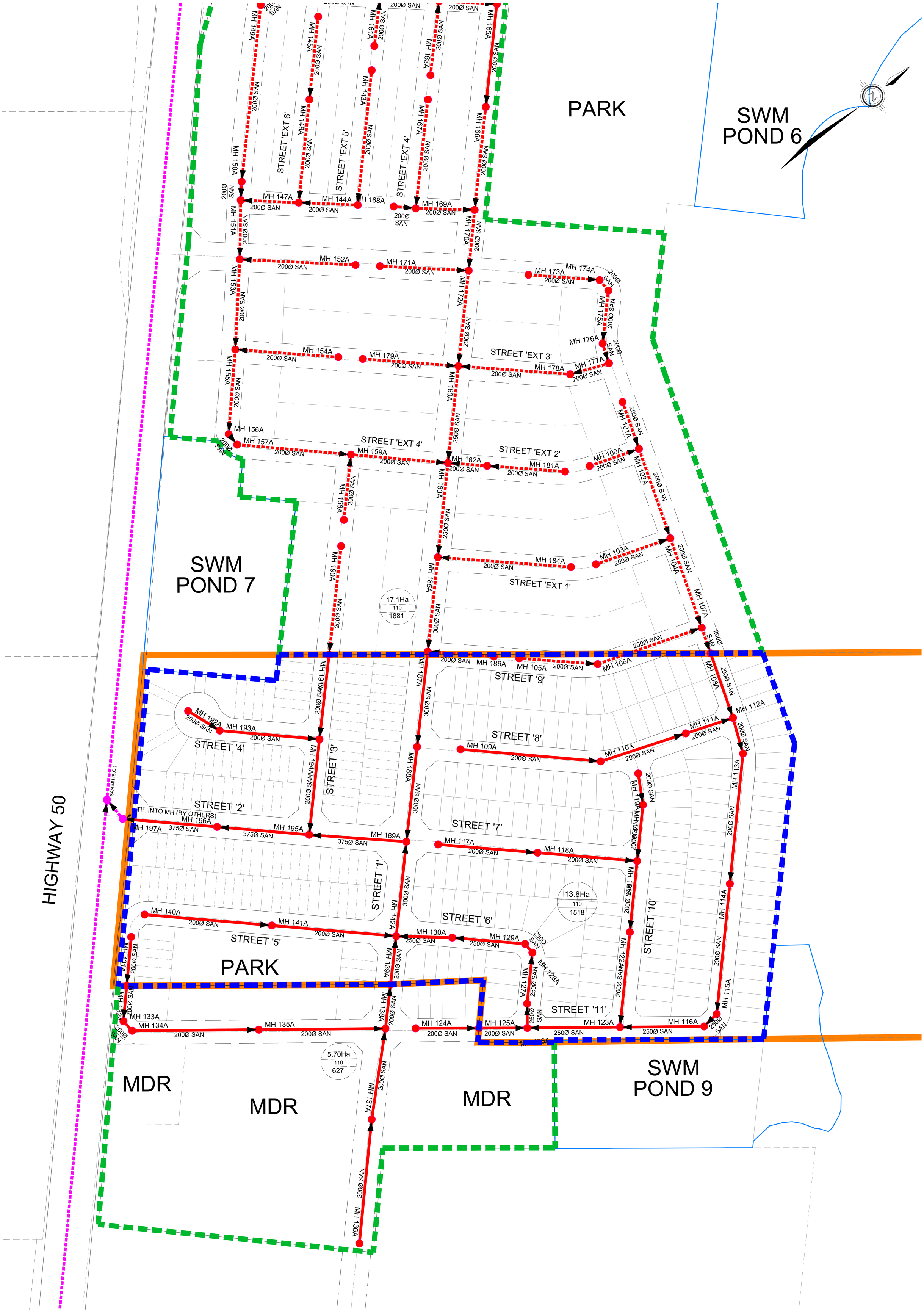
PROPOSED GRADE LOWERING BELOW EXISTING	PROPOSED GRADE RAISE ABOVE EXISTING
0 - 0.5	0 - 0.5
0.5-1.0	0.5-1.0
1.0-1.5	1.0-1.5
> 1.5	> 1.5

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Markham, Ontario, L3R 0E7
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GRADING PLAN

SCALE:	1:1250	PROJECT No.:	25-1433
DATE:	MARCH 2026	DRAWING:	1



LEGEND

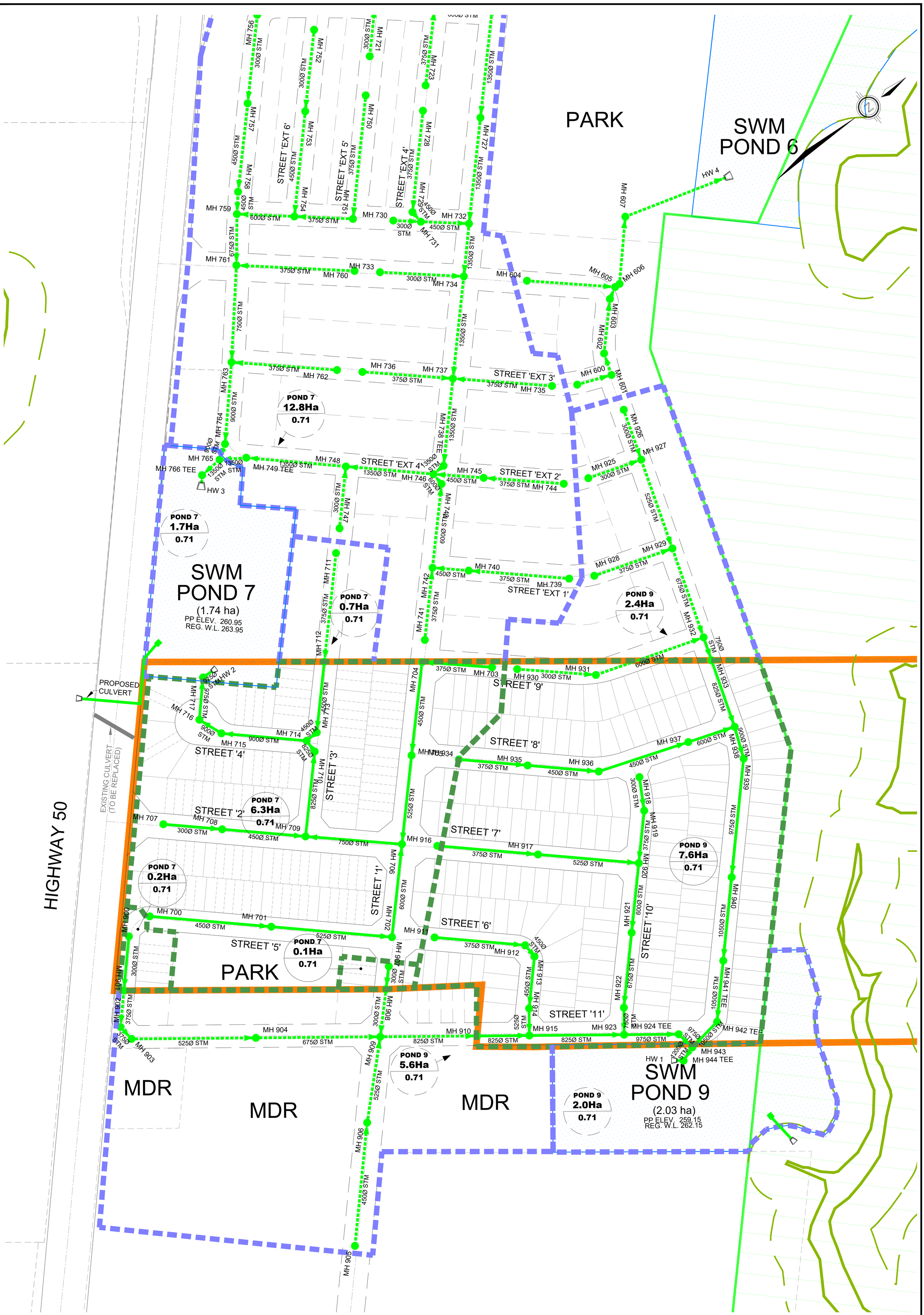
- SUBJECT LANDS
- PROPOSED SANITARY MAINTENANCE HOLE
- PROPOSED SANITARY SEWER
- PROPOSED EXTERNAL SANITARY SEWER
- INTERNAL DRAINAGE AREA
- EXTERNAL DRAINAGE AREA
- PROPOSED SANITARY TRUNK SEWER BY OTHERS
- PROPOSED SANITARY MAINTENANCE HOLE BY OTHERS
- 5.7Ha / 110 / 627 — DRAINAGE AREA, POPULATION DENSITY, POPULATION
- 5.60Ha / 100 / 560 — EXTERNAL DRAINAGE AREA, EXTERNAL POP. DENSITY, EXTERNAL POPULATION

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SANITARY SERVICING PLAN

SCALE:	1:2500	PROJECT No.:	25-1433
DATE:	MARCH 2026	DRAWING:	2



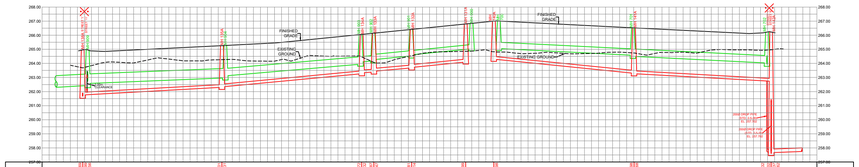
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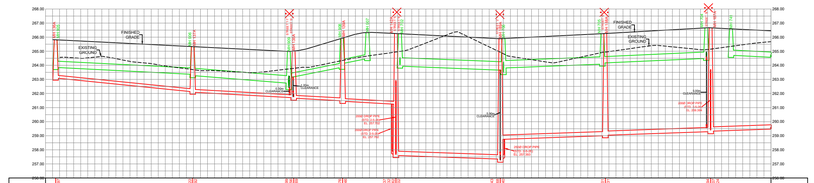
STORM SERVICING PLAN

SCALE:	1:2500	PROJECT No.:	25-1433
DATE:	MARCH 2026	DRAWING:	3

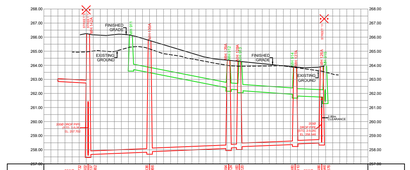
STREET '5'



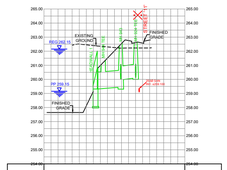
STREET '1'



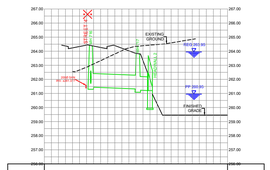
STREET '6'



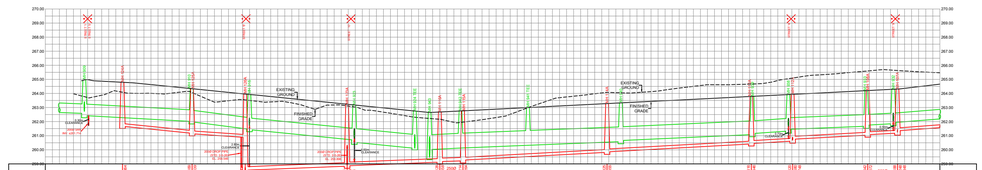
POND 9 INLET



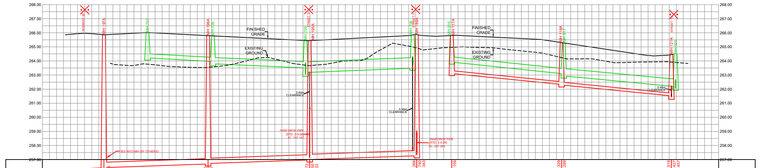
POND 7 SOUTH INLET



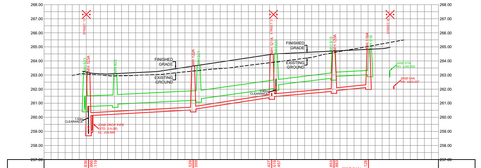
STREET '11'



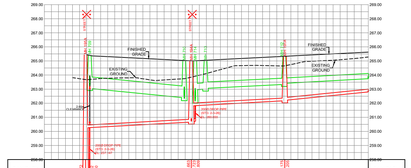
STREET '2' & STREET '7'



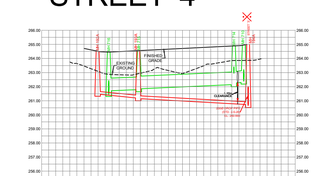
STREET '10'



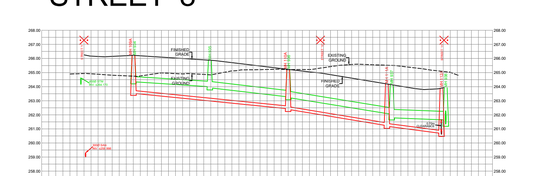
STREET '3'



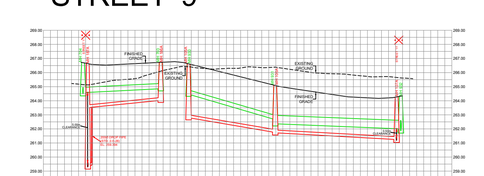
STREET '4'



STREET '8'



STREET '9'



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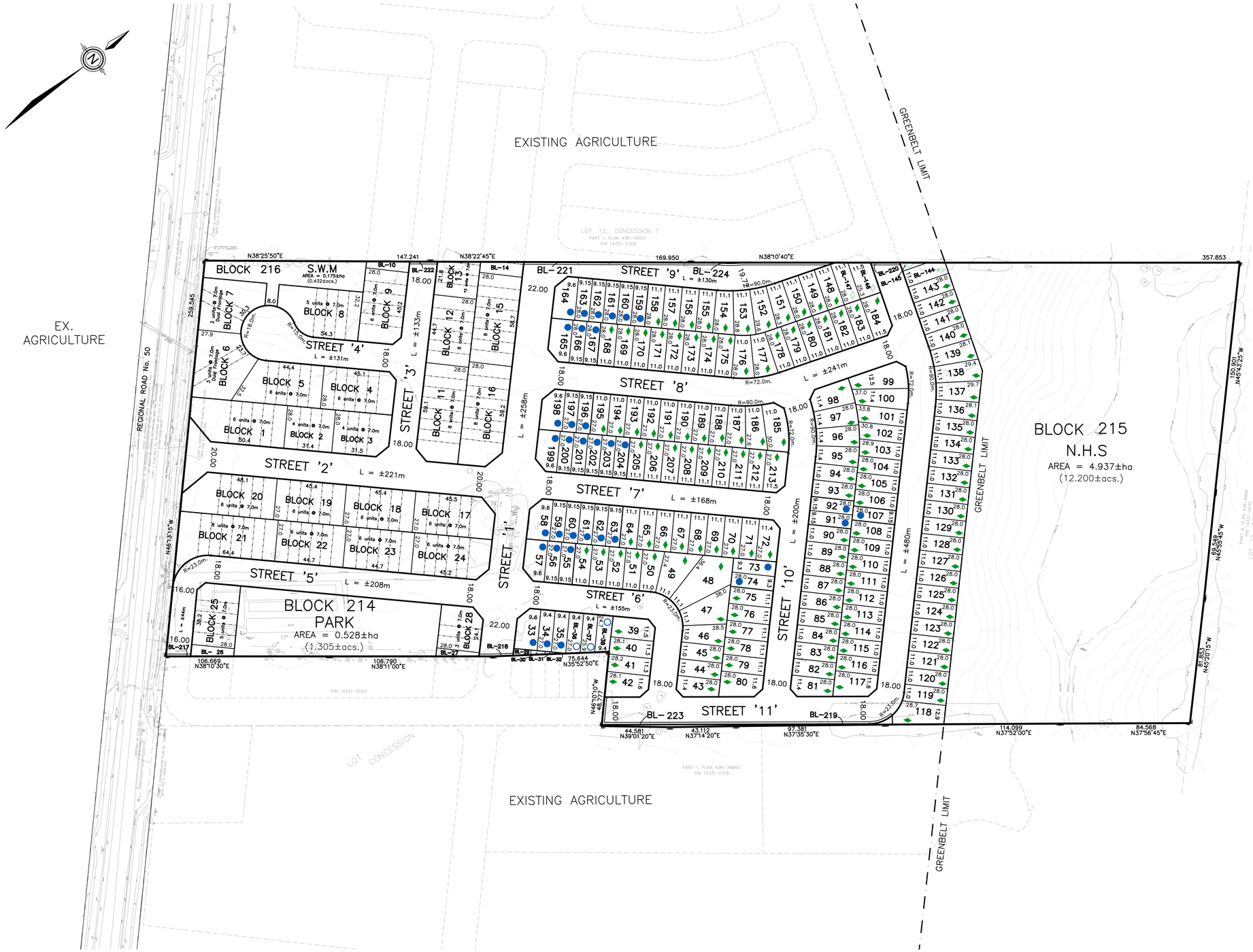
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PLAN AND PROFILE FOR ALL ROADS
BLOCKS AND EASEMENTS

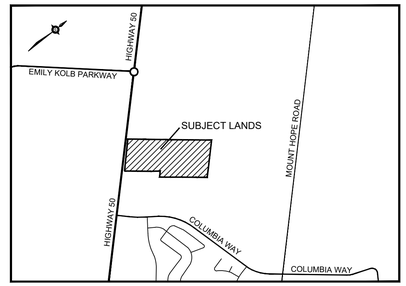
SCALE:	1:2000	PROJECT No.:	25-1433
DATE:	MARCH 2026	DRAWING:	4

APPENDIX A
Draft Plan

DRAFT PLAN OF SUBDIVISION
PART OF LOT 12, CONCESSION 7
(GEOGRAPHIC TOWNSHIP OF ALBION)
TOWN OF CALEDON
REGIONAL MUNICIPALITY OF PEEL
SCALE: 1:250



DRAFT PLAN T-



KEY PLAN NTS

SECTION 51(17), PLANNING ACT, ADDITIONAL INFORMATION

A. AS SHOWN ON DRAFT PLAN
B. AS SHOWN ON DRAFT PLAN
C. AS SHOWN ON DRAFT PLAN
D. SEE SCHEDULE OF LAND USE
E. AS SHOWN ON DRAFT PLAN
F. AS SHOWN ON DRAFT PLAN
G. AS SHOWN ON DRAFT PLAN
H. MUNICIPAL PIPED WATER AVAILABLE AT TIME OF DEVELOPMENT
I. CLAY-LOAM
J. AS SHOWN ON DRAFT PLAN
K. SANITARY AND STORM SEWERS, GARBAGE COLLECTION, FIRE PROTECTION
L. AS SHOWN ON DRAFT PLAN

SURVEYOR'S CERTIFICATE

I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LAND TO BE SUBDIVIDED AS SHOWN ON THIS PLAN, AND THEIR RELATIONSHIP TO THE ADJACENT LAND ARE ACCURATELY AND CORRECTLY SHOWN.

DATE _____, 2025

_____
Ophir N. Dzaldov B.Sc, O.L.S, O.L.I.P.
SCHAEFFER DZALDOV PURCELL LTD.

OWNER'S CERTIFICATE

I AUTHORIZE KLM PLANNING PARTNERS INC. TO PREPARE AND SUBMIT THIS DRAFT PLAN OF SUBDIVISION TO THE CITY OF TOWN OF CALEDON FOR APPROVAL.

OWNER

TIFFANY ROX

C/O BRETT SAITOWITZ
601 MAGNETIC DRIVE
NORTH YORK, ON
M3J3J2

_____
Brett Saitowitz A.S.O

SCHEDULE OF LAND USE

TOTAL AREA OF LAND OWNED BY APPLICANT = 19.047 ± Ha. (47.066 ± Acs.)

	BLOCKS	LOTS	UNITS	± Ha.	± Acs.
SINGLE DETACHED DWELLING					
♦ LOTS 39-54, 64-72, 75-90, 93-106, 108-143, 148-158, 168-195, AND 205-213 MIN. LOT FRONTAGE - 11.0m MIN. AREA - 298.5m²		139	139	4.581	11.320
◇ PLUS BLOCKS 144-147	4		3 *	0.096	0.237
● LOTS 33-35, 55-63, 73, 74, 91, 92, 107, 159-167, AND 196-204 MIN. LOT FRONTAGE - 9.15m MIN. AREA - 247.0m²		35	35	0.967	2.390
○ PLUS BLOCKS 29-32, AND 36-38	7		3 *	0.080	0.198
STANDARD TOWNHOUSE DWELLING					
BLOCK 1-5, 8, 9, 11, 12, 15-25 AND 28 MIN. UNIT FRONTAGE - 7.0m	21		125	2.853	7.050
PLUS BLOCKS 10, 13, 14, 26 AND 27	5		4.5 *	0.134	0.331
DUAL FRONTAGE TOWNHOUSE DWELLING					
BLOCK 6 AND 7 MIN. UNIT FRONTAGE - 7.0m	2		6	0.269	0.665
SUBTOTAL	39	174	315.5	8.980	22.190
BLOCK 214 - PARK	1			0.528	1.305
BLOCK 215 - NATURAL HERITAGE SYSTEM	1			4.937	12.200
BLOCK 216 - STORMWATER MANAGEMENT	1			0.175	0.432
BLOCK 217-222 - 0.3m RESERVE	6			0.015	0.037
BLOCK 223 & 224 - FUTURE DEVELOPMENT	2			0.022	0.054
STREETS				4.390	10.848
22.0m WIDE TOTAL LENGTH = 258±m. AREA = 0.569±Ha.					
20.0m WIDE TOTAL LENGTH = 221±m. AREA = 0.443±Ha.					
18.0m WIDE TOTAL LENGTH = 1834±m. AREA = 3.307±Ha.					
16.0m WIDE TOTAL LENGTH = 43±m. AREA = 0.071±Ha.					
TOTAL LENGTH = 2356±m. AREA = 4.390±Ha.					
TOTAL	**50**	**174**	**315.5**	**19.047**	**47.066**

NOTES:
- ELEVATIONS RELATED TO CANADIAN GEODETIC DATUM
- 0.3M RESERVES HAVE BEEN EXAGGERATED TO 1.5M FOR VISUAL PURPOSES
- (*) UNITS SUBJECT TO FINAL CALCULATION

PROJECT No. P-3600
SCALE 1:1250 5 March 2026
(3600DES2) X-REF: (3600MAS1 & 3600TOPO1)
DWG. No. - 26:1

KLM PLANNING
64 JARDIN DRIVE - UNIT 1B, CONCORD ONT. L4K 3P3 TEL: (905)669-4055

File Location: P:\3600s\3600_Tiffany Rox North Ltd - 14331 Highway 50, Bolton\2 - Design\1 - Draft Plans\3600des2.dwg Plotted By: Stephanie Chalmers March 5, 2026 Last Saved By: schalmers

APPENDIX B
Preliminary Storm Design Sheets

STORM SEWER CALCULATION SHEET (RATIONAL METHOD)

	LOCATION			AREA (Ha)														FLOW						SEWER DATA							
				2 YEAR				5 YEAR				10 YEAR				100 YEAR				Time of	Intensity	Intensity	Intensity	Intensity	Peak Flow	DIA. (mm)	DIA. (mm)	TYPE	SLOPE	LENGTH	CAPACITY	VELOCITY	TIME OF	RATIO
				AREA	R	Indiv.	Accum.	AREA	R	Indiv.	Accum.	AREA	R	Indiv.	Accum.	AREA	R	Indiv.	Accum.	Conc.	2 Year	5 Year	10 Year	100 Year										
Location	From Node	To Node	Phase	(Ha)		2.78 AC	2.78 AC	(Ha)		2.78 AC	2.78 AC	(Ha)		2.78 AC	2.78 AC	(Ha)		2.78 AC	2.78 AC	(min)	(mm/h)	(mm/h)	(mm/h)	(mm/h)	Q (l/s)	(actual)	(nominal)		(%)	(m)	(l/s)	(m/s)	LOW (min	Q/Q full
STREET '3'																																	
	711	712				0.00	0.00	0.48	0.71	0.95	0.95			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	104	375	375	PVC	0.55	86.0	130	1.18	1.22	0.80
	712	713				0.00	0.00	0.31	0.71	0.61	1.56			0.00	0.00			0.00	0.00	11.22	80.88	104.34	127.76	188.62	163	450	450	CONC	0.50	56.0	202	1.27	0.74	0.81
	713	714				0.00	0.00	0.06	0.71	0.12	1.68			0.00	0.00			0.00	0.00	11.95	78.24	101.40	124.19	184.00	170	450	450	CONC	0.55	10.5	211	1.33	0.13	0.80
To STREET '4', Pipe 714 - 715							0.00				1.68				0.00				0.00	12.09													
Contribution From STREET '2', Pipe 706 - 709							0.00				5.96				0.00				0.00	14.08													
Contribution From STREET '2', Pipe 708 - 709							0.00				1.24				0.00				0.00	11.68													
	709	710				0.00	0.00	0.38	0.71	0.75	7.95			0.00	0.00			0.00	0.00	14.08	71.57	93.81	114.97	171.90	746	825	825	CONC	1.00	67.0	1435	2.69	0.42	0.52
To STREET '4', Pipe 710 - 714							0.00				7.95				0.00				0.00	14.49													
STREET '4'																																	
Contribution From STREET '3', Pipe 709 - 710							0.00				7.95				0.00				0.00	14.49													
	710	714				0.00	0.00	0.06	0.71	0.12	8.07			0.00	0.00			0.00	0.00	14.49	70.40	92.47	113.33	169.72	746	825	825	CONC	1.00	10.5	1435	2.69	0.07	0.52
Contribution From STREET '3', Pipe 713 - 714							0.00				1.68				0.00				0.00	12.09													
	714	715				0.00	0.00	0.38	0.71	0.75	10.50			0.00	0.00			0.00	0.00	14.56	70.22	92.26	113.08	169.38	969	900	900	CONC	0.45	67.5	1214	1.91	0.59	0.80
	715	716				0.00	0.00	0.11	0.71	0.22	10.72			0.00	0.00			0.00	0.00	15.15	68.64	90.43	110.85	166.39	969	900	900	CONC	0.45	20.5	1214	1.91	0.18	0.80
To POND 7 S INLET, Pipe 716 - 717							0.00				10.72				0.00				0.00	15.33													
POND 7 S INLET																																	
Contribution From STREET '4', Pipe 715 - 716							0.00				10.72				0.00				0.00	15.33													
	716	717				0.00	0.00	0.19	0.71	0.38	11.09			0.00	0.00			0.00	0.00	15.33	68.17	89.89	110.19	165.50	997	975	975	CONC	0.40	33.5	1417	1.90	0.29	0.70
	717	HW2				0.00	0.00	0.09	0.65	0.16	11.26			0.00	0.00			0.00	0.00	15.62	67.43	89.02	109.13	164.07	1002	975	975	CONC	0.40	9.0	1417	1.90	0.08	0.71
STREET '8'																																	
	934	935				0.00	0.00	0.38	0.71	0.75	0.75			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	82	375	375	PVC	0.65	54.0	141	1.28	0.70	0.58
	935	936				0.00	0.00	0.40	0.71	0.79	1.54			0.00	0.00			0.00	0.00	10.70	82.84	106.52	130.38	191.99	164	450	450	CONC	1.15	56.0	306	1.92	0.49	0.54
	936	937				0.00	0.00	0.52	0.71	1.03	2.57			0.00	0.00			0.00	0.00	11.19	80.99	104.46	127.90	188.81	268	450	450	CONC	1.60	74.0	361	2.27	0.54	0.74
	937	938				0.00	0.00	0.27	0.71	0.53	3.10			0.00	0.00			0.00	0.00	11.73	79.01	102.26	125.24	185.37	317	600	600	CONC	0.40	38.5	388	1.37	0.47	0.82
To STREET '11', Pipe 938 - 939							0.00				3.10				0.00				0.00	12.20													
STREET '9'																																	
	703	704				0.00	0.00	0.31	0.71	0.61	0.61			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	67	375	375	PVC	0.50	55.0	124	1.12	0.82	0.54
To STREET '1', Pipe 704 - 705							0.00				0.61				0.00				0.00	10.82													
	930	931				0.00	0.00	0.44	0.71	0.87	0.87			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	95	300	300	PVC	2.90	62.0	165	2.33	0.44	0.58
	931	932				0.00	0.00	0.63	0.71	1.24	2.11			0.00	0.00			0.00	0.00	10.44	83.87	107.65	131.75	193.73	227	600	600	CONC	0.40	89.5	388	1.37	1.09	0.59
To STREET '11', Pipe 932 - 933							0.00				2.11				0.00				0.00	11.53													
STREET 'EXT 1'																																	
	928	929				0.00	0.00	0.46	0.71	0.91	0.91			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	100	375	375	PVC	1.55	65.0	218	1.98	0.55	0.46
To STREET '11', Pipe 929 - 932							0.00				0.91				0.00				0.00	10.55													
STREET 'EXT 2																																	
	925	927				0.00	0.00	0.31	0.71	0.61	0.61			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	67	300	300	PVC	1.00	44.0	97	1.37	0.54	0.69
To STREET '11', Pipe 927 - 929							0.00				0.61				0.00				0.00	10.54													
STREET '2'																																	
	707	708				0.00	0.00	0.26	0.71	0.51	0.51			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	56	300	300	PVC	0.55	46.5	72	1.01	0.76	0.78
	708	709				0.00	0.00	0.37	0.71	0.73	1.24			0.00	0.00			0.00	0.00	10.76	82.61	106.25	130.07	191.58	132	450	450	CONC	0.45	66.0	191	1.20	0.91	0.69
To STREET '3', Pipe 709 - 710							0.00				1.24				0.00				0.00	11.68													
	916	917				0.00	0.00	0.56	0.71	1.11	1.11			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	121	375	375	PVC	1.05	79.5	180	1.63	0.81	0.67
	917	920				0.00	0.00	0.56	0.71	1.11	2.21			0.00	0.00			0.00	0.00	10.81	82.41	106.04	129.81	191.25	234	525	525	CONC	0.95	79.5	419	1.94	0.68	0.56
To STREET '10', Pipe 920 - 921							0.00				2.21				0.00				0.00	11.50													

Definitions:
Q = 2.78 AIR, where
Q = Peak Flow in Litres per second (L/s)
A = Areas in hectares (ha)
I = Rainfall Intensity (mm/h)
R = Runoff Coefficient

Designed: CJM	PROJECT: TIFFANY ROX NORTH HILL LTD.		
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Dwg. Reference: 3	File Ref: 25-1417	Date: 23 Mar 2026	Sheet No. SHEET 1 OF 3

STORM SEWER CALCULATION SHEET (RATIONAL METHOD)

	LOCATION			AREA (Ha)														FLOW						SEWER DATA								
				2 YEAR				5 YEAR				10 YEAR				100 YEAR				Time of	Intensity	Intensity	Intensity	Intensity	Peak Flow	DIA. (mm)	DIA. (mm)	TYPE	SLOPE	LENGTH	CAPACITY	VELOCITY	TIME OF	RATIO
				AREA	R	Indiv.	Accum.	AREA	R	Indiv.	Accum.	AREA	R	Indiv.	Accum.	AREA	R	Indiv.	Accum.	Conc.	2 Year	5 Year	10 Year	100 Year										
Location	From Node	To Node	Phase	(Ha)		2.78 AC	2.78 AC	(Ha)		2.78 AC	2.78 AC	(Ha)		2.78 AC	2.78 AC	(Ha)		2.78 AC	2.78 AC	(min)	(mm/h)	(mm/h)	(mm/h)	(mm/h)	Q (l/s)	(actual)	(nominal)		(%)	(m)	(l/s)	(m/s)	LOW (min	Q/Q full
Contribution From STREET '1', Pipe 702 - 706							0.00				2.92				0.00				0.00	13.28													
Contribution From STREET '1', Pipe 705 - 706							0.00				2.19				0.00				0.00	12.77													
	706	709				0.00	0.00	0.43	0.71	0.85	5.96			0.00	0.00			0.00	0.00	13.28	73.92	96.51	118.25	176.24	575	750	750	CONC	0.40	76.0	704	1.59	0.79	0.82
To STREET '3', Pipe 709 - 710							0.00				5.96				0.00				0.00	14.08													
STREET '10'																																	
	918	919				0.00	0.00	0.19	0.71	0.38	0.38			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	41	300	300	PVC	0.50	26.5	68	0.97	0.46	0.60
	919	920				0.00	0.00	0.29	0.71	0.57	0.95			0.00	0.00			0.00	0.00	10.46	83.82	107.59	131.68	193.65	102	375	375	PVC	1.20	41.5	192	1.74	0.40	0.53
Contribution From STREET '2', Pipe 917 - 920							0.00				2.21				0.00				0.00	11.50													
	920	921				0.00	0.00	0.39	0.71	0.77	3.93			0.00	0.00			0.00	0.00	11.50	79.85	103.20	126.37	186.83	405	600	600	CONC	1.40	55.0	727	2.57	0.36	0.56
	921	922				0.00	0.00	0.42	0.71	0.83	4.76			0.00	0.00			0.00	0.00	11.86	78.58	101.78	124.65	184.61	484	675	675	CONC	0.50	59.0	594	1.66	0.59	0.81
	922	923				0.00	0.00	0.15	0.71	0.30	5.05			0.00	0.00			0.00	0.00	12.45	76.57	99.52	121.91	181.03	503	750	750	CONC	0.40	21.5	704	1.59	0.22	0.71
To STREET '11', Pipe 923 - 924TEE							0.00				5.05				0.00				0.00	12.67													
STREET '11'																																	
	911	912				0.00	0.00	0.51	0.71	1.01	1.01			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	110	375	375	PVC	1.90	71.5	242	2.19	0.54	0.46
	912	913				0.00	0.00	0.08	0.71	0.16	1.16			0.00	0.00			0.00	0.00	10.54	83.47	107.20	131.22	193.05	125	450	450	CONC	0.40	11.5	180	1.13	0.17	0.69
	913	914				0.00	0.00	0.29	0.71	0.57	1.74			0.00	0.00			0.00	0.00	10.71	82.80	106.47	130.33	191.92	185	450	450	CONC	0.65	41.0	230	1.45	0.47	0.80
	914	915				0.00	0.00	0.15	0.71	0.30	2.03			0.00	0.00			0.00	0.00	11.19	81.00	104.47	127.91	188.82	212	525	525	CONC	0.40	21.5	272	1.26	0.29	0.78
To STREET '11', Pipe 915 - 923							0.00				2.03				0.00				0.00	11.47													
STREET '1'																																	
	905	906				0.00	0.00	0.69	0.71	1.36	1.36			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	149	450	450	CONC	0.50	97.5	202	1.27	1.28	0.74
	906	909				0.00	0.00	0.48	0.71	0.95	2.31			0.00	0.00			0.00	0.00	11.28	80.64	104.08	127.44	188.21	240	525	525	CONC	0.80	68.0	385	1.78	0.64	0.62
To STREET '11', Pipe 909 - 910							0.00				2.31				0.00				0.00	11.92													
	907	908				0.00	0.00	0.10	0.71	0.20	0.20			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	22	300	300	PVC	2.00	18.0	137	1.93	0.16	0.16
	908	909				0.00	0.00	0.27	0.71	0.53	0.73			0.00	0.00			0.00	0.00	10.16	85.05	108.94	133.31	195.71	80	300	300	PVC	2.00	37.5	137	1.93	0.32	0.58
To STREET '11', Pipe 909 - 910							0.00				0.73				0.00				0.00	10.48													
Contribution From STREET '9', Pipe 703 - 704							0.00				0.61				0.00				0.00	10.82													
	704	705				0.00	0.00	0.41	0.71	0.81	1.42			0.00	0.00			0.00	0.00	10.82	82.40	106.03	129.79	191.23	151	450	450	CONC	0.45	74.0	191	1.20	1.03	0.79
	705	706				0.00	0.00	0.39	0.71	0.77	2.19			0.00	0.00			0.00	0.00	11.84	78.63	101.83	124.71	184.69	223	525	525	CONC	0.40	70.0	272	1.26	0.93	0.82
To STREET '2', Pipe 706 - 709							0.00				2.19				0.00				0.00	12.77													
Contribution From STREET '1', Pipe 701 - 702							0.00				2.11				0.00				0.00	12.39													
	702	706				0.00	0.00	0.41	0.71	0.81	2.92			0.00	0.00			0.00	0.00	12.39	76.76	99.74	122.17	181.38	291	600	600	CONC	0.40	73.5	388	1.37	0.89	0.75
To STREET '2', Pipe 706 - 709							0.00				2.92				0.00				0.00	13.28													
STREET '1'																																	
	700	701				0.00	0.00	0.54	0.71	1.07	1.07			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	117	450	450	CONC	0.50	96.0	202	1.27	1.26	0.58
	701	702				0.00	0.00	0.53	0.71	1.05	2.11			0.00	0.00			0.00	0.00	11.26	80.72	104.16	127.53	188.33	220	525	525	CONC	0.50	95.0	304	1.40	1.13	0.72
To STREET '1', Pipe 702 - 706							0.00				2.11				0.00				0.00	12.39													
	900	901				0.00	0.00	0.24	0.71	0.47	0.47			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	52	300	300	PVC	1.00	43.0	97	1.37	0.52	0.54
	901	902				0.00	0.00	0.20	0.71	0.39	0.87			0.00	0.00			0.00	0.00	10.52	83.55	107.29	131.33	193.19	93	375	375	PVC	0.95	28.5	171	1.55	0.31	0.55
	902	903				0.00	0.00	0.09	0.71	0.18	1.05			0.00	0.00			0.00	0.00	10.83	82.35	105.97	129.72	191.14	111	375	375	PVC	0.60	13.0	136	1.23	0.18	0.82
	903	904				0.00	0.00	0.69	0.71	1.36	2.41			0.00	0.00			0.00	0.00	11.01	81.67	105.22	128.82	189.98	253	525	525	CONC	0.85	97.5	396	1.83	0.89	0.64
	904	909				0.00	0.00	0.69	0.71	1.36	3.77			0.00	0.00			0.00	0.00	11.89	78.45	101.63	124.47	184.37	383	675	675	CONC	0.40	97.5	532	1.49	1.09	0.72
To STREET '11', Pipe 909 - 910							0.00				3.77				0.00				0.00	12.99													
STREET '11'																																	
Contribution From STREET '1', Pipe 904 - 909							0.00				3.77				0.00				0.00	12.99													
Contribution From STREET '1', Pipe 906 - 909							0.00				2.31				0.00				0.00	11.92													
Contribution From STREET '1', Pipe 908 - 909							0.00				0.73				0.00				0.00	10.48													
	909	910				0.00	0.00	0.54	0.71	1.07	7.88			0.00	0.00			0.00	0.00	12.99	74.83	97.54	119.51	177.89	768	825	825	CONC	0.55	76.0	1065	1.99	0.64	0.72
	910	915				0.00	0.00	0.29	0.71	0.57	8.45			0.00	0.00			0.00	0.00	13.62	72.88	95.33	116.81	174.34	805	825	825	CONC	1.30	41.0	1637	3.06	0.22	0.49
Contribution From STREET '11', Pipe 914 - 915							0.00				2.03				0.00				0.00	11.47													
	915	923				0.00	0.00	0.53	0.71	1.05	11.53			0.00	0.00			0.00	0.00	13.85	72.23	94.57	115.90	173.13	1090	825	825	CONC	1.00	74.5	1435	2.69	0.46	0.76

Definitions:
Q = 2.78 AIR, where
Q = Peak Flow in Litres per second (L/s)
A = Areas in hectares (ha)
I = Rainfall Intensity (mm/h)
R = Runoff Coefficient

Designed: CJM	PROJECT: TIFFANY ROX NORTH HILL LTD.		
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Dwg. Reference: 3	File Ref: 25-1417	Date: 23 Mar 2026	Sheet No. SHEET 2 OF 3

STORM SEWER CALCULATION SHEET (RATIONAL METHOD)

	LOCATION			AREA (Ha)														FLOW						SEWER DATA							
				2 YEAR				5 YEAR				10 YEAR				100 YEAR				Time of	Intensity	Intensity	Intensity	Intensity	Peak Flow	DIA. (mm)	DIA. (mm)	TYPE	SLOPE	LENGTH	CAPACITY	VELOCITY	TIME OF	RATIO
				AREA	R	Indiv.	Accum.	AREA	R	Indiv.	Accum.	AREA	R	Indiv.	Accum.	AREA	R	Indiv.	Accum.	Conc.	2 Year	5 Year	10 Year	100 Year										
Location	From Node	To Node	Phase	(Ha)		2.78 AC	2.78 AC	(Ha)		2.78 AC	2.78 AC	(Ha)		2.78 AC	2.78 AC	(Ha)		2.78 AC	2.78 AC	(min)	(mm/h)	(mm/h)	(mm/h)	(mm/h)	Q (l/s)	(actual)	(nominal)		(%)	(m)	(l/s)	(m/s)	LOW (min	Q/Q full
Contribution From STREET '10', Pipe 922 - 923							0.00				5.05				0.00				0.00	12.67													
	923	924TEE				0.00	0.00	0.30	0.71	0.59	17.17			0.00	0.00			0.00	0.00	14.31	70.90	93.05	114.05	170.66	1598	975	975	CONC	1.00	43.0	2241	3.00	0.24	0.71
To POND 9 INLET, Pipe 924TEE - 943							0.00				17.17				0.00				0.00	14.55													
	926	927				0.00	0.00	0.29	0.71	0.57	0.57			0.00	0.00			0.00	0.00	10.00	85.69	109.64	134.16	196.80	63	300	300	PVC	1.00	41.5	97	1.37	0.51	0.65
Contribution From STREET 'EXT 2, Pipe 925 - 927							0.00				0.61				0.00				0.00	10.54													
	927	929				0.00	0.00	0.52	0.71	1.03	2.21			0.00	0.00			0.00	0.00	10.54	83.50	107.24	131.26	193.11	237	525	525	CONC	0.50	74.0	304	1.40	0.88	0.78
Contribution From STREET 'EXT 1', Pipe 928 - 929							0.00				0.91				0.00				0.00	10.55													
	929	932				0.00	0.00	0.52	0.71	1.03	4.14			0.00	0.00			0.00	0.00	11.41	80.16	103.54	126.78	187.37	429	675	675	CONC	0.80	74.0	752	2.10	0.59	0.57
Contribution From STREET '9', Pipe 931 - 932							0.00				2.11				0.00				0.00	11.53													
	932	933				0.00	0.00	0.14	0.71	0.28	6.53			0.00	0.00			0.00	0.00	12.00	78.08	101.21	123.97	183.72	661	750	750	CONC	0.55	19.0	826	1.87	0.17	0.80
	933	938				0.00	0.00	0.39	0.71	0.77	7.30			0.00	0.00			0.00	0.00	12.17	77.50	100.56	123.18	182.69	734	825	825	CONC	0.40	55.5	908	1.70	0.54	0.81
Contribution From STREET '8', Pipe 937 - 938							0.00				3.10				0.00				0.00	12.20													
	938	939				0.00	0.00	0.18	0.71	0.36	10.76			0.00	0.00			0.00	0.00	12.72	75.70	98.53	120.71	179.47	1060	900	900	CONC	0.70	26.0	1515	2.38	0.18	0.70
	939	940				0.00	0.00	0.66	0.71	1.30	12.06			0.00	0.00			0.00	0.00	12.90	75.12	97.87	119.91	178.42	1180	975	975	CONC	0.45	93.5	1503	2.01	0.77	0.79
	940	941TEE				0.00	0.00	0.47	0.71	0.93	12.99			0.00	0.00			0.00	0.00	13.67	72.74	95.17	116.62	174.08	1236	1050	1050	CONC	0.40	66.0	1727	1.99	0.55	0.72
	941TEE	942TEE				0.00	0.00	0.34	0.71	0.67	13.66			0.00	0.00			0.00	0.00	14.22	71.15	93.33	114.39	171.12	1275	1050	1050	CONC	0.40	48.5	1727	1.99	0.41	0.74
	942TEE	943				0.00	0.00	0.20	0.71	0.39	14.05			0.00	0.00			0.00	0.00	14.63	70.02	92.04	112.81	169.01	1293	1050	1050	CONC	0.40	28.0	1727	1.99	0.23	0.75
To POND 9 INLET, Pipe 943 - 944TEE							0.00				14.05				0.00				0.00	14.86													
POND 9 INLET																																	
Contribution From STREET '11', Pipe 923 - 924TEE							0.00				17.17				0.00				0.00	14.55													
	924TEE	943				0.00	0.00	0.11	0.71	0.22	17.39			0.00	0.00			0.00	0.00	14.55	70.24	92.29	113.12	169.42	1605	975	975	CONC	1.00	15.0	2241	3.00	0.08	0.72
Contribution From STREET '11', Pipe 942TEE - 943							0.00				14.05				0.00				0.00	14.86													
	943	944TEE				0.00	0.00	0.12	0.65	0.22	31.66			0.00	0.00			0.00	0.00	14.86	69.39	91.30	111.92	167.82	2891	1200	1200	CONC	0.85	12.5	3594	3.18	0.07	0.80
	944TEE	HW1				0.00	0.00	0.06	0.65	0.11	31.77			0.00	0.00			0.00	0.00	14.93	69.22	91.10	111.67	167.49	2894	1200	1200	CONC	0.85	5.5	3594	3.18	0.03	0.81

Definitions:
Q = 2.78 AIR, where
Q = Peak Flow in Litres per second (L/s)
A = Areas in hectares (ha)
I = Rainfall Intensity (mm/h)
R = Runoff Coefficient

Designed: CJM	PROJECT: TIFFANY ROX NORTH HILL LTD.		
Checked: CMK	LOCATION: BOLTON NORTH HILL		
Dwg. Reference: 3	File Ref: 25-1417	Date: 23 Mar 2026	Sheet No. SHEET 3 OF 3

APPENDIX C
Hydro Geotechnical Report and Water Balance

PATERSON GROUP

Hydrogeological Study

Proposed Residential Development

14337 Highway 50
Bolton, Ontario

Prepared for Tiffany Rox North Hill Ltd.

Report TH0106-REP.01 dated March 11, 2026

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Appendices

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Appendix 2 Soil Profile and Test Data Sheets
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Table 6 - Post-Development Annual Water Budget Calculations
Sample Calculations - Dupuit Forchheimer
Hydraulic Conductivity Results - Falling and Rising Head Tests
Water Quality Results

1.0 INTRODUCTION

1.1 Background

Paterson Group (Paterson) was retained by Tiffany Rox North Hill Ltd to complete a hydrogeological study for the proposed residential development at 14337 Highway 50, Bolton, Ontario (hereinafter referred to as the "subject site"). The location of the subject site is shown on Drawing TH0106-1 - Site Plan appended to this report. This report incorporates the findings of Paterson Report TG0257-REP.01 Revision 1 dated March 6, 2026.

1.2 Scope of Work

Paterson has completed this report in accordance with the scope prepared by Paterson based on our understanding of the proposed development at the time of report preparation as it relates to the local/regional hydrogeological system. As per the agreed upon scope, the purpose of this study was to:

- Characterize the hydrogeological setting of the subject site. Consideration was given to bedrock and surficial geology, aquifer systems, groundwater levels, hydraulic properties and catchment characteristics.
- Provide a preliminary estimate of dewatering rates during construction.
- Assess potential risks to the surrounding infrastructure, environment and water supply of existing well users.
- Complete a site-specific water budget assessment.

2.0 PREVIOUS REPORTS

In addition to a review of the general literature summarized in the following sections and in the 'References' section of this report (MECP water well mapping, available geological and physiographic mapping), Paterson reviewed the following site-specific reports:

- TG0257-1 Revision 1"Geotechnical Investigation – 14337 Highway 50 – Bolton, Ontario" - prepared by Paterson Group – March 6, 2026.
- TE0197-Rep.01 - Phase I ESA – 14337 Highway 50 Bolton - prepared by Paterson Group – January 23, 2026.
- TE0197-REP.02 – Phase II ESA – 14337 Highway 50 Bolton - prepared by Paterson Group – February 11, 2026.

3.0 METHOD OF INVESTIGATION

3.1 Records Review

A review of available geological, and hydrogeological data was completed as a part of this assessment. However, the literature review and previous reports did not provide site-specific data regarding overburden and bedrock aquifers, recharge and discharge conditions or flow contributions to the nearby water features. Further detail is provided in the following sections.

3.2 Field Program

The geotechnical and hydrogeological field programs were developed to assess geology, groundwater conditions, hydraulic gradients and the overall hydrologic/hydrogeologic function of the subject site. The test holes were advanced to various depths across the site to assess hydrogeological and geotechnical conditions.

The geotechnical field investigations completed by Paterson have been carried out at the subject site between January 28, 2025 and December 9, 2025. During that time, a total of thirteen (13) boreholes were advanced to a maximum depth of 9.8 m below ground surface (bgs). The location of the test holes are shown on Drawing TG0257-1- Test Hole Location Plan located in Appendix 2.

Soil samples were obtained from the boreholes by means of split spoon sampling and the sampling of shallow soils directly from auger flights. Split-spoon samples were taken at approximate 0.76 m intervals. The depth at which split-spoon and auger flight samples were obtained from the test holes are shown as "**SS**" and "**AU**" respectively on the Soil Profile and Test Data sheets, appended to this report. All samples were classified on site, placed in sealed plastic bags and were transported to our laboratory for further review and testing. Transportation of the samples was completed in accordance with ASTM D4220-95 (2007) - Standard Practice for Preserving and Transporting Soils.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split-spoon sampler 300 mm into the ground after an initial penetration of 150 mm using a 63.5 kg hammer falling from a height of 760 mm. This test was done in accordance with ASTM D1586-11 - Standard Method for Penetration Test and Split-Barrel Sampling of Soils.

The thickness of the overburden was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at BH 12-25. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profiles encountered at the test hole locations.

Drawdown Analysis - Hydraulic Conductivity Testing

Hydraulic conductivity testing was completed at select monitoring wells installed during the geotechnical investigations. Falling head and rising head tests ("slug tests") were completed in accordance with ASTM Standard Test Method D 4404 - Field Procedure for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers.

Slug testing was completed on February 4 and 5, 2025 and January 22, 2026, by Paterson personnel. The general test method consisted of measuring the static water level in the well, followed by inducing a near-instantaneous change of head in the monitoring well and subsequent monitoring of water level recovery with an electronic water level meter and a water level data logger. The change in head was induced by the introduction of a 0.9 m in length and 19 or 38 mm in diameter acetal slug. The slug was introduced to raise the water level in the monitoring well, following which the decrease in water level over time was monitored (falling head test). Once the water level had stabilized (or nearly stabilized), the slug was then removed to lower the groundwater level, following which the increase in water level over time was monitored (rising head test).

Following the completion of the slug tests, the test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent, zero-storage assumption, and a screen length significantly greater than the monitoring well diameter. The assumption regarding aquifer storage is considered to be appropriate for groundwater flow through the overburden aquifer. The assumption regarding screen length and well diameter is considered to be met based upon a typical length of approximately 1.5 m and a diameter of 0.05 m.

While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site. Hvorslev analysis is based on the line of best fit through the field data (hydraulic head recovery vs. time), plotted on a semi-logarithmic scale. In cases where the initial hydraulic head displacement is known with relative certainty, such as in this case where a physical slug has been introduced, the line of best fit is considered to pass through the origin.

In cases where the initial hydraulic head displacement is known with less certainty (e.g. a bail test, where water is pumped rapidly from the well), the best-fit line is drawn regardless of the origin.

3.3 Laboratory Testing

All soil samples were retained for laboratory review following the field portion of the subsurface investigation. The soils were classified in general accordance with ASTM D2488-09a, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Based on the soil descriptions across the subject site during the geotechnical investigations, these samples are considered to be sufficiently representative of the site.

3.4 Monitoring Well Installations

As part of the field program, Monitoring wells were installed in nine (9) boreholes to permit the monitoring of groundwater levels and conduct drawdown analyses. All monitoring well installations were constructed in accordance with ASTM D5092 standards.

3.5 Water Level Measurements

Following the completion of the drilling program, groundwater levels were measured at the monitoring well and Piezometer locations. Water levels were measured relative to the ground surface elevation at each location and are noted on the Soil Profile and Test Data sheets, appended to this report. In addition, a subsequent groundwater level monitoring program is ongoing using data loggers installed at select monitoring well locations to accurately monitor fluctuations in the groundwater levels across the subject site. The groundwater monitoring program will be completed in June 2026, following the spring freshet. The results from the monitoring report will be included in an update to the hydrogeological report.

3.6 Surveying

The test hole locations and ground surface elevations at each test hole location completed by Paterson were surveyed using a GPS unit with respect to a geodetic datum. The locations and ground surface elevations for each test hole are presented in Drawing TG0257-1 - Test Hole Location Plan.

4.0 REVIEW AND EVALUATION

4.1 Physical Setting

Currently, the subject site consists primarily of agricultural lands. Commercial and residential buildings and associated gravel surfaced access lane are located within the southwest corner of the property. A greenbelt area consisting of a pond, wetland and an unnamed tributary to the Humber River have been noted within the eastern limits of the site. The subject site is bordered to the west by Highway 50, to the north, east, and southeast by vacant agricultural lands, and to the southwest by a commercial development. The site generally slopes downward from west to east with an approximate elevation difference of 7 to 8 m.

Based on TRCA watershed mapping and subwatershed mapping, the subject site is located within the Humber River watershed, specifically the Main Humber subwatershed. Surface water features within the subject site include a pond, wetland and an unnamed tributary to the Humber River within the eastern limits of the site. Several unnamed drainage ditches, creeks and conveyance channels have also been noted within the subject area.

According to available mapping from the Ontario Geologic Survey (OGS; MRD228), the subject site is located in a Till Plains (Drumlinized) physiographic region and generally coincides with our field observations discussed below.

4.2 Geology

Surficial Geology

Overburden mapping provided by the OGS online mapping tool was reviewed as part of this assessment. Available mapping (MRD 128) indicates that overburden soils throughout the subject site consist of glacial till deposits consisting of clay to silt-textured till. Overburden soil mapping is shown on Drawing TH0106-2 - Surficial Geology Plan within Appendix 1.

Overburden soils identified during the geotechnical investigation completed by Paterson generally consisted of topsoil or an agricultural disturbed zone overlying silty clay and/or a glacial till deposit. The glacial till was typically comprised of a brown silty clay to clayey silt with varying amounts of sand and gravel. A fill material was observed underlying the topsoil layer at borehole BH 7-25 and BH 8-25 and consisted of brown silty sand with varying amounts of clay, gravel and organics. Discontinuous layers of compact silty sand to sandy silt layer was also encountered at BH1-25, BH4-25, BH5-25 and BH12-25. Overburden thickness was recorded at approximately 9.8 m across the subject site.

Specific details are provided on the Soil Profile and Test Data Sheets attached within Appendix 2 of this report.

Bedrock Geology

Bedrock mapping, provided by the Ontario Geologic Survey was reviewed as a part of this assessment. Available mapping (MRD 219) indicates that bedrock at the subject site consists of shale and limestone of the Georgian Bay Formation.

Practical refusal to the DCPT was encountered at an approximate depth of 8.2 m below the existing ground surface at Borehole BH 12-25.

Bedrock geology mapping is shown on Drawing TH0106-3 - Bedrock Geology Plan within Appendix 1.

Karst Features

The term “karst” refers to a geologic formation characterized by the dissolution of carbonate bedrock, such as limestone or dolostone. In order for karstification to occur, precipitation must be able to infiltrate the top of the bedrock, causing dissolution which enlarges previously existing joints and bedding planes.

Based on available mapping by the OGS (GRS 005), the subject site is located within an area that does not contain any known, inferred, or potential karstic landforms.

4.3 Hydrogeological Setting

Existing Aquifer Systems

Aquifer systems may be defined as geological media, either overburden soils or fractured bedrock, which permit the movement of groundwater under hydraulic gradients. In general, aquifer systems may be present in overburden soils or bedrock. Given that bedrock was not identified at the subject site during Paterson's geotechnical investigation, it is Paterson's opinion that overburden materials found at the site are representative of the local aquifer system for the proposed development.

The deeper bedrock aquifer system is likely to include the shale and limestone bedrock of the Georgian Bay Formation.

Groundwater Levels

Groundwater was observed at piezometer and monitoring well locations at the subject site on January 8, 2025, December 22, 2025 and January 8, 2026, using an electronic water level meter. Manual groundwater measurements ranged between 1.04 to 7.21 m bgs and are displayed on the Soil Profile and Test Data sheets in Appendix 2. The manual measured groundwater levels are presented in Table 1.

Additional information regarding groundwater levels will be available once the on-going groundwater monitoring program is completed in June 2026.

Table 1 – Summary of Groundwater Level readings					
Test Hole ID	**Structure**	**Ground Surface Elevation (m)**	**Groundwater Depth (m)**	**Groundwater Elevation (m)**	**Recording Date**
BH 1-25	PZ	261.60	Dry	N/A	December 22, 2025
BH 2-25	MW	263.86	7.21	256.65	
BH 3-25	PZ	263.03	Dry	N/A	
BH 4-25	PZ	266.5	Dry	N/A	
BH 5-25	MW	264.25	Dry	N/A	
BH 6-25	MW	264.09	Dry	N/A	
BH 7-25	MW	265.3	3.03	262.27	
BH 8-25	MW	266.20	4.35	261.85	
BH 10-25	MW	263.78	1.5	262.28	
BH 11-25	MW	266.05	4.45	261.6	
BH 12-25	MW	264.18	2.96	261.22	
BH 13-25	MW	266.11	1.04	265.07	January 8, 2026
BH 11-25	MW	266.05	Dry	N/A	February 4, 2025
BH 12-25	MW	264.18	1.30	262.88	
BH 13-25	MW	266.11	1.33	264.78	
Note: The ground surface elevation at each monitoring well location was surveyed using a handheld GPS unit and references to a geodetic datum. MW – Monitoring Well and PZ - Piezometer					

Horizontal Hydraulic Gradients

Due to the nature of the water levels obtained from field work conducted at the subject site (groundwater monitoring wells), the absolute direction of horizontal hydraulic gradients in the vicinity of the subject site was not determined. However, using the available data, it was possible to approximate the horizontal hydraulic gradients in the overburden given that the horizontal hydraulic gradient between any 2 points is the slope of the hydraulic head between those points:

$$i=h_2-h_1/L$$

Where: i = horizontal gradient
h = water level (m bgs)
L = horizontal distance between test hole locations

Using the above noted formula, the horizontal hydraulic gradients were observed to be in an approximate northeasterly orientation with a magnitude generally ranging between 0.00012 to 0.03205 m/m in the overburden materials. The regional groundwater flow is anticipated towards the Humber River tributary located northeasterly of the subject site.

Hydraulic Conductivity

Hydraulic conductivity testing (slug testing) was completed by Paterson as part of the field investigations at the subject site. The test data was analyzed as per the method set out by Hvorslev (1951). The result of the testing yielded hydraulic conductivity values of approximately $1.82x10^{-6}$ to $3.10x10^{-8}$ m/sec for glacial till layer with a geometric mean hydraulic conductivity of 1.45×10^{-7} m/sec. The hydraulic conductivity results are summarized in Table 2 below and have been included in Appendix 3.

Table 2 – Summary of Hydraulic Conductivity Testing Results

Test Hole ID	Ground Surface Elevation (m asl)	Testing Depth Interval (m bgs)	Testing Elevation Interval (m asl)	Hydraulic Conductivity (m/sec)	Test Type	Strata
BH 2-25	263.86	9.1-7.6	254.76-256.26	$4.10x10^{-8}$	Falling Head	Glacial Till
BH 7-25	265.30	3.0-6.0	262.3-259.3	$2.06x10^{-7}$	Falling Head	Glacial Till
BH 8-25	266.2	3.0-6.0	263.2-260.2	$1.09x10^{-8}$	Falling Head	Glacial Till
BH 12-25	264.18	4.6-6.1	259.6-258.1	$3.10x10^{-8}$	Falling Head	Glacial Till
BH 13-25	266.11	4.6-6.1	261.5-260	$1.82x10^{-6}$	Falling Head	Glacial Till
				$1.83x10^{-6}$	Rising Head	Glacial Till

Water Quality

Monitoring well BH7-25 was sampled and analyzed for all the parameters listing in Peel Region - Sanitary Sewer Discharge By-Law 53-2010. Non-filtered samples were submitted to Paracel Laboratories Inc., a Canadian laboratory accredited by the Canadian Association for Laboratory Accreditation.

The sample results show that the groundwater quality is generally compliant with the standards outlined in Peel Region - Sanitary Sewer Discharge By-Law 53-2010. The water quality results can be found in Appendix 3.

Groundwater Recharge and Discharge

In general, groundwater will follow the path of least resistance from areas of higher hydraulic head to areas of lower hydraulic head. While upward and downward hydraulic gradients may be indicative of areas of discharge and recharge respectively, other factors must be considered.

Based on Paterson's site specific field investigation , it is our interpretation that the majority of surface water will flow down gradient towards east as sheet drainage with minimal infiltration within the underlying silty clay layer. In select isolated areas where fill material has been encountered, surface water may infiltrate to the underlying glacial till layer where it will flow laterally down gradient as perched water (interflow). Therefore, the volume of recharge occurring within the site boundaries is expected to be minimal. However, it should be noted that the site is mapped as a significant groundwater recharge area (SGRA) by the MECP.

With regards to discharge zones, neither the topographical nor geological conditions are suitable for discharge to be occurring on a large scale at the subject site.

It should be noted that the subject site is not identified by the MECP as a drinking water protection zone.

Catchment Areas

As previously discussed, the subject site is located within the Humber River watershed specifically within Main Humber subwatershed. As shown on Drawing TH0106-4 – Groundwater Contour Map, the general groundwater flow direction across the subject site was measured to be in a northeasterly direction, toward the pond, wetland and watercourse along the eastern boundary of the property (greenbelt area). Based on the general site topography, surficial drainage across the majority of the site is also anticipated to be eastward towards the previously noted water features.

The subject site currently consists of a residential property surrounded by a mixture of developed, undeveloped and agricultural land with single general flow direction occurring within the subject site. Therefore, it is Paterson's opinion that the subject site is predominantly characterized by one catchment area.

4.4 Groundwater Extraction and Discharge

Based on available conceptual drawings for the site, three (3) potential sources of dewatering have been identified at the subject site. The sources consist of the excavation footprints related to the building foundations, servicing trenches and stormwater management pond (SWMP). Details regarding the excavation footprints and depths for each potential dewatering source were unavailable at the time of report preparation. Therefore, the building, servicing and SWMP excavations are assumed to encompass an area of approximately 750 m^2, 125 m^2 and 1,700 m^2, respectively, for a preliminary dewatering assessment.

Excavation sizes for the above noted water taking sources have been estimated based on previous experience at similar sites and the proposed SWMP footprint.

The infiltration rates provided for the following sources were calculated using the Dupuit Forchheimer method:

$Q = \pi k((h_0^2-h_p^2)/ln(R/r))$

- k = hydraulic conductivity (m/sec)
- h_0 = thickness of the aquifer (m)
- h_p = thickness of the aquifer from the base of the excavation to the base of the aquifer (m)
- R = effective drawdown radius for the excavation (m)
- r = equivalent radius of the excavation (m)

The groundwater infiltration calculations for the excavation footprints are provided in Appendix 3 of this report.

The stratigraphy within the anticipated saturated depth of the building excavations generally consists of a glacial till deposit. Specific design details were not available at the time of report preparation. For the purpose of this study, it has been conservatively assumed the buildings will consist of one basement level with a maximum excavation depth of 3 m bgs. Calculations are based on an excavation size of approximately 750 m^2 and water level depth at 2.5 m. Using a representative hydraulic conductivity of 1.45 x 10^{-7} m/sec (geometric mean of the calculated hydraulic conductivities), the steady state volume of groundwater anticipated to be less than 25,000 L/day, per excavation.

The stratigraphy within the anticipated saturated depth of the servicing excavations generally consists of a glacial till deposit. Specific design details were not available at the time of report preparation. For the purpose of this study, it has been assumed the servicing trenches will have a maximum excavation depth of 5 m bgs. Calculations are based on an excavation size of approximately 125 m^2 and water level depth at 2.5 m. Using representative hydraulic conductivity of 1.45 x 10^{-7} m/sec (geometric mean of the calculated hydraulic conductivities), the steady state volume of groundwater anticipated is less than 25,000 L/day, per excavation.

The stratigraphy within the anticipated saturated depth of the SWMP excavation generally consists of a glacial till deposit. Specific design details were not available at the time of report preparation. For the purpose of this study, it has been assumed the SWMP excavation will have a maximum excavation depth of 5 m bgs.

Calculations are based on an excavation size of approximately 1,700 m^2 and water level depth at 2.5 m. Using a representative hydraulic conductivity of 1.45 x 10^{-7} m/sec (geometric mean of the calculated hydraulic conductivities), the steady state volume of groundwater anticipated is approximately 40,000 to 50,000 L/day.

It is recommended that source specific dewatering calculations be completed once more specific development details are available.

5.0 SITE SPECIFIC WATER BUDGET ASSESSMENT

The site-specific water budget assessment (SSWB) was conducted to determine the hydrogeological function of the subject site, to identify infiltration potential and to identify opportunities for supplemental stormwater management measures. At the time of the field investigations the study area mainly consisted of agricultural land and some mature trees, grass and shrubs. The pre- and post-development terrain compositions are illustrated on Drawings TH0106-5 - Pre-Development Terrain Composition Plan and TH0106-6 - Post-Development Terrain Composition Plan, appended to this report. It should be noted the greenbelt area located within the eastern portion is subject site has been excluded from the assessment as is outside the development limits.

5.1 Calculations

Thornthwaite and Mather Water Balance Calculations

When falling precipitation intercepts the ground, three possible outcomes arise. The water can either evaporate/transpire back into the atmosphere (evapotranspiration), infiltrate into the surface soils (infiltration) or leave the area as runoff.

The method employed by Thornthwaite and Mather (1957) was used along with modelling software by Environment Canada's Engineering Climate Services Unit (EC-ECS) to determine the partitioning of water throughout various portions of the hydrologic cycle. Inputs into the modelling program included monthly temperature, precipitation, water holding capacities and site latitude. Using the long-term averages of these variables, it was possible to calculate annual potential and actual evapotranspiration, change in soil moisture storage and the water surplus.

The formula employed by Thornthwaite and Mather is as follows:

$$S = R + I = P - ET$$

Where: S = surplus (mm/year)
R = annual runoff (mm/year)
I = annual infiltration (mm/year)
P = annual precipitation (mm/year)
ET = annual evapotranspiration (mm/year).

Shallow unsaturated soils within the study area generally consisted of an agriculturally disturbed zone overlying a silty clay to clayey silt layer and/or glacial till comprised of a silty clay to clayey silt matrix.

Based on the location of the site, climatic data was obtained from the climate station located at the Toronto Pearson International Airport covering the period of January 1940 to December 2025. The information was provided by Environment Canada's Engineering Climate Services Unit.

Table 3, below, displays the soil types present within the study area and their associated water holding capacities (WHC) as well as the actual evapotranspiration (AET) and surplus data. For the purposes of this study, AET values were used as they account for accumulated soil moisture deficit. This deficit represents the volume of water retained within the available pore spaces of the soil and is subtracted from the potential evapotranspiration (PET) value to more accurately calculate the water surplus. For the purpose of this study, 70% of the low-density property footprints are considered impervious surfaces (100% of surplus will result in runoff), with the remaining 30% considered urban lawns. Similarly, 85% of the medium-density property footprints are treated as impervious surfaces (100% of surplus will result in runoff), and 15% as urban lawns.

Table 3 - Site Specific Water Surplus Information

Land Use Unit	Water Holding Capacity (mm)	Actual Evapotranspiration (mm/year)	Surplus Water (mm/year)
Impervious Surfaces	N/A	126*	660
Agricultural Land (Silt Loam)	200	591	197
Pasture and Shrubs (Silt Loam)	250	605	183
Urban Lawn/Shallow Rooted Crops (Silt Loam)	125	550	238
Urban Lawn (Fine Sand Loam)	75	506	279
Mature Forests (Silt Loam)	400	626	162

Table reproduced using WHC values from MOE (2003) - Stormwater Management Planning and Design Manual and modelling data from Environment Canada's Engineering Climate Services Unit.
*Values based on evaporation information for urban areas (16% of precipitation) included in the Eastern Ontario Water Resources Management Study prepared by CH2M HILL Canada Limited (March 30, 2001).

Infiltration Factors

In order to break down the surplus water values for the various materials into infiltration and runoff, various factors must be considered. The MOE Stormwater Management Planning and Design Manual (2003) lists three main factors that contribute to surface water infiltration rates.

The first factor is topography, which is broken down further into three sections: flat and average slope, rolling land and hilly land. Flat and average slope provides the greatest potential for infiltration and has the largest infiltration factor applied to it (0.3), while the other two have progressively lower infiltration factors (rolling land is 0.2 and hilly land is 0.1).

The second factor is soil, which is also broken down further into three sections: tight impervious clay, medium combinations of clay and loam and open sandy loam. Open sandy loam provides the greatest potential for infiltration (infiltration factor of 0.4) while the other two have progressively lower potential for infiltration to occur (infiltration factor for medium combinations of clay and loam is 0.2 and for tight impervious clay is 0.1).

The final factor the MOE manual uses to partition infiltration from runoff is land cover. It is broken down into two sections: open fields/cultivated lands and woodlands. Woodlands have greater infiltration potential and an infiltration factor of 0.2. Open fields and cultivated lands have lower potential and with an infiltration factor of 0.1. A summary of the MOE manual's descriptors and their associated infiltration factors is shown below in Table 4.

Table 4 - MOE (2003) Infiltration Factors

Description of Area/Development Site	Value of Infiltration Factor
Topography	
Flat and average slope (<0.6 m/km)	0.30
Rolling land (slope of 2.8-3.8 m/km)	0.20
Hilly land (slope of 28-47 m/km)	0.10
Soil	
Tight impervious clay	0.10
Medium combinations of clay and loam	0.20
Open sandy loam	0.40
Cover	
Open fields/cultivated lands	0.10
Woodlands	0.20

Table reproduced from MOE (2003) - Stormwater Management Planning and Design Manual.

The topography of the study area is classified as Rolling land (slope of 2.8-3.8 m/km throughout the subject site) Therefore, a pre-development topography infiltration factor of 0.2 was given for the materials analysed on this property. Further, the topography of the subject site under post-development conditions will also consist of a rolling land well graded and was therefore assigned a post-development topography infiltration factor of 0.2. An infiltration factor of 0 was assigned to the impervious surfaces and SWMP due to its negligible infiltration capacity.

As previously discussed, soils within the study area generally consisted of an agriculturally disturbed zone overlying a silty clay to clayey silt layer and/or glacial till comprised of a silty clay to clayey silt matrix with low infiltration capacity. Therefore, a pre-development soil infiltration factor of 0.15 was given for the materials analysed on this property in exemption to urban lawn where an infiltration factor of 0.3 was given. Under post-development conditions, the majority of the site will consist of either impervious surfaces or urban lawn and parks. Therefore, an infiltration factor of 0.15 was assigned to urban lawn and parks and 0 for impervious surfaces and SWMP.

At the time of the field investigations, the subject site generally consisted of agricultural land with grass and shrub around the existing building and farm outbuilding and mature tree area at the southeast part of the site. A pre-development vegetation infiltration factor of 0.15 was used for agricultural land and shrub area, 0.2 for mature tree area, and 0.1 for urban lawns adjacent to the existing residential building. Post-development, it is expected the majority of the trees remaining on site will be removed to accommodate buildings, parking areas and roadways. As such, a post-development vegetation infiltration factor of 0.1 was assigned to the site, except for impervious surfaces and SWMP, which were given an infiltration factor of 0 due to its negligible potential to benefit from vegetation cover.

5.2 Pre and Post-Development Water Budget

The pre-development water budget analysis conducted for the study area determined that an estimated 13,415,643 L/year of surplus water currently infiltrates the surface soils. The remaining estimated 16,397,402 L/year of surplus leaves the site as runoff.

The post-development water budget analysis determined that an estimated 2,935,718 L/year of surplus water will infiltrate the surface soils and approximately 77,872,804 L/year will leave the site as runoff. These values equate to an approximate decrease in infiltration of 78% and an increase in runoff of 375%.

The main variable that changed from pre-development conditions to post-development conditions was the addition of approximately 11 hectares of impervious surfaces. This results in reducing the area of pervious materials throughout the subject site, therefore, reducing the overall infiltration potential of the subject site. The remaining areas that are not being converted to impervious surfaces will become landscaped surfaces characterized by urban lawn (well graded silt loam) material. Furthermore, the SWMP area is designed to manage stormwater runoff and does not contribute to the site's overall infiltration or runoff potential. As previously noted, the greenbelt area was excluded from the post-development water budget analysis as it is located outside the limits of development.

It is important to note that the post-development water budget analysis for the subject site does not consider any potential infiltration of the impervious surfaces (100% runoff was taken as a conservative approach). In reality, some portion of surface water that lands on impervious surfaces infiltrates (asphalt is not 100% impervious) or is diverted to grassed areas where additional infiltration may occur. As such, the post-development runoff volumes should be considered a conservative estimate and not expected to definitively represent future conditions.

Details of the pre- and post-development water budget analyses are presented in Tables 5 and 6, included in Appendix 2 of this report.

6.0 GROUNDWATER IMPACT ASSESSMENT

6.1 Impact of Proposed Development on Surrounding Infrastructure

As previously discussed, soils within the subject site generally consisted of an agriculturally disturbed zone overlying a silty clay to clayey silt layer and/or glacial till comprised of a silty clay to clayey silt matrix. The expected groundwater infiltration will be encountered within the glacial till deposit.

The steady-state radius of influence calculations completed were based upon the Sichardt equation as shown below. The assumed setting for the analytical solution was one in which open cut trenches were used to install the services at the subject site, creating an unconfined condition which would allow use of the equation to determine the radius of influence.

$$R = 3000 * \Delta h(K^{0.5})$$

Where: R = radius of influence (m)
Δh = expected groundwater drawdown (m)
K = hydraulic conductivity (m/sec).

For the purposes of completing the calculations, the following values were used in the analysis for the glacial till:

- Δh = 0.5 to 2.5 m
- K = 1.45 x 10^{-7} m/sec (geometric mean hydraulic conductivity), based on site specific hydraulic conductivity values of the glacial till.

Using the above equation and assumptions, a radius of influence of 1 to 3 m will develop as a steady state condition within the glacial till, extending from the edge of the excavation, in the area of the subject site, depending on the groundwater levels and hydraulic properties of the specific soils encountered.

The surrounding area consists of mostly agricultural land with some commercial and an institutional development. No buildings are located within the theoretical radius of influence. Furthermore, any nearby buildings are generally expected to be founded on the glacial till deposit. The majority of the groundwater infiltration is expected to occur within the glacial till with minimal compressibility. Water takings are also expected to be short term in duration, given the nature of the development. As such, adverse effects to surrounding infrastructure related to dewatering activities at the subject site are expected to be negligible.

6.2 Impact of Proposed Development on Existing Well Users

A search of the Ontario Water Well Records online mapping database indicates there are a number of wells within 500 m of the site as depicted on Drawing TH0106-7 - MECP Water Well Location Plan, appended to this report. However, all domestic wells in proximity to the subject site are located well outside the theoretical radius of influence and extend well below the anticipated excavation depths. Given the vertical and horizontal separation of the wells from the proposed development, dewatering activities at the subject site are not expected to cause any interference to the water supply of surrounding properties or other negative impacts.

Existing Water Quality and Construction Dewatering Plan

Monitoring well BH7-25 was sampled and analyzed for all the parameters listed in Peel Region - Sanitary Sewer Discharge By-Law 53-2010. The sample results show that the groundwater quality is generally compliant with the standards outlined in Peel Region - Sanitary Sewer Discharge By-Law 53-2010. The water quality results can be found in Appendix 3. Further, as the potential to interfere with the water quality/quantity of existing well users in the area is negligible, a water well monitoring program is not recommended for the aforementioned water takings. If wells are found to remain in existence on the subject site, they should be decommissioned in accordance with Ontario Regulation 903.

In the unlikely event that construction activities are shown to cause negative impacts to the water supplies of existing well users, the contractor shall take action to make available a supply of water equivalent in quality and quantity of their typical takings or shall compensate those affected for reasonable costs for doing so, or shall reduce water taking amounts to alleviate the negative impacts. The contractor shall provide temporary water supplies, to those affected, to meet their typical takings or compensate such persons for reasonable costs associated to do so until permanent restoration of the affected water supply or an equivalent source.

Prior to and during site development, it is recommended that Best Management Practices (BMPs) with respect to fuels and chemical handling, spill prevention, and erosion and sediment control be followed. This will minimize the potential for the introduction of contaminants to the soil, surface water, or groundwater at the subject site.

Also, BMPs related to water discharge quality should be implemented which may include but not limited to:

- ❒ Controlling sediment loading before pumping by:
  - placing the pump in a bucket at the bottom of a sump pit to reduce sediment build up around the pump.
  - adding a float to the suction hose/pipe and keep it off the bottom of the sump pit to avoid pumping sediment that has settled at the bottom of the pit.
- ❒ Filtering discharge through a sediment bag or use filter cloth around the suction and/or discharge hose/pipe.

As the discharge point for the pumped water is anticipated to be directed to overland drainage, it is expected that a multi-barrier approach (such as hay bales, geosocks, silt fence, etc.) to a non-frozen, well vegetated area will be utilized in order to promote re-infiltration prior to reaching a water body. If overland drainage occurs within 30 m of a water body, the turbidity of the water shall not exceed 8 NTU above background levels. The contractor will be required to maintain appropriate BMPs with respect to sediment and erosion control to ensure negative effects to the surrounding environment are minimized.

If water is to be discharged to municipal sewers, discharge activities must be compliant with the Regional Municipality of Peel By-law 53-2010. It is anticipated that discharge rates will generally be < 5 L/s and should not exceed the local municipal sewer system's capacity. Final discharge rates will be determined closer to the time of construction.

Post-Development Water Quality

With respect to stormwater runoff quality, it is recommended that BMPs with respect to operational standards be maintained for any stormwater management facilities constructed for the proposed development. It is also recommended that adherence to the local road salting management regulations is enforced to ensure that chloride levels in stormwater runoff are as low as possible.

6.3 Impact of Proposed Development on the Environment

A search of the MECP Environmental Site Registry for Records Site Condition (RSCs) was conducted as part of the assessment of the site, neighbouring properties and the general area. No RSCs were identified within the 500 m of the subject site.

A Phase II Environmental Site Assessment (ESA) was completed by Paterson for the subject site. All soil samples were found to be in compliance with MECP Table 1 Residential standards.

6.4 Adjacent PTTW/EASRs/ECAs

A search of the MECP Permit to Take Water (PTTW) and Environmental Activity and Sector Registry (EASR) database provided no active PTTW or EASR within a 500 m radius of the subject site and with respect to Environmental Compliance Approvals (ECAs), no ECAs were found neighboring the subject site.

7.0 ASSESSMENT AND RECOMMENDTIONS

7.1 Sources of Contamination

Based on the soil and groundwater samples collected at the subject site as part of the Phase II ESA investigation, all soil and groundwater samples were found to be in compliance with MECP Table 1 Residential standards.

Prior to and during site development, it is recommended that construction best management practices with respect to fuels and chemical handling, spill prevention, and erosion and sediment control be followed. This will minimize the potential for the introduction of contaminants to the soil, surface water, or groundwater at the subject site.

It is anticipated that the material on site will be disposed of as per Ontario Regulation 406/19 – On-site and Excess Soil Management.

With respect to stormwater runoff quality, it is recommended that best management practices with respect to operational standards be maintained for any stormwater management facilities constructed for the proposed development.

7.2 Surface Water Features

There are several surface water features located within the subject site and within 500 m of the subject site. These include unnamed scratch ditches, drainage ditches, pond and unnamed tributaries of Humber River. However, as previously discussed, the stormwater management strategy will employ measures to increase hydration to adjacent surface water features. Therefore, by implementing the proposed stormwater management strategy no significant impacts are expected from the proposed development on the overall hydrologic function of the area or sensitive features.

With respect to water discharge, water that is pumped from on site excavations must be managed in an appropriate manner. The contractor will be required to implement a water management program to dispose of the pumped water. If the discharge point for the pumped water is directed to overland drainage, it is expected that a multi-barrier approach (such as hay bales, geosocks, silt fence, etc.) to a non-frozen, well vegetated area will be utilized in order to promote re-infiltration prior to reaching a watercourse. Furthermore, if the discharged water is to be directed to overland drainage within 30 m of a water body/watercourse, the turbidity of the water shall not exceed 8 NTU above background levels of the

nearest water body. The contractor will be required to maintain appropriate BMPs with respect to sediment and erosion control to ensure negative effects to the surrounding environment are minimized.

7.3 Water Taking Permitting Requirements

If water taking volumes are greater than 50,000 L/day, a MECP water taking Environmental Activity Sector Registry (EASR) or Permit to Take Water (PTTW) will be required. Depending on the nature of the proposed water takings, an additional hydrogeological investigation may be required.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

7.4 Infiltration Potential

As previously discussed, surficial soils within the study area generally consisted of an agriculturally disturbed zone overlying a silty clay to clayey silt layer and/or glacial till comprised of a silty clay to clayey silt matrix. Infiltration rates for the overburden materials are anticipated to be low.

As noted above, the results of the water budget analyses completed at the subject site indicated that 13,415,643 L/year of infiltration and 16,397,402 L/year of surface runoff are occurring under pre-development conditions. Under post development conditions, it is expected that there will be a 78% infiltration deficit and a 372% increase in runoff. Therefore, it will be necessary to incorporate various stormwater management measures into the design of the development. It should be noted that Paterson's water budget assessment is based on mean water budget values for the soil types at the subject site that were calculated by modeling conducted by EC-ECS. The EC-ECS model is calibrated to historical climate data and does not account for climate change predictions. Therefore, based on the National Capital Commission and City of Toronto and surrounding area climate change predictions, the stormwater management design team could consider potential seasonal changes (longer spring and shorter winter) and increases in temperature and precipitation when developing their stormwater management strategy.

The stormwater management strategy should target methods best suited to mitigate the impacts that may arise due to the post-development decrease in infiltration and increase in runoff at the subject site. Based on the site constraints observed at the subject site (i.e. low permeable soil conditions), infiltration-type Low Impact Development (LID) measures are not recommended at the time of

report preparation. Therefore, it is understood that the stormwater management strategy will include conventional stormwater management measures (i.e., end of pipe quality and quantity control) as well as the implementation of Best Management Practices to mitigate impacts related to the post-development water balance. Despite the post-development infiltration deficit, the majority of runoff will continue to be directed to areas within the same subwatershed, where infiltration will occur offsite, thereby maintaining the natural function of the hydrologic and hydrogeologic systems.

8.0 CLOSURE

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only, and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Tiffany Rox North Hill Ltd. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.



Nicholas Zulinski, M.Sc., P.Geo., géo.





Balaji T. Nirmala, M.Eng., P.Eng.

9.0 REFERENCES

Chapman, L.J., and Putnam, D. F. “The Physiography of Southern Ontario, Third Edition”. Ontario Ministry of Natural Resources, 1984.

Freeze, R.A., and Cherry, J.A. “Groundwater”. Prentice-Hall, Inc., 1979.

Ontario Geologic Survey Mapping - https://www.geologyontario.mndm.gov.on.ca/ogsearth.html

Ontario Regulation 387/04.

APPENDIX 1

DRAWING TH0106-1 - SITE PLAN

DRAWING TH0106-2 - SURFICIAL GEOLOGY PLAN

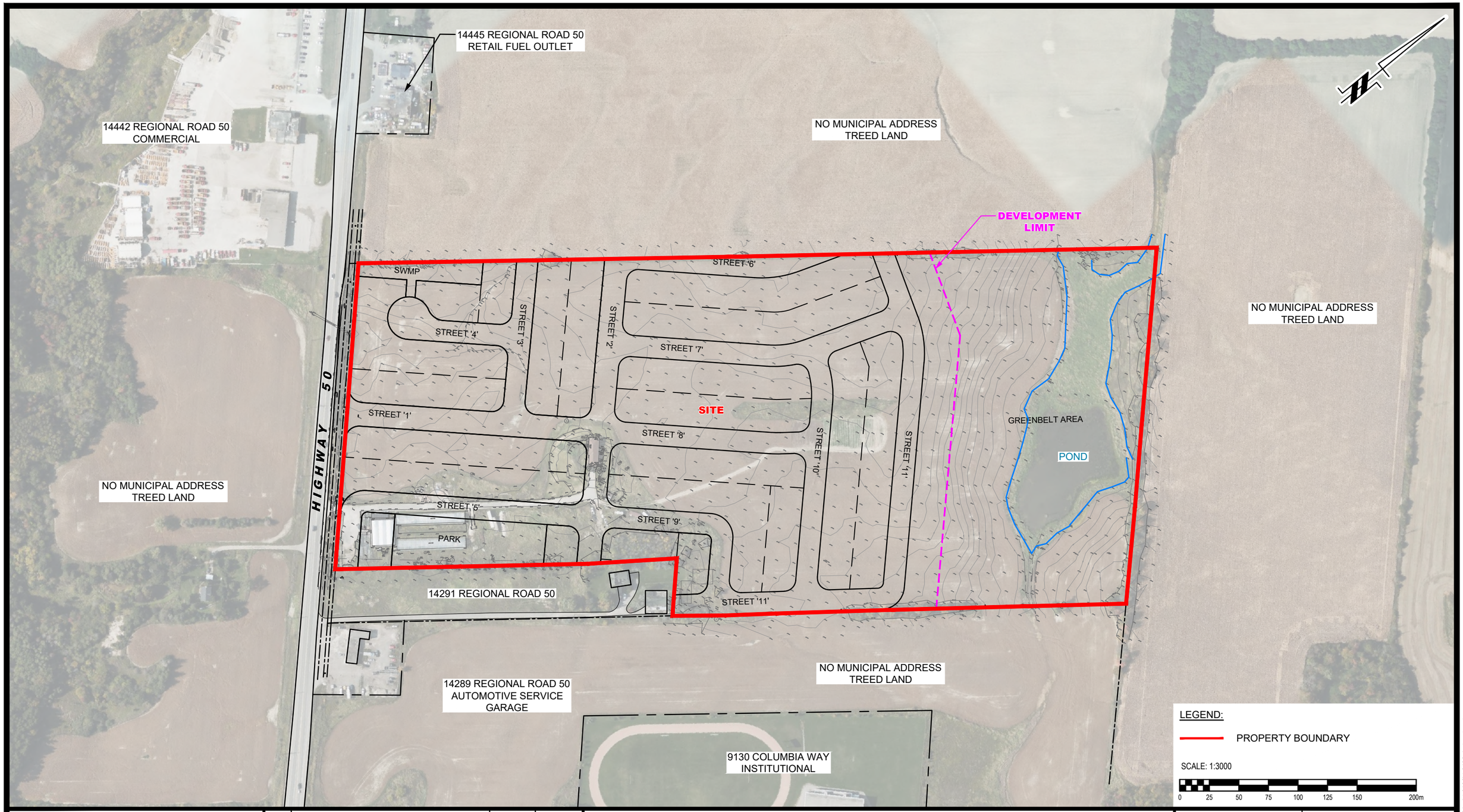
DRAWING TH0106-3 - BEDROCK GEOLOGY PLAN

DRAWING TH0106-4 - GROUNDWATER CONTOUR MAP

DRAWING TH0106-5 - PRE-DEVELOPMENT TERRAIN COMPOSITION PLAN

DRAWING TH0106-6 - POST-DEVELOPMENT TERRAIN COMPOSITION PLAN

DRAWING TH0106-7 - MECP WATER WELL LOCATION PLAN



PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

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HYDROGEOLOGICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
14337 HIGHWAY 50

BOLTON, ONTARIO

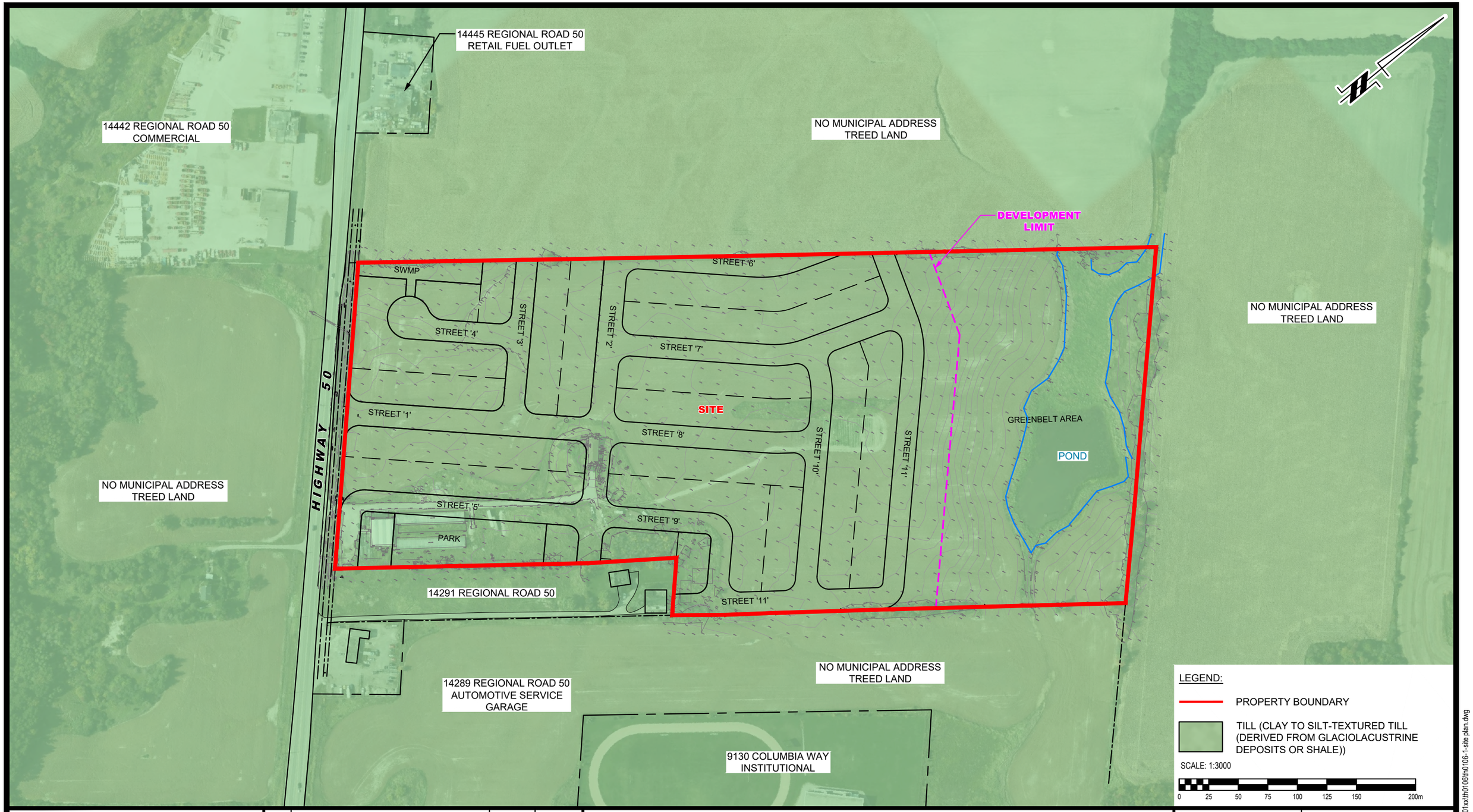
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Scale: 1:3000	Date: 01/2026
Drawn by: GK	Report No.: TH0106-1
Checked by: BN	Dwg. No.: TH0106-1
Approved by: ML	Revision No.:

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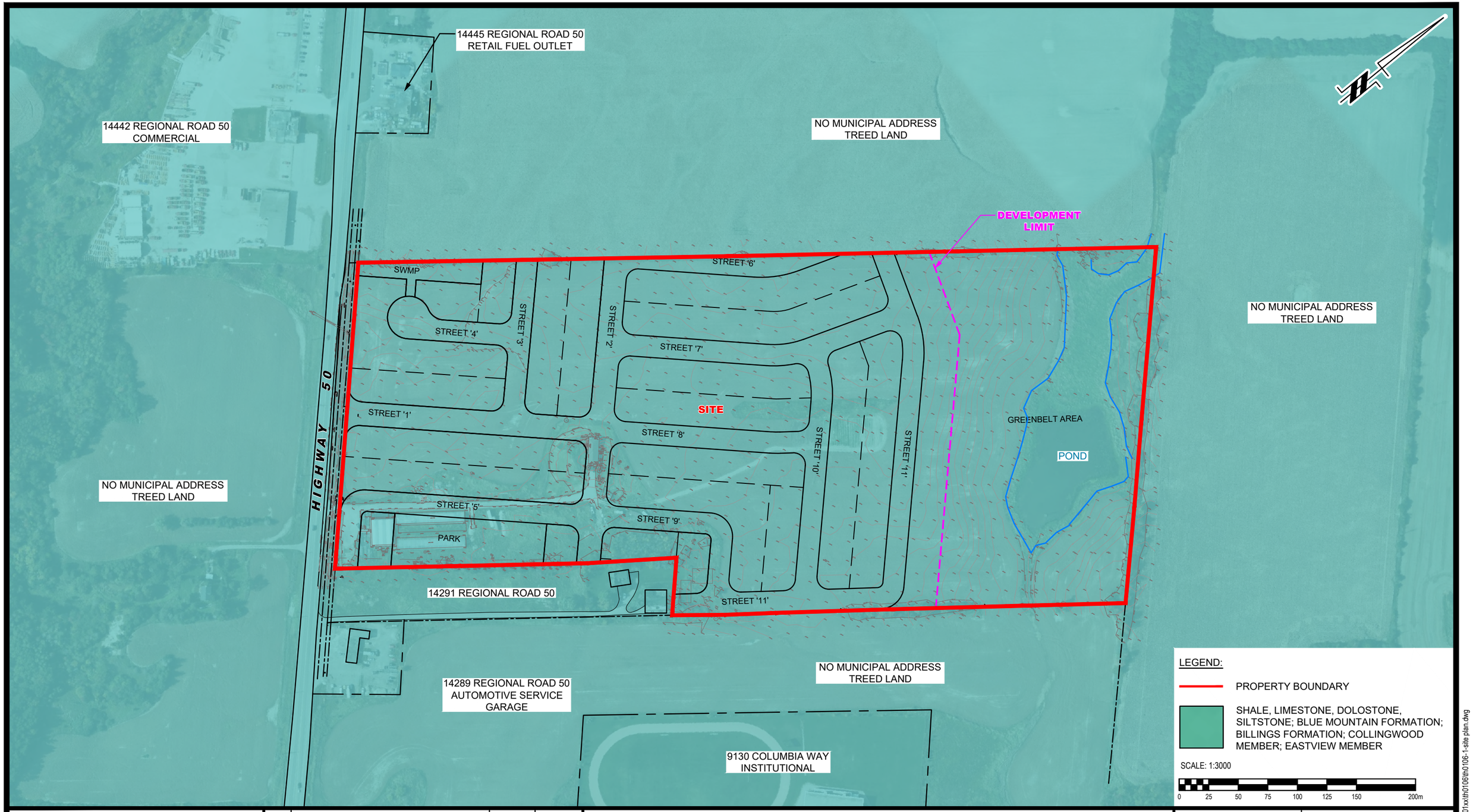
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Drawn by: GK	**Report No.:** TH0106-1
Checked by: BN	**Dwg. No.:** TH0106-2
Approved by: ML	**Revision No.:**

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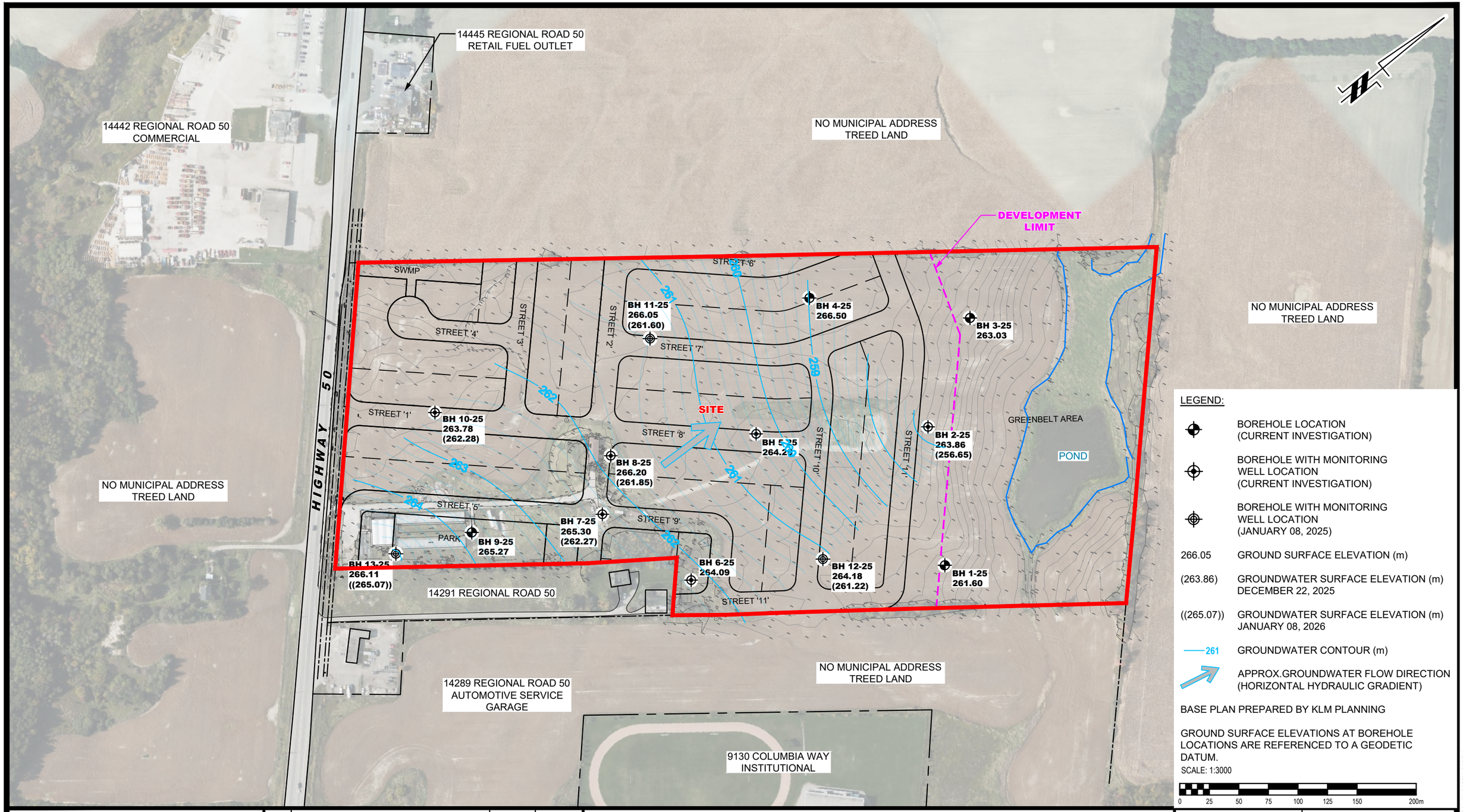
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BEDROCK GEOLOGY PLAN

Scale: 1:3000	Date: 01/2026
Drawn by: GK	Report No.: TH0106-1
Checked by: BN	Dwg. No.: TH0106-3
Approved by: ML	Revision No.:

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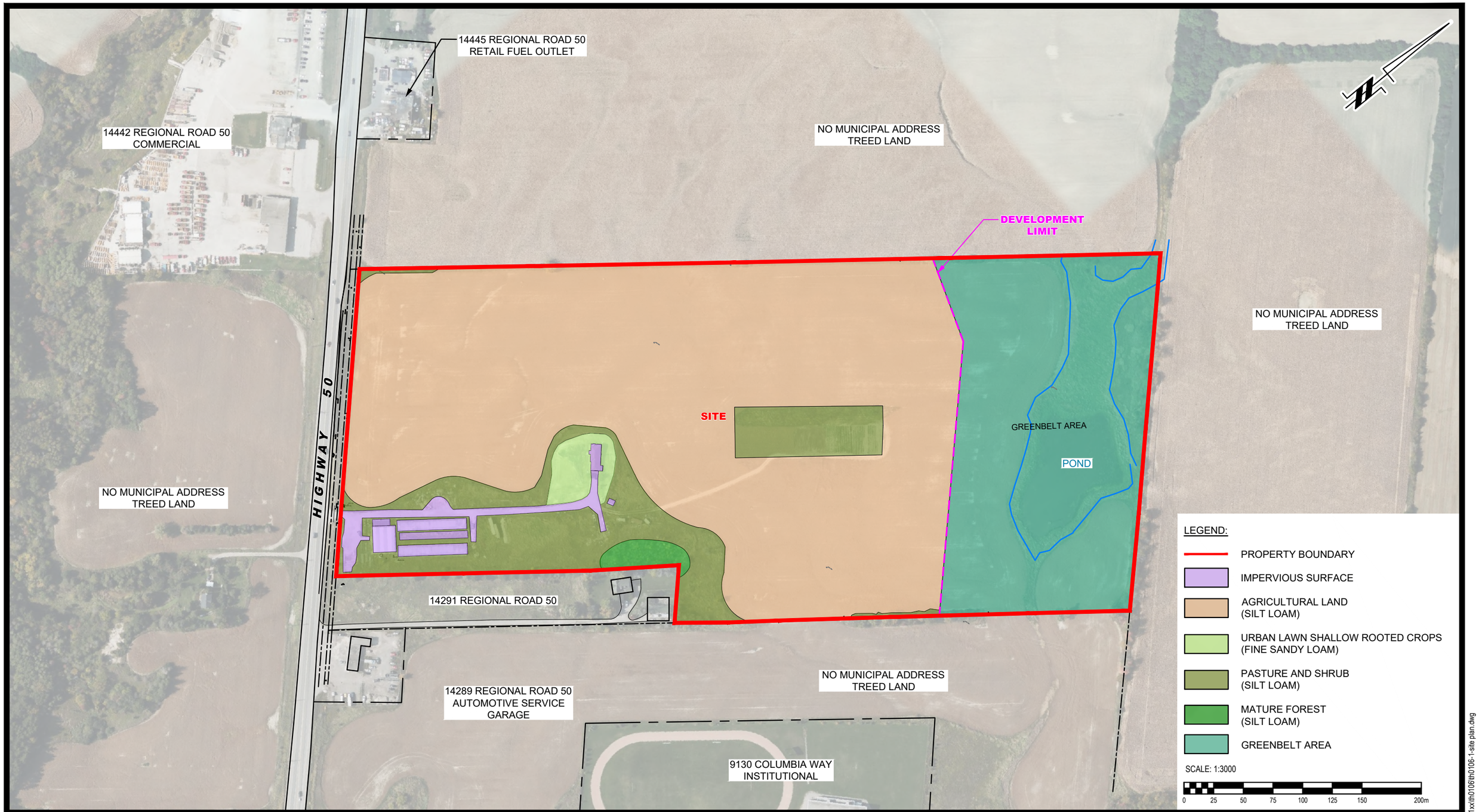
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GROUNDWATER CONTOUR PLAN

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Drawn by: GK	Report No.: TH0106-1
Checked by: BN	Dwg. No.: TH0106-4
Approved by: ML	Revision No.:

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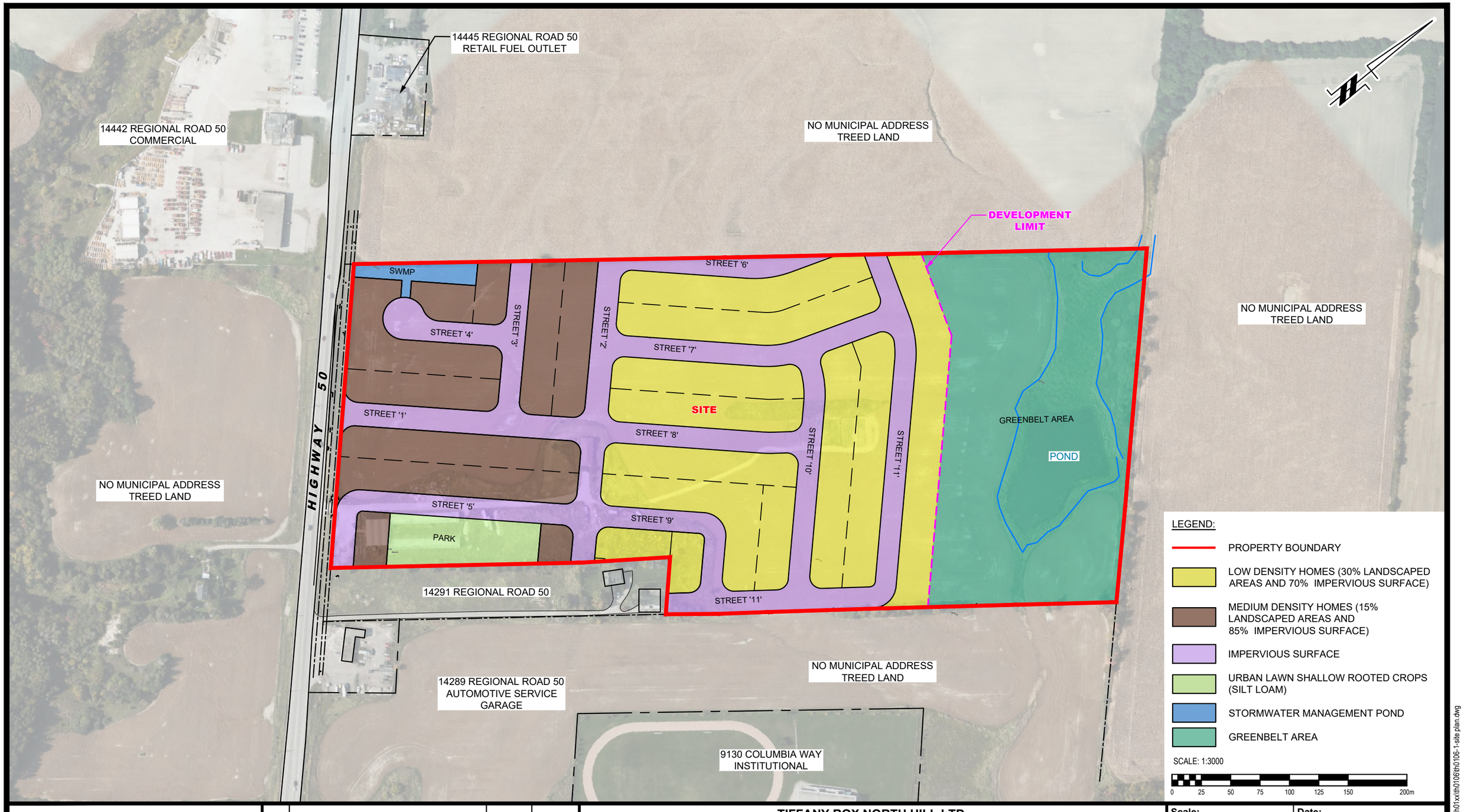
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PROPOSED RESIDENTIAL DEVELOPMENT
14337 HIGHWAY 50
BOLTON, ONTARIO

Title:
PRE-DEVELOPMENT TERRAIN COMPOSITION PLAN

Scale: 1:3000	Date: 02/2026
Drawn by: GK	Report No.: TH0106-1
Checked by: BN	Dwg. No.: TH0106-5
Approved by: ML	Revision No.:

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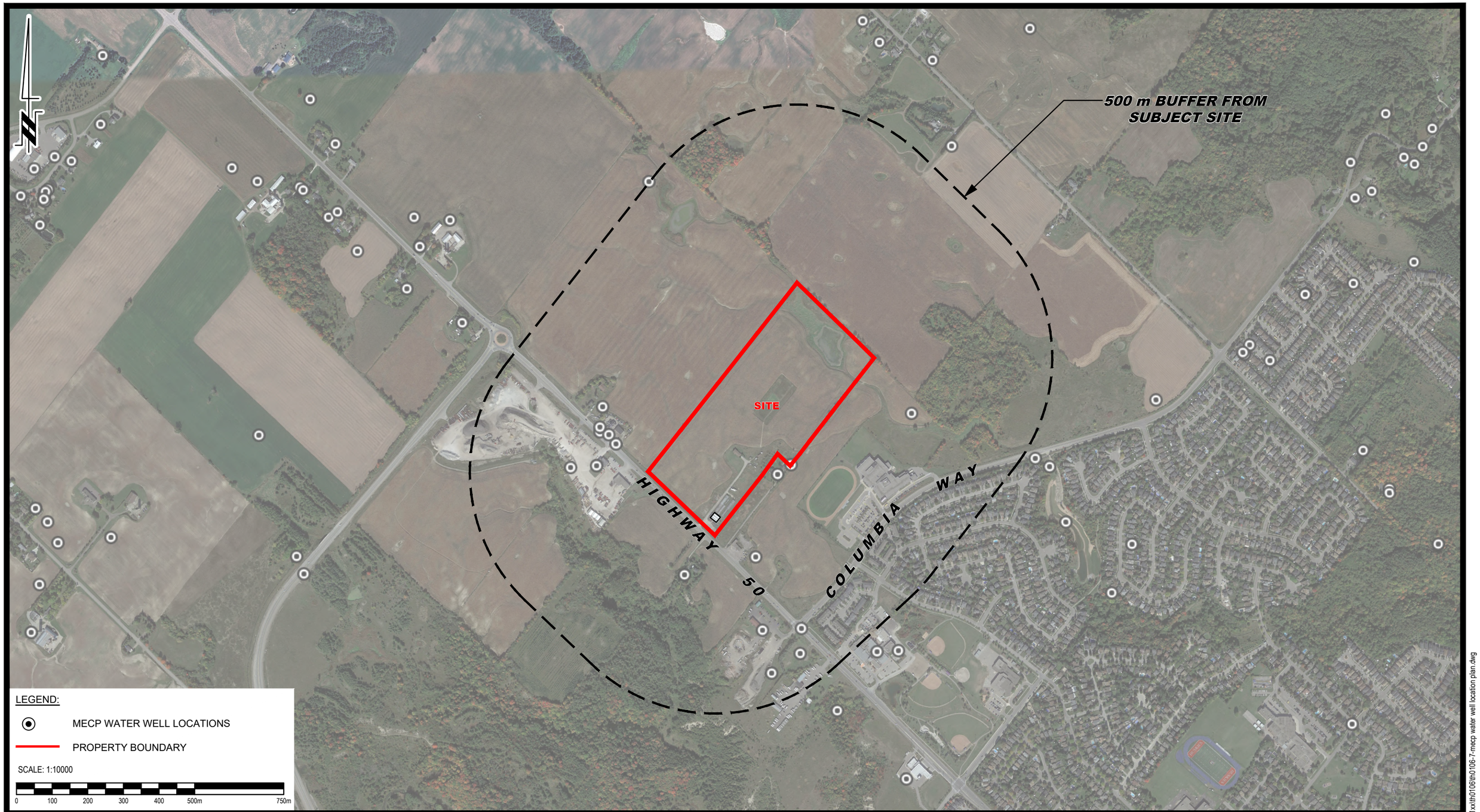
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PROPOSED RESIDENTIAL DEVELOPMENT
14337 HIGHWAY 50
BOLTON, ONTARIO

Title:
POST-DEVELOPMENT TERRAIN COMPOSITION PLAN

Scale: 1:3000	Date: 02/2026
Drawn by: GK	Report No.: TH0106-1
Checked by: BN	Dwg. No.: TH0106-6
Approved by: ML	Revision No.:

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BOLTON, ONTARIO

Title:

MECP WATER WELL LOCATION PLAN

Scale: 1:10000	Date: 01/2026
Drawn by: GK	Report No.: TH0106-1
Checked by: BN	Dwg. No.: TH0106-7
Approved by: ML	Revision No.:

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APPENDIX 2

SOIL PROFILE AND TEST DATA SHEETS

DRAWING TG0257-1 - TEST HOLE LOCATION PLAN

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 600124.69 **NORTHING:** 4861227.07 **ELEVATION:** 261.60

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 3, 2025

FILE NO. : TG0257

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
AGRICULTURALLY DISTURBED SOIL: Brown silty clay to clayey silt with sand organics and rootlets		0	AU 1			18			261
0.69m [260.91m]									
Compact, brown **SILTY SAND** to **SANDY SILT**		1	SS 2	12	3-7-4-5 11	16			260
2.13m [259.47m]		2	SS 3	33	3-3-7-11 10				
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel		3	SS 4	100	4-8-7-6 15	19			259
			SS 5	92	5-8-7-8 15	17			258
		4	SS 6	100	3-4-7-9 11	21			257
		5	SS 7	100	3-5-6-7 11	20			256
		6	SS 8	100	3-6-7-5 13	21			255
6.71m [254.89m]			SS 9	100	4-5-8-7 13	20			254
End of Borehole		7							253
(BH Dry on - December 22, 2025)		8							252
		9							251
		10							
		11							

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 600025.21 **NORTHING:** 4861290.43 **ELEVATION:** 263.86

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 3, 2025

FILE NO. : TG0257

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80	REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80	PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE											
AGRICULTURALLY DISTURBED SOIL: Brown silty clay to clayey silt with sand, organics and rootlets			AU 1			24					
0.69m [263.17m]											
GLACIAL TILL: Very stiff to hard, brown silty clay to clayey silt with some sand		1	SS 2	75	5-7-9-8 16	16					263
		2	SS 3	58	4-7-7-8 14	16 22					262
			SS 4	100	3-6-8-9 14	23					
		3	SS 5	75	4-6-9-12 15	12					261
- Silty sand seam by 3.9 m depth		4	SS 6	83	4-7-8-6 15	13 17 17					260
4.27m [259.59m]											
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand		5	SS 7	75	3-4-7-8 11	20					259
		6	SS 8	12	4-10-13-16 23	17					258
			SS 9	100	4-6-6-10 12	10					
		7	SS 10	83	4-7-9-11 16	17					257
		8	SS 11	100	3-6-7-12 13	22					256
		9	SS 12	100	3-7-8-12 15	20					255
			SS 13	100	6-7-13-16 20	14					
9.75m [254.11m]											
End of Borehole		10									254
(GWL at 7.21 m depth - December 22, 2025)		11									253

129

7.21 m 2025-12-22

7.55m

9.07m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599976.62 **NORTHING:** 4861376.31 **ELEVATION:** 263.03

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 3, 2025

FILE NO. : TG0257

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
AGRICULTURALLY DISTURBED SOIL: Brown silty clay to clayey silt with sand, organics and rootlets		0	AU 1			23			263
0.61m [262.42m]									
GLACIAL TILL: Very stiff to hard, brown silty clay some sand, trace gravel		1	SS 2	42	2-5-7-7 12	23			262
		2	SS 3	67	3-10-9-11 19	18			261
			SS 4	100	4-9-13-18 22	19			
		3	SS 5	100	4-12-13-18 25	20			260
		4	SS 6	100	6-13-16-21 29	19			259
		5	SS 7	100	5-9-12-19 21	21			258
5.46m [257.57m]						19			
GLACIAL TILL: Very stiff to hard, grey silty clay, trace gravel		6	SS 8	100	4-8-10-13 18	18			257
6.71m [256.32m]			SS 9	100	3-9-7-10 16	19			
End of Borehole		7							256
(BH Dry on - December 22, 2025)		8							255
		9							254
		10							253
		11							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599877.34 **NORTHING:** 4861282.17 **ELEVATION:** 266.50

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 5, 2025

FILE NO. : TG0257

HOLE NO. : BH 4-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
AGRICULTURALLY DISTURBED SOIL: Brown silty clay to clayey silt with sand, organics and rootlets			AU 1			22			266
0.69m [265.81m]									
Compact, brown **SILTY SAND**		1	SS 2	83	9-10-10-13 20	11			
1.12m [265.38m]						18			265
GLACIAL TILL: Very stiff to hard, brown silty clay, trace gravel		2	SS 3	100	9-11-12-14 23	15			
			SS 4	100	6-11-15-22 26	18			264
2.97m [263.53m]		3							
Dense, brown **SILTY SAND** to **SANDY SILT**, some clay			SS 5	75	10-19-14-25 33	21			263
3.73m [262.77m]		4							
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel			SS 6	83	5-12-10-10 22	16			262
		5	SS 7	100	6-10-13-14 23	19			
			SS 8	75	7-9-9-7 18	16			261
6.71m [259.79m]		6							
End of Borehole			SS 9	83	9-12-19-20 31	18			260
(BH Dry on - December 22, 2025)		7							259
		8							258
		9							257
		10							256
		11							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599937.51 **NORTHING:** 4861174.35 **ELEVATION:** 264.25

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 5, 2025

FILE NO. : TG0257

HOLE NO. : BH 5-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
AGRICULTURALLY DISTURBED SOIL: Brown silty sand to sandy silt with clay, organics and rootlets			AU 1			18			264
0.69m [263.56m]									
GLACIAL TILL: Very stiff to hard, brown silty clay, some sand, trace gravel		1	SS 2	67	5-7-8-9 15	17			263
1.83m [262.42m]									
Compact, brown **SILTY SAND**		2	SS 3	83	4-6-8-8 14				262
2.59m [261.66m]						20			
Hard, brown **SILTY CLAY** 2.74m [261.51m]			SS 4	100	6-12-13-16 25	18 20			
Compact, brown **SILTY SAND**, some clay		3	SS 5	75	8-13-15-12 28	18 18			261
3.51m [260.74m]									
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel		4	SS 6	100	6-8-11-11 19	18		4.27m	260
		5	SS 7	83	4-7-9-14 16	19			259
		6	SS 8	83	5-5-7-8 12	20		5.72m	258
6.71m [257.54m]			SS 9	92	4-5-8-8 13	22			
End of Borehole		7							257
(BH Dry on - December 22, 2025)		8							256
		9							255
		10							254
		11							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599998.26 **NORTHING:** 4861053.38 **ELEVATION:** 264.09

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 5, 2025

FILE NO. : TG0257

HOLE NO. : BH 6-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80	REMOULDED SHEAR STRENGTH (kPa) △ UNDRAINED SHEAR STRENGTH (kPa) ▲ 20 40 60 80	PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE											264
AGRICULTURALLY DISTURBED SOIL: Brown silty clay to clayey silt with sand, organics and rootlets			AU 1			17					
0.61m [263.48m]											
GLACIAL TILL: Stiff to hard, brown silty clay with some sand and garvel		1	SS 2	100	7-10-16-23 26	16					263
		2	SS 3	100	6-10-16-23 26	17					262
			SS 4	100	4-11-17-22 28	19					
3.00m [261.09m]		3									261
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel			SS 5	100	3-6-8-10 14	17					
		4	SS 6	92	4-5-7-9 12	19					260
		5	SS 7	100	3-6-9-13 15	19					259
			SS 8	100	5-7-9-9 16	19					
6.71m [257.38m]		6	SS 9	100	4-6-10-11 16						258
End of Borehole		7									257
(BH Dry on - December 22, 2025)		8									256
		9									255
		10									254
		11									

3.58m

6.63m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599907.52 **NORTHING:** 4861030.66 **ELEVATION:** 265.30

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 8, 2025

FILE NO. : TG0257

HOLE NO. : BH 7-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE							265
FILL: Stiff, brown silty sand with clay and gravel			AU 1			17	
0.61m [264.69m]							
GLACIAL TILL: Stiff to hard, brown silty clay, some sand and gravel		1	SS 2	50	3-2-3-4 5		264
		2	SS 3	50	3-3-6-9 9	24	263
			SS 4	100	5-7-10-12 17	17	
		3	SS 5	100	5-11-13-14 24	18	262
		4	SS 6	67	4-8-8-14 16	17	261
		5	SS 7	42	5-10-17-15 27	19	
5.18m [260.12m]							260
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt, some gravel		6	SS 8	100	3-5-7-10 12	25	
6.71m [258.59m]			SS 9	83	4-6-8-8 14	20	259
End of Borehole		7					258
(GWL at 3.03 m depth - December 22, 2025)		8					257
		9					256
		10					255
		11					

PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) — 20 40 60 80

REMOULDED SHEAR STRENGTH (kPa)

UNDRAINED SHEAR STRENGTH (kPa) — 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) — 20 40 60 80

MONITORING WELL CONSTRUCTION

3.03 m 2025-12-22

6.03m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599873.80 **NORTHING:** 4861067.65 **ELEVATION:** 266.20

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 8, 2025

FILE NO. : TG0257

HOLE NO. : BH 8-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80	REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80	PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE											266
TOPSOIL 0.05m [266.15m]			AU 1			9					
FILL: Brown silty sand with clay and organics		1	SS 2	42	3-5-5-4 10	19					265
		2	SS 3	75	8-4-7-6 11	16					264
2.44m [263.76m]			SS 4	100	6-9-11-14 20	19 24					
GLACIAL TILL: Very stiff to hard, brown silty clay to clayey silt with some sand and gravel		3	SS 5	83	3-7-10-13 17	18				3.00m	263
		4	SS 6	100	4-16-15-18 31	17				4.35 m 2025-12-22	262
4.72m [261.48m]			SS 7	100	5-17-27-23 44	20					
Dense, brown **SILTY SAND** with clay		5									261
5.18m [261.02m]			SS 8	100	3-6-12-15 18	13					
GLACIAL TILL: Very stiff, grey silty clay to clayey silt with some sand and gravel		6								6.05m	260
6.71m [259.49m]			SS 9	75	2-6-8-9 14	19					
End of Borehole		7									259
(GWL at 4.35 m depth - December 22, 2025)		8									258
		9									257
		10									256
		11									

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599849.33 **NORTHING:** 4860935.63 **ELEVATION:** 265.27

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 8, 2025

FILE NO. : TG0257

HOLE NO. : BH 9-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
TOPSOIL 0.05m [265.22m]			AU 1			14			265
GLACIAL TILL: Stiff to very stiff, brown silty clay to clayey silt with trace of organics		1	SS 2	50	5-6-6-6 12	23			264
2.13m [263.14m]		2	SS 3	75	2-3-3-4 6	20			263
GLACIAL TILL: Very stiff to hard, brown silty clay to clayey silt with some sand and gravel		3	SS 4	83	4-8-12-13 20	20			262
3.96m [261.31m]		4	SS 5	100	3-6-9-12 15	17			261
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel		5	SS 6	100	3-9-12-14 21	17			260
6.71m [258.56m]		6	SS 7	100	2-5-10-11 15	20			259
End of Borehole		7	SS 8	100	5-10-10-10 20	15			258
		8	SS 9	83	4-10-6-6 16	19			257
		9							256
		10							255
		11							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599751.17 **NORTHING:** 4860975.99 **ELEVATION:** 263.78

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 9, 2025

FILE NO. : TG0257

HOLE NO. : BH10-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE							263
AGRICULTURALLY DISTURBED SOIL: Brown silty clay to clayey silt with sand, organics and rootlets		0	AU 1			21	
0.71m [263.07m]							
GLACIAL TILL: Very stiff to hard, brown silty clay with some sand and gravel		1	SS 2	33	2-8-10-16 18	19	
		2	SS 3	83	2-9-9-13 18	21	262
			SS 4	100	3-10-10-12 20	19	
3.35m [260.43m]		3	SS 5	100	3-7-7-8 14	21 14	261
GLACIAL TILL: Stiff to very stiff, grey silty clay to clayey silt with some sand and gravel		4	SS 6	100	2-3-6-6 9	20	260
		5	SS 7	100	2-3-6-7 9	21	259
5.94m [257.84m]		6	SS 8	67	3-5-7-8 12	20	258
End of Borehole							
(GWL at 1.50 m depth - December 22, 2025)		7					257
		8					256
		9					255
		10					254
		11					253

PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) — 20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)

▲ UNDRAINED SHEAR STRENGTH (kPa) — 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) — 20 40 60 80

MONITORING WELL CONSTRUCTION

1.50 m 2025-12-22

2.62m

4.14m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599818.32 **NORTHING:** 4861156.18 **ELEVATION:** 265.19

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: EGM96 (Global) **DATE:** January 28, 2025

FILE NO. : TG0257

HOLE NO. : BH11-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE							265
AGRICULTURALLY DISTURBED SOIL: Brown silty clay to clayey silt with sand, organics and rootlets		0	G 1			23	
0.69m [264.50m]							
GLACIAL TILL: Hard to very stiff, brown silty clay with some sand and gravel		1	SS 2	58	4-6-11-14 17	19	264
		2	SS 3	100	2-7-10-11 17	18	263
			SS 4	75	P	16	
2.79m [262.40m]						26	
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel		3	SS 5	83	2-12-11-12 23	20	262
		4	SS 6	71	7-11-13-16 24	17	261
		5	SS 7	100	4-9-17-21 26	18	260
		6	SS 8	100	3-10-16-17 26	19	259
6.71m [258.48m]			SS 9	100	3-12-13-21 25	18	
End of Borehole		7					258
(BH Dry on - February 4, 2025)		8					257
(GWL at 4.45 m depth - December 22, 2025)		9					256
		10					255
		11					

PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

REMOULDED SHEAR STRENGTH (kPa)

UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

MONITORING WELL CONSTRUCTION

249

4.45 m 2025-12-22

5.97m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 600055.24 **NORTHING:** 4861150.74 **ELEVATION:** 263.33

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: EGM96 (Global) **DATE:** January 28, 2025

FILE NO. : TG0257

HOLE NO. : BH12-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE							263
AGRICULTURALLY DISTURBED SOIL: Brown silty clay to clayey silt with sand, organics and rootlets		0	G 1			16	
0.80m [262.53m]							
Compact, brown **SILTY SAND**, trace gravel		1	SS 2	100	3-5-7-13 12	17	
		2	SS 3	75	2-11-15-17 26	20	
			SS 4	50	9-11-13-14 24	23	261
3.20m [260.13m]		3					
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel			SS 5	75	5-10-8-9 18	22 19	260
		4	SS 6	0	6-8-12-16 20		259
		5	SS 7	100	2-12-14-20 26	17	258
			SS 8	75	5-11-17-26 28	17	
		6	SS 9	100	7-16-20-25 36	16	257
6.71m [256.62m]							
Dynamic Cone Penetration Test commenced at 6.71 m depth		7					256
		8					
8.23m [255.10m]							255
End of Borehole		9					
(GWL at 1.30 m depth - February 4, 2025)							254
(GWL at 2.96 m depth - December 22, 2025)		10					
							253
		11					

PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) — 20 40 60 80

REMOULDED SHEAR STRENGTH (kPa)

UNDRAINED SHEAR STRENGTH (kPa) — 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) — 20 40 60 80

45

52

68

118

153

MONITORING WELL CONSTRUCTION

1.30 m 2025-02-04

2.96 m 2025-12-22

4.51m

6.03m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599822.52 **NORTHING:** 4860874.63 **ELEVATION:** 265.25

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: EGM96 (Global) **DATE:** January 28, 2025

FILE NO. : TG0257

HOLE NO. : BH13-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE							265
TOPSOIL 0.05m [265.20m]		0	G 1			21	
GLACIAL TILL: Very stiff to hard, brown silty clay to clayey silt with some sand and gravel		1	SS 2	100	2-5-10-16 15	20	
		2	SS 3	100	3-9-10-16 19	19	263
		3	SS 4	100	4-12-18-29 30	19	
			SS 5	100	3-8-14-16 22	19	262
4.57m [260.68m]		4	SS 6	8	3-6-11-14 17	20	
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel		5	SS 7	100	3-8-17-28 25	19	260
			SS 8	100	3-7-14-13 21	23	
6.71m [258.54m]		6	SS 9		5-6-14-18 20	18	259
End of Borehole		7					258
(GWL at 1.33 m depth - February 4, 2025)		8					257
(GWL at 1.04 m depth - January 08, 2026)		9					256
		10					255
		11					

PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) — 20 40 60 80

REMOULDED SHEAR STRENGTH (kPa)

UNDRAINED SHEAR STRENGTH (kPa) — 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) — 20 40 60 80

MONITORING WELL CONSTRUCTION

1.04 m 2026-01-08

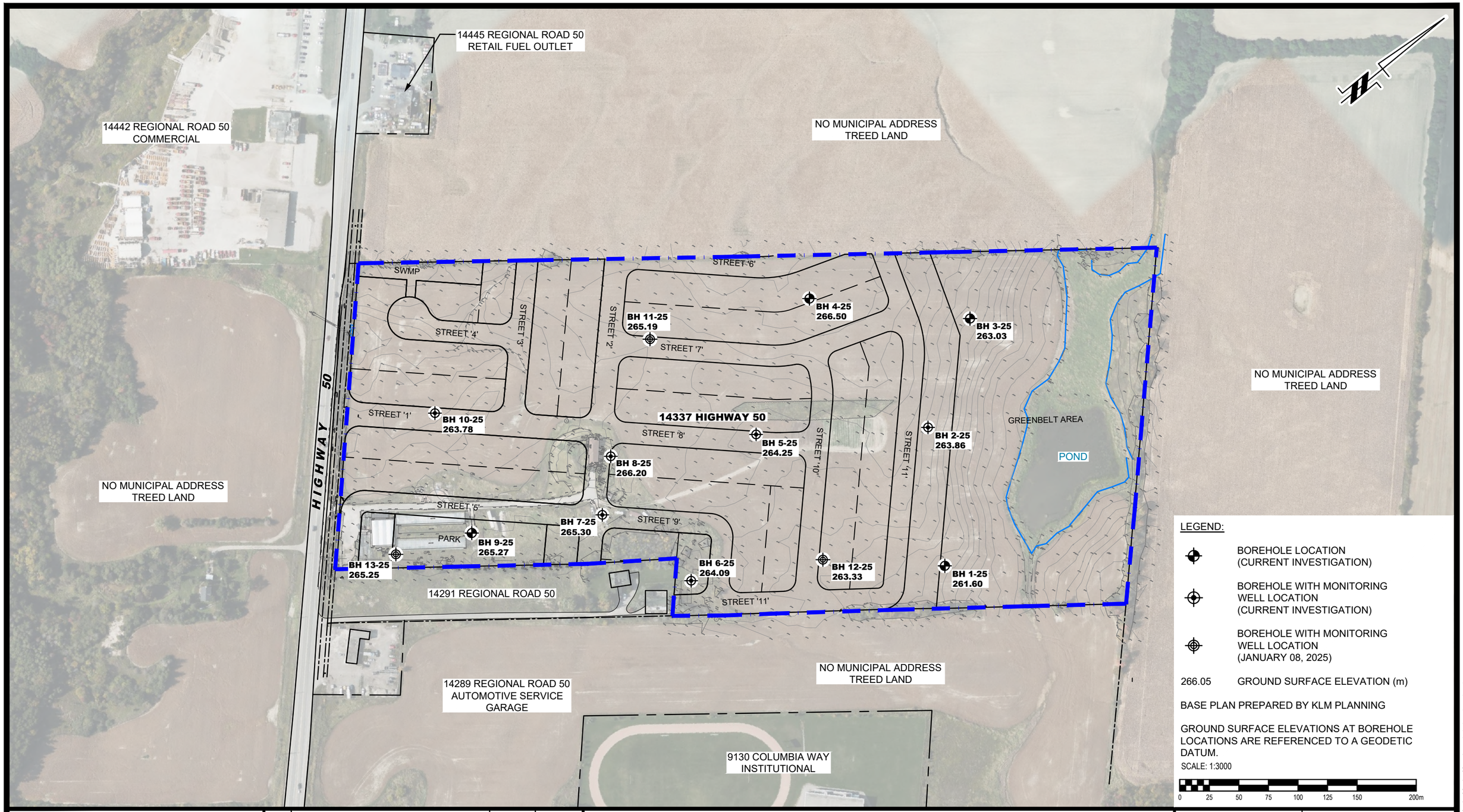
1.33 m 2025-02-04

4.26m

5.78m

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LEGEND:

- BOREHOLE LOCATION (CURRENT INVESTIGATION)
- BOREHOLE WITH MONITORING WELL LOCATION (CURRENT INVESTIGATION)
- BOREHOLE WITH MONITORING WELL LOCATION (JANUARY 08, 2025)

266.05 GROUND SURFACE ELEVATION (m)

BASE PLAN PREPARED BY KLM PLANNING

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:3000

0 25 50 75 100 125 150 200m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

TIFFANY ROX NORTH HILL LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
14337 HIGHWAY 50

BOLTON, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:3000	Date: 01/2026
Drawn by: YA	Report No.: TG0257-1
Checked by: BN	Dwg. No.: TG0257-1
Approved by: DG	Revision No.:

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11x17

APPENDIX 3

Table 5 - Pre-Development Annual Water Budget Calculations

Table 6 - Post-Development Annual Water Budget Calculations

Sample Calculations - Dupuit Forchheimer

Hydraulic Conductivity Results - Falling and Rising Head Tests

Water Quality Results

Table 5 - Pre-Development Annual Water Budget Calculations

Land Use Unit	Area (m^2)	Water Surplus (mm)	Topography Factor	Soil Factor	Vegetation Factor	Infiltration Factor	Runoff Factor	Total Infiltration (mm/year)	Total Infiltration (L/year)	Total Runoff (mm/year)	Total Runoff (L/year)
Agricultural Land (Silt Loam)	109,135	197	0.2	0.15	0.15	0.5	0.5	98.5	10,749,817	98.5	10,749,817
Mature Trees (Silt Loam)	1,546	162	0.2	0.15	0.2	0.55	0.45	89.1	137,737	72.9	112,694
Pasture and Shrubs (Silt Loam)	23,215	183	0.2	0.15	0.15	0.5	0.5	91.5	2,124,129	91.5	2,124,129
Urban Lawns (Fine Sand Loam)	2,413	279	0.2	0.3	0.1	0.6	0.4	167.4	403,960	111.6	269,306
Impervious Surfaces	4,760	660	0	0	0	0	1	0	0	660	3,141,455
Total	**141,069**								**13,415,643**		**16,397,402**

Table 6 - Post-Development Annual Water Budget Calculations

Land Use Unit	Area (m^2)	Water Surplus (mm)	Topography Factor	Soil Factor	Vegetation Factor	Infiltration Factor	Runoff Factor	Total Infiltration (mm/year)	Total Infiltration (L/year)	Total Runoff (mm/year)	Total Runoff (L/year)
Impervious Surfaces	111,904	660	0	0	0	0	1	0	0	660.0	73,856,640
Urban Lawn/Shallow Rooted Crops (Silt Loam)	27,411	238	0.2	0.15	0.1	0.45	0.55	107.1	2,935,718	130.9	3,588,100
Stormwater Management Pond	1,754	244	0	0	0	0	1	0	0	244	428,064
Totals	**141,069**								**2,935,718**		**77,872,804**
Difference (L/year)									**-10,479,925**		**61,475,402**
Percentage Variation									**-78%**		**375%**

File: TG0106

Estimated Groundwater Inflow

Tiffany Rox North Hill Ltd. - 14337 Highway 50, Caledon - Building Excavation

Dupuit-Forchheimer Equation $Q = \pi K((h_0^2 - h_p^2)/\ln(R/r))$

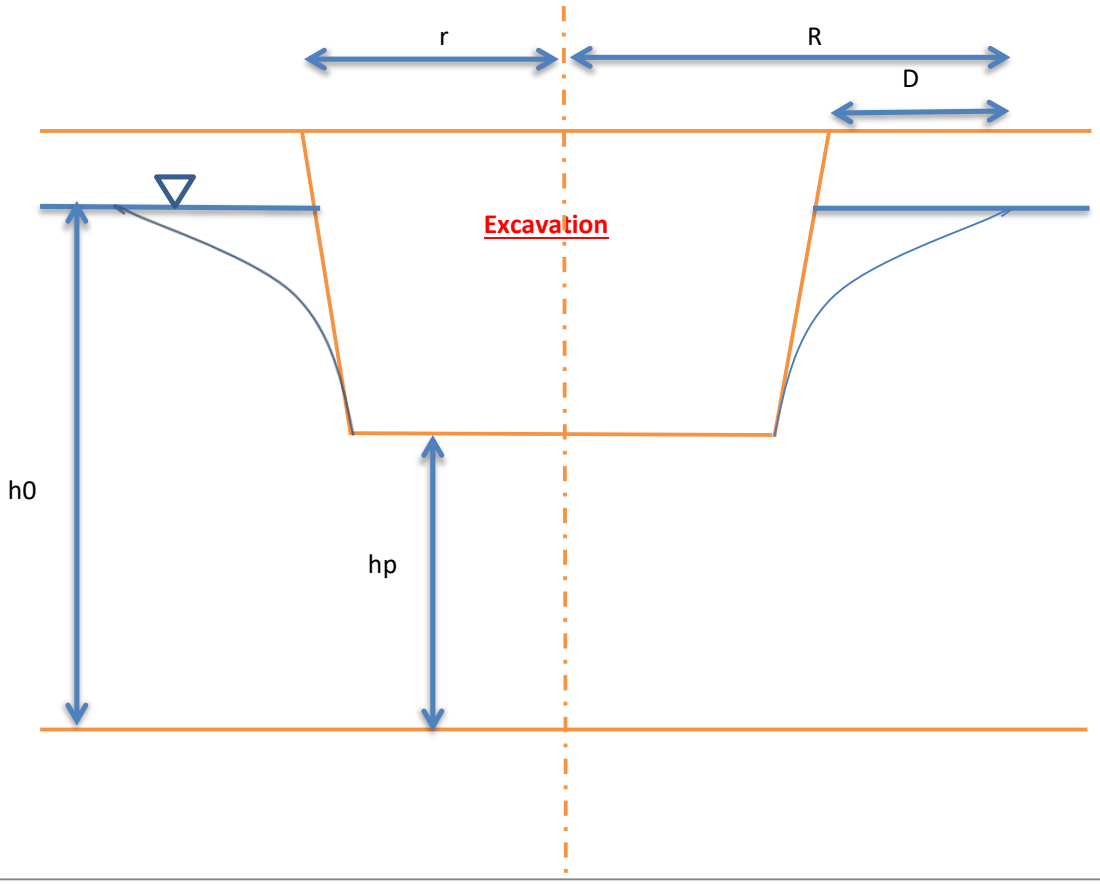
Equivalent Radius of Excavation = A+B=Pi*r

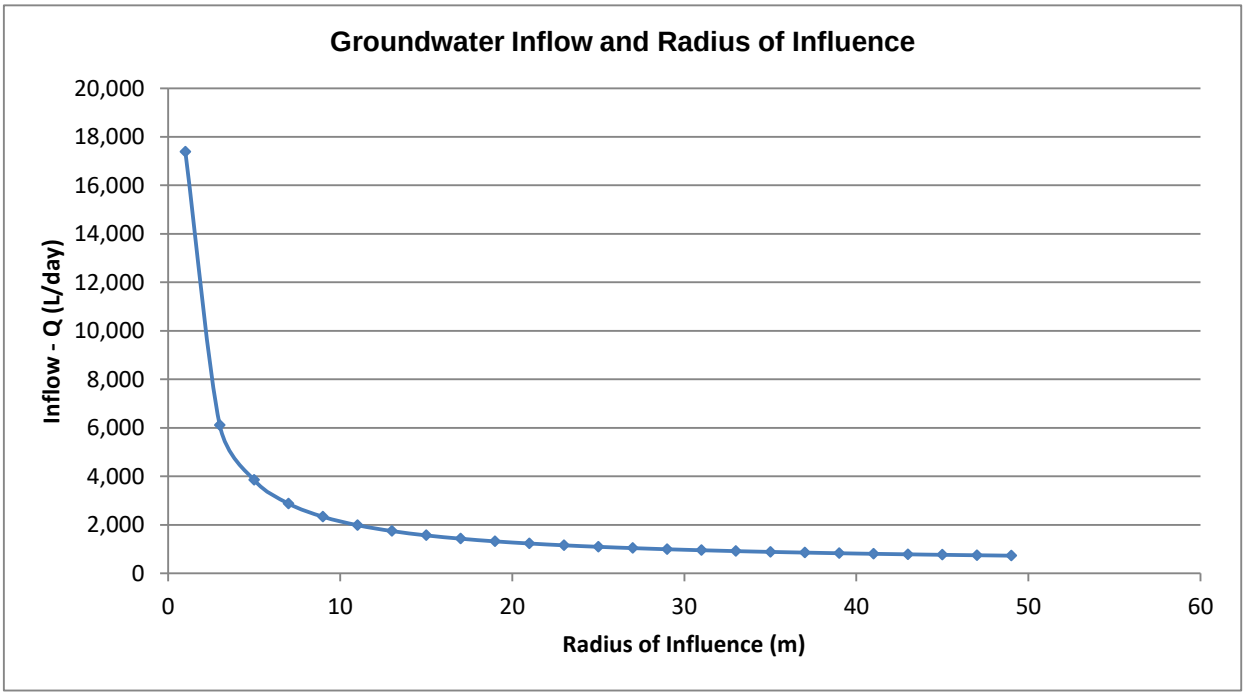
K (m/sec) =	1.45E-07			
h0 (m) =	25	Excavation Width (A) =	26.5	m
hp (m) =	24.5	Excavation Length (B) =	28	m
r (m) =	17.35	Perimeter Length =	109	m
		Equivalent Radius (r) =	17.35	m

R	Distance to edge of excavation (D)	Q (m^3/s)	Q (m^3/day)	Q (L/day)
18.35	**1.00**	**0.0002**	**17.4**	**17,381**
20.35	3.00	0.0001	6.1	6,107
22.35	5.00	0.0000	3.8	3,846
24.35	7.00	0.0000	2.9	2,874
26.35	9.00	0.0000	2.3	2,331
28.35	11.00	0.0000	2.0	1,984
30.35	13.00	0.0000	1.7	1,742
32.35	15.00	0.0000	1.6	1,563
34.35	17.00	0.0000	1.4	1,426
36.35	19.00	0.0000	1.3	1,317
38.35	21.00	0.0000	1.2	1,228
40.35	23.00	0.0000	1.2	1,154
42.35	25.00	0.0000	1.1	1,092
44.35	27.00	0.0000	1.0	1,038
46.35	29.00	0.0000	1.0	991
48.35	31.00	0.0000	1.0	950
50.35	33.00	0.0000	0.9	914
52.35	35.00	0.0000	0.9	882
54.35	37.00	0.0000	0.9	853
56.35	39.00	0.0000	0.8	827
58.35	41.00	0.0000	0.8	803
60.35	43.00	0.0000	0.8	781
62.35	45.00	0.0000	0.8	761
64.35	47.00	0.0000	0.7	743
66.35	49.00	0.0000	0.7	726
68.35	51.00	0.0000	0.7	710
70.35	53.00	0.0000	0.7	696
72.35	55.00	0.0000	0.7	682
74.35	57.00	0.0000	0.7	669
76.35	59.00	0.0000	0.7	657

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File: TG0106


r
R
D
Excavation
h0
hp


Groundwater Inflow and Radius of Influence
Inflow - Q (L/day)
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
0
10
20
30
40
50
60
Radius of Influence (m)

File: TG0106

Estimated Groundwater Inflow

Tiffany Rox North Hill Ltd. - 14337 Highway 50, Caledon - Servicing Excavation

Dupuit-Forchheimer Equation $Q = \pi K((h_0^2 - h_p^2)/\ln(R/r))$

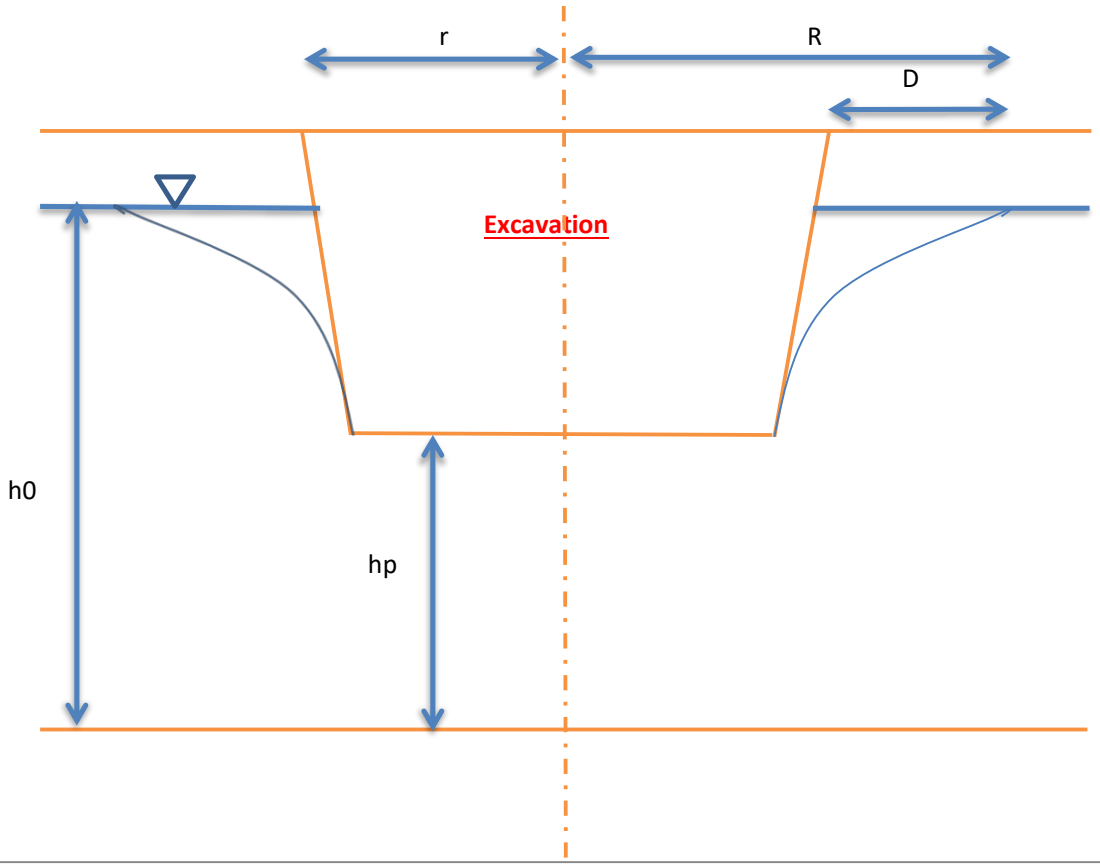
Equivalent Radius of Excavation = A+B=Pi*r

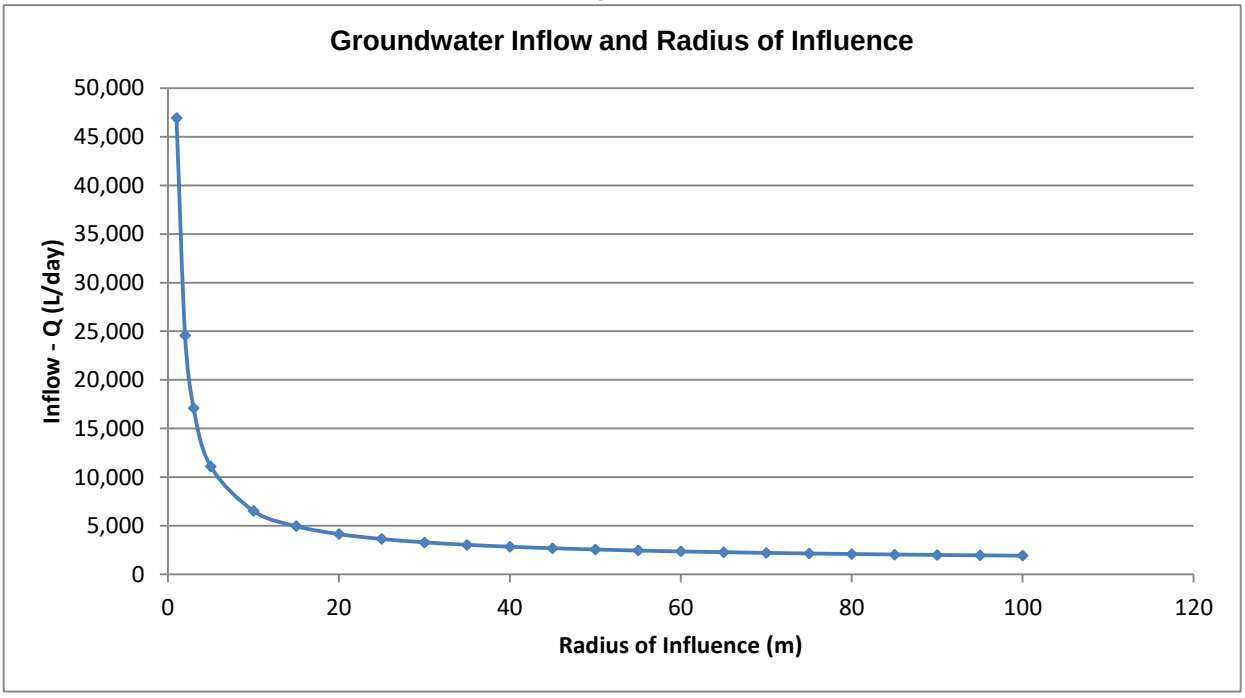
K (m/sec) =	1.45E-07			
h0 (m) =	25	Excavation Width (X) =	5	m
hp (m) =	22.5	Excavation Length (Y) =	25	m
r (m) =	9.55	Perimeter Length =	60	m
		Equivalent Radius (r) =	9.55	m

R	Distance to edge of excavation (D)	Q (m^3/s)	Q (m^3/day)	Q (L/day)
10.55	1.00	0.0005	46.9	46,929
11.55	2.00	0.0003	24.6	24,578
12.55	**3.00**	**0.0002**	**17.1**	**17,108**
14.55	5.00	0.0001	11.1	11,100
19.55	10.00	0.0001	6.5	6,523
24.55	15.00	0.0001	4.9	4,950
29.55	20.00	0.0000	4.1	4,138
34.55	25.00	0.0000	3.6	3,635
39.55	30.00	0.0000	3.3	3,289
44.55	35.00	0.0000	3.0	3,035
49.55	40.00	0.0000	2.8	2,839
54.55	45.00	0.0000	2.7	2,682
59.55	50.00	0.0000	2.6	2,553
64.55	55.00	0.0000	2.4	2,446
69.55	60.00	0.0000	2.4	2,354
74.55	65.00	0.0000	2.3	2,274
79.55	70.00	0.0000	2.2	2,205
84.55	75.00	0.0000	2.1	2,143
89.55	80.00	0.0000	2.1	2,088
94.55	85.00	0.0000	2.0	2,039
99.55	90.00	0.0000	2.0	1,994
104.55	95.00	0.0000	2.0	1,953
109.55	100.00	0.0000	1.9	1,916
114.55	105.00	0.0000	1.9	1,881
119.55	110.00	0.0000	1.8	1,849
124.55	115.00	0.0000	1.8	1,820
129.55	120.00	0.0000	1.8	1,792
134.55	125.00	0.0000	1.8	1,767
139.55	130.00	0.0000	1.7	1,743
144.55	135.00	0.0000	1.7	1,720

265.06

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File: TG0106

Estimated Groundwater Inflow

Tiffany Rox North Hill Ltd. - 14337 Highway 50, Caledon - SWMP Excavation

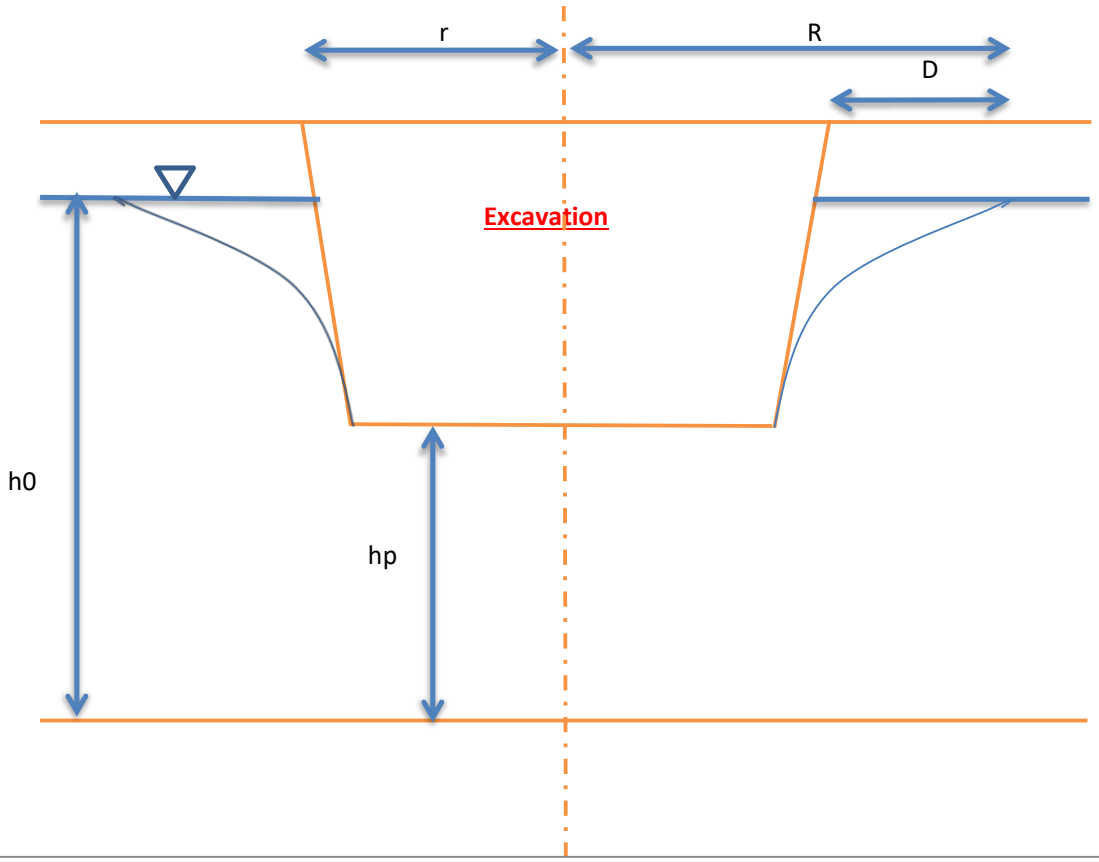
Dupuit-Forchheimer Equation **Q= πK((h0^2-hp^2)/ln(R/r))**

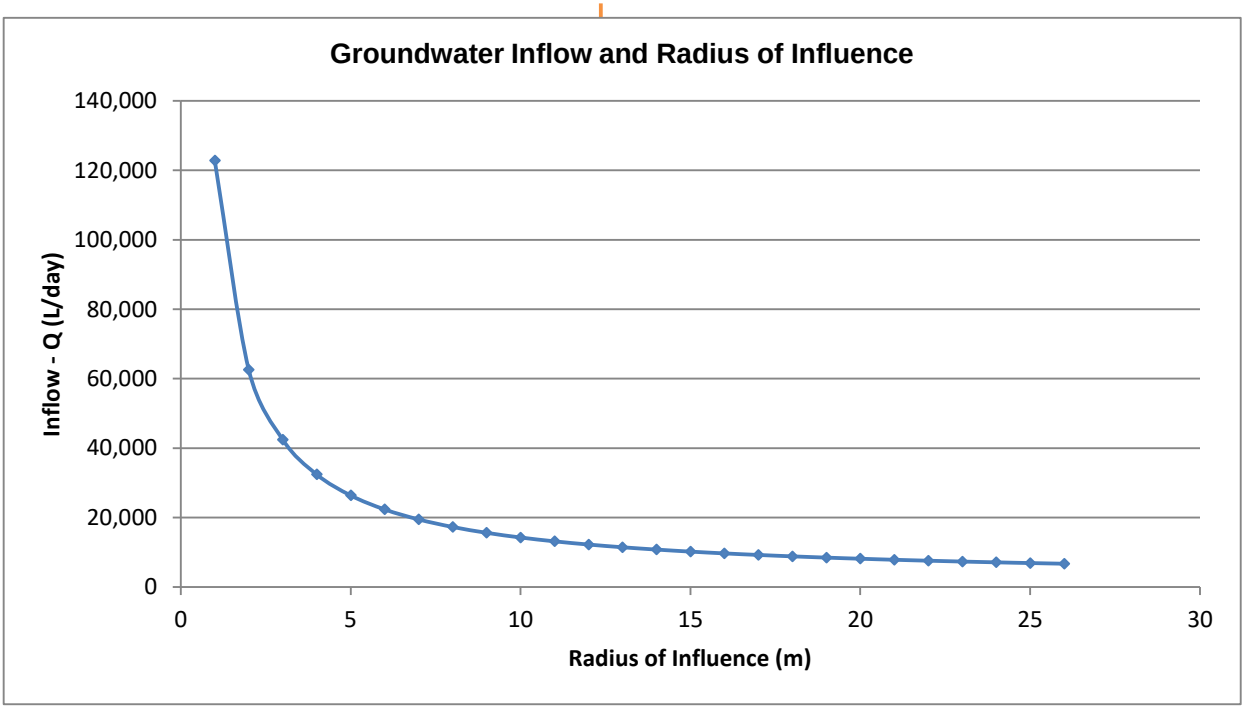
		Equivalent Radius of Excavation =	A+B=Pi + r
K (m/sec) =	1.45E-07		
h0 (m) =	25	Excavation Width (X) =	40 m
hp (m) =	22.5	Excavation Length (Y) =	41 m
r (m) =	25.78	Perimeter Length =	241 m
		Equivalent Radius (r) =	25.78 m

R	Distance to edge of excavation (D)	Q (m^3/s)	Q (m^3/day)	Q (L/day)
26.78	1.00	0.0014	122.8	122,826
27.78	2.00	0.0007	62.6	62,560
28.78	**3.00**	**0.0005**	**42.5**	**42,462**
29.78	4.00	0.0004	32.4	32,407
30.78	5.00	0.0003	26.4	26,369
31.78	6.00	0.0003	22.3	22,339
32.78	7.00	0.0002	19.5	19,458
33.78	8.00	0.0002	17.3	17,295
34.78	9.00	0.0002	15.6	15,610
35.78	10.00	0.0002	14.3	14,260
36.78	11.00	0.0002	13.2	13,154
37.78	12.00	0.0001	12.2	12,230
38.78	13.00	0.0001	11.4	11,448
39.78	14.00	0.0001	10.8	10,776
40.78	15.00	0.0001	10.2	10,192
41.78	16.00	0.0001	9.7	9,681
42.78	17.00	0.0001	9.2	9,229
43.78	18.00	0.0001	8.8	8,826
44.78	19.00	0.0001	8.5	8,465
45.78	20.00	0.0001	8.1	8,140
46.78	21.00	0.0001	7.8	7,844
47.78	22.00	0.0001	7.6	7,576
48.78	23.00	0.0001	7.3	7,329
49.78	24.00	0.0001	7.1	7,103
50.78	25.00	0.0001	6.9	6,895
51.78	26.00	0.0001	6.7	6,702
52.78	27.00	0.0001	6.5	6,523
53.78	28.00	0.0001	6.4	6,357
54.78	29.00	0.0001	6.2	6,201
55.78	30.00	0.0001	6.1	6,056

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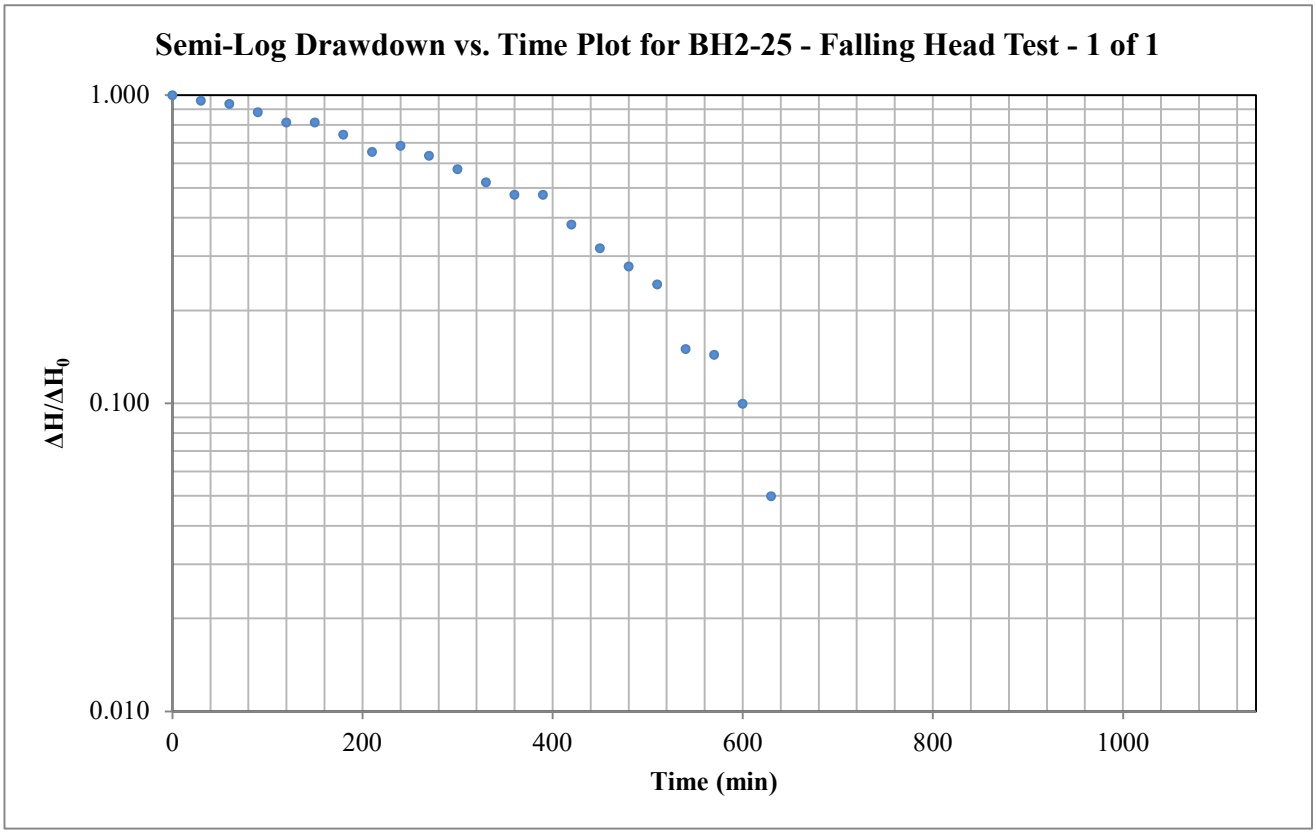
File: TG0106


r
R
D
Excavation
h0
hp


Groundwater Inflow and Radius of Influence
140,000
120,000
100,000
80,000
60,000
40,000
20,000
0
Inflow - Q (L/day)
0
5
10
15
20
25
30
Radius of Influence (m)

Hvorslev Hydraulic Conductivity Analysis

Project: Tiffancy Rox North Hill Ltd/ - Highway 50, Bolton - Parcel 6
Test Location: BH2-25
Test: Falling Head - 1 of 1
Date: January 22, 2026



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.31086

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

Data Points (from plot):

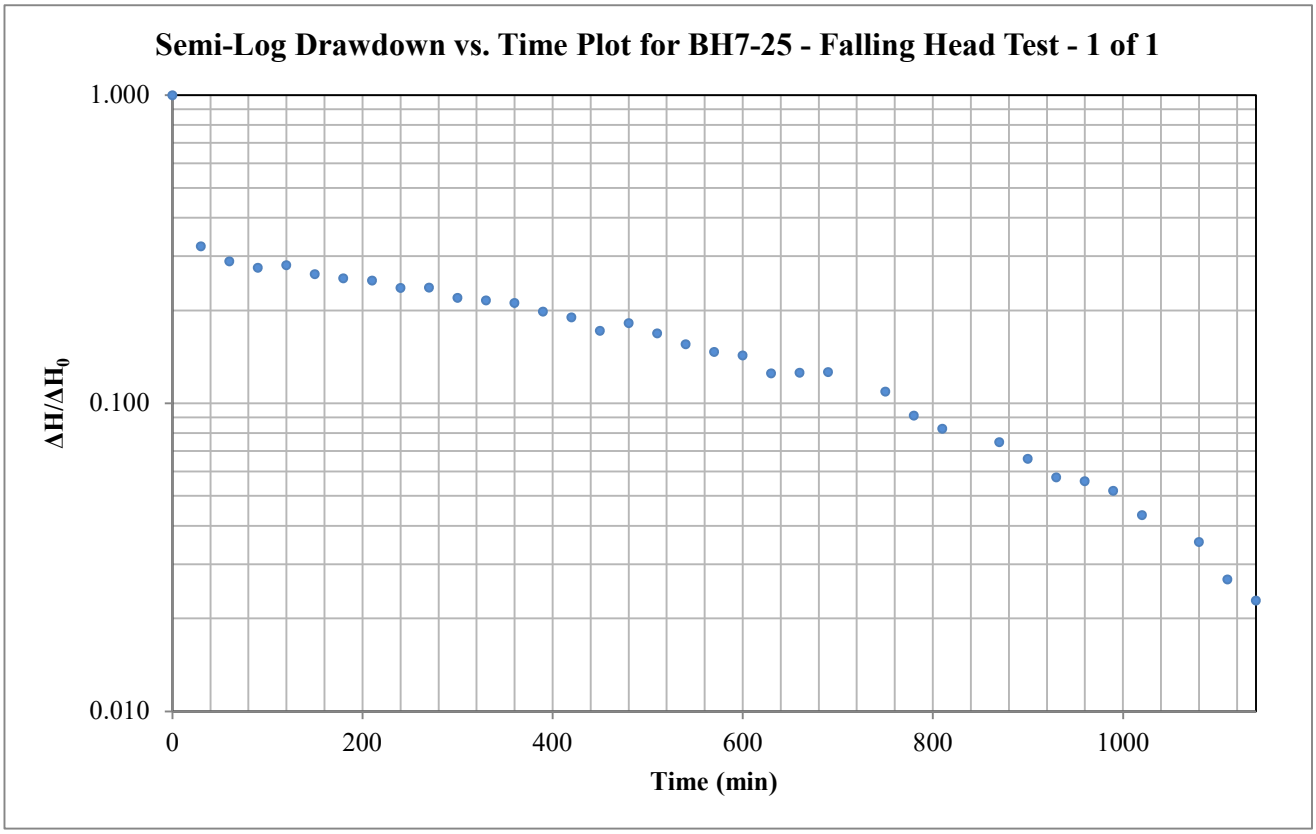
t*: 354.789 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.10E-08 m/sec

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Hvorslev Hydraulic Conductivity Analysis

Project: Tiffancy Rox North Hill Ltd/ - Highway 50, Bolton - Parcel 6
Test Location: BH7-25
Test: Falling Head - 1 of 1
Date: January 22, 2026



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.59613

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.015875	m	Radius of well

Data Points (from plot):

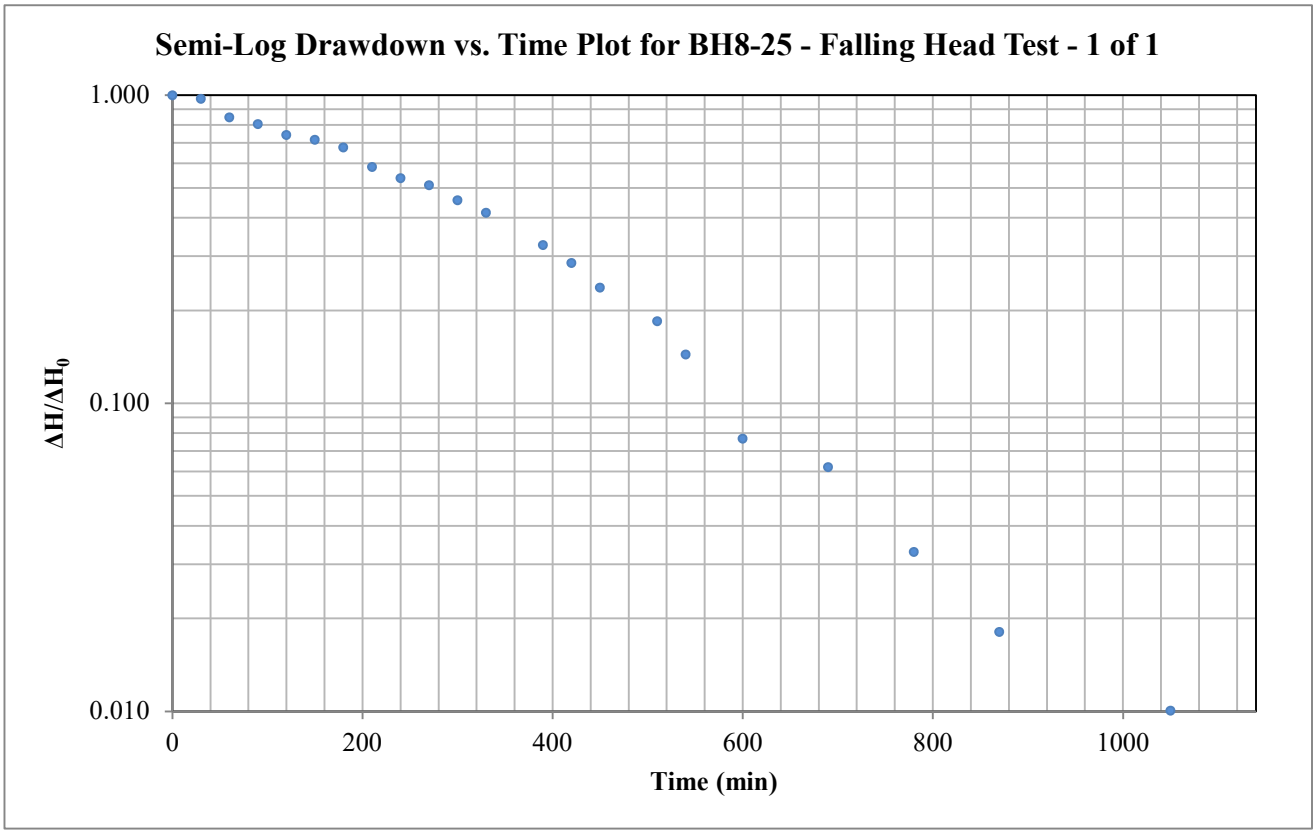
t*: 17.684 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.06E-07 m/sec

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Hvorslev Hydraulic Conductivity Analysis

Project: Tiffancy Rox North Hill Ltd. - Highway 50, Bolton - Parcel 6
Test Location: BH8-25
Test: Falling Head - 1 of 1
Date: January 22, 2026



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.59613

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.015875	m	Radius of well

Data Points (from plot):

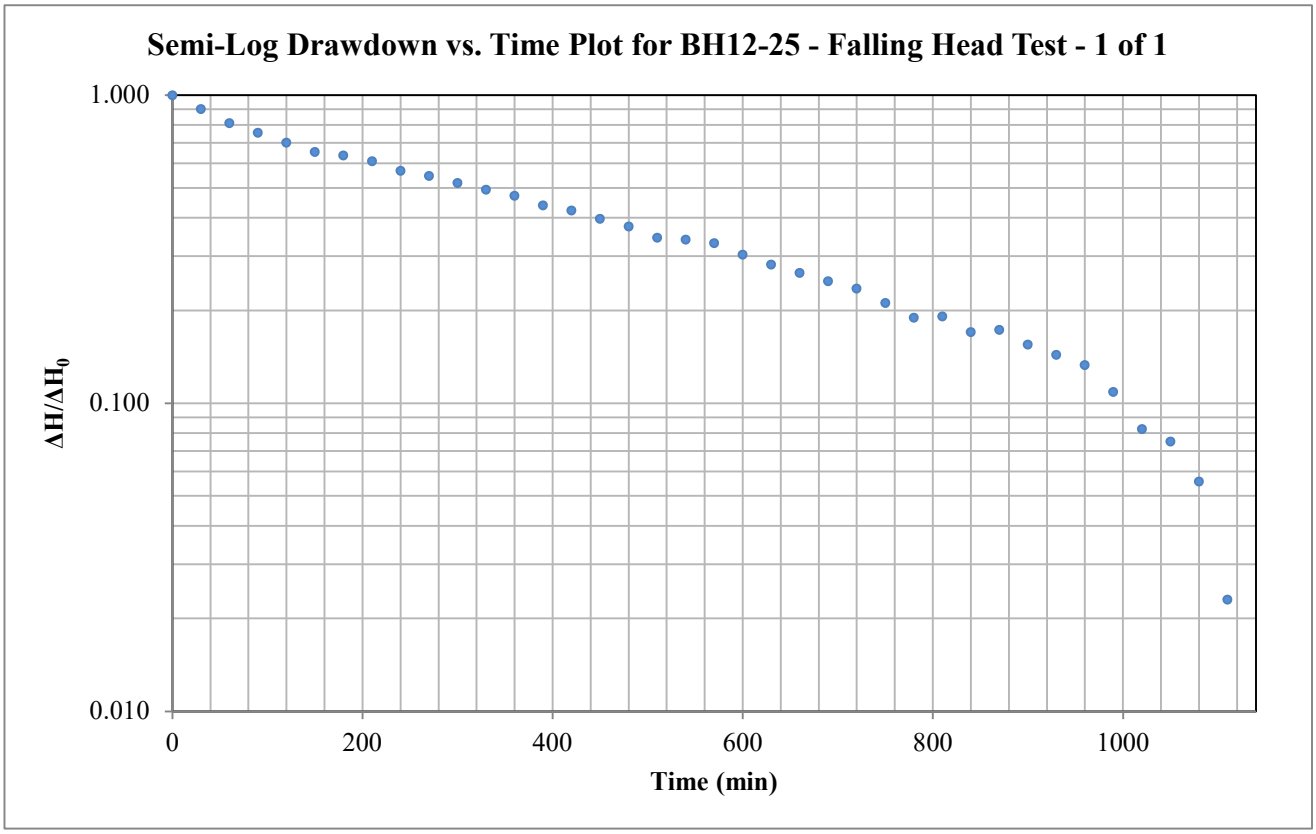
t*: 333.868 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.09E-08 m/sec

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Hvorslev Hydraulic Conductivity Analysis

Project: Tiffany Rox North Hill Ltd. - Highway 50, Bolton - Parcel 6
Test Location: BH12-25
Test: Falling Head - 1 of 1
Date: February 5, 2025



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.31086

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

Data Points (from plot):

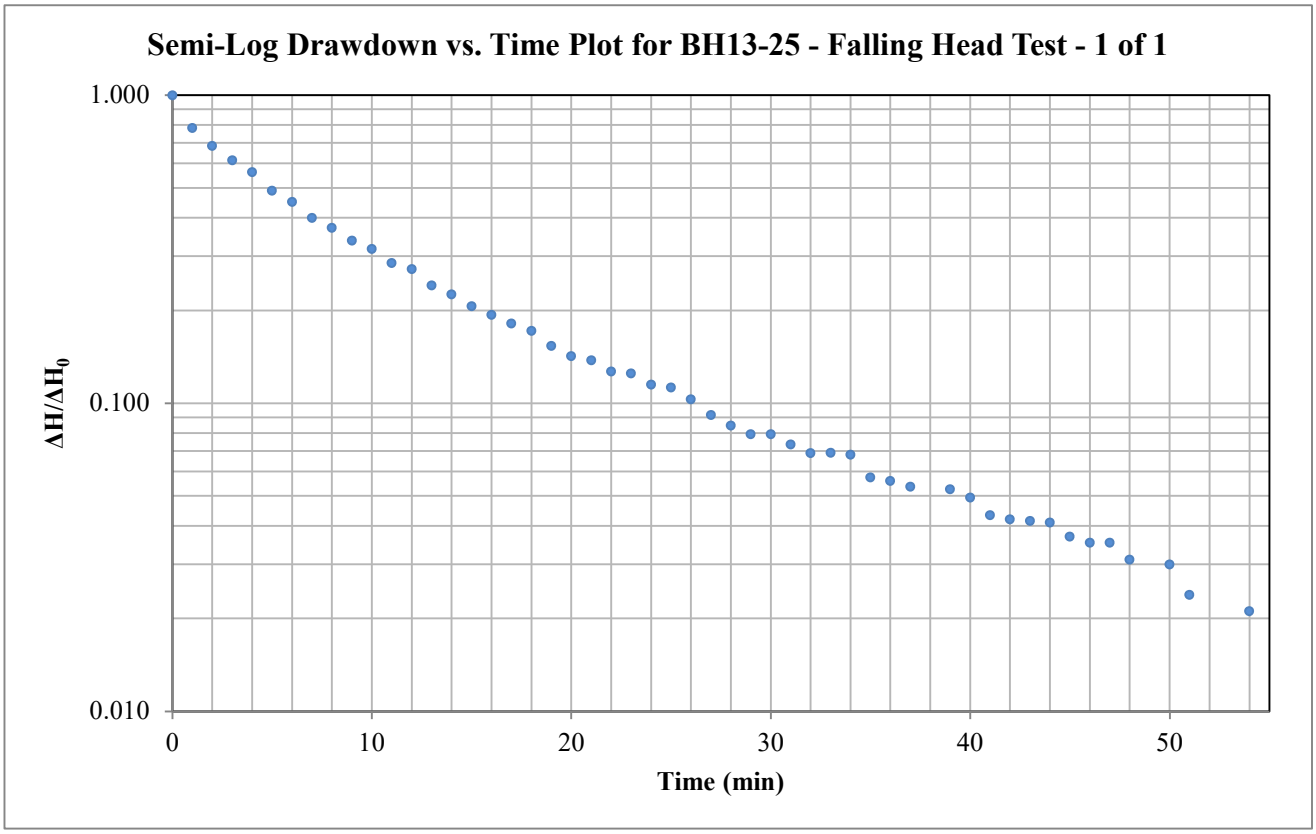
t*: 469.332 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.10E-08 m/sec

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Hvorslev Hydraulic Conductivity Analysis

Project: Tiffany Rox North Hill Ltd. - Highway 50, Bolton - Parcel 6
Test Location: BH13-25
Test: Falling Head - 1 of 1
Date: February 4, 2025



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.31086

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

Data Points (from plot):

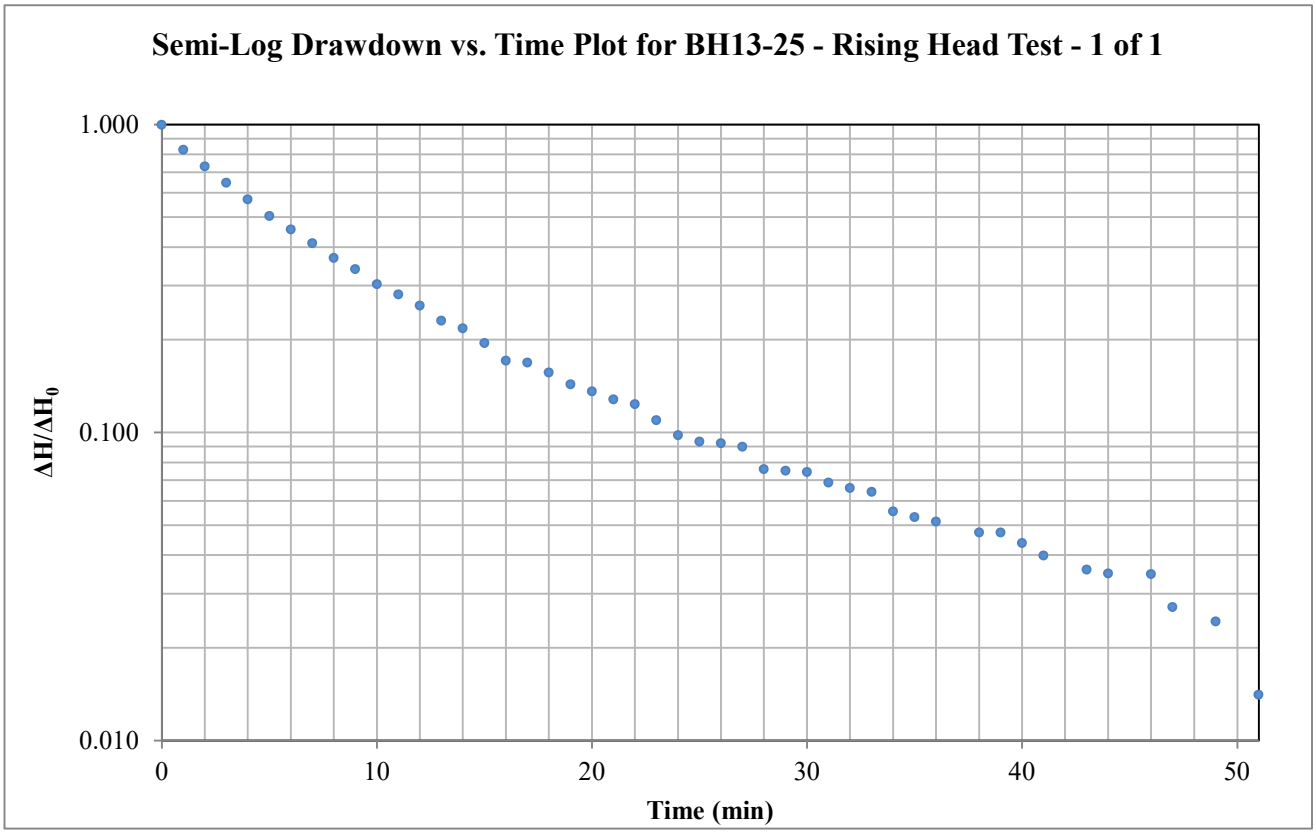
t*: 8.005 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.82E-06 m/sec

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Hvorslev Hydraulic Conductivity Analysis

Project: Tifany Rox North Hill Ltd. - Highway 50, Bolton - Parcel 6
Test Location: BH13-25
Test: Rising Head - 1 of 1
Date: February 4, 2025



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.31086

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

Data Points (from plot):

t*: 7.949 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.83E-06 m/sec

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Certificate of Analysis

Paterson Group Consulting Engineers (Mississauga)
1535 Meyerside Drive, Unit 7 & 8
Mississauga, ON L5T 1M9
Attn: Balaji Nirmala

Client PO: 64929
Project:
Custody: 149717

Report Date: 28-Jan-2026
Order Date: 20-Jan-2026

Order #: 2604007

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
2604007-01	BH7-25

Approved By:

Mark Foto

Mark Foto, M.Sc.
Laboratory Director

Analysis Summary Table

Analysis	Method Reference/Description	Lab Location	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC	Ottawa	22-Jan-26	22-Jan-26
CBOD	SM 5210B - DO Probe	Ottawa	21-Jan-26	21-Jan-26
Cyanide, total	MOE E3015 - Auto Colour	Ottawa	23-Jan-26	23-Jan-26
E. coli	MOE E3407	Ottawa	21-Jan-26	21-Jan-26
Mercury by CVAA	EPA 245.2 - Cold Vapour AA	Ottawa	22-Jan-26	22-Jan-26
Metals, ICP-MS	EPA 200.8 - ICP-MS	Ottawa	23-Jan-26	23-Jan-26
Oil & Grease, mineral/synthetic	SM5520F - Gravimetric	Ottawa	22-Jan-26	23-Jan-26
Oil & Grease, total	SM5520B - Gravimetric, hexane soluble	Ottawa	22-Jan-26	23-Jan-26
PCBs, total	EPA 608 - GC-ECD	Ottawa	21-Jan-26	22-Jan-26
Peel - Sanitary: Nonylphenols & Ethoxylates	Subcontract - ASTM D7065-06, GC/MS	Ottawa	21-Jan-26	21-Jan-26
Oil & Grease, animal/vegetable	SM5520 - Gravimetric	Ottawa	22-Jan-26	23-Jan-26
Peel - Sanitary: Phthalates	EPA 625	Ottawa	21-Jan-26	22-Jan-26
Peel - Sanitary: VOCs	EPA 624 - P&T GC-MS	Ottawa	21-Jan-26	21-Jan-26
pH	SM 4500-H+	Ottawa	21-Jan-26	22-Jan-26
Phenolics	EPA 420.2 - Auto Colour, 4AAP	Ottawa	21-Jan-26	21-Jan-26
Phosphorus, total, water	EPA 365.4 - Auto Colour, digestion	Ottawa	23-Jan-26	23-Jan-26
Total Kjeldahl Nitrogen	EPA 351.2 - Auto Colour, digestion	Ottawa	23-Jan-26	23-Jan-26
Total Suspended Solids	SM 2540D - Gravimetric	Ottawa	23-Jan-26	23-Jan-26

	Client ID:	BH7-25	-	-	-	-	-
	Sample Date:	20-Jan-26 09:30	-	-	-		
	Sample ID:	2604007-01	-	-	-		
	Matrix:	Ground Water	-	-	-		
	MDL/Units						
Microbiological Parameters							
E. coli	1 CFU/100mL	<10 [1] [2]	-	-	-	-	-
General Inorganics							
CBOD	2 mg/L	<2	-	-	-	-	-
Cyanide, total	0.01 mg/L	<0.01	-	-	-	-	-
pH	0.1 pH Units	7.2	-	-	-	-	-
Phenolics	0.001 mg/L	<0.001	-	-	-	-	-
Phosphorus, total	0.01 mg/L	0.27	-	-	-	-	-
Total Suspended Solids	2 mg/L	275	-	-	-	-	-
Total Kjeldahl Nitrogen	0.1 mg/L	0.4	-	-	-	-	-
Anions							
Fluoride	0.1 mg/L	0.2	-	-	-	-	-
Sulphate	1 mg/L	74	-	-	-	-	-
Metals - Total							
Aluminum	0.01 mg/L	6.09	-	-	-	-	-
Antimony	0.001 mg/L	<0.001	-	-	-	-	-
Arsenic	0.01 mg/L	<0.01	-	-	-	-	-
Cadmium	0.001 mg/L	<0.001	-	-	-	-	-
Chromium	0.05 mg/L	<0.05	-	-	-	-	-
Cobalt	0.001 mg/L	0.006	-	-	-	-	-
Copper	0.005 mg/L	0.010	-	-	-	-	-
Lead	0.001 mg/L	0.004	-	-	-	-	-
Mercury	0.0001 mg/L	<0.0001	-	-	-	-	-
Manganese	0.05 mg/L	0.44	-	-	-	-	-
Molybdenum	0.005 mg/L	<0.005	-	-	-	-	-
Nickel	0.005 mg/L	0.012	-	-	-	-	-

	Client ID:	BH7-25	-	-	-	-	-
	Sample Date:	20-Jan-26 09:30	-	-	-		
	Sample ID:	2604007-01	-	-	-		
	Matrix:	Ground Water	-	-	-		
	MDL/Units						
Metals - Total							
Selenium	0.005 mg/L	<0.005	-	-	-	-	-
Silver	0.001 mg/L	<0.001	-	-	-	-	-
Tin	0.01 mg/L	<0.01	-	-	-	-	-
Titanium	0.01 mg/L	0.21	-	-	-	-	-
Zinc	0.02 mg/L	0.02	-	-	-	-	-
Volatiles							
Benzene	0.0005 mg/L	<0.0005	-	-	-	-	-
Chloroform	0.0005 mg/L	<0.0005	-	-	-	-	-
1,2-Dichlorobenzene	0.0005 mg/L	<0.0005	-	-	-	-	-
1,4-Dichlorobenzene	0.0005 mg/L	<0.0005	-	-	-	-	-
cis-1,2-Dichloroethylene	0.0005 mg/L	<0.0005	-	-	-	-	-
trans-1,3-Dichloropropylene	0.0005 mg/L	<0.0005	-	-	-	-	-
Ethylbenzene	0.0005 mg/L	<0.0005	-	-	-	-	-
Methyl Ethyl Ketone (2-Butanone)	0.0050 mg/L	<0.0050	-	-	-	-	-
Methylene Chloride	0.0050 mg/L	<0.0050	-	-	-	-	-
Styrene	0.0005 mg/L	<0.0005	-	-	-	-	-
1,1,2,2-Tetrachloroethane	0.0005 mg/L	<0.0005	-	-	-	-	-
Tetrachloroethylene	0.0005 mg/L	<0.0005	-	-	-	-	-
Toluene	0.0005 mg/L	<0.0005	-	-	-	-	-
Trichloroethylene	0.0005 mg/L	<0.0005	-	-	-	-	-
Xylenes, total	0.0005 mg/L	<0.0005	-	-	-	-	-
Toluene-d8	Surrogate	92.4%	-	-	-	-	-
Dibromofluoromethane	Surrogate	113%	-	-	-	-	-
4-Bromofluorobenzene	Surrogate	98.6%	-	-	-	-	-
Hydrocarbons							

	Client ID:	BH7-25	-	-	-	-	-
	Sample Date:	20-Jan-26 09:30	-	-	-		
	Sample ID:	2604007-01	-	-	-		
	Matrix:	Ground Water	-	-	-		
	MDL/Units						
Hydrocarbons							
Oil & Grease, animal/vegetable	0.5 mg/L	<0.5	-	-	-	-	-
Oil & Grease, mineral/synthetic	0.5 mg/L	<0.5	-	-	-	-	-
Oil & Grease, total	0.5 mg/L	<0.5	-	-	-	-	-
Semi-Volatiles							
Bis(2-ethylhexyl)phthalate	0.001 mg/L	<0.001	-	-	-	-	-
Di-n-butylphthalate	0.001 mg/L	0.007	-	-	-	-	-
Terphenyl-d14	Surrogate	65.8% [6]	-	-	-	-	-
PCBs							
PCBs, total	0.05 ug/L	<0.05	-	-	-	-	-
Decachlorobiphenyl	Surrogate	80.0%	-	-	-	-	-
Subcontract							
Nonylphenol	0.00100 mg/L	<0.00100 [7]	-	-	-	-	-
Total Nonylphenol Ethoxylates	0.0100 mg/L	<0.0100 [7]	-	-	-	-	-
Nonylphenol Monoethoxylates	0.0100 mg/L	<0.0100 [7]	-	-	-	-	-
Nonylphenol Diethoxylates	0.0100 mg/L	<0.0100 [7]	-	-	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions								
Fluoride	ND	0.1	mg/L					
Sulphate	ND	1	mg/L					
General Inorganics								
CBOD	ND	2	mg/L					
Cyanide, total	ND	0.01	mg/L					
Phenolics	ND	0.001	mg/L					
Phosphorus, total	ND	0.01	mg/L					
Total Suspended Solids	ND	2	mg/L					
Total Kjeldahl Nitrogen	ND	0.1	mg/L					
Hydrocarbons								
Oil & Grease, mineral/synthetic	ND	0.5	mg/L					
Oil & Grease, total	ND	0.5	mg/L					
Metals - Total								
Aluminum	ND	0.01	mg/L					
Antimony	ND	0.001	mg/L					
Arsenic	ND	0.01	mg/L					
Cadmium	ND	0.001	mg/L					
Chromium	ND	0.05	mg/L					
Cobalt	ND	0.001	mg/L					
Copper	ND	0.005	mg/L					
Lead	ND	0.001	mg/L					
Manganese	ND	0.05	mg/L					
Mercury	ND	0.0001	mg/L					
Molybdenum	ND	0.005	mg/L					
Nickel	ND	0.005	mg/L					
Selenium	ND	0.005	mg/L					
Silver	ND	0.001	mg/L					
Tin	ND	0.01	mg/L					
Titanium	ND	0.01	mg/L					
Zinc	ND	0.02	mg/L					
Microbiological Parameters								
E. coli	ND	1	CFU/100mL					

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%REC	%REC Limit	RPD	RPD Limit	Notes
PCBs								
PCBs, total	ND	0.05	ug/L					
Surrogate: Decachlorobiphenyl	*0.221*		*%*	*88.5*	*60-140*			
Semi-Volatiles								
Bis(2-ethylhexyl)phthalate	ND	0.001	mg/L					
Di-n-butylphthalate	ND	0.001	mg/L					
Surrogate: 2-Fluorobiphenyl	*0.0207*		*%*	*104*	*76-125*			
Surrogate: Nitrobenzene-d5	*0.0209*		*%*	*105*	*68-125*			
Surrogate: Terphenyl-d14	*0.0181*		*%*	*90.3*	*70-125*			
Volatiles								
Benzene	ND	0.0005	mg/L					
Chloroform	ND	0.0005	mg/L					
1,2-Dichlorobenzene	ND	0.0005	mg/L					
1,4-Dichlorobenzene	ND	0.0005	mg/L					
cis-1,2-Dichloroethylene	ND	0.0005	mg/L					
trans-1,3-Dichloropropylene	ND	0.0005	mg/L					
Ethylbenzene	ND	0.0005	mg/L					
Methyl Ethyl Ketone (2-Butanone)	ND	0.0050	mg/L					
Methylene Chloride	ND	0.0050	mg/L					
Styrene	ND	0.0005	mg/L					
1,1,2,2-Tetrachloroethane	ND	0.0005	mg/L					
Tetrachloroethylene	ND	0.0005	mg/L					
Toluene	ND	0.0005	mg/L					
Trichloroethylene	ND	0.0005	mg/L					
Xylenes, total	ND	0.0005	mg/L					
Surrogate: 4-Bromofluorobenzene	*0.0728*		*%*	*91.1*	*50-140*			
Surrogate: Dibromofluoromethane	*0.0846*		*%*	*106*	*50-140*			
Surrogate: Toluene-d8	*0.0801*		*%*	*100*	*50-140*			

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Fluoride	0.47	0.1	mg/L	0.47			0.1	20	
Sulphate	75.9	1	mg/L	74.8			1.5	10	
General Inorganics									
Cyanide, total	ND	0.01	mg/L	ND			NC	20	
pH	7.7	0.1	pH Units	7.7			0.3	3.3	
Phenolics	ND	0.001	mg/L	ND			NC	10	
Phosphorus, total	ND	0.01	mg/L	ND			NC	15	
Total Suspended Solids	ND	2	mg/L	ND			NC	10	
Total Kjeldahl Nitrogen	0.22	0.1	mg/L	0.23			4.9	20	
Metals - Total									
Aluminum	0.01	0.01	mg/L	0.02			48.4	20	QR-05
Antimony	0.007	0.001	mg/L	ND			NC	20	
Arsenic	ND	0.01	mg/L	ND			NC	20	
Cadmium	ND	0.001	mg/L	ND			NC	20	
Chromium	ND	0.05	mg/L	ND			NC	20	
Cobalt	ND	0.001	mg/L	ND			NC	20	
Copper	0.018	0.005	mg/L	0.033			NC	20	
Lead	ND	0.001	mg/L	ND			NC	20	
Mercury	ND	0.0001	mg/L	ND			NC	20	
Manganese	ND	0.05	mg/L	ND			NC	20	
Molybdenum	ND	0.005	mg/L	ND			NC	20	
Nickel	ND	0.005	mg/L	ND			NC	20	
Selenium	ND	0.005	mg/L	ND			NC	20	
Silver	ND	0.001	mg/L	ND			NC	20	
Tin	ND	0.01	mg/L	ND			NC	20	
Titanium	ND	0.01	mg/L	ND			NC	20	
Zinc	ND	0.02	mg/L	ND			NC	20	
Microbiological Parameters									
E. coli	ND	10	CFU/100mL	ND			NC	30	BAC06, BAC12
Volatiles									

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Benzene	0.0098	0.0005	mg/L	0.0100			2.5	30	
Chloroform	ND	0.0005	mg/L	ND			NC	30	
1,2-Dichlorobenzene	ND	0.0005	mg/L	ND			NC	30	
1,4-Dichlorobenzene	ND	0.0005	mg/L	ND			NC	30	
cis-1,2-Dichloroethylene	0.0009	0.0005	mg/L	0.0010			6.2	30	
trans-1,3-Dichloropropylene	ND	0.0005	mg/L	ND			NC	30	
Ethylbenzene	0.0175	0.0005	mg/L	0.0178			1.8	30	
Methyl Ethyl Ketone (2-Butanone)	0.0341	0.0050	mg/L	0.0350			2.6	30	
Methylene Chloride	ND	0.0050	mg/L	ND			NC	30	
Styrene	0.0010	0.0005	mg/L	0.0010			4.1	30	
1,1,2,2-Tetrachloroethane	ND	0.0005	mg/L	ND			NC	30	
Tetrachloroethylene	ND	0.0005	mg/L	ND			NC	30	
Toluene	0.0773	0.0005	mg/L	0.0782			1.2	30	
Trichloroethylene	ND	0.0005	mg/L	ND			NC	30	
m,p-Xylenes	0.0662	0.0005	mg/L	0.0665			0.4	30	
o-Xylene	0.0261	0.0005	mg/L	0.0265			1.3	30	
Surrogate: 4-Bromofluorobenzene	*0.0763*		*%*		*95.4*	*50-140*			
Surrogate: Dibromofluoromethane	*0.0870*		*%*		*109*	*50-140*			
Surrogate: Toluene-d8	*0.0788*		*%*		*98.4*	*50-140*			

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Fluoride	1.45	0.1	mg/L	0.47	97.9	70-130			
Sulphate	84.7	1	mg/L	74.8	99.4	74-126			
General Inorganics									
CBOD	182	2	mg/L	ND	90.8	62-129			
Cyanide, total	0.048	0.01	mg/L	ND	96.9	64-136			
Phenolics	0.024	0.001	mg/L	ND	95.3	80-120			
Phosphorus, total	1.06	0.01	mg/L	ND	106	80-120			
Total Suspended Solids	21.0	2	mg/L	ND	94.6	84-124			
Total Kjeldahl Nitrogen	1.21	0.1	mg/L	0.23	98.2	75-125			
Hydrocarbons									
Oil & Grease, mineral/synthetic	7.45	0.5	mg/L	ND	74.5	65-110			
Oil & Grease, total	21.4	0.5	mg/L	ND	107	85-110			
Metals - Total									
Aluminum	57.0	0.01	mg/L	1.90	110	80-120			
Antimony	45.0	0.001	mg/L	ND	90.0	80-120			
Arsenic	57.5	0.01	mg/L	0.092	115	80-120			
Cadmium	53.2	0.001	mg/L	0.003	106	80-120			
Chromium	65.7	0.05	mg/L	0.053	131	80-120			QM-07
Cobalt	61.2	0.001	mg/L	0.004	122	80-120			QM-07
Copper	60.6	0.005	mg/L	3.26	115	80-120			
Lead	48.7	0.001	mg/L	0.017	97.3	80-120			
Mercury	0.0028	0.0001	mg/L	ND	93.6	70-130			
Manganese	63.4	0.05	mg/L	0.094	127	80-120			QM-07
Molybdenum	61.1	0.005	mg/L	0.139	122	80-120			QM-07
Nickel	61.7	0.005	mg/L	0.075	123	80-120			QM-07
Selenium	53.7	0.005	mg/L	0.038	107	80-120			
Silver	52.5	0.001	mg/L	0.002	105	80-120			
Tin	56.1	0.01	mg/L	0.036	112	80-120			
Titanium	64.2	0.01	mg/L	0.015	128	80-120			QM-07
Zinc	54.7	0.02	mg/L	0.436	109	80-120			

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
PCBs									
PCBs, total	0.807	0.05	ug/L	ND	80.7	65-135			
Surrogate: Decachlorobiphenyl	*0.269*		*%*		*108*	*60-140*			
Semi-Volatiles									
Bis(2-ethylhexyl)phthalate	0.0109	0.001	mg/L	ND	109	50-140			
Di-n-butylphthalate	0.0106	0.001	mg/L	ND	106	50-140			
Surrogate: 2-Fluorobiphenyl	*0.0169*		*%*		*84.7*	*76-125*			
Surrogate: Nitrobenzene-d5	*0.0157*		*%*		*78.4*	*68-125*			
Surrogate: Terphenyl-d14	*0.0145*		*%*		*72.3*	*70-125*			
Volatiles									
Benzene	0.0420	0.0005	mg/L	ND	105	60-130			
Chloroform	0.0418	0.0005	mg/L	ND	105	60-130			
1,2-Dichlorobenzene	0.0419	0.0005	mg/L	ND	105	60-130			
1,4-Dichlorobenzene	0.0414	0.0005	mg/L	ND	103	60-130			
cis-1,2-Dichloroethylene	0.0422	0.0005	mg/L	ND	106	60-130			
trans-1,3-Dichloropropylene	0.0437	0.0005	mg/L	ND	109	60-130			
Ethylbenzene	0.0407	0.0005	mg/L	ND	102	60-130			
Methyl Ethyl Ketone (2-Butanone)	0.107	0.0050	mg/L	ND	107	50-140			
Methylene Chloride	0.0443	0.0050	mg/L	ND	111	60-130			
Styrene	0.0406	0.0005	mg/L	ND	101	60-130			
1,1,2,2-Tetrachloroethane	0.0446	0.0005	mg/L	ND	112	60-130			
Tetrachloroethylene	0.0406	0.0005	mg/L	ND	102	60-130			
Toluene	0.0404	0.0005	mg/L	ND	101	60-130			
Trichloroethylene	0.0426	0.0005	mg/L	ND	106	60-130			
m,p-Xylenes	0.0813	0.0005	mg/L	ND	102	60-130			
o-Xylene	0.0409	0.0005	mg/L	ND	102	60-130			
Surrogate: 4-Bromofluorobenzene	*0.0876*		*%*		*109*	*50-140*			
Surrogate: Dibromofluoromethane	*0.0851*		*%*		*106*	*50-140*			
Surrogate: Toluene-d8	*0.0782*		*%*		*97.8*	*50-140*			

Qualifier Notes:

Sample Qualifiers :

1: Increased detection limit was a consequence of dilution/reduced analysis volume due to high background bacteria levels / sample matrix limitations.
Applies to Samples: BH7-25

2: Confluent background/interfering flora: May interfere with target colony growth and the analysts' ability to count discrete colonies. The target colonies may be under-represented.
Applies to Samples: BH7-25

6: Surrogate recovery outside of control limits. The data was accepted based on valid recovery of the remaining surrogate.
Applies to Samples: BH7-25

7: Subcontracted analysis - SGS

QC Qualifiers:

BAC06	Increased detection limit was a consequence of dilution/reduced analysis volume due to high background bacteria levels / sample matrix limitations.
BAC12	Confluent background/interfering flora: May interfere with target colony growth and the analysts' ability to count discrete colonies. The target colonies may be under-represented.
QM-07	The spike recovery was outside acceptance limits for the MS and/or MSD. The batch was accepted based on other acceptable QC.
QR-05	Duplicate RPDs higher than normally accepted. Remaining batch QA\QC was acceptable. May be sample effect.

Sample Data Revisions:

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

NC: Not Calculated

Any use of these results implies your agreement that our total liabilty in connection with this work, however arising, shall be limited to the amount paid by you for this work, and that our employees or agents shall not under any circumstances be liable to you in connection with this work.

PARACEL

Paracel ID: 2604007

Paracel Order Number (Lab Use Only)

Chain Of Custody (Lab Use Only)
№ 149717

Client Name: Paterson Group	Project Ref:	Page __ of __
Contact Name: Balaji Nirmala	Quote #:	**Turnaround Time**
Address: 1535 Meyerside Drive, Unit 7&8, Mississauga	PO #: 64929	☐ 1 day ☐ 3 day
	E-mail: bnirmala@patersongroup.ca	☐ 2 day ☒ Regular
Telephone:		Date Required:

☐ REG 153/04 ☐ REG 406/19

☐ Table 1 ☐ Agri/Other ☐ Med/Fine
☐ Table 2 ☐ Res/Park ☐ Coarse
☐ Table 3 ☐ Ind/Comm
☐ Table___
For RSC: ☐ Yes ☐ No

Other Regulation

☐ REG 558 ☐ PWQO
☐ CCME ☐ MISA
☐ SU - Sani ☐ SU - Storm
Mun:
☐ Other:

Matrix Type: S (Soil/Sed.) GW (Ground Water) SW (Surface Water) SS (Storm/Sanitary Sewer) P (Paint) A (Air) O (Other)

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Field Filtered	Sample Taken: Date	Sample Taken: Time	PHCs F1-F4+BTEX	VOCs	PAHs	Metals by ICP	Hg	CrVI	B (HWE)	Regulation for Fuel Storage Tank Sites 2019			
1	BH 7-25	GW		16	N	Jan 20, 2026	9:30 a.m								X			
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

Comments:

Method of Delivery: RABSEX

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Relinquished By (Sign): Balaji Nirmala	Received at Depot: JW	Received at Lab: SO	Verified By: SO
Relinquished By (Print): Balaji Nirmala	Date/Time: 01/20/2026 1322	Date/Time: Jan 21, 2026 10:00am	Date/Time: Jan 21, 2026 10:30am
Date/Time: Jan 20, 2025	Temperature: 9.5 °C	Temperature: 4.2 °C	pH Verified: ☑ By: SO

APPENDIX D
Background Information and
Relevant Excerpts

FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

BOLTON NORTH HILL
OPTION 1 LANDS

TOWN OF CALEDON
REGION OF PEEL

PREPARED FOR:

BOLTON NORTH HILL LANDOWNERS GROUP

PREPARED BY:

C.F. CROZIER & ASSOCIATES INC.
211 YONGE STREET, SUITE 600
TORONTO, ON M5B 1M4

MARCH 2026

CFCA FILE NO. 708-3446

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CROZIER

2.0 Water Servicing

2.1 Background

R.J. Burnside was retained by the Bolton North Hill Landowners Group in cooperation/ coordination with the Option 3 Landowners Group to complete an analysis on the existing water distribution system in the Bolton Residential Expansion Study (BRES) lands. The goal of this analysis was to provide a proposed water servicing strategy for the full build-out scenario of the Option 1, 2, and 3 Lands. The latest conclusions of their Technical Memorandum (July 14, 2025) with respect to Option 1 and 2 Lands are summarized below and Appendix A contains R.J. Burnside's Technical Memorandum from July 14, 2025.

2.2 Pressure Zone 7

According to the Bolton Residential Expansion Study (Region of Peel, September 24, 2020), the development of Option 1 and 2 Lands will require a new pressure zone to facilitate water servicing. The Site is outside of the range of elevations associated with the water distribution infrastructure for Pressure Zone 6 (214.5 m – 259.1 m), as stated in the Water and Wastewater Master Plan (Region of Peel, 2020). Therefore, ultimate development of Option 1 and 2 Lands will require the addition of Pressure Zone 7 (PZ7), accompanied by a new Zone 7 booster pumping station at King Street and Coleraine Drive.

Storage is proposed in the form of two elevated tanks to provide flow equalization and to meet the required fire and emergency demands. One of the proposed elevated tanks is in the northeast corner of Option 1 Lands, adjacent to Regional Road 50. The serviceable elevation of PZ7 ranges from 243.4 m to 289.6 m as stated in the Water and Wastewater Master Plan (Region of Peel, 2020).

2.3 Interim Water Servicing Strategy

R.J. Burnside and HydraTek have updated their analysis on the water servicing for both the interim and ultimate development scenarios. HydraTek's Hydraulics Technical Memorandum No. 1 (June 2025) uses hydraulic modelling analysis to propose that the Macville Booster Pump Station (BPS) can service an interim population of 2,960 persons within the Option 1 Lands. This memo provides analysis for the Macville BPS and characterization for PZ7 under existing, interim, and ultimate conditions. The model demonstrated that in the interim period, Bolton PZ7 will operate as a closed pressure system with a supply HGL of 327.7m and system pressures controlled by the Macville BPS.

Interim servicing will be maintained until development within the Option 1/2 Lands exceeds approximately 2,960 persons. Beyond this threshold, additional water storage will be required to satisfy flow equalization, fire protection, and emergency supply requirements. At that stage, floating storage is proposed in the form of two elevated tanks (ETs). One elevated tank is proposed within the Option 1 Lands, with its final location to be confirmed through the ongoing Environmental Assessment.

2.4 Ultimate Water Servicing Strategy

The full build-out modelling presented in HydraTek's Hydraulics Technical Memorandum No. 1 demonstrates that the ultimate PZ 7 servicing solution can accommodate full build-out conditions. The ultimate servicing strategy includes two ETs located within Option 1 and Option 3 Lands, a 600 mm diameter feedermain along Emil Kolb Parkway to Regional Road 50, and a 400mm along Regional Road 50 and Columbia Way. This infrastructure is sufficient to service the full PZ7 build-out, subject to confirmation of elevated tank sizing and final locations through the ongoing Environmental Assessment.

Under the ultimate build-out scenario, a looped watermain system is proposed to interconnect the Option 1, 2, & 3 Lands, providing system redundancy and operational reliability. The elevated tank proposed within the Option 1 Lands is currently anticipated to be located on landowner property outside of the development limits within the Oak Ridges Moraine (ORM). The Ontario Regulation 140/02 policy of the ORM allows for infrastructure within its limits but not development. Internally, water servicing is proposed to provide the necessary domestic and fire demands. A looped system will be maintained, and all Region of Peel standards will be adhered to for the design of the watermains and appurtenances.

The proposed Macville Booster PS will ultimately work in conjunction with the elevated water tanks that will provide water supply for the entire Pressure Zone 7. This design is being undertaken in consultation with the Region, as it is proposed on Region owned- lands and will be assumed by the Region for operation and maintenance. Alongside the detailed design, R.J. Burnside and HydraTek is updating their analysis on the water servicing for both the interim and ultimate development scenarios.

Further details of the full build-out water servicing strategy can be seen on Drawing C709 – Preliminary Water Distribution Plan. The water servicing strategy for the Option 1 lands will need to respect the findings of any updated analysis conducted by R.J. Burnside and/or the Region of Peel.

3.0 Sanitary Servicing

3.1 Existing Sanitary Servicing

The existing sanitary servicing infrastructure for lake-based systems within the Region of Peel is described in detail in the Water and Wastewater Master Plan (Region of Peel, 2020). Volume 3 and Volume 4 of this Master Plan discuss existing water and sanitary infrastructure in the Region, including an examination of the capacity of Bolton's existing sanitary infrastructure. The Master Plan is expected to be updated in 2025 (Volume 5); the servicing strategy moving forward from that point will need to respect the approach and conclusions set forth by the Region.

Bolton is predominantly serviced by two sanitary trunk sewers: a 900 mm diameter trunk sewer on Albion-Vaughan Road in the east end and a 525 mm diameter trunk sewer on Coleraine Drive in the west end. These two trunk sewers merge outside of the southern limits of Bolton at the intersection of Coleraine Drive and Regional Road 50. The wastewater then continues flowing south in a sanitary sewer along Regional Road 50 to the McVean Sewage Pumping Station, and ultimately for treatment at the G.E. Booth Wastewater Treatment Plant.

The 525 mm diameter sanitary trunk sewer on Coleraine Drive was installed in 1983, expanding to approximately 800 m north of George Bolton Parkway. This sanitary sewer was then extended 400 m further north at 600 mm diameter in 1989, then terminated at 375 mm diameter at King Street West in 1999.

The 900 mm diameter Albion-Vaughan (AV) trunk sewer as it exists today was completed in early 2015 and currently terminates north of the railway corridor on Albion Vaughan Road. The Region of Peel's capital works program included completing this trunk sewer by the spring of 2021 to terminate at the northern most extent of Nunnville Road, which also involves the decommissioning of the existing Albion Vaughan and Harvestview Sewage Pumping Stations along a similar timeline. The AV trunk sewer has an approximate full flow capacity of 1,000 L/s.

3.2 Design Sanitary Flow

The Water and Wastewater Master Plan, Volume 4 (Region of Peel, 2020) was referenced to calculate the sanitary design flows for the Site. Dry weather unit sewage flows of 290 L/capita/day and 270 L/capita/day were used for residential and employment populations, respectively. Population yields (derived from the unit count of the Concept Plan), an infiltration flow of 0.26 L/s/ha, and a peaking factor were applied to the unit sewage flow to obtain the total estimated design sanitary flow. Table 1 summarizes the results and Appendix B2 contains detailed calculations.

Table 1: Estimated Sanitary Design Flows

Land Parcel	Development Area (ha)	Average Daily Flow (L/s)	Average Peaking Factor	Peak Flow (L/s)	Infiltration Flow (L/s)	Total Sanitary Flow (L/s)*
Option 1	150.4	105.1	3.6	171.8	39.1	274.2
Option 2	4.64	3.2	4.00	6.46	1.2	9.95

*30% contingency factor has been applied to address potential scope changes

Therefore, the proposed development of Option 1 and 2 Lands will generate a total of 284.2 L/s in sanitary discharge.

3.3 Proposed Sanitary Servicing

The external servicing strategy has been developed in consideration of the Region's advancing Water and Wastewater Municipal Class EA Crozier continues to work closely with the Region and its consultants through monthly coordination meetings to ensure alignment of the Bolton North Hill servicing strategy with Regional objectives.

The preferred approach remains unchanged from the previous edition of this report and for all new developments north of Columbia Way, which is to discharge into a proposed sanitary system on Highway 50 and Emil Kolb Parkway. Flows are then pumped via twin 450 mm diameter forcemains to a 525 mm diameter trunk sewer flowing west via gravity on King Street, and ultimately to a 1200 mm diameter trunk sewer flowing south on Humber Station Road.

The trunk sewer on Humber Station Road is being designed to convey flows from all the currently proposed development and future Whitebelt Lands development in the north and west ends of Bolton. The analysis, design, and construction for the furthest downstream portion of this sewer (between Mayfield Rd and Healey Rd) is being undertaken by the Region and its consultants, with Urbantech responsible for the upstream portion north of Healey Rd, including the trunk sewer on King St.

Internal Sanitary Servicing Strategy

The internal servicing strategy aims to convey flows via gravity to trunk sewer connections on Regional Road 50 and Emil Kolb Parkway. While there are significant negative elevation changes from these connection points to the outer edges of development, we understand the Region is proposing the trunk sewer to be deep enough such that no sewage pumping stations will be necessary within the Subject Lands.

All proposed sanitary sewers will be Polyvinyl chloride (PVC) pipes. PVC was chosen over reinforced concrete to reduce the infiltration into the sanitary sewer system. Additionally, unless otherwise stated all sanitary sewers are proposed to be 250 mm diameter with minimum slope to provide self-cleansing velocity, in accordance with the Region of Peel standards.

Interim Sanitary Servicing Strategy

An interim servicing strategy has also been provided for the 4.3 ha parcel in the Option 2 Lands, since it is likely that they would develop well ahead of the timeline for the remaining Option 2 Lands. The upgrades necessary within the existing system south of Columbia Way to accommodate the development are minor and may be proven unnecessary through more detailed modelling. For this solution, the Option 2 Lands would discharge into the existing sanitary sewer on Mount Hope Road south of Columbia Way and eventually discharge to the Bolton Sanitary Pumping Station south of King Street East, where flows would then be pumped via a forcemain into the existing AV trunk sewer.

Table 2 below breaks down the flows from the drainage area being sent to the existing sanitary sewer system south of Columbia Way during the interim condition. A total of 7.0 L/s of sanitary flows from the proposed development will be sent to the existing system south of Columbia Way.

Table 2: Sanitary Flows to the Existing System (Interim Condition)

Drainage Area Number	Area (ha)	Average Daily Flow (L/s)	Peaking Factor	Peak Flow (L/s)	Infiltration Flow (L/s)	Total Sanitary Flow (L/s)
8	4.64	2.8	4.0	6.46	1.2	7.7

Sanitary flows from Drainage Area 8 will be connected to the existing system at Columbia Way and Mount Hope Drive via a 250 mm diameter gravity sewer at 0.5% grade.

Ultimate Sanitary Servicing Strategy

Table 3 below breaks down the flows from each drainage area being sent to Emil Kolb Parkway, and detailed calculations are provided in Appendix B2.

Table 3: Sanitary Flows to Emil Kolb Parkway (Ultimate Condition)

Drainage Area Number	Area (ha)	Average Daily Flow (L/s)	Peaking Factor	Peak Flow (L/s)	Infiltration Flow (L/s)	Total Sanitary Flow (L/s)*
1	41.0	23.6	3.35	43.50	10.7	70.41
2	16.4	9.7	3.66	19.89	4.3	31.4
3	16.2	8.6	3.72	16.33	4.2	27.09
4	22.0	13.30	3.54	26.97	5.7	42.50
5	14.7	10.0	3.60	22.52	3.8	34.25
6	15.4	9.0	3.68	18.48	4.0	29.23
7	24.7	13.0	3.59	23.83	6.4	39.33
8	4.64	2.8	4.00	6.46	1.2	9.95

*30% contingency factor has been applied to address potential scope changes

It should be noted that the design for the Emil Kolb Parkway trunk sewer is to be completed by the Region.

3.4 Proposed Sewage Pumping Station

We understand the Region's preferred approach is to provide one (1) sewage pumping station (SPS) within the Humber River Valley along Emil Kolb Parkway that will service the development of the Option 1 and 2 lands. Crozier has been working collaboratively with the Region to share and review preliminary design information. The ultimate design will be undertaken by the Region based on the results of the Municipal Class EA.

Table 4 shows the total tributary flows to the SPS.

Table 4: Sewage Pumping Station (SPS) Summary

Lands	Area (ha)	Cumulative Area (ha)	Population	Cumulative Population	Dry Weather Flow (L/s)	Peaking Factor	Infiltration & Inflow (L/s)	Total Flow (L/s)
Option 1	150.4	150.4	14415	14415	49.1	2.8	38.9	274.2*
Option 2 (BNHLG)	4.64	155.0	483	14898	50.6	2.8	40.1	180.5
Option 2 (Remaining Whitebelt)	117.7	272.7	8077	22975	77.7	2.6	70.6	271.7
Rounding Out Area C	7.5	280.2	538	23513	79.5	2.6	72.5	277.5
Columbia Square	3.3	283.5	2247	25760	87.0	2.5	73.4	294.4

*30% contingency factor has been applied to address potential scope changes

4.0 Proposed Grading & Roads

4.1 Grading Constraints & Objectives

The following constraints were taken into consideration in preparing the grading plan for the Site:

- The existing elevations of Emil Kolb Parkway, Regional Road 50, and Columbia Way.
- The existing elevations at the property lines of all land parcels.
- Matching the existing drainage patterns within the Site in the post-development condition, and keeping overland flows sent to the Humber River tributaries as close to existing as possible.
- Sending overland flows to the proposed Stormwater Management Pond locations.
- Conveying external flows through or around the Site as necessary.
- Minimum and maximum road slopes per the Town of Caledon standards.

Further to the above constraints, proposed grading for the Site adheres to the following objectives:

- Optimize cut and fill operations and achieve a balanced site.
    - Prevent / minimize the need for retaining walls.
    - Special consideration for the preservation of existing ground elevations in the vicinity of natural features.
- Maintain required cover for municipal infrastructure and utilities including reducing the need for pump stations to convey sanitary flows.
- Accommodate the future lot grading, based on preliminary concepts in accordance with the Town standards.
- Utilize the opportunity for walkout lot types where parcels border areas outside of the development limit.
- Maximize the reuse of topsoil within the Site to limit hauling operations.

4.2 Preliminary Grading Design

The preliminary grading design for the Site is shown on Drawing C704 – General Grading. The grading plans include proposed road centerline elevation and slopes, existing surface contours and the direction of overland flow routes. The following sections outline critical elements of the grading design.

Compatibility with Environmental Policy Areas

As mentioned in the above constraints, proposed elevations adjacent to Environmental Policy Areas will match existing as close as possible to minimize impact to these sensitive areas. Grading adjacent to natural heritage features considers natural heritage buffers/constraints, erosion setbacks, and external overland flow. As the Concept Plan and design progress we will continue to monitor the grading impacts in these areas and minimize the need for retaining walls, sloping, etc.

Lot or park grading adjacent to Environmental Policy Areas will aim to discharge clean stormwater runoff from landscaped areas to the natural features to promote water balance.

Road Centerline Gradients

The preliminary grading design conforms to the Town of Caledon design standard of maintaining a minimum and maximum asphalt gradient of 0.5% and 6.0%, respectively in most areas.

Stormwater Management Ponds

In some instances, the proposed stormwater management ponds (SWMPs) create boundary conditions that the road centerline elevations must adhere to for each drainage area. This occurs where the permanent pool elevations for the SWMPs were fixed based on the respective receiving outlet elevations, which are set at 0.10 m above the 25-year flood elevation of each tributary. These permanent pool elevations then set the road centerline elevation required to facilitate minor and major flow conveyance to the SWMPs.

Grading of roads, lots, and SWMP Blocks considers a buffer required to match proposed or existing boundary elevations. Ponds have been sized such that no retaining walls are required to facilitate grading.

Overland and Emergency Flow Routes

The roads have been graded to ensure positive drainage of major system flows towards the SWMPs within each drainage area. The major system drainage is directed overland to the SWMPs and will operate in the event of a minor system blockage or when the minor system capacity is exceeded (above the 10-year storm event). Each SWMP includes an overland flow path to direct major flow from the roadways into the ponds and an emergency spillway that will operate in the event of an outlet blockage or in the event of a large storm exceeding the pond design capacity. Emergency spillways will be sized for the Regulatory event (larger of the 100-year uncontrolled flow or the Regional uncontrolled flow).

Other Drainage Considerations

If road grading cannot accommodate the conveyance of overland flow to the SWMP, localized capture of the 100-year storm will be contemplated. This will be further determined at detailed design based on road grading and stormwater management requirements for capturing and treating flows.

There are small areas identified across the Site that propose uncontrolled runoff directed to adjacent Environmental Policy Areas. The typical location is split or walkout lots that back onto undevelopable land where the rear-yard drainage cannot be captured. The drainage from these areas is considered inherently 'clean' runoff. At this time, uncontrolled flows from developable areas have been eliminated but are subject to change based on constraints encountered at detailed design. Uncontrolled drainage has been accounted for in the sizing of the SWMPs.

4.3 Earthworks Analysis

An earthworks analysis was performed based on the preliminary grading plan for the Site to determine the locations within the Site that require cut or fill and derive a net earthworks requirement. Matching the pre-development drainage divides helps to match proposed and existing elevations closely across the site, however, the natural topography is quite undulated, and areas of large cut and fill have still resulted.

Drawing C705 – Preliminary Earthworks Map illustrates the cut and fill areas for the Site with colour coded banding and Table 5 below for the resultant volumes.

Table 5: Earthworks Volumes

Land Area	Cut (m3)	Fill (m3)	Net (m3)
Option 1 Lands	1,424,000	1,121,000	303,000 (CUT)
Option 2 Lands	37,000	1,100	35,900 (CUT)

The earthworks were calculated with an average topsoil depth and pre-grade across the entire Site. In absence of a geotechnical report for the Site, the stripped topsoil depth was assumed to be 0.3 m, which is consistent with historically agricultural lands across southern Ontario. The pre-grade depth was assumed to be 0.8 m, which is a weighted average of the typical pre-grade for parks (0.3 m), roads/institutional/commercial (0.6 m), single residential lots (0.8 m), townhomes (1.0 m), and apartments (2.0 m).

At this stage, the earthworks analysis was completed to understand which areas of the Site could be refined in subsequent revisions so that cut and fill and the resultant net volume (import or export) is minimized. As the development is advanced, the proposed grading will be further refined to achieve balance where possible. It is not feasible that each parcel of land will balance, and therefore collaboration between the landowners will likely be required to share cuts and fills.

In preparing subsequent development staging/phasing plans, the cut and fill areas will be further analyzed to coordinate the earthworks program between individual owners.

5.0 Drainage Conditions

5.1 Existing Drainage Conditions

The Site is located within the regulation limits of Toronto and Region Conservation Authority (TRCA) and discharges directly to the Main Humber River and Cold Creek subwatersheds. A network of small tributaries, ditches, culverts, and bridges convey runoff from the Site to the larger watercourses downstream.

A hydrologic model (Visual OTTHYMO v.6.2) was received from TRCA for the Main Humber River and Cold Creek. It is noted that subcatchment boundaries received from TRCA have been further refined by Crozier based on the detailed topographic data obtained from the Land Information Ontario Dataset by the Ministry of Natural Resources and Forestry and existing drainage features (ditches, culverts, bridges, etc.) to clarify existing drainage flow paths, boundaries, and outlets. The existing hydrologic and hydraulic modeling was obtained for the purposes of assessing the Floodplain. Refer to Section 8.0 for further details regarding the floodplain modeling.

Both Option 1 and 2 Lands are located within the Main Humber River Subwatershed. There is a minor watershed divide bisecting Option 1 Lands that directs surface flows east/west towards the Main Humber River tributaries. From this drainage divide, surface flows from both internal and external lands generally drain south to tributaries of the Main Humber River. Lands east of this drainage divide are within Sub Basin 10 of the Humber River and are subject to unit flow controls per TRCA criteira. Option 2 Lands are located north of the intersection at Mount Hope Road and Columbia Way and drain north towards Cold Creek, a tributary of the Main Humber River.

A total of five (5) nearby tributaries of the Main Humber River (labelled as Tributaries 'A-E') are located downstream of the site and receive surface water runoff from both the Option 1 and external lands. One tributary of Cold Creek (labelled as Tributary 'F') is located downstream of the site and receives runoff from the Option 2 lands. To model the existing drainage conditions to each of the minor receiving tributaries, the Humber River catchments have been further discretized into a total of six (6) drainage areas which comprise of both the Option 1 and 2 lands as well as external areas draining to the subject minor tributaries.

Existing drainage, both internal and external to Option 1 Lands, generally flows south towards one of the five (5) nearby tributaries of the Main Humber River (labelled as Tributaries 'A-E'). Option 2 Lands are located northeast of the intersection at Mount Hope Road and Columbia Way and have been delineated as one (1) internal drainage area. Drainage from the Option 2 Lands generally drains north towards Cold Creek, a tributary of the Main Humber River.

Drawing C703 – Pre-Development Storm Drainage Plan illustrates the existing drainage patterns within the Site and surrounding area.

Existing land use within the subject lands is primarily agricultural, with some commercial, forest, meadow, rural residential and wetlands present. Due to these lands being primarily agricultural, there is no storm sewer infrastructure on-site (except for existing storm sewer infrastructure along road right-of-ways). As such, surface runoff from the subject site drains via overland flow towards the receiving tributaries under existing conditions.

Since the Humber River hydrology model received from TRCA is a calibrated model used for regulation, the drainage parameters (including time to peak values, CN* values, initial abstraction values, etc.) have remain unchanged from the original model received. As such, only the catchment areas have been updated based on the local refinement of catchments described above.

According to the Ontario Soil Survey mapping for Peel County, there are mainly four soil types present on Site: King Clay Loam, Bottom Land, Monoghan Clay Loam and Pontypool Sandy Loam. King Clay Loam is the dominant soil type found across the majority of the Site. Figure SOIL-1 (provided in Appendix E – Figures) illustrates the predominant soil types across the Site and surrounding area.

Existing land use on Site is primarily agricultural, with some commercial, forest, meadow, rural residential and wetlands present. Refer to Figure LAND-1, provided in Appendix E – Figures, which depicts the existing land use conditions.

Table 6 below provides a summary of the existing drainage catchments and catchment areas used for modeling. Refer to Appendix C2 for a detailed summary of the calculated hydrologic parameters.

Table 6: Pre-Development Catchments – Draining towards Site

Catchment ID	**Drainage Area** (ha)	**Initial Abstraction, IA** (mm)	**Modified Curve Number, CN*** (2-5yr)	**Time to Peak** (2-5yr) (hours)	**Modified Curve Number, CN*** (10-500yr)	**Time to Peak** (10-500yr) (hours
TRIB A	37.79	10	69	3.69	90	2.46
TRIB B	36.99	10	69	3.69	90	2.46
TRIB C	109.13	10	70	4.04	92	2.69
TRIB D	168.1810	70	4.04	92	2.69	
TRIB E	151.92	10	71	4.2	89	2.1
TRIB COLD	67.64	10	70	1.78	88	0.89
Total	**571.66**	**-**	**-**	**-**	**-**	**-**

The catchments listed in Table 6 represent those that contribute drainage towards the Site and receiving minor tributaries of the Humber River and Cold Creek.

5.2 Proposed Drainage Conditions

Existing drainage patterns will be preserved as much as possible under the proposed development conditions, with topography generally sloping southwards toward the tributaries of the West Humber River. The drainage divide between sub-basin 10 of the Main Humber River and areas of the Main Humber River Watershed will be generally maintained by the proposed grading and servicing concept. Minor and major system flows will be directed to the SWM facilities via storm sewers and road grading.

The Option 1 lands have been discretized into a total of twelve (12) internal drainage catchments under post-development conditions. The Option 2 lands have been delineated as a single drainage catchment. Proposed land uses were assigned based on the Concept Plan (Bousfields, February 11, 2026), comprising of low to high density residential, commercial, schools, parks, open space, and stormwater management ponds. Per the concept plan land uses, percent impervious values for each catchment were calculated using the area-weighted method and values from Table 2-2 of the 2018 Humber River Hydrology Update (Civica, April 2018). For the purposes of the stormwater management analysis, mixed-use, commercial, and/or high-density land use areas proposed on the Concept Plan were assigned TIMP and XIMP values of 70% and 60% respectively, consistent with medium-density (townhouses). In the event that these areas will be developed at a higher density, on-site controls will be implemented at Draft Plan stage to reduce peak flows to be equivalent to the runoff associated with a 70% TIMP value. Detailed percent impervious value calculations and Table 2-2 are provided in Appendix C2.

Refer to Drawing C706 – Preliminary Post-Development Storm Drainage Plan, depicting the proposed drainage boundaries and conceptual SWM pond locations. A summary of the proposed Site drainage areas, composite runoff coefficients, and corresponding percent impervious values are provided in Table 7 below.

Table 7: Post-Development Drainage Area Summary

Catchment ID	**Drainage Area (ha)**	**Total Impervious** (%)	**Directly Connected Impervious** (%)	**Receiving Tributary**
201	8.50	58%	48%	Tributary A
202	7.34	55%	46%	Tributary B
203	8.71	62%	52%	Tributary B
204	18.33	61%	52%	Tributary C
205	17.71	68%	59%	Tributary D
206	5.83	63%	53%	Tributary C
207	17.86	64%	57%	Tributary D
208	34.45	68%	58%	Tributary D
209A	22.25	62%	53%	Tributary E
209B	1.33	60%	50%	Tributary E
210	17.67	62%	52%	Tributary E
212	3.51	10%	0%	Tributary E
213	5.03	70%	60%	Tributary F (Cold Creek)
Total Site	**168.52**	**-**	**-**	**-**

Changes to the quantity and quality of stormwater runoff resulting from the proposed development will be mitigated by a comprehensive stormwater management (SWM) strategy. This strategy includes a total of eleven (11) SWM wet pond facilities to provide stormwater management for the Site, including quantity, quality and erosion control of post-development runoff.

External Drainage

Runoff from external areas, which previously drained through the Option 1 lands under existing conditions will not be altered by the proposed development. Positive drainage for these external catchments will be maintained by the proposed grading and servicing concept to route these flows through or around the development area.

Flows from external drainage area EXT3 and the western portion of external drainage area EXT4 will be captured into a storm pipe, conveyed through the Site and will outlet to the upper reaches of Tributary C. Flows from the 1.87 ha woodlot area, external drainage area EXT5, and the eastern portion of external drainage area EXT4 will be captured into a storm sewer pipe, conveyed through the Site and will outlet to the existing concrete culvert at Emil Kolb Parkway (adjacent to the outlet for SWM Pond 3). These storm sewer pipes were sized to convey the 100-year storm event. Flows from extreme storm events that exceed the conveyance pipe capacity (i.e., storm events greater than the 100-year storm) will drain overland through the Site via the proposed road network and overland flow paths.

The remaining external drainage areas will be conveyed by swales around the proposed development. Runoff from these areas will be directed to the adjacent tributaries, thereby maintaining existing drainage patterns/outlets. Table 8 below provides a summary of the external catchment areas draining through and in the vicinity of the Site.

Table 8: Post-Development Drainage Area Summary (External)

Catchment ID	Drainage Area (ha)	Receiving Tributary
EXT1	28.75	Tributary A
EXT2	25.18	Tributary B
EXT3A	36.40	Tributary C
EXT3B	47.61	Tributary C
EXT4	97.24	Tributary D
EXT5	108.20	Tributary E
EXT6	62.60	Tributary F (Cold Creek)
Total External	**405.97**	**-**

Minor System Drainage

Storm sewers internal to the development will be designed to convey post-development minor system flows to the stormwater management ponds. The Town of Caledon (Development Standards Manual, 2019) requires that storm sewers without foundation drains be designed to convey the 5-year storm event at a minimum. For systems where foundation drains will be connected to the storm sewers, the Town requires the system to be designed to accommodate a 10-year storm event. To be conservative at this stage in the development, the storm sewers for the Site were designed to convey the 10-year storm peak flows, which is required when foundation drains are used.

Sizing of these storm sewers was completed using the Rational Method and using the Town's standard Intensity-Duration-Frequency Rainfall Curves (Town Standard Drawing No. 103). Refer to Appendix C1 for the storm sewer design sheet.

Major System Drainage

Major system stormwater runoff that exceeds the capacity of the storm sewers will drain overland via the proposed road right-of-ways (ROWs) to the respective stormwater management ponds. Per development standards, overland flows shall be contained within the ROWs and shall not cross private property. Depths of flooding permitted on streets and at intersections for all classes of road during the 100-year design storm event are as follows:

- The product of depth of water (m) at the gutter times the velocity of flow (m/s) shall not exceed 0.65 m^2/s.
- The depth of water should not exceed 0.30m

6.0 Stormwater Management

6.1 Stormwater Design Criteria & Targets

Stormwater management design for the Site is governed by criteria set out in the following documents:

- Development Standards Manual (Town of Caledon, 2019)
- Stormwater Management Criteria (Toronto and Region Conservation Authority (TRCA), 2012)
- Stormwater Management Planning and Design Manual (Ministry of Environment, (MOE), 2003)

A Subwatershed Study (being prepared under separate cover) will ultimately be the overarching document for directing more detailed stormwater management criteria for the Site.

The applicable stormwater management criteria are summarized below.

Quality Control

The Development Standards Manual (Town of Caledon, 2019) states that all new developments must adhere to the quality control requirements in the Stormwater Management and Planning & Design Manual (MOE, 2003). Additionally, all new stormwater management facilities shall be designed to meet Enhanced (Level 1) protection per the MOE SWMPD manual, requiring 80% removal of Total Suspended Solids (TSS) from 90% of the total annual runoff volume.

Erosion Control

The Development Standards Manual (Town of Caledon, 2019) and MOE SWMPD manual (MOECC, 2003) requires that developments with proposed stormwater management ponds must provide extended detention control of the runoff volume generated from a 4-hour, 25 mm storm event over a period of 48 hours.

Since the Site is considered greenfield development, a detailed erosion analysis based on a geomorphic assessment will need to be completed to determine if more stringent erosion controls are required to mitigate erosion within the receiving tributaries.

Quantity Control

The Site is located within the Humber River Watershed, therefore post-development flow rates must be controlled to the prescribed TRCA unit flow rates for the Humber River. TRCA Stormwater Management Criteria (2012) states that the main branches of the Lower, Main, East, West and Upper Humber River watershed do not require any quantity control measures. Lands that do not discharge to these main branches are required to adhere to the quantity control flow rates for the Humber River Watershed.

Most of the Site drains to the main branch of the West Humber River, therefore quantity controls are not required for these areas. However, the runoff from these areas is conveyed to the West Humber River through small tributaries. The impacts of releasing uncontrolled flows to these tributaries must be evaluated to determine the level of quantity control required. Refer to Section X.X for downstream analysis results.

Areas of the Site located within sub-basin 10 (generally located east of Highway 50) of the Humber River watershed must be designed to meet targets calculated using the Humber River Unit Flow Relationships for sub-basin 10. The Unit Flow Relationship Equations for the 2 through 100-year storm events within sub-basin 10 (Stormwater Management Criteria, TRCA 2012) are presented in Table 9 below.

Table 9: Humber River Watershed - Sub-Basin 10 Unit Flow Relationships

Return Period	Equation C: Sub-Basin 10
2-Year	Q = 3.142 - 0.233 * ln(A)
5-Year	Q = 5.557 - 0.427 * ln(A)
10-Year	Q = 7.443 - 0.578 * ln(A)
25-Year	Q = 9.838 - 0.757 * ln(A)
50-Year	Q = 11.920 - 0.921 * ln(A)
100-Year	Q = 14.140 - 1.096 * ln(A)

In accordance with Table 3-1 of the Stormwater Management Criteria (TRCA, 2012) calculation of return period peak flows within the Humber River Watershed shall be based on the 6-hour and 12-hour AES storm distributions. The storms used by TRCA for the Humber River model include the 6-hour, 12-hour, and 24-hour AES storm distributions and have therefore been used for this analysis.

Flood Control

As the Bolton North Hill Lands are upstream of a flood vulnerable area additional modeling is required to assess the impact of the development on dowsream land owners. Modeling has been completed for the 350 year, 500 year and regional storm to identify if additional quantity control is required to mitigate downstream impacts. Please refer to Section XX for details of this analysis.

Water Balance

Based on TRCA mapping, the Site is located within a Significant Groundwater Recharge Area and a High Groundwater Recharge Area (Stormwater Management Criteria, TRCA 2012). Due to these vulnerable areas, the Site will be required to maintain pre-development groundwater recharge rates and appropriate distribution, ensuring the protection of related hydrologic and ecologic functions. Further analysis of the water balance requirements will be conducted following the completion of a hydrogeological report.

6.2 Stormwater Management Strategy

The proposed grading and servicing strategy for the Site will generally maintain existing drainage patterns as well as the sub-watershed divide between the two sub-basins of the Humber River Watershed; sub-basin 10, which is subject to prescribed water quantity and quality controls, and a sub-basin that does not require quantity control.

Runoff from most of the development lands will be directed to on-site stormwater management (SWM) wet ponds designed to provide quantity, quality, and erosion control. A total of eleven (11) end-of-pipe stormwater management wet pond facilities are proposed to treat runoff from post-development drainage areas as illustrated on Drawing C706 – Preliminary Post-Development Storm Drainage Plan.

A total of eight (8) ponds will be located within an area of the Main Humber River Watershed, which does not require prescribed quantity controls per TRCA criteria. Although the Main Humber does not require quantity control, runoff from these ponds is conveyed through small drainage features to the Main Humber. These small drainage features will be evaluated as part of the Local Subwatershed Study to determine if they can safely convey uncontrolled flows from the development. The ponds located within the Main Humber Watershed are therefore designed to provide post to pre quantity control as well as quality and erosion control, providing conservative pond block sizing for this FSR. The remaining three (3) SWM ponds are located within sub-basin 10 of the Humber River which requires quantity control to meet unit flow rates specific to this subbasin and have been designed accordingly to provide quantity control as well as quality and erosion control.

Conceptual locations for the SWM wet pond facilities are shown on Drawing C706 – Preliminary Post-Development Storm Drainage Plan and have been selected according to the following:

- Located within natural low points, to minimize earthworks and maintain existing drainage patterns.
- Proximity to an outlet, such as an existing receiving tributary, ditch, or storm sewer (there are no existing storm sewers in the vicinity of the Subject Lands, however they may be proposed if urbanization of the existing arterial roads is considered).
- Minimize storm sewer infrastructure size and avoid potential service crossing conflicts (should storm sewers be proposed as a receiver).
- Optimize land use by maximizing developable tableland and serviceable areas.

In addition to the parameters listed above, options to expand SWM ponds into adjacent Greenbelt areas can be explored throughout further refinement of the design in order to maximize the developable area. The Greenbelt Plan 2017 identifies where urbanization should be limited in order to provide permanent protection to the agricultural land base and the ecological and hydrological features and functions occurring on the landscape within the Greater Golden Horseshoe. Existing, expanded, and new infrastructure is permitted within the Greenbelt Plan Area provided it serves significant growth and economic development expected in southern Ontario beyond the Greenbelt (Planning Act, Section 4.3.1.1 (b)). Stormwater management infrastructure is permitted within the Greenbelt Plan Area, however SWM facilities are prohibited in key natural heritage features, key hydrologic features, and their associated vegetation protection zones (Planning Act, Section 4.2.3.3).

Of note, SWM ponds 2/3 are two hydraulically connected wet pond cells, which will function as one stormwater management facility, which outlets to Tributary B. SWM pond 8 will include a flow splitter outlet to maintain flows to both Tributaries C and D under post-development conditions.

SWM Pond Grading & Design Considerations

This report outlines the preliminary quantity, quality and erosion control storage volumes required by each of the conceptual stormwater management ponds. Stormwater management criteria will be refined through the completion of the Local Subwatershed Study by Crozier (under separate cover). Block sizing for each of these stormwater management facilities will be quantified through progression of the Tertiary Plan and landowner's individual Draft Plans.

Each of the stormwater management (SWM) ponds shall be designed with consideration for grading, public safety, future maintenance access, and safe conveyance of emergency flows. A summary of the criteria to be taken into consideration for design of the stormwater management ponds is as follows:

- Site grading to provide safe overland conveyance of flows into the stormwater management facilities.
- Ponds graded with maximum side slopes of 4:1 from the bottom of the pond to 0.5 m below the normal water level.
- Maximum side slopes of 7:1 applied above the 4:1 sloping to the top of pond berm/ access road, with maximum 4:1 sloping from the top of berm to match into the pond block perimeter grades outside the pond.
- Permanent pool volume sized to achieve an Enhanced Level of quality protection with a depth of 1.5 m in the forebay and the main cell.
- Permanent pool elevation above the 25-year water level of the receiving watercourse.
- Extended detention storage and drawdown of the 25-mm rainfall event over a minimum period of 48 hours, at a maximum depth of 1.0 m per MECP guidelines.
- Active storage ponds located within sub-Basin 10, sized according to TRCA Unitary Flow Rate criteria for the Humber River Watershed (sub-Basin 10) with a maximum active storage depth of 2.0 m.
- A minimum 0.30 m of freeboard provided above the maximum (100-year) water level to the top of pond berm.
- Emergency spillways sized to convey the unattenuated Regulatory storm event runoff, which is defined as the greater of the Regional or 100-year unattenuated flow.
- A 5.0 m wide maintenance access road located with maximum longitudinal slopes of 8.0% and a maximum cross-fall slope of 2.0%. The maintenance access road will facilitate access to the inlet structure, forebay, and outlet structure for maintenance.

Site-Specific Stormwater Management Considerations

The stormwater management strategy for the Bolton North Hill development relies mainly on communal stormwater management ponds to meet quantity (if needed), quality, and erosion control requirements. As noted within **Section 5.2**, the Mixed-use, commercial and/or high-density blocks have been modelled with a percent impervious value consistent with medium-density (townhouse) development for the purposes of this report and communal SWM pond sizing. In the event that these areas will be developed at a higher density than 70% imperviousness, on-site stormwater management controls shall be explored to mitigate increased peak flows directed to the downstream communal ponds.

6.3 Quality Control

As mentioned in Section 6.1, all new developments (including all new stormwater management facilities) are required to provide Enhanced (Level 1) Protection (80% removal of TSS) on an annual loading basis. This quality control criteria will be achieved by the permanent pool of each of the proposed stormwater management (SWM) wet pond facilities.

Permanent pool volumes required to achieve Enhanced (Level 1) Protection were calculated based on the drainage area and associated imperviousness (calculated per the Concept Plan) tributary to each pond, in accordance with Table 3.2 of the Planning and Design Manual (Ministry of Environment, Conservation and Parks (MECP), 2003). Detailed permanent pool volume sizing calculations are provided in Appendix C2 and the results are summarized in Table 10 below.

Table 10: SWM Pond Permanent Pool Volume Summary

SWM Facility	Contributing Drainage Area (ha)	Percent Impervious of Drainage Area (%)	Required Permanent Pool Volume[1] (m^3)
1	7.34	55%	1,101
2/3	8.71	62%	1,455
4	18.33	61%	3,020
5	17.71	68%	3,202
6	23.59	62%	3,942
7	34.45	68%	6,228
8	23.69	63%	4,015
9	17.67	62%	2,952
10	8.50	58%	1,338
11	5.03	70%	931

[1] Required storage volumes interpolated from Table 3.2 of SWM Planning and Design Manual (MECP, 2003) are based on upstream impervious area, less 40 m^3/ha required for extended detention.

Each of the proposed SWM ponds will be graded to provide sufficient volume to meet or exceed the permanent pool volume required to achieve Enhanced Protection quality criteria, as shown in Table 10 above.

Permanent pool water levels within each of the proposed SWM pond facilities will be set based on the 25-year water level of the receiving tributary. Per TRCA criteria, the elevation between SWM pond outfalls and the receiving channel shall be minimized, by placing the outfall as low down on the slope as possible, while still above the 25-year floodline. From these outfall elevations, an outlet pipe slope was assumed to establish the minimum permanent pool water level within each pond; the pipe slope varies between ponds and was iterated so that the top of pond matches the minimum grade in the pond block. SWM ponds within catchments 203 and 207 outlet to an existing culvert or roadside ditch. Outfall inverts for these SWM pond facilities were based on the ditch or culvert invert.

Forebay Sizing

At the Draft Plan stage, forebays within each of the SWM wet ponds will be designed to achieve required dispersion and settling lengths in accordance with water quality sizing objectives in Section 4.6.2. of the SWM Planning and Design Manual (MECP, 2003). Forebay berms will be used to achieve these criteria as well as to meet a minimum forebay length to width ratio of 2:1, as recommended by the MECP.

6.4 Erosion Control

At a minimum, a 40 m^3/ha erosion control volume is required (MECP, 2003). Per Town and TRCA criteria, erosion controls for each SWM wet pond are required to provide extended detention and drawdown of a 25 mm rainfall event over a minimum 48-hour period.

A VO model was prepared for the post-development drainage conditions using the 25 mm 4-hour Chicago design storm to determine the resulting runoff volumes directed to each stormwater management facility. Detailed output from the VO model is provided in Appendix C3**.** Erosion control volume requirements are summarized in Table 11 below.

Table 11: Erosion Control Volume Requirements

SWM Facility	Contributing Drainage Area (ha)	Percent Impervious of Drainage Area (%)	Required MECP Erosion Control Volume[1] (m^3)	Required 25mm Event Erosion Control Volume[2] (m^3)
1	7.34	55%	294	1,021
2/3	8.71	62%	348	1,313
4	18.33	61%	733	2,749
5	17.71	68%	709	2,883
6	23.59	62%	943	3,574
7	34.45	68%	1,378	5,573
8	23.69	63%	948	3,702
9	17.67	62%	707	2,664
10	8.50	58%	340	1,278
11	5.03	70%	201	832

[1] MECP Erosion Control Volume required is 40 m^3/ha, obtained from the MOE SWM Planning and Design Manual (2003)
[2] 25mm Event Erosion Control Volume obtained from the Visual Otthymo model

6.5 Downstream Analysis

The majority of the proposed development area is located within the Main Humber watershed, which does not require quantity controls as prescribed within TRCA Stormwater Management Criteria (2012). Since runoff from the proposed development is conveyed through small drainage features to the Main Humber, a downstream analysis has been completed to assess the impact of increased runoff to these smaller features. Additionally, there is a Flood Vulnerable Area (FVA) currently identified in the downtown area of Bolton. As such, the downstream analysis also examined any potential increases in peak flows and flood risk associated with the main Humber River and this associated Flood Vulnerable Area.

The Subwatershed study being prepared under separate cover will also include a downstream analysis of the smaller drainage features downstream of the site as well as the Flood Vulnerable Area. Results presented in the Subwatershed Study will ultimately be the overarching document for directing more detailed stormwater management criteria for the Site.

The Humber River hydrology model was run for both the existing and proposed (uncontrolled) development conditions. Peak flow rates resulting from each model were then compared at specified flow nodes to understand the impact of the development on peak flow conditions within the downstream tributaries and Main Humber River. A comparison of peak flows is summarized in Table 12 below.

Table 12: Downstream Analysis – Comparison of Peak Flows

StFlow Node (NHYD #)	Design Storm Event	Existing Condition Peak Flow (m^3/s)	Proposed (Uncontrolled) Condition Peak Flow (m^3/s)	Difference (%)
1682	2-Year 6-Hour AES	4.71	4.70	0%
	5-Year 6-Hour AES	9.12	9.11	0%
	10-Year 6-Hour AES	15.74	15.49	-2%
	25-Year 6-Hour AES	22.55	22.26	-1%
	50-Year 6-Hour AES	29.55	29.55	0%
	100-Year 6-Hour AES	37.40	37.39	0%
	350-Year 12-Hour AES	88.22	88.11	0%
	500-Year 12-Hour AES	98.70	98.56	0%
	Hurricane Hazel	431.31	430.47	0%

1471	2-Year 6-Hour AES	4.77	5.29	11%
	5-Year 6-Hour AES	9.31	9.24	-1%
	10-Year 6-Hour AES	19.48	18.11	-7%
	25-Year 6-Hour AES	27.39	25.61	-7%
	50-Year 6-Hour AES	34.60	32.60	-6%
	100-Year 6-Hour AES	42.84	40.57	-5%
	350-Year 12-Hour AES	89.69	89.16	-1%
	500-Year 12-Hour AES	100.69	99.85	-1%
	Hurricane Hazel	441.94	435.94	-1%
2048	2-Year 6-Hour AES	7.05	8.98	27%
	5-Year 6-Hour AES	9.78	12.80	31%
	10-Year 6-Hour AES	20.26	18.99	-6%
	25-Year 6-Hour AES	28.28	26.65	-6%
	50-Year 6-Hour AES	35.90	33.89	-6%
	100-Year 6-Hour AES	44.20	41.95	-5%
	350-Year 12-Hour AES	90.83	90.13	-1%
	500-Year 12-Hour AES	101.78	100.81	-1%
	Hurricane Hazel	442.31	437.97	-1%
2043	2-Year 6-Hour AES	16.51	17.50	6%
	5-Year 6-Hour AES	23.44	25.08	7%
	10-Year 6-Hour AES	35.32	37.39	6%
	25-Year 6-Hour AES	43.29	45.93	6%
	50-Year 6-Hour AES	50.17	53.29	6%
	100-Year 6-Hour AES	56.49	60.04	6%
	350-Year 12-Hour AES	90.84	90.16	-1%
	500-Year 12-Hour AES	101.48	100.75	-1%
	Hurricane Hazel	439.81	436.00	-1%

2128	2-Year 6-Hour AES	5.82	7.80	34%
	5-Year 6-Hour AES	8.19	11.14	36%
	10-Year 6-Hour AES	12.21	15.74	29%
	25-Year 6-Hour AES	15.05	19.57	30%
	50-Year 6-Hour AES	17.66	22.87	29%
	100-Year 6-Hour AES	19.95	25.84	30%
	350-Year 12-Hour AES	17.05	22.00	29%
	500-Year 12-Hour AES	18.09	23.38	29%
	Hurricane Hazel	22.36	26.91	20%
7520	2-Year 6-Hour AES	3.10	4.26	37%
	5-Year 6-Hour AES	4.44	6.33	43%
	10-Year 6-Hour AES	7.86	10.38	32%
	25-Year 6-Hour AES	10.00	13.37	34%
	50-Year 6-Hour AES	11.79	15.80	34%
	100-Year 6-Hour AES	13.44	17.85	33%
	350-Year 12-Hour AES	15.88	20.18	27%
	500-Year 12-Hour AES	16.96	21.58	27%
	Hurricane Hazel	24.63	28.65	16%
894	2-Year 6-Hour AES	0.51	0.50	-2%
	5-Year 6-Hour AES	0.99	0.97	-1%
	10-Year 6-Hour AES	5.73	5.67	-1%
	25-Year 6-Hour AES	8.38	8.33	-1%
	50-Year 6-Hour AES	10.96	10.90	-1%
	100-Year 6-Hour AES	14.22	14.18	0%
	350-Year 12-Hour AES	32.83	32.77	0%
	500-Year 12-Hour AES	37.33	37.27	0%
	Hurricane Hazel	157.38	157.16	0%

1850	2-Year 6-Hour AES	3.34	4.55	36%
	5-Year 6-Hour AES	4.93	6.81	38%
	10-Year 6-Hour AES	22.01	21.75	-1%
	25-Year 6-Hour AES	30.77	30.35	-1%
	50-Year 6-Hour AES	38.40	37.86	-1%
	100-Year 6-Hour AES	46.62	45.97	-1%
	350-Year 12-Hour AES	87.25	86.44	-1%
	500-Year 12-Hour AES	96.47	95.58	-1%
	Hurricane Hazel	311.36	309.43	-1%
1794	2-Year 6-Hour AES	17.38	19.60	13%
	5-Year 6-Hour AES	25.20	29.12	16%
	10-Year 6-Hour AES	43.08	47.93	11%
	25-Year 6-Hour AES	56.54	60.43	7%
	50-Year 6-Hour AES	71.12	70.37	-1%
	100-Year 6-Hour AES	88.05	85.05	-3%
	350-Year 12-Hour AES	168.53	164.83	-2%
	500-Year 12-Hour AES	186.93	182.74	-2%
	Hurricane Hazel	690.00	681.76	-1%

As can be seen from the table above, peak flows increases are generally observed as a result of the development. Within the Main Humber peak flows during the smaller 2-5 -year storm events present the largest change in peak flows, whereas the larger storms show decreases in peak flows. During the Regional (Hurricane Hazel) storm event, only a 1% increase in peak flow rates is observed through the main reach of the Humber River.

The portion of the development area located within sub-basin 10 which outlets to Tributary E and Cold Creek, is required to meet prescribed unit flow rates per TRCA criteria for events up to and including the 100-year design storm event. Results of the downstream analysis showed significant increases in peak flows during the 350 yr, 500yr and Regional (Hurricane Hazel) storm event, therefore peak flow controls for these larger storm events will be implemented for SWM ponds which outlet to Tributary E and Cold Creek. It was also noted that for Tributary E and Cold Creek the unit flow rates for the 2 and 5 year storm did not fully control the peak flows to pre-development levels. Due to the erosive potential of the downstream reaches a more conservative control target has been established for these facilities. These target flow rates will be reassessed based on the results of the downstream erosion assessment.

6.6 Quantity Control

SWM Pond Quantity Control Criteria Summary

Based on the prescribed TRCA unit flow rates (where applicable) and the results of the downstream analysis presented in Section 6.5, Table 13 summarizes the quantity control criteria applicable to each individual SWM facility.

Table 13: SWM Facility Quantity Control Requirement Summary

SWM Facility	Humber River Sub-basin	Receiving Tributary	Design Storm Control Required	Flood Control Required
1	Main Humber	Tributary B	2 – 5 year and 100 year Post to Pre	350 year, 500 year and Regional Post to Pre
2/3	Main Humber	Tributary B	2 – 5 year and 100 year Post to Pre	350 year, 500 year and Regional Post to Pre
4	Main Humber	Tributary C	2 – 5 year and 100 year Post to Pre	No
5	Main Humber	Tributary D	2 – 5 year and 100 year Post to Pre	350 year, 500 year and Regional Post to Pre
6	Sub-basin 10	Tributary E	Unitary Flow Controls[1]	350 year, 500 year, Hazel
7	Main Humber	Tributary D	2 – 5 year and 100 year Post to Pre	350 year, 500 year and Regional Post to Pre
8	Main Humber	Tributary C/D	2 – 5 year and 100 year Post to Pre	350 year, 500 year and Regional Post to Pre
9	Sub-basin 10	Tributary E	Unitary Flow Controls[1]	350 year, 500 year, Hazel
10	Main Humber	Tributary A	2-5 year post to pre.	No
11	Sub-basin 10	Tributary F (Cold Creek)	Unitary Flow Controls[1]	350 year, 500 year, Hazel

Note 1 - Unitary Flow Controls for 2-5 year adjusted per table 14 below, controls for the 10-100 year) per TRCA criteria (sub-basin 10)

The Subwatershed study being prepared under separate cover will ultimately be the overarching document for directing more detailed stormwater management criteria for the Site.

Sub-Basin 10 SWM Ponds

As previously noted, the eastern portion of the development is located within sub-basin 10 of the Humber River Watershed, where control of post-development peak flows is required to meet the Unit Flow Relationships for the Humber River (sub-basin 10), as previously identified in Table 9 of this report. See Table 14 below for the target flows to each pond based on pre-development areas to the pond outlets.

Table 14: Target Flows for SWM Ponds in Sub-Basin 10

Storm Event	SWMP 6		SWMP 9		SWMP 11	
	Pre-Dev. Area to Pond (ha)	Target Flow Rate (m^3/s)	Pre-Dev. Area to Pond (ha)	Target Flow Rate (m^3/s)	Pre-Dev. Area to Pond (ha)	Target Flow Rate (m^3/s)
2-Year	22.95	23.9	16.13	16.7	5.03	10.9
5-Year		52.2		37.1		21.5
10-Year		129.3		94.1		32.7
25-Year		171.4		124.7		43.3
50-Year		207.3		151.0		52.5
100-Year		245.7		178.9		62.2
500-Year		-		-		-
350-Year		-		-		-
Regional		2.125		0.910		-

A VO model was prepared for the post-development drainage conditions using the 6-hour, 12-hour, and 24-hour AES design storms. Target flow rates for each pond were calculated using the Unit Flow Relationships for sub-basin 10 based on the total drainage area contributing to each pond. Target flows were then coded into VO as a route reservoir command to determine the resultant active storage volumes required. Detailed Humber River target flow calculations are provided in Appendix C2 and Visual Otthymo model outputs for the SWM Pond Designs are provided in Appendix C3.

Results of the modeling determined that the 12-hour and 24-hour AES storms produced the larger volume requirements compared to the 6-hour AES storm. As such, ponds shall be sized to meet the larger volume requirement for the ponds to achieve Unit Flow criteria under both 6-hour, 12-hour, and 24-hour AES events for all storms up to and including the 100-year event.

Since SWM ponds 6 and 9 both outlet to Tributary E, quantity control of the Regional (Hurricane Hazel) storm event will also be provided to mitigate increases in peak flows downstream. As such, a sizing exercise has also been completed in VO to determine the required volume to mitigate increases in peak flows associated with the Regional storm event.

Tables 15, 16, and 17 summarize the resultant active storage volumes for each SWM pond facility located within sub-basin 10 of the Humber River Watershed.

Table 15: SWM Pond 6 Active Storage Volume Summary

Storm Event	Post-Development Contributing Area (ha)	Percent Imperviousness (%)	Target Flow[1] Rate (m^3/s)	Required Active Storage Volume (m^3)
2-Year	22.92	62%	23.9	6,300
5-Year			52.2	8,000
10-Year			129.3	9,300
25-Year			171.4	10,900
50-Year			207.3	12,100
100-Year			245.7	13,300
350-Year			-	-
500-Year			-	-
Regional			1.215	24,500

[1] Target flow rates calculated based on contributing area to SWM pond and Unit Flow Rate Equations for Sub-Basin 10 of the Humber River Watershed (TRCA, 2012).

Table 16: SWM Pond 9 Active Storage Volume Summary

Storm Event	Post-Development Contributing Area (ha)	Percent Imperviousness (%)	Target Flow[1] Rate (m^3/s)	Required Active Storage Volume (m^3)
2-Year	17.67	62%	16.7	4,800
5-Year			37.1	6,100
10-Year			94.1	7,000
25-Year			124.7	8,200
50-Year			151.0	9,100
100-Year			178.9	10,100
350-Year			-	-
500-Year			-	-
Regional			910.0	18,100

[1] Target flow rates calculated based on contributing area to SWM pond and Unit Flow Rate Equations for Sub-Basin 10 of the Humber River Watershed (TRCA, 2012).

Table 17: SWM Pond 11 Active Storage Volume Summary

Storm Event	Post-Development Contributing Area (ha)	Percent Imperviousness (%)	Target Flow[1] Rate (m^3/s)	Required Active Storage Volume (m^3)
2-Year	5.03	70%	10.9	1,400
5-Year			21.5	1,800
10-Year			32.7	2,100
25-Year			43.3	2,400
50-Year			52.5	2,650
100-Year			62.2	2,900

[1] Target flow rates calculated based on contributing area to SWM pond and Unit Flow Rate Equations for Sub-Basin 10 of the Humber River Watershed (TRCA, 2012).

Main Humber River SWM Ponds

Target flows for the SWM facilities within the Main Humber River watershed were coded into VO as route reservoir commands to determine the resultant active storage volumes required. Detailed Humber River target flow calculations are provided in Appendix C2 and Visual Otthymo model outputs for the SWM Pond Designs are provided in Appendix C3.

Results of the modeling determined that, overall, the 24-hour AES storm produced the larger volume requirement compared to the 12-hour and 6-hour AES storm. As such, ponds shall be sized to meet the larger volume requirement for the ponds to achieve quantity control under both 6-hour and 12- and 24-hour AES events for all storms up to and including the 100-year event. Tables 18-24 summarize the resultant active storage volumes for each SWM pond facility discharging to the Main Humber River. Table 25 summarizes the total active storage required for each SWM pond facility.

Table 18: SWM Pond 1 Active Storage Volume Summary

Storm Event	Post-Development Contributing Area (ha)	Percent Imperviousness (%)	Target Flow Rate (m^3/s)	Required Active Storage Volume (m^3)
2-Year	7.34	55%	0.005	2,010
5-Year			0.012	2,700
10-Year			-	-
25-Year			-	-
50-Year			-	-
100-Year			0.149	3,550
350-Year	-	-	0.213	4,490
500-Year	-	-	0.227	4,730
Hazel	-	-	0.405	7,060

Table 19: SWM Pond 2/3 Active Storage Volume Summary

Storm Event	Post-Development Contributing Area (ha)	Percent Imperviousness (%)	Target Flow Rate (m^3/s)	Required Active Storage Volume (m^3)
2-Year	8.71	62%	0.007	2,600
5-Year			0.015	3,400
10-Year			-	-
25-Year			-	-
50-Year			-	-
100-Year			0.169	4,440
350-Year	-	-	0.251	5,580
500-Year	-	-	0.269	5,870
Hazel	-	-	0.482	8,500

Table 20: SWM Pond 4 Active Storage Volume Summary

Storm Event	Post-Development Contributing Area (ha)	Percent Imperviousness (%)	Target Flow Rate (m^3/s)	Required Active Storage Volume (m^3)
2-Year	18.33	61%	0.020	5,300
5-Year			0.042	7,000
10-Year			-	-
25-Year			-	-
50-Year			-	-
100-Year			-	-

Table 21: SWM Pond 5 Active Storage Volume Summary

Storm Event	Post-Development Contributing Area (ha)	Percent Imperviousness (%)	Target Flow Rate (m^3/s)	Required Active Storage Volume (m^3)
2-Year	17.71	68%	0.019	5,500
5-Year			0.041	7,000
10-Year			-	-
25-Year			-	-
50-Year			-	-
100-Year			0.506	8,840
350-Year	-	-	0.715	10,680
500-Year	-	-	0.766	11,190
Hazel	-	-	1.321	13,277

Table 22: SWM Pond 7 Active Storage Volume Summary

Storm Event	Post-Development Contributing Area (ha)	Percent Imperviousness (%)	Target Flow Rate (m^3/s)	Required Active Storage Volume (m^3)
2-Year	34.45	68%	0.038	10,500
5-Year			0.080	13,500
10-Year			-	-
25-Year			-	-
50-Year			-	-
100-Year			0.979	17,150
Hazel	-	-	2.567	25,640

Table 23: SWM Pond 8 Active Storage Volume Summary

Storm Event	Post-Development Contributing Area (ha)	Percent Imperviousness (%)	Target Flow Rate (m^3/s)	Required Active Storage Volume (m^3)
2-Year	23.69	63%	0.026	7,000
5-Year			0.055	9,000
10-Year			-	-
25-Year			-	-
50-Year			-	-
100-Year			0.677	11,470
Hazel	-	-	1.361	18,319

Table 24: SWM Pond 10 Active Storage Volume Summary

Storm Event	Post-Development Contributing Area (ha)	Percent Imperviousness (%)	Target Flow Rate (m^3/s)	Required Active Storage Volume (m^3)
2-Year	8.50	58%	0.010	2,500
5-Year			0.021	3,300
10-Year			-	-
25-Year			-	-
50-Year			-	-
100-Year			-	-

Table 25: SWM Total Active Storage Volume Summary

SWM Pond Location	Total Active Storage Volume (m^3)
SWM Pond 1	10,603
SWM Pond 2/3	12,953
SWM Pond 4	6,829
SWM Pond 5	22,111
SWM Pond 6	13,300
SWM Pond 7	42,786
SWM Pond 8	24,745
SWM Pond 9	10,100
SWM Pond 10	3,300
SWM Pond 11	2,900

Stormwater Facility Outfalls

The Stormwater Management Criteria (TRCA, 2012) outlines criteria required for stormwater management outfalls to reduce the risk of erosion to the outfall structures, channels, and receiving watercourse and floodplain. The outfall criteria are listed as follows:

- Hard structure outfalls (i.e., headwalls or anything made of concrete) are required to be located outside of the 100-year erosion limit, unless not technically feasible.
- Outfalls should not impact the flood levels.
- Avoid backwater conditions in outfalls, unless explicitly accounted for in the design.
- Minimize elevation between the outfall and receiving channel, placing it as low down on the slope as possible, while still above the 25-year floodline.
- Plunge pools are to be designed at the outfall outlet, unless it can be demonstrated that it is not required to dissipate flow energy, or a plunge pool cannot reasonably be constructed due to topographic constraints.
- Outfalls that are set-back 10 m to 20 m from the channel require a flow spreader and/or a plunge pool design, where feasible.

Several outfall pipes are likely to require steep slopes due to the deep valleys where the receiving watercourses are located. For the outfall pipes with steep slopes, opportunities to reduce slopes, such as incorporating drop structures or cascades will be evaluated at the detailed design stage.

7.0 Water Balance & Low Impact Development

7.1 Site Water Balance and Wetland Feature-Based Water Balance

A water balance assessment and feature based water balance assessment will be completed for the Site as part of the Hydrogeological Report being prepared by Crozier under separate cover. Well drilling took place during October and November 2021 and the results of the monitoring program will be provided in the Hydrogeological Report Submission.

7.2 Low Impact Development

Per TRCA mapping, the Site is located within a Significant Groundwater Recharge Area and a High Groundwater Recharge Area (Stormwater Management Criteria, TRCA 2012). Due to these vulnerable areas, the Site will be required to maintain pre-development groundwater recharge rates and appropriate distribution, ensuring the protection of related hydrologic and ecologic functions.

Low Impact Development (LID) methods can be implemented on-site to mitigate water balance within the Subject Lands and/or to a wetland. Detailed infiltration facility sizing, including in-situ percolation tests and drawdown calculations for each facility, will be completed as part of the Draft Plan stage.

A list of potential LID options, including the SWM criteria which can be achieved via each LID option, is presented in Table 26 below for consideration.

Table 26: LID Options Feasibility Summary

LID Type	Location	Stormwater Management Benefits					
		Un-Lined System			Lined System		
		Water Balance Via Infiltration	Water Quality	Stream Erosion Control	Water Balance Via Infiltration	Water Quality	Stream Erosion Control
Infiltration Gallery (Trenches, Chambers, & Soakaway)	Low & Med-Density Residential (Rear-Yard)	☑	☑	☑	☒	☒	☑
	Mixed-Use Blocks						
	High-Density Blocks						
	Boulevard						
	Park Blocks						
	Pond Blocks						
	LID Blocks						
	Greenbelt Areas						
Bioretention	Mixed-Use Blocks	☑	☑	☑	☒	☑	☑
	High-Density Blocks						
	Boulevards						
	Park Blocks						
	Pond Blocks						
	LID Blocks						
	Greenbelt Areas						

LID Type	Location	Stormwater Management Benefits					
		Un-Lined System			Lined System		
		Water Balance Via Infiltration	Water Quality	Stream Erosion Control	Water Balance Via Infiltration	Water Quality	Stream Erosion Control
Enhanced Grass Swales	Low & Med-Density Residential	☑	☑	☑	☒	☑	☑
	Mixed-Use Blocks						
	High-Density Blocks						
	Boulevard						
	Park Blocks						
	Pond Blocks						
	LID Blocks						
	Greenbelt Areas						
Disconnected Downspouts	Residential lots	☑	☒	☑	N/A	N/A	N/A
Green Roofs	Mixed-Use Blocks	☒	☑	☑	N/A	N/A	N/A
	High-Density Blocks						
	Schools						
Permeable Pavements	Residential Driveways	☑	☑	☑	☒	☑	☒
	Local Roads						
	Mixed-Use Blocks						
	High-Density Blocks						
	Schools						
	SWM Pond Access Roads						

Tree Pits / Planters	Boulevards	☑	☑	☑	☒	☑	☑
	Park Blocks						
	Mixed-Use Blocks						
	High-Density Blocks						
	Schools						
Rainwater Harvesting	Residential lots (Rain Barrels)	☑ (via irrigation)	☑	☑	N/A	N/A	N/A
	Mixed-Use Blocks (Rainwater Cistern)						
	High-Density Blocks (Rainwater Cistern)						
Additional Depth Topsoil	Residential lots	☑	☑	☑	N/A	N/A	N/A
	Boulevards						
	Parks						
Pocket Wetlands	Mixed-Use Blocks	☒	☑	☑	☒	☑	☑
	High-Density Blocks						
	Parks						
	Boulevards						
	Greenbelt Areas						

Note that further investigation of these LID options, including understanding the feasibility of their implementation and in-situ soil and groundwater conditions, will be conducted at the Draft Plan stage once the results of the hydrogeological study are available.

8.0 Floodplain Analysis

8.1 Headwater Drainage Feature Assessment

The Site is situated within the headwaters of the Main Humber River Watershed. A headwater drainage feature assessment specific to the Site has been prepared under separate cover as the Option 1/2 Lands Secondary Plan Natural Heritage Study Report (Dillon, December 2024).

According to the Natural Heritage Study, there are no headwater drainage features within the Site which are protected or require conservation. Therefore, none of the headwater drainage features are maintained within the Site and no floodplain mapping is required or provided for these features within the Site boundary. Conveyance of external drainage discharging to the Site under existing conditions will be achieved by routing the flow through sewers. Additional information regarding the conveyance of external drainage is described in more detail in Section 5.2.

8.2 Hydraulic Modelling

Existing hydraulic (HEC-RAS) models for the Main Humber River and Cold Creek were provided by the Toronto and Region Conservation Authority (TRCA). The received models include:

- Engineered HEC-RAS model for the Main Humber River and the associated TRCA floodplain Map Sheet Hum_77, Hum_81 and Hum_82
- Estimated HEC-RAS model titled Middle Main for tributaries of the Main Humber River Watershed
- Estimated HEC-RAS model titled Cold Creek for tributaries of the Cold Creek Subwatershed
- Engineered HEC-RAS model titled Cold Creek-R2 for tributaries of Cold Creek West and Tributary E.

Updated hydraulic modelling was completed in HEC-RAS v.6.4.1. Digital copies of the HEC-RAS models are provided with this submission. The estimated models for tributaries of the Main Humber River Watershed and Cold Creek Subwatershed were revised as part of this analysis due to their proximity to the Site. The Engineered TRCA HEC RAS model received on February 4, 2026 was used to complete the floodplain analysis for Tributary E and Cold Creek West. Based on pre-consultation with the TRCA, the existing engineered hydraulic model for the Main Humber River does not require re-modelling since the associated floodplain is well outside of the Site area and does not experience any impact in terms of water surface elevation due to development.

From the TRCA estimated models, four (4) tributaries (MM_9_2 - 9.203, MM_9_2_trib - 9.204, MM_9_2 - 9.205, and Cold_10_7 - 10.7) were selected for further analysis. Upon further investigation of the Site's existing and proposed drainage conditions, three (3) new tributaries were added to the model to provide a more comprehensive analysis of the floodplain in the immediate vicinity of the Site. To simplify the review process, the naming convention of each tributary has been updated in the model to reflect the naming convention of tributaries described Section 5.0. The updated naming convention of each tributary, listed from west to east across the Site, is summarized below in Table 27.

Table 27: HEC-RAS Model River Reach Naming Convention

HEC-RAS Model	Original HEC-RAS Tributary ID	Updated HEC-RAS Tributary ID
Middle Main	X[1]	Tributary A
	X[1]	Tributary B
	MM_9_2 - 9.203	Tributary C
	MM_9_2_trib - 9.204	Tributary D
	MM_9_2 - 9.205	Tributary C + Tributary D
Cold Creek	Cold_10_7 - 10.7	Tributary E
	X[1]	Tributary F (Cold Creek)

1. "X" Refers to a new tributary that has been added to the HEC-RAS model.

Modelled cross-sections within the Site were updated to reflect topographic survey information derived from Drone Survey Canada (July 2021). Modelled cross-sections outside of the Site were updated to reflect the Digital Terrain Model (LiDAR-Derived) data obtained from the Land Information Ontario (LIO) Dataset by the Ministry of Natural Resources and Forestry (October 2023). As described in Section 8.1, floodplain mapping has only been completed outside the Site where significant headwater drainage features are located. Since there are no changes to the topography outside of the Site, the cross-section geometries are the same under both existing and proposed conditions.

Two watercourse crossings were added to the existing hydraulic model. Watercourse crossing information was obtained from as-built drawings available on the Region of Peel Public Works – Peel Asset Locator Application. At locations where as-built drawings were not available, watercourse crossings were measured manually during field visits. Watercourse crossing information is summarized in Table 28 below.

Table 28: Modelled Watercourse Crossings

Location	HECRAS Tributary ID	Material	Culvert Length (m)	Culvert Diameter (mm)	Data Source	As-Built Drawing No.
Duffy's Ln approximately 175 m north of Emil Kolb Pkwy	Tributary B	Corrugated Steel Pipe	30.4	600	Field Measurement	N/A
Emil Kolb Pkwy approximately 770 m west of HWY 50	Tributary C	Concrete Bridge with Piers	NA	NA	As-Built	55280-D

Detailed HEC-RAS model schematics, including invert elevations of the above culverts, for Middle Main and Cold Creek are provided in Appendix D1 for reference. Copies of as-built drawings are provided in Appendix D1.

HEC-RAS steady state flow data was updated using the results of the existing and proposed hydrologic models described in Section 6.0. To analyze the Regional floodplain elevations of each tributary, the following storm events were run in Visual Otthymo (v.6.2):

- 100-Year 6-hour AES Design Storm
- 100-Year 12-hour AES Design Storm
- Hurricane Hazel Design Storm

The results of the hydrologic model prove that Hurricane Hazel produces the greatest peak flows. Therefore, Hurricane Hazel was used to perform Regional analysis of the floodplain as per the TRCA's Technical Guidelines for Flood Hazard Mapping (March 2017).

In addition to the Regional event, 2-Year, 25-Year, and 100-Year design storms were also included in the hydraulic model to perform further analysis. The 6-hour AES design storm produced greater peak flow rates than the 12-hour AES design storm after running the events in the hydrologic model. Therefore, the 6-hour AES design storm was selected for further analysis to achieve the most conservative results. A summary of the design storms and method of analysis is provided below:

- 2-Year 6-hour AES: To analyze changes in erosive forces between the existing and proposed conditions models
- 25-Year 6-hour AES: To evaluate acceptable outfall elevations for proposed stormwater management facilities described in Section 6.0
- 100-Year 6-hour AES: To assess the overall changes in floodplain elevations during the 100-year event as part of standard floodplain mapping practices

Peak flow rates were input at various cross-sections of each tributary. The HEC-RAS flow change locations are defined by the upstream limits of each reach, points of confluence, watercourse crossings, and the locations proposed SWM facilities. The existing and proposed Regional flow rates, HEC-RAS flow change locations, and corresponding VO flow node are summarized below in Table 29.

Table 29: HEC-RAS Regional Flow Change Locations

Tributary ID	VO Flow Node Existing	VO Flow Node Proposed	HEC-RAS Station	Regional Flow Rate (m^3/s)	
				Existing	Proposed
A	Transposition	Transposition	1017	0.503	0.473
	7696	7717	996	2.821	2.654
B	Transposition	Transposition	1005	1.703	2.115
	7695	7728	999	2.761	3.429
C	Transposition	Transposition	2179.936	4.559	4.259
	Transposition	Transposition	1855.838	5.325	4.975
	7691	7731	993.3609	7.790	7731
D	Transposition	Transposition	1195.92	5.227	6.093
	Transposition	Transposition	1192.146	8.930	10.410
	7692	7863	886.4161	12.209	14.233
C + D	7690	93	294	0.523	0.523
E	Transposition	Transposition	4822.04	2.448	2.920
	Transposition	Transposition	4460.59	3.914	4.668
	7686	7685	3837.47	9.002	10.737
	7518	7518	10.011	303.179	300.389
R1	1682	1682	107.233	385.589	384.839
	3448	3448	106.631	384.227	383.841
	1471	1471	102.613	395.094	389.732
	Transposition	Transposition	102.398	374.948	374.948
	2048	2048	100.401	395.422	391.543
	1379	1379	11.39	639.230	630.957

1. Flow rates have been prorated based on an area-weighted calculation of each drainage area.

The above proposed flow rates represent the uncontrolled condition. The analysis assumes that the proposed SWM facilities would be under maximum capacity during the Regional event and would not provide quantity control of runoff from the Site. The 2-Year to 100-Year flow rates at each flow change location are provided in Appendix D2. Under the proposed condition, the 2-Year to 100-Year events represent the controlled condition. The analysis of design storms assumes that runoff from the Site would be controlled due to the proposed SWM facilities.

8.3 Hydraulic Model Results

WSE and velocities (left overbank, main channel, and right overbank) were generated and compared for both the existing and proposed conditions models. Tables summarizing the 2-Year, 25-Year, 100-Year WSE and velocity data are summarized in Appendix D3. Table 30 provides a summary of the Regional results at Tributary A.

Table 30: Tributary A – Regional Existing Vs. Proposed WSE and Velocity Data

X Section	Water Surface Elev.			Left Bank Vel.			Main Channel Vel.			Right Bank Vel.		
	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ
1017	254.86	254.85	0.01	0	0	0	0.96	0.95	0.01	0	0	0
1000	253.53	253.53	0	0	0	0	0.9	0.89	0.01	0	0	0
999	252.4	252.4	0	0	0	0	0.12	0.12	0	0	0	0
998.6	252.38	252.38	0	0	0	0	0.21	0.2	0.01	0	0	0
998.4	252.34	252.34	0	0	0	0	0.34	0.33	0.01	0	0	0
998	251.93	251.93	0	0	0	0	0.85	0.83	0.02	0	0	0
997.7	249.47	249.46	0.01	0	0	0	1.29	1.28	0.01	0	0	0
997	245.8	245.79	0.01	0	0	0	1.04	1.07	-0.03	0	0	0
996.6	242.27	242.26	0.01	0	0	0	1.01	0.99	0.02	0	0	0
996	237.8	237.79	0.01	0	0	0	1.15	1.14	0.01	0	0	0
995.8	236.04	236.03	0.01	0	0	0	1.32	1.29	0.03	0	0	0
995.6	234.69	234.69	0	0	0	0	1.02	1.01	0.01	0	0	0
995	232.59	232.58	0.01	0	0	0	1.63	1.6	0.03	0	0	0
932	232	232	0	0.01	0.01	0	0.05	0.04	0.01	0.02	0.01	0.01

1. A "–" indicates that the channel component does not contain runoff at this location

Tributary A experiences minimal decrease in WSE under the proposed condition. As a result, the main channel velocities are also subject to some minor reduction. This decrease occurs because the proposed drainage patterns within the Site are different compared to the existing conditions topography. Less runoff will discharge to Tributary A and more runoff will then discharge to Tributary B. The minimal decrease in WSE and velocity at Tributary A does not pose any increase to flood risk.

Table 31 provides a summary of the Regional results at Tributary B.

Table 31: Tributary B – Regional Existing Vs. Proposed WSE and Velocity Data

X Section	Water Surface Elev.			Left Bank Vel.			Main Channel Vel.			Right Bank Vel.		
	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ
1005	257.12	257.17	-0.05	0	0	0	1.72	1.8	-0.08	0	0	0
1004.6	255.95	256	-0.05	0	0	0	1.7	1.8	-0.1	0	0	0
1004	254.97	255.01	-0.04	0	0	0	1.58	1.67	-0.09	0	0	0
1003	251.72	251.76	-0.04	0	0	0	1.52	1.58	-0.06	0	0	0
1002	250.89	250.96	-0.07	0	0	0	0.18	0.21	-0.03	0	0	0
1001.4	250.89	250.96	-0.07	0	0	0	0.06	0.07	-0.01	0	0	0
1001	250.89	250.96	-0.07	0	0	0	0.05	0.06	-0.01	0	0	0
1000.8	250.89	250.96	-0.07	0	0	0	0.01	0.01	0	0	0	0
1000.2	250.89	250.96	-0.07	0	0	0	0.02	0.02	0	0	0	0
1000	250.89	250.96	-0.07	0	0	0	0.01	0.02	-0.01	0	0	0
999.64	0											
999	243.3	243.43	-0.13	0	0	0	2.82	3.03	-0.21	0	0	0
998	241.43	241.5	-0.07	0	0	0	2.1	2.2	-0.1	0	0	0
997.4	239.49	239.55	-0.06	0	0	0	1.78	1.89	-0.11	0	0	0
997	237.43	237.46	-0.03	0	0	0	1.47	1.54	-0.07	0	0	0
996	234.81	234.84	-0.03	0	0	0	1.43	1.51	-0.08	0	0	0
995.4	232.27	232.33	-0.06	0	0	0	1.92	2.02	-0.1	0	0	0
994.4	231.85	231.85	0	0.01	0.01	0	0.03	0.03	0	0.01	0.01	0
994	231.85	231.85	0	0	0	0	0.01	0.01	0	0	0	0

1. A "–" indicates that the channel component does not contain runoff at this location.

As mentioned in the analysis of Tributary A results, the proposed condition routes less runoff to Tributary A and more runoff to Tributary B. As a result of allocating more runoff to Tributary B, there are increases in WSE throughout the channel. The most significant increase (0.29 m) occurs immediately downstream of the crossing at Duffy's Lane. This increase in WSE can likely be attributed to the increase in flow and undersized culvert at Duffy's Lane. While the increase is considerable, it occurs for a small length of channel is located deep in a significant valley corridor. As indicated in the velocity results, water is still contained well within the banks of the main channel. Since the steep side slopes of the valley provide a narrow channel corridor, there is minimal increase to the extent of floodplain area. Downstream of Duffy's Lane, there is only significant woodland area where no future development will occur. Therefore, the downstream increase in WSE poses no threat to developable land or public safety.

According to the Ministry of Natural Resources – Technical Guide River & Stream Systems: Flooding Hazard Limit (2002), safe access and egress are defined by the following conditions:

- Depths no greater than 0.3 m.
- Velocities no greater than 1.7 m/s.
- Product depth-velocities no greater than 0.4 m^2/s.

At Duffy's Lane, the maximum depth over the road under the proposed condition is 0.28 m. The velocity and product depth-velocity are 0.67 m/s and 0.18 m^2/s, respectively. Therefore, safe access along Duffy's Lane can still be achieved given the increase in flow at Tributary B.

Table 32 provides a summary of the Regional results at Tributary C.

Table 32: Tributary C – Regional Existing Vs. Proposed WSE and Velocity Data

X Section	Water Surface Elev.			Left Bank Vel.			Main Channel Vel.			Right Bank Vel.		
	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ
2179.936	257.07	257.06	0.01	0	0	0	0.7	0.7	0	0	0	0
2159.385	256.99	256.98	0.01	0	0	0	0.5	0.5	0	0	0	0
2097.132	256.83	256.82	0.01	0	0	0	0.58	0.57	0.01	0	0	0
2030.564	256.34	256.33	0.01	0	0	0	1.45	1.42	0.03	0	0	0
1994	256.21	256.19	0.02	0	0	0	0.34	0.33	0.01	0	0	0
1970.384	256.2	256.18	0.02	0	0	0	0.2	0.19	0.01	0	0	0
1911.614	256.2	256.18	0.02	0	0	0	0.21	0.2	0.01	0	0	0
1881.066	0											
1855.838	255.84	255.83	0.01	0	0	0	1.39	1.37	0.02	0	0	0
1813.506	254.99	254.98	0.01	0	0	0	1.15	1.12	0.03	0	0	0
1775.696	254.41	254.41	0	0	0	0	0.95	0.92	0.03	0	0	0
1726.57	253.3	253.29	0.01	0	0	0	1.37	1.35	0.02	0	0	0
1684.01	252.16	252.15	0.01	0	0	0	1.17	1.18	-0.01	0	0	0
1603.573	252	251.99	0.01	0	0	0	0.23	0.23	0	0	0	0
1539.326	251.89	251.88	0.01	0	0	0	0.71	0.7	0.01	0	0	0
1486.021	251.35	251.33	0.02	0	0	0	1.18	1.16	0.02	0	0	0
1450.067	250.78	250.77	0.01	0	0	0	1.61	1.58	0.03	0	0	0
1406.638	250.2	250.18	0.02	0	0	0	1.17	1.14	0.03	0	0	0
1338.455	249.33	249.32	0.01	0	0	0	1.58	1.55	0.03	0	0	0
1304.352	248.58	248.56	0.02	0	0	0	1.67	1.65	0.02	0	0	0
1265.187	247.93	247.89	0.04	0	0	0	1.8	1.9	-0.1	0	0	0
1206.875	246.81	246.78	0.03	0	0	0	1.76	1.76	0	0	0	0
1175.661	246.29	246.28	0.01	0	0	0	1.57	1.53	0.04	0	0	0

X Section	Water Surface Elev.			Left Bank Vel.			Main Channel Vel.			Right Bank Vel.		
	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ
1120.613	245.37	245.35	0.02	0	0	0	1.18	1.15	0.03	0	0	0
1083.716	244.89	244.86	0.03	0	0	0	1.84	1.85	-0.01	0	0	0
1042.153	243.73	243.71	0.02	0	0	0	1.15	1.13	0.02	0	0	0
1006.177	243.16	243.14	0.02	0	0	0	1.81	1.77	0.04	0	0	0
993.3609	243.04	243.02	0.02	0	0	0	1.04	1.02	0.02	0	0	0
935.4169	242.42	242.4	0.02	0	0	0	1.9	1.9	0	0	0	0
870.5434	240.62	240.58	0.04	0	0	0	2.59	2.55	0.04	0	0	0
836.4684	239.53	239.5	0.03	0	0	0	2.57	2.53	0.04	0	0	0
799.8246	238.76	238.73	0.03	0	0	0	2.42	2.37	0.05	0	0	0
745.4448	236.54	236.51	0.03	0	0	0	2.3	2.27	0.03	0	0	0
716.4415	235.51	235.5	0.01	0	0	0	2.06	1.97	0.09	0	0	0
687.3773	235.1	235.07	0.03	0	0	0	1.64	1.67	-0.03	0	0	0
661.4518	234.27	234.24	0.03	0	0	0	2.35	2.3	0.05	0	0	0
627.6722	233.4	233.37	0.03	0	0	0	2.36	2.32	0.04	0	0	0
589.2625	232.75	232.73	0.02	0	0	0	1.35	1.32	0.03	0	0	0
540.5079	232.16	232.13	0.03	0	0	0	2.3	2.26	0.04	0	0	0
510.2238	231.19	231.17	0.02	0	0	0	2.03	1.98	0.05	0	0	0
457.5412	229.95	229.91	0.04	0	0	0	2.45	2.41	0.04	0	0	0
414.0466	229.3	229.28	0.02	0	0	0	2.25	2.18	0.07	0	0	0
374.4607	228.69	228.67	0.02	0	0	0	2.08	2.05	0.03	0	0	0
330.2584	227.84	227.8	0.04	0	0	0	1.1	1.31	-0.21	0	0	0

1. A "–" indicates that the channel component does not contain runoff at this location.

Tributary C experiences a minimal decrease in WSE and velocity throughout the channel. There are no concerns in terms of increasing the flood risk to adjacent properties and public safety.
Table 33 provides a summary of the Regional results at Tributary D.

Table 33: Tributary D – Regional Existing Vs. Proposed WSE and Velocity Data

X Section	Water Surface Elev.			Left Bank Vel.			Main Channel Vel.			Right Bank Vel.		
	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ
1195.922	264.33	264.36	-0.03	0.56	0.61	-0.05	1.48	1.53	-0.05	0.36	0.43	-0.07
1194	263.71	263.74	-0.03	0.51	0.51	0	1.24	1.3	-0.06	0	0	0
1193.146	263.4	263.43	-0.03	0	0	0	0.52	0.55	-0.03	0.32	0.34	-0.02
1193.1	263.2	263.22	-0.02	0.98	1.04	-0.06	1.6	1.68	-0.08	0	0	0
1193.002	263.18	263.2	-0.02	0.06	0.07	-0.01	0.2	0.22	-0.02	0.07	0.07	0
1193.001	263.08	263.12	-0.04	0.78	0.73	0.05	1.6	1.48	0.12	0	0	0
1193	262.48	262.52	-0.04	0.12	0.14	-0.02	0.76	0.81	-0.05	0	0	0
1192.777	262.36	262.39	-0.03	0	0	0	0.94	0.99	-0.05	0	0	0
1192.631	262.07	262.11	-0.04	0	0	0	0.94	0.98	-0.04	0	0	0
1192.407	261.3	261.34	-0.04	0	0.14	-0.14	1.7	1.79	-0.09	0	0	0
1192.296	260.6	260.65	-0.05	0	0	0	1.96	2.03	-0.07	0	0	0
1192.146	257.43	257.49	-0.06	0	0	0	2.15	2.22	-0.07	0	0	0
1158.742	255.85	255.9	-0.05	0	0	0	1.51	1.54	-0.03	0	0	0
1117.042	255.09	255.16	-0.07	0	0	0	2.34	2.44	-0.1	0	0	0
1085.498	254.46	254.52	-0.06	0	0	0	2.23	2.31	-0.08	0	0	0
1056.979	253.48	253.5	-0.02	0	0	0	1.43	1.49	-0.06	0	0	0
1006.094	250.35	250.43	-0.08	0	0	0	2.51	2.6	-0.09	0	0	0
954.6843	248.23	248.31	-0.08	0	0	0	2.57	2.67	-0.1	0	0	0
927.8772	247.55	247.64	-0.09	0	0	0	2.74	2.83	-0.09	0	0	0
886.4161	246.94	247.05	-0.11	0	0	0	2.07	2.11	-0.04	0	0	0
830.2587	246.14	246.25	-0.11	0	0	0	2.78	2.84	-0.06	0	0	0
765.2102	245.23	245.3	-0.07	0	0	0	1.75	1.73	0.02	0	0	0
719.386	244.58	244.6	-0.02	0	0	0	1.69	1.82	-0.13	0	0	0
691.6686	244.17	244.21	-0.04	0	0	0	1.54	1.59	-0.05	0	0	0
639.0789	243.24	243.29	-0.05	0	0	0	1.89	1.95	-0.06	0	0	0
605.7167	242.58	242.62	-0.04	0	0	0	1.87	1.96	-0.09	0	0	0
552.2101	240.9	240.98	-0.08	0	0	0	1.98	1.9	0.08	0	0	0
508.8769	240.12	240.21	-0.09	0	0	0	2.45	2.48	-0.03	0	0	0
457.6058	239.16	239.2	-0.04	0	0	0	1.78	1.85	-0.07	0	0	0
393.3919	237.75	237.8	-0.05	0	0	0	2.16	2.26	-0.1	0	0	0
345.8717	237.22	237.27	-0.05	0	0	0	1.43	1.54	-0.11	0	0	0
308.4069	236.81	236.84	-0.03	0	0	0	1.76	1.84	-0.08	0	0	0
261.2717	236.03	236.11	-0.08	0	0	0	1.67	1.64	0.03	0	0	0
237.4458	235.48	235.55	-0.07	0	0	0	2.6	2.71	-0.11	0	0	0

X Section	Water Surface Elev.			Left Bank Vel.			Main Channel Vel.			Right Bank Vel.		
	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ
197.6155	234.13	234.17	-0.04	0	0	0	1.39	1.48	-0.09	0	0	0
176.7205	233.84	233.87	-0.03	0	0	0	1.78	1.88	-0.1	0	0	0
163.5648	233.51	233.55	-0.04	0	0	0	1.85	1.92	-0.07	0	0	0
151.4771	232.33	232.42	-0.09	0	0	0	2.63	2.72	-0.09	0	0	0
140.92	231.85	231.9	-0.05	0	0	0	2.07	2.16	-0.09	0	0	0
123.28	230.94	231.01	-0.07	0	0	0	2.49	2.59	-0.1	0	0	0
89.70679	230.31	230.41	-0.1	0	0	0	2.65	2.72	-0.07	0	0	0
72.5771	229.73	229.79	-0.06	0	0	0	2.1	2.17	-0.07	0	0	0
51.16022	228.96	228.99	-0.03	0	0	0	1.57	1.63	-0.06	0	0	0
26.54893	228.21	228.25	-0.04	0	0	0	1.75	1.8	-0.05	0	0	0

1. A "–" indicates that the channel component does not contain runoff at this location.
2. An "NA" indicates that the open channel has been removed and converted to pipe flow under the proposed condition.

From cross sections 1195.992 to 1192.296, Tributary D will be converted to piped flow under the proposed condition. Therefore, the flood risk along this section does not exist. Similar to Tributary B, the maximum increase in WSE (0.14 m) occurs within a deep valley corridor. The results indicate that the water is contained within the main channel as there are no values for the left and right bank velocities. Even though there is a considerable increase in depth, the steep side slopes of the valley contain the water and prevent the floodplain area from increasing. The described increase will not pose any risk to public safety or neighbouring properties.

Table 34 provides a summary of the Regional results at Tributary C + Tributary D.

Table 34: Tributary C + Tributary D – Regional Existing Vs. Proposed WSE and Velocity Data

X Section	Water Surface Elev.			Left Bank Vel.			Main Channel Vel.			Right Bank Vel.		
	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ
294	227.13	227.13	0	0	0	0	0.62	0.62	0	0	0	0
266.3048	225.08	225.08	0	0	0	0	1.25	1.25	0	0	0	0
236.2558	224.21	224.21	0	0	0	0	0.2	0.2	0	0	0	0
205.6305	224.21	224.21	0	0	0	0	0.11	0.11	0	0	0	0
173.8239	224.21	224.21	0	0	0	0	0.04	0.04	0	0	0	0
151.7834	224.21	224.21	0	0	0	0	0.02	0.02	0	0	0	0
116.6732	224.21	224.21	0	0	0	0	0.01	0.01	0	0	0	0
66.5837	224.21	224.21	0	0	0	0	0	0	0	0	0	0
34.76245	224.21	224.21	0	0	0	0	0	0	0	0	0	0

1. A "–" indicates that the channel component does not contain runoff at this location.

The increase to WSE and velocity at Tributary C + Tributary D is minimal and can be considered negligible (0.01 m to 0.02 m). There will be no increase in flood risk due to the proposed development.

Table 35 provides a summary of the Regional results at Tributary E.

Table 35: Tributary E – Regional Existing Vs. Proposed WSE and Velocity Data

X Section	Water Surface Elev.			Left Bank Vel.			Main Channel Vel.			Right Bank Vel.		
	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ
4822.04	263.65	263.69	-0.04	0.09	0.2	-0.11	0.81	0.81	0	0.5	0.52	-0.02
4783.59	263.17	263.18	-0.01	0.32	0.37	-0.05	0.78	0.89	-0.11	0.46	0.52	-0.06
4731.68	262.11	262.15	-0.04	0.48	0.38	0.1	1.18	1.05	0.13	0.52	0.57	-0.05
4652.11	262.11	262.18	-0.07	0.03	0.03	0	0.09	0.09	0	0.03	0.03	0
4592.65	262.11	262.18	-0.07	0.03	0.03	0	0.08	0.09	-0.01	0.02	0.02	0
4515.64	262.08	262.15	-0.07	0	0.08	-0.08	0.67	0.67	0	0.31	0.33	-0.02
4460.59	262.07	262.13	-0.06	0.09	0.1	-0.01	0.16	0.17	-0.01	0.09	0.1	-0.01
4407.62	262.02	262.09	-0.07	0.2	0.22	-0.02	0.49	0.51	-0.02	0.22	0.23	-0.01
4354.33	261.84	261.91	-0.07	0.18	0.22	-0.04	0.63	0.67	-0.04	0.4	0.42	-0.02
4322.62	261.72	261.78	-0.06	0.22	0.25	-0.03	0.65	0.69	-0.04	0.29	0.3	-0.01
4293.51	261.53	261.59	-0.06	0.25	0.32	-0.07	0.91	0.97	-0.06	0.56	0.58	-0.02
4265.3	260.95	260.98	-0.03	0	0	0	1.52	1.62	-0.1	0.57	0.68	-0.11
4240.34	260.23	260.27	-0.04	0	0	0	0.62	0.65	-0.03	0.47	0.49	-0.02
4181.11	259.92	259.96	-0.04	0.21	0.24	-0.03	0.52	0.55	-0.03	0.38	0.4	-0.02
4121.04	259.2	259.22	-0.02	0.66	0.71	-0.05	1.37	1.44	-0.07	0.97	1.05	-0.08
4055.36	258.95	259.01	-0.06	0.06	0.06	0	0.1	0.11	-0.01	0.07	0.07	0
4004.48	258.95	259.01	-0.06	0.03	0.03	0	0.07	0.07	0	0.02	0.02	0
3947.08	258.95	259.01	-0.06	0.01	0.01	0	0.04	0.05	-0.01	0.01	0.01	0
3869.46	258.95	259.01	-0.06	0.06	0.06	0	0.1	0.11	-0.01	0.05	0.06	-0.01
3837.47	258.84	258.91	-0.07	0.68	0.69	-0.01	1.33	1.3	0.03	0.54	0.53	0.01
3758.91	258.66	258.73	-0.07	0.26	0.27	-0.01	0.37	0.39	-0.02	0.23	0.25	-0.02
3700.91	258.56	258.63	-0.07	0.37	0.41	-0.04	0.54	0.57	-0.03	0.38	0.42	-0.04
3650.13	258.39	258.45	-0.06	0.38	0.42	-0.04	0.66	0.7	-0.04	0.57	0.61	-0.04
3587.5	258.15	258.2	-0.05	0.45	0.49	-0.04	0.54	0.58	-0.04	0.48	0.51	-0.03
3530.86	257.85	257.9	-0.05	0.56	0.59	-0.03	0.74	0.79	-0.05	0.58	0.62	-0.04
3486.25	257.47	257.52	-0.05	0.39	0.39	0	0.69	0.74	-0.05	0.6	0.64	-0.04
3456.94	257.19											
3435.76	256.73	256.77	-0.04	0.69	0.74	-0.05	0.95	1.01	-0.06	0.93	0.98	-0.05
3426.56	256.48	256.52	-0.04	0.51	0.56	-0.05	0.75	0.82	-0.07	0.8	0.86	-0.06
3396.62	256.13	256.18	-0.05	0.8	0.85	-0.05	0.91	0.97	-0.06	0.78	0.84	-0.06
3377.5	255.86	255.92	-0.06	0.72	0.74	-0.02	0.7	0.73	-0.03	0.67	0.69	-0.02
3359.36	255.64	255.77	-0.13	0.77	0.67	0.1	0.84	0.76	0.08	0.77	0.69	0.08
3339.67	255.5	255.69	-0.19	0.58	0.51	0.07	0.68	0.6	0.08	0.62	0.54	0.08

X Section	Water Surface Elev.			Left Bank Vel.			Main Channel Vel.			Right Bank Vel.		
	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ
3321.23	255.43	255.66	-0.23	0.49	0.44	0.05	0.66	0.57	0.09	0.47	0.41	0.06
3287.37	255.38	255.63	-0.25	0.3	0.28	0.02	0.46	0.44	0.02	0.41	0.39	0.02
3269.41	255.33	255.59	-0.26	0	0.14	-0.14	0.72	0.67	0.05	0	0.15	-0.15
3191.63	255.24	255.53	-0.29	0.15	0.18	-0.03	0.43	0.4	0.03	0.12	0.13	-0.01
3188.65	255.21	255.49	-0.28	0	0	0	0.75	0.79	-0.04	0	0	0

1. A "–" indicates that the channel component does not contain runoff at this location.

Tributary E experiences no impact to WSE and velocity throughout the channel. There is no flood risk at Tributary E due to the proposed development.

Table 36 provides a summary of the Regional results at Cold Creek West.

Table 36: Tributary Cold Creek West– Regional Existing Vs. Proposed WSE and Velocity Data

X Section	Water Surface Elev.			Left Bank Vel.			Main Channel Vel.			Right Bank Vel.		
	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ	Ex.	Pr.	Δ
1784.71	254.64	254.56	0.08	0	0	0	1.66	1.55	0.11	0	0	0
1752.69	253.6	253.55	0.05	0	0	0	1.42	1.26	0.16	0	0	0
1724.34	252.75	252.72	0.03	0	0	0	0.99	0.89	0.1	0	0	0
1663.99	250.42	250.34	0.08	0	0	0	1.84	1.72	0.12	0	0	0
1612.71	248.72	248.74	-0.02	0	0	0	0.87	0.9	-0.03	0	0	0
1586	247.25	247.26	-0.01	0	0	0	1.14	1.18	-0.04	0	0	0
1531	244.34	244.35	-0.01	0	0	0	1.3	1.34	-0.04	0	0	0
1458	239.52	239.53	-0.01	0	0	0	1.6	1.63	-0.03	0	0	0
1397	236.72	236.72	0	0	0	0	1.65	1.64	0.01	0	0	0
1339	234.88	234.87	0.01	0	0	0	1.02	1.02	0	0	0	0
1206	232.06	232.05	0.01	0	0	0	2.48	2.48	0	0	0	0
1205	230.83	230.83	0	0	0	0	0.4	0.39	0.01	0	0	0
1202.128 10.09.38	230.23	230.23	0	0.83	0.83	0	3.77	3.77	0	0.83	0.83	0
1202.052 10.09.37	230.2	230.2	0	0.59	0.59	0	2.51	2.51	0	0.65	0.65	0
1201.976 10.09.36	229.92	229.92	0	0.62	0.62	0	3.71	3.71	0	1	1	0
1201.9 10.09.35	229.5	229.5	0	0.91	0.91	0	4.03	4.03	0	0.79	0.79	0
1201.824 10.09.34	229.64	229.64	0	0.61	0.61	0	2.06	2.06	0	0.46	0.46	0
1201.748 10.09.33	228.93	228.93	0	1.17	1.17	0	4.61	4.61	0	0.94	0.94	0
1201.672 10.09.32	228.97	228.97	0	0.55	0.55	0	2.52	2.52	0	0.78	0.78	0
1201.596 10.09.31	228.81	228.81	0	0.77	0.77	0	2.57	2.57	0	0.82	0.82	0

1. A "–" indicates that the channel component does not contain runoff at this location.

There is generally a decrease in WSE and velocity at the beginning section of Cold Creek West. This occurs because the proposed grading sends water further downstream compared to the existing conditions. The decreases are not a concern in terms of flood risk. There is minimal increase at the mid-tributary that can be considered negligible (0.01 m to 0.02 m).

As per the TRCA's Technical Guidelines for Flood Hazard Mapping (March 2017), Regional floodplain mapping for the Hurricane Hazel event was generated for both the existing and proposed conditions. Drawings C711A to C711D depict the Regional floodplain mapping and associated 10 m development setback. As illustrated in the floodplain mapping, the property boundary and extent of development is outside of the Regulatory floodplain. A comparison of the existing and proposed conditions floodplain mapping demonstrates that there is minimal increase to floodplain area due to development.

9.0 Secondary Plan Natural Heritage Study Report

Dillon Consulting Ltd. has prepared a Secondary Plan Natural Heritage Study Report (herein referred to as the Report), dated December 2024, under separate cover. The purpose of this Report is to document the existing conditions of the natural environment within 120 m of the Site. The Report also addresses the potential limits of development, evaluates potential environmental impacts associated with development, and recommends mitigation, restoration, and enhancement measures to preserve and/or restore natural features.

The Report includes discussion on the following with respect to existing conditions:

- Designated Natural Heritage Features
- Landforms, Soils and Geology
- Aquatic Environment
- Terrestrial Environment
- Significant Wildlife Habitat
- Species at Risk

Additional studies will be required for future development proposal and particularly on lands where access was not granted for preparation of the Secondary Plan Natural Heritage Study Report.

10.0 Regional Official Plan Amendment 30 Lands

Regional Official Plan Amendment 30 "ROPA 30" identified lands at the south end of the Bolton North Hill Option 1 lands as the ROPA 30 lands, which were included within the Bolton Settlement Area. Following approval with modifications by the Minister of Municipal Affairs and housing for the Reel Region Official Plan 2022, as well as the approval of the Future Caledon Official Plan (2025), and the Town's approved Growth Management and Phasing Plan ("GMPP"), a comprehensive servicing strategy has been developed for the Bolton North Hill Lands. The servicing plan would allow any development parcel to proceed with adequate municipal servicing. Interim water servicing is being provided through the Macville Booster Pumping Station up to 2,960 people, with the ultimate water servicing strategy reliant on a new elevated tank. Sanitary servicing is being provided through new gravity trunk sewers on Highway 50 and Emil Kolb Parkway, with ultimate discharge to Humber Station Road via sewage pump station, forcemain, and gravity sewer on King St.

11.0 Conclusions & Recommendations

The proposed development of the Site can be serviced for water, sanitary and stormwater in accordance with the Town of Caledon and Region of Peel requirements and standards. A summary of our conclusions and recommendations includes:

1. Water servicing is in line with the R.J. Burnside Technical Memorandum dated May 13, 2021, which includes the creation of a new Pressure Zone 7 to service the north end of Bolton. An elevated water tank will be included within Option 1 and a 400 mm diameter feedermain is required along Emil Kolb Parkway and Regional Road 50 to service the Subject Lands. A water booster station (designed by R.J. Burnside) will be utilized for water supply prior to the construction of the elevated water tank.

2. The Region and other consulting teams are completing the design of the sanitary servicing strategy for the Subject Lands. Through consultation with these teams, we understand that sanitary servicing is designed to convey all sanitary flows from the Option 1 and Option 2 Lands west to Humber Station Road. From the Option 2 Lands to the intersection of Regional Road 50 and Emil Kolb Parkway, flows will be conveyed via a forcemain or gravity sewer depending on the outcome of the Region's Water and Wastewater Municipal Class Environmental Assessment. From Regional Road 50, flows will then discharge via a gravity sewer to a proposed sewage pumping station on Emil Kolb Parkway. Flows will then be pumped south on Emil Kolb Parkway to a gravity sewer on King Street, which will outlet to the trunk sewer on Humber Station Road.

3. Grading of the Site is constrained by existing elevations (matching drainage divides and elevations at the boundary), the normal water level in the stormwater management (SWM) facilities, and minimum cover on the gravity sanitary system. Stormwater flows in excess of the 10-year event are conveyed to the SWM facilities via overland flow path. Existing elevations on the arterial roadways (Regional Road 50, Emil Kolb, and Duffy's) will be maintained and matched at the entrances to the Site.

4. Several drainage divides exist throughout the Site, which direct flows to tributaries of the West Humber River. The drainage divide between sub-basin 10 of the West Humber River (which requires quantity controls) and areas of the West Humber River Watershed (which do not require quantity control) will be generally maintained by the proposed grading and servicing concept.

5. The floodplain analysis includes mapping of the existing floodplain and an evaluation of the post-development floodplain. The proposed development plan will require tributaries within the development area to be realigned in order to safely convey external drainage though or around the proposed development. Based on the findings of the Dillon headwater drainage feature assessment this realignment may need to include replication of the floodplain storage provided onsite and natural channel design.

6. Stormwater management facilities (wet ponds) will be implemented within each of the drainage divides to maintain outlets to the existing watersheds. A total of eight (8) ponds will be located within an area of the Main Humber watershed, which does not require typically quantity control; some of these ponds require quantity control to mitigate increases in peak flows to downstream features, as well as quality and erosion control. The remaining three (3) SWM ponds are located within sub-basin 10 of the Humber River and designed to provide quantity control as well as quality and erosion control. Outlets from the ponds will be set at or above the 25-year water level of the receiving watercourse.

7. A water balance assessment will be included in a future hydrogeological report, which will be prepared by C.F. Crozier & Associates Inc. under separate cover. Low impact development (LID) features were investigated to assist with water balance and environmentally friendly stormwater management solutions. Potential LID methods to be utilized on Site include green roofs, roof downspout disconnection, infiltration galleries, rainwater harvesting, bioretention, vegetated filter strips, permeable pavement, and enhanced grass swales.

8. A Secondary Plan Natural Heritage Study Report (the Report) was prepared by Dillon Consulting Ltd. and provided under separate cover. The Report provides a detailed background review and description of the existing terrestrial and aquatic conditions within 120 m of the Site where access was permitted.

Based on the above conclusions, we support the proposed development application of the Site from the perspective of water supply, sanitary servicing, and stormwater management.

C.F. CROZIER & ASSOCIATES INC.



Travis Gibson, P.Eng., PMP
Manager, Land Development

C.F. CROZIER & ASSOCIATES INC.

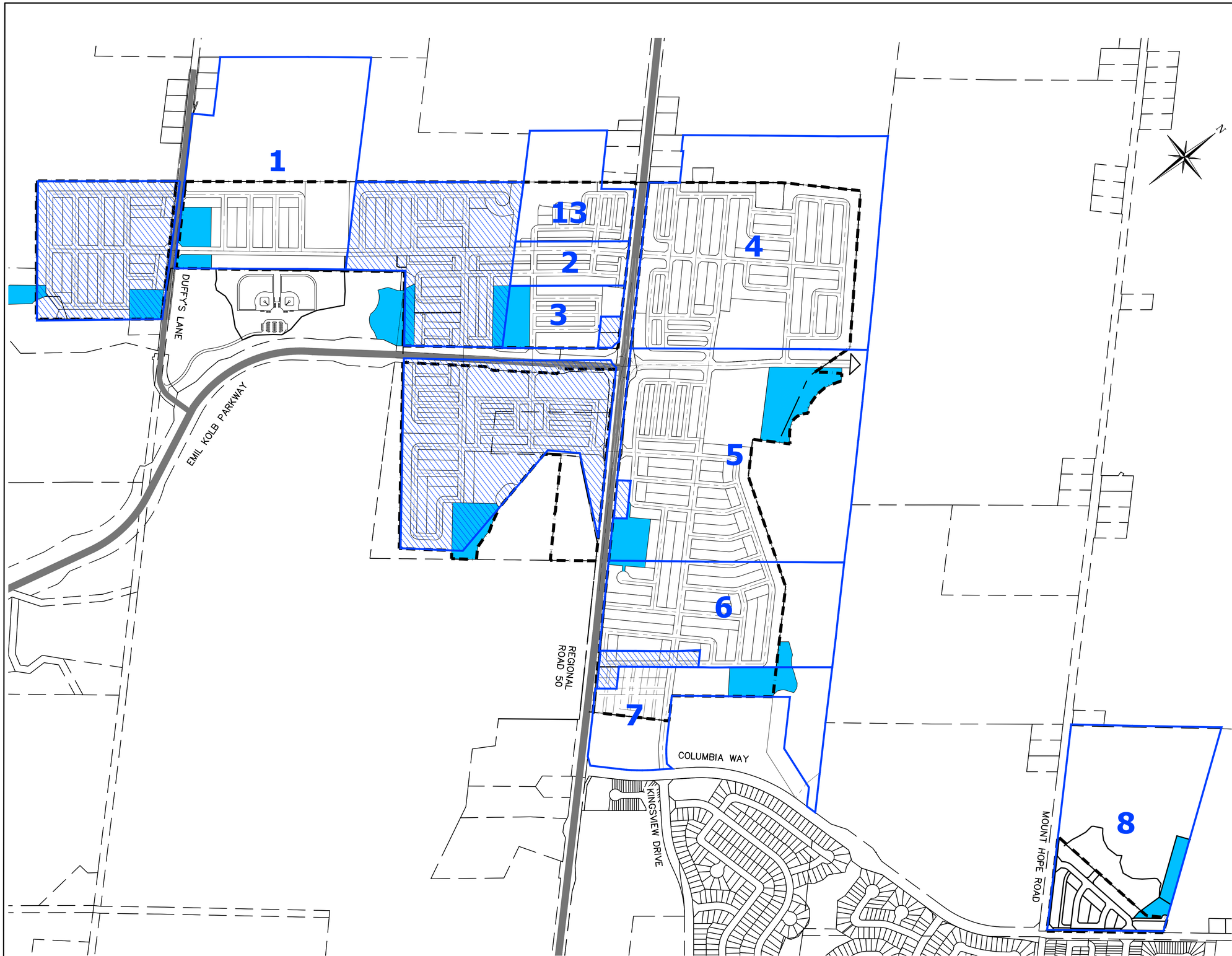


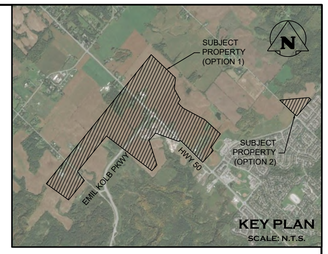
Rebecca Archer, P.Eng.
Project Engineer, Land Development

JL:RA/tc

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APPENDIX F – DRAWINGS





LEGEND

1 — PROPERTY BOUNDARY AND BOLTON NORTH HILL OWNERSHIP ID

NON-PARTICIPANT PROPERTY

PROPERTY BOUNDARY

NOTES

ALL OF THESE PROPERTIES, INCLUDING WHAT IS TRADITIONALLY REFERRED TO AS THE OPTION 1 LANDS (PARECELS 1–7, 13) AND OPTION 2 LANDS (PARCEL 8) ARE INCLUDED IN THE PHASE 1 DEVELOPMENT PLANS PER THE TOWN OF CALEDON'S OFFICIAL PLAN AMENDMENT TO ADDRESS GROWTH MANAGEMENT AND PHASING.

PLAN PREPARED BY BOUSFIELDS INC. DATED FEBRUARY 2026

1. THIS DRAWING IS THE EXCLUSIVE PROPERTY OF C.F. CROZIER & ASSOCIATES INC. AND THE REPRODUCTION OF ANY PART WITHOUT PRIOR WRITTEN CONSENT OF THIS OFFICE IS STRICTLY PROHIBITED.

2. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, LEVELS, AND DATUMS ON SITE AND REPORT ANY DISCREPANCIES OR OMISSIONS TO THIS OFFICE PRIOR TO CONSTRUCTION.

3. THIS DRAWING IS TO BE READ AND UNDERSTOOD IN CONJUNCTION WITH ALL OTHER PLANS AND DOCUMENTS APPLICABLE TO THIS PROJECT.

4. DO NOT SCALE THE DRAWINGS.

5. ALL EXISTING UNDERGROUND UTILITIES TO BE VERIFIED IN THE FIELD BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

EXISTING GROUND CONTOURS DERIVED FROM AERIAL SURVEY BY DRONE SURVEY CANADA (JULY 2021). ADDITIONAL CONTOUR DATA ADAPTED FROM THE ONTARIO DIGITAL TERRAIN MODEL (LIDAR-DERIVED) LAND INFORMATION ONTARIO (LIO) DATASET (MINISTRY OF NATURAL RESOURCES AND FORESTRY, 2017).

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2	ISSUED FOR SECOND SUBMISSION	02/18/2025
3	ISSUED FOR THIRD SUBMISSION	02/27/2026

Engineer

Engineer

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Project

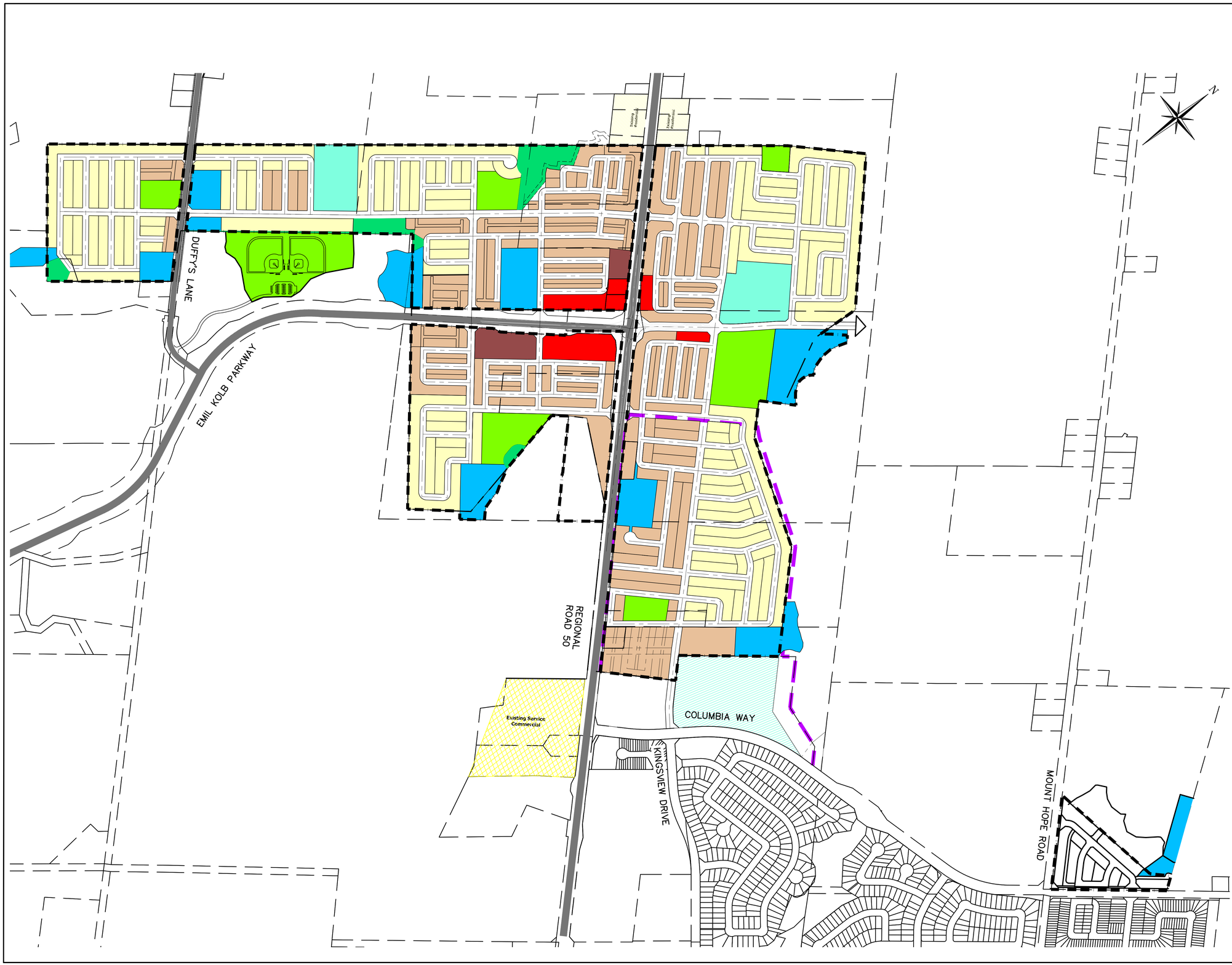
BOLTON NORTH HILL
TOWN OF CALEDON

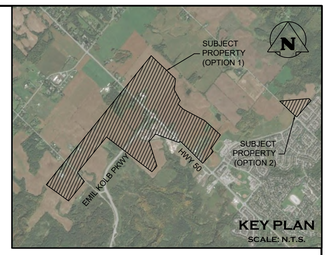
Drawing

LANDOWNERSHIP MAP

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2800 HIGH POINT DRIVE
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Drawn By	Design By	Project	
S.v.N./J.B.	S.v.N./J.L.		708-3446
Check By	Check By	Scale	Drawing
J.L./J.B.	T.G.	1:6000	C701





LEGEND

- PROPERTY BOUNDARY
- LOW DENSITY SINGLES
- MEDIUM DENSITY (TOWNS)
- MEDIUM/HIGH DENSITY
- MIXED USE
- SCHOOLS
- PARKS
- OPEN SPACE
- STORM WATER PONDS
- ROUNDING OUT AREA
- EXISTING SEPARATE SECONDARY SCHOOL
- EXISTING RESIDENTIAL
- ROPA 30 BOUNDARY

PLAN PREPARED BY BOUSFIELDS INC. DATED JANUARY 27, 2025

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Engineer

Engineer

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BOLTON NORTH HILL
TOWN OF CALEDON

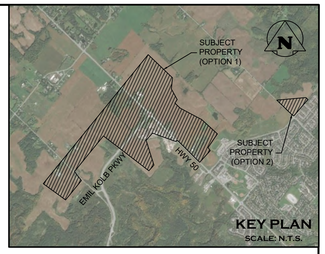
Drawing

PRELIMINARY FRAMEWORK PLAN

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S.v.N./J.B.	S.v.N./J.L.		708-3446
Check By	Check By	Scale	Drawing
J.L./J.B.	T.G.	1:6000	C702

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KEY PLAN
SCALE: N.T.S.

EXT1 37.80 83
EXT2 36.99 83
EXT3 109.1 83
EXT4 168.29 83
EXT5 151.99 83
EXT6 67.63 83

EMIL KOLB PARKWAY
DUFFY'S LANE
REGIONAL ROAD 50
KINGSVIEW DRIVE
MOUNT HOPE ROAD
HUMBER RIVER
COLD CREEK
TRIBUTARY A
TRIBUTARY B
TRIBUTARY C
TRIBUTARY D
TRIBUTARY E
EX. 52.0m 12500 HOPE CULVERT
EX. 52.5m 2.44x2.13m CONC. BOX CULVERT
EX. ±9508 HOPE CULVERT
EX. 83x1.22m CONC. BOX CULVERT
EX. 62.5m 1200Ø CSP CULVERT
EX. ±9508 HOPE CULVERT

LEGEND

- 198.00 — EXISTING MAJOR CONTOURS (2.0m)
- 197.00 — EXISTING MINOR CONTOURS (1.0m)
- DRAINAGE AREA
- PROPERTY BOUNDARY
- EXISTING OVERLAND FLOW ARROW
- 100 / 5.28 92 — CATCHMENT ID; CN NUMBER (PERVIOUS AREAS) % IMPERVIOUS (IMPERVIOUS AREAS); CATCHMENT AREA (ha)

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Engineer | Engineer

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BOLTON NORTH HILL
TOWN OF CALEDON

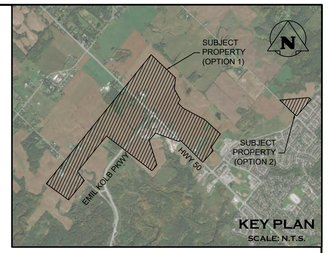
Drawing

PRE-DEVELOPMENT STORM DRAINAGE PLAN

CROZIER CONSULTING ENGINEERS
2800 HIGH POINT DRIVE
SUITE 100
MILTON, ON, L9T 6P4
905-875-0026 T
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Drawn By	Design By	Project	708-3446
S.v. / E.D.	E.D.		
Check By	Check By	Scale	Drawing
J.L. / J.B.	T.G.	1:6000	C703

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KEY PLAN
SCALE: N.T.S.

N

LEGEND

263.42	EXISTING ELEVATION
×198.11	PROPOSED ELEVATION
1.6%	PROPOSED SLOPE
	PROPERTY BOUNDARY
	OVERLAND FLOW ARROW
264.00	EXISTING MAJOR CONTOURS
	EXISTING MINOR CONTOUR

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Engineer

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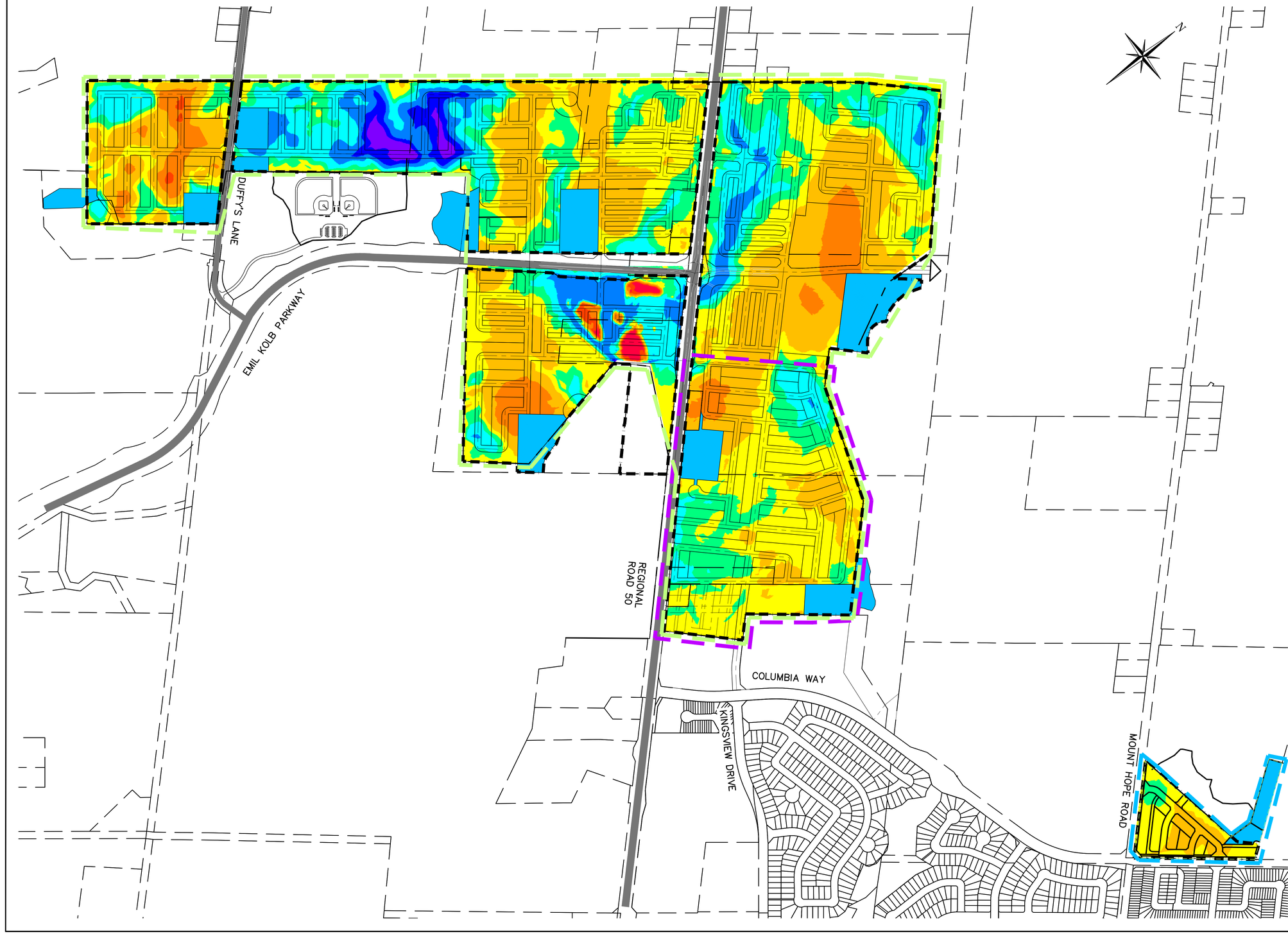
BOLTON NORTH HILL
TOWN OF CALEDON

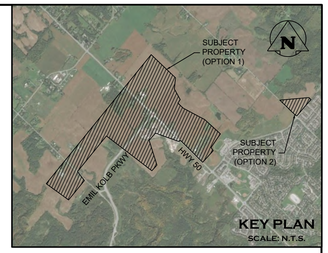
Drawing

GENERAL GRADING

CROZIER CONSULTING ENGINEERS
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Drawn By	Design By	Project	
S.v.N./J.B.	S.v.N./J.L.		708-3446
Check By	Check By	Scale	Drawing
J.L./J.B.	T.G.	1:6000	C704





KEY PLAN
SCALE: N.T.S.

LEGEND

PROPERTY BOUNDARY

CUT / FILL ANALYSIS				
Number	Minimum Elevation	Maximum Elevation	Color	VOL (cum)
1	-12.3	-7.0		10809
2	-7.0	-5.0		13280
3	-5.0	-3.0		84724
4	-3.0	-1.0		547833
5	-1.0	0.0		805645
6	0.0	1.0		477569
7	1.0	3.0		461294
8	3.0	5.0		132203
9	5.0	7.0		41215
10	7.0	10.1		9622

ROPA 30 BOUNDARY

OPTION 1

OPTION 2

EARTHWORKS NOTES
- ASSUMED TOPSOIL STRIPPING DEPTH = 0.30m
- ASSUMED AVERAGE PRE-GRADE DEPTH = 0.80m
- OPTION 1 VOLUMES ARE INCLUSIVE OF ROPA 30 LAND

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Engineer

Engineer

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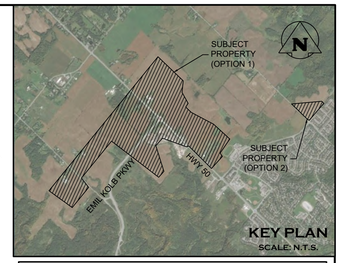
BOLTON NORTH HILL
TOWN OF CALEDON

Drawing

PRELIMINARY
EARTHWORKS MAP

CROZIER
CONSULTING ENGINEERS
2800 HIGH POINT DRIVE
SUITE 100
MILTON, ON. L9T 6P4
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S.V.N./J.B.	S.V.N./J.L.		708-3446
Check By	Check By	Scale	Drawing
J.L./J.B.	T.G.	1:6000	C705



LEGEND

- EXISTING MAJOR CONTOURS (2.0m)
- EXISTING MINOR CONTOURS (1.0m)
- PROPERTY BOUNDARY
- INTERNAL DRAINAGE AREA
- EXTERNAL DRAINAGE AREA
- WATERCOURSE (FROM TRCA MODEL)
- PROPOSED STORM SEWER
- CATCHMENT ID
- CN NUMBER (PERVIOUS AREAS) % IMPERVIOUS (IMPERVIOUS AREAS)
- CATCHMENT AREA (ha)
- PROPOSED OVERLAND FLOW DIRECTION

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Engineer

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BOLTON NORTH HILL
TOWN OF CALEDON

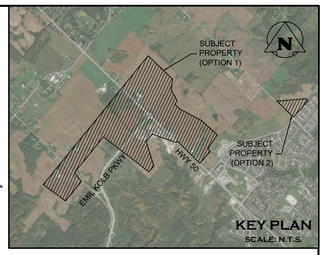
Drawing

PRELIMINARY POST DEVELOPMENT STORM DRAINAGE PLAN

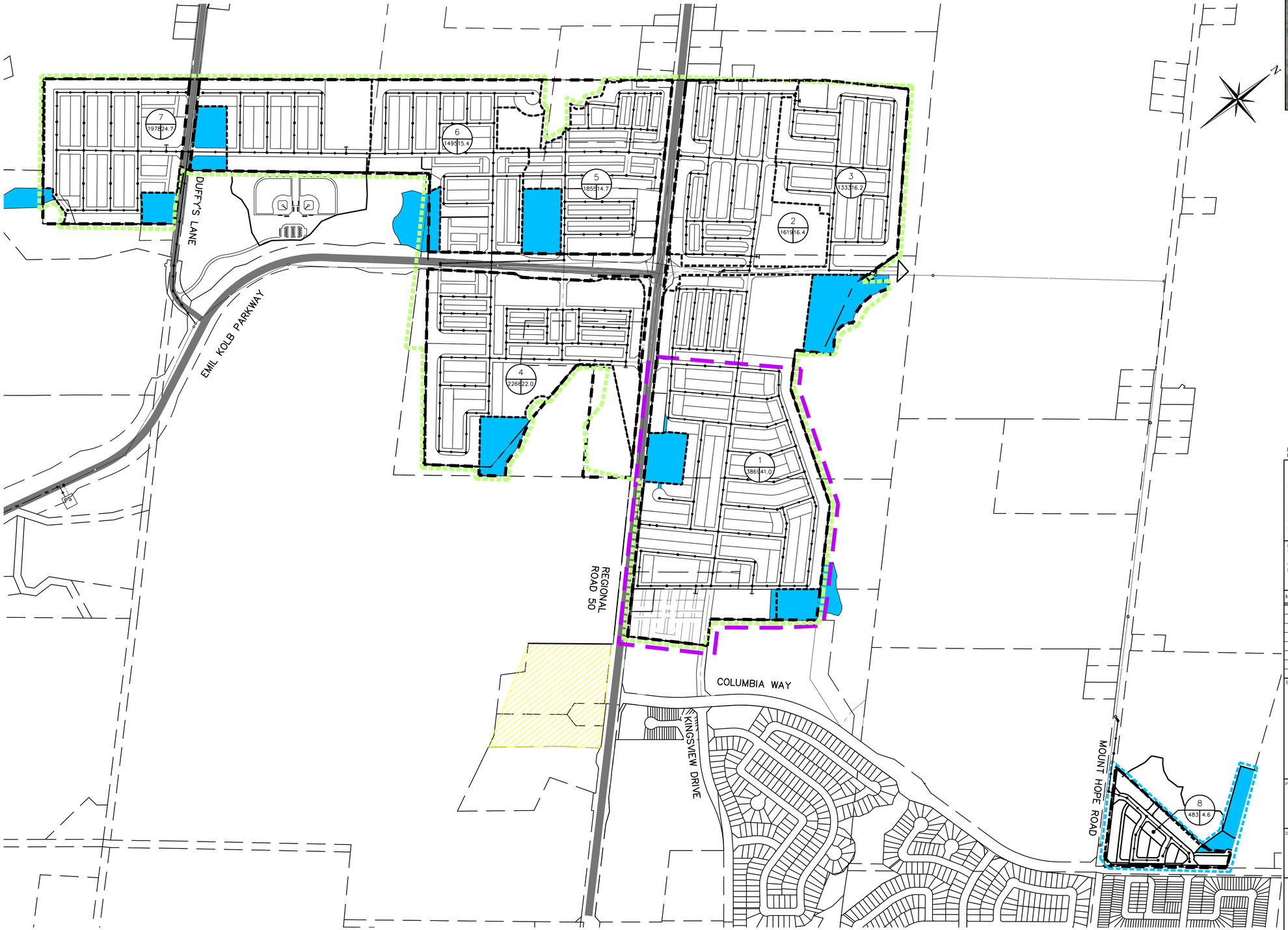
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S.v.N./J.B.	S.v.N./J.L.	708-3446
Check By	Check By	Scale / Drawing
J.L./J.B.	T.G.	1:6000 / C706

N:\700\708-Bolton NH Landowners Grp\3446-Bolton North Hill\CAD\Civil_Sheets\Figures\3446_C706 - STM.dwg, 2026-03-02 3:40:17 PM, AutoCAD PDF (General Documentation).pc3



KEY PLAN
SCALE: N.T.S.



LEGEND

- PROPOSED SANITARY SEWER
- PROPOSED 150mmø FORCEMAIN
- PS — PROPOSED PUMP STATION
- PROPOSED SANITARY CATCHMENT
- PROPOSED LOT LINES
- PROPERTY LIMITS
- REGION SANITARY SEWER
- EXISTING SANITARY SEWER
- 1 / 320 | 3.8 — CATCHMENT AREA ID / AREA (ha) / POPULATION
- ROPA 30 BOUNDARY
- OPTION 1
- OPTION 2

NOTE: ALL SANITARY PIPES ARE 250mmø @ 0.5% UNLESS STATED OTHERWISE

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Engineer

Engineer

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BOLTON NORTH HILL
TOWN OF CALEDON

Drawing

PRELIMINARY INTERNAL
SANITARY DRAINAGE PLAN

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S.v.N./J.B.	S.v.N./J.L.		708-3446
Check By	Check By	Scale	Drawing
J.L./J.B.	T.G.	1:6000	C707

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DESIGN OF SANITARY TRUNK SEWER ON HUMBER STATION CONSIDERS FUTURE DEVELOPMENT OF WHITEBELT LANDS IN TOWN OF CALEDON'S OFFICIAL PLAN (2024)

REFER TO PRELIMINARY EXTERNAL SANITARY SERVICING PLAN BY URBANTECH (2023) FOR SANITARY SERVICING IN OPTION 3 LANDS

CONNECTION TO MH5A OF PROPOSED HUMBER STATION ROAD TRUNK SEWER DESIGNED BY OTHERS INV. SE 248.80

PROPOSED 1200mm ø TRUNK SEWER

RAILWAY CROSSING

PROPOSED 525mmø TRUNK SEWER

PUMP STATION

DESIGN OF ULTIMATE SERVICING STRATEGY SUBJECT TO THE COMPLETION OF AN ENVIRONMENTAL ASSESSMENT (REGION OF PEEL)

REFER TO SANITARY DRAINAGE PLAN FOR PROPOSED SANITARY SERVICING IN OPTION 1 AND 2 LANDS

ULTIMATE SERVICING STRATEGY FOR OPTION 2 LANDS

INTERIM SERVICING STRATEGY FOR LANDS AT THE NORTHEAST CORNER OF COLUMBIA WAY & MOUNT HOPE. INTERIM STRATEGY TO BE DECOMMISSIONED AFTER ULTIMATE STRATEGY IS CONSTRUCTED.

1 — 150.4ha

2 — 122ha

3 — 173ha

4/5 — 169ha

A — 22.2ha

B — 15.3ha

C — 7.5ha

3.3ha

EMIL KOLB PARKWAY

DUFFY'S LANE

REGIONAL ROAD 50

HUMBER STATION ROAD

KING STREET

COLUMBIA WAY

MOUNT HOPE ROAD

GUARDHOUSE DR

MANSVIEW DRIVE

GREENPANE DRIVE

KING STREET E

BOLTON SANITARY PUMPING STATION

450mmø

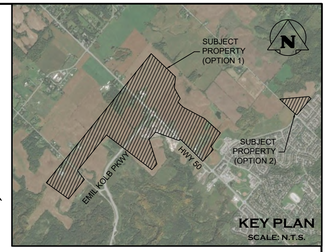
525mmø

500mmø FM

525mmø FM

500mmø FM

900mmø



KEY PLAN
SCALE: N.T.S.

LEGEND

- PROPOSED GRAVITY SEWER TRUNK ON EMIL KOLB PKWY
- EX. SANITARY SEWER UPGRADE TO 300mm
- EX. SANITARY SEWER UPGRADE TO 450mm
- EX. SANITARY SEWER WITH NO UPGRADE
- PROPOSED FORCEMAIN
- PROPOSED LOT LINES
- PROPERTY LIMITS
- PROPOSED SANITARY CATCHMENT
- 3 — AREA I.D.; 173ha — LAND AREA (ha)
- B.R.E.S. OPTION LANDS / B.R.E.S. ROUNDING OUT AREAS
- EX. SANITARY PUMPING STATION
- EX. SANITARY SEWER
- PROPOSED SANITARY TRUNK SEWER BY OTHERS

TO LIMIT THE EXTENT OF DISTURBANCE IN THE EXISTING SYSTEM SOUTH OF COLUMBIA WAY, THE CAPACITY ANALYSIS WAS LIMITED TO INDIVIDUAL LEGS OF SEWER, RATHER THAN UPGRADING THE ENTIRE SYSTEM DOWNSTREAM.

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Engineer | Engineer

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Project

BOLTON NORTH HILL
TOWN OF CALEDON

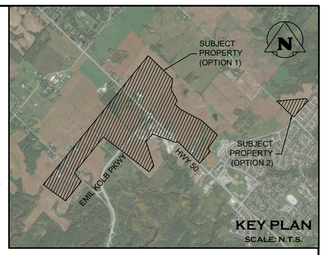
Drawing

PRELIMINARY EXTERNAL SANITARY SERVICING PLAN

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SUITE 100
MILTON, ON. L9T 6P4
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Check By	Check By	Scale	Drawing
J.L./J.B.	T.G.	1:10000	C708

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KEY PLAN
SCALE: N.T.S.

ELEVATED WATER TANK
(LOCATION TO BE CONFIRMED THROUGH COMPLETION OF ENVIRONMENTAL ASSESSMENT)

DUFFY'S LANE

PROPOSED 600ø WATERMAIN

EMIL KOLB PARKWAY

PROPOSED 600ø WATERMAIN

WATERMAIN TRUNK CONTINUES SOUTH ON EMIL KOLB PKWY WITH CONNECTION TO THE OPTION 3 LANDS TRUNK ON KING STREET

PROPOSED 400ø WATERMAIN

PROPOSED 400ø WATERMAIN

REGIONAL ROAD 50

PROPOSED 400ø WATERMAIN

PROPOSED 400ø WATERMAIN

COLUMBIA WAY

EX. 400ø WATERMAIN

KINGSVIEW DRIVE

EX. 400ø WATERMAIN

EX. 400ø WATERMAIN

PROPOSED 400ø WATERMAIN

DECOMMISSIONING OF EX. 300ø WATERMAIN TO BE CONFIRMED THROUGH COMPLETION OF ENVIRONMENTAL ASSESSMENT

EX. 300ø WATERMAIN

PROPOSED 400ø WATERMAIN

MOUNT HOPE ROAD

EX. 300ø WATERMAIN

EX. 300ø WATERMAIN

EX. 300ø WATERMAIN

LEGEND

- PROPOSED SUB-TRUNK WATERMAIN
- PROPOSED 400ø WATERMAIN
- PROPOSED 600ø WATERMAIN
- EXISTING WATERMAIN
- PROPOSED ELEVATED TANK
- PROPOSED CONNECTION TO EXISTING
- PROPOSED CONNECTION
- PROPOSED LOT LINES
- PROPERTY LIMITS

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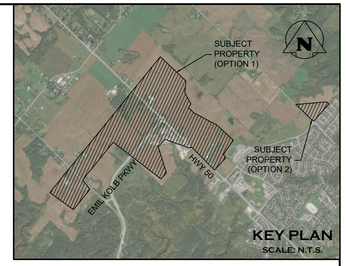
BOLTON NORTH HILL
TOWN OF CALEDON

Drawing

PRELIMINARY WATER DISTRIBUTION PLAN

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J.L./J.B.	T.G.	1:6000	C709



KEY PLAN
SCALE: N.T.S.

LEGEND

- PROPERTY BOUNDARY
- PROP. REGIONAL FLOODLINE
- EX. REGIONAL FLOODLINE
- PROP. REGIONAL FLOODLINE BUFFER (10m)
- EX. REGIONAL FLOODLINE BUFFER (10 m)
- TRCA WATERCOURSE
- EXISTING REGIONAL FLOODLINE – CHANNEL TO BE REALIGNED AND CONVERTED TO STORM SEWER UNDER THE PROPOSED CONDITION

CROSS-SECTION ID: 1830.512 — EXISTING REGIONAL WSEL 268.49 / PROPOSED REGIONAL WSEL 268.44

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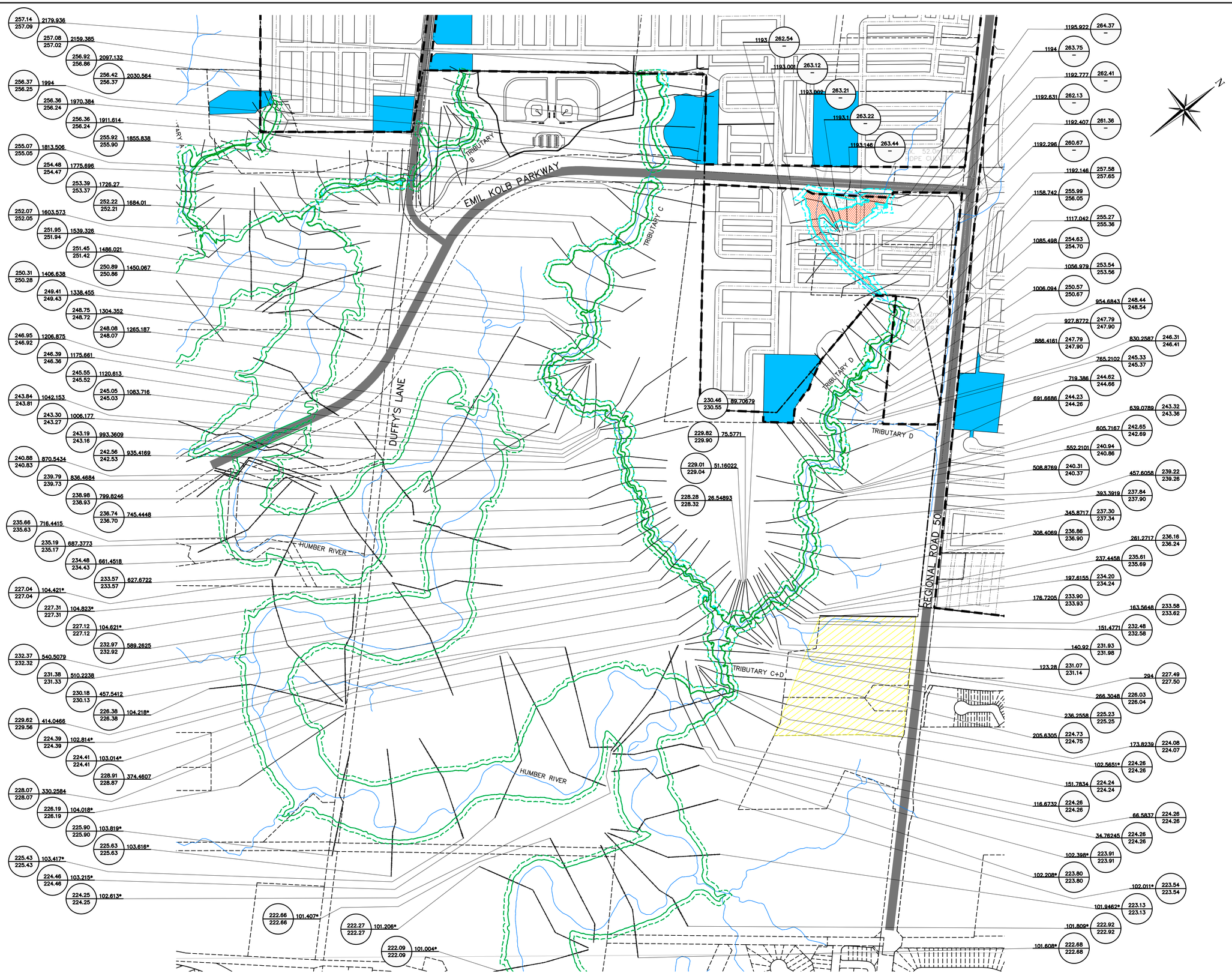
BOLTON NORTH HILL
TOWN OF CALEDON

Drawing

REGIONAL STORM
FLOODPLAIN MAPPING

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J.L.	T.G.	1: 4000	C710-FP1



LEGEND

- PROPERTY BOUNDARY
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CROSS-SECTION ID — EXISTING REGIONAL WSEL — PROPOSED REGIONAL WSEL

Example: 1830.512 — 268.49 / 268.44

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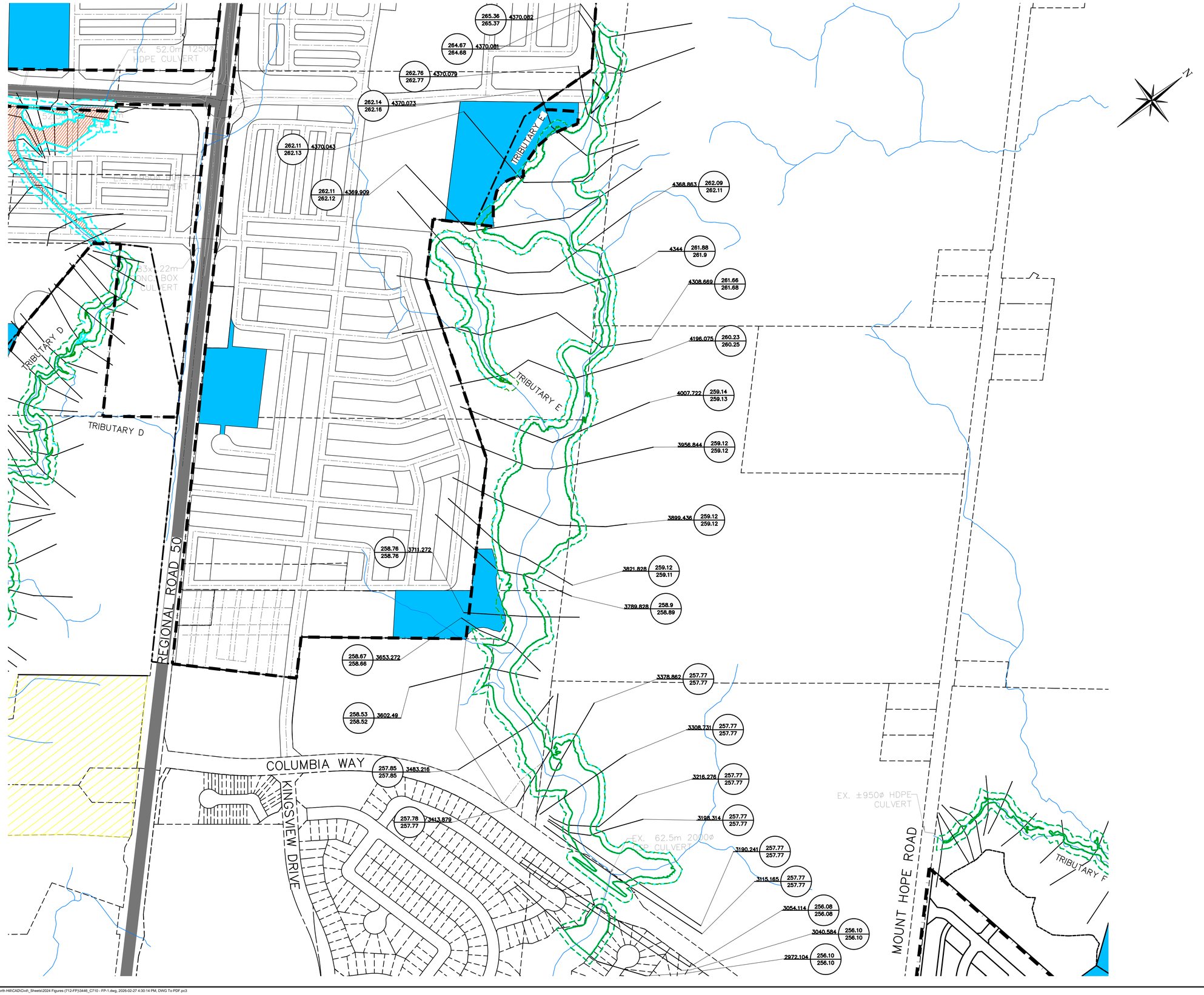
BOLTON NORTH HILL
TOWN OF CALEDON

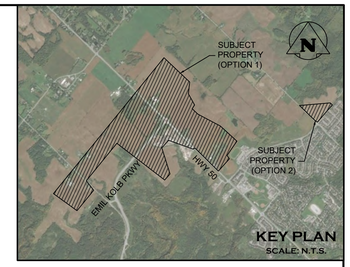
Drawing

REGIONAL STORM
FLOODPLAIN MAPPING

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Check By	Check By	Scale	Drawing
J.L.	T.G.	1: 4000	C710-FP2





LEGEND

- PROPERTY BOUNDARY
- PROP. REGIONAL FLOODLINE
- EX. REGIONAL FLOODLINE
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Engineer

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BOLTON NORTH HILL
TOWN OF CALEDON

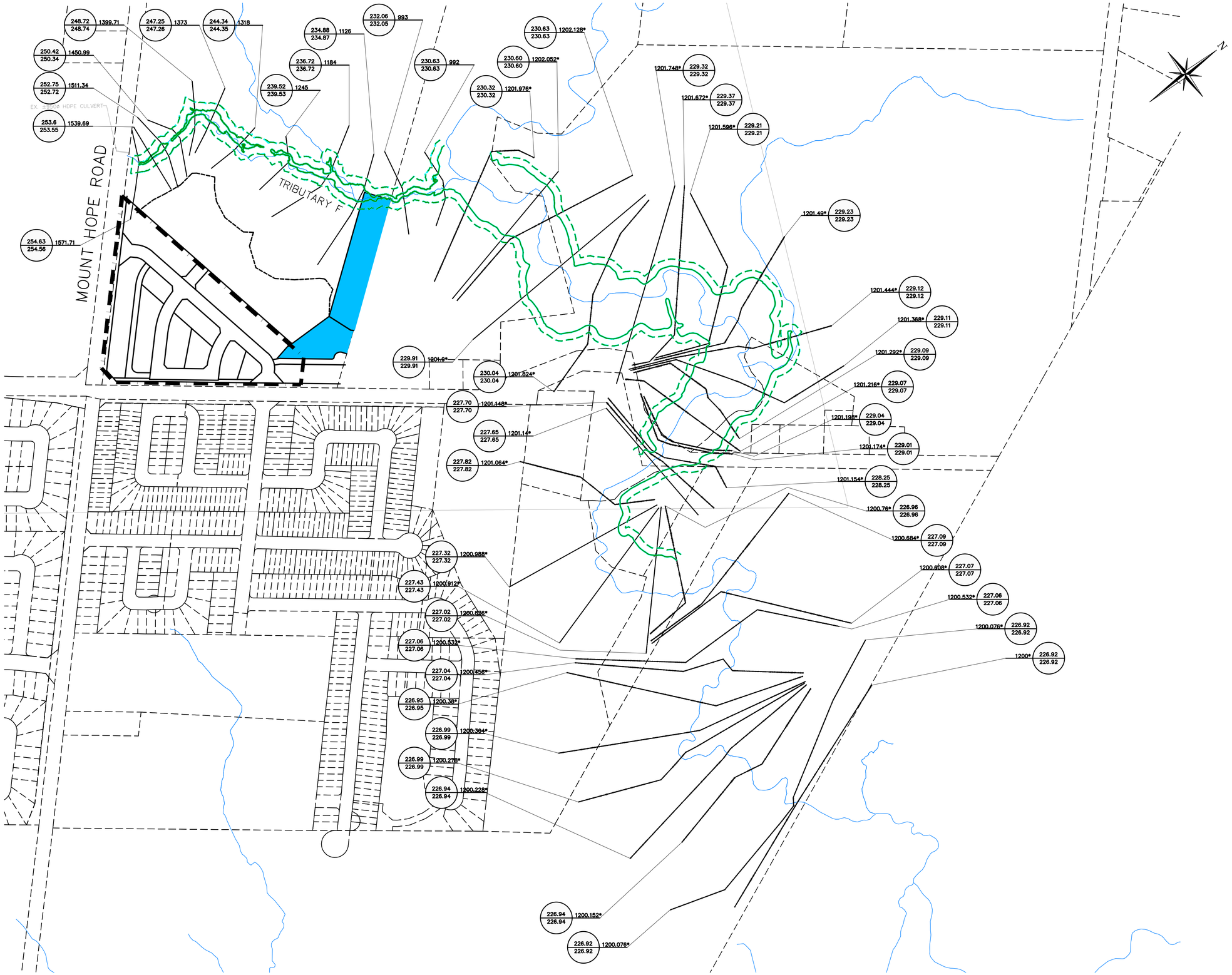
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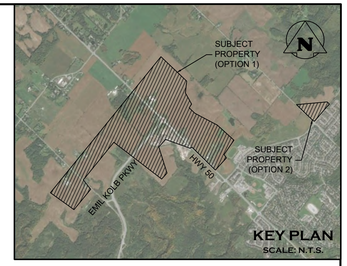
REGIONAL STORM
FLOODPLAIN MAPPING

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2800 HIGH POINT DRIVE
SUITE 100
MILTON, ON, L9T 6P4
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905 875 4915 F
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A.O.	J.W.		708-3446
Check By	Check By	Scale	Drawing
J.L.	T.C.	1: 3000	C710-FP3

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LEGEND

- PROPERTY BOUNDARY
- PROP. REGIONAL FLOODLINE
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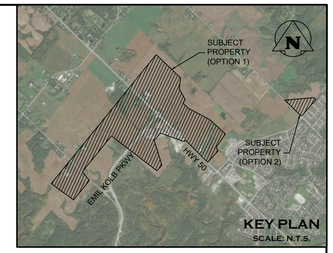
BOLTON NORTH HILL
TOWN OF CALEDON

Drawing

REGIONAL STORM
FLOODPLAIN MAPPING

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DIVERTED CATCHMENT AREA SUMMARY TABLE

TRIBUTARY	AREA ADDED TO TRIBUTARY (ha)	AREA REMOVED FROM TRIBUTARY (ha)	NET CHANGE (ha)
A	0.94	1.47	-0.53
B	5.15	0.94	4.21
C	2.23	6.56	-4.33
D	8.90	9.35	-0.45
E	7.12	6.02	1.10
COLD CREEK	0.00	0.00	NO CHANGE

LEGEND

- 198.00 — EXISTING MAJOR CONTOURS
- 198.50 — EXISTING MINOR CONTOURS
- ×199.11 — EXISTING ELEVATION
- PRE-DEVELOPMENT DRAINAGE AREA
- POST-DEVELOPMENT DRAINAGE AREA
- DIVERTED CATCHMENT AREA

1.47ha AREA DIVERTED FROM TRIBUTARY A TO TRIBUTARY B

3.68ha AREA DIVERTED FROM TRIBUTARY C TO TRIBUTARY B

3.44ha AREA DIVERTED FROM TRIBUTARY D TO TRIBUTARY E

0.94ha AREA DIVERTED FROM TRIBUTARY B TO TRIBUTARY A

2.23ha AREA DIVERTED FROM TRIBUTARY D TO TRIBUTARY C

6.02ha AREA DIVERTED FROM TRIBUTARY E TO TRIBUTARY D

HALF OF 5.77ha AREA (2.88 TOTAL) DIVERTED FROM TRIBUTARY C TO TRIBUTARY D

3.68ha AREA DIVERTED FROM TRIBUTARY D TO TRIBUTARY E

DUFFY'S LANE
EMIL KOLB PARKWAY
REGIONAL ROAD 50
COLUMBIA WAY
KINGSVIEW DRIVE
MOUNT HOPE ROAD
HUMBER RIVER
HUMBER RIVER
COLD CREEK
TRIBUTARY A
TRIBUTARY B
TRIBUTARY C
TRIBUTARY D
TRIBUTARY E
EX. 62.5m 2400ø CSP CULVERT
EX. 83.12m² CONCRETE BOX CULVERT
EX. ±950ø HDPE CULVERT

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Engineer | Engineer

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Project
BOLTON NORTH HILL
TOWN OF CALEDON

Drawing
STORMWATER DRAINAGE DIVERSION PLAN

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J.L./J.B.	T.G.	1:6000	C711

R.J. Burnside & Associates Limited 3 Ronell Crescent Collingwood ON L9Y 4J6 CANADA
telephone (705) 446-0515 **fax** (226) 526-9660 **web** www.rjburnside.com

BURNSIDE
[THE DIFFERENCE IS OUR PEOPLE]

Technical Memorandum – Zone 7 Demands

Date: July 14, 2025 **Project No.:** 300034976.0001

Project Name: Bolton Option 1, 2 & 3 Lands

Client Name: Option 1/2 Landowners Group and Bolton Option 3 Landowners Group

Submitted To: Ms. Miriam Polga, P.Eng., Region of Peel

Submitted By: Rhys Cavill, CET

Reviewed By: Jeff Langlois, P.Eng., MBA

1.0 Introduction

R.J. Burnside & Associates Limited (Burnside) has been retained by the Option 1/2 Landowners and Bolton Option 3 Landowners Group to complete a preliminary hydraulic water modelling analysis of future development areas within a new pressure Zone 7 in the community of Bolton. The water distribution system in Bolton is part of the Region of Peel (Region) water distribution system.

The Town previously completed the Bolton Residential Expansion Study (BRES) to determine the best approach to meet their urban boundary expansion needs. The BRES examined six different options for expansion of the Bolton settlement area and examined how each area could be serviced.

As had been determined in the previous BRES work, the Option 1, 2, 3, and Rounding Out Area (ROA) lands are generally outside of the range of elevations associated with Pressure Zone 6 of the existing water distribution infrastructure in Bolton. The northern section of Option 4 Lands will also generally be outside of the range of elevations associated with Pressure Zone 6. As such, development of Option 1, 2, 3, 4 and ROA lands will ultimately require the development of a new Pressure Zone 7 within Bolton. The land areas to be serviced by the future Zone 7 are shown in Figure 1 attached.

Previous studies completed in support of BRES identified a new Zone 7 booster pumping station. Ultimately, floating storage is proposed in the form of an elevated tank (ET) to provide storage for flow equalization, fire demands, and emergencies. Two ETs have been ultimately proposed. The first is to be situated in the vicinity of the northwest corner of the Option 3 lands

and the second to be situated generally to the north of the Option 1 lands. The general locations of the Zone 7 ETs are shown in Figure 1.

The purpose of this technical memorandum is to outline the assumptions made to determine the demands for each of the areas to be serviced by the new Zone 7 Booster Pumping Station (BPS). A version of this memo dated September 19, 2023 was previously circulated to the Region, and the memo has been updated to reflect feedback provided by the Region in an email dated November 21, 2023. In support of the design of the Booster Pumping station, a revised version of the memo was circulated to the Region January 2, 2024. The memo has been further updated to reflect direction from the Region with regard to fire flows (increase to 250 L/s) and detailing the demand implications associated with Zone 6 Lands that may transition to Zone 7.

2.0 Demand Calculations

The demand calculations for each of the land areas examined have been enclosed.

2.1 Peaking Factor

Peaking factors have been referenced from the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010, as requested by the Region in their November 21, 2023 email. The peaking factors used in the modelling are as follows:

- Residential Maximum Day Factor (MDF): 2
- Non-Residential Maximum Day Factor (MDF): 1.4
- Peak Hour Factor (PHF): 3

2.2 Water Demand

Water Demands for the model have been calculated based on the following per capita demands:

- Residential: 280 L/cap/d
- Employment: 300 L/cap/d

These per capita demands above are referenced from the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010 as requested by the Region in their November 21, 2023 email. Previously, this estimate was based on the Region of Peel 2020 Water and Wastewater Master Plan for Lake-Based Systems.

Preliminary Block Plans for the Option 1 2031 Lands and the Option 3 Lands were prepared by Gerrard Designs and provided for modelling purposes.

To calculate the population for Option 1 2031 and Option 3 using the block plans provided, the following densities were assumed:

- Townhouse: 3.4 ppu
- Single Detached: 4.2 ppu
- Apartment: 3.1 ppu
- Employment: 50 pp/ha

The densities were referenced from the Region of Peel Linear Wastewater Standards, 2023.

Block plans were not available for the Option 2 Lands, Option 4 Lands, 2051 Rounding Out Areas and Whitebelt Lands. If block plans are available in the future, the demands can be updated to reflect the proposed unit densities.

The following population densities were assumed in the absence of a block plan:

- Residential Lands: 141 pp/ha
- Employment: 50 jobs/ha

The Residential Lands density has been referenced from the GSAI report for the Caledon Station Secondary Plan, dated May 10, 2023. This is a more conservative number compared to the 67.5 pp/ha previously used referenced from the Region of Peel's Official Plan. The density is more reflective of the block plans that have been submitted for Zone 7 thus far.

The Employment density has been referenced from the Region of Peel Linear Wastewater Standards, 2023.

It is anticipated that as development design progresses the demand will be further refined. Refer to Appendix A for the demand calculations.

2.2.1 Option 1 Lands

The calculated population yield for the 2031 Option 1 Lands and the 2051 Option 1 Lands is based on the Functional Servicing and Stormwater Management Report Bolton Option 1 & Option 2 Lands, dated December 2021, prepared by Crozier.

The total estimated population yield for the Option 1 Lands are as follows:

- Residential: 14,749 persons
- Employment: 960 jobs (including schools)

The calculated 2051 Option 1 lands demands are:

- Average Day Demand (ADD): 51.1 L/s
- Maximum Day Demand (MDD): 100.3 L/s
- Peak Hour Demand (PHD): 153.4 L/s

32.03 ha of land has been allocated for development in 2031 and an additional 146.6 ha of land has been allocated for development in 2051. The total Option 1 Land area is approximately 178.6 ha.

2.2.2 Option 2 Lands

The Option 2 Lands are entirely within the 2051 buildout scenario.

A land area for the Option 2 Lands of 95 ha has been provided by the Region of Peel as part of the ROPA 30 Lands. The Functional Servicing and Stormwater Management Report Bolton Option 1 & Option 2 Lands, dated December 2021, prepared by Crozier provided a block plan for 4.3 ha of the Option 2 Land Area. The unit count from the Crozier report was taken and the balance of the land area from the Population Allocation Plan was used.

The total estimated population yield for the Option 2 Lands are as follows:

- Residential: 13,221 persons

The calculated 2051 Option 2 Lands demands are:

- Average Day Demand (ADD): 42.8 L/s
- Maximum Day Demand (MDD): 85.7 L/s
- Peak Hour Demand (PHD): 128.5 L/s

2.2.3 Option 3 Lands

The calculated population yield for the Option 3 Lands is based on the proposed Block Plan dated May 1, 2023, prepared by Gerrard Design. This block plan includes the 2051 full build out of the Option 3 Lands. The population yields assume employment at the school commercial areas at the ground level of the mixed-use blocks, and commercial at the GO Station blocks. The estimated population yields are as follows:

- Residential: 28,736 persons
- Employment: 1,927 jobs (including schools)

The calculated 2051 Option 3 Lands demands are:

- Average Day Demand (ADD): 99.8 L/s
- Maximum Day Demand (MDD): 195.6 L/s
- Peak Hour Demand (PHD): 299.4 L/s

The total Option 3 Land area is 182.4 ha.

2.2.4 Option 4 Lands

The area of Option 4 Lands that would be brought into Zone 7 was estimated based on the existing Zone 6 pressure elevation boundary of 259.1 m. Using the existing topography within Option 4, an approximate total land area of 28 ha has been established.

The existing land area that would be brought into Pressure Zone 7 was estimated by the Region of Peel. The Region then determined a total demand for the area based on historical billing records. These values were used in a previous version of this memo; however, this version incorporates the latest projections for these lands as included in the Region's model as provided to Hydratek.

The two existing areas that are assumed to be brought online to Zone 7 are generally west of Coleraine Drive. Largely on the south side of King Street and east of Highway 50, south of Columbia Way. These are areas where the Region has reported complaints by residents of inadequate pressure from time to time. While the Region has made improvements to the system to work toward addressing these concerns, the pressure remains at the lower end of the Region's preferred operating range, primarily due to elevation.

The 2031 demands provided by the Region of Peel are:

- Average Day Demand (ADD): 24.4 L/s
- Maximum Day Demand (MDD): 38.4 L/s
- Peak Hour Demand (PHD): 50.8 L/s

The 2051 demands provided by the Region of Peel are:

- Average Day Demand (ADD): 27.1 L/s
- Maximum Day Demand (MDD): 42.4 L/s
- Peak Hour Demand (PHD): 56.3 L/s

2.2.7 Whitebelt Lands

The Whitebelt Lands represent lands having future development potential beyond the 2051 planning horizon. Using ground elevation 259.1 m as an approximation of the Pressure Zone Boundary, it is estimated that approximately 290 ha of the Whitebelt Lands will be in Pressure Zone 7.

The total estimated population yield for the Whitebelt lands are as follows:

- Residential: 40,890

The calculated Whitebelt Lands Demands are:

- Average Day Demand (ADD): 132.5 L/s
- Maximum Day Demand (MDD): 265.0 L/s
- Peak Hour Demand (PHD): 397.5 L/s

2.2.8 Total Demands

The total estimated demands considered for this exercise are summarized in Table 1 below:

Table 1: Demand Summary

Land Area	Area (ha)	Pop.	Demand (L/s)			
			ADD	MDD	PHD[1]	PHD[2]
Option 1 ROPA 30 2031	32.1	2,963	9.6	19.2	28.8	30.7
Option 3 MZO 2031	77.47	19,133	62.2	122.3	186.7	196
ROA 2031 (ROA B)	8.5	1,199	3.9	7.8	11.7	12.5
2031 Demand w/o Existing Zone 6			**75.7**	**149.3**	**227.2**	**239.2**
2031 Ex. Zone 6 Lands ***	N/A	N/A	24.4	38.4	50.8[2]	50.8
Interim - Total 2031*	**118.07**	**23,294**	**100.1**	**187.7**	**278.0**	**290.0**
Interim - Total 2031* (ML/d)			**8.6**	**16.2**	**24.0**	**25.0**
Option 1 Full Buildout 2051	178.6	15,709	51.1	100.3	153.4	160
Option 2 Full Buildout 2051	95	13,221	42.8	85.7	128.5	137
Option 3 Full Buildout 2051	182.4	30,662	99.8	195.6	299.4	313
Option 4 Zone 7 Lands 2051	28	3,948	12.8	25.6	38.4	41
ROA 2051 (ROA A, B, C)	29.2	4,117	13.3	26.7	40.0	42.7
Whitebelt - post 2051	290	40,890	132.5	265.0	397.5	424
Ultimate w/o 2051 Zone 6			**352**	**699**	**1057**	**1118**
2051 Ex. Zone 6 Lands ***	N/A	N/A	27.1	42.4	56.3[2]	56.3
Ultimate – Total 2051**	**803.2**	**108,547**	**379**	**741**	**1113**	**1174**
Ultimate - Total Demands (ML/Day)**			**32.7**	**64.1**	**96.2**	**101.4**

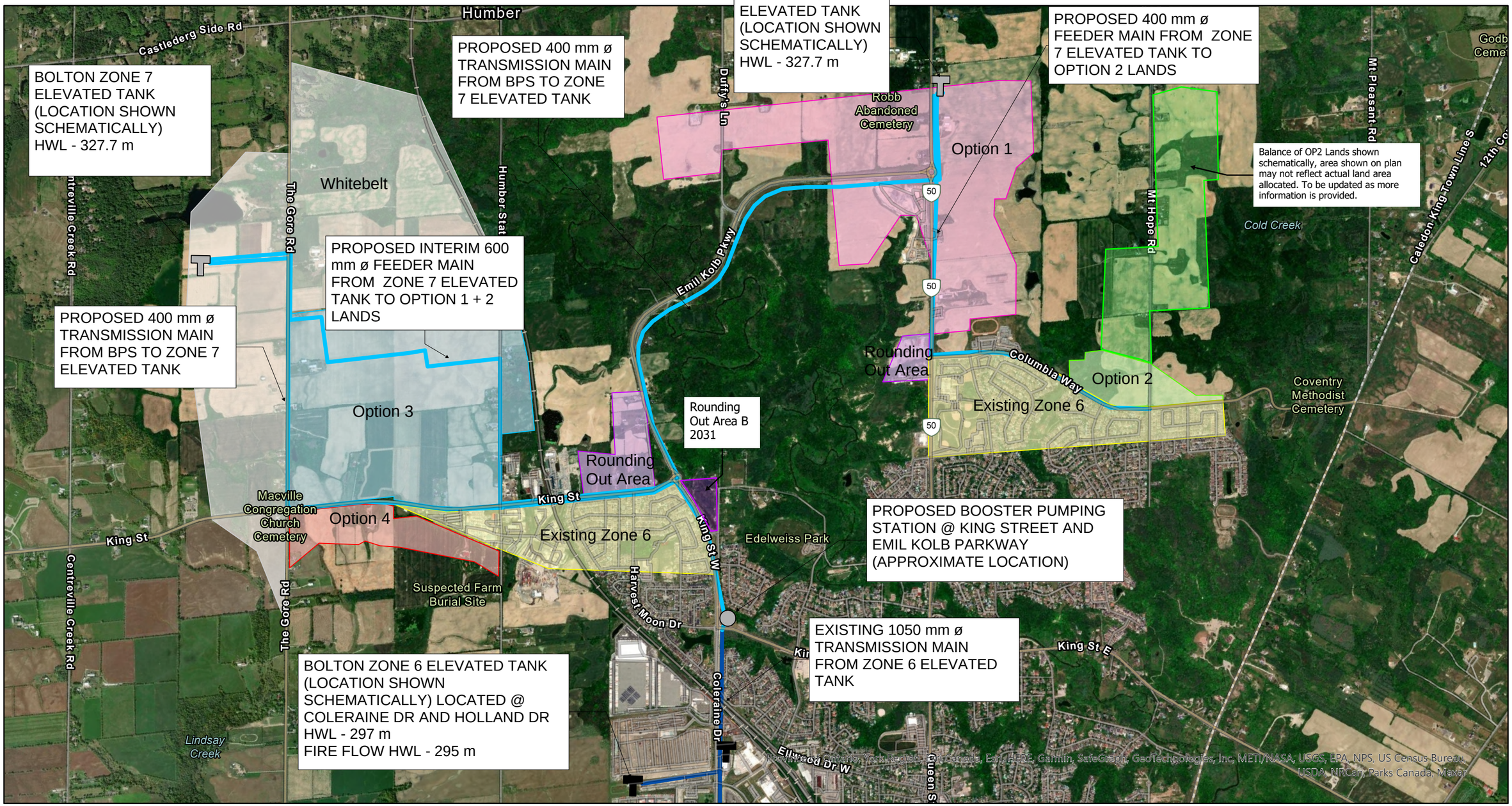
*Total 2031= Option 1 ROPA 30, Option 3 MZO, ROA 2031 and Existing Zone 6 Lands (Land Area and Service Population associated with Existing Zone 6 Lands to be incorporated not provided by the Region).

**Total 2051 = Option 1, 2, 3, 4 2051 Lands + ROA 2051 + Ex Zone 6 + Whitebelt.

Option 1 ROPA 30, Option 3 MZO and ROA 2031 included in Option 1 2051, Option 3 2051 Total and ROA 2051 Total.

*** Land Area and Service Population associated with Existing Zone 6 Lands to be incorporated not provided by the Region and hence not included in shown totals for area and populations.

1. Based on PHD as calculated in Appendices
2. Based on diurnal demand pattern hourly peaking factor of 1.6 from Region's InfoWater Model.



Datum:	
Coord. System:	
Projection:	
Central Meridian:	
False Easting: m	False Northing: m
Page Orientation: °	Scale Factor:

Grid North

BURNSIDE

Client

BOLTON OPTION 3 LANDOWNERS GROUP

Figure Title

BOLTON ZONE 7

WATER INFRASTRUCTURE

Drawn	Checked	Date	Figure No.
RW	JLL	2023/09/19	1
Scale: H 1:6,000		Project No.: 300034975	

File Path: C:\Users\RWalton\Documents\230510 Bolton OP3 Infowater Update\230510 Bolton OP3 Infowater Update.aprx Print Date: 2023/09/19 Time: 12:54 PM

Bolton Zone 7 Demand Summary

Prepared by:	RW
Checked by:	JLL
Project No:	300034976
Date:	1/2/2024

Assumptions

Water Demands

Residential	Residential Per Capita Flow	280	L/person/day	From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Max Day Factor	2		From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Peak Hour Factor	3		
Employment	Employment Per Capita Flow	300	L/employee/day	From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Max Day Factor	1.4		From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Peak Hour Factor	3		

Fire Flow

220 L/s	From Bolton Residential Expansion Study Infrastructure Report, June 16, 2014

Density

Unit Type	People Per Unit (ppu)*	People Per Hectare (pp/ha)**	
Townhomes	3.4	141	
Single Detached	4.2	141	
Large Apartment	3.1	141	
Employment (Commercial)		50	Region of Peel Linear Wastewater Standards, 2023
School (pp/school)		445	Region of Peel Linear Wastewater Standards, 2023. Schools designed for 850 students. Assume Sr Public Schools

*Region of Peel Linear Wastewater Standards, 2023

** from GSAI Report for Caledon Station Secondary Plan, dated May 10, 2023

assumptions used in absences of a development plan

Demands per Land Area

	Area (ha)	Population	Demand L/s			
			ADD	MDD	PHD	
Option 1 ROPA 30 2031	32.1	2963	9.6	19.2	28.8	
Option 1 Full Buildout 2051	178.6	15709	51.1	100.3	153.4	
Option 2 Full Buildout 2051	95	13221	42.8	85.7	128.5	
Option 3 MZO 2031	77.47	19133	62.2	122.3	186.7	
Option 3 Full Buildout 2051	182.4	30662	99.8	195.6	299.4	
Option 4 Zone 7 Lands 2051	28	3948	12.8	25.6	38.4	
ROA 2031	8.5	1199	3.9	7.8	11.7	
ROA 2051	29.2	4117	13.3	26.7	40.0	
Existing Zone 6 Lands	0	0	16.0	32.0	48.0	*demands based on Historical Billing Records from Region
Whitebelt - post 2051	290	40890	132.5	265.0	397.5	
Interim - Total 2031*	**118.07**	**23294**	**91.7**	**181.3**	**275.2**	
Ultimate - Total 2051**	**803.2**	**108547**	**368.4**	**730.9**	**1105.3**	
Ultimate 2051 MDD (ML/Day)			**31.8**	**63.1**	**95.5**	

*Total 2031 = Option 1 ROPA 30, Option 3 MZO, ROA 2031 and Existing Zone 6 Lands

**Total 2051 = Option 1, 2, 3,4 2051 Lands + ROA 2051+Ex. Zone 6 + Whitebelt

Option 1 ROPA 30,Option 3 MZO and ROA 2031 included in Option 1 2051, Option 3 2051 and ROA 2051 total

Bolton Option 1 2031 Zone 7 Demand Calculations

Prepared by:	RW
Checked by:	JLL
Project No:	300034976
Date:	1/2/2024

BURNSIDE

Assumptions

Water Demands

Residential	Residential Per Capita Flow	280	L/person/day	From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Max Day Factor	2		From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Peak Hour Factor	3		
Employment	Employment Per Capita Flow	300	L/employee/day	From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Max Day Factor	1.4		From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Peak Hour Factor	3		

Fire Flow

220 L/s	From Bolton Residential Expansion Study Infrastructure Report, June 16, 2014

Density

Unit Type	People Per Unit (ppu)*	People Per Hectare (pp/ha)**	
Townhomes	3.4	141	
Single Detached	4.2	141	
Large Apartment	3.1	141	
Employment (commercial)		50	Region of Peel Linear Wastewater Standards, 2023
School (pp/school)		445	Region of Peel Linear Wastewater Standards, 2023. Schools designed for 850 students. Assume Sr Public Schools

*Region of Peel Linear Wastewater Standards, 2023
** from GSAI Report for Caledon Station Secondary Plan, dated May 10, 2023
assumptions used in absences of a development plan

Demand Allocation

2031 Buildout

				L/ha/s			L/s		
Unit Type	Total Area (ha)*	# of units*	Population	ADD	MDD	PHD	ADD	MDD	PHD
Single Detached		288	1210				3.9	7.8	11.8
Townhomes		473	1608				5.2	10.4	15.6
Apartments		37	115				0.4	0.7	1.1
Commercial	0.6		30				0.10	0.1	0.3
Total	**32.1**	**798**	**2963**	0.3	0.4	0.9	**9.6**	**19.2**	**28.8**

*From Functional Servicing and Stormwater Management Report Bolton Option 1 & Option 2 Lands, dated Dec 2021, prepared by Crozier
Appendix B3- Draiange Area Population Breakdown (OP1 ROPA 30 Lands correlate with Sanitary Drainage Area 1)

Bolton Option 1 2051 Zone 7 Demand Calculations

Prepared by:	RW
Checked by:	JLL
Project No:	300034976
Date:	1/2/2024

BURNSIDE

Assumptions

Water Demands

Residential	Residential Per Capita Flow	280	L/person/day	From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Max Day Factor	2		From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Peak Hour Factor	3		
Employment	Employment Per Capita Flow	300	L/employee/day	From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Max Day Factor	1.4		From the Region of Peel Linear Infrastructure Watermain Design Criteria, 2010
	Peak Hour Factor	3		

Fire Flow

220 L/s	From Bolton Residential Expansion Study Infrastructure Report, June 16, 2014

Density

Unit Type	People Per Unit (ppu)*	People Per Hectare (pp/ha)**	
Townhomes	3.4	141	
Single Detached	4.2	141	
Large Apartment	3.1	141	
Employment (commercial)		50	Region of Peel Linear Wastewater Standards, 2023
School (pp/school)		445	Region of Peel Linear Wastewater Standards, 2023. Schools designed for 850 students. Assume Sr Public Schools

*Region of Peel Linear Wastewater Standards, 2023
** from GSAI Report for Caledon Station Secondary Plan, dated May 10, 2023
assumptions used in absences of a development plan

Demand Allocation

Buildout Year	Unit Type	Area (ha)	# of units*	Population	L/ha/s			L/s		
					ADD	MDD	PHD	ADD	MDD	PHD
2051	Single Detached	46.04	1554	6527				**21.2**	**42.3**	**63.5**
	Townhomes	33.04	2038	6929				**22.5**	**44.9**	**67.4**
	Apartment Units	4.37	417	1293				**4.2**	**8.4**	**12.6**
	Commercial	1.4		70				**0.2**	**0.3**	**0.7**
	School (2 based on Block Plan)	4.9		890				**3.1**	**4.3**	**9.3**
Total	****	**178.6**	**4009**	**15709**	**0.3**	**0.6**	**0.9**	**51.1**	**100.3**	**153.4**

*From Functional Servicing and Stormwater Management Report Bolton Option 1 & Option 2 Lands, dated Dec 2021, prepared by Crozier for Option 1 lands - Appendix B2
**total 178.6 ha area includes non serviceable area such as: park, SWMP, Open space, Roads, Elevated Tank etc, not listed

Pressure Zone	Description	Location	Region DC Study IDs		Year Active		
			Component	Project	Region DC Study	As-Received Model	Analysis
	600 mm King PZ7 watermain	King (Emil Kolb to west of Humber Station)	53992	24-1190	2026	2027	2031
	400 mm Highway 50 PZ7 watermain	Highway 50 (Emil Kolb to Columbia _	54021	24-1191	2026	2027	2031
	400 mm Columbia PZ7 watermain	Columbia (Highway 50 to Mount Hope)	56947	24-1191	2026	2027	2031
	900 mm King PZ7 watermain	King (The Gore to west of Humber Station)	31439	29-1299	2032	n/a	2032
	900 mm Macville PZ7 watermain	King and The Gore (Sandhill PS to Macville ET)	31439	29-1299	2032	n/a	2032
	750 mm Castlederg PZ7 watermain	King, Emil Kolb, and new road (Humber Station to Castlederg ET)	31817	29-1299	2035	n/a	2036
	400 mm Mount Hope PZ7 watermain	Mount Hope (Columbia to Option 2B lands)	n/a	n/a	n/a	2041	2041
	400/300 mm Northeast PZ7 watermains	Various around Option 1 and Option 2B lands	n/a	n/a	n/a	2041	2051

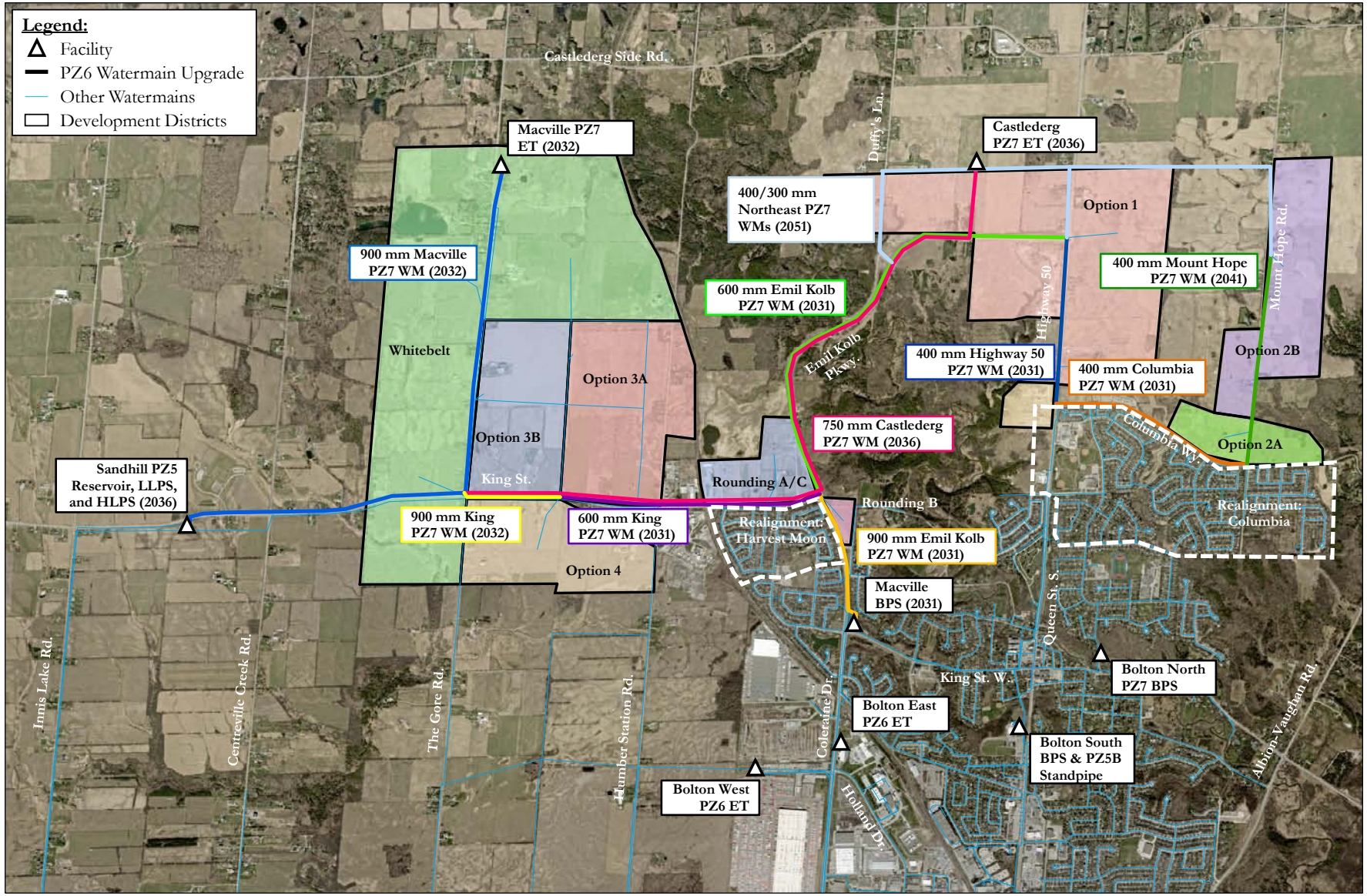


Figure 7: Background – map of Bolton PZ7 water system upgrades from Region DC study

Table 7: Methodology – development statistics and water demand estimates

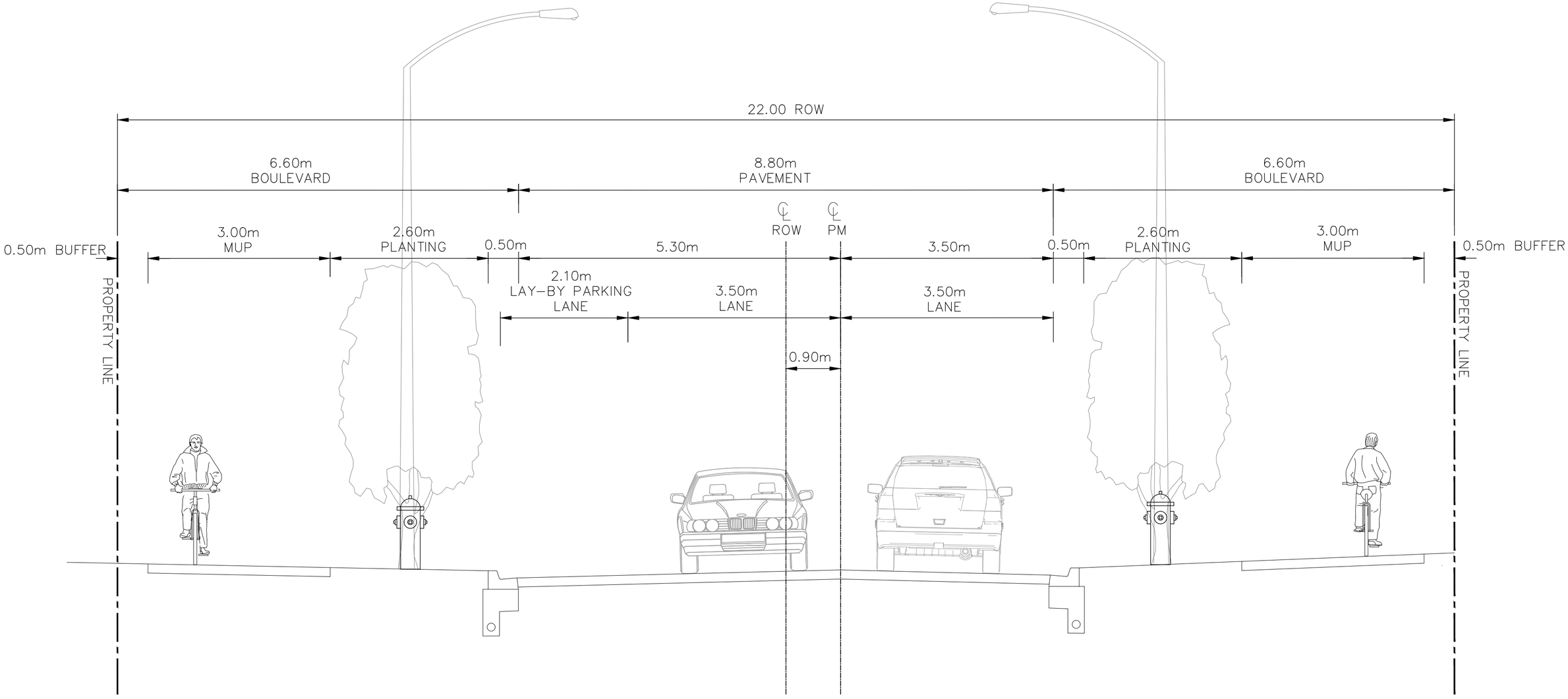
Development Land	Time Frame	Area (ha)	Population	Demand (L/s)				Elevation Range (m)	
				ADD[1]	MDD[1]	PHD[1]	PHD[2]	Min.	Max.
Option 1A	2031	32.1	2,960	9.6	19.2	28.8	30.7	258	269
Option 1B	2051	146.5	12,700	41.5	81.1	125	130	246	277
Option 1 Total	-	178.6	15,700	51.1	100	153	160	246	277
Option 2A	2051	23.8	3,310	10.7	21.4	32.1	34.3	238	263
Option 2B	2051	71.3	9,920	32.1	64.3	96.4	103	235	265
Option 2 Total	2051	95.0	13,200	42.8	85.7	129	137	235	265
Option 3A	2031	77.5	19,100	62.2	122	187	196	261	277
Option 3B	2051	104.9	11,500	37.6	73.3	113	117	261	281
Option 3 Total	-	182.4	30,700	99.8	196	299	313	261	281
Option 4	2051	28.0	3,950	12.8	25.6	38.4	41.0	255	266
Rounding 'A/C'	2051	20.7	2,920	9.4	18.9	28.3	30.2	248	266
Rounding 'B'	2031	8.5	1,200	3.9	7.8	11.7	12.5	247	263
Rounding Total	-	29.2	4,120	13.3	26.7	40.0	42.7	247	266
Whitebelt	2051	290.0	40,900	133	265	398	424	250	299
Total 2031	**-**	**118.1**	**23,300**	**75.7**	**149**	**227**	**239**	**-**	**-**
Total 2051	**-**	**803.2**	**108,500**	**352**	**699**	**1057**	**1118**	**-**	**-**

1 Demand estimates were provided by Burnside and are based on the Region's design criteria.
2 Based on (i) MDD estimates provided by Burnside and (ii) a diurnal demand pattern hourly peaking factor of 1.60 from the Region's InfoWater model.

Table 8: Methodology – Bolton PZ7 system water demands

Model Year	Demand Condition	PZ7 Demand (L/s) by Area				PZ7 Demand (ML/d) by Area			
		Harvest Moon	Columbia	New PZ7	Total	Harvest Moon	Columbia	New PZ7	Total
2026*	MHD	5.02	11.0	0	16.0	0.43	0.95	0	1.39
	ADD	7.43	16.0	0	23.4	0.64	1.38	0	2.03
	MDD	11.4	25.4	0	36.9	0.99	2.20	0	3.18
	PHD	14.7	33.7	0	48.4	1.27	2.91	0	4.18
2031	MHD	5.24	11.3	38.4	54.9	0.5	0.98	3.31	4.74
	ADD	7.82	16.6	76.5	101	0.68	1.43	6.61	8.72
	MDD	12.0	26.4	149	188	1.04	2.28	12.9	16.2
	PHD	15.5	35.3	239	290	1.34	3.05	20.6	25.0
2036	MHD	5.93	12.2	72.7	90.7	0.5	1.05	6.28	7.84
	ADD	9.19	18.3	145	173	0.79	1.58	12.5	14.9
	MDD	14.4	29.5	287	331	1.24	2.54	24.8	28.6
	PHD	18.8	40.0	459	518	1.63	3.46	39.6	44.7
2041	MHD	5.40	12.2	107	125	0.47	1.05	9.25	10.8
	ADD	8.14	18.3	214	240	0.70	1.58	18.5	20.8
	MDD	12.7	28.2	424	465	1.10	2.44	36.6	40.2
	PHD	16.6	37.5	679	733	1.44	3.24	58.6	63.3
2051	MHD	5.56	12.4	176	194	0.48	1.07	15.2	16.8
	ADD	8.47	18.6	352	379	0.73	1.61	30.4	32.8
	MDD	13.3	29.1	699	741	1.15	2.52	60.4	64.1
	PHD	17.8	38.5	1118	1174	1.53	3.32	96.6	101

* 2026 demands represent the existing Harvest Moon and Columbia areas that will be realigned to PZ7.



CALEDON CROSS–SECTION – 22m ROW
RESIDENTIAL MINOR COLLECTOR WITH MULTI–USE PATH BOTH SIDES
SINGLE PARKING LANE

N.T.S.

No.	ISSUE	DATE: YYYY/MM/DD
0	ISSUED FOR 1st SUBMISSION	YYYY/MM/DD

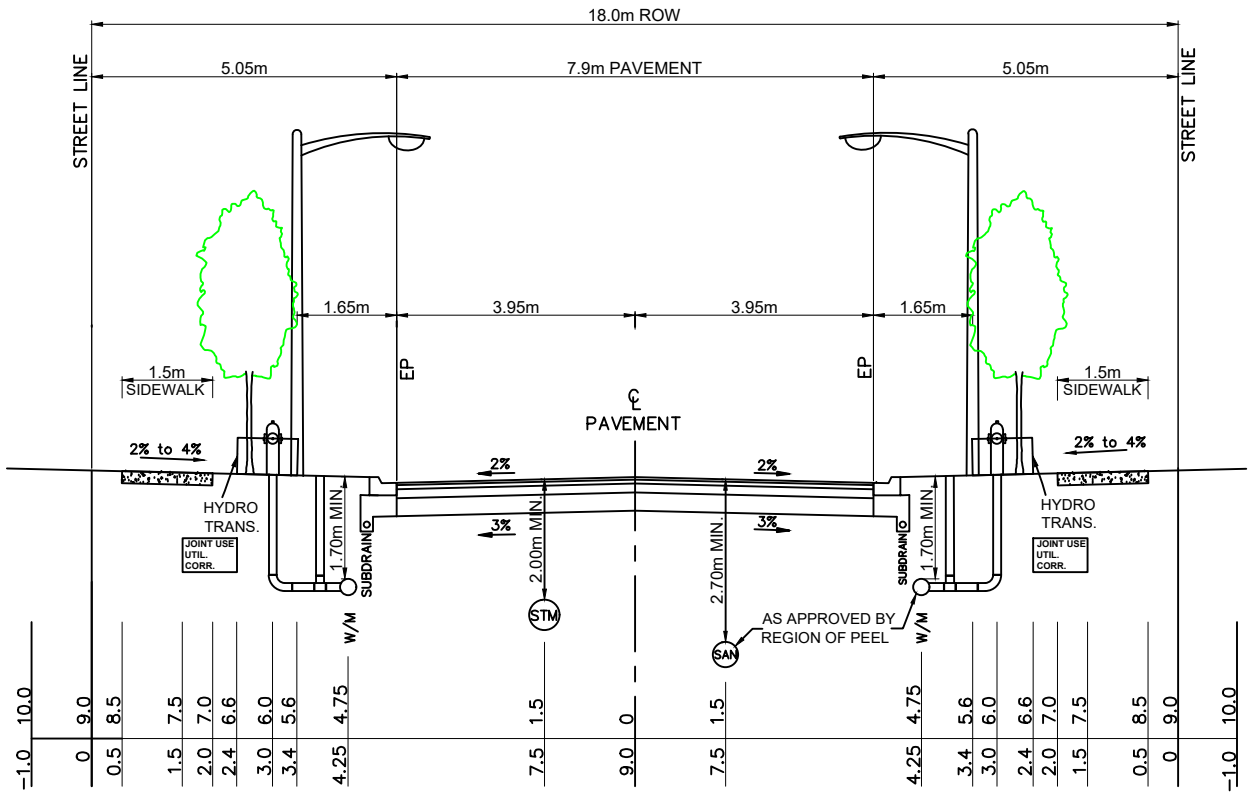
PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

Project
BOLTON NORTH HILL LANDOWNERS GROUP
TOWN OF CALEDON, REGION OF PEEL

Drawing
MINOR COLLECTOR CROSS–SECTION

CROZIER
CONSULTING ENGINEERS

Drawn By	Design By	Project
J.W.	J.W./M.L.	0708–3446
Check By	Check By	Drawing
J.W/M.L.	M.L./K.K.	T–102



NOTES

1. WATERMAIN TO HAVE MINIMUM COVER OF 1.7m.
2. UTILITY CORRIDOR TO HAVE A MINIMUM COVER OF 0.9m.
3. TREES TO BE PLACED IN LOCATIONS PER APPROVED LANDSCAPE PLAN.
4. STREETLIGHT FIXTURE PER APPROVED TOWN STANDARD.
5. THE FOLLOWING IS A MINIMUM ROAD BASE AND WILL REQUIRE A SOILS REPORT VERIFICATION
   - 40 mm HL3
   - 65 mm HL8
   - 150 mm GRANULAR "A"
   - 300 mm GRANULAR "B"
6. THE BOULEVARDS REQUIRE A MINIMUM OF 300mm OF TOPSOIL AND NURSERY SOD.
7. ON A CRESCENT THE WATERMAIN SHALL BE PLACED ON THE OUTSIDE.
8. FULL LENGTH MINIMUM 100 MM DIA.SUB-DRAINS C/W FILTERCLOTH SHALL BE INSTALLED, AS PER APPROVED TOWN OF CALEDON STANDARD NO. 219.
9. SUB-GRADE SHALL BE COMPACTED TO A MINIMUM 95% OF S.P.D. AT OPTIMUM MOISTURE CONTENT.
10. WHERE POSSIBLE MANHOLE LIDS TO BE LOCATED OUT OF TIRE LANE OF TRAFFIC.
11. LONG DIMENSION OF TRANSFORMER TO BE PARALLEL TO STREETLINE.

TOWN OF CALEDON

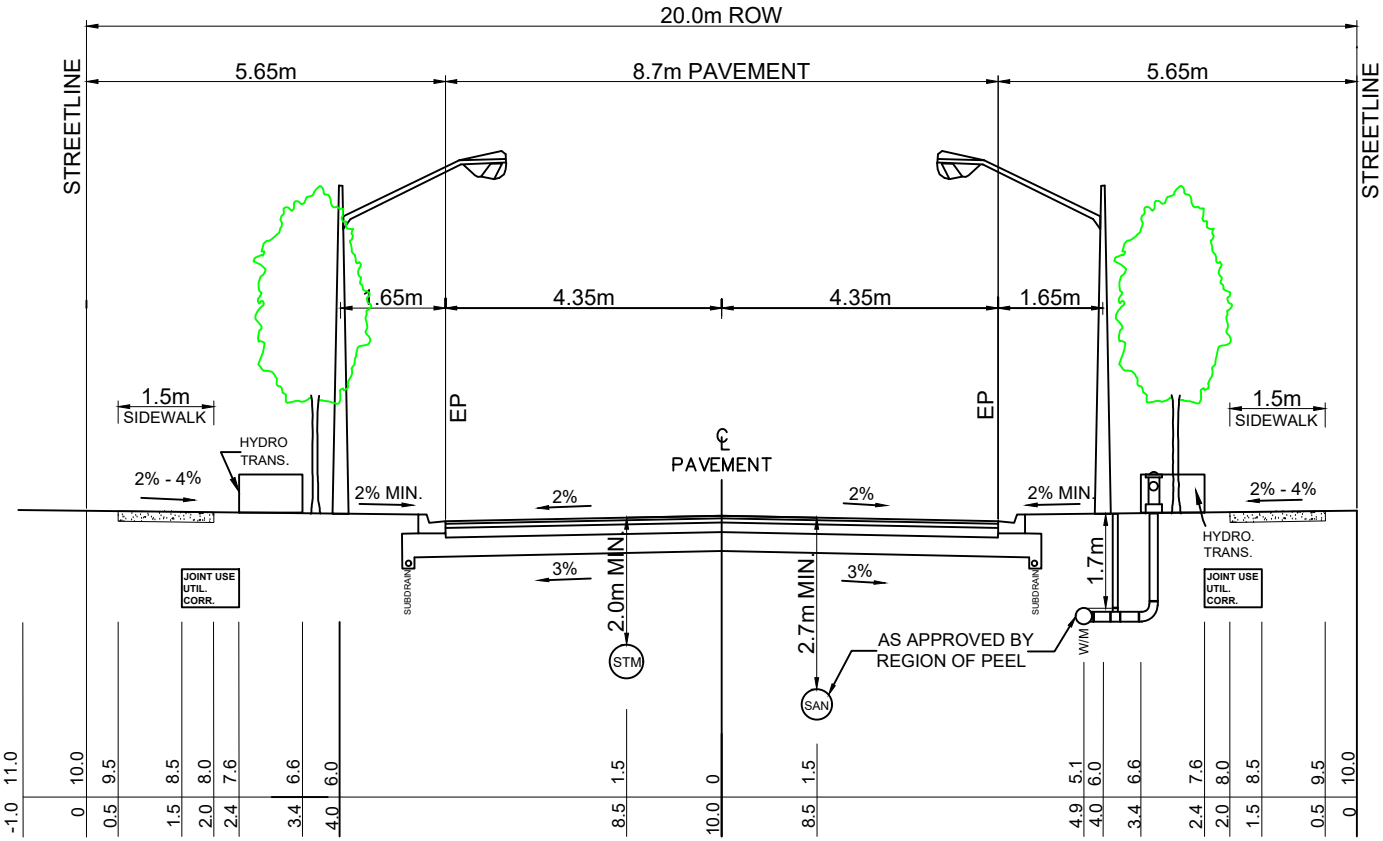
18.0m LOCAL ROAD
8.5m ROADWAY (7.9m PAVEMENT)

NO.	REVISION	APR'D	DATE
4	TEXT AND SLOPE REVISIONS		MAY 19
3	DIMENSION AND TEXT REVISION		JAN. 09
2	DIMENSION AND LAYOUT REVISION		JULY 08
1	DIMENSION EDIT		JUNE 08

APR'D: C.C. | DATE: JUNE 08

DRAWN: | SCALE: N.T.S.

STANDARD No. 202



NOTES:

1. UTILITY CORRIDOR TO HAVE A MINIMUM COVER OF 0.9m.
2. WATERMAIN TO HAVE MINIMUM COVER OF 1.7m.
3. THE FOLLOWING IS A MINIMUM ROAD BASE AND WILL REQUIRE A SOILS REPORT VERIFICATION
   40mm HL3
   65mm HL8
   150mm GRANULAR "A"
   300mm GRANULAR 'B'
4. THE BOULEVARDS REQUIRE A MINIMUM OF 300mm OF TOPSOIL AND NURSERY SOD
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6. STREETLIGHT FIXTURE PER APPROVED TOWN STANDARD.
7. ON A CRESCENT THE WATERMAIN SHALL BE PLACED ON THE OUTSIDE.
8. FULL LENGTH MINIMUM 100MM DIA.SUB-DRAINS C/W FILTERCLOTH SHALL BE INSTALLED TO TOWN OF CALEDON STANDARD NO. 218.
9. SUB-GRADE SHALL BE COMPACTED TO A MINIMUM 95% OF S.P.D. AT OPTIMUM MOISTURE CONTENT.
10. WHERE POSSIBLE MANHOLE LIDS TO BE LOCATED OUT OF TIRE LANE OF TRAFFIC.
11. ESTATE RESIDENTIAL 4.0 METERS LANE WIDTH.

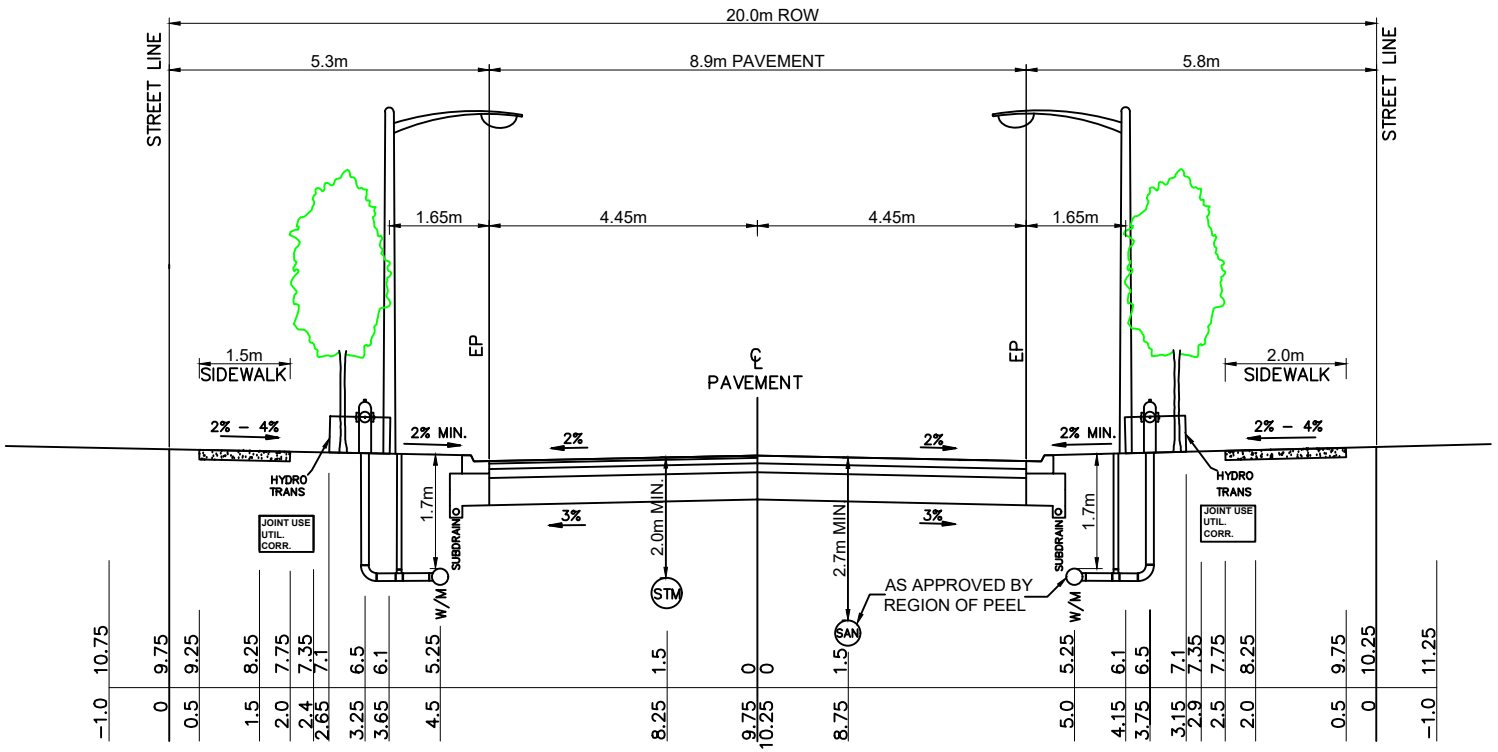
TOWN OF CALEDON

20.0m LOCAL URBAN/RURAL
9.3m ROADWAY (8.7m PAVEMENT)

NO.	REVISION	APR'D	DATE
5	TEXT AND SLOPE REVISIONS		JULY 19
4	DIMENSION AND TEXT REVISION		JAN. 09
3	DIMENSION, LAYOUT REVISION		JULY 08
2	DIMENSION EDIT, STD No. 201 NOW 203		JUNE 08
1	OFFSETS		APRIL 08

APR'D: C.C. DATE: 19/01/00
DRAWN: BJM SCALE: N.T.S.

STANDARD No. 203



NOTES

1. UTILITY CORRIDOR TO HAVE A MINIMUM COVER OF 0.9 m.
2. WATERMAIN TO HAVE A MINIMUM COVER OF 1.7 m.
3. THE FOLLOWING IS A MINIMUM ROAD BASE AND WILL REQUIRE A SOILS REPORT VERIFICATION
   - 40 mm HL3
   - 90 mm HL8
   - 150 mm GRANULAR "A"
   - 450 mm GRANULAR "B"
4. THE BOULEVARDS REQUIRE A MINIMUM OF 300mm OF TOPSOIL AND NURSERY SOD.
5. TREES TO BE PLACED IN LOCATIONS PER APPROVED LANDSCAPE PLANS.
6. STREETLIGHT FIXTURE PER APPROVED TOWN STANDARD.
7. FULL LENGTH MINIMUM 100 MM DIA.SUB-DRAINS C/W FILTERCLOTH SHALL BE INSTALLED, AS PER APPROVED TOWN OF CALEDON STANDARD NO. 218.
8. SUB-GRADE SHALL BE COMPACTED TO A MINIMUM 95% OF S.P.D. AT OPTIMUM MOISTURE CONTENT.
9. WHERE POSSIBLE MANHOLE LIDS TO BELOCATED OUT OF TIRE LANE OF TRAFFIC.
10. LONG DIMENSION OF TRANSFORMER TO BE PARALLEL TO STREETLINE.
11. 1.0 m SPLASH PAD TO BE PATTERNED CONCRETE AS PER APPROVED TOWN OF CALEDON STANDARD NO. 903.

TOWN OF CALEDON

20.0m NEIGHBOURHOOD COLLECTOR
9.5m ROADWAY (8.9m PAVEMENT)

NO.	REVISION	APR'D	DATE
4	TEXT AND SLOPE REVISION		JULY 19
3	DIMENSION AND TEXT REVISION		JAN. 09
2	DIMENSION AND LAYOUT REVISION		JULY 08
1	DIMENSION EDIT, STD No. 206 NOW 204		JUNE 08

APR'D: C.C. | DATE: JUNE 08
DRAWN: | SCALE: N.T.S.

STANDARD No. 204

APPENDIX E
Preliminary Pond Sizing Calculations

Project Name:	Bolton Hill
Project Number:	1433
Designed By:	LH
Checked By:	AVN
Date	20-Mar-26

David Schaeffer Engineering Limited
600 Alden Road, Suite 606
Markham, ON L3R 0E7

ALLOWABLE POND VOLUME AND DISCHARGE RATES - POND 07 - Chicago 4hr

Post Drainage Area	34.45	ha
Post Imp %	74.3	%

Design Storm	Target Release Rate	Required Pond Volume
	m^3/s	m^3
2 Year Design Storm CHI-4hr	0.038	9171
5 Year Design StormCHI-4hr	0.080	13831
100 Year Design StormCHI-4hr	0.979	20994
Regional Design Storm	2.567	28938

1) Post development drainage areas and target release rates taken from FS&SWMR completed by CF. Crozier, March 2026

Project Name:	Bolton Hill
Project Number:	1433
Designed By:	LH
Checked By:	AVN
Date	20-Mar-26

David Schaeffer Engineering Limited
600 Alden Road, Suite 606
Markham, ON L3R 0E7

ALLOWABLE POND VOLUME AND DISCHARGE RATES - POND 09 - Chicago 4hr

Post Drainage Area	17.66	ha
Post Imp %	74.3	%

Design Storm	Target Release Rate	Required Pond Volume
	m^3/s	m^3
2 Year Design Storm CHI-4hr	0.017	4848
5 Year Design Storm CHI-4hr	0.037	7297
10 Year Design Storm CHI-4hr	0.094	8349
25 Year Design Storm CHI-4hr	0.125	10342
50 Year Design Storm CHI-4hr	0.151	11652
100 Year Design Storm CHI-4hr	0.179	13094
Regional Design Storm	0.910	21925

1) Post development drainage areas and target release rates taken from FS&SWMR completed by CF. Crozier, March 2026

APPENDIX F
Sanitary Design Sheet

SANITARY SEWER CALCULATION SHEET

DSEL

STREET	MANHOLE		LENGTH (m)	PHASE	TRIBUTARY AREA HECTARE											POPULATION TRIBUTARY											AVG. L/s	AVG. L/s	PEAKING	MAX	INF.	MAX FLOW	SEWER					PIPE		Q/Qf	REMARKS
					INCREMENT										TOTAL	INCREMENT										TOTAL														
	FROM	TO			SINGLE F.	SEMI.	TOWNHOUSE	CONDO	SCHOOL	PARK	INSTITUTIONAL	COMMERCIAL	EXTERNAL	INFILT		SINGLE F.	SEMI.	TOWNHOUSE	CONDO	SCHOOL	PARK	INSTITUTIONAL	COMMERCIAL	EXTERNAL		INC.	TOTAL	FACTOR	L/s	L/s	EXP.	SIZE	SLOPE	Q L/s	VEL (m/s) FULL	VEL (m/s) ACT.	TYPE	CLASS		
STREET '4'																																						
	192A	193A	29.00		0.18										0.18	20									20	0.07	0.07	4.00	0.27	0.05	0.32	200	1.00	32.80	1.04	0.32	PVC	SDR-35	0.01	
	193A	194A	78.00		0.49										0.67	54									74	0.18	0.25	4.00	0.99	0.17	1.17	200	0.55	24.32	0.77	0.39	PVC	SDR-35	0.05	
To STREET '3', Pipe 194A - 195A															0.67										74														
STREET '10'																																						
	119A	120A	24.50		0.16										0.16	18									18	0.06	0.06	4.00	0.24	0.04	0.28	200	1.00	32.80	1.04	0.32	PVC	SDR-35	0.01	
	120A	121A	44.00		0.28										0.44	31									49	0.10	0.16	4.00	0.66	0.11	0.77	200	0.90	31.12	0.99	0.41	PVC	SDR-35	0.02	
Contribution From STREET '2', Pipe 118A - 121A															0.98										108														
	121A	122A	56.00		0.35										1.77	39									196	0.13	0.66	4.00	2.63	0.46	3.09	200	1.55	40.83	1.30	0.76	PVC	SDR-35	0.08	
	122A	123A	74.50		0.47										2.24	52									248	0.17	0.83	4.00	3.33	0.58	3.91	200	0.55	24.32	0.77	0.57	PVC	SDR-35	0.16	
To STREET '11', Pipe 123A - 126A															2.24										248														
STREET '8'																																						
	109A	110A	110.00		0.69										0.69	76									76	0.26	0.26	4.00	1.02	0.18	1.20	200	1.00	32.80	1.04	0.49	PVC	SDR-35	0.04	
	110A	111A	70.00		0.44										1.13	49									125	0.16	0.42	4.00	1.68	0.29	1.97	200	1.65	42.13	1.34	0.68	PVC	SDR-35	0.05	
	111A	112A	39.00		0.24										1.37	27									152	0.09	0.51	4.00	2.04	0.36	2.40	200	1.30	37.40	1.19	0.66	PVC	SDR-35	0.06	
To STREET '11', Pipe 112A - 113A															1.37										152														
STREET '6'																																						
Contribution From STREET '11', Pipe 123A - 126A															9.22										1024														
Contribution From STREET '11', Pipe 125A - 126A															0.55										62														
	126A	127A	19.00		0.12										9.89	14									1100	0.05	3.69	3.77	13.92	2.57	16.49	250	0.40	37.61	0.77	0.74	PVC	SDR-35	0.44	
	127A	128A	40.00		0.25										10.14	28									1128	0.09	3.79	3.77	14.27	2.64	16.91	250	0.40	37.61	0.77	0.75	PVC	SDR-35	0.45	
	128A	129A	9.00		0.06										10.20	7									1135	0.02	3.81	3.76	14.32	2.65	16.98	250	0.40	37.61	0.77	0.75	PVC	SDR-35	0.45	
	129A	130A	57.00		0.36										10.56	40									1175	0.13	3.94	3.75	14.79	2.75	17.54	250	0.40	37.61	0.77	0.75	PVC	SDR-21	0.47	
	130A	142A	43.50		0.27										10.83	30									1205	0.10	4.04	3.75	15.17	2.82	17.98	250	0.40	37.61	0.77	0.76	PVC	SDR-21	0.48	
To STREET '1', Pipe 142A - 189A															10.83										1205														
STREET '5'																																						
	140A	141A	100.00		0.63										0.63	70									70	0.23	0.23	4.00	0.94	0.16	1.10	200	1.00	32.80	1.04	0.48	PVC	SDR-35	0.03	
	141A	142A	97.50		0.61										1.24	68									138	0.23	0.46	4.00	1.85	0.32	2.18	200	0.55	24.32	0.77	0.48	PVC	SDR-35	0.09	
To STREET '1', Pipe 142A - 189A															1.24										138														
	131A	132A	38.50		0.24										0.24	27									27	0.09	0.09	4.00	0.36	0.06	0.42	200	1.00	32.80	1.04	0.35	PVC	SDR-35	0.01	
	132A	133A	27.50		0.17										0.41	19									46	0.06	0.15	4.00	0.62	0.11	0.72	200	0.85	30.24	0.96	0.40	PVC	SDR-35	0.02	
	133A	134A	10.00		0.06										0.47	7									53	0.02	0.18	4.00	0.71	0.12	0.83	200	0.55	24.32	0.77	0.36	PVC	SDR-35	0.03	
	134A	135A	99.00		0.62										1.09	69									122	0.23	0.41	4.00	1.64	0.28	1.92	200	0.95	31.97	1.02	0.56	PVC	SDR-35	0.06	
	135A	138A	99.00		0.62										1.71	69									191	0.23	0.64	4.00	2.56	0.44	3.01	200	0.55	24.32	0.77	0.52	PVC	SDR-35	0.12	
To STREET '1', Pipe 138A - 139A															1.71										191														
STREET '9'																																						
	186A	187A	52.00		0.32										0.32	36									36	0.12	0.12	4.00	0.48	0.08	0.57	200	1.00	32.80	1.04	0.39	PVC	SDR-35	0.02	
To STREET '1', Pipe 187A - 188A															0.32										36														
	105A	106A	61.50		0.39										0.39	43									43	0.14	0.14	4.00	0.58	0.10	0.68	200	1.70	42.76	1.36	0.49	PVC	SDR-35	0.02	
	106A	107A	86.00		0.54										0.93	60									103	0.20	0.35	4.00	1.38	0.24	1.62	200	0.55	24.32	0.77	0.44	PVC	SDR-35	0.07	
To STREET '11', Pipe 107A - 108A															0.93										103														

DESIGN PARAMETERS

Single Family:	110	pph	School:	40	pph
Semi-detached:	110	pph	Park:	0	pph
Townhouse:	110	pph	Community Services:	40	pph
Condo:	110	pph	Commercial:	90	pph
			Infil.Flow (INF):	0.26	L/s/ha

Note: To be conservative, industrial Kav is considered to be 1
Manning's n = (Conc) 0.013 (Pvc) 0.013

Designed: CJM
Checked: CMK
Dwg. Reference: 2

PROJECT: **TIFFANY ROX NORTH HILL LTD.**
LOCATION: **BOLTON NORTH HILL**
File Ref: **25-1417**
Date: **23 Mar 2026**
Sheet No. 1 of 5

SANITARY SEWER CALCULATION SHEET

STREET	MANHOLE		LENGTH (m)	PHASE	TRIBUTARY AREA HECTARE											POPULATION TRIBUTARY											AVG. L/s	AVG. L/s	PEAKING	MAX	INF.	MAX FLOW	SEWER					PIPE		Q/Qf	REMARKS
					INCREMENT										TOTAL	INCREMENT										TOTAL														
	FROM	TO			SINGLE F.	SEMI.	TOWNHOUSE	CONDO	SCHOOL	PARK	INSTITUTIONAL	COMMERCIAL	EXTERNAL	INFILT.		SINGLE F.	SEMI.	TOWNHOUSE	CONDO	SCHOOL	PARK	INSTITUTIONAL	COMMERCIAL	EXTERNAL		INC.	TOTAL	FACTOR	L/s	L/s	EXP.	SIZE	SLOPE	Q L/s	VEL (m/s) FULL	VEL (m/s) ACT.	TYPE	CLASS		
STREET 'EXT 1'																																						
	103A	104A	61.50		0.39										0.39	43									43	0.14	0.14	4.00	0.58	0.10	0.68	200	2.30	49.74	1.58	0.55	PVC	SDR-35	0.01	
To STREET '11', Pipe 104A - 107A															0.39										43														
	184A	185A	104.00		0.65										0.65	72									72	0.24	0.24	4.00	0.97	0.17	1.14	200	1.00	32.80	1.04	0.48	PVC	SDR-35	0.03	
To STREET '1', Pipe 185A - 187A															0.65										72														
STREET 'EXT 2'																																						
	100A	102A	41.00		0.26										0.26	29									29	0.10	0.10	4.00	0.39	0.07	0.46	200	1.00	32.80	1.04	0.36	PVC	SDR-35	0.01	
To STREET '11', Pipe 102A - 104A															0.26										29														
	181A	182A	61.00		0.38										0.38	42									42	0.14	0.14	4.00	0.56	0.10	0.66	200	1.00	32.80	1.04	0.41	PVC	SDR-35	0.02	
	182A	183A	30.50		0.19										0.57	21									63	0.07	0.21	4.00	0.85	0.15	0.99	200	0.55	24.32	0.77	0.37	PVC	SDR-35	0.04	
To STREET '1', Pipe 183A - 185A															0.57										63														
STREET '3'																																						
	158A	159A	51.00		0.32										0.32	36									36	0.12	0.12	4.00	0.48	0.08	0.57	200	1.00	32.80	1.04	0.39	PVC	SDR-35	0.02	
To STREET 'EXT 4', Pipe 159A - 183A															0.32										36														
	190A	191A	85.50		0.53										0.53	59									59	0.20	0.20	4.00	0.79	0.14	0.93	200	1.00	32.80	1.04	0.46	PVC	SDR-35	0.03	
	191A	194A	66.50		0.42										0.95	47									106	0.16	0.36	4.00	1.42	0.25	1.67	200	0.55	24.32	0.77	0.44	PVC	SDR-35	0.07	
Contribution From STREET '4', Pipe 193A - 194A															0.67										74														
	194A	195A	75.00		0.47										2.09	52									232	0.17	0.78	4.00	3.11	0.54	3.66	200	0.55	24.32	0.77	0.55	PVC	SDR-35	0.15	
To STREET '2', Pipe 195A - 196A															2.09										232														
STREET 'EXT 6'																																						
	145A	146A	65.00		0.41										0.41	46									46	0.15	0.15	4.00	0.62	0.11	0.72	200	1.60	41.49	1.32	0.50	PVC	SDR-35	0.02	
	146A	147A	80.50		0.51										0.92	57									103	0.19	0.35	4.00	1.38	0.24	1.62	200	1.55	40.83	1.30	0.63	PVC	SDR-35	0.04	
To Unknown Road4 - 08, Pipe 147A - 151A															0.92										103														
STREET 'EXT 3'																																						
	154A	155A	80.50		0.51										0.51	57									57	0.19	0.19	4.00	0.77	0.13	0.90	200	2.30	49.74	1.58	0.59	PVC	SDR-35	0.02	
To STREET 'EXT 4', Pipe 155A - 156A															0.51										57														
	179A	180A	75.00		0.47										0.47	52									52	0.17	0.17	4.00	0.70	0.12	0.82	200	1.50	40.17	1.28	0.50	PVC	SDR-35	0.02	
To STREET '1', Pipe 180A - 183A															0.47										52														
Contribution From STREET '11', Pipe 176A - 177A															0.78										87														
	177A	178A	31.50		0.20										0.98	22									109	0.07	0.37	4.00	1.46	0.25	1.72	200	0.55	24.32	0.77	0.44	PVC	SDR-35	0.07	
	178A	180A	87.50		0.55										1.53	61									170	0.20	0.57	4.00	2.28	0.40	2.68	200	0.55	24.32	0.77	0.50	PVC	SDR-35	0.11	
To STREET '1', Pipe 180A - 183A															1.53										170														
STREET '11'																																						
	152A	153A	90.50		0.57										0.57	63									63	0.21	0.21	4.00	0.85	0.15	0.99	200	2.65	53.39	1.70	0.64	PVC	SDR-35	0.02	
To STREET 'EXT 4', Pipe 153A - 155A															0.57										63														
	171A	172A	69.50		0.43										0.43	48									48	0.16	0.16	4.00	0.64	0.11	0.76	200	1.25	36.67	1.17	0.45	PVC	SDR-35	0.02	
To STREET '1', Pipe 172A - 180A															0.43										48														

DESIGN PARAMETERS

Single Family:	110	pph	School:	40	pph
Semi-detached:	70	pph	Park:	0	pph
Townhouse:	135	pph	Community Services:	40	pph
Condo:	55	pph	Commercial:	90	pph
			Infil.Flow (INF):	0.26	L/s/ha

Note: To be conservative, industrial Kav is considered to be 1

Manning's n = (Conc) 0.013 (Pvc) 0.013

Designed: CJM

Checked: CMK

Dwg. Reference: 2

PROJECT: **TIFFANY ROX NORTH HILL LTD.**

LOCATION: **BOLTON NORTH HILL**

File Ref: **25-1417**

Date: **23 Mar 2026**

Sheet No. 2 of 5

SANITARY SEWER CALCULATION SHEET

STREET	MANHOLE		LENGTH (m)	PHASE	TRIBUTARY AREA HECTARE											POPULATION TRIBUTARY											AVG. L/s	AVG. L/s	PEAKING	MAX	INF.	MAX FLOW	SEWER					PIPE		Q/Qf	REMARKS
					INCREMENT										TOTAL	INCREMENT										TOTAL														
	FROM	TO			SINGLE F.	SEMI.	TOWNHOUSE	CONDO	SCHOOL	PARK	INSTITUTIONAL	COMMERCIAL	EXTERNAL	INFILT		SINGLE F.	SEMI.	TOWNHOUSE	CONDO	SCHOOL	PARK	INSTITUTIONAL	COMMERCIAL	EXTERNAL		INC.	TOTAL	FACTOR	L/s	L/s	EXP.	SIZE	SLOPE	Q L/s	VEL (m/s) FULL	VEL (m/s) ACT.	TYPE	CLASS		
	124A	125A	49.50		0.31										0.31	35									35	0.12	0.12	4.00	0.47	0.08	0.55	200	1.00	32.80	1.04	0.39	PVC	SDR-35	0.02	
	125A	126A	37.50		0.24										0.55	27									62	0.09	0.21	4.00	0.83	0.14	0.98	200	0.70	27.44	0.87	0.40	PVC	SDR-35	0.04	
To STREET '6', Pipe 126A - 127A															0.55										62														
	173A	174A	55.50		0.35										0.35	39									39	0.13	0.13	4.00	0.52	0.09	0.61	200	3.05	57.28	1.82	0.58	PVC	SDR-35	0.01	
	174A	175A	10.50		0.07										0.42	8									47	0.03	0.16	4.00	0.63	0.11	0.74	200	0.55	24.32	0.77	0.34	PVC	SDR-35	0.03	
	175A	176A	41.50		0.26										0.68	29									76	0.10	0.26	4.00	1.02	0.18	1.20	200	0.55	24.32	0.77	0.40	PVC	SDR-35	0.05	
	176A	177A	16.00		0.10										0.78	11									87	0.04	0.29	4.00	1.17	0.20	1.37	200	0.55	24.32	0.77	0.42	PVC	SDR-35	0.06	
To STREET 'EXT 3', Pipe 177A - 178A															0.78										87														
	101A	102A	38.50		0.24										0.24	27									27	0.09	0.09	4.00	0.36	0.06	0.42	200	1.75	43.39	1.38	0.42	PVC	SDR-35	0.01	
Contribution From STREET 'EXT 2', Pipe 100A - 102A															0.26										29														
	102A	104A	74.00		0.46										0.96	51									107	0.17	0.36	4.00	1.44	0.25	1.69	200	0.55	24.32	0.77	0.44	PVC	SDR-35	0.07	
Contribution From STREET 'EXT 1', Pipe 103A - 104A															0.39										43														
	104A	107A	74.00		0.46										1.81	51									201	0.17	0.67	4.00	2.70	0.47	3.17	200	0.75	28.40	0.90	0.60	PVC	SDR-35	0.11	
Contribution From STREET '9', Pipe 106A - 107A															0.93										103														
	107A	108A	21.00		0.13										2.87	15									319	0.05	1.07	4.00	4.28	0.75	5.03	200	0.55	24.32	0.77	0.61	PVC	SDR-35	0.21	
	108A	112A	53.50		0.33										3.20	37									356	0.12	1.19	4.00	4.78	0.83	5.61	200	0.55	24.32	0.77	0.63	PVC	SDR-35	0.23	
Contribution From STREET '8', Pipe 111A - 112A															1.37										152														
	112A	113A	29.00		0.18										4.75	20									528	0.07	1.77	3.96	7.02	1.24	8.25	200	0.60	25.41	0.81	0.72	PVC	SDR-35	0.32	
	113A	114A	102.00		0.64										5.39	71									599	0.24	2.01	3.93	7.90	1.40	9.30	200	0.55	24.32	0.77	0.72	PVC	SDR-35	0.38	
	114A	115A	102.00		0.64										6.03	71									670	0.24	2.25	3.91	8.79	1.57	10.36	200	0.55	24.32	0.77	0.74	PVC	SDR-35	0.43	
	115A	116A	14.00		0.09										6.12	10									680	0.03	2.28	3.90	8.90	1.59	10.49	250	0.40	37.61	0.77	0.66	PVC	SDR-35	0.28	
	116A	123A	65.50		0.41										6.53	46									726	0.15	2.44	3.89	9.48	1.70	11.18	250	0.40	37.61	0.77	0.66	PVC	SDR-35	0.30	
Contribution From STREET '10', Pipe 122A - 123A															2.24										248														
	123A	126A	72.50		0.45										9.22	50									1024	0.17	3.44	3.79	13.03	2.40	15.42	250	0.40	37.61	0.77	0.73	PVC	SDR-35	0.41	
To STREET '6', Pipe 126A - 127A															9.22										1024														
Unknown Road4 - 08																																							
	168A	169A	17.50		0.11										0.11	13									13	0.04	0.04	4.00	0.17	0.03	0.20	200	1.90	45.21	1.44	0.35	PVC	SDR-35	0.00	
Contribution From STREET 'EXT 4', Pipe 167A - 169A															0.54										60														
	169A	170A	46.00		0.29										0.94	32									105	0.11	0.35	4.00	1.41	0.24	1.65	200	0.55	24.32	0.77	0.44	PVC	SDR-35	0.07	
To STREET '1', Pipe 170A - 172A															0.94										105														
Contribution From STREET 'EXT 5', Pipe 143A - 144A															0.66										73														
	144A	147A	46.00		0.29										0.95	32									105	0.11	0.35	4.00	1.41	0.25	1.66	200	2.65	53.39	1.70	0.76	PVC	SDR-35	0.03	
Contribution From STREET 'EXT 6', Pipe 146A - 147A															0.92										103														
	147A	151A	45.50		0.28										2.15	31									239	0.10	0.80	4.00	3.21	0.56	3.77	200	0.80	29.34	0.93	0.64	PVC	SDR-35	0.13	
To STREET 'EXT 4', Pipe 151A - 153A															2.15										239														
STREET 'EXT 4'																																							
	163A	164A	58.00		0.36										0.36	40									40	0.13	0.13	4.00	0.54	0.09	0.63	200	1.00	32.80	1.04	0.41	PVC	SDR-35	0.02	
To STREET 'EXT 4', Pipe 164A - 165A															0.36										40														
	167A	169A	85.50		0.54										0.54	60									60	0.20	0.20	4.00	0.81	0.14	0.95	200	1.50	40.17	1.28	0.53	PVC	SDR-35	0.02	
To Unknown Road4 - 08, Pipe 169A - 170A															0.54										60														

DESIGN PARAMETERS

Single Family:	110	pph	School:	40	pph
Semi-detached:	70	pph	Park:	0	pph
Townhouse:	135	pph	Community Services:	40	pph
Condo:	55	pph	Commercial:	90	pph
			Infil.Flow (INF):	0.26	L/s/ha

Note: To be conservative, industrial Kav is considered to be 1

Manning's n = (Conc) 0.013 (Pvc) 0.013

Designed: CJM

Checked: CMK

Dwg. Reference: 2

PROJECT: **TIFFANY ROX NORTH HILL LTD.**

LOCATION: **BOLTON NORTH HILL**

File Ref: **25-1417**

Date: **23 Mar 2026**

Sheet No. 3 of 5

SANITARY SEWER CALCULATION SHEET

STREET	MANHOLE		LENGTH (m)	PHASE	TRIBUTARY AREA HECTARE											POPULATION TRIBUTARY											AVG. L/s	AVG. L/s	PEAKING	MAX	INF.	MAX FLOW	SEWER				PIPE		Q/Qf	REMARKS
					INCREMENT										TOTAL	INCREMENT										TOTAL							SIZE	SLOPE	Q	VEL (m/s)				
	FROM	TO			SINGLE F.	SEMI.	TOWNHOUSE	CONDO	SCHOOL	PARK	INSTITUTIONAL	COMMERCIAL	EXTERNAL	INFILT		SINGLE F.	SEMI.	TOWNHOUSE	CONDO	SCHOOL	PARK	INSTITUTIONAL	COMMERCIAL	EXTERNAL		INC.	TOTAL	FACTOR	L/s	L/s	EXP.			L/s	FULL	ACT.	TYPE	CLASS		
STREET 'EXT 5'																																						
	143A	144A	106.00		0.66										0.66	73									73	0.25	0.25	4.00	0.98	0.17	1.15	200	1.00	32.80	1.04	0.48	PVC	SDR-35	0.04	
To Unknown Road4 - 08, Pipe 144A - 147A															0.66										73														
	161A	162A	37.50		0.24										0.24	27									27	0.09	0.09	4.00	0.36	0.06	0.42	200	1.00	32.80	1.04	0.35	PVC	SDR-35	0.01	
To STREET 'EXT 4', Pipe 162A - 164A															0.24										27														
STREET 'EXT 4'																																						
	160A	162A	59.00		0.37										0.37	41									41	0.14	0.14	4.00	0.55	0.10	0.65	200	1.15	35.17	1.12	0.42	PVC	SDR-35	0.02	
Contribution From STREET 'EXT 5', Pipe 161A - 162A															0.24										27														
	162A	164A	46.00		0.29										0.90	32									100	0.11	0.34	4.00	1.34	0.23	1.58	200	0.55	24.32	0.77	0.43	PVC	SDR-35	0.06	
Contribution From STREET 'EXT 4', Pipe 163A - 164A															0.36										40														
	164A	165A	45.00		0.28										1.54	31									171	0.10	0.57	4.00	2.30	0.40	2.70	200	0.55	24.32	0.77	0.50	PVC	SDR-35	0.11	
To STREET '1', Pipe 165A - 166A															1.54										171														
	148A	149A	12.50		0.08										0.08	9									9	0.03	0.03	4.00	0.12	0.02	0.14	200	1.35	38.11	1.21	0.27	PVC	SDR-35	0.00	
	149A	150A	138.50		0.87										0.95	96									105	0.32	0.35	4.00	1.41	0.25	1.66	200	1.35	38.11	1.21	0.60	PVC	SDR-35	0.04	
	150A	151A	14.50		0.09										1.04	10									115	0.03	0.39	4.00	1.54	0.27	1.81	200	3.30	59.58	1.90	0.83	PVC	SDR-35	0.03	
Contribution From Unknown Road4 - 08, Pipe 147A - 151A															2.15										239														
	151A	153A	46.00		0.29										3.48	32									386	0.11	1.30	4.00	5.18	0.90	6.09	200	1.30	37.40	1.19	0.87	PVC	SDR-35	0.16	
Contribution From STREET '11', Pipe 152A - 153A															0.57										63														
	153A	155A	70.50		0.44										4.49	49									498	0.16	1.67	3.98	6.65	1.17	7.82	200	0.90	31.12	0.99	0.82	PVC	SDR-35	0.25	
Contribution From STREET 'EXT 3', Pipe 154A - 155A															0.51										57														
	155A	156A	66.50		0.42										5.42	47									602	0.16	2.02	3.93	7.94	1.41	9.35	200	0.55	24.32	0.77	0.72	PVC	SDR-35	0.38	
	156A	157A	10.50		0.07										5.49	8									610	0.03	2.05	3.93	8.05	1.43	9.47	200	0.55	24.32	0.77	0.72	PVC	SDR-35	0.39	
	157A	159A	89.00		0.56										6.05	62									672	0.21	2.26	3.90	8.80	1.57	10.37	200	0.55	24.32	0.77	0.74	PVC	SDR-35	0.43	
Contribution From STREET '3', Pipe 158A - 159A															0.32										36														
	159A	183A	76.00		0.48										6.85	53									761	0.18	2.55	3.87	9.89	1.78	11.67	200	0.55	24.32	0.77	0.76	PVC	SDR-35	0.48	
To STREET '1', Pipe 183A - 185A															6.85										761														
STREET '1'																																						
Contribution From STREET 'EXT 4', Pipe 164A - 165A															1.54										171														
	165A	166A	81.00		0.51										2.05	57									228	0.19	0.77	4.00	3.06	0.53	3.59	200	0.55	24.32	0.77	0.55	PVC	SDR-35	0.15	
	166A	170A	81.00		0.51										2.56	57									285	0.19	0.96	4.00	3.83	0.67	4.49	200	0.55	24.32	0.77	0.59	PVC	SDR-35	0.18	
Contribution From Unknown Road4 - 08, Pipe 169A - 170A															0.94										105														
	170A	172A	47.50		0.30										3.80	33									423	0.11	1.42	4.00	5.68	0.99	6.67	200	0.55	24.32	0.77	0.66	PVC	SDR-35	0.27	
Contribution From STREET '11', Pipe 171A - 172A															0.43										48														
	172A	180A	75.00		0.47										4.70	52									523	0.17	1.76	3.96	6.95	1.22	8.17	200	0.55	24.32	0.77	0.69	PVC	SDR-35	0.34	
Contribution From STREET 'EXT 3', Pipe 178A - 180A															1.53										170														
Contribution From STREET 'EXT 3', Pipe 179A - 180A															0.47										52														
	180A	183A	76.00		0.48										7.18	53									798	0.18	2.68	3.86	10.34	1.87	12.21	250	0.40	37.61	0.77	0.68	PVC	SDR-35	0.32	
Contribution From STREET 'EXT 4', Pipe 159A - 183A															6.85										761														
Contribution From STREET 'EXT 2', Pipe 182A - 183A															0.57										63														
	183A	185A	74.00		0.46										15.06	51									1673	0.17	5.62	3.64	20.44	3.92	24.36	250	0.40	37.61	0.77	0.81	PVC	SDR-21	0.65	
Contribution From STREET 'EXT 1', Pipe 184A - 185A															0.65										72														
	185A	187A	74.00		0.46										16.17	51									1796	0.17	6.03	3.62	21.82	4.20	26.03	300	0.35	57.21	0.81	0.79	PVC	SDR-21	0.45	
Contribution From STREET '9', Pipe 186A - 187A															0.32										36														
	187A	188A	75.00		0.47										16.96	52									1884	0.17	6.32	3.61	22.83	4.41	27.24	300	0.35	57.21	0.81	0.80	PVC	SDR-21	0.48	

DESIGN PARAMETERS

Single Family:	110	pph	School:	40	pph
Semi-detached:	70	pph	Park:	0	pph
Townhouse:	135	pph	Community Services:	40	pph
Condo:	55	pph	Commercial:	90	pph
			Infil.Flow (INF):	0.26	L/s/ha

Note: To be conservative, industrial Kav is considered to be 1

Manning's n = (Conc) 0.013 (Pvc) 0.013

Designed: CJM

Checked: CMK

Dwg. Reference: 2

PROJECT: **TIFFANY ROX NORTH HILL LTD.**

LOCATION: **BOLTON NORTH HILL**

File Ref: **25-1417**

Date: **23 Mar 2026**

Sheet No. 4 of 5

SANITARY SEWER CALCULATION SHEET

STREET	MANHOLE		LENGTH (m)	PHASE	TRIBUTARY AREA HECTARE											POPULATION TRIBUTARY											AVG. L/s	AVG. L/s	PEAKING	MAX	INF.	MAX FLOW	SEWER					PIPE		Q/Qf	REMARKS
					INCREMENT										TOTAL	INCREMENT										TOTAL							SIZE	SLOPE	Q	VEL (m/s)				
	FROM	TO			SINGLE F.	SEMI.	TOWNHOUSE	CONDO	SCHOOL	PARK	INSTITUTIONAL	COMMERCIAL	EXTERNAL	INFILT		SINGLE F.	SEMI.	TOWNHOUSE	CONDO	SCHOOL	PARK	INSTITUTIONAL	COMMERCIAL	EXTERNAL		INC.	TOTAL	FACTOR	L/s	L/s	EXP.			L/s	FULL	ACT.	TYPE	CLASS		
	188A	189A	74.50		0.47										17.43	52									1936	0.17	6.50	3.60	23.39	4.53	27.93	300	0.35	57.21	0.81	0.80	PVC	SDR-21	0.49	
To STREET '2', Pipe 189A - 195A															17.43									1936														
	136A	137A	97.50		0.61										0.61	68									68	0.23	0.23	4.00	0.91	0.16	1.07	200	1.00	32.80	1.04	0.47	PVC	SDR-35	0.03	
	137A	138A	71.50		0.45										1.06	50									118	0.17	0.40	4.00	1.58	0.28	1.86	200	0.55	24.32	0.77	0.45	PVC	SDR-35	0.08	
Contribution From STREET '5', Pipe 135A - 138A															1.71									191														
	138A	139A	35.00		0.22										2.99	25									334	0.08	1.12	4.00	4.48	0.78	5.26	200	0.55	24.32	0.77	0.62	PVC	SDR-35	0.22	
	139A	142A	38.00		0.24										3.23	27									361	0.09	1.21	4.00	4.85	0.84	5.69	200	0.55	24.32	0.77	0.63	PVC	SDR-35	0.23	
Contribution From STREET '6', Pipe 130A - 142A															10.83									1205														
Contribution From STREET '5', Pipe 141A - 142A															1.24									138														
	142A	189A	74.00		0.47										15.77	52									1756	0.17	5.89	3.63	21.40	4.10	25.50	300	0.35	57.21	0.81	0.78	PVC	SDR-21	0.45	
To STREET '2', Pipe 189A - 195A															15.77									1756														
STREET '2'																																							
	117A	118A	78.00		0.49										0.49	54									54	0.18	0.18	4.00	0.73	0.13	0.85	200	1.00	32.80	1.04	0.44	PVC	SDR-35	0.03	
	118A	121A	78.00		0.49										0.98	54									108	0.18	0.36	4.00	1.45	0.25	1.70	200	1.00	32.80	1.04	0.54	PVC	SDR-35	0.05	
To STREET '10', Pipe 121A - 122A															0.98									108														
Contribution From STREET '1', Pipe 142A - 189A															15.77									1756														
Contribution From STREET '1', Pipe 188A - 189A															17.43									1936														
	189A	195A	76.00		0.48										33.68	53									3745	0.18	12.57	3.36	42.24	8.76	50.99	375	0.35	103.73	0.94	0.94	PVC	SDR-21	0.49	
Contribution From STREET '3', Pipe 194A - 195A															2.09									232														
	195A	196A	72.00		0.45										36.22	50									4027	0.17	13.52	3.33	45.01	9.42	54.43	375	0.35	103.73	0.94	0.95	PVC	SDR-21	0.52	
	196A	197A	74.00		0.46										36.68	51									4078	0.17	13.69	3.33	45.58	9.54	55.12	375	0.35	103.73	0.94	0.95	PVC	SDR-21	0.53	

DESIGN PARAMETERS

Single Family:	110	pph	School:	40	pph
Semi-detached:	70	pph	Park:	0	pph
Townhouse:	135	pph	Community Services:	40	pph
Condo:	55	pph	Commercial:	90	pph
			Infil.Flow (INF):	0.26	L/s/ha

Note: To be conservative, industrial Kav is considered to be 1

Manning's n = (Conc) 0.013 (Pvc) 0.013

Designed: CJM

Checked: CMK

Dwg. Reference: 2

PROJECT: **TIFFANY ROX NORTH HILL LTD.**

LOCATION: **BOLTON NORTH HILL**

File Ref: **25-1417**

Date: **23 Mar 2026**

Sheet No. 5 of 5

APPENDIX G
Geotechnical Report

PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

14337 Highway 50
Bolton, Ontario

Prepared for Tiffany Rox North Hill Ltd.

Report TG0257-1 dated January 23, 2026

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Appendices

Appendix 1 Soil Profile and Test Data Sheets
Symbols and Terms
Grain Size Distribution and Hydrometer Testing Results
Atterberg Limits Testing Results
Analytical Testing Results

Appendix 2 Figure 1 - Key Plan
Drawing TG0257-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Tiffany Rox North Hill Ltd. to conduct a geotechnical investigation for the proposed residential development to be located at 14337 Highway 50, Bolton, Ontario (refer to Figure 1 - Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of boreholes.
- Provide geotechnical recommendations pertaining to the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available conceptual drawings, it is understood that the proposed development will consist of a residential development inclusive of single-family dwellings, townhomes, a stormwater management pond, and parklands.

At finished grades, the proposed residential dwellings will generally be surrounded by driveways, local right-of-ways, walkways, and landscaped areas. It is also understood that the proposed development is to be municipally serviced.

It is expected that the existing commercial and residential dwellings will be demolished to accommodate the construction of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on December 3, 5, 8 and 9, 2025, and consisted of advancing a total of ten (10) boreholes to a maximum depth of 9.8 m below the existing ground surface.

The test hole locations were distributed in a manner to provide general coverage of the proposed development, taking into consideration existing site features. The approximate locations of the test holes are shown on Drawing TG0257-1 - Test Hole Location Plan included in Appendix 2.

A previous field investigation was carried out on January 28, 2025, by Paterson Group. At that time three (3) boreholes BH11-25, BH12-25 and BH13-25 were drilled within the current property limit to a maximum depth of 6.7 m. The borehole logs from the investigation are shown in Appendix 1. The borehole locations from this investigation are shown on Drawing TG0257-1 - Test Hole Location Plan, included in Appendix 2.

The boreholes were put down using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The testing procedure for boreholes consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In-Situ Testing

Borehole samples were recovered from a 50 mm diameter split-spoon (SS) or the auger flights (AU). All soil samples were visually inspected and initially classified on site. The split-spoon and auger samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

The thickness of the overburden was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at BH 12-25. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm.

The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Undrained shear strength testing was carried out in BH 2-25 and BH 11-25 cohesive soils using a field vane apparatus. The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Monitoring wells were installed in nine (9) boreholes and standpipe piezometer was installed in three (3) boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are further discussed in Section 4.3 and are presented in the Soil Profile and Test Data Sheets in Appendix 1.

Sample Storage

All samples will be stored in the laboratory for a period of six (6) months after the issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations, and the ground surface elevations at each test hole location, were surveyed by Paterson using a handheld GPS unit, and referenced to a geodetic datum. The borehole locations are presented on Drawing TG0257-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soils samples were collected from the subject site during the investigation and were visually examined in our laboratory to review the results of the field logging. Moisture content testing was completed on all recovered soil samples.

Additionally, four (4) grain-size distribution tests and four (4) Atterberg's test were conducted on selected soil samples recovered during the current field investigation and two (2) grain-size distribution tests were conducted during the previous field investigation, results are presented in Appendix 1.

3.4 Analytical Testing

Three (3) soil samples were submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures the samples were submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7

4.0 Observations

4.1 Surface Conditions

The majority of the subject site consists of agricultural lands. Agricultural outbuildings and residential buildings and associated gravel surfaced access lane are located within the southeast corner of the property. A marsh area is located within the northern limits of the site.

The subject site is bordered to the west by Highway 50, to the north, east, and southeast by vacant agricultural lands, and to the southwest by a commercial development. The site generally slopes downward from west to east, from approximate geodetic elevation 258 to 265.5 m.

A 4 to 6 m high gradual slope exists within the east portion of the subject site. The slope profile ranges between 10H:1V to 13H:1V with no signs of instability. the slope was noted to be part of the farmer's field with an existing pond located at the toe of the slope.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of topsoil underlain by a hard to very stiff brown silty clay to clayey silt deposit or compact silty sand to sandy silt. A fill material was observed underlying the topsoil layer at borehole BH 7-25, BH 8-25 and BH 11-25 and consisted of brown to grey silty clay with trace grave, organics and sand.

A hard to very stiff brown silty clay deposit was encountered underlying the topsoil and/or silt sand at all boreholes. The brown silty clay was observed to transition to very stiff to hard, grey silty clay to clayey silt deposit at approximate depths ranging 2.1 to 5.46 m. Further, a silty sand to sandy silt layer was interbedded between the clay layers or underneath the topsoil at boreholes BH 1-25, 4-25, 5-25, 8-25, 9-25 and BH 12-25. All boreholes were terminated within the silty clay to clayey silt deposit.

Practical refusal to the DCPT was encountered at an approximate depth of 8.2 m below the existing ground surface at Borehole BH 12-25.

Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for details of the soil profiles encountered at each borehole location.

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of shale, limestone, dolostone and siltstone of varying geological formations consisting of the Georgian Bay, Blue Mountain, and Billings Formation. The overburden drift thickness is anticipated to be greater than 50 m.

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) testing was completed on selected soil samples. The results of the grain size analysis are summarized in Table 1 and presented on the Grain-Size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 1 – Summary of Grain Size Distribution Analysis

Borehole Number	Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 1-25	SS3	1.5 – 2.1	0	44.2	53.1	2.7
BH 5-25	SS3	1.5 – 2.1	0.2	6.1	59.9	33.9
BH 7-25	SS4	2.3 - 2.9	2.6	10.7	53.3	33.4
BH 9-25	SS4	2.3 - 2.9	0	5.7	48.5	45.9
BH 11-25	SS5	3.0 - 3.7	0.2	7.9	59.5	32.5
BH 13-25	SS3	1.5 – 2.1	0	11.6	45.4	43

Atterberg Limit Tests

Four (4) samples were submitted for Atterberg Limits testing during the current investigation. The results are summarized in Table 2 below.

Table 2 – Summary of Atterberg Limits Results

Borehole Number	Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 1-25	SS4	2.3 - 2.9	45	23	22	CL
BH 4-25	SS3	1.5 – 2.1	38	24	14	CL
BH 6-25	SS4	2.3 - 2.9	35	22	13	CL
BH 8-25	SS4	2.3 - 2.9	34	22	12	CL

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; CL: Inorganic Clay of Low Plasticity

The test results indicate that the soil samples are generally classified as Inorganic Clay of Low Plasticity (CL). These classifications are in accordance with the Unified Soil Classification System.

4.3 Groundwater

Monitoring wells were installed in nine (9) boreholes and standpipe piezometer was installed in three (3) boreholes as part of our geotechnical investigation. Groundwater level measurements were recorded at all test hole locations on February 4, 2025, December 22, 2025, and January 8, 2026. The observed groundwater levels are shown in Table 3 below and are presented in the Soil Profile and Test Data Sheets in Appendix 1.

Table 3 – Summary of Groundwater Level readings

Test Hole ID	Structure	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-25	PZ	261.60	Dry	N/A	December 22, 2025
BH 2-25	MW	263.86	7.21	256.65	
BH 3-25	PZ	263.03	Dry	N/A	
BH 4-25	PZ	266.5	Dry	N/A	
BH 5-25	MW	264.25	Dry	N/A	
BH 6-25	MW	264.09	Dry	N/A	
BH 7-25	MW	265.3	3.03	262.27	
BH 8-25	MW	266.20	4.35	261.85	
BH 10-25	MW	263.78	1.5	262.28	
BH 11-25	MW	266.05	4.45	261.6	
BH 12-25	MW	264.18	2.96	261.22	
BH 13-25	MW	266.11	1.04	265.07	January 8, 2026
BH 11-25	MW	266.05	Dry	N/A	February 4, 2025
BH 12-25	MW	264.18	1.30	262.88	
BH 13-25	MW	266.11	1.33	264.78	

Note: The ground surface elevation at each monitoring well location was surveyed using a handheld GPS unit and references to a geodetic datum.
MW – Monitoring Well and PZ - Piezometer

It should be noted that the groundwater level is subject to seasonal fluctuations. Therefore, groundwater could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is anticipated that the proposed buildings founded on conventional footings placed on an undisturbed hard, to very stiff brown silty clay, compact silty sand, or on engineered fill placed over an approved bearing surface.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and any fill containing significant amounts of deleterious or organic materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities.

Paterson should be consulted to assess potential areas of concern for root-impacted subsoils during topsoil stripping and/or the pre-engineering phase of the site. Where encountered, root-impacted subsoils may need to be removed beneath settlement-sensitive structures to provide an adequate bearing surface.

Further, existing foundations and other construction debris should be entirely removed from within the proposed residential dwelling footprints. Under paved areas, existing construction remnants, such as foundation walls should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill placed for grading beneath building footprints should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II or approved equivalent. The imported fill material should be tested and approved by Paterson prior to site delivery. The fill should be placed in maximum 300 mm loose lifts and compacted by suitable compaction equipment as specified in OPSS 501. Fill placed beneath the building footprints should be compacted to a minimum 98% of the Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids.

Non-specified fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in maximum 225 mm thin lifts to a minimum density of 95% SPMDD using suitable compaction equipment as per OPSS 501.

Any soft or poor performing areas should be removed and replaced with engineered fill consisting of OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm or approved equivalent. The engineered fill should be placed in maximum 300 mm loose lifts and compacted to 98% SPMDD using suitable vibratory equipment as per OPSS 501.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed very stiff to hard silty clay to clayey silt, or undisturbed, compact silty sand, or on an approved engineered fill pad placed directly over the approved native soil can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa.** A geotechnical resistance factor of 0.5 was applied to the above-noted bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

Footings designed using the bearing resistance value at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit at the site, a permissible grade raise restriction of **3.0 m** is recommended.

If a higher permissible grade raise is required, preloading with or without surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction and differential settlements.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in-situ soil.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class X_D** for the foundations considered as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2024 for a full discussion of the earthquake design requirements.

5.5 Floor Slab Construction

With the removal of all topsoil and deleterious fill from within the footprint of the proposed buildings, the native soil surface or approved engineered fill pad will be considered acceptable subgrades on which to commence backfilling for floor slab construction.

For structures with slab-on-grade construction, it is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed structures should be placed in a maximum of 300 mm thick loose layers and compacted to a minimum of 98% of the material's SPMDD.

If a basement level is considered for the proposed buildings, it is recommended that the upper 300 mm of sub-floor fill consists of 19 mm clear crush stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the material's SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

5.6 Pavement Design

For preliminary design purposes, the pavement structures in Tables 4 and 5 are recommended for the design of the proposed pavement structures.

Table 4 - Recommended Pavement Structure – Car only Parking Areas/Driveways	
Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
300	**SUBBASE** – OPSS Granular B Type II
SUBGRADE – Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

Table 5 - Recommended Pavement Structure – Local Residential Roadways	
Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
400	**SUBBASE** – OPSS Granular B Type II
SUBGRADE – Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. Cement asphalt should be compacted to a minimum average density of 93% and no more than 98%.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable compaction equipment as per OPSS 501.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Due to the low permeability of the subgrade materials, consideration should be given to installing subdrains during the pavement construction. Where subdrains are to be connected to catch basins, the subdrain invert should be approximately 300 mm below subgrade level. The subdrains should be outfitted with a filter sock, surrounded by 19 mm clear stone and wrapped in a non-woven geotextile.

Where the heavy and light duty pavement structures intersect, the subdrain should be placed within subbase of the heavy-duty pavement structure. The subdrain should be outfitted with a filter cloth, surrounded by 100 mm of 19 mm clear stone and wrapped in a non-woven geotextile.

The road subgrade surface should be crowned to promote waterflow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage & Backfill

Foundation Drainage

Should the proposed buildings include below-grade space, a perimeter foundation drainage system is recommended to be provided for the proposed structures. The system should consist of a 150 mm diameter perforated and corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipe should have positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.4 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided in this regard.

Exterior unheated foundations, such as isolated piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure, and require additional protection, such as soil cover of 1.8 m, or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. For the proposed development, it is anticipated that sufficient room will be available for the greater part of the excavations to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes in the overburden soils above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. Excavations below the groundwater level should be cut back at a maximum slope of 2H:1V. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent material specifications and standard detail drawings from the Region of Peel’s Public Works Design Standard, Specifications, & Procedures Manual.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil. It is recommended to carry a provision to increase the bedding layer thickness to a minimum thickness of 300 mm if the subgrade consists of relatively loose and saturated grey silty clay or grey silty sand and is difficult to achieve compaction over the relatively loose subgrade. The bedding should extend to the spring line of the pipe.

Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in a maximum of 225 mm thick lifts and compacted to 99% of the SPMDD.

It should generally be possible to re-use the site generated fill materials (moist, not wet) above the cover material if excavation and filling operations are carried out in dry and non-freezing weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.4 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavation should be controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required. In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan, to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means.

In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the samples indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a non-aggressive to slightly aggressive corrosive environment.

7.0 Recommendations

It is recommended that the following be carried out by Paterson as design details of the proposed development are prepared by other members of the design team:

- ❑ Review preliminary and detailed grading, servicing, and structural plan(s) from a geotechnical perspective during the design phase of the project.

- ❑ Review geotechnical aspects of the excavation contractor's subgrade preparation and excavation work plans from a geotechnical perspective and prior to construction.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ❑ Review detailed grading plan(s) from a geotechnical perspective.

- ❑ Observation of all bearing surfaces prior to the placement of concrete.

- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

- ❑ Observation of all subgrades prior to backfilling.

- ❑ Field density tests to ensure that the specified level of compaction has been achieved.

- ❑ Sampling and testing of concrete and fill materials; including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per ***Ontario Regulation 406/19: On-Site and Excess Soil Management.***

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8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Tiffany Rox North Hill Ltd., or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.



Balaji T. Nirmala, M.Eng., P.Eng. David J. Gilbert, P.Eng.

Report Distribution:

- ❑ Tiffany Rox North Hill Ltd. (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTURBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMIT TESTING RESULTS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 600124.69 **NORTHING:** 4861227.07 **ELEVATION:** 261.60

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 3, 2025

FILE NO. : TG0257

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
AGRICULTURALLY DISTURBED ZONE: with organics and rootlets			AU 1			18			
0.33m [261.27m]									
Firm, brown **SILTY CLAY** some sand and rootlets			SS 2	12	3-7-4-5 11	16			
0.69m [260.91m]		1							
Compact, brown **SILTY SAND** to **SANDY SILT**			SS 3	33	3-3-7-11 10				
2.13m [259.47m]		2							
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel			SS 4	100	4-8-7-6 15	19			
		3	SS 5	92	5-8-7-8 15	17			
		4	SS 6	100	3-4-7-9 11	21			
		5	SS 7	100	3-5-6-7 11	20			
			SS 8	100	3-6-7-5 13	21			
		6	SS 9	100	4-5-8-7 13	20			
6.71m [254.89m]		7							
End of Borehole									
(BH Dry on - December 22, 2025)		8							
		9							
		10							
		11							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 600025.21 **NORTHING:** 4861290.43 **ELEVATION:** 263.86

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 3, 2025

FILE NO. : TG0257

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE							263
AGRICULTURALLY DISTURBED ZONE: with organics and rootlets			AU 1			24	
0.15m [263.71m]							
Firm, brown **SILTY CLAY** some sand and rootlets		1	SS 2	75	5-7-9-8	16	
0.69m [263.17m]					16		
GLACIAL TILL: Very stiff to hard, brown silty clay to clayey silt with some sand		2	SS 3	58	4-7-7-8	16	262
					14	22	
			SS 4	100	3-6-8-9	23	
		3			14		261
			SS 5	75	4-6-9-12	12	
					15		
- Silty sand seam by 3.9 m depth		4	SS 6	83	4-7-8-6	13	260
4.27m [259.59m]					15	17	
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand		5	SS 7	75	3-4-7-8	20	259
					11		
			SS 8	12	4-10-13-16	17	
		6			23		258
			SS 9	100	4-6-6-10	10	
		7			12		257
			SS 10	83	4-7-9-11	17	
					16		256
		8	SS 11	100	3-6-7-12	22	
					13		
			SS 12	100	3-7-8-12	20	255
		9			15		
			SS 13	100	6-7-13-16	14	
9.75m [254.11m]					20		254
End of Borehole		10					
(GWL at 7.21 m depth - December 22, 2025)							253
		11					

PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) — 20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)

▲ UNDRAINED SHEAR STRENGTH (kPa) — 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) — 20 40 60 80

MONITORING WELL CONSTRUCTION

129

7.21 m 2025-12-22

7.55m

9.07m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599976.62 **NORTHING:** 4861376.31 **ELEVATION:** 263.03

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 3, 2025

FILE NO. : TG0257

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
AGRICULTURALLY DISTURBED ZONE: with organics and rootlets		0	AU 1			23			263
0.05m [262.98m]									
Firm, brown **SILTY CLAY** with sand, organics and gravel		1	SS 2	42	2-5-7-7 12	23			262
0.61m [262.42m]									
GLACIAL TILL: Very stiff to hard, brown silty clay some sand, trace gravel		2	SS 3	67	3-10-9-11 19	18			261
			SS 4	100	4-9-13-18 22	19			
		3	SS 5	100	4-12-13-18 25	20			260
		4	SS 6	100	6-13-16-21 29	19			259
		5	SS 7	100	5-9-12-19 21	21			258
5.46m [257.57m]									
GLACIAL TILL: Very stiff to hard, grey silty clay, trace gravel		6	SS 8	100	4-8-10-13 18	19 18			257
6.71m [256.32m]									
End of Borehole		7	SS 9	100	3-9-7-10 16	19			256
(BH Dry on - December 22, 2025)		8							255
		9							254
		10							253
		11							

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599877.34 **NORTHING:** 4861282.17 **ELEVATION:** 266.50

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 5, 2025

FILE NO. : TG0257

HOLE NO. : BH 4-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
AGRICULTURALLY DISTURBED ZONE: with organics and rootlets 0.20m [266.30m]			AU 1			22			266
Firm, brown **SILTY CLAY** with sand, trace organics 0.69m [265.81m]		1	SS 2	83	9-10-10-13 20	11			
Compact, brown **SILTY SAND** 1.12m [265.38m]						18			265
GLACIAL TILL: Very stiff to hard, brown silty clay, trace gravel		2	SS 3	100	9-11-12-14 23	15			
2.97m [263.53m]			SS 4	100	6-11-15-22 26	18			264
Dense, brown **SILTY SAND** to **SANDY SILT**, some clay 3.73m [262.77m]		3	SS 5	75	10-19-14-25 33	21			263
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel		4	SS 6	83	5-12-10-10 22	16			
									262
		5	SS 7	100	6-10-13-14 23	19			
			SS 8	75	7-9-9-7 18	16			261
		6							
6.71m [259.79m]			SS 9	83	9-12-19-20 31	18			260
End of Borehole		7							
(BH Dry on - December 22, 2025)									259
		8							
									258
		9							
									257
		10							
									256
		11							

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599937.51 **NORTHING:** 4861174.35 **ELEVATION:** 264.25

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 5, 2025

FILE NO. : TG0257

HOLE NO. : BH 5-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
AGRICULTURALLY DISTURBED ZONE: with organics and rootlets 0.30m [263.95m]			AU 1			18			264
Brown **SILTY SAND**, organics and rootlets 0.69m [263.56m]		1	SS 2	67	5-7-8-9 15	17			263
GLACIAL TILL: Very stiff to hard, brown silty clay, some sand, trace gravel 1.83m [262.42m]		2	SS 3	83	4-6-8-8 14				262
Compact, brown **SILTY SAND** 2.59m [261.66m]			SS 4	100	6-12-13-16 25	20 18 20			
Hard, brown **SILTY CLAY** 2.74m [261.51m]		3							261
Compact, brown **SILTY SAND**, some clay 3.51m [260.74m]			SS 5	75	8-13-15-12 28	18 18			
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel		4	SS 6	100	6-8-11-11 19	18		4.27m	260
		5	SS 7	83	4-7-9-14 16	19			259
		6	SS 8	83	5-5-7-8 12	20		5.72m	258
6.71m [257.54m]			SS 9	92	4-5-8-8 13	22			
End of Borehole		7							257
(BH Dry on - December 22, 2025)		8							256
		9							255
		10							254
		11							

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599998.26 **NORTHING:** 4861053.38 **ELEVATION:** 264.09

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 5, 2025

FILE NO. : TG0257

HOLE NO. : BH 6-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									264
AGRICULTURALLY DISTURBED ZONE: with organics and rootlets 0.15m [263.94m]			AU 1			17			
Brown **SILTY CLAY** and organics 0.61m [263.48m]									
GLACIAL TILL: Stiff to hard, brown silty clay with some sand and garvel		1	SS 2	100	7-10-16-23 26	16			263
		2	SS 3	100	6-10-16-23 26	17			262
			SS 4	100	4-11-17-22 28	19			
3.00m [261.09m]		3							261
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel			SS 5	100	3-6-8-10 14	17			
		4	SS 6	92	4-5-7-9 12	19			260
		5	SS 7	100	3-6-9-13 15	19			259
			SS 8	100	5-7-9-9 16	19			
6.71m [257.38m]		6	SS 9	100	4-6-10-11 16				258
End of Borehole		7							257
(BH Dry on - December 22, 2025)		8							256
		9							255
		10							254
		11							

3.58m

6.63m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599907.52 **NORTHING:** 4861030.66 **ELEVATION:** 265.30

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 8, 2025

FILE NO. : TG0257

HOLE NO. : BH 7-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
FILL: Stiff, brown silty sand with clay and gravel			AU 1			17			265
0.61m [264.69m]									
GLACIAL TILL: Stiff to hard, brown silty clay, some sand and gravel		1	SS 2	50	3-2-3-4 5				264
		2	SS 3	50	3-3-6-9 9	24			263
			SS 4	100	5-7-10-12 17	17			
		3	SS 5	100	5-11-13-14 24	18			262
		4	SS 6	67	4-8-8-14 16	17			261
5.18m [260.12m]		5	SS 7	42	5-10-17-15 27	19			
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt, some gravel			SS 8	100	3-5-7-10 12	25			260
6.71m [258.59m]		6	SS 9	83	4-6-8-8 14	20			259
End of Borehole		7							258
(GWL at 3.03 m depth - December 22, 2025)		8							257
		9							256
		10							255
		11							

3.03 m - 2025-12-22

6.03m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599873.80 **NORTHING:** 4861067.65 **ELEVATION:** 266.20

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 8, 2025

FILE NO. : TG0257

HOLE NO. : BH 8-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80	REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80	PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE											266
TOPSOIL 0.05m [266.15m]			AU 1			9					
FILL: Brown silty sand with clay and organics		1	SS 2	42	3-5-5-4 10	19					265
		2	SS 3	75	8-4-7-6 11	16					264
2.44m [263.76m]			SS 4	100	6-9-11-14 20	19 24					
GLACIAL TILL: Very stiff to hard, brown silty clay to clayey silt with some sand and gravel		3	SS 5	83	3-7-10-13 17	18				3.00m	263
		4	SS 6	100	4-16-15-18 31	17				4.35 m 2025-12-22	262
4.72m [261.48m]											
Dense, brown **SILTY SAND** with clay		5	SS 7	100	5-17-27-23 44	20					261
5.18m [261.02m]											
GLACIAL TILL: Very stiff, grey silty clay to clayey silt with some sand and gravel		6	SS 8	100	3-6-12-15 18	13				6.05m	260
6.71m [259.49m]			SS 9	75	2-6-8-9 14	19					
End of Borehole		7									259
(GWL at 4.35 m depth - December 22, 2025)		8									258
		9									257
		10									256
		11									

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599849.33 **NORTHING:** 4860935.63 **ELEVATION:** 265.27

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 8, 2025

FILE NO. : TG0257

HOLE NO. : BH 9-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
TOPSOIL 0.05m [265.22m]			AU 1			14			265
Stiff to very stiff, brown **SILTY CLAY** to **CLAYEY SILT** with organics		1	SS 2	50	5-6-6-6 12	23			264
2.13m [263.14m]		2	SS 3	75	2-3-3-4 6	20			263
GLACIAL TILL: Very stiff to hard, brown silty clay to clayey silt with some sand and gravel			SS 4	83	4-8-12-13 20	20			
3.96m [261.31m]		3	SS 5	100	3-6-9-12 15	17			262
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel		4	SS 6	100	3-9-12-14 21	17			261
		5	SS 7	100	2-5-10-11 15	20			260
			SS 8	100	5-10-10-10 20	15			
6.71m [258.56m]		6	SS 9	83	4-10-6-6 16	19			259
End of Borehole		7							258
		8							257
		9							256
		10							255
		11							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599751.17 **NORTHING:** 4860975.99 **ELEVATION:** 263.78

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** December 9, 2025

FILE NO. : TG0257

HOLE NO. : BH10-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
AGRICULTURALLY DISTURBED ZONE: with organics and rootlets			AU 1			21			
0.15m [263.63m]									
Brown **SILTY CLAY** with organics									
0.71m [263.07m]									
GLACIAL TILL: Very stiff to hard, brown silty clay with some sand and gravel		1	SS 2	33	2-8-10-16 18	19			263
		2	SS 3	83	2-9-9-13 18	21		1.50 m 2025-12-22	262
		3	SS 4	100	3-10-10-12 20	19		2.62m	261
3.35m [260.43m]			SS 5	100	3-7-7-8 14	21 14			260
GLACIAL TILL: Stiff to very stiff, grey silty clay to clayey silt with some sand and gravel		4	SS 6	100	2-3-6-6 9	20		4.14m	259
		5	SS 7	100	2-3-6-7 9	21			258
5.94m [257.84m]		6	SS 8	67	3-5-7-8 12	20			257
End of Borehole		7							256
(GWL at 1.50 m depth - December 22, 2025)		8							255
		9							254
		10							253
		11							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599818.32 **NORTHING:** 4861156.18 **ELEVATION:** 265.19

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: EGM96 (Global) **DATE:** January 28, 2025

FILE NO. : TG0257

HOLE NO. : BH11-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE							265
AGRICULTURALLY DISTURBED ZONE: with organics and rootlets 0.18m [265.01m]			G 1			23	
Firm to stiff, brown to grey **SILTY CLAY**, trace gravel and sand 0.69m [264.50m]		1	SS 2	58	4-6-11-14 17	19	264
GLACIAL TILL: Hard to very stiff, brown silty clay with some sand and gravel		2	SS 3	100	2-7-10-11 17	18	263
2.79m [262.40m]			SS 4	75	P	16 26	
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel		3	SS 5	83	2-12-11-12 23	20	262
		4	SS 6	71	7-11-13-16 24	17	261
		5	SS 7	100	4-9-17-21 26	18	260
		6	SS 8	100	3-10-16-17 26	19	259
6.71m [258.48m]			SS 9	100	3-12-13-21 25	18	
End of Borehole		7					258
(BH Dry on - February 4, 2025)		8					257
(GWL at 4.45 m depth - December 22, 2025)		9					256
		10					255
		11					

PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

REMOULDED SHEAR STRENGTH (kPa)

UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

MONITORING WELL CONSTRUCTION

249

4.45 m 2025-12-22

5.97m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 600055.24 **NORTHING:** 4861150.74 **ELEVATION:** 263.33

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: EGM96 (Global) **DATE:** January 28, 2025

FILE NO. : TG0257

HOLE NO. : BH12-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE							263
AGRICULTURALLY DISTURBED ZONE: with organics and rootlets			G 1			16	
0.23m [263.10m]							
Brown **SILTY CLAY**, trace roots to 0.6 m depth		1	SS 2	100	3-5-7-13 12	17	
1.20m [262.13m]							
Compact, brown **SILTY SAND**, trace gravel		2	SS 3	75	2-11-15-17 26	20	262
			SS 4	50	9-11-13-14 24	23	261
3.20m [260.13m]		3					
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel			SS 5	75	5-10-8-9 18	22 19	260
		4	SS 6	0	6-8-12-16 20		259
		5	SS 7	100	2-12-14-20 26	17	258
			SS 8	75	5-11-17-26 28	17	
		6	SS 9	100	7-16-20-25 36	16	257
6.71m [256.62m]							
Dynamic Cone Penetration Test commenced at 6.71 m depth		7					256
8.23m [255.10m]		8					255
End of Borehole							
(GWL at 1.30 m depth - February 4, 2025)		9					254
(GWL at 2.96 m depth - December 22, 2025)		10					253
		11					

PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)

20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)

▲ UNDRAINED SHEAR STRENGTH (kPa)

20 40 60 80

PL (%) WATER CONTENT (%) LL (%)

20 40 60 80

45

52

68

118

153

MONITORING WELL CONSTRUCTION

1.30 m 2025-02-04

2.96 m 2025-12-22

4.51m

6.03m

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

14337 Highway 50, Bolton, Ontario

COORD. SYS.: UTM ZONE 17N **EASTING:** 599822.52 **NORTHING:** 4860874.63 **ELEVATION:** 265.25

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: EGM96 (Global) **DATE:** January 28, 2025

FILE NO. : TG0257

HOLE NO. : BH13-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80	REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80	PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE											265
TOPSOIL 0.05m [265.20m]			G 1			21					
GLACIAL TILL: Very stiff to hard, brown silty clay to clayey silt with some sand and gravel		1	SS 2	100	2-5-10-16 15	20				1.04 m 2026-01-08	
		2	SS 3	100	3-9-10-16 19	19				1.33 m 2025-02-04	263
		3	SS 4	100	4-12-18-29 30	19					
			SS 5	100	3-8-14-16 22	19					262
4.57m [260.68m]		4	SS 6	8	3-6-11-14 17	20				4.26m	261
GLACIAL TILL: Very stiff to hard, grey silty clay to clayey silt with some sand and gravel		5	SS 7	100	3-8-17-28 25	19					
		6	SS 8	100	3-7-14-13 21	23				5.78m	260
6.71m [258.54m]			SS 9		5-6-14-18 20	18					259
End of Borehole		7									258
(GWL at 1.33 m depth - February 4, 2025)		8									257
(GWL at 1.04 m depth - January 08, 2026)		9									256
		10									255
		11									

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

P:/AutoCAD Drawings/Test Hole Data Files/TG02xx/TG0257/data.sqlite 2026-01-23, 08:24 Paterson_Template JM

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their "sensitivity". The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called "mechanical breaks") are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu	-	Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

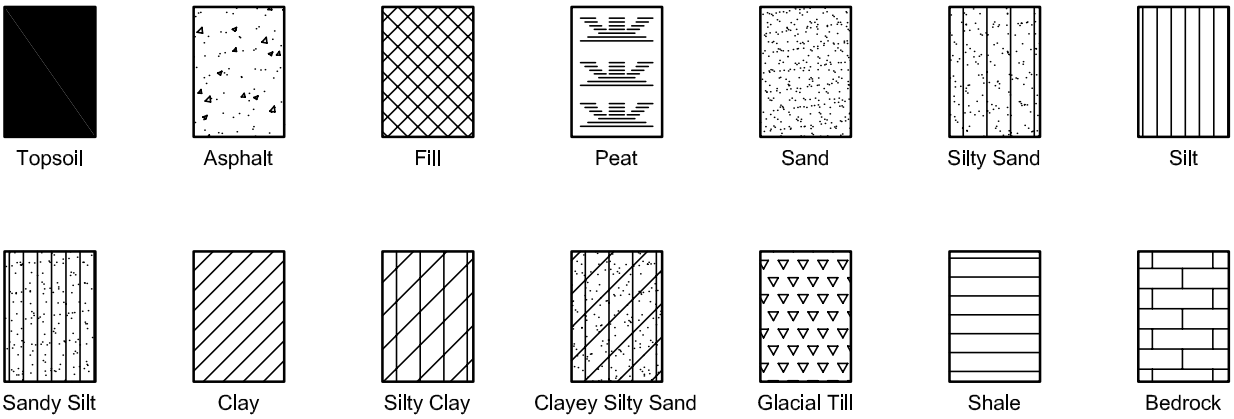
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

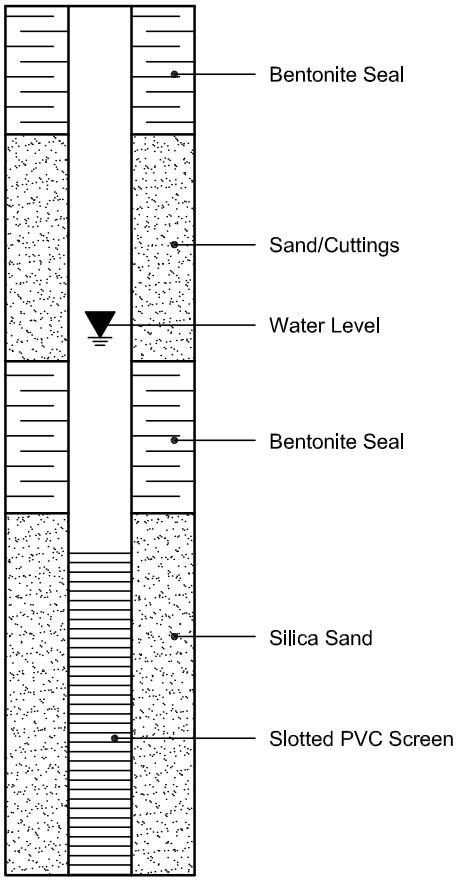
SYMBOLS AND TERMS (continued)

STRATA PLOT

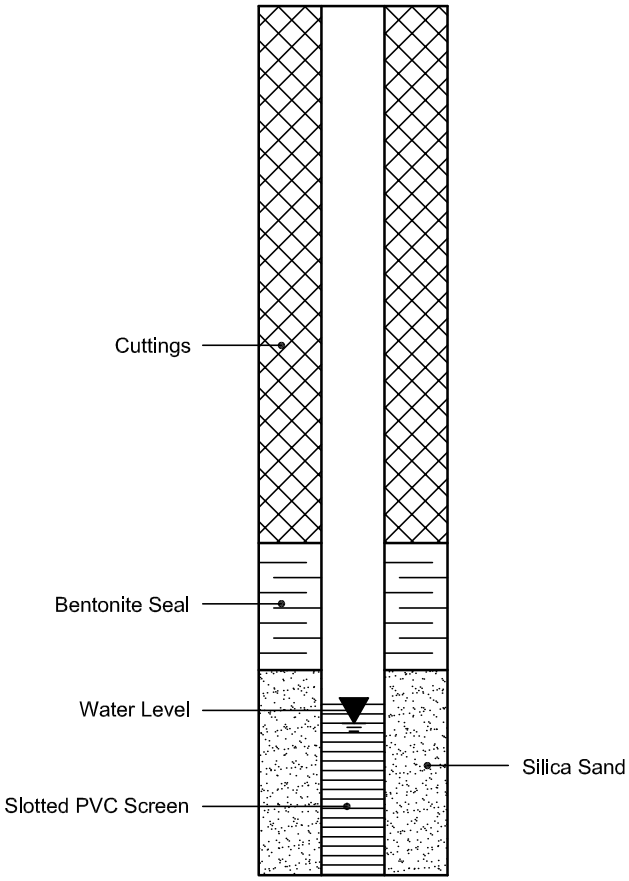


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



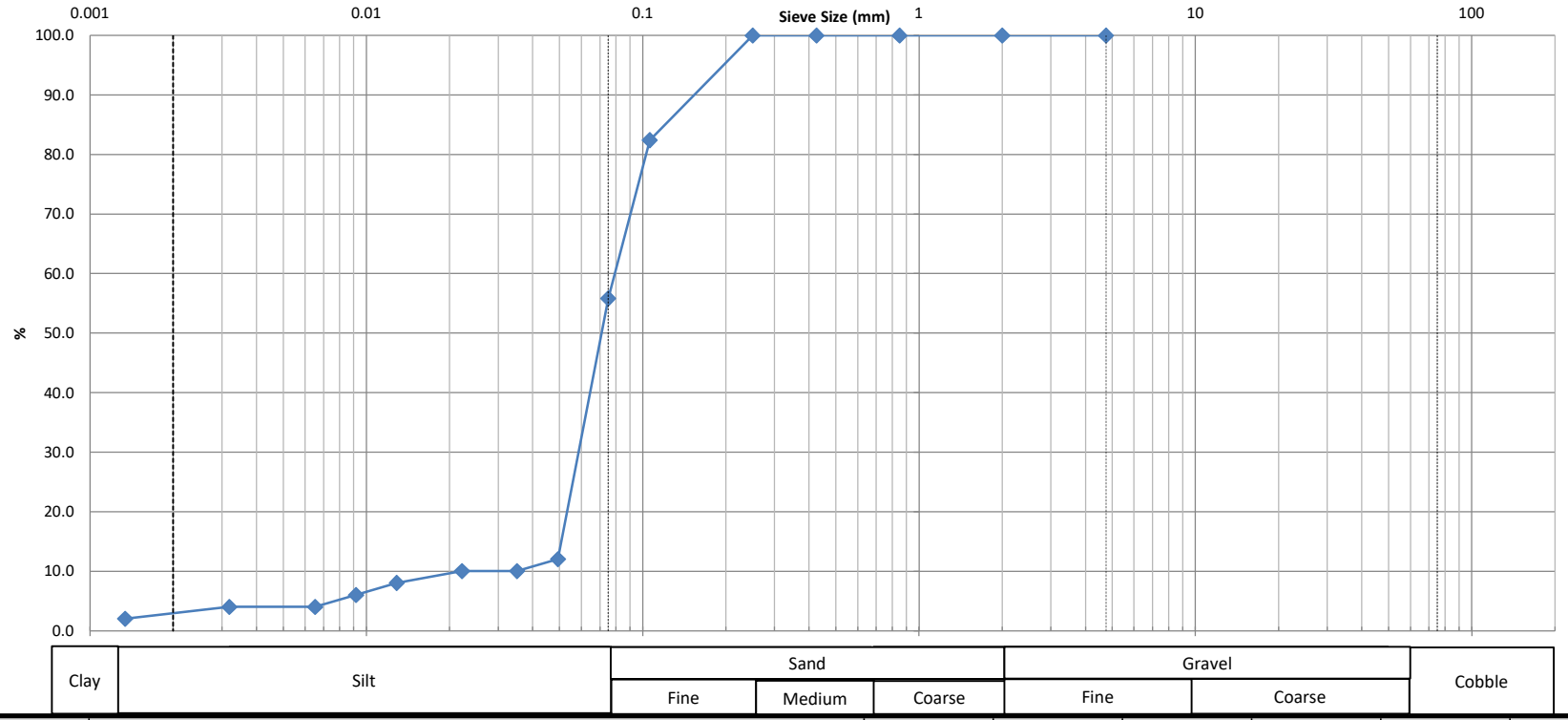
PIEZOMETER CONSTRUCTION



PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	1.5-2.1 m	FILE NO:	TG0257
CONTRACT NO.:		BH OR TP No.:	BH1-25 SS3	LAB NO:	112
PROJECT:	14337 Highway 50, Caledon			DATE RECEIVED:	9-Dec-25
				DATE TESTED:	11-Dec-25
DATE SAMPLED:	9-Dec-25			DATE REPORTED:	14-Dec-25
SAMPLED BY:	B.N			TESTED BY:	S.E



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						22.4%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.0	44.2		53.1		2.7	

Comments: The soil is classified as brown sandy silt with trace clay.

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	1.5-2.1 m	FILE NO.:	TG0257
PROJECT:	14337 Highway 50, Caledon	BH OR TP No.:	BH1-25 SS3	DATE SAMPLED:	09-Dec-25
LAB No. :	112	TESTED BY:	S.E	DATE RECEIVED:	09-Dec-25
SAMPLED BY:	B.N	DATE REPT'D:	14-Dec-25	DATE TESTED:	11-Dec-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
125.0	2.700

INITIAL WEIGHT	50.10	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	49.27	TARE WEIGHT	169.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	22.30	AIR DRY	296.10	127.10
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	294.00	125.00
		CORRECTED	0.983	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75	**0.0**	0.0	**100.0**
2.0	**0.0**	0.0	**100.0**
Pan	**124.6**		
0.850		0.0	**100.0**
0.425		0.0	**100.0**
0.250		0.0	**100.0**
0.106	**8.80**	17.6	**82.4**
0.075	**22.12**	44.2	**55.8**
Pan	**22.30**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (°C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	10:05	11.0	5.0	23.0	0.0494	12.0	**12.0**
2	10:06	10.0	5.0	23.0	0.0351	10.0	**10.0**
5	10:09	10.0	5.0	23.0	0.0222	10.0	**10.0**
15	10:19	9.0	5.0	23.0	0.0129	8.0	**8.0**
30	10:34	8.0	5.0	23.0	0.0092	6.0	**6.0**
60	11:04	7.0	5.0	23.0	0.0065	4.0	**4.0**
250	14:14	7.0	5.0	23.0	0.0032	4.0	**4.0**
1440	10:04	6.0	5.0	23.0	0.0013	2.0	**2.0**

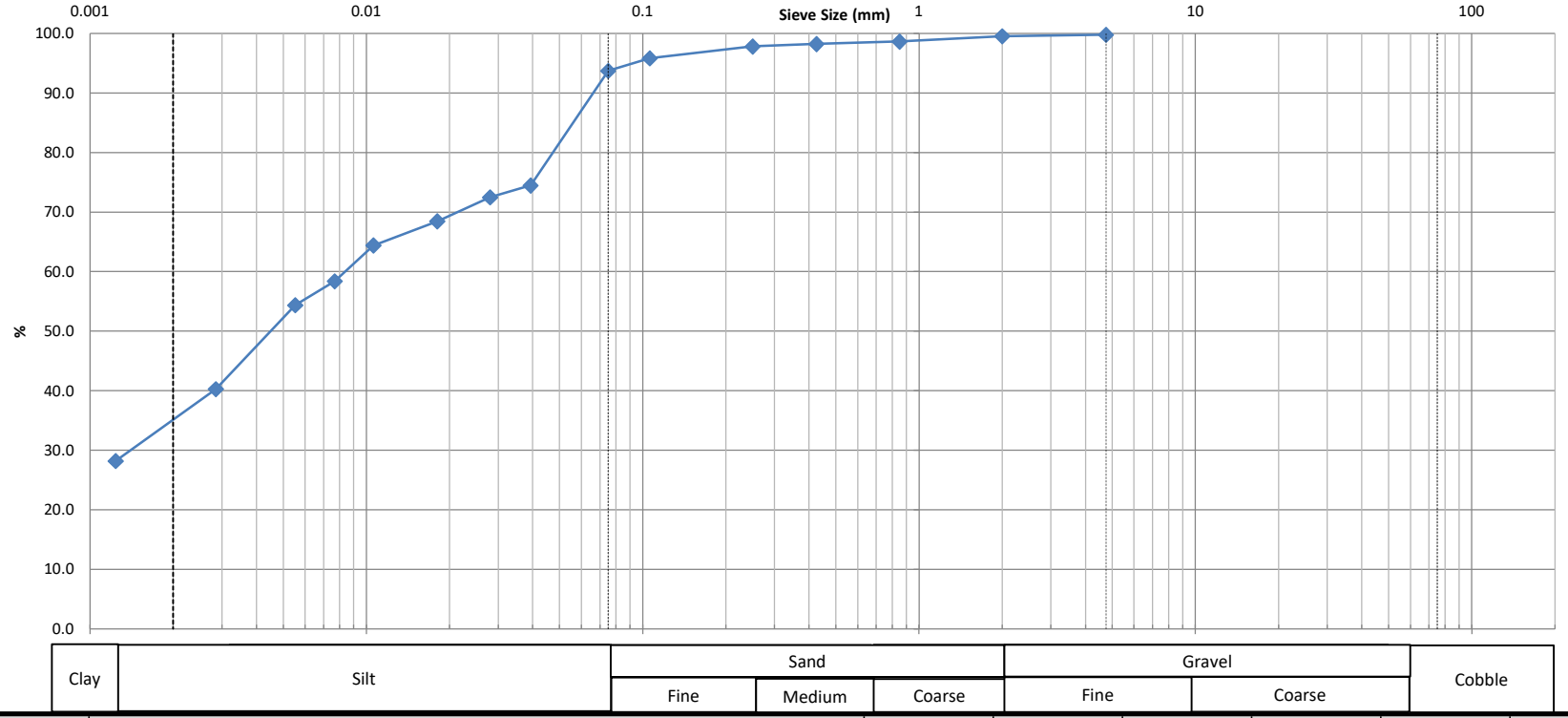
Moisture = 22.4%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	1.5 – 2.1 m	FILE NO:	TG0257
CONTRACT NO.:		BH OR TP No.:	BH5-25 SS3	LAB NO:	113
PROJECT:	14337 Highway 50			DATE RECEIVED:	9-Dec-25
				DATE TESTED:	11-Dec-25
DATE SAMPLED:	9-Dec-25			DATE REPORTED:	14-Dec-25
SAMPLED BY:	B.N			TESTED BY:	S.E



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						20.7%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.2	6.1		59.9		33.9	

Comments: This soil is classified as brown clayey silt with trace sand.

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	1.5 – 2.1 m	FILE NO.:	TG0257
PROJECT:	14337 Highway 50	BH OR TP No.:	BH5-25 SS3	DATE SAMPLED:	09-Dec-25
LAB No. :	113	TESTED BY:	S.E	DATE RECEIVED:	09-Dec-25
SAMPLED BY:	B.N	DATE REPT'D:	14-Dec-25	DATE TESTED:	11-Dec-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
154.0	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	48.89	TARE WEIGHT	166.60	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	2.92	AIR DRY	324.10	157.50
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	320.60	154.00
		CORRECTED	0.978	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75	**0.3**	0.2	**99.8**
2.0	**0.7**	0.5	**99.5**
Pan	**152.5**		
0.850	**0.45**	1.4	**98.6**
0.425	**0.65**	1.7	**98.3**
0.250	**0.86**	2.2	**97.8**
0.106	**1.86**	4.2	**95.8**
0.075	**2.91**	6.2	**93.8**
Pan	**2.92**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (°C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	10:25	42.0	5.0	23.0	0.0393	74.8	**74.5**
2	10:26	41.0	5.0	23.0	0.0281	72.8	**72.5**
5	10:29	39.0	5.0	23.0	0.0181	68.8	**68.5**
15	10:39	37.0	5.0	23.0	0.0106	64.7	**64.4**
30	10:54	34.0	5.0	23.0	0.0077	58.7	**58.4**
60	11:24	32.0	5.0	23.0	0.0055	54.6	**54.4**
250	14:34	25.0	5.0	23.0	0.0029	40.5	**40.3**
1440	10:24	19.0	5.0	23.0	0.0012	28.3	**28.2**

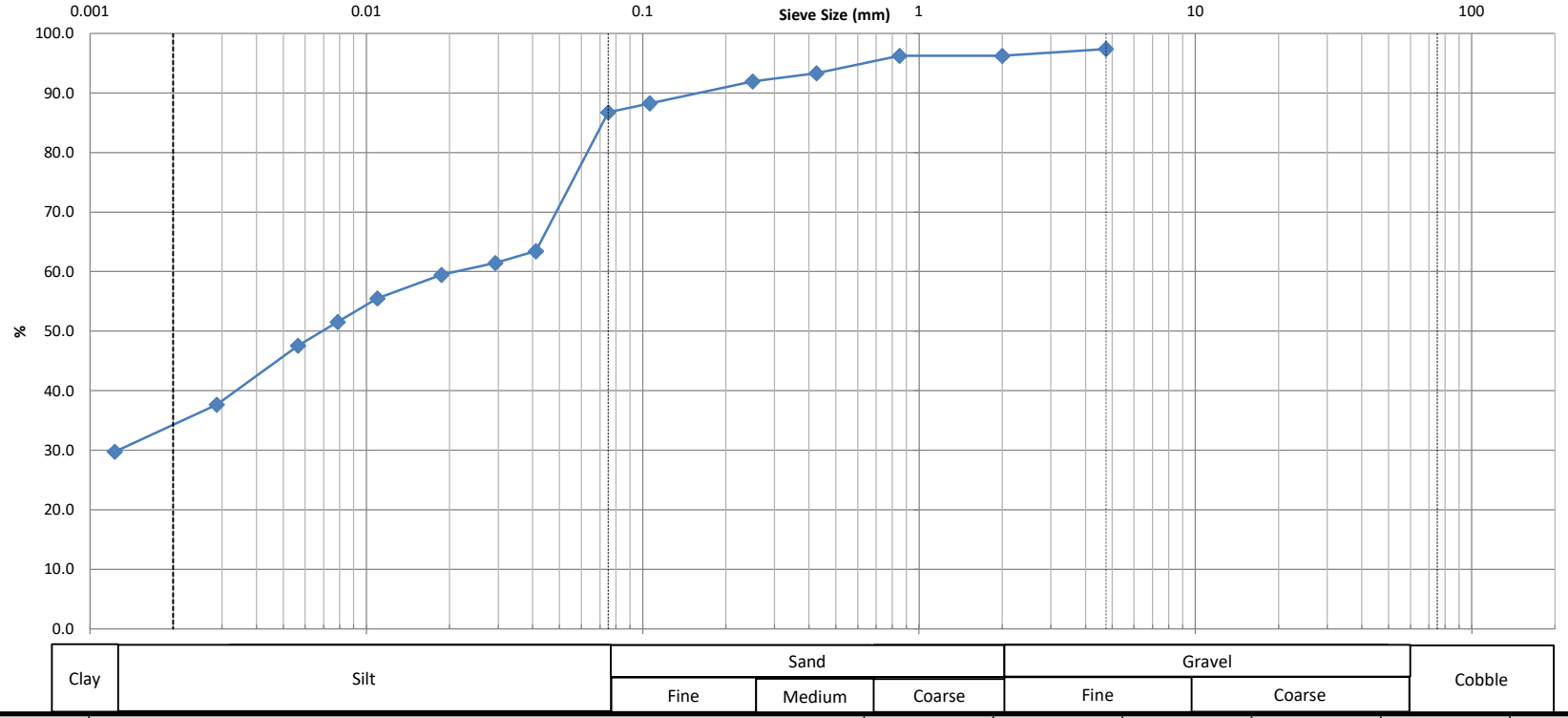
Moisture = 20.7%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	2.3 - 2.9 m	FILE NO:	TG0257
CONTRACT NO.:		BH OR TP No.:	BH7-25 SS4	LAB NO:	114
PROJECT:	14337 Highway 50			DATE RECEIVED:	9-Dec-25
				DATE TESTED:	11-Dec-25
DATE SAMPLED:	9-Dec-25			DATE REPORTED:	14-Dec-25
SAMPLED BY:	B.N			TESTED BY:	S.E



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						17.7%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					2.6	10.7		53.3		33.4	

Comments: This soil is classified as brown clayey silt with minor sand and trace gravel.

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	2.3 - 2.9 m	FILE NO.:	TG0257
PROJECT:	14337 Highway 50	BH OR TP No.:	BH7-25 SS4	DATE SAMPLED:	09-Dec-25
LAB No. :	114	TESTED BY:	S.E	DATE RECEIVED:	09-Dec-25
SAMPLED BY:	B.N	DATE REPT'D:	14-Dec-25	DATE TESTED:	11-Dec-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
155.3	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	48.02	TARE WEIGHT	167.40	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	4.98	AIR DRY	329.10	161.70
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	322.70	155.30
		CORRECTED	0.960	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75	**4.0**	2.6	**97.4**
2.0	**5.8**	3.7	**96.3**
Pan	**149.0**		
0.850	**0.00**	3.7	**96.3**
0.425	**1.52**	6.7	**93.3**
0.250	**2.24**	8.0	**92.0**
0.106	**4.14**	11.7	**88.3**
0.075	**4.94**	13.2	**86.8**
Pan	**4.98**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (°C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	10:45	37.0	5.0	23.0	0.0411	65.9	**63.4**
2	10:46	36.0	5.0	23.0	0.0293	63.8	**61.4**
5	10:49	35.0	5.0	23.0	0.0187	61.8	**59.5**
15	10:59	33.0	5.0	23.0	0.0110	57.7	**55.5**
30	11:14	31.0	5.0	23.0	0.0079	53.5	**51.5**
60	11:44	29.0	5.0	23.0	0.0057	49.4	**47.6**
250	14:54	24.0	5.0	23.0	0.0029	39.1	**37.7**
1440	10:44	20.0	5.0	23.0	0.0012	30.9	**29.7**

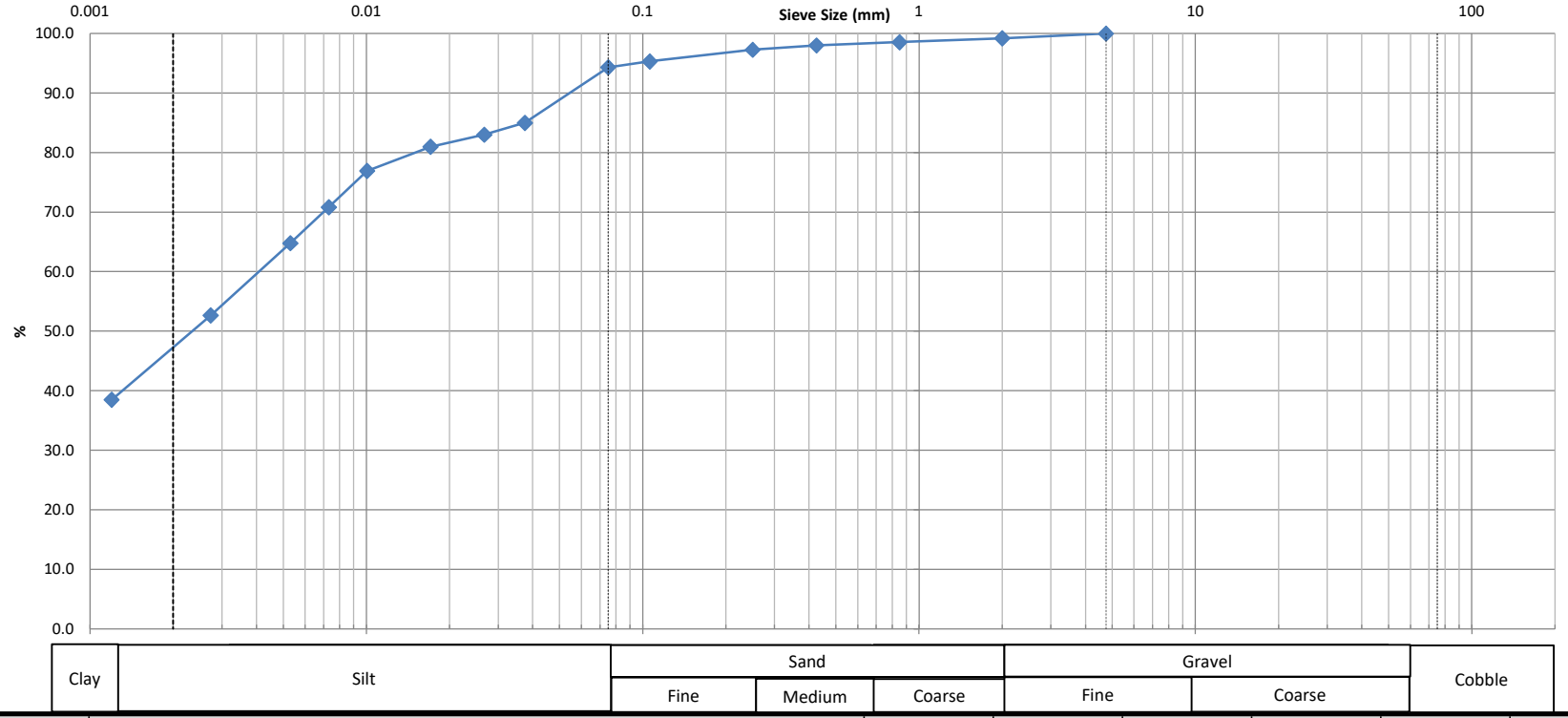
Moisture = 17.7%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	2.3 - 2.9 m	FILE NO:	TG0257
CONTRACT NO.:		BH OR TP No.:	BH9-25 SS4	LAB NO:	115
PROJECT:	14337 Highway 50			DATE RECEIVED:	9-Dec-25
				DATE TESTED:	11-Dec-25
DATE SAMPLED:	9-Dec-25			DATE REPORTED:	14-Dec-25
SAMPLED BY:	B.N			TESTED BY:	S.E



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						21.1%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.0	5.7		48.5		45.9	

Comments: This soil is classified as brown clayey silt with trace sand.

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	2.3 - 2.9 m	FILE NO.:	TG0257
PROJECT:	14337 Highway 50	BH OR TP No.:	BH9-25 SS4	DATE SAMPLED:	09-Dec-25
LAB No. :	115	TESTED BY:	S.E	DATE RECEIVED:	09-Dec-25
SAMPLED BY:	B.N	DATE REPT'D:	14-Dec-25	DATE TESTED:	11-Dec-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
153.8	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.10			
WEIGHT CORRECTED	48.46	TARE WEIGHT	166.30	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	2.47	AIR DRY	325.30	159.00
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	320.10	153.80
		CORRECTED	0.967	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75	**0.0**	0.0	**100.0**
2.0	**1.2**	0.8	**99.2**
Pan	**151.1**		
0.850	**0.34**	1.5	**98.5**
0.425	**0.61**	2.0	**98.0**
0.250	**0.98**	2.7	**97.3**
0.106	**1.96**	4.7	**95.3**
0.075	**2.46**	5.7	**94.3**
Pan	**2.47**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (°C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	11:08	47.0	5.0	23.0	0.0374	85.7	**85.0**
2	11:09	46.0	5.0	23.0	0.0267	83.7	**83.0**
5	11:12	45.0	5.0	23.0	0.0171	81.6	**81.0**
15	11:22	43.0	5.0	23.0	0.0101	77.5	**76.9**
30	11:37	40.0	5.0	23.0	0.0073	71.4	**70.9**
60	12:07	37.0	5.0	23.0	0.0053	65.3	**64.8**
250	15:17	31.0	5.0	23.0	0.0027	53.0	**52.6**
1440	11:07	24.0	5.0	23.0	0.0012	38.8	**38.5**

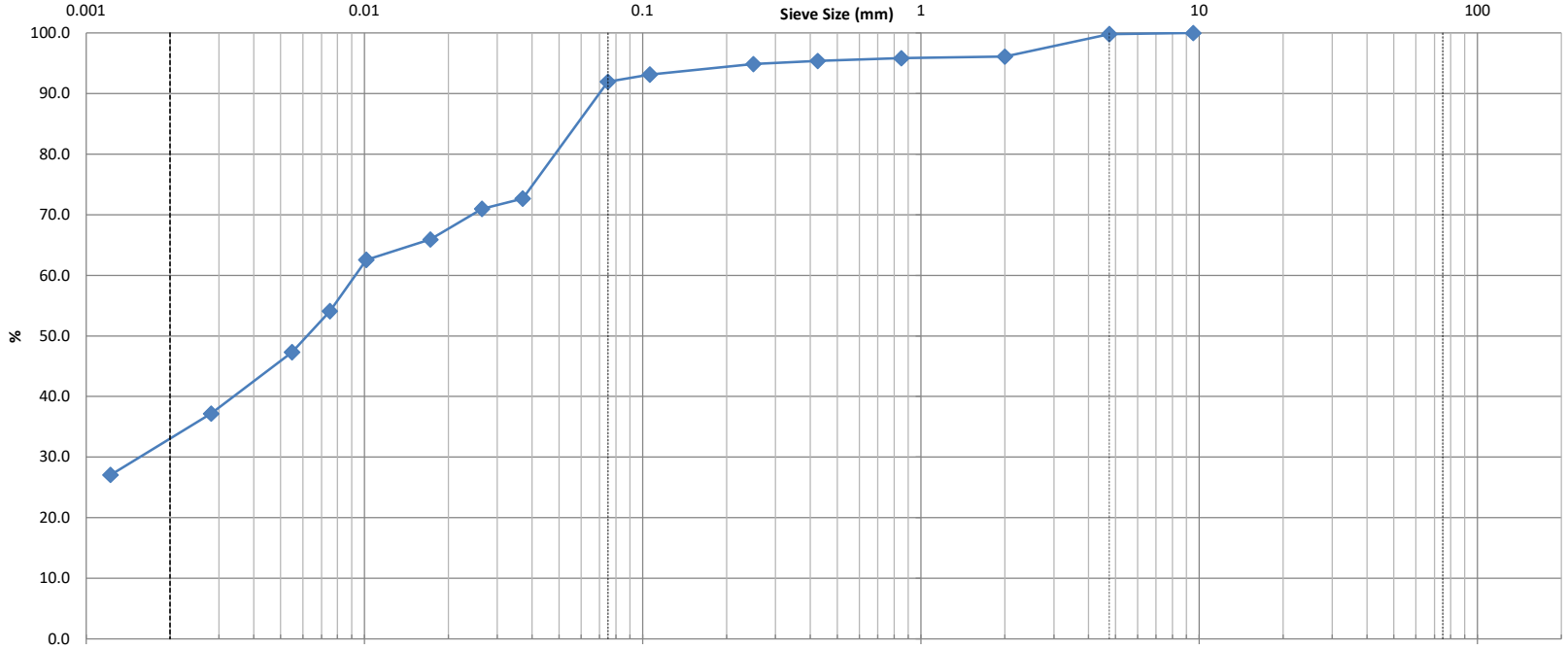
Moisture = 21.1%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	3.0 - 3.7 m	FILE NO:	TG0257
CONTRACT NO.:		BH OR TP No.:	BH11-25 SS5	LAB NO:	00054
PROJECT:	Highway 50, Bolton			DATE RECEIVED:	28-Jan-25
				DATE TESTED:	27-Feb-25
DATE SAMPLED:	28-Jan-25			DATE REPORTED:	1-Mar-24
SAMPLED BY:	JM			TESTED BY:	J.R



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					25.0%					

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
				0.2	7.9	59.5	32.5

Comments:

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	3.0 - 3.7 m	FILE NO.:	TG0257
PROJECT:	Highway 50, Bolton	BH OR TP No.:	BH11-25 SS5	DATE SAMPLED:	28-Jan-25
LAB No. :	00054	TESTED BY:	J.R	DATE RECEIVED:	28-Jan-25
SAMPLED BY:	JM	DATE REPT'D:	01-Mar-24	DATE TESTED:	27-Feb-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
100.0	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	48.90	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	2.20	AIR DRY	100.00	100.00
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	97.80	97.80
		CORRECTED	0.978	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5	**0.0**	0.0	**100.0**
4.75	**0.2**	0.2	**99.8**
2.0	**3.9**	3.9	**96.1**
Pan	**96.1**		
0.850	**0.14**	4.2	**95.8**
0.425	**0.37**	4.6	**95.4**
0.250	**0.64**	5.1	**94.9**
0.106	**1.53**	6.8	**93.2**
0.075	**2.15**	8.0	**92.0**
Pan	**2.20**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (°C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	9:29	48.0	5.0	23.0	0.0371	75.6	**72.7**
2	9:30	47.0	5.0	23.0	0.0265	73.9	**71.0**
5	9:33	44.0	5.0	23.0	0.0173	68.6	**65.9**
15	9:43	42.0	5.0	23.0	0.0102	65.1	**62.5**
30	9:58	37.0	5.0	23.0	0.0075	56.3	**54.1**
60	10:28	33.0	5.0	23.0	0.0055	49.2	**47.3**
250	13:38	27.0	5.0	23.0	0.0028	38.7	**37.2**
1440	9:28	21.0	5.0	23.0	0.0012	28.1	**27.0**

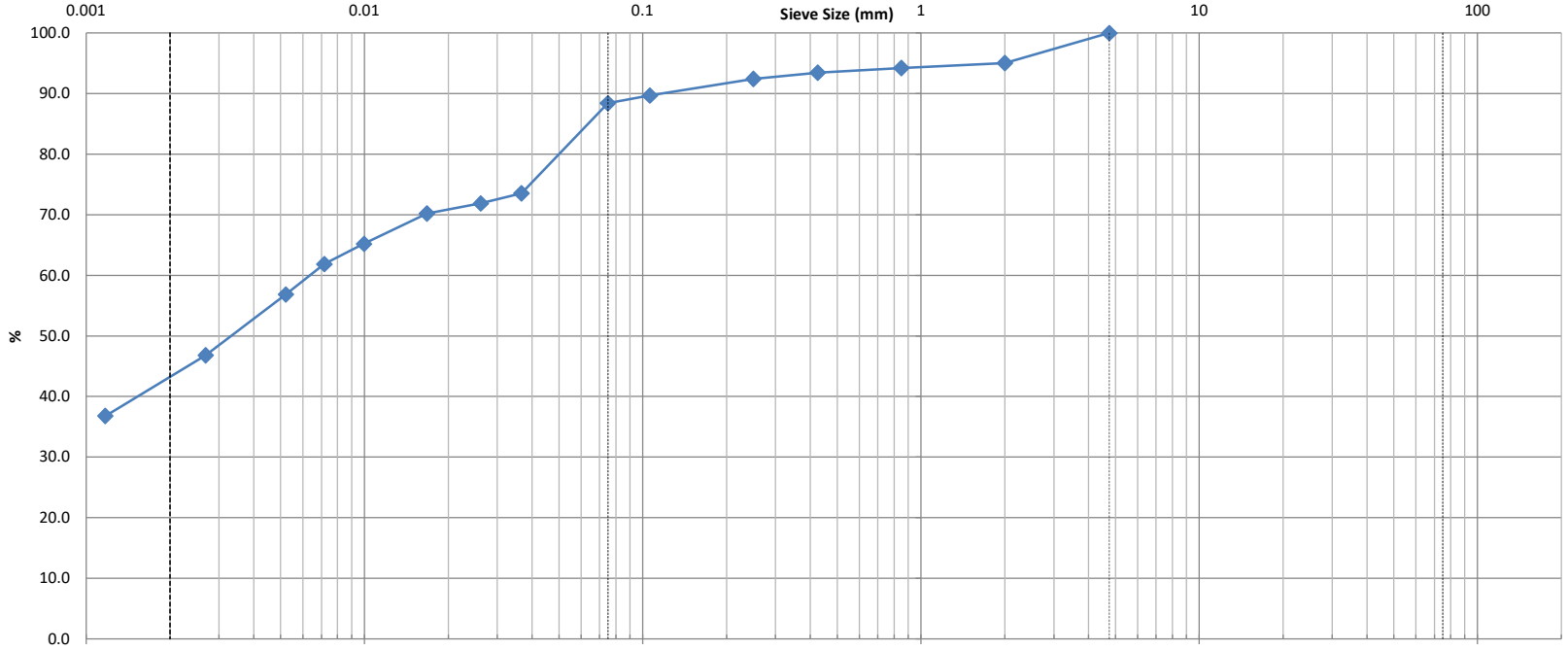
Moisture = 25.0%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	1.5 – 2.1 m	FILE NO:	TG0257
CONTRACT NO.:		BH OR TP No.:	BH13-25 SS3	LAB NO:	00054
PROJECT:	Highway 50, Bolton			DATE RECEIVED:	28-Jan-25
				DATE TESTED:	27-Feb-25
DATE SAMPLED:	28-Jan-25			DATE REPORTED:	1-Mar-24
SAMPLED BY:	JM			TESTED BY:	J.R



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					25.0%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)
					0.0	11.6		45.4		43.0

Comments:

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Tiffany Rox North Hill Ltd.	DEPTH:	1.5 – 2.1 m	FILE NO.:	TG0257
PROJECT:	Highway 50, Bolton	BH OR TP No.:	BH13-25 SS3	DATE SAMPLED:	28-Jan-25
LAB No. :	00054	TESTED BY:	J.R	DATE RECEIVED:	28-Jan-25
SAMPLED BY:	JM	DATE REPT'D:	01-Mar-24	DATE TESTED:	27-Feb-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
97.9	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	48.90	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	3.52	AIR DRY	100.00	100.00
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	97.80	97.80
		CORRECTED	0.978	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75	**0.0**	0.0	**100.0**
2.0	**4.8**	4.9	**95.1**
Pan	**93.1**		
0.850	**0.43**	5.8	**94.2**
0.425	**0.83**	6.5	**93.5**
0.250	**1.38**	7.6	**92.4**
0.106	**2.82**	10.3	**89.7**
0.075	**3.49**	11.6	**88.4**
Pan	**3.52**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (°C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	9:00	49.0	5.0	23.0	0.0367	77.4	**73.6**
2	9:01	48.0	5.0	23.0	0.0262	75.6	**71.9**
5	9:04	47.0	5.0	23.0	0.0167	73.9	**70.2**
15	9:14	44.0	5.0	23.0	0.0100	68.6	**65.2**
30	9:29	42.0	5.0	23.0	0.0072	65.1	**61.9**
60	9:59	39.0	5.0	23.0	0.0052	59.8	**56.8**
250	13:09	33.0	5.0	23.0	0.0027	49.2	**46.8**
1440	8:59	27.0	5.0	23.0	0.0012	38.7	**36.8**

Moisture = 25.0%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

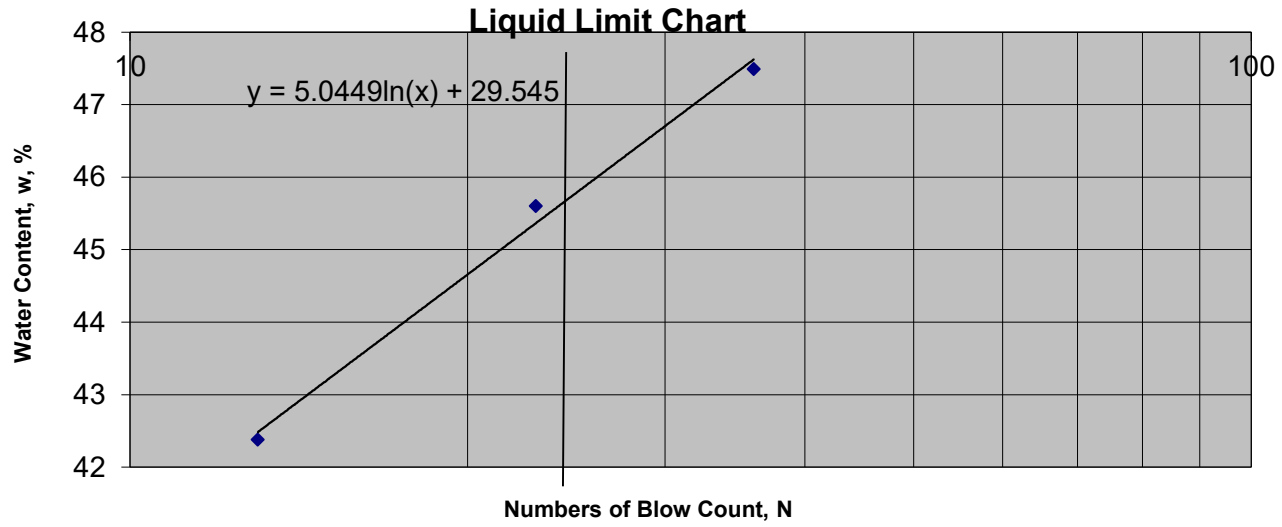
ATTERBERG LIMITS
LS-703/704

CLIENT:	Tiffany Rox North Hill Ltd.	FILE NO.:	TG0257
PROJECT:	116	DATE SAMPLED:	08-Dec
LOCATION:	BH1-25 SS4	DATE REPORTED:	17-Dec

CAN NO.	7	12	13				
WT. OF CAN	13.91	14.01	14.12				
WT. OF SOIL & CAN	45.9	46.90	41.57				
WT. OF DRY SOIL & CAN	35.60	36.60	33.40				
WT. OF MOISTURE	10.3	10.3	8.17				
WT. OF DRY SOIL & CAN	21.69	22.59	19.28				
WATER CONTENT, w, %	**47.49**	**45.6**	**42.38**				
NO. OF BLOWS, N	36	23	13				

CAN NO.	15	16
WT. OF CAN	14	14.13
WT. OF SOIL & CAN	25.46	26.80
WT. OF DRY SOIL & CAN	23.32	24.40
WT. OF MOISTURE	2.14	2.4
WT. OF DRY SOIL & CAN	9.32	10.27
WATER CONTENT, w, %	**22.96**	**23.37**

RESULTS	
LIQUID LIMIT	**45**
PLASTIC LIMIT	**23**
PLASTICITY INDEX	**22**



TECHNICIAN: SE	REVIEWED BY:	C. Beadow	J. Forsyth, P. Eng.

PATERSON GROUP

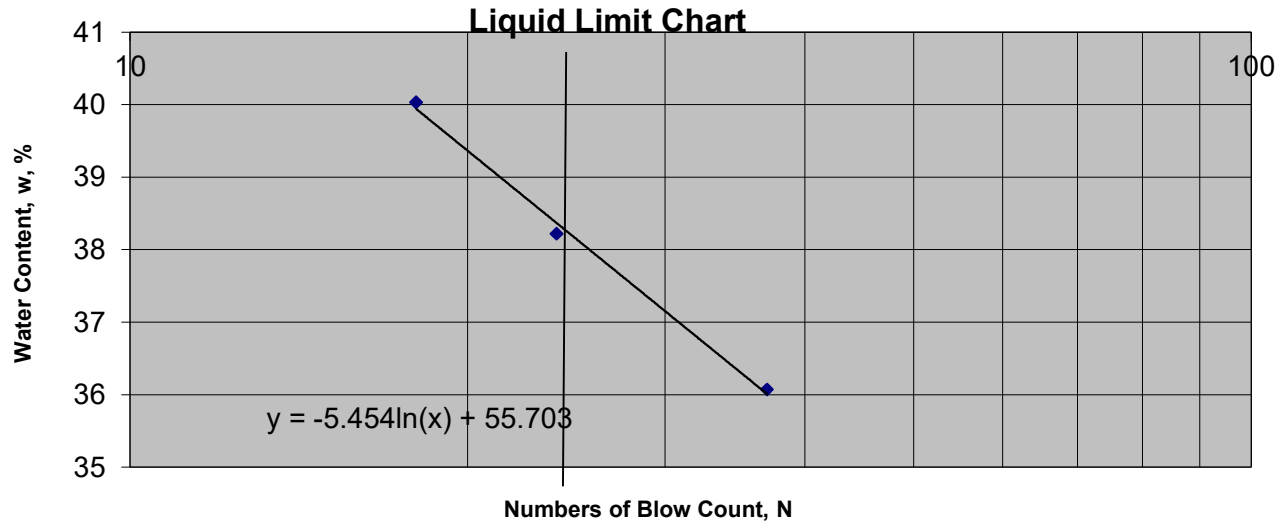
ATTERBERG LIMITS
LS-703/704

CLIENT:	Tiffany Rox North Hill Ltd.	FILE NO.:	TG0257
PROJECT:	117	DATE SAMPLED:	08-Dec
LOCATION:	BH4-25 SS3	DATE REPORTED:	17-Dec

CAN NO.	9	8	10				
WT. OF CAN	13.86	13.92	13.98				
WT. OF SOIL & CAN	42.79	47.70	46.99				
WT. OF DRY SOIL & CAN	34.52	38.36	38.24				
WT. OF MOISTURE	8.27	9.34	8.75				
WT. OF DRY SOIL & CAN	20.66	24.44	24.26				
WATER CONTENT, w, %	**40.03**	**38.22**	**36.07**				
NO. OF BLOWS, N	18	24	37				

CAN NO.	17	18
WT. OF CAN	14.09	13.86
WT. OF SOIL & CAN	34.68	36.71
WT. OF DRY SOIL & CAN	30.66	32.23
WT. OF MOISTURE	4.02	4.48
WT. OF DRY SOIL & CAN	16.57	18.37
WATER CONTENT, w, %	**24.26**	**24.39**

RESULTS	
LIQUID LIMIT	**38**
PLASTIC LIMIT	**24**
PLASTICITY INDEX	**14**



TECHNICIAN: SE	REVIEWED BY:	C. Beadow	J. Forsyth, P. Eng.

PATERSON GROUP

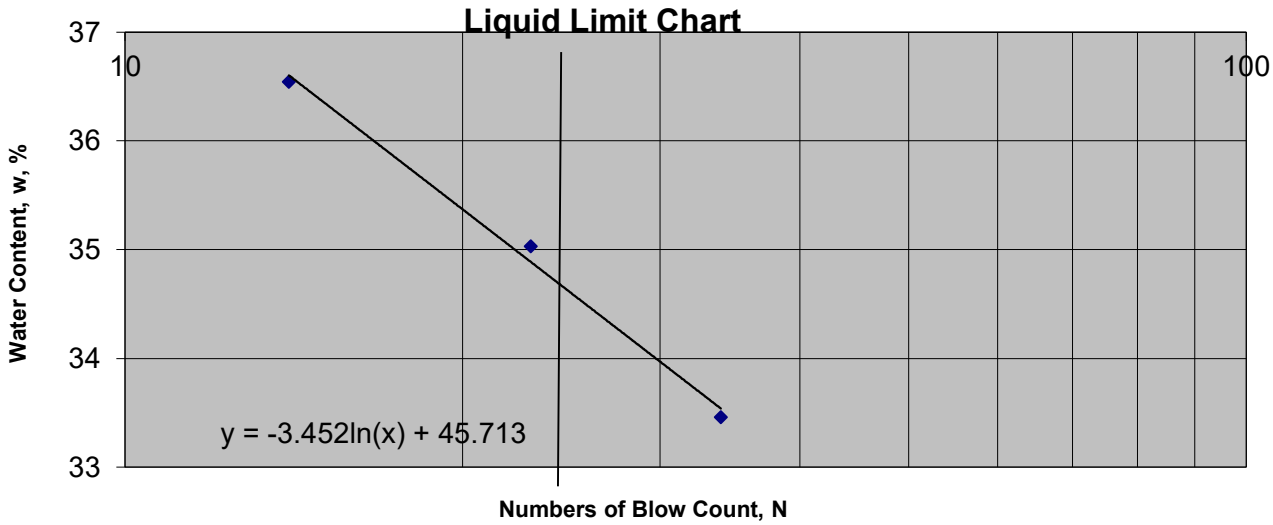
ATTERBERG LIMITS
LS-703/704

CLIENT:	Tiffany Rox North Hill Ltd.	FILE NO.:	TG0257
PROJECT:	118	DATE SAMPLED:	08-Dec
LOCATION:	BH6-25 SS4	DATE REPORTED:	17-Dec

CAN NO.	5	4	11				
WT. OF CAN	14.00	13.91	13.99				
WT. OF SOIL & CAN	50.77	49.80	52.96				
WT. OF DRY SOIL & CAN	40.93	40.49	43.19				
WT. OF MOISTURE	9.84	9.31	9.77				
WT. OF DRY SOIL & CAN	26.93	26.58	29.2				
WATER CONTENT, w, %	**36.54**	**35.03**	**33.46**				
NO. OF BLOWS, N	14	23	34				

CAN NO.	14	2
WT. OF CAN	14.13	13.90
WT. OF SOIL & CAN	29.07	25.00
WT. OF DRY SOIL & CAN	26.42	23.03
WT. OF MOISTURE	2.65	1.97
WT. OF DRY SOIL & CAN	12.29	9.13
WATER CONTENT, w, %	**21.56**	**21.58**

RESULTS	
LIQUID LIMIT	**35**
PLASTIC LIMIT	**22**
PLASTICITY INDEX	**13**



TECHNICIAN: SE	REVIEWED BY:	C. Beadow	J. Forsyth, P. Eng.

PATERSON GROUP

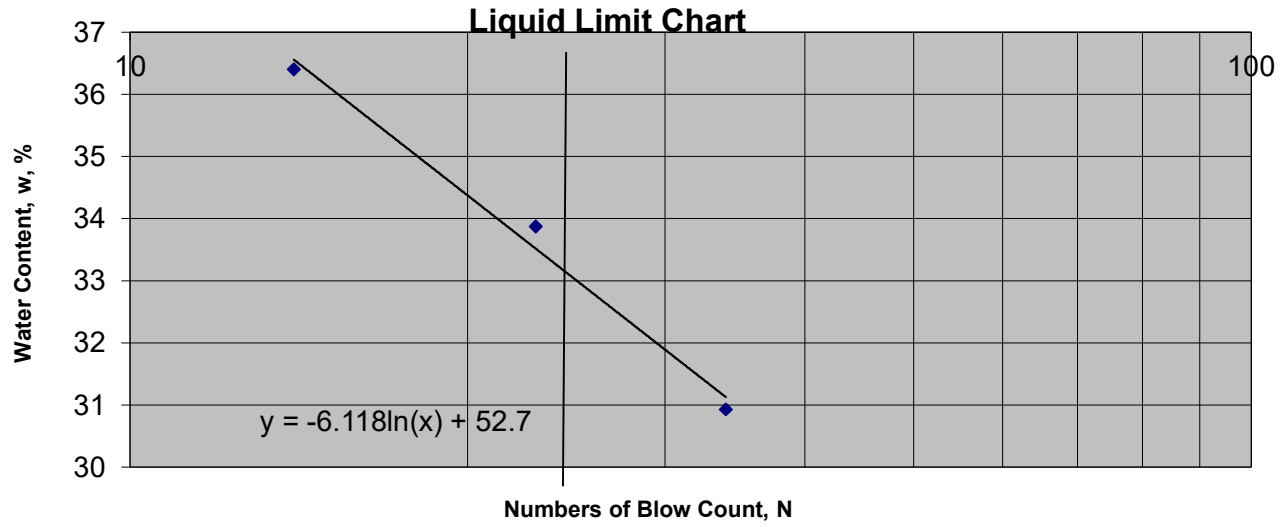
ATTERBERG LIMITS
LS-703/704

CLIENT:	Roxborough Developments	FILE NO.:	TG0257
PROJECT:	112	DATE SAMPLED:	08-Dec
LOCATION:	BH8-25 SS4	DATE REPORTED:	11-Dec

CAN NO.	C16	C32	B9				
WT. OF CAN	38.02	38.02	38.17				
WT. OF SOIL & CAN	83.96	92.21	77.62				
WT. OF DRY SOIL & CAN	71.70	78.50	68.30				
WT. OF MOISTURE	12.26	13.71	9.32				
WT. OF DRY SOIL & CAN	33.68	40.48	30.13				
WATER CONTENT, w, %	**36.4**	**33.87**	**30.93**				
NO. OF BLOWS, N	14	23	34				

CAN NO.	C29	B13
WT. OF CAN	37.9	38.01
WT. OF SOIL & CAN	68.72	64.05
WT. OF DRY SOIL & CAN	63.10	59.30
WT. OF MOISTURE	5.62	4.75
WT. OF DRY SOIL & CAN	25.2	21.29
WATER CONTENT, w, %	**22.3**	**22.31**

RESULTS	
LIQUID LIMIT	**34**
PLASTIC LIMIT	**22**
PLASTICITY INDEX	**12**



TECHNICIAN: SE	REVIEWED BY:	C. Beadow	J. Forsyth, P. Eng.

Certificate of Analysis

Report Date: 17-Dec-2025

Client: Paterson Group Consulting Engineers (Mississauga)

Order Date: 11-Dec-2025

Client PO: 64700

Project Description: TG0257

	Client ID:	BH-10-25-SS3	-	-	-
	Sample Date:	09-Dec-25 12:00	-	-	-
	Sample ID:	2550304-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	82.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.59	-	-	-
Resistivity	0.1 Ohm.m	37.6	-	-	-
Anions					
Chloride	10 ug/g dry	<10	-	-	-
Sulphate	10 ug/g dry	100	-	-	-

Certificate of Analysis

Report Date: 19-Jan-2026

Client: **Paterson Group Consulting Engineers (Mississauga)**

Order Date: 29-Jan-2025

Client PO: **62254**

Project Description: TG0257

	Client ID:	BH12-25-SS4	-	-	-
	Sample Date:	28-Jan-25 12:00	-	-	-
	Sample ID:	2505304-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	83.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.27	-	-	-
Resistivity	0.10 Ohm.m	91.2	-	-	-
Anions					
Chloride	5 ug/g dry	7	-	-	-
Sulphate	5 ug/g dry	7	-	-	-

Certificate of Analysis

Report Date: 19-Jan-2026

Client: **Paterson Group Consulting Engineers (Mississauga)**

Order Date: 31-Jan-2025

Client PO: **62254**

Project Description: TG0257

	Client ID:	BH13-25-SS3	-	-	-
	Sample Date:	28-Jan-25 12:00	-	-	-
	Sample ID:	2506060-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	84.2	-	-	-
General Inorganics					
pH	0.05 pH Units	6.94	-	-	-
Resistivity	0.10 Ohm.m	63.4	-	-	-
Anions					
Chloride	5 ug/g dry	7	-	-	-
Sulphate	5 ug/g dry	43	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING TG0257-1 - TEST HOLE LOCATION PLAN

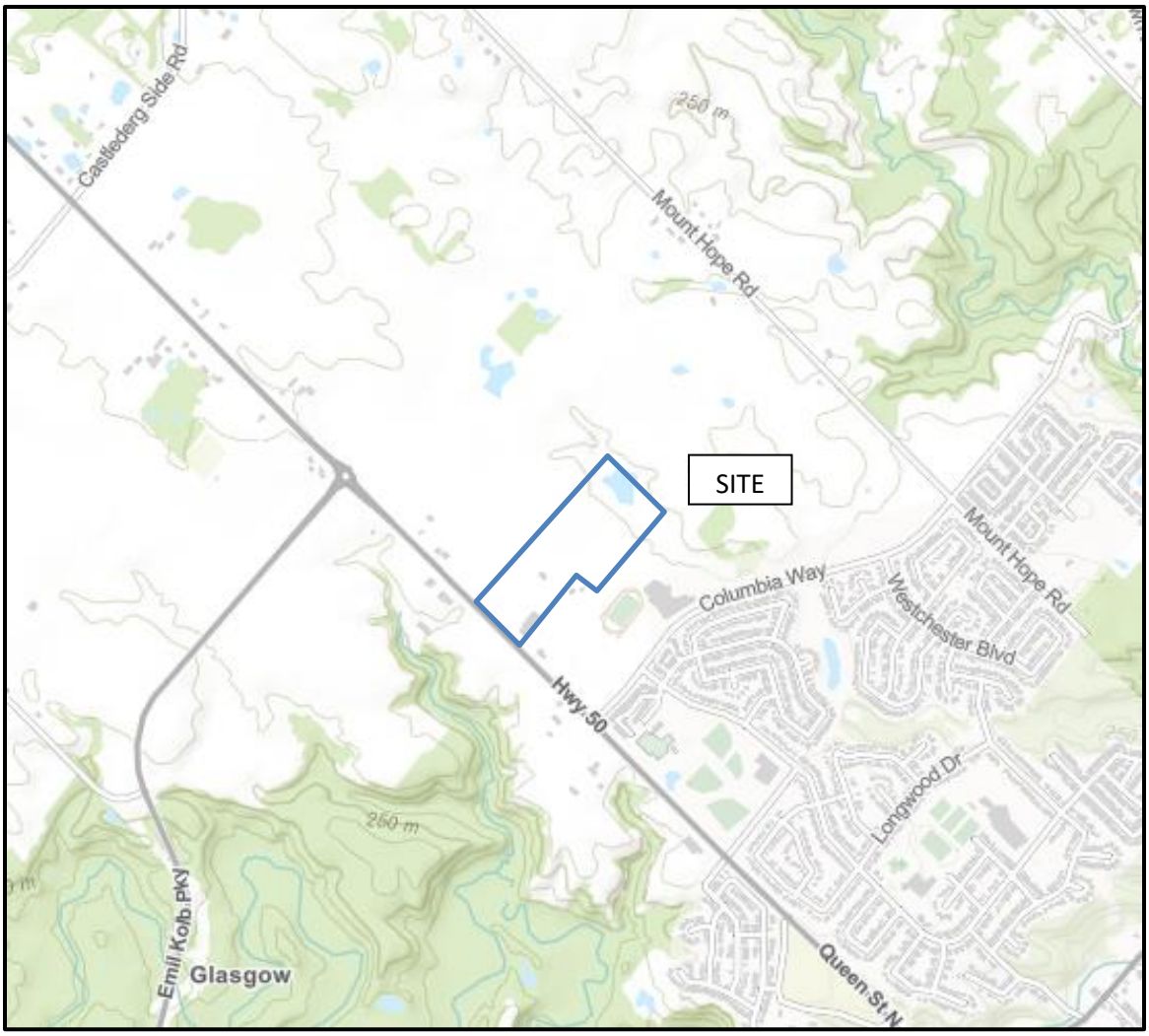
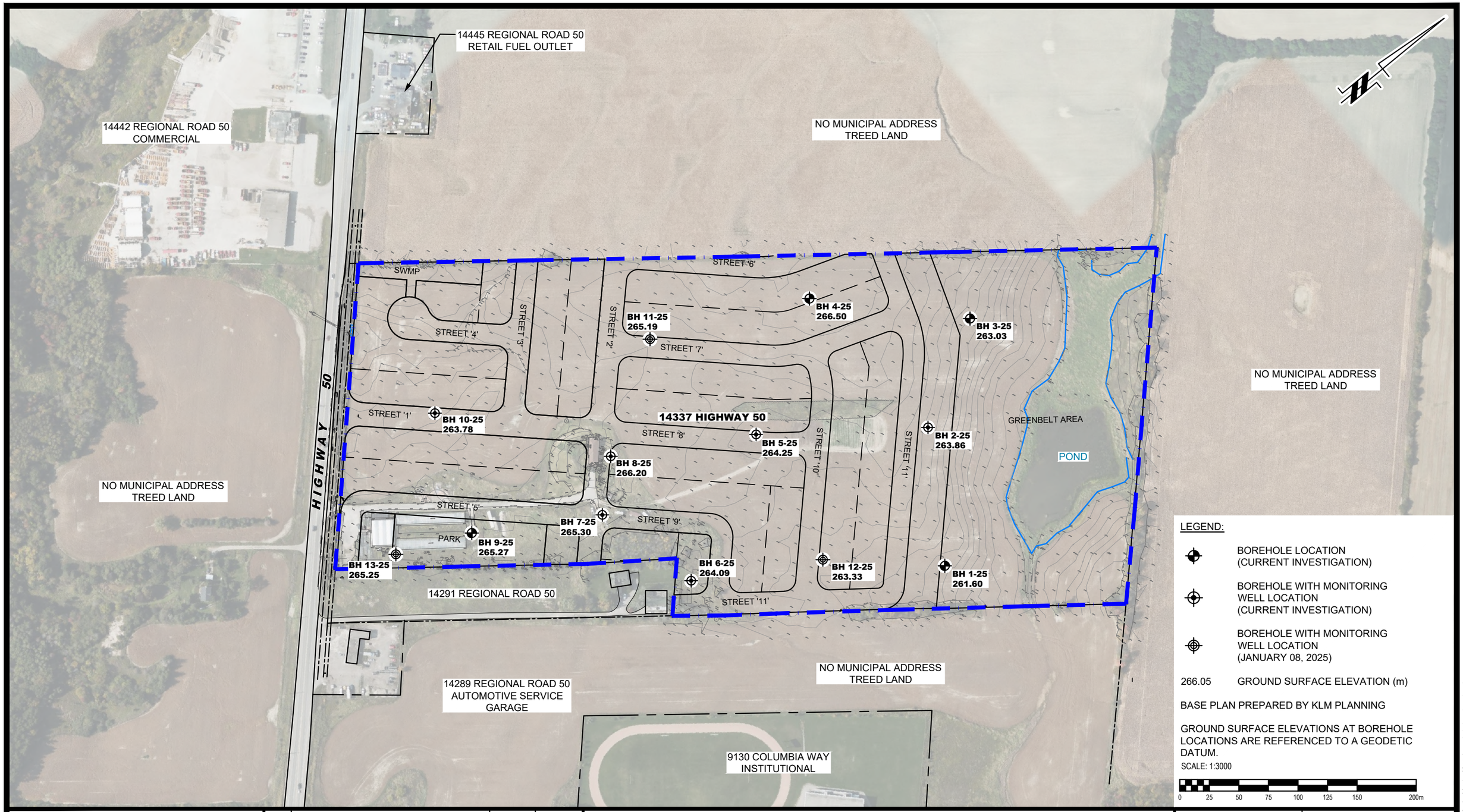


FIGURE 1

KEY PLAN



LEGEND:

- BOREHOLE LOCATION (CURRENT INVESTIGATION)
- BOREHOLE WITH MONITORING WELL LOCATION (CURRENT INVESTIGATION)
- BOREHOLE WITH MONITORING WELL LOCATION (JANUARY 08, 2025)

266.05 GROUND SURFACE ELEVATION (m)

BASE PLAN PREPARED BY KLM PLANNING

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:3000

0 25 50 75 100 125 150 200m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

TIFFANY ROX NORTH HILL LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
14337 HIGHWAY 50

BOLTON, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:3000	Date: 01/2026
Drawn by: YA	Report No.: TG0257-1
Checked by: BN	Dwg. No.: TG0257-1
Approved by: DG	Revision No.:

p:\autocad drawings\geotechnical\tg02xx\tg0257\tg0257-1-test hole location plan.dwg

11x17