

PROPOSED TRUCKING FACILITY

6404 MAYFIELD ROAD

Stormwater Management Report

Town of Caledon, ON

June 19, 2026

Project: 25034

Prepared By: Camila Canizares

Reviewed By: Adam McEwen



15955 Airport Road, Suite 304, Caledon East, Ontario, Canada, L7C 1H9

TEL: 905 584-1458 FAX: 905 584-1461 E-MAIL: urbanwater@grnland.com

WEBSITE: www.grnland.com

**PROPOSED TRUCKING FACILITY
6404 Mayfield Road, Caledon**

TABLE OF CONTENTS

1.0	INTRODUCTION.....	1
2.0	STORMWATER MANAGEMENT.....	2
2.1	Stormwater Management Criteria.....	2
2.2	Site Statistics.....	3
2.3	Peak Flow Control.....	4
2.3.1	Uncontrolled Drainage	5
2.3.2	Peak Flow Control Orifice Sizing.....	5
2.3.3	Emergency Overflow Weir Sizing.....	6
2.3.4	SWM Detention Volumes	7
2.4	Water Quality Control	8
2.4.1	TSS Removal	8
2.4.2	Thermal Mitigation Measures.....	9
2.4.3	Maintenance Considerations.....	9
2.4.4	Construction Phase Erosion and Sediment Control.....	10
2.5	Water Balance Analysis.....	10
2.6	Design of SWM Features	11
2.6.1	Pre - Treatment	11
2.6.2	Native Soils Conditions and Water Table Elevations.....	11
2.6.3	Stormwater Management Pond Design	12
2.6.4	Infiltration Basin Design	13
2.6.5	Bioswale Design.....	13
3.0	CLOSURE.....	14

TECHNICAL APPENDICES

Appendix A:	Modified Rational Method Calculations
Appendix B:	Hydrogeological Report Excerpts

Stormwater Management Report
PROPOSED TRUCKING FACILITY
6404 Mayfield Road, Caledon

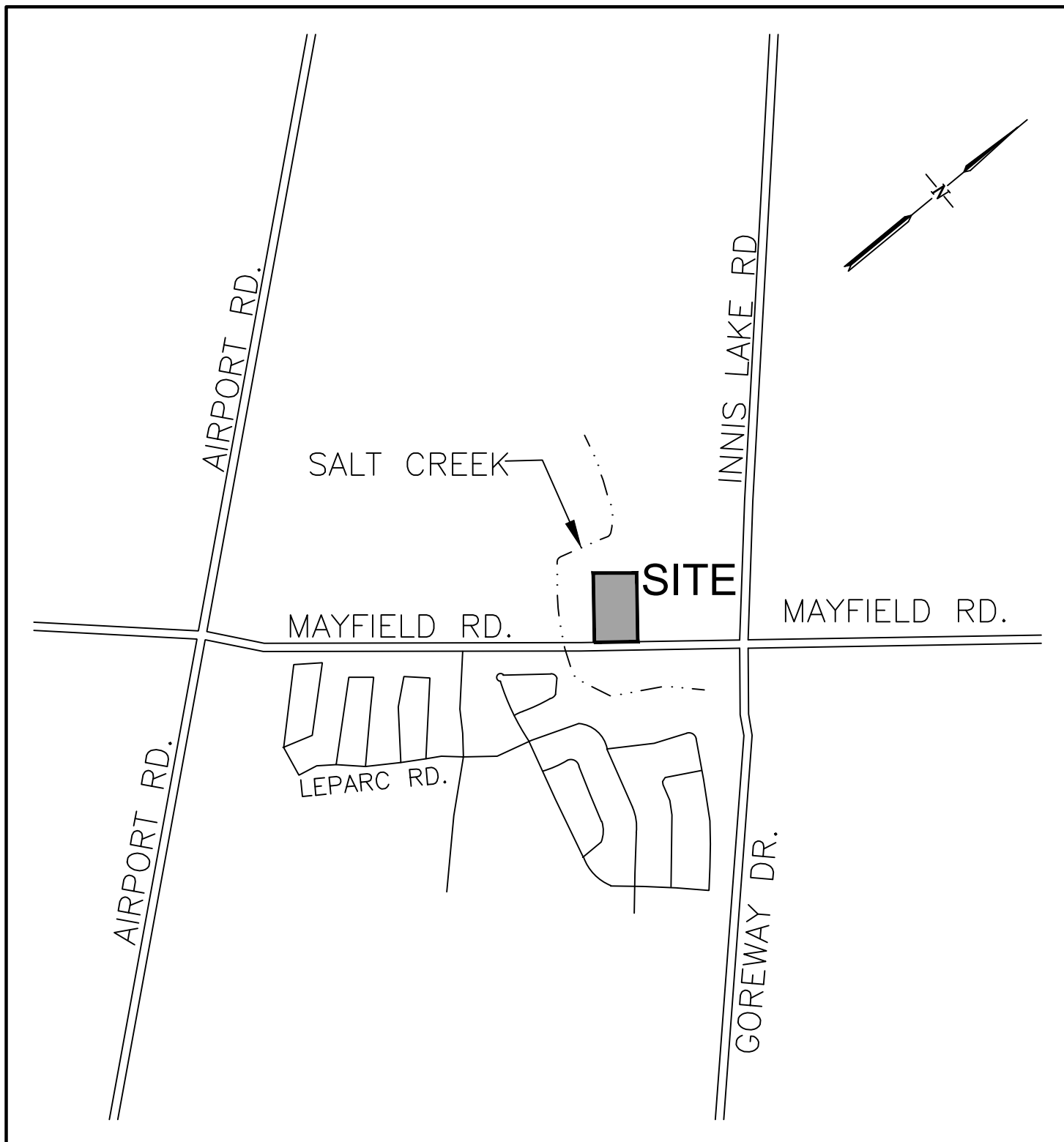
1.0 INTRODUCTION

The following Stormwater Management Report has been prepared by Urban Watershed Group Limited. (UWGL), in support of the temporary land use Zoning By-Law amendment for the proposed trucking facility located on 6404 Mayfield Road in the Town of Caledon. The 0.92-hectare property is legally described as Lot 1, Concession 1 (Geographic Township of Albion) – Town of Caledon – Regional Municipality of Peel and is located about 350 meters south-west of the intersection of Mayfield Road and Innis Lake Road (See **Figure No. 1**).

The site is located in a predominantly agricultural area, currently occupied by an existing single-storey residential building and an ancillary garage building that will both remain in the proposed development. The site is bounded by a watercourse on the west, residential/agricultural to the north and an existing residential building to the east side. It is proposed to construct a gravel surface parking lot of 4,150.7 sq.m. Site access will be provided onto Mayfield Road through a proposed driveway. A total of 36 truck and trailer parking spaces and 5 car parking spaces are proposed on the site.

The primary purpose this report is to develop a stormwater management plan for the site that accounts for the increase in imperviousness due to proposed parking lot and restricts the site's release rate to satisfy the criteria set out by the governing agencies (Town of Caledon and Toronto and Region Conservation Authority (T.R.C.A.)), which include the following:

- Propose SWM controls and stormwater detention volumes required to maintain the corresponding pre-development release rate for stormwater from the property during the 2-year and 100-year events;
- Provide an analysis of stormwater quality from the property and provide a minimum TSS removal rate of 80% corresponding to an "Enhanced" level of protection under current TRCA standards; and,
- Allow for retention of a 5mm rainfall event from the site to mitigate increase in runoff volume.
- Protection of sensitive habitat within the Salt Creek Watercourse corridor.



Drawing Title

SITE LOCATION PLAN

PROPOSED TRUCKING FACILITY
6404 MAYFIELD ROAD
CALEDON, ONTARIO



15955 AIRPORT ROAD, SUITE 304
CALEDON EAST, ONTARIO, L7C 1H9
PHONE: (905) 584-1458 FAX: (905) 584-1461

Urban Watershed Group Ltd.

A Member of the **GREENLAND** Group of Companies
urbanwater@grnland.com

Client

Drawn By

B. KLESS

Checked By

P. ELLIS

Drawing No.

FIG1

Scale

N.T.S.

Project No.

25034

2.0 STORMWATER MANAGEMENT

2.1 Stormwater Management Criteria

The property is located within a regulated area of the Toronto and Region Conservation Authority (TRCA) Watershed. A portion of the property is within the Regulatory floodplain of the Salt Creek (of Humber River Watershed). The existing runoff from the property is primarily conveyed by sheet flow directly onto the neighbouring lands to the west, and directly into the Salt Creek. A small portion of the front yard currently drains overland towards the municipal ditch within the Mayfield Road right-of-way.

The policies set out in the TRCA Stormwater Management Criteria (August 2012, Version 1.0) outline the below applicable SWM criteria for the development which are to be followed:

- **Water Quantity**
 - o Peak Flow Control (post to pre for up to and including 100-year storms)
- **Water Quality**
 - o Suspended Solids Removal
 - o Erosion and Sediment Control
- **Water Balance / Groundwater Recharge**
- **Stormwater Outfalls**

Salt Creek is also considered a potential habitat for Redside Dace, a Species at Risk and considered endangered by the Federal Department of Fisheries and Oceans (DFO). Provincial guidance under the Endangered Species Act sets additional water quality targets for ensuring the habitat is not negatively affected:

- Discharge of water from urban development SWMF into Redside Dace habitat should not exceed 25 mg/L of TSS above the background stream level of total suspended solids in baseflow conditions.
- Discharge temperatures for stormwater management facilities connected to Redside Dace streams should be below 24°C and have dissolved oxygen concentrations of at least seven milligrams per litre.

As the proposed development is associated with a temporary land-use, implementation of underground site services is to be kept to a minimum in order to reduce expense related to installation and removal of infrastructure and disposal. As such, stormwater management shall primarily be provided via a SWM Pond.

The criteria above will be defined in detail and addressed within the succeeding sections of the report.

2.2 Site Statistics

The total lot area is 9,216.5 sq.m and is occupied by two existing buildings (the primary residence and an accessory detached garage), a driveway and patio space.

The land partially slopes in a south westerly direction, across a change in grade of approximately 0.6 meters with an average slope of 1.0%. Storm runoff from the property shall be controlled to the existing peak flow rates for the 2-year and 100-year storm events, considering a pre-development runoff coefficient of 0.36. The front portion of the site drains to the municipal ditch on the Mayfield Road right-of-way uncontrolled. The subject site's existing and proposed conditions have been summarized in the following tables:

Existing Condition (Figure No. 2)

Land Use	Area	C	AC
Building Area	358.8	0.90	322.9
Impervious Area	1,184.1	0.90	1,065.7
Pervious Area	7,673.6	0.25	1,918.4
Total Development Area	9,216.5	0.36	3,307.0

Proposed Condition (Figure No. 3)

Land Use	Area	C	AC
Controlled Building Area	184.2	0.90	165.8
Controlled Impervious Area	4,000.0	0.90	3,600.0
Controlled Pervious Area	754.7	0.25	188.7
Controlled SWM Pond Area	1,741.0	0.50	870.5
Total Sub-Catchment Area	6,679.9	0.72	4,825.0
Existing Uncontrolled Building Area	174.6	0.90	157.1
Existing Uncontrolled Impervious Area	517.9	0.90	466.1
Existing Uncontrolled Pervious Area	1,128.0	0.25	282.0
Total Sub-Catchment Area	1,820.5	0.50	905.2
Proposed Uncontrolled Impervious Area	150.2	0.90	135.2
Proposed Uncontrolled Pervious Area	565.9	0.25	141.5
Total Sub-Catchment Area	716.1	0.39	276.7
Total Development Area	9,216.5	0.65	6,006.9

$$\% \text{ Impervious of Development Site} = (358.8 + 4,000.0 + 517.9 + 150.2) / 9,216.5 = 54.5\%$$

i:\URBAN WATERSHED GROUP\25034 - 6404 MAYFIELD ROAD - CALEDON\ENGINEERING\25034 - 6404 MAYFIELD RD - 2025-09-23\25034 - 6404 MAYFIELD RD - 2025-09-23.dwg
Sep 23, 2025 - 9:12am

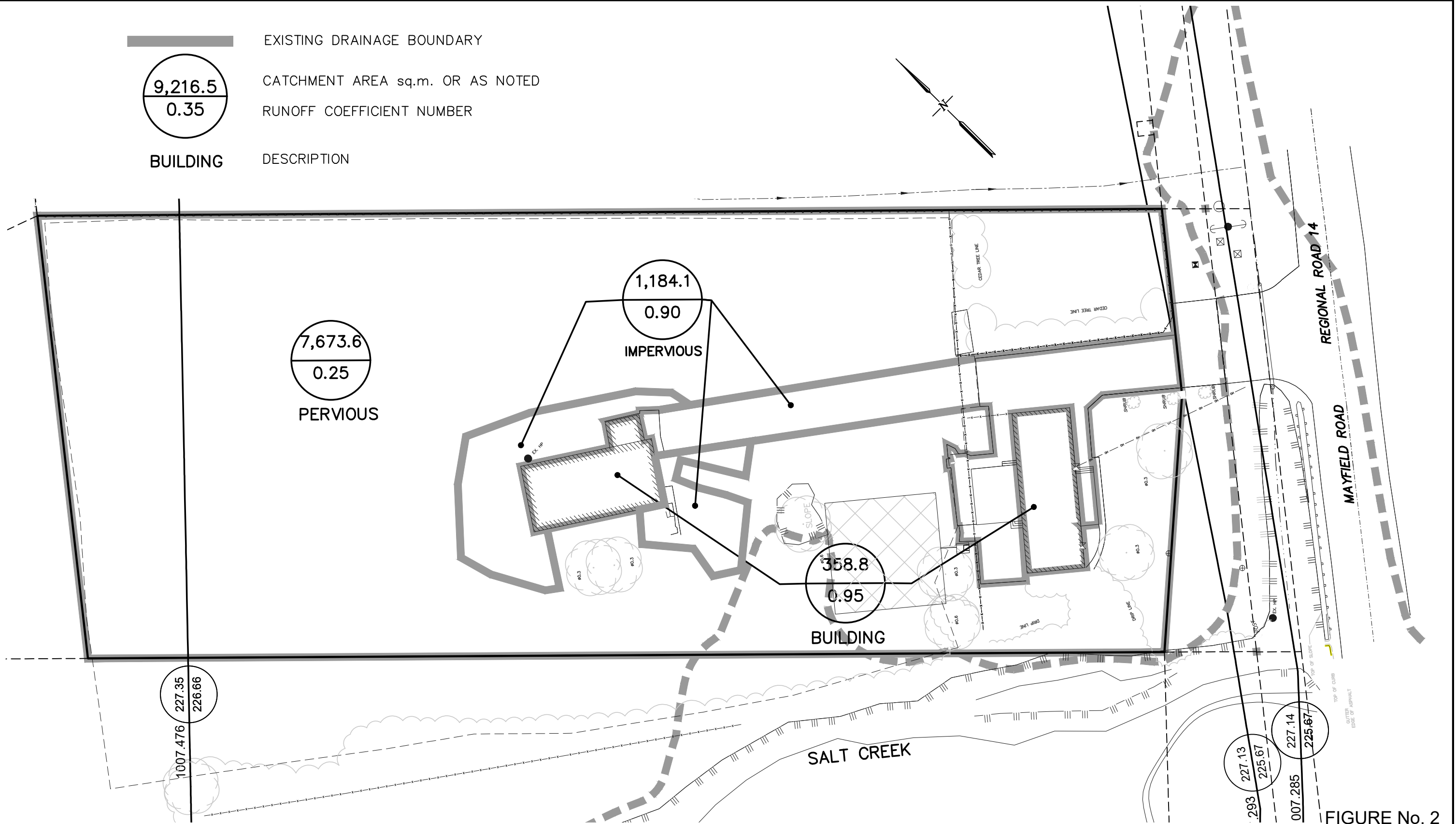
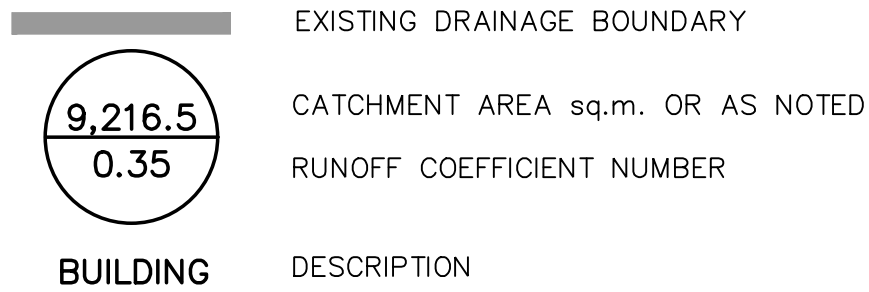


FIGURE No. 2

PROPOSED TRUCKING FACILITY
6404 MAYFIELD ROAD
CALEDON, ONTARIO



15955 AIRPORT ROAD, SUITE 304
CALEDON EAST, ONTARIO, L7C 1H9
PHONE: (905) 584-1458 FAX: (905) 584-1461

Urban Watershed Group Ltd.

A Member of the **GREENLAND** Group of Companies
urbanwater@grnland.com

**PRE DEVELOPMENT
DRAINAGE AREA PLAN**

SCALE 1:500

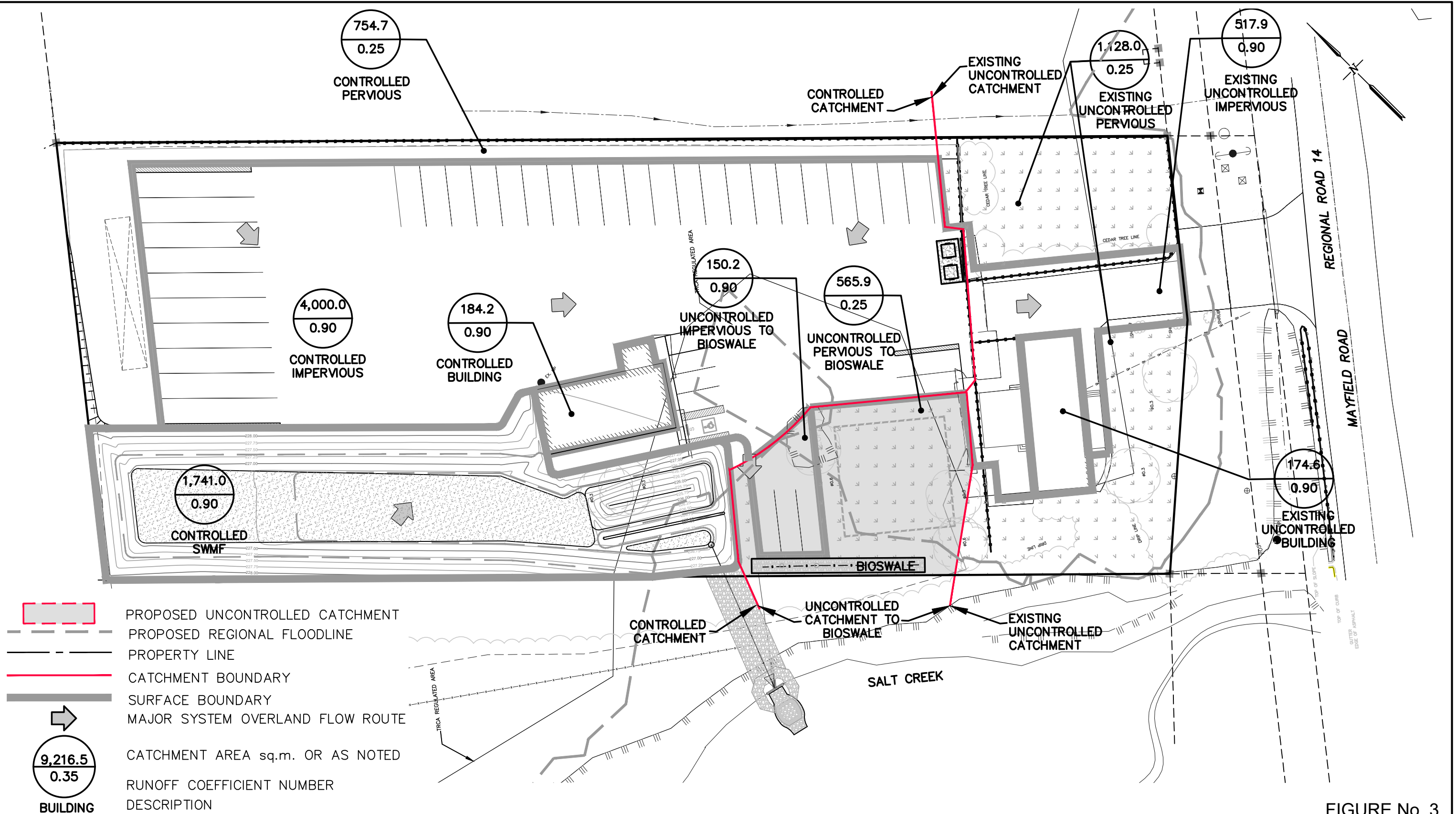


FIGURE No. 3

PROPOSED TRUCKING FACILITY
6404 MAYFIELD ROAD
CALEDON, ONTARIO



15955 AIRPORT ROAD, SUITE 304
CALEDON EAST, ONTARIO, L7C 1H9
PHONE: (905) 584-1458 FAX: (905) 584-1461

Urban Watershed Group Ltd.

A Member of the **GREENLAND** Group of Companies
urbanwater@grnland.com

**POST DEVELOPMENT
DRAINAGE AREA PLAN**

SCALE 1:1000

2.3 Peak Flow Control

TRCA and Town criteria require runoff controls be implemented to maintain the peak flow rate generated from the property under existing conditions.

As per the TRCA's Stormwater Management Criteria, the subject site falls within the Sub-Basin 36 of the Humber River watershed, the unitary flow rates for the 2-year and 100-year events have also been calculated using the provided formulas and are summarized below:

Return Period	Unit Flow Rate Equation	Unit Flow Rate
100-Year	$Q = (29.912 - 2.316 \times \ln(A)) \times A$	27.7 L/s
2-Year	$Q = (9.506 - 0.719 \times \ln(A)) \times A$	8.8 L/s

The overall site area and pre-development runoff coefficient (9,216.5 sq.m, $C_{2YR} = 0.36$) has been used to calculate the existing release rates based on Caledon's rainfall IDF Curve given in the Development Standards Manual - Version 5.0 (2019), the intensity for the 2-year and 100-year storms (with an initial time of concentration of 10 minutes) are calculated as follows:

Rainfall Intensities

$$\begin{aligned} I_{2 \text{ year}} &= 1070 / (10 + 7.85)^{(0.8759)} & I_{100 \text{ year}} &= 4688 / (10 + 17)^{(0.9624)} \\ &= 85.72 \text{ mm/hr} & &= 196.54 \text{ mm/hr} \end{aligned}$$

The 2-year existing release rate from the site can then be calculated:

Existing Release Rates

$$\begin{aligned} Q_{\text{EXISTING, 2-YR}} &= ACI / 3600 \\ &= 9,216.5 \times 0.36 \times 85.72 / 3600 \\ &= 79.00 \text{ L/sec} \end{aligned}$$

As with the unit flow rates, the 2-year to 100-year existing release rates, are also summarized below:

Return Period	Rainfall Intensity	Existing Peak Flow Rate
100-Year	195.5 mm/hr	181.1 L/s
2-Year	85.7 mm/hr	79.0 L/s

Due to limitations associated with the minimum recommended size to implement effective stormwater controls, it is not feasible to achieve the unit flow rates calculated for this site per the TRCA's Stormwater Management Criteria. The development area will instead be controlled to no more than the 2-year existing peak release rate of 79.0 L/s under a 100-year storm event. Runoff from the site is to be conveyed by overland flow into a dry pond cell of the proposed Stormwater Management (SWM) Pond facility for peak flow control.

2.3.1 Uncontrolled Drainage

As shown on the Post-Development Drainage Plan, a 2,536.6 sq.m (C= 0.47) portion of the site area will drain uncontrolled in order to minimize disturbance outside of the development limits and to prevent displacement of floodplain storage.

A 716.1 sq.m area of the uncontrolled catchment drains directly to the watercourse, and is required to be treated by a proposed bioswales system to provide water quality control.

The other 1,820.5 sq.m of the uncontrolled site area is located at the front and side yard of the building and will remain as uncontrolled drainage to the Mayfield Road right-of-way following the existing site conditions. As no development is proposed within this uncontrolled area of the site, the peak flow rate to the municipal road allowance will not be altered.

The remaining 6,679.9 sq. m of the development site area will be controlled by the SWM Pond. To account for the uncontrolled storm runoff from the property, the total site allowable release rate from the SWM Pond will be adjusted to accommodate the peak flow rates produced from this area.

Post-Development Uncontrolled Storm Drainage

$$\begin{aligned} Q_{100\text{-YR UNCONTROLLED}} &= ACI / 360 \\ &= 2,536.5 \times 0.47 \times 196.54 / 3600 \\ &= 65.1 \text{ L/s} \end{aligned}$$

Adjusted Allowable Release Rate

$$\begin{aligned} Q_{100\text{YR POST}} &= Q_{2\text{-YR EXISTING FLOW RATE}} - Q_{100\text{-YR UNCONTROLLED}} \\ &= 79.0 \text{ L/s} - 65.1 \text{ L/s} \\ &= 13.9 \text{ L/s} \end{aligned}$$

Therefore, the release rate from the proposed SWM Pond must be controlled to no greater than 13.9 L/s for the 100-year storm in order to achieve the desired level of peak flow control.

2.3.2 Peak Flow Control Orifice Sizing

The proposed allowable release rate from the SWM Pond of 13.9 L/s is below the level of control achievable by a single pipe acting as an orifice tube, and will require a proposed standpipe and orifice plate configuration. The proposed orifice is to be set at the base of the standpipe with an elevation of 226.70 in order to maintain the water level within the water quality control cell.

Per the Floodplain Assessment (February, 2026) completed by EXP. the highest water level within the watercourse which would act as tailwater on the orifice during a 100-year storm is 226.66masl, and is not anticipated to impede flow through the orifice control.

Water Quantity control will be provided through a dry pond cell located above the proposed orifice device. The highwater level for the dry pond will be limited to 227.37m, in order to maintain a minimum 150mm freeboard from the finished floor of the unoccupied ancillary building (FFE = 227.52m).

The maximum allowable diameter for a single orifice to achieve these parameters has been calculated below:

A 55mm diameter orifice plate has been proposed

$$\begin{aligned} 100\text{-yr Ponding Elevation (m)} &= 227.37 \text{ masl} \\ \text{Orifice Springline Elevation (m)} &= 226.70 \text{ masl} \\ \Delta h \text{ (head)} &= 0.67 \text{ m} \end{aligned}$$

$$\begin{aligned} \text{Orifice Area} &= D^2 \times \pi / 4 \\ &= 0.055^2 \times \pi / 4 \\ &= 0.00237 \text{ sq.m} \end{aligned}$$

$$\begin{aligned} \text{Release rate (Q)} &= A_{\text{orifice}} \times (c \sqrt{2g\Delta h}) \\ &= 0.00237 \text{ sq.m} \times (0.64 \sqrt{2 \times 9.81 \times 0.67}) \\ &= 5.5 \text{ L/s} \end{aligned}$$

Therefore, the proposed stand pipe fitted with a 55mm diameter orifice plate will release a 5.5 L/s flow, which is below the 13.9 L/s allowable release rate. The orifice shall be protected by a stand pipe and rip-rap collar to reduce risk of blockages due to the minimal size of opening.

2.3.3 Emergency Overflow Weir Sizing

An emergency overflow weir shall be provided with a base elevation equal to the maximum 100-year highwater level of 227.37m to control the release of flow during any event larger than the 100-year storm. The regional water level within the downstream water course is noted by EXP. to be 227.35, and will not impact the proposed emergency overflow weir.

Considering an uncontrolled 100-year inflow to the SWM Pond of 0.28 cms, an overflow weir with a length of 5.0m will provide the required conveyance at a 0.08m depth and velocity of 0.73 m/s.

The proposed SWM pond shall be built up to an elevation of 227.47m to contain the required detention storage volume, and will be able to contain the maximum flow depth over the weir at elevation 227.45m (227.37m + 0.08m) below the berm to prevent overtopping.

The overflow weir will be lined with 150mm diameter rip-rap, capable of withstanding velocities up to 2.5m/s.

2.3.4 SWM Detention Volumes

The Modified Rational Method approach was used to establish the 100-year post-development flows and associated stormwater detention volumes required to satisfy the peak flow rate control criteria developed above. The maximum anticipated storage requirements were developed from the figures in **Appendix A** and are based upon the maximum allowable controlled release rate for the site of 13.9 L/s. **Figures No. 4 & 5** in **Appendix A** present the required storage considering the Town of Caledon and TRCA criteria.

$$\text{Total 100-year Post Detention Volume Required} = 343.1 \text{ cu.m}$$

The proposed dry pond cell will generate the required storage within the SWM Pond between the elevations 226.70 and 227.37 (0.77m maximum depth). The proposed Dry Pond Cell volume has been calculated utilizing a comparison surface in Civil 3D as 695 cu.m. Based on a stage storage discharge curve for the pond below a summary of the proposed and allowable release rates from the SWM Pond during each design storm has been determined:

Elevation (masl)	Volume (cu.m)	Release Rate (L/s)
227.37	695	5.5
227.35	671	5.4
227.25	567	5.0
227.15	464	4.5
227.12	430	4.4
227.10	412	4.3
227.05	360	4.0
226.95	256	3.4
226.85	153	2.6
226.75	49	1.5
226.7	0	0

Return Period	Unit Flow Rate	Existing Peak Flow Rate	Uncontrolled Release Rate	SWM Pond Release Rate	Proposed Release Rate
100-Year	27.7 L/s	181.1 L/s	65.1 L/s	4.4 L/s	69.5 L/s
2-Year	8.8 L/s	79.0 L/s	28.4 L/s	2.6 L/s	31.0 L/s

2.4 Water Quality Control

Runoff from the development area is received directly by the adjacent watercourse, which is considered a sensitive habitat for Redside Dace. Runoff is discharged from the site in two forms, controlled flow from the proposed SWM Pond outfall, and uncontrolled via overland flow. Both sources of runoff must be treated to a maximum 25 mg/L TSS concentration and temperature below 24 degrees Celsius under provincial guidance for protection of endangered species. A water quality design storm for 10mm of rainfall will be considered. Larger and subsequent events will experience low initial TSS loading and thermal impacts as the initial storm will lift loose sediments and cool the impervious surfaces.

Additionally, a net 80% TSS removal efficiency is required for the development as a whole under TRCA criteria.

The existing front yard, building and driveway area (1,820.5 sq.m) are not proposed to be altered by the proposed development, and are not considered to require specific water quality treatment. The remaining 7,396.0 sq.m area (9,216.5 site area – 1,820.5 sq.m existing area) will be subject to a treatment train as detailed herein. An infiltration basin will be installed at the SWM Pond outlet to provided the stormwater retention necessary to meet the controlled catchment area's stormwater quality requirements. A bioswale will provide supplemental water quality treatment to the uncontrolled impervious surface associated with the proposed development.

2.4.1 TSS Removal

A TSS mass balance has been prepared below in order to demonstrate the anticipated discharge concentration along with the overall treatment train efficiency.

Catchment Type	Catchment Area (sq.m)	% Impv.	Water Quality Design Volume (10mm event)	Initial TSS Concentration* (mg/L)	Initial TSS Loading (kg)	TSS Removal Efficiency	Discharge TSS Loading (kg)	Discharge TSS Concentration (mg/L)
Controlled Surfaces	6,679.9	63%	43,897 L	77.7	3.41	80% (Infil. Basin)	0.68	15.5
Uncontrolled Surfaces	716.1	21%	2,767 L	49.4	0.14	80% (Bioswale)	0.03	9.9
Total Development Site	7,396.0	65%	46,664 L	76.1	3.55	80.0%	0.89	15.2

*Composite TSS concentration calculated as weighted average by surface area considering;

Pervious/ vegetated surface TSS Concentration: 25 mg/L

Impervious / paved surface TSS Concentration: 90 mg/L

Building roof TSS Concentration: 10 mg/L

The infiltration basin has been sized in accordance with the Stormwater Management Planning and Design Manual (MECP, 2003). Per Table 3.2 – Water Quality Storage Requirements based on Receiving Waters an infiltration storage volume of 23.5 cu.m (35 cu.m/ha x 0.67 ha) will be adequate to achieve an 80% TSS removal efficiency for an 70% imperviousness level.

Similarly, the bioswale is proposed with a total retention volume of 1.8 cu.m (25 cu.m/ha x 0.07 ha) to achieve an 80% TSS removal efficiency for an 35% imperviousness level per MECP Table 3.2.

2.4.2 Thermal Mitigation Measures

The stormwater management systems on this site are designed to retain frequent rainfall within as part of the water quality control and water balance requirements. From both the controlled and uncontrolled catchments the runoff which will be infiltrated from the facilities experiences the most heat transfer as it the first to come into contact with warm surfaces. Since this water is infiltrated into the ground and not released to the watercourse the associated warming effect is not introduced into the adjacent watercourse.

From the controlled catchment areas, which represents the majority of runoff discharged from the site, a dry pond feature was selected to provide only temporary detention of runoff on the surface, avoiding the warming effect typically associated with the permanent pool of wet pond features.

Planting of shade vegetation around the uncontrolled catchment area perimeter will also be reviewed with the project landscape architect at further design stages to see whether additional surface cooling benefits can be suitably implemented.

At the outfall, turbulence normally occurs which results in energy dissipation thereby causing a further temperature drop while also aerating the flow to restore dissolved oxygen levels.

In the view of the foregoing, it can be concluded that the storm water management plan proposed for this site will amply mitigate thermal impacts on the receiving watercourse.

2.4.3 Maintenance Considerations

Sediment and debris will gradually accumulate within the SWM Pond. The buildup of sediment will degrade the provided TSS removal efficiency of the facility if not properly maintained, and could result in discharge of stormwater runoff from the site at qualities unacceptable to the municipality. Annual inspection of stormwater infrastructure for debris or sediment accumulation should be carried out in early spring or following significant rainfall events (>25mm).

Assessment of the pond via topographic survey during the dry season to determine whether maintenance is required is recommended after a period of 10-years. Maintenance can typically be expected to include draining of the pond and regrading to removal accumulated sediment.

Sediment and Debris will also gradually accumulate within the bioswale which could impede the performance of the system and should be maintained at the same time as the SWM Pond Maintenance.

Accumulated sediment and debris should be cleaned from the surface of the infiltration facilities annually.

2.4.4 Construction Phase Erosion and Sediment Control

A temporary single row and double row silt fence as per Town of Caledon standards is to be installed along all property lines where there is a risk of sediment runoff onto the adjacent property during construction. This fencing shall be installed prior to site grading and shall be maintained throughout the construction period. A mud mat per Town standards shall be constructed at the onset of the works at proposed site entrance to prevent sediment from being tracked onto the street, and street sweeping shall be carried out as necessary. Additionally, a Rock Check Dam must be installed at the Salt Creek discharge point to prevent erosion.

The proposed SWM pond can be utilized as a temporary settling basin to remove sediment from runoff generated during construction.

The complete erosion control plan has been illustrated on Dwg. **25034-ESC1**, and shall be inspected and maintained until all exposed soil is stabilized on the site.

2.5 **Water Balance Analysis**

TRCA's Stormwater Management Criteria establish criteria to provide erosion control for the downstream watercourses. A 5mm rainfall depth over the 9,216.5 sq.m site area is to be retained. The required retention volume has been calculated below as:

5mm Retention Volume	= 9,216.5 sq.m x 0.005m = 46.1 cu.m
Initial Abstraction	= 4,189.6 sq.m x 0.005m = 20.9 cu.m
Site Retention Volume	= 46.1 cu.m – 20.9 cu.m = 25.2 cu.m

The proposed water quality treatment cells will provide a combined retention capacity of 28.5 cu.m (26 cu.m Infil. Basin + 2.5 cu.m Bioswale) to allow for infiltration of the required runoff volumes.

2.6 Design of SWM Features

Several on-site SWM features are proposed on the site to achieve the water quantity, water quality, and water balance criteria discussed in the previous sections.

The proposed SWM Pond shall be constructed of a combination dry-pond and infiltration basin as the primary form of water quantity and quality control for the development. Water balance objectives will also be addressed within the facility.

A secondary bioswale will augment the site's water quality and water balance criteria within the uncontrolled catchment of the site. Due to the presence of the Salt Creek floodline, water quantity control within the bioswale is not feasible.

2.6.1 Pre - Treatment

While infiltration basins do not typically require a forebay, pretreatment is recommended to control accumulation of sediment and prevent loss of storage. Therefore, a sediment forebay and vegetated filter strip are proposed at the inlet to the SWM Pond to promote the settling of sediments, and reduce long-term maintenance requirements for the remainder of the SWM facility.

2.6.2 Native Soils Conditions and Water Table Elevations

The native soil underlying the site are understood to be predominately a clay loam material of imperfect drainage qualities (classified as the 'Peel Loam' formation in the Soil Map of Peel County, Report No 18 of the Ontario Soil Survey). Single Well Response testing has been carried out by Soil Engineers Ltd. as outlined in their Hydrogeological Assessment (dated January 21, 2026) provided in **Appendix B**. The observed hydraulic conductivity for the native soils is 1.2×10^{-5} m/sec at BH/MW 1 located near the proposed infiltration facility. Using the correlation of hydraulic conductivity to percolation time presented in Appendix C of the CVC Low Impact Development Stormwater Management Planning and Design Guide, an infiltration rate for the native soils can be calculated as follows:

$$\begin{aligned}\text{Hydraulic Conductivity} &= 6\text{e-}11\text{cm/s} \times (\text{Infiltration Rate})^{3.7363} \\ \text{Infiltration Rate} &= (\text{HC} / 6\text{e-}11)^{1/3.7363} \\ &= (1.2 \times 10\text{e-}3 / 6\text{e-}11)^{1/3.7363} \\ &= 90 \text{ mm/hr}\end{aligned}$$

$$\begin{aligned}\text{Design Infiltration Rate} &= \text{Measured Infiltration Rate} / \text{Safety Factor} \\ &= 90 \text{ mm/hr} / 3.5 \\ &= 25.7 \text{ mm/hr}\end{aligned}$$

Boreholes were drilled by Soil Engineers Ltd. as part of their Geotechnical Investigation (August 2025). Auger refusal was encountered in several locations during borehole drilling, and is interpreted to represent the depth of a bedrock outcrop which may consist of shale with limestone. Borehole 2 in the vicinity of the proposed SWM Pond experienced auger refusal at a depth of 2.4mbgs, or 225.1masl.

Groundwater observations were made in BH/MW 2 between July 28 and August 27, 2025 and indicate a high groundwater level of 225.8masl (on August 14, 2025).

2.6.3 Stormwater Management Pond Design

In order to minimize the volume of fill required to develop the subject property the proposed SWMP is proposed to be located at the boundary of the existing floodplain. A perimeter berm set above the 100-year flood line shall be constructed along the limits of SWM Pond to prevent the watercourse from flooding onto the facility.

The Ministry of the Environment, Conservation and Parks (MECP) provide four (4) requirements to be met for multi-purpose SWMPs located within the flood plain, and the below summarizes how each shall be met.

- 1) *The cumulative effects resulting from changes in floodplain storage and balancing cut and fill do not adversely impact existing or future development.*

Effects on the floodplain from the development have been reviewed and found to be negligible herein.

- 2) *Effects on corridor requirements and functional valleyland values must be assessed. SWMPs would not be allowed in the floodplain if detrimental impacts could occur to the valleyland values or corridor processes.*

An Environmental Impact Study has been carried out to verify the development will not have detrimental impacts to natural features.

- 3) *The SWMPs must not affect the fluvial processes in the floodplain; and*

SWMP and outlet has been located outside the 20 metre setback from the primary watercourse channel.

- 4) *The outlet invert elevation of the SWMP should be higher than the 2 year floodline and the overflow elevation must be above the 25 year floodline.*

As the SWMP is a multi-purpose facility the overflow has been set above the 100-year floodline, as discussed herein.

2.6.4 Infiltration Basin Design

The proposed infiltration basin has been designed with a retention volume of 26 cu.m (25.3 cu.m required per Section 2.4.1), to reduce the flow of sediment latent runoff from the site to achieve 80% TSS removal from captured stormwater runoff.

The allowable draw down depth and required footprint have been calculated to ensure the facility will drain within 24 hours.

P = infiltration rate (mm/hr), t = draw down time (hrs), V = Retention volume (m^3) n = void ratio

$$\begin{aligned}\text{Equation 4.2} \quad \text{depth} &= Pt \\ &= 25.7 \text{ mm/hr} \times 24 \text{ hr} \\ &= 617 \text{ mm}\end{aligned}$$

A void ratio of 0.10 has been selected to reflect the proposed sand filter layer.

$$\begin{aligned}\text{Equation 4.3} \quad \text{Area} &= 1000V / (Pnt), \\ &= 1000 \times 26 / (25.7 \times 0.1 \times 24) \\ &= 421 \text{ sq.m}\end{aligned}$$

The base of the sand filter shall be installed at an elevation of 226.15-226.05 in order to achieve an average 1.0m vertical separation from the observed bedrock layer and 0.30m vertical separation from the observed groundwater level. While a total media depth of 617mm is permissible based on the native soils, a reduced thickness of 450mm has been selected to improve clearance from the groundwater table, and the surface footprint has consequentially been increased to 600 sq.m to maintain the required retention volume.

2.6.5 Bioswale Design

A bioswale system has been incorporated into the design of the site with a retention volume of 2.5 cu.m (1.8 cu.m required per Section 2.4.1), to manage part of the uncontrolled drainage of the site.

Detailed infiltration trench design calculations, following Section 4.5.8 of the MOE Stormwater Planning and Design Manual (2003), have been completed below:

$$\begin{aligned}\text{Equation 4.2} \quad \text{depth} &= Pt \\ &= 25.7 \text{ mm/hr} \times 8 \text{ hr} \\ &= 206 \text{ mm}\end{aligned}$$

A void ratio of 0.40 has been selected to reflect the proposed clear stone storage layer.

$$\begin{aligned}\text{Equation 4.3} \quad \text{Area} &= 1000V / (Pnt), \\ &= 1000 \times 2.5 / (25 \times 0.40 \times 8) \\ &= 31 \text{ sq.m}\end{aligned}$$

The proposed bioswale system has an area of 48 sq.m (2 m x 24 m) to provide the necessary storage volume and allow the retained rainwater to infiltrate within 8 hours. A shorter 8 hour draw down time has been selected to reflect the function of the facility to provide water quality treatment primarily for smaller more frequent rainfall events. The base of the bioswale clear stone reservoir is proposed at 226.27, and has similar constraints to the infiltration basin with respect to depth.

3.0 CLOSURE

The proposed Stormwater Management Plan demonstrates that the development will meet the established criteria with respect to stormwater management set forth in governing documents. It is recommended the following site works be completed to accompany the required site servicing necessary as part of development for the site:

- Installation of SWM Pond to provide necessary SWM controls, treatment and retention through a multi-purpose facility, including:
 - o A 695 cu.m Dry Pond Cell for Peak Flow Control
 - o A 600 sq.m x 450mm Deep Infiltration Basin for Water Quality Control
 - o Sediment Forebay for Pre-treatment
 - o Outlet Control Structure with a 55mm diameter Orifice; and,
 - o Emergency Overflow Weir
- Construction of a bioswale with clear stone reservoir bed system to provide an additional detention storage volume of 2.5 cu.m for water quality benefits.
- Energy dissipation pool for aeration at the site outlet to enhance protection of sensitive watercourse habitats.
- Maintain the existing flood plain across the site as per the grading plan 25034-GR1.

In conclusion, the proposed stormwater management and grading plan supports the concept of an environmentally sustainable development. The proposed plan will mitigate anticipated stormwater impacts associated with the construction of the proposed development and aims to mimic the pre-development hydrological characteristics of the site.

Please do not hesitate to contact us with any questions pertaining to the enclosed.

Yours truly,

Urban Watershed Group Ltd.

Adam McEwen, P.Eng.

(905) 584-1458 ext. 233

Appendix A

Modified Rational Method Calculations Figure No. 4 & 5

STORMWATER MANAGEMENT ANALYSIS - FIGURE No. 4

JOB NUMBER: 25034
CLIENT: G.N. Transport
LOCATION: 6404 Mayfield Road
IDF STATION: Town of Caledon

SITE INFORMATION

Number of Subcatchment Areas: 1

This Analysis is for Subcatchment Area No. 1

	AREA (m2)	RUNOFF COEFFICIENT
Building Area	184.2	0.90
Impervious Area	4,000.0	0.90
Pervious Area	754.7	0.25
SWM Pond Area	1,741.0	0.50
Controlled Ground Surface Area	6,679.9	0.72

Allowable Release from Surface Areas (L/s): 13.9
Time of Concentration (tc) = 10.0 min

DETERMINATION OF NECESSARY STORAGE VOLUME - FIGURE No. 5**This Analysis is for Subcatchment Area No. 1**

Modified Rational Method used to determine storage volume:

$$Q = CiA/360, \quad i = A / (tc/60)^B \text{ for: } A = 4688, \quad B = 0.9624, \quad C = 17$$

TIME	i	INFLOW (m3/sec)	ACCUMULATED RUNOFF VOLUME (m3)	ALLOWABLE RELEASE VOL. (m3)	REQ'D STORAGE (m3)
(t)		($Q = CiA/360$)	($V_{acc} = Q \times tc$)	($V_{rel} = Q100yr \times (t + tc)/2$)	($V_{req'd} = V_{acc} - V_{rel}$)
(MIN.)	(mm/hr)	SURFACE	SURFACE	SURFACE	SURFACE
1	290.3	0.389	23.3	4.6	18.8
2	275.6	0.369	44.3	5.0	39.3
3	262.3	0.352	63.3	5.4	57.9
4	250.3	0.335	80.5	5.8	74.7
5	239.4	0.321	96.2	6.3	90.0
6	229.3	0.307	110.6	6.7	104.0
7	220.1	0.295	123.9	7.1	116.8
8	211.6	0.284	136.2	7.5	128.7
9	203.8	0.273	147.5	7.9	139.6
10	196.5	0.263	158.0	8.3	149.7
15	166.9	0.224	201.3	10.4	190.9
20	145.1	0.195	233.4	12.5	220.9
25	128.5	0.172	258.3	14.6	243.7
30	115.3	0.155	278.1	16.7	261.4
35	104.6	0.140	294.4	18.8	275.6
40	95.7	0.128	308.0	20.9	287.1
50	82.0	0.110	329.5	25.0	304.5
60	71.7	0.096	345.9	29.2	316.7
90	52.2	0.070	378.0	41.7	336.3
120	41.2	0.055	397.3	54.2	343.1
240	22.5	0.030	433.7	104.3	329.5
360	15.5	0.021	449.9	154.3	295.7
720	8.2	0.011	472.1	304.4	167.7
1440	4.2	0.006	490.0	604.7	0.0

Appendix B

Hydrogeological Report Excerpts



Soil Engineers Ltd.

CONSULTING ENGINEERS

GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE

90 WEST BEAVER CREEK ROAD, SUITE 100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BARRIE
TEL: (705) 721-7863
FAX: (705) 721-7864

MISSISSAUGA
TEL: (905) 542-7605
FAX: (905) 542-2769

OSHAWA
TEL: (905) 440-2040
FAX: (905) 725-1315

NEWMARKET
TEL: (905) 853-0647
FAX: (905) 881-8335

MUSKOKA
TEL: (705) 684-4242
FAX: (705) 684-8522

HAMILTON
TEL: (905) 777-7956
FAX: (905) 542-2769

**A REPORT TO
G.N. TRANSPORT INC.**

**HYDROGEOLOGICAL ASSESSMENT
PROPOSED TRANSPORTATION TERMINAL**

**6404 MAYFIELD ROAD
TOWN OF CALEDON**

REFERENCE NO. 2506-W157

January 21, 2026

DISTRIBUTION

Digital Copy - G.N. Transport Inc.

Digital Copy - Soil Engineers Ltd. (Richmond Hill)



4.0 METHODOLOGY

4.1 Borehole Advancement and Monitoring Well Installation

Drilling of boreholes and construction of monitoring wells were conducted for geotechnical and hydrogeological investigations by SEL on July 08, 2025. The program consisted of the drilling of four (4) boreholes (BH) and installation of three (3) monitoring wells within selected geotechnical borehole. The locations of the boreholes and monitoring wells are shown on **Drawing No. 2**.

Extension of boreholes and installation of monitoring wells approximately 4.6 and 6.1 metres below ground surface (mbgs) were proposed for the current assessment. However, the boreholes could not be extended due to auger refusal and shallow bedrock that was contacted in the proposed monitoring well' locations. Borehole drilling and monitoring well construction were completed by a licensed water well contractor, under the full-time supervision of a geotechnical field technician from SEL, who logged the soil strata encountered during borehole advancement and collected representative soil samples for textural classification. The boreholes were drilled using a track mounted machine equipped with continuous flight, hollow stem augers. Detailed descriptions of the encountered subsoil and groundwater conditions are provided by SEL and presented on the borehole and monitoring well logs, on the enclosed **Appendix A**.

The monitoring wells were constructed using 50-mm diameter Trilock pipes and 0.9 m or 1.5 m long 10-slot well screens, which were installed in each of the selected geotechnical boreholes. All the monitoring wells were equipped with flush-mount casings.

The UTM coordinates and ground surface elevations at the monitoring wells' locations, as well as the monitoring well construction details, are presented in **Table 4-1**. The ground surface elevations and horizontal coordinates at the monitoring well locations were determined at the time of the investigation, using a Trimble TSC3 handheld Global Navigation Satellite System.

Table 4-1- Monitoring Well Installation Details

Well ID	Installation Date	UTM Coordinates East (m)	UTM Coordinates North (m)	Ground Elevation (masl)	Soil Medial in the Screen Interval (mbgs)	Screen Interval (mbgs)	Casing Dia. (mm)
BH/MW 1	July 08, 2025	600321.8	4850322.3	228.3	Earth Fill, Silty Clay Till	1.4–2.3	50
BH/MW 2	July 08, 2025	600380.3	4850266.3	227.5	Earth fill and Silty Clay Till	0.8–2.3	50
BH/MW 3	July 08, 2025	600358.7	4850346.6	228.2	Silty Clay Till	3.1–4.6	50

Notes:

mbgs-metres below ground surface

masl-metres above sea level



5.0 REGIONAL AND LOCAL SITE SETTING

5.1 Regional Geology

The current understanding of the surface geological setting of the Subject Site is based on scientific work conducted by the OGS (OGS, 2003). The Subject Site is located within an area mapped as till (5d), comprising of clay to silt-textured till. **Drawing No. 3** illustrates the mapped surficial geology for the Subject Site and the surrounding area.

The Oak Ridges Moraine Groundwater Program (ORMGP) produced a cross-sectional geological map to aid in the characterization of the general area. Considering the regional cross-section, it is understood that the overburden units prevalent in this area are as follows, with the youngest unit at the top:

- *Undifferentiated Sediments*: Undifferentiated sediments present in the ground surface, with an approximate thickness varying between 0.5 m to 2.4 m beneath the Subject Site.
- *Thorncliffe Formation*: The Thorncliffe Formation consists of glaciofluvial and glaciolacustrine sand and silt deposited approximately 30,000 to 50,000 years ago. The Thorncliffe Formation shows a considerable variation in grain size and thickness, both locally and regionally. It acts as a regional aquifer. Based on the ORMGP cross-section, the approximate thickness of the Thorncliffe at the Subject Site is 13.9 to 16.1 m across the Subject Site.

The underlying bedrock at the Subject Site is the Blue Mountain Formation, which consists of shale along with shale and minor limestone (OGS, 2007). A review of the ORMGP cross-section indicates that the bedrock could be contacted approximately between 14.4 to 18.5 mbgs beneath the Subject Site. However, findings of the current investigation suggests that the bedrock was contacted shallower.

5.2 Regional Physiography

The Subject Site lies partially within the physiographic region of Southern Ontario known as the South Slope, which is situated on the Till Plains (Drumlinized) physiographic feature and partially lies within the physiographic region of Southern Ontario known as the Peel Plain, which is situated on the Bevelled Till Plains physiographic feature. **Drawing No. 4** shows the location of the Subject Site within the regional physiography map.

5.3 Regional Topography and Drainage

A review of a regional topography map presented on **Drawing No. 5** indicates that the topography of the Subject Site and surrounding area is relatively flat and exhibits a slight decline in elevation towards south direction. The ground surface elevation ranges approximately between 227.5 metres above sea level



6.0 SOIL LITHOLOGY AND SUBSURFACE INVESTIGATION

The subsoil investigation has revealed that beneath a layer of earth fill, the Subject Site is underlain by a stratum of silty clay till extending to the termination depth of 5.1 mbgs. Auger refusal was observed at three (3) boreholes' locations indicating a shallow bedrock beneath the Subject Site. Information regarding borehole logs is presented in **Appendix A (Figures 1 to 4)**. The approximate locations of installed monitoring wells are shown on **Drawing No. 2**. Additionally, a cross-section Key plan and a subsoil profile are presented on **Drawing No. 8-1** and **8-2**, respectively. Based on a review of the geotechnical investigation report prepared by SEL (SEL Reference No. 2506-S157), the stratigraphy beneath the investigated areas of the Subject Site generally consists of the following:

6.1 Granular Fill

A layer of granular fill, 30 to 36 cm thick, was encountered at all BHs within the Subject Site.

6.2 Earth Fill

A layer of earth fill, contacted at the ground surface, was contacted at all the borehole and monitoring well (BH/MW) locations. It consisted of sandy silt with gravel, traces to some clay traces and occasional organic inclusions, and extends from ground surface to 1.4 mbgs. The natural water content values for the retrieved soil samples indicated very moist to wet conditions.

6.3 Silty Clay Till

A native silty clay till layer was contacted beneath the earth fill layer at all BH/MWs locations. It consists of a trace of sand and was found to be grey in color. The silty clay till unit was contacted at depths between 1.4 mbgs extending to maximum termination depth of 5.1 mbgs at BH/MW 4. It varies from stiff to hard in consistency. The moisture content for the retrieved samples indicates damp to moist conditions. Grain size analyses were performed on two selected samples of the silty clay till and the results are plotted on (**Appendix-A Figure 5**).



7.0 LOCAL HYDROGEOLOGICAL STUDY

7.1 Monitoring Well Development and Groundwater Level Monitoring

The groundwater levels in the monitoring wells were monitored, manually over three (3) monitoring events from July 28 to August 27, 2025, to record the fluctuation of the shallow groundwater table beneath the Subject Site. SEL measured the groundwater levels using an interface probe (Heron Water Tape Series #1900).

A summary of the groundwater level observations and their corresponding elevations are provided in **Table 7-1**.

Table 7-1- A Summary of Groundwater Monitoring

MW ID	Unit	Groundwater Level		
		July 28, 2025*	August 14, 2025	August 27, 2025
BH/MW 1	mbgs	1.5	1.5	1.6
	masl	226.8	226.8	226.7
BH/MW 2	mbgs	1.7	1.8	1.9
	masl	225.8	225.7	225.6
BH/MW 3	mbgs	2.2	2.5	2.4
	masl	226.0	225.7	225.8

Notes:

mbgs metres below ground surface

masl metres above sea level

**Groundwater table measured in undeveloped monitoring well*

As shown in **Table 7-1**, the highest and lowest water levels were measured at El. 226.8 masl and 225.6 masl at BH/MW 1 and BH/MW 2, respectively.

As previously discussed, the boreholes could not be extended deeper to install monitoring wells, due to the auger refusal and shallow bedrock at the proposed monitoring wells' locations. Since the BH/MWs 1 and 2 were partially screened within the fill material, the recorded water table may present the contribution of perched water anticipated from fill material.

7.2 Groundwater Flow Pattern

Since BH/MWs 1 and 2 were partially screened within the fill material, interpreting the shallow groundwater flow pattern is not feasible.



7.3 Hydraulic Conductivity Testing

7.3.1 Single Well Response Test

BH/MW 1 and 3 underwent a single well response testing (SWRTs) to assess the hydraulic conductivity (K) for the saturated water-bearing unit at the depths of the well screens. Each monitoring well was equipped with a digital transducer to record the fluctuation made to complete the SWRT. The results of the SWRT tests are presented in **Appendix C**, with a summary of the findings provided in **Table 7-2**.

Table 7-2- A Summary of Falling Head Hydraulic Conductivity Testing

Well ID	Ground El. (masl)	Monitoring Well Depth (mbgs)	Screen Interval (mbgs)	Screened Soil Strata	Hydraulic Conductivity (m/sec)	Test Method
BH/MW 1	228.3	2.3	1.4–2.3	Earth Fill, Silty Clay Till	1.2×10^{-5}	Falling Head
BH/MW 3	228.2	4.6	3.1–4.6	Silty Clay Till	4.0×10^{-5}	Falling Head

Notes:

mbgs metres below ground surface

masl metres above sea level

The estimated hydraulic conductivity measured at BH/MW 1 and BH/MW 3 for screened soil of Silty Clay Till and Silty Clay Till along with Fill material is 4.0×10^{-5} m/sec and 1.2×10^{-5} m/sec beneath the Subject Site, respectively.

7.4 Groundwater Quality

One (1) set of groundwater samples were collected, from one (1) selected monitoring well (BH/MW 3) to characterize its quality for evaluation against Peel Region Sanitary and Storm Sewer Use By-Law (BL_53_2010) parameters. Upon sampling, all of the bottles were placed in a cooler for shipment to the analytical laboratory. Sample analysis was performed by SGS Canada Inc., which is accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA). Results of the analysis are provided in **Appendix D**, with a discussion of the findings provided below. The chain of custody (COC) number for the submitted samples that underwent analysis is 045059.

The exceedances, together with the Storm and Sanitary Sewer Use Standards, are presented in **Table 7-3** below.



10.0 CLOSURE

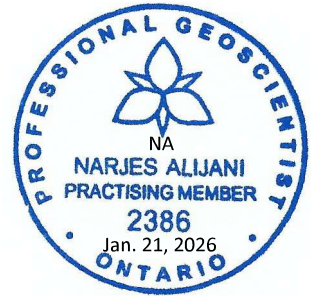
We trust that the above-noted information is suitable for your review. If you have any questions regarding this information, please do not hesitate to contact the undersigned.

Yours truly,

SOIL ENGINEERS LTD.

Alaa Alborno, B.Eng., EIT
Project Manager-Hydrogeological Services

Narjes Alijani, M.Sc., P.Geo.
Department Manager-Hydrogeological Services





N

References: Ontario Ministry of Natural Resources and Forestry
© King's Printer for Ontario, 2025

Key Map

Service Layer Credits: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community
Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

Legend

Approximate Boundary of Subject Site

Major Road

Waterbody

Watercourse

Borehole (1)

Borehole With Monitoring Well (3)

Soil Engineers Ltd.

Borehole and Monitoring Well Location Plan

Hydrogeological Assessment
Proposed Residential Development
6404 Mayfield Road
Town of Caledon

Reference No. 2506-W157

Date: December 12, 2025

Scale:

Metres

Drawing No. 2

FIGURE NO.: 1

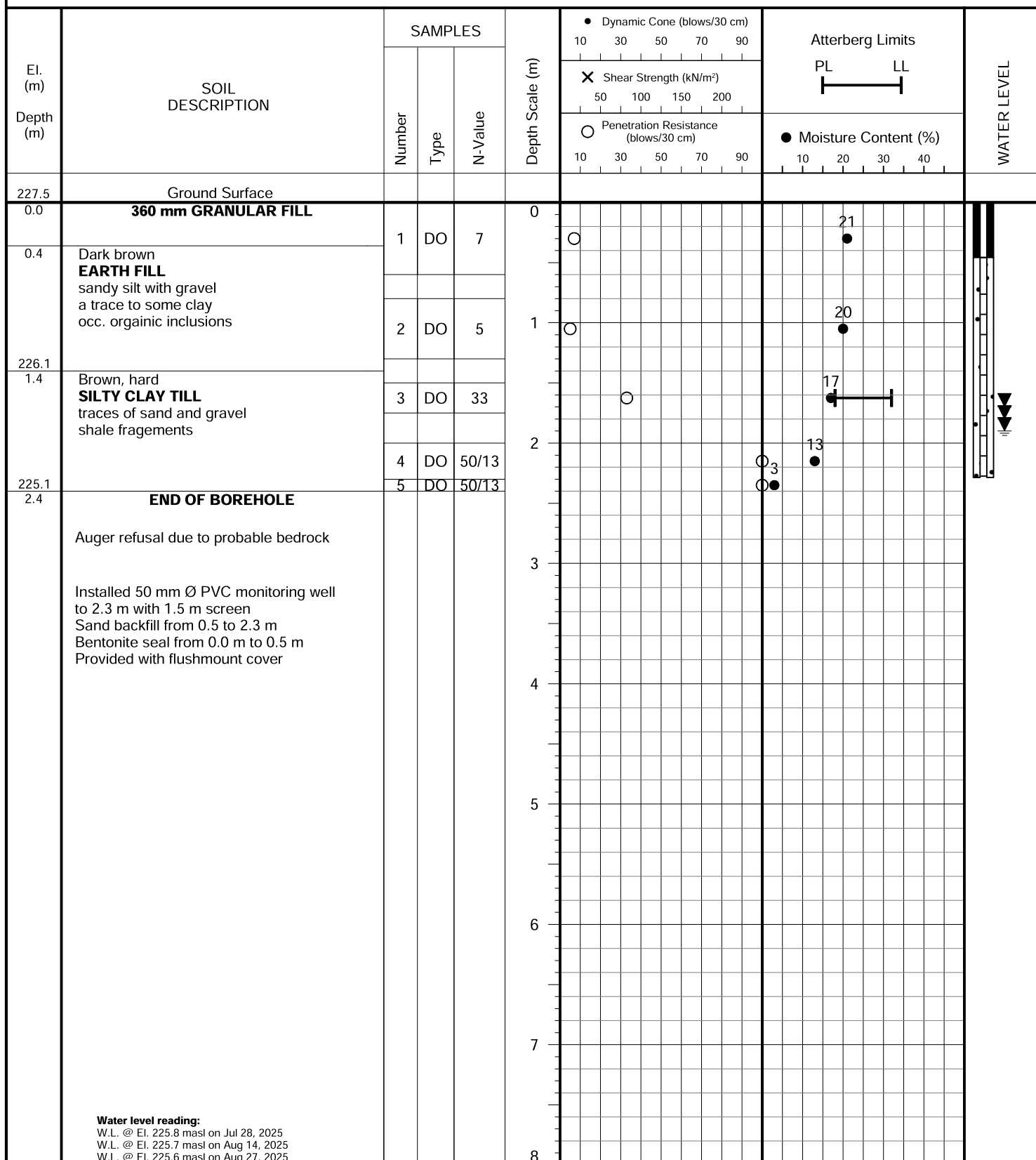
METHOD OF BORING: Hollow Stem Augers

DRILLING DATE: July 8, 2025

JOB NO.: 2506-W157

LOG OF BOREHOLE: BH/MW 2

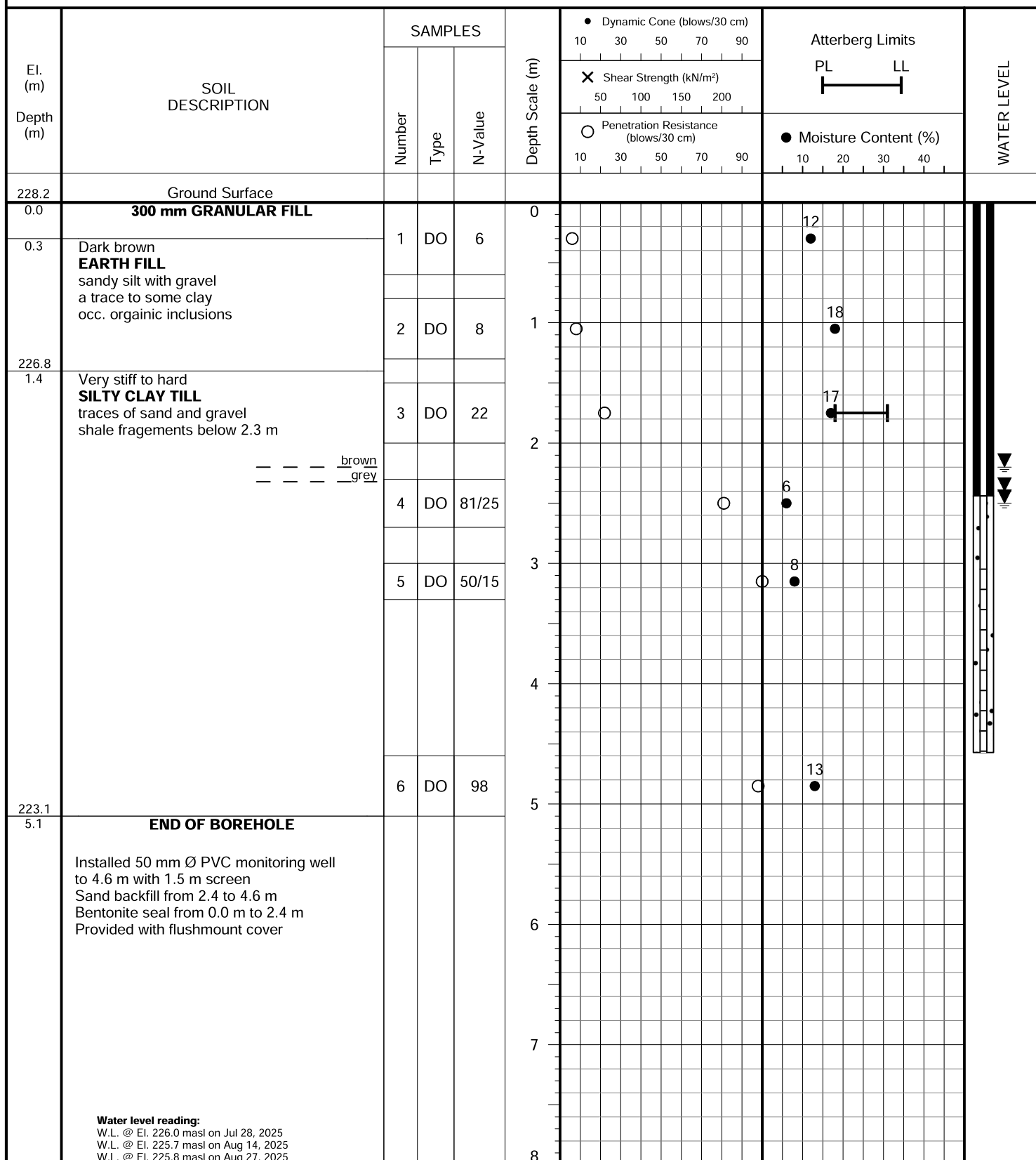
FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Transportation Terminal**METHOD OF BORING:** Hollow Stem Augers**PROJECT LOCATION:** 6404 Mayfield Road, Town of Caledon**DRILLING DATE:** July 8, 2025**Soil Engineers Ltd.**

JOB NO.: 2506-W157

LOG OF BOREHOLE: BH/MW 3

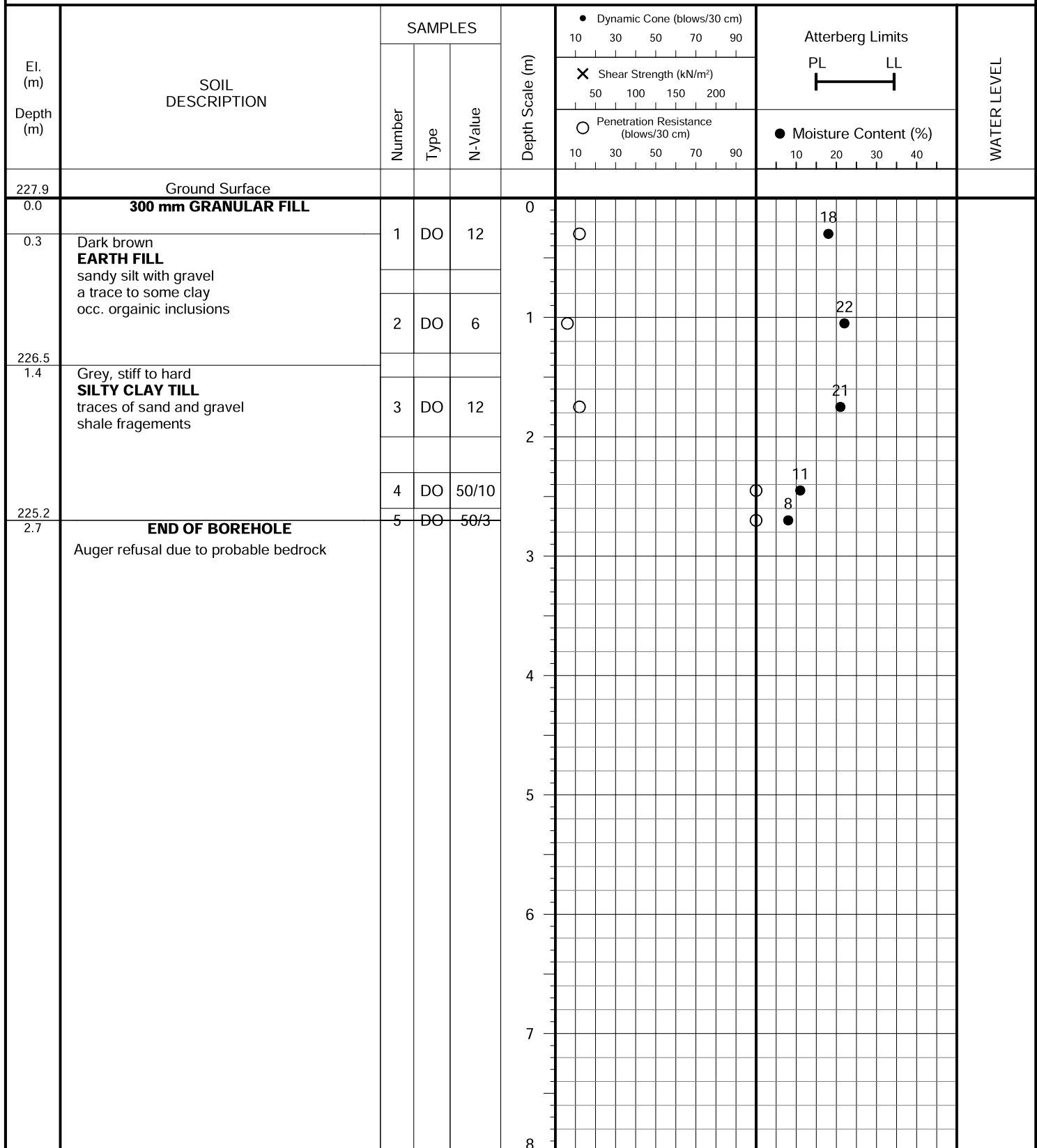
FIGURE NO.: 3

PROJECT DESCRIPTION: Proposed Transportation Terminal**METHOD OF BORING:** Hollow Stem Augers**PROJECT LOCATION:** 6404 Mayfield Road, Town of Caledon**DRILLING DATE:** July 8, 2025**Soil Engineers Ltd.**

JOB NO.: 2506-W157

LOG OF BOREHOLE: BH 4

FIGURE NO.: 4

PROJECT DESCRIPTION: Proposed Transportation Terminal**METHOD OF BORING:** Hollow Stem Augers**PROJECT LOCATION:** 6404 Mayfield Road, Town of Caledon**DRILLING DATE:** July 8, 2025**Soil Engineers Ltd.**