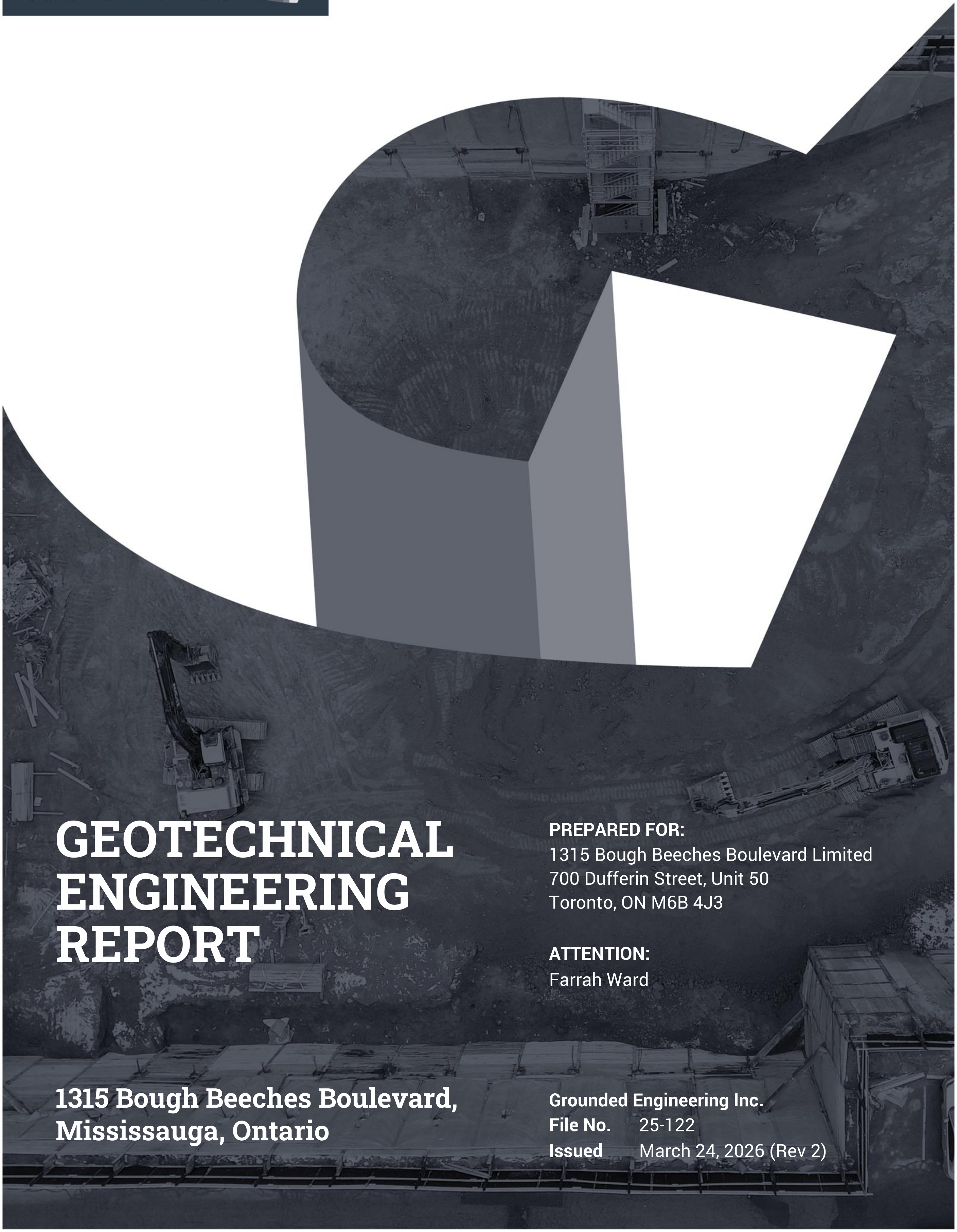

GEOTECHNICAL ENGINEERING REPORT

PREPARED FOR:
1315 Bough Beeches Boulevard Limited
700 Dufferin Street, Unit 50
Toronto, ON M6B 4J3

ATTENTION:
Farrah Ward

1315 Bough Beeches Boulevard, Mississauga, Ontario

Grounded Engineering Inc.
File No. 25-122
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1 Introduction

1315 Bough Beeches Boulevard Limited has retained Grounded Engineering Inc. to provide geotechnical engineering design advice for their proposed development on a portion of the property at 1315 Bough Beeches Boulevard, in Mississauga, Ontario.

The property is located east of Dixie Road, north of Rathburn Road East, and west of Bough Beeches Boulevard. The property is rectangular in shape with a total area of about 23,727± m^2. The property is currently occupied by a twenty (20) storey apartment building in the approximate middle of the site, connected to a 1 storey building housing a swimming pool to the west and one (1) level of underground parking extending for a major portion of the property surrounding the existing apartment building. The existing buildings are surrounded by asphalt pavements and parking lots to the north, east, and south. The rest of the property is surrounded by landscaped portions consisting of berms, and trees. There are two outdoor sports facilities (tennis and basketball courts) west and southwest of the existing building, separated by chain-link fencing from the landscaped portions. There are multiple berms in the landscaped portion of the site.

A new development consisting of a thirteen (13) storey building is proposed at the southeast portion of the property near the intersection of Rathburn Road East and Bough Beeches Boulevard (the “Subject Site”, See Figure 3). The area of the subject site presently consists of an on-grade asphalt parking lot and a landscaped berm. Two levels of underground parking (P2) are proposed for this development. The underground structure is designed in a corkscrew arrangement with the higher portion of the P2 set at an FFE of Elev. 141.58 m, and the lowest portion of the P2 (referenced as the Lower P2 in the architectural drawings) set at an FFE of Elev. 140.05± m. The new P2 underground structure will be situated immediately adjacent to the existing 1-storey underground parking garage.

The following is the revision history for this geotechnical report:

- Report originally issued on March 6, 2026
- Revision 1 (March 11, 2026): Report was updated with shear wave velocity testing results conducted for the proposed development. The seismic site designation was updated based on these results.
- Revision 2 (March 24, 2026): Report has been updated with groundwater level measurements from March 2026.

Grounded has been provided with the following documents to assist in our geotechnical scope of work:

- Site survey, prepared by RPE Surveying Ltd. (July 4, 2025).

- Architectural Drawings, “1315 Bough Beeches, Mississauga, Ontario”; Project 30280666, dated February 12, 2026, prepared by Arcadis.

Grounded has previously conducted subsurface investigation at this site for a separate development proposed at a different portion of the property (west portion of the property) and had previously issued Geotechnical Engineering and Hydrogeological Review reports for it.

Grounded’s subsurface investigation of the site to date includes seventeen (17) boreholes:

- Boreholes 1 to 6 were advanced in the west portion of the property as part of a previous geotechnical investigation scope between July 22, 2025, and August 8, 2025.
- Boreholes 7 to 9 were advanced at the eastern portion of the property between August 9 and 13, 2025 for the purposes of environmental assessment of the property under a separate scope. Among these boreholes, Borehole 7 is within the footprint of the currently proposed development.
- Boreholes 101 to 108 were advanced within the footprint of the currently proposed (southeast) development between December 15 and 23, 2025.
    - Among these boreholes, Boreholes 102, 103 and 107 were advanced as part of environmental engineering scope.
    - Boreholes 101, and 104, 105, 106 and 108 were advanced within the proposed building footprint to provide geotechnical engineering recommendations for the proposed development.
    - Borehole 101 was drilled adjacent to supplement the subsurface information obtained through Borehole 7. Since Borehole 7 did not extend / infer top of bedrock, Borehole 101 was straight drilled to without sampling to a depth of 13.7± m adjacent to Borehole 7 and split spoon sampling was commenced at Elev. 137.2± m.

Based on the borehole findings, geotechnical engineering advice for the proposed development is provided for foundations, seismic site designation, earth pressure design, slab on grade design, basement drainage, and pavement design. Construction considerations including excavation, groundwater control, and geostructural engineering design advice are also provided.

At the time of this investigation, it was understood by Grounded that a raft foundation design was the preferred foundation option, and that caissons were not required. For this reason, rock coring was not conducted within the subject portion of the site. Regardless, this geotechnical engineering report provides preliminary recommendations for caisson foundation design. If caissons are deemed to be the preferred foundation option, additional boreholes with rock coring and rock mechanics laboratory testing will be required for detailed design of these elements.

Grounded Engineering must conduct the on-site evaluation of founding subgrade as foundation and slab construction proceeds. This is a vital and essential part of the geotechnical engineering function and must not be grouped together with other “third-party inspection services”. Grounded will not accept responsibility for foundation performance if Grounded is not retained to carry out all the foundation evaluations during construction.

2 Ground Conditions

The borehole results are detailed on the attached borehole logs. Our assessment of the relevant stratigraphic units is intended to highlight the strata as they relate to geotechnical engineering. The ground conditions reported here will vary between and beyond the borehole locations.

The stratigraphic boundary lines shown on the borehole logs are assessed from non-continuous samples supplemented by drilling observations. These stratigraphic boundary lines represent transitions between soil types and should be regarded as approximate and gradual. They are not exact points of stratigraphic change.

Elevations are measured relative to geodetic datum (as established by the RPE Survey). The horizontal coordinates are provided relative to the Universal Transverse Mercator (UTM) geographic coordinate system.

2.1 Stratigraphy

The following stratigraphic summary is based on the results of the relevant boreholes and the geotechnical laboratory testing.

A subsurface profile showing stratigraphy and engineering units is appended (Figure 4).

2.1.1 Surficial and Earth Fill

Surficial fill (pavements, aggregate, topsoil, etc.) thicknesses were observed in individual borehole locations through the top of the open borehole. Thicknesses may vary between and beyond each borehole location.

Boreholes 103 to 108 encountered asphalt pavement at the existing ground surface, ranging in thickness from 125 to 150 mm. The asphalt pavement was immediately underlain by a 65 to 280 mm thick aggregate layer which forms the pavement base/subbase. Boreholes 7, 9, and 102 encountered approximately 100 to 225± mm of topsoil at the existing ground surface. Borehole 101 was straight-drilled adjacent to Borehole 7 to a depth of 13.7± m and sampling was commenced from a depth of 13.7± m onwards.

Underlying the surficial materials, the boreholes observed a layer of earth fill that extends to depths of 0.8 to 2.6± m below grade (Elev. 148.9 to 147.0± m). Boreholes 102 and 103 were terminated at their target investigative depths of 2.1 m below grade (Elev. 148.7 and Elev. 147.5 m respectively) in this earth fill. The earth fill varies in composition but generally consists of sandy silt with a varying clay content (trace to clayey), and silty sand with trace gravel. The existing fill also contains occasional glass fragments, brick fragments, wood fragments, rootlets, and rock fragments (which may imply the presence of cobbles or larger rock pieces within the fill). The earth fill is typically dark brown with orange to light brown with grey, and moist.

Boreholes 102, 103 and 107 were sampled using a 75± mm diameter split spoon sampler instead of the standard 50± mm diameter split spoon sampler. Hence, Standard Penetration Test (SPT) results (N-values) are not measured and reported for Boreholes 102, 103 and 107. SPT N-Values measured in the fill range from 5 to more than 50 blows per 300 mm of penetration ("bpf"). Due to inconsistent placement and the inherent heterogeneity of earth fill materials, the relative density of the earth fill is variable.

2.1.2 Glacial Till

Underlying the fill materials, the boreholes encountered an undisturbed native glacial till deposit with a matrix that ranges between cohesionless to slightly cohesive silts (sandy silt to sandy clayey silt to silt and sand to silty sand). These soils are grouped together as the "**glacial till**" unit. This unit was encountered at 0.8 to 2.6 m below grade (Elev. 148.9 to 147.0± m) and extends down to depths of 9.1 to 12.4 m below grade (Elev. 140.5 to 137.2± m). Boreholes 8 and 107 were terminated at their target investigation depth within this unit. The glacial till is generally light brown with orange, to grey, and moist to wet, and contains occasional rock fragments (which may infer the presence of cobbles within the till). SPT N-values measured in the glacial till unit range from 12 to greater than 50 bpf (compact to very dense in the cohesionless portions and stiff to hard in the cohesive portions).

2.1.3 Sands and Silts

Underlying the glacial till unit, all boreholes encountered an undisturbed cohesionless deposit consisting of sands with varying amounts of silts and gravel (ranging between sand and silt with trace gravel to gravelly, silty sands to silt, some sand with trace clay). There are occasional gravelly deposits in this unit as observed in Boreholes 105 and 108. These soils are grouped together as the "**sands and silts**" unit. This unit was encountered at depths of 9.1 to 13.7± m below grade (Elev. 140.5 to 137.2± m) and extends down to depths of 15.2 to 21.0± m below grade (Elev. 134.3 to 128.6± m). Boreholes 7 was terminated at its target investigative depth in this unit.

The sands and silts are generally grey, and wet, and contain occasional shale fragments. SPT N-values measured in the sands unit are consistently over 50 bpf (very dense).

2.1.4 Inferred Bedrock

Within the proposed development portion of the site, bedrock was inferred in Boreholes 101, 104, 105, 106, and 108 underlying the overburden soils at depths of 15.2 to 21.0± m below grade (Elev. 134.3 to 128.6± m). These boreholes indirectly inferred the top of weathered bedrock through auger cuttings, split spoon samples, and auger grinding/resistance observations. Each of these boreholes was terminated due to auger and sampler refusal (at target investigation depth) at elevations ranging from Elev. 134.0 to 128.2 m.

Rock coring was not conducted within the subject portion of the site for development, however, bedrock was confirmed by rock cores recovered in Boreholes 2 and 6 (in the western portion of the property) to depths of 25.9 and 22.0± m below grade (Elev. 123.9 and 129.2± m, respectively).

It should be noted that the bedrock changes in elevation across the site, with Boreholes 4, 5 and 6 in the southwestern portion of the site encountering bedrock between Elev. 134.1 to 135.9 m, and Boreholes 1, 2 and 3 in the northwestern portion of the property encountering bedrock between Elev. 128.5 to 129.7± m. Boreholes 101, 104, 105, 106, and 108 drilled in the southeastern portion of the property encountered bedrock between Elev. 134.3 to Elev. 128.6± m.

Based on the boreholes in other areas of this property that were cored, the bedrock beneath the property is the Georgian Bay Formation, which comprises thin to medium bedded grey shale and limestone of Ordovician age. The fissile shale is interbedded with non-fissile calcareous shale, limestone, dolostone, and calcareous sandstone (conventionally grouped together as “limestone”) which are typically laterally discontinuous. Per the appended terminology, the Georgian Bay shale is typically classified as “weak” whereas the limestone interbedding is classified as “medium strong to strong”.

Joints occurring within the shale are closely to very closely spaced, and typically weathered with a veneer to coating of clay. Widely-spaced subvertical joints (closed, planar, clean) were also observed within the shale.

A summary of the engineering properties of the Georgian Bay Formation is presented in the Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects* (March 1983). The relevant parameters from that document are as follows:

Table 2.1 – Summary of MTO Georgian Bay Formation Parameters

	Uniaxial Compressive Strength (MPa)	Young's Modulus (GPa)	Dynamic Modulus (GPa)	Poisson's Ratio
Average	28	4	19	0.19
Range	8 to 41	0.5 to 12	6 to 38	0.1 to 0.25

Directly below the overburden soils, the uppermost portion of bedrock is typically weathered. The MTO[1] provides a *typical weathering profile of a low durability shale* reproduced from Skempton, Davis, and Chandler, which characterizes weathered versus unweathered shale as follows:

Table 2.2 – Typical Weathering Profile of a Low Durability Shale

	Zone	Description	Notes
Fully Weathered	IVb	soil-like matrix only	indistinguishable from glacial drift deposits, slightly clayey, may be fissured
Partially Weathered	IVa	soil-like matrix with occasional pellets of shale less than 3 mm dia.	little or no trace of rock structure, although matrix may contain relic fissures
	III	soil-like matrix with frequent angular shale particles up to 25 mm dia.	moisture content of matrix greater than the shale particles
	II	angular blocks of unweathered shale with virtually no matrix separated by weaker chemically weathered but intact shale	spheroidal chemical weathering of shale pieces emanating from relic joints and fissures, and bedding planes
Unweathered (Sound)	I	shale	regular fissuring

In glacial till overburden soils directly overlying bedrock, a zone of soil with fragmented shale is often observed and interpreted as either the lowest portion of the native overburden, or as partially weathered Zone III rock. This interpretation is subjective and depends on the investigator. There is occasionally a concentration of boulders in the soil just above the bedrock that can be mistakenly identified as bedrock where rock coring is not performed. Weathering Zones III and IV are frequently not present due to glacial scouring action, which often removes these zones from the bedrock surface.

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[1] Franklin, J.A., Gruspier, J.E., 1983. "Evaluation of Shales for Construction Projects – An Ontario Shale Rating System", Ontario Ministry of Transportation and Communication, Research Report RR229.

The bedrock surface as indicated on the Borehole Logs from this investigation is intended to be consistently interpreted as the surface of Zone II unless noted otherwise. Weathered and sound bedrock elevations are summarized as follows:

Borehole	Ground Surface Elevation (m)	Partially Weathered (Zone II) Bedrock		Sound (Zone I) Bedrock	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
101	150.9	18.3	132.6	*n/a*	*n/a*
104	149.5	15.2	134.3	*n/a*	*n/a*
105	149.4	18.4	131.0	*n/a*	*n/a*
106	149.7	20.0	129.7	*n/a*	*n/a*
108	149.6	21.0	128.6	*n/a*	*n/a*

n/a: Bedrock was not cored at these borehole locations and hence, depth to and elevation of sound bedrock is not available.

Rock Quality Designation (RQD) is an index measurement that refers to the total length of pieces of sound core in a core run that are at least 100 mm in length, expressed as a percentage of the total length of that core run. Only natural discontinuities are used in assessing RQD. The RQD of the recovered rock cores was typically 0 to 26% in the weathered bedrock, and between 10% and 100% in the sound bedrock.

RQD underrepresents the competency of the Georgian Bay Formation and is not appropriate for horizontally bedded fissile shale. In this formation, the RQD is typically low due to the fissility of the shale as well as the closely spaced horizontal bedding planes. Our results are typical of this formation.

There are near-vertical joint sets within this shale that are typically very widely spaced at over 2 m apart. There are also several faults typically referred to as “shear zones” found within the formation, which are observed as zones of rock rubble within the cores. These faults defy discovery in conventional vertical boreholes.

The jointing and crush zones in the rock are related to the state of stress in the deposit. Research in the Greater Toronto Area has revealed that the bedrock contains locked-in horizontal stresses that could be remnants of the foreshortening that occurred in the earth’s crust during continental glaciation several thousand years ago. Documented experiments have indicated that the major principal stress is of the order of 2 MPa in the upper 1 to 2 metres of the deposit where the rock is weathered and contains more fractures. Intact rock can have an internal major principal stress as high as 4 to 5 MPa. The major and minor principal stresses are horizontal and may be oriented in any direction. The empirical approach to vertical stress below the top of bedrock is to use a uniform pressure distribution below the top of bedrock elevation that is equal to the maximum earth pressure calculated for the lowest level of soil in the profile.

The Georgian Bay Formation has been known to issue gases. There are instances where both methane and hydrogen sulphide gas emissions have been detected in excavations made in the Georgian Bay Formation. While there was no specific indication of gas emissions from the boreholes made in this investigation, the potential for gas emissions from this formation should be taken into account during the design and construction phases of the project.

2.2 Groundwater

On completion of drilling, the boreholes were filled with drill fluid (from mud rotary drilling) and measuring the unstabilized groundwater level after drilling was not practical. The boreholes were cased by hollow stem augers on completion, and cave measurement was not practical.

Monitoring wells were installed in all boreholes except Boreholes 8, 101, 102, 103, 106, and 107, and stabilized groundwater levels were measured in each of the installed monitoring wells. The groundwater observations are shown on the Borehole Logs and a detailed table of monitoring well observation data is appended (Table 1).

Groundwater levels fluctuate with time depending on the amount of precipitation and surface runoff, and may be influenced by known or unknown dewatering activities at nearby sites.

The design groundwater table for engineering purposes is at Elev. 144.0± m. The groundwater table is in all the native soil units. The glacial till unit has a low permeability and will yield only minor seepage in the long term. The sand and silt unit has a high permeability and will yield free-flowing water when penetrated below the groundwater table.

There is also infiltrated stormwater perched in the earth fill which is flowing down towards the groundwater table.

Grounded has prepared a hydrogeological report for this site (File No. 25-122) under separate cover.

2.3 Corrosivity and Sulphate Attack

Three (3) soil samples were submitted for corrosivity testing parameters (pH, Resistivity, Electrical Conductivity, Redox Potential, Sulphate, Sulphide and Chloride). The Certificate of Analyses and interpretation sheet is appended.

The soil samples were analysed for soluble sulphate concentration and compared to the Canadian Standard CAN3/CSA A23.1-M94 Table 3, *Additional Requirements for Concrete Subjected to Sulphate Attack*. Corrosivity parameters are also used for assessing soil corrosivity applicable to cast iron alloys, according to the 10-point soil evaluation procedure described in the American Water Work Association (AWWA) C-105-18 standard[2].

The analytical results only provide an indication of the potential for corrosion. The results of this analysis are in reference to only the soil samples collected from specific locations, and soil chemistry may vary between and beyond the locations of the analysed samples. In summary:

- All of the samples have negligible sulphate concentrations.
- All three samples scored more than 10 points in the AWWA C-105 evaluation. Corrosion protective measures are **recommended** for cast iron alloys.

2.4 Frost Heave Susceptibility of Soils

Frost heave can occur in soils that are frost-susceptible, when there is b) a source of water (e.g., the groundwater table or stormwater infiltration) and c) those soils with a source of water are exposed to freezing temperatures.

Frost susceptibility in soils refers to the tendency of soils to grow ice lenses and heave during freezing. This tendency varies between soil types, with fine-grained soils with low cohesion and high capillarity generally having the highest susceptibility.

The U.S. Army Corps of Engineers (USACE) provides a soil classification system[3] relating soil types to different levels of frost susceptibility. The Canadian Foundation Engineering Manual ("CFEM") 5th Edition adopts and updates this frost design screening criteria approach (see table below). Frost-susceptible soils are classified in groups, F1 to F4, generally coinciding with increasing order of susceptibility. Soil in the latter groups tend to have higher rates of frost heave and lower strength after freeze-thaw cycles.

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[2] ANSI/AWWA C105/A21.5-18, Appendix A

[3] U.S. Army Corps of Engineers. April 1984. Pavement Criteria for Seasonal Frost Conditions. Engineer Manual No. 1110-3-138.

Table 2.3 – CFEM 5th Ed. Table 14.1, Frost Design Soil Classification adapted from USACE

Frost Group	Soil Type	Percentage Finer than 0.02 mm, by Weight	Typical USCS[4] Soil Types
F1	Gravelly soils	3-10	GM, GW-GM, GP-GM
F2	Gravelly soils	10-20	GM, GW, GP-GM
	Sands	3-15	SM, SW-SM, SP-SM
F3	Gravelly soils	> 20	GM-GC
	Sands, except very fine silty sands	> 15	SM, SC
	Clays, PI > 12	-	CL, CH
F4	All silts	-	ML, MH
	Very fine silty sands	> 15	SM
	Clays, PI < 12	-	CL, CL-ML
	Varved clays and other fined-grained, banded sediments	-	CL and ML; CL, ML, and SM; CL, CH, ML, and SM

The table above is interpreted by Grounded as follows:

- All soils in these groups are frost-susceptible to some degree per the USACE.
- Non-frost susceptible groups are not listed.
- Within each group, the soil's frost susceptibility can vary from very low to very high (e.g., an F4 soil could potentially heave between 1 to 10+ mm/day), though this has never been quantitatively standardized.
- Soils in the F4 group are especially susceptible to frost.

The site soils are classified by their Frost Group (level of frost susceptibility) according to their grain size data and USCS classification.

Stratum	Percentage Finer than 0.02 mm, by Weight	USCS Symbol	Frost Group
Earth Fill	Est. 55% or higher	ML, MH, SM, CL*	F3/F4
Glacial Till	Est. > 15%	ML, MH, SM, SC, GM-GC	F3/F4

* inferred

2.5 Pressuremeter Testing

In situ pressuremeter testing (PMT) was conducted by Grounded Engineering using an N-size Texam Pressuremeter. Our equipment is lab calibrated before every project, and field calibrated on each day of field testing. The raw data is corrected for membrane stiffness and system

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[4] ASTM D2487-17

volume loss to obtain a corrected plot of probe pressure versus change in probe volume, from which we obtain a pressuremeter modulus. Calibrations and data correction are in accordance with ASTM D4719. The field test data are appended.

The PMT modulus is converted to an equivalent Young's modulus using the following simplified relationship:

$$E_{PMT} \,/\, \alpha = E$$

E_{PMT} = Pressuremeter Modulus (MPa)
α = Menard Factor (unitless)
E = Young's Modulus (MPa)
E_{ur} = Young's Modulus, unload-reload (MPa)

Alpha is interpreted using a first principles derivation which assumes the soil around a pressuremeter behaves according to the general orthotropic elastic equations. This is compared to the results given by the Menard table and the Pressiorama chart, as well as the methods for PMT Young's Modulus interpretation outlined by Mair and Wood[5] and others. As such, the Young's Modulus reported is interpreted based on engineering judgement for present purposes.

The detailed pressuremeter test results are appended, and the estimated Young's Modulus results are also shown on the attached Borehole Logs and Subsurface Profile. The test results are summarized as follows:

Table 2.4 – Summary of Pressuremeter Test Results

Borehole	Depth of Test (m)	Elevation of Test (masl)	E (MPa)	E_{ur} (MPa)	Engineering Unit
104	6.9	142.6	16	132	Glacial Till
104	9.9	139.6	120	1172	Glacial Till
104	13.0	136.5	70	558	Sand and Silt
108	7.6	142.0	*n/a**	*n/a**	Glacial Till
108	11.4	138.2	453	4810	Sand and Silt
108	14.5	135.1	188	1171	Sand and Silt

n/a: Unsuccessful Test – Probe ruptured during testing*

A measurement of the lateral earth pressure coefficient (K_0) is also made directly from the PMT data. This measurement likely represents $K_{0\text{-}OCR}$ and not the real design K_0 values (in the unload-

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[5] Mair, R.J. and Wood, D.M. (1987) Pressuremeter Testing: Methods and interpretation, CIRIA/Butterworths, London.

reload condition for example), as reported by many including Alpan[6], Hamouche et al.[7], and Mayne and Kulhawy[8]. It is also heavily affected by borehole disturbance in the pre-bored PMT application (Mair and Wood). This data is appended for regulatory review purposes only.

3 Geotechnical Engineering Recommendations

Based on the factual data summarized above, we are providing the following geotechnical engineering design recommendations. Contractors must review the factual data while bidding or scoping services for this project and must provide their own opinion as to means, methods, and schedule.

This report assumes that the design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards, and guidelines of practice. If there are any changes to the site development features, or there is any additional information relevant to the interpretations made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Grounded should be retained to review the implications of these changes with respect to the contents of this report.

The proposed development will consist of a thirteen (13) storey residential building with a 1-storey podium structure, and two (2) levels of underground parking (P2). The underground structure is designed in a corkscrew arrangement with the higher portion of the P2 set at an FFE of Elev. 141.58 m, and the lowest portion of the P2 (referenced as the Lower P2 in the architectural drawings) set at an FFE of Elev. 140.05 m. The new P2 underground structure will be constructed immediately adjacent to the existing P1 underground structure.

The design groundwater table for engineering purposes is at Elev. 144.0± m. The P2 excavation will extend several metres below the groundwater table in cohesionless soils (cohesionless glacial till and sand and silts). It will be necessary to positively dewater the site to a minimum 1.2 m below the lowest excavation elevation prior to excavation to preserve the in situ integrity of the native soils. If the subsurface is not dewatered prior to excavation, the native soils will become disturbed by the ingress of groundwater and the recommendations for bearing capacity below will not be valid.

In the permanent condition, if the structure is designed as a drained structure, perimeter and subfloor drainage systems are required for the underground structure. These systems should be designed with a filtering mechanism such that it prevents removal of fines during dewatering.

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[6] Alpan, I. (1967) The Empirical Evaluation of the Coefficient K_0 and K_{0R}

[7] Hamouche, K.K., Leroueil, S., Roy, M., and Lutenegger, A.J. (1995) In Situ Evaluation of K_0 in Eastern Canada Clays, in *Can Geotech J.* ***32****: 677-688.*

[8] Mayne, P.W., and Kulhawy, F.H. (1982) K_0-OCR Relationships in Soil, in *Journal ASCE,* ***108*** *(GT6), 851-72.*

Alternatively, if the proposed structure is designed as a watertight structure, then the below-grade structure should be designed to resist the hydrostatic pressure (horizontal and uplift) using a static groundwater table at Elev. 144.0± m.

3.1 Foundation Design Parameters

The following foundation options have been considered and are discussed in the following sub-sections.

- Spread footings (Podium structures only)
- Raft foundation
- Caissons (end-bearing into sound bedrock – preliminary considerations only)
- Alternative Foundation Options: Helical Piles and Micropiles

The top of bedrock elevation varies across the property as discussed in Section 2.1.4. Boreholes 101, 104, 105, 106, and 108, which were drilled in the subject portion of the site, encountered bedrock at elevations ranging from Elev. 134.3 to 128.6± m. It should be noted that rock coring was not conducted in this portion of the property. If caissons extending into bedrock are to be considered for detailed design, additional boreholes extending to bedrock with rock coring and rock mechanics laboratory testing will be required.

Since the proposed P2 underground structure will be constructed immediately adjacent to the existing P1 underground structure, consideration must be given to the potential effects of the proposed development on the existing adjacent structure via induced settlement, which can potentially propagate from new foundation loads in proximity to the existing underground structure. This analysis is typically conducted at the detailed design stage. This is separate from the need to underpin the foundations to maintain at-rest conditions during construction as discussed in section 5.1.1.

3.1.1 General Foundation Recommendations

Grounded should be retained by the Owner to review the structural engineering drawings prior to issue or construction, to ensure that the recommendations in this report have been appropriately implemented.

It will be necessary to positively dewater the site to a minimum 1.2 m below the lowest excavation elevation prior to excavation to preserve the in situ integrity of the native soils. If the subsurface is not dewatered prior to excavation, the native soils will become disturbed by the ingress of groundwater and the recommendations for bearing capacity below will not be valid.

Footings stepped from one elevation to another should be offset at a slope not steeper than 7 vertical to 10 horizontal. This requirement exists to avoid undermining adjacent footings at the higher elevation.

The lowest levels of unheated underground parking structures two or more levels deep are, although unheated, still warmer than typical outdoor winter temperatures in the Greater Toronto Area. Interior foundations (or pile caps) with 900 mm of frost cover perform adequately, as do perimeter foundations with 600 mm of frost cover. Where foundations are next to ventilation shafts or are exposed to typical outdoor temperatures, 1.2 m of earth cover (or equivalent insulation) is required for frost protection.

The founding subgrade must be cleaned of all unacceptable materials and approved by Grounded prior to pouring concrete for the footings. Such unacceptable materials may include disturbed or caved soils, ponded water, or similar as indicated by Grounded during founding subgrade inspection. During the winter, adequate temporary frost protection for the footing bases and concrete must be provided if construction proceeds during freezing weather conditions.

3.1.2 Spread Footings

Foundations made below the P2 underground structure will bear on undisturbed very dense glacial till or very dense silts and sands. Conventional spread footings made to bear on these soils may be designed using a maximum factored geotechnical resistance at ULS of 900 kPa. The geotechnical reaction at SLS is 750 kPa, for an estimated total settlement of 25 mm.

The capacities provided above are based on an individual spread footing foundations that are 2 to 3 m wide, spaced one footing width apart, and embedded a minimum of 1.5± m below FFE. These minimum requirements apply in conjunction with the above recommended geotechnical resistance regardless of loading considerations. The geotechnical reaction at SLS refers to an estimated settlement which for practical purposes is linear and non-recoverable. Differential settlement is related to column spacing, column loads, and footing sizes.

3.1.3 Raft Foundation

A 20 x 46± m raft underlying the tower is considered in the bearing capacity discussion below. Raft slabs for a podium structure will be subjected to much less load, and will not govern design.

Considering the P2 FFE of 141.6± m (and a lower P2 set at an FFE of Elev. 140.0± m), it is assumed that a raft would be founded around Elev. 139.0± m, on undisturbed (dewatered) very dense native soils.

The preliminary raft design parameters assume a uniform load at the base of the raft. In reality, raft loads are non-uniform; they are typically highest at the core and lowest at the perimeter. The preliminary parameters below are provided as the initial step in determining raft feasibility (a structural task). The detailed design process is described below.

Bulk excavation to underside of raft elevation (Elev. 139.0± m) will induce a reduction in effective stress of 170± kPa, which is the unload stress. Utilizing measured soil stiffness parameters, analysis of a uniformly loaded raft foundation shows that a uniform total applied SLS bearing pressure of 500± kPa (incorporating a 0.9 factor as per the CFEM 5th edition) at the base of the raft will generate an estimated 25± mm of settlement. Similarly, a uniform geotechnical reaction at SLS of 850± kPa will generate an estimated 50± mm of settlement.

The modulus of subgrade reaction for design of a raft slab is a function of the size of the raft, the applied load, and whether loading is within the recompression range or the virgin range. On the basis of our preliminary stiffness parameters and the assumption of uniform raft loading, the preliminary modulus of subgrade reaction appropriate for 20 x 46 m raft design at this site is about 13,800 kPa/m for loads over 170± kPa SLS.

The maximum factored geotechnical resistance at ULS of this 19 x 41 m raft foundation is 3,600 kPa. Raft foundation design is typically governed by service load criteria.

Detailed raft design is an iterative process between the structural and the geotechnical engineer. Once a draft structural design is completed by the structural engineer, the resulting non-uniform raft pressure distribution is provided to us (typically as a contour plot of SLS pressures). Grounded then models that non-uniform pressure distribution to more accurately estimate the settlement at each point under the raft. The resulting estimated settlement distribution is then sent back to the structural engineer to assess the total and differential settlements under the raft, as well as lateral impacts on adjacent footings and structures. The structural design is then modified as required.

During construction, the subgrade at founding elevation should be cut neat, inspected, and immediately protected by a mud slab (lean concrete) to provide a working surface. The subsurface must not be proofrolled as this activity would further weaken these soils. The raft slab is then constructed on top of the mud slab. Prior to pouring the mud mat and foundation, the foundation subgrade must be cleaned of all deleterious materials such as softened, disturbed or caved materials, or standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the raft foundation base and concrete must be provided.

As the raft slab is to be fully watertight, the structure must be designed to resist uplift and lateral hydrostatic pressure on foundation walls. During construction, it will be necessary to consider the potential uplift pressure on the underside of a raft foundation due to hydrostatic forces. Dewatering operations during construction must continue until such time as the structural dead load exceeds the potential uplift forces (with suitable partial factors (LRFD) included in this assessment). A design groundwater elevation of 144.0± m is to be used.

Differential settlement is related to real non-uniform raft load distribution and must be assessed as part of the detailed design process. Impacts to adjacent structures caused by settlement within the raft's lateral zone of influence will also need to be reviewed by the structural engineer. At this site in particular, the existing apartment building should be assessed for settlement impacts by the proposed raft.

3.1.4 Tiedown Anchors for Rafts / Watertight Structure

If deemed necessary by the structural engineer, micropile tiedowns can be designed to resist uplift.

One or more prototype anchors must be performance-tested to at least 200% of design load to demonstrate the anchor capacity and validate design assumptions for these permanent tiedowns, per OPSS 942.07.12.05.02. For permanent applications, the performance test anchor is sacrificial.

In the compact to very dense subgrade below founding elevation, post-grouted micropile ground anchors in tension can be designed using a maximum factored geotechnical resistance at ULS of 60 to 80 kN/m of adhered anchor length (at a nominal diameter of 150 mm). This capacity is provided assuming that a site specific tension load test is performed, implying a resistance factor of 0.6. Following the load test, the micropile capacity can be re-evaluated and potentially improved.

In the Georgian Bay Formation below founding elevation, conventional micropile anchors (at a nominal borehole diameter of 115 mm) can be designed using an ultimate resistance of 1,800 kPa. Assuming that a site specific tension load test is performed, a resistance factor of 0.65 is applicable. The maximum factored geotechnical resistance at ULS is taken as the ultimate resistance multiplied by a resistance factor. Following the load test, the micropile capacity can be re-evaluated and potentially improved.

Anchors will not engage the soil and rock simultaneously, due to strain incompatibility.

After installation, 5% of the permanent ground anchors must be proof tested to not less than 160% of ULS design load, per OPSS 942.07.12.05.02. After proof testing, the anchor is unloaded.

Micropile anchors are made with high-strength hot-rolled threadbar conforming to ASTM A615 or CSA G30.18. For permanent installations they should be made within grouted HDPE corrugated sheaths to provide “double corrosion protection”. Industry-standard grout cover may be used as a corrosion protection mechanism, subject to a review of the corrosivity and sulphate attack data.

Helical pile anchors are also feasible, subject to consultation from the design-build contractor. The project geotechnical information should be provided to a specialist design/build contractor to assess the feasibility of this foundation system and to determine probable helical pile refusal/installation depths. Adequate corrosion protection must be provided.

Tiedown anchors act in side shear, and should therefore be separated from each other by at least 5 borehole diameters (measured on centres) to avoid inducing a capacity reduction due to group effect.

In addition to designing the anchors for grout-soil adhesion capacity, global stability must also be checked. Tie-down anchors must also be designed to a depth sufficient to engage the necessary bulk unit weight of soil and/or rock. Soil anchors are made to engage a 30-45 degree cone of soil per anchor, measured from vertical[9]. Rock anchors are made to engage a 30-45 degree cone of rock per anchor, measured from vertical[10]. The anchor spacing and overlapping zones of influence must be considered. A typical detail is appended.

3.1.5 Caissons End-Bearing in Bedrock

End-bearing caissons may be used to support the proposed structure. End-bearing caissons made to bear on unweathered (sound) bedrock may be designed using a maximum factored geotechnical resistance at ULS of 12 MPa. The geotechnical reaction at SLS is 10 MPa, for up to 20 mm of estimated settlement at pile tip elevation, for individual caissons no larger than 2 m diameter and not subject to group effects.

It should be noted that rock coring to confirm the elevation of sound bedrock was not conducted within the subject portion of the site. If caissons are to be considered for detailed design, additional boreholes extending to bedrock and including rock coring and rock mechanics laboratory testing are required within the tower footprint to confirm the elevation and quality of rock, which then will be used to update the rock caisson capacity provided above.

In addition to the displacement of the rock, there will be compression of the concrete caisson shaft under loading which will increase the apparent settlement at the structure level. Caisson shaft compression must be assessed by the structural engineer.

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[9] FHWA. “Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems.” Publication No. FHWA-IF-99-015, June 1999, Figure 54.

[10] FHWA. “Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems.” Publication No. FHWA-IF-99-015, June 1999, Figure 53.

Caissons should be separated from each other by at least 2 times the largest caisson diameter (2D, measured on centres) to avoid inducing additional settlement from group effect. Caissons placed closer than this will induce group effects, and a reduced bearing capacity will apply, which is dependent on caisson sizing, bearing stratum, founding elevation, and separation distance. If this situation is unavoidable from a structural engineering perspective, Grounded should be retained to review the structural drawings and estimate the expected settlement of the caisson group.

If caissons are to be used to resist uplift as well (due to buoyancy, see below), they must be separated by 5D.

The soil at this site are cohesionless, permeable and wet. There are zones of soil at this site that are cohesionless, permeable and wet. Augered boreholes for caissons will require temporary liners, polymer mud drilling techniques, tremie pour concrete, pre-advancing casing, or other means and methods as deemed necessary by the contractor to prevent groundwater inflow or loss of soil into the drill holes, disturbance to placed concrete, or similar issues. Concrete for caissons must be placed by tremie method where there is more than 300 mm of water or fluid at the base of the hole.

Grounded recommends sonic caliper testing (or equivalent) to confirm verticality and diameter. Grounded generally recommends carrying such tests on at least the first five (5) caissons of each diameter, and 10% of each caisson diameter thereafter. The structural engineer should specify the number of tests to verify the quality of the contractor’s installation.

To confirm concrete placement, thermal integrity profiling (TIP), crosshole logging, or other similar tests are recommended.

Grounded reserves the right to increase the testing frequency, subject to the results of the initial testing.

Caissons in Uplift

If a watertight basement structure is required at this site and if the hydrostatic uplift loading is excessive (e.g. for podiums), the design approach may require the caissons to act as anchors in uplift.

Caissons designed to resist uplift are made typically deeper in the rock to engage a larger cone of rock per caisson. As uplift caissons act in side shear only, they must be separate at least 5D to avoid group effects. In addition to designing the caissons for a depth sufficient to engage the necessary weight of rock (see below), caisson depths are also sized for adhesion based on a bond between concrete and rock of 550 kPa factored ULS (using a deep foundations tension reduction factor of 0.3, assuming that a performance test is not conducted). If an uplift performance test is conducted, the ultimate adhesion (anticipated at 1800 kPa or higher) may be factored by 0.6 to achieve the design ULS adhesion. Caissons designed in uplift require a minimum 3 m embedment into sound rock.

In addition to designing the anchors for grout-rock adhesion capacity, global stability must also be checked. Uplift caissons must also be designed to a depth sufficient to engage the necessary bulk unit weight of soil and/or rock. Rock anchors are made to engage a 30-45 degree cone of rock per anchor, measured from vertical[11]. The anchor spacing and overlapping zones of influence must be considered. A typical detail is appended.

3.1.6 Alternative Foundation Options

In addition to the above foundation options, other alternative foundation options may also be considered for the support of the proposed structure. Helical piles torqued into the sand and silt unit or micropiles acting in compression bonded to the sand and silt unit or conventional micropiles bearing on bedrock may be considered. Grounded can provide additional details recommendations if these are the preferred foundation options for this proposed development.

3.2 Seismic Site Designation

The Ontario Building Code (2024) stipulates the methodology for earthquake load and effects analysis and design, as set out in Subsection 4.1.8. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration, and the site classification.

The site designation, X, is determined using the average shear wave velocity, V_{s30}, calculated from in situ measurements of shear wave velocity, in accordance with ground profiles provided in Table 4.1.8.4.-A. For all other ground profiles, the site designation is X_V, where V is the value of V_{s30}. At sites where V_{s30} is not available, the site designation is X_S, where S is the Site Class as determined from rational analysis of average undrained shear strength (s_u) or energy-corrected average standard penetration resistance (SPT N-values) in accordance with Table 4.1.8.4.-B.

The structural commentaries to the NBC 2020, on which the OBC 2024 are based, have been recently released. Based on the structural commentaries, site designation must be evaluated in the top 30 m of site stratigraphy.

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[11] FHWA. “Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems.” Publication No. FHWA-IF-99-015, June 1999, Figure 53.

Multichannel Analysis of Surface Waves (MASW) was performed at this site to determine the average shear wave velocity in the 30 metres of site stratigraphy (V_{s30}). The reported results are appended. An average V_{s30} value of 529 m/s was assessed from grade. Based on the measured shear wave velocities, the site designation for seismic analysis is $\mathbf{X_{529}}$.

3.3 Earth Pressure Design Parameters

At this site, the design parameters for structures subject to unbalanced earth pressures such as basement walls and retaining walls are shown in the table below.

Stratigraphic Unit	**γ**	**φ**	**K_a**	**K_o**	**K_p**
Compact Granular Fill Granular 'B' (OPSS.MUNI 1010)	21	32	0.31	0.47	3.25
Existing Earth Fill	19	29	0.35	0.52	2.88
Glacial Till Unit, Elev. 148± to 139± m as applicable	21	36	0.26	0.41	3.85
Sand and Silt Unit, Elev. 139± to 129± m as applicable	21	38	0.24	0.38	4.2
Inferred Bedrock	26	28	n/a		

γ = soil bulk unit weight (kN/m^3)
φ = internal friction angle (degrees)
K_a = active earth pressure coefficient (Rankine, dimensionless)
K_o = at-rest earth pressure coefficient (Rankine, dimensionless)
K_p = passive earth pressure coefficient (Rankine, dimensionless)

These earth pressure parameters assume that grade is horizontal behind the retaining structure. If retained grade is inclined, these parameters do not apply and must be re-evaluated.

The following equation can be used to calculate the unbalanced earth pressure imposed on walls:

$$P = K[\gamma(h - h_w) + \gamma' h_w + q] + \gamma_w h_w$$

P = horizontal pressure (kPa) at depth h
h = the depth at which P is calculated (m)
K = earth pressure coefficient
h_w = height of groundwater (m) above depth h
γ = soil bulk unit weight (kN/m^3)
γ' = submerged soil unit weight (γ - 9.8 kN/m^3)
q = total surcharge load (kPa)

If the wall backfill is drained such that hydrostatic pressures on the wall are effectively eliminated, this equation simplifies to:

$$P = K[\gamma h + q]$$

Where walls are made directly against shoring, prefabricated composite drainage panel covering the blind side of the wall is used to provide drainage. Water from the composite drainage panel is collected and discharged through the basement wall in solid ports directly to the sumps. This is discussed in Section 3.5.

If the underground structure is to be designed as a fully watertight structure, then, the full height of the basement walls should be watertight and designed to withstand horizontal hydrostatic pressure below Elev. 144.0± m.

The possible effects of frost on retaining earth structures must be considered. In frost-susceptible soils, pressures induced by freezing pore water are basically irresistible. Insulation typically addresses this issue. Alternatively, non-frost-susceptible backfill may be specified.

Foundation resistance to sliding is proportional to the friction between the subgrade and the base of the footing. The factored geotechnical resistance to friction (**R_f**) at ULS provided in the following equation:

$$R_f = \Phi N \tan\varphi$$

R_f = frictional resistance (kN)
Φ = reduction factor per CFEM 5th Ed. (0.8 for cohesionless soils or rock; 0.6 for cohesive soils)
N = normal load at base of footing (kN)
φ = internal friction angle (see table above)

3.4 Slab on Grade Design Parameters

3.4.1 Watertight Structure Approach

If the structure is designed as a fully watertight structure, it should be designed to withstand uplift and hydrostatic pressures, with no permanent drainage. The lowest floor will be made as a pressure slab spanning between foundation elements, to be designed by the structural engineer. If a fully watertight raft foundation approach is adopted (with no permanent drainage system), design parameters are provided in Section 3.1.

3.4.2 Drained Structure Approach

The slab-on-grade parameters provided here apply to a conventional slab on grade and drained basement approach only. At the proposed P2 elevation, the undisturbed native soils will provide adequate subgrade for the support of a conventional slab on grade. The modulus of subgrade reaction for slab-on-grade design supported by undisturbed native soils (glacial till) is 60,000 kPa/m.

If this basement structure is made as a conventional drained structure, a permanent drainage system including subfloor drains is required (see section below). In this case, the slab on grade must be provided with a drainage layer and capillary moisture break, which is achieved by forming the slab on a minimum 300 mm thick layer of 9.5 mm clear stone (OPSS.MUNI 1004), High Performance Bedding (HPB), or approved equivalent vibrated to a dense state.

If this basement structure is made as a conventional drained structure, a permanent drainage system including subfloor drains is required (see section below). In this case, the slab on grade must be provided with a drainage layer and capillary moisture break.

Due to the cohesionless subgrade, volume of seepage, and hydrostatic pressure anticipated, the drainage layer is to be made as follows:

- The drainage layer must be separated from the subgrade using a fine concrete sand filter overlain with a non-woven geotextile, which separates the clear stone from the fine concrete sand. In all areas where native soils are exposed and outside of foundation bearing areas, the clear stone drainage layer must be separated from the native subgrade using this makeup.
- The bulk excavation is subexcavated a min. 150 mm and replaced with fine concrete sand (OPSS.MUNI 1002 Table 1 “Fine Aggregate”, or CSA A23.1 (FA1)) compacted to 98% of SPMDD.
- A non-woven geotextile (with an apparent opening size of less than 0.075 mm and a tear resistance of more than 200 N) is to be placed on the surface of the compacted fine concrete sand.
- The subfloor drains are then laid directly on the flat subgrade and backfilled with a minimum 300 mm thick layer of 9.5 mm clear stone (OPSS.MUNI 1004), High Performance Bedding (HPB), or approved equivalent, vibrated to a dense state.
- Any solid collection pipes must be sloped so that they positively discharge to the sumps.

Without this filtering layer, fines from the underlying subgrade may enter the drainage layer potentially resulting in loss of ground, loss of slab support, and clogging of the subfloor drainage system.

Given the nature of the soils at this site, recompaction or proof rolling of the undisturbed native subgrade will weaken these materials. These activities should be specifically prohibited when preparing native subgrade. The subgrade should be cut neat and inspected by Grounded prior to placement of the capillary moisture break and construction of the slab. Disturbed or otherwise unacceptable material (as determined by Grounded) must be subexcavated and replaced with Granular B (OPSS.MUNI 1010) compacted to a minimum of 98% SPMDD. The slab on grade should not be placed on frozen subgrade, to prevent excessive settlement of the slab as the subgrade thaws. Areas of frozen subgrade should be removed during subgrade preparation.

3.5 Long-Term Groundwater and Seepage Control

To limit seepage to the extent practicable, exterior grades adjacent to foundation walls should be sloped at a minimum 2 percent gradient away from the wall for 1.2 m minimum.

The requirement for a permanent basement drainage system depends on whether a fully watertight approach is adopted for this site. Grounded’s Hydrogeological Report (File No. 25-122) provides further discussion on this.

A watertight basement implies that the basement structure is designed to withstand hydrostatic pressures, with no permanent drainage system. The full height of the basement walls should be watertight (no drainage) and designed to withstand hydrostatic pressure (horizontal and uplift) using a static groundwater table at Elev. 144.0± m. A connection to the City’s sewer for emergency repair services is recommended.

The following discussion pertains to a drained basement approach only.

For a conventional drained basement approach, perimeter and subfloor drainage systems are required for the underground structure. Subfloor drainage systems collect and remove the seepage that infiltrates under the floor. Perimeter drainage systems collect and remove seepage or stormwater that infiltrates at the foundation walls. Perimeter drainage must be collected and conveyed directly to the building sumps, and not discharged into the subfloor drainage system, the granular layer, or beneath the floor slab.

Subfloor drainage pipes are to be spaced at a maximum 6 m (measured on-centres).

The walls of the substructure are to be fully drained to eliminate hydrostatic pressure. Where drained basement walls are made directly against shoring, prefabricated composite drainage panel covering the blind side of the wall is used to provide drainage. Seepage from the composite drainage panel is collected and discharged through the basement wall in solid ports directly to the sumps.

In an open cut excavation, basement wall drainage is installed directly against the basement wall from the open cut side. Perimeter foundation drains made in this application comprise perforated pipe (minimum 100 mm diameter) surrounded by a granular filter of OPSS.MUNI HL-8 Coarse Aggregate providing a minimum 300 mm of cover over the drain pipe.

Even with a fully drained below-grade structure, the relative humidity at the interface between the foundation wall and the soil/shoring system will still be 100%. A layer of waterproofing placed between the drainage layer and the foundation wall is recommended to protect interior finishes and reinforcing steel from moisture. The building science engineer should confirm this and can provide further advice, as well as specifications for waterproofing products.

Typical basement drainage details are appended.

The perimeter and subfloor drainage systems are critical structural elements since they eliminate hydrostatic pressure from acting on the basement walls and floor slab. The sumps that ensure the performance of these systems must have a duplexed pump arrangement providing 100% redundancy, and they must be on emergency power. The sumps should be sized by the mechanical engineer to adequately accommodate the estimated volume of water seepage.

The permanent dewatering requirements for the subject portion of the site are provided in Grounded's (site specific) Hydrogeological Report (File No. 25-122) under separate cover.

If any water is to be discharged to the storm or sanitary sewers, the City will require Discharge Agreements to be in place.

3.6 Site Servicing

All services must have at least 1.2 metres of earth cover or equivalent insulation for frost protection.

Where site services extend beyond the building footprint, the following recommendations apply.

3.6.1 Bedding

The soil subgrade encountered within the proposed site servicing trenches will consist of either earth fill or native soil. The trench base must be inspected for obvious loose, wet, or disturbed material. Any unsuitable material must be subexcavated and replaced with imported fill compacted to 98% SPMDD. If suitable earth fill is encountered, the subgrade must be compacted in place to a minimum 98% SPMDD.

Site servicing drawings are not available for review. It is assumed that trenches will be made at least 1.2 m above the groundwater table.

If trenches extend below the groundwater table, dewatering should be considered to lower the groundwater table below the lowest trench invert prior to excavation. At this site, the subgrade soils are moderately permeable and may yield free-flowing water when penetrated below the groundwater table. Positive dewatering prior to trench excavation will be required.

Test pits may be advanced as part of a future investigation scope of work to better observe seepage at trench invert elevation, to inform or confirm dewatering requirements. Test pits should be left open for 24h, with any observed seepage recorded. Grounded can provide these services on request.

Above the groundwater table, bedding material may consist of 19 mm clear stone (OPSS.MUNI 1004) or similar, vibrated to a dense state. Where the bedding material consists of clear stone, the bedding must be separated from the subgrade with a non-woven geotextile. Alternatively, a well graded granular fill such as Granular A (OPSS.MUNI 1010) compacted to 98% SPMDD may be considered.

3.6.2 Backfill

Excavated earth fill and native soils on site will constitute adequate backfill material if the soil meets the following backfill specifications:

- Any deleterious material in the earth fill is removed prior to reuse as backfill.
- Backfill materials are not frozen.
- The moisture content is within 2% of optimum, or moisture conditioned to within 2% of optimum.
- The backfill must be compacted to a minimum 98% SPMDD.

Excavated shale material is **not** a suitable material for trench backfill. The shale cannot be broken down and effectively compacted. Reused shale will slake and degrade with time, causing settlement or heave.

4 Pavement Engineering Recommendations

4.1 Asphalt Pavements Above Underground Parking Structure

It is expected that most of the pavements will be placed on top of the reinforced concrete parking structure and not on soil subgrade. In this case, the pavements resting on parking structure should consist of the following:

Component	Compaction Requirement	Pavement on Concrete Parking Structure Minimum Component Thickness
Asphalt Top Lift HL-3 (OPSS.MUNI 1150), and PG 58-28 (OPSS.MUNI 1101)	OPSS.MUNI 310	40 mm
Asphalt Base Course HL-8 (OPSS.MUNI 1150), and PG 58-28 (OPSS.MUNI 1101)	OPSS.MUNI 310	50 mm
Granular Base Course Granular A (OPSS.MUNI 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm

Component	Compaction Requirement	Pavement on Concrete Parking Structure Minimum Component Thickness
Total Thickness		**240 mm**

A waterproof membrane will be required between the asphalt and concrete parking structure deck. For pavements placed on top of the underground parking structure, all drainage, waterproofing, and protection considerations for these areas must be designed separately and in conjunction with the civil engineering design of the underground parking structure. Wherever they have to connect to the adjacent roadways or driveways, those adjacent pavement profiles will be different and so taper transitions and run-outs must be designed for the connections.

The design presented below is only for areas in which the pavements will rest on a soil subgrade.

4.2 Asphalt Pavements on Subgrade Soils

The following design pertains to asphaltic concrete pavements ('pavement') where the pavement will rest on a soil subgrade as described above.

The following Ontario Provincial Standards Specifications (OPSS.MUNI) apply to the pavement construction and material requirements:

- OPSS.MUNI 310 - Hot Mix Asphalt
- OPSS.MUNI 501 - Compacting
- OPSS.MUNI 1010 - Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
- OPSS.MUNI 1101 - Performance Graded Asphalt Cement
- OPSS.MUNI 1150 - Hot Mix Asphalt

The pavement construction and material should also follow the relevant city specifications, as applicable.

4.2.1 Pavement Subgrade Preparation

The subgrade must be adequately prepared prior to pavement construction.

Topsoil and existing wet or organic-rich earth fill soils are considered unsuitable for the pavement subgrade. These materials must be stripped down to acceptable subgrade prior to pavement construction.

Existing earth fill (free of organic-rich or wet soils) and native subgrade will provide adequate subgrade for the support of the pavement. The subgrade must be proof-rolled and inspected under the supervision of Grounded for obvious loose or disturbed soils or where there is deleterious materials or moisture. These areas can either be recompacted in place and retested, or replaced with Granular B (OPSS.MUNI 1010) in 150 mm thick lifts, compacted to a minimum of 98% SPMDD.

The existing subgrade may not be readily compacted in small volumes, such as trenches or in areas adjacent to foundations or catch basins. For areas of limited extent, compactable aggregate-source backfills like Granular B (OPSS.MUNI 1010) are recommended for post-construction grade integrity. All new fill shall be compacted to a minimum of 98% SPMDD.

The subgrade for all pavement structures shall be frost tapered at a 3H to 1V slope (or flatter) to match with existing pavement structures, to reduce differential settlements due to frost heave.

4.2.2 Asphalt Pavement Design

Minimum and performance asphaltic concrete pavement designs are outlined in the tables below.

The following **basic pavement design** will last for 8 to 10 years before significant maintenance is required, depending on the traffic volume.

Basic Pavement Structure	Compaction Requirement	Car Parking Minimum Component Thickness	Bus/Truck Traffic Minimum Component Thickness
Asphalt Top Lift HL-3 (OPSS.MUNI 1150), and PG 58-28 (OPSS.MUNI 1101)	OPSS.MUNI 310	65 mm	40 mm
Asphalt Base Course HL-8 (OPSS.MUNI 1150), and PG 58-28 (OPSS.MUNI 1101)	OPSS.MUNI 310	N/A	50 mm
Granular Base Course 19 mm diameter crusher run limestone or Granular A (OPSS.MUNI 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Granular Subbase Course 50 mm diameter crusher run limestone or Granular B Type II (OPSS.MUNI 1010)	98% Standard Proctor Maximum Dry Density (ASTM-D698)	300 mm	400 mm
Total Thickness		**515 mm**	**640 mm**

The following **performance pavement design** will last approximately twice as long before significant maintenance is required. The performance pavement design considers that the top layer of asphalt will be damaged over time, and therefore, will contribute less to the structural strength of the asphalt.

<u>Performance</u> Pavement Structure	Compaction Requirement	Car Parking Minimum Component Thickness	Bus/Truck Traffic Minimum Component Thickness
Asphalt Top Lift HL-3 (OPSS.MUNI 1150), and PG 58-28 (OPSS.MUNI 1101)	OPSS.MUNI 310	40 mm	40 mm
Asphalt Base Course HL-8 (OPSS.MUNI 1150), and PG 58-28 (OPSS.MUNI 1101)	OPSS.MUNI 310	50 mm	80 mm
Granular Base Course 19 mm diameter crusher run limestone or Granular A (OPSS.MUNI 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Granular Subbase Course 50 mm diameter crusher run limestone or Granular B Type II (OPSS.MUNI 1010)	98% Standard Proctor Maximum Dry Density (ASTM-D698)	400 mm	500 mm
Total Thickness		**640 mm**	**770 mm**

The existing subgrade soils have a low to moderate susceptibility to frost heave, and pavement on these materials must be designed accordingly. To reduce frost heave, soil subgrade that is susceptible to frost (as defined in Section 2.4) should be replaced to a depth of 70 percent of the frost penetration depth (i.e. 0.85± m below proposed top of pavement) with non-frost susceptible soils or with granular materials. The most effective ways of dealing with potential frost heave are to construct a good subsurface drainage system, and to stay above the groundwater table.

4.2.3 Pavement Drainage

Adequate drainage of the pavement subgrade is required. Prior to paving, the subgrade should be free of any depressions and sloped at a minimum grade of 2% to provide positive drainage. Perforated plastic subdrains (100 mm diameter) should be designed to collect subgrade water and positively outlet it at the catch basins. Typical pavement drainage details are appended.

Controlling surface water is important in keeping pavements in good maintenance. Grading adjacent pavement areas must be designed so that water is not allowed to pond adjacent to the outside edges of the pavement or curb.

5 Considerations for Construction

5.1 Excavations

Excavations must be carried out in accordance with the *Occupational Health and Safety Act – Regulation 213/91 – Construction Projects (Part III - Excavations, Section 222 through 242)*. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. For practical purposes:

- The earth fill is a Type 3 soil
- The native soils are Type 4 soils if wet or below the groundwater table, and can be considered Type 3 soils if dewatered or above the groundwater table

In accordance with the regulation's requirements, the soil must be suitably sloped and/or braced where workers must enter a trench or excavation deeper than 1.2 m. Safe excavation slopes (of no more than 3 m in height) by soil type are stipulated as follows, per Section 234:

Soil Type	Base of Slope	Steepest Slope Inclination
1	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
2	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
3	from bottom of trench	1 horizontal to 1 vertical
4	from bottom of trench	3 horizontal to 1 vertical

Minimum support system requirements for steeper excavations are stipulated in Sections 235 through 239 and 241 of the Act and Regulations and include provisions for timbering, shoring and moveable trench boxes. Any excavation slopes greater than 3 m in height should be checked by Grounded for global stability issues.

Larger obstructions (e.g. buried concrete debris, other obstructions) not directly observed in the boreholes are likely present in the earth fill. Similarly, larger inclusions (e.g. cobbles and boulders) may be encountered in the native soils. The size and distribution of these obstructions cannot be predicted with boreholes, as the split spoon sampler is not large enough to capture particles of this size. Provision must be made in excavation contracts to allocate risks associated with the time spent and equipment utilized to remove or penetrate such obstructions when encountered.

Excess soil is governed by Ontario Regulation 406/19: On-Site and Excess Soil Management (ESM). The Project Leader (typically the owner) may be required to file a notice in the excess soil registry and a Qualified Person (within the meaning of O.Reg. 153/04) may be required to prepare the associated planning documents and/or develop and implement a tracking system in accordance with the Soil Rules, to track each load of excess soil during its transportation and deposit before removing excess soil from the project area.

5.1.1 Underpinning Existing Footings

Excavation for the proposed development will be close to and extend below the base of the existing P1 parking level of the existing apartment building. Excavation without providing adequate support to the existing parking level will undermine the existing structure. This can likely be achieved by the use of a rigid shoring (caisson wall) along the west excavation wall (adjacent to the existing P1 underground structure) as discussed in Section 5.3 or by deepening the existing foundations to bear at deeper stratums using either helical piles or micropiles. Grounded can provide additional geotechnical recommendations during the detailed design stage.

5.2 Short-Term Groundwater Control

Considerations pertaining to groundwater discharge quantities and quality are discussed in Grounded's hydrogeological report for the site, under separate cover.

The groundwater table at Elev. 144.0± m is above the bulk excavation level for P2. Positive dewatering to lower the groundwater table will be required to facilitate construction as well as to maintain the integrity of the subgrade for foundation and slab-on-grade support. Dewatering will take some time to accomplish prior to the start of excavation. The water level must be kept at least 1.2 m below the lowest excavation elevation during construction. Failure to dewater prior to excavation will result in unrecoverable disturbance of the subgrade, which will render advice provided for undisturbed subgrade conditions inapplicable.

A professional dewatering contractor should be consulted to review the subsurface conditions and to design a site-specific dewatering system. It is the dewatering contractor's responsibility to assess the factual data and to provide recommendations on dewatering system requirements.

If a watertight basement approach is adopted, it will be necessary to consider the potential uplift pressure on the underside of a watertight foundation due to hydrostatic forces during construction. Positive dewatering operations during construction must begin prior to excavation and must continue until such time as the structural dead load exceeds the potential uplift forces (with suitable partial factors (LRFD) included in this assessment). A design groundwater elevation of 144.0± m is to be used in this assessment.

Should the excavation be supported using permeable soldier pile and lagging shoring, positive dewatering will be required on a continuous ongoing basis during excavation and throughout construction.

5.3 Earth-Retention Shoring Systems

No excavation shall extend below the foundations of existing adjacent structures without adequate alternative support being provided. Excavation zone of influence guidelines are appended.

Continuous interlocking caisson wall shoring is to be used where the excavation must be constructed as a rigid shoring system. Caisson wall shoring preserves the support capabilities and integrity of the soil beneath existing foundations of adjacent buildings, in a state akin to the at-rest condition. Otherwise, excavations can be supported using conventional soldier pile and lagging walls with active dewatering prior to and during construction.

5.3.1 Lateral Earth Pressure Distribution

If the shoring is supported with a single level of earth anchor or bracing, a triangular earth pressure distribution like that used for the basement wall design is appropriate.

Where multiple rows of lateral supports are used to support the shoring walls, research has shown that a distributed pressure diagram more realistically approximates the earth pressure on a shoring system of this type, when restrained by pre-tensioned anchors. A multi-level supported shoring system can be designed based on an earth pressure distribution with a maximum pressure defined by:

$$\boldsymbol{P = 0.65\ K[\gamma H + q] + \gamma_w h_w}$$ … in cohesionless soils

P = maximum horizontal pressure (kPa)
K = earth pressure coefficient (see Section 3.3)
H = total depth of the excavation (m)
h_w = height of groundwater (m) above the base of excavation
γ = soil bulk unit weight (kN/m3)
q = total surcharge loading (kPa)

Where shoring walls are drained to effectively eliminate hydrostatic pressure on the shoring system (e.g. pile and lagging walls), $\boldsymbol{h_w}$ is equal to zero. For the design of impermeable shoring, a design groundwater table at Elev. 144.0± m must be accounted for. There is infiltrated stormwater perched in the earth fill and upper native soils which may accumulate behind a caisson wall. This hydrostatic pressure needs to be accounted for in shoring design. In cohesionless soils, the lateral earth pressure distribution is rectangular.

5.3.2 Soldier Pile Toe Embedment

Soldier pile toes will be made in very dense sand and silt unit or in the bedrock. Soldier pile toes resist horizontal movement due to the passive earth pressure acting on the toe below the base of excavation.

The subgrade soils at this site are cohesionless, wet, and permeable. Augered holes for piles made into these soils will be prone to caving and blowback. Temporarily cased holes advanced to the bedrock surface are required to prevent borehole caving during installations in drilled holes. To prevent groundwater issues (groundwater inflow, caving and blowback into the drill holes, disturbance to placed concrete, etc.) during drilling and installation, construction methods such as utilizing temporary liners, pre-advancing liners deeper than the augered holes, mud/slurry/polymer drilling techniques, tremie pour concrete, or other methods as deemed necessary by the shoring contractor are required. Concrete for shoring piles and fillers must be placed by tremie method wherever there is more than 300 mm of water or fluid at the base of the drill hole.

5.3.3 Lateral Bracing Elements

The shoring system at this site will require lateral bracing. If feasible, the shoring system should be supported by pre-stressed soil anchors (tiebacks) extending into the subgrade of the adjacent properties. To limit the movement of the shoring system as much as is practically possible, tiebacks are installed and stressed as excavation proceeds. The use of tiebacks through adjacent properties requires the consent (through encroachment agreements) of the adjacent property owners.

Anchors can be designed to react in either the overburden native soils, or the rock. They will not engage the soil and rock simultaneously, due to strain incompatibility.

In the very dense subgrade, post-grouted micropile ground anchors in tension can be designed using a maximum factored geotechnical resistance at ULS of 80 to 100 kN/m of adhered anchor length (at a nominal diameter of 150 mm). If the anchors are to be made in the stiff to hard plastic till, they can be designed using a maximum factored geotechnical resistance at ULS of 40-60 kN/m of adhered anchor length (at a nominal diameter of 150 mm). These capacities are provided assuming that site-specific tension load tests will be performed, implying a resistance factor of 0.6.

If anchors in the gravelly sand are required, the gravelly sand may need to be dewatered to enable tieback installation to proceed.

In the Georgian Bay Formation, conventional micropile anchors (at a nominal borehole diameter of 115 mm) can be designed using an ultimate resistance of 1,800 kPa. Assuming that a site specific tension load test is performed, a resistance factor of 0.65 is applicable. The maximum factored geotechnical resistance at ULS is taken as the ultimate resistance multiplied by a resistance factor. Following the load test, the micropile capacity can be re-evaluated and potentially improved.

Production tiebacks require a minimum 3 m socket length.

At least one prototype anchor per tieback level must be performance-tested to 200% of the design load to demonstrate the anchor capacity and validate design assumptions. For temporary applications, the performance test anchor may be used as a production anchor.

Every production anchor must be proof tested to 133% of design load, and then locked in at 100% of design load. For permanent applications, they must be proof tested to 150% of design load, and then locked in at 110% of design load.

The very dense / very stiff to hard till and the very dense sand and silt unit below the proposed FFE is suitable for the placement of raker foundations. Raker footings established at an inclination of 45 degrees can be designed for a maximum factored geotechnical resistance at ULS of 350 kPa if bearing on very dense till / very dense sands and silts.

5.4 Site Work

To better protect wet undisturbed subgrade, excavations exposing wet soils must be cut neat, inspected, and then immediately protected with a skim coat of concrete (i.e. a mud mat). Wet sands are susceptible to degradation and disturbance due to even mild site work, frost, weather, or a combination thereof.

The effects of work on site can greatly impact soil integrity. Care must be taken to prevent this damage. Site work carried out during periods of inclement weather may result in the subgrade becoming disturbed, unless a granular working mat is placed to preserve the subgrade soils in their undisturbed condition. Subgrade preparation activities should not be conducted in wet weather and the project must be scheduled accordingly.

If site work causes disturbance to the subgrade, removal of the disturbed soils and the use of granular fill material for site restoration or underfloor fill will be required at additional cost to the project.

It is construction activity itself that often imparts the most severe loading conditions on the subgrade. Special provisions such as end dumping and forward spreading of earth and aggregate fills, restricted construction lanes, and half-loads during placement of the granular base and other work may be required, especially if construction is carried out during unfavourable weather.

Adequate temporary frost protection for the founding subgrade must be provided if construction proceeds in freezing weather conditions. The subgrade at this site is susceptible to frost damage. The slab on grade should not be placed on frozen subgrade, to prevent excess settlement of the slab as the subgrade thaws. Areas of frozen subgrade should be removed during subgrade preparation. Depending on the project context, consideration should be given to frost effects (heaving, softening, etc.) on exposed subgrade surfaces.

5.5 Engineering Review

By issuing this report, Grounded Engineering has assumed the role of Geotechnical Engineer of Record for this site. Grounded should be retained by the Owner to review the structural and geostructural engineering drawings prior to issue or construction, to ensure that the recommendations in this report have been appropriately implemented.

All foundation installations must be reviewed in the field by Grounded, the Geotechnical Engineer of Record, as they are constructed. The on-site review of foundation installations and the condition of the founding subgrade as the foundations are constructed is as much a part of the geotechnical engineering design function as the design itself; it is also required by Section 4.2.2.3. of the 2024 Ontario Building Code. If Grounded is not retained to carry out foundation engineering field review during construction, then Grounded accepts no responsibility for the performance or non-performance of the foundations, even if they are constructed in general conformance with the engineering design advice contained in this report.

Strict procedures must be maintained during construction to maintain the integrity of the subgrade to the extent possible. The design advice in this report is based on an assessment of the subgrade support capabilities as indicated by the boreholes. These conditions may vary across the site depending on the final design grades and therefore, the preparation of the subgrade should be monitored by Grounded at the time of construction to confirm material quality, and thickness.

A visual pre-construction survey of adjacent lands and buildings is recommended to be completed prior to the start of any construction. This documents the baseline condition and can prevent unwarranted damage claims. Any shoring system, regardless of the execution and design, has the potential for movement. Small changes in stress or soil volume can cause cracking in adjacent buildings.

6 Limitations and Restrictions

The current investigation was designed to provide a range of foundation options for the proposed development but wasn't designed to provide detailed foundation recommendations for each foundation option. Once a foundation strategy has been selected (raft vs caissons vs other alternative foundation options), additional subsurface investigation to characterize the elastic modulus of the soils and/or the top of sound bedrock may be required. This geotechnical engineering report therefore provides only preliminary recommendations for the following elements:

- Raft foundation design (based on assumed UDL, detailed raft analysis can not yet completed without structural engineering input at detailed design stage)
- Caisson design (rock coring was not conducted in this area of the site, additional boreholes with rock coring are required to carry this option to detailed design)
- Alternative foundation options: Helical Piles, Micropiles (if required, Grounded can provide recommendations for these options and required additional scopes of work, if necessary)

At detailed design, depending on the chosen foundation option, additional boreholes (possibly with rock coring), and updated detailed geotechnical engineering advice may be required. Once completed, a future geotechnical engineering report by Grounded Engineering would then supersede this report. Note that preliminary findings can vary significantly from the findings of a detailed comprehensive study.

The detailed design of a raft foundation is typically carried out in conjunction with the structural engineer at the detailed design stage, as discussed in Section 3.1.3 of this report. This may not require additional subsurface investigation.

6.1 Investigation Procedures

The geotechnical engineering analysis and advice provided are based on the factual borehole information observed and recorded by Grounded. The investigation methodology and engineering analysis methods used to carry out this scope of work are consistent with Grounded’s standard of practice as well as other reasonable and prudent geotechnical consultants, working under similar conditions and constraints (time, financial, and physical).

Borehole drilling services were provided to Grounded by a specialist professional contractor. The drilling was observed and recorded by Grounded’s field supervisor on a full-time basis. Drilling was conducted using conventional drilling rigs equipped with hollow stem augers and mud rotary drilling equipment. Rock coring was carried out with HQ size diamond bit core drilling barrels. As drilling proceeded, groundwater observations were made in the boreholes. Based on examination of recovered borehole samples, our field supervisor made a record of borehole and drilling observations. The field samples were secured in air-tight clean jars and bags and taken to the Grounded soil laboratory where they were each logged and reviewed by the geotechnical engineering team and the senior reviewer.

The Split-Barrel Method technique (ASTM D1586) was used to obtain the soils samples. The sampling was conducted at conventional intervals and not continuously. As such, stratigraphic interpolation between samples is required and stratigraphic boundary lines do not represent exact depths of geological change. They should be taken as gradual transition zones between soil or rock types.

A carefully conducted, fully comprehensive investigation and sampling scope of work carried out under the most stringent level of oversight may still fail to detect certain ground conditions. As such, users of this report must be aware of the risks inherent in using engineered field investigations to observe and record subsurface conditions. As a necessary requirement of working with discrete test locations, Grounded has assumed that the conditions between test locations are the same as the test locations themselves, for the purposes of providing geotechnical engineering advice.

It is not possible to design a field investigation with enough test locations that would provide complete subsurface information, nor is it possible to provide geotechnical engineering advice that completely identifies or quantifies every element that could affect construction, scheduling, or tendering. Contractors undertaking work based on this report (in whole or in part) must make their own determination of how they may be affected by the subsurface conditions, based on their own analysis of the factual information provided and based on their own means and methods. Contractors using this report must be aware of the risks implicit in using factual information at discrete test locations to infer subsurface conditions across the site and are directed to conduct their own investigations as needed.

6.2 Site and Scope Changes

Natural occurrences, the passage of time, local construction, and other human activity all have the potential to directly or indirectly alter the subsurface conditions at or near the project site. Contractual obligations related to groundwater or stormwater control, disturbed soils, frost protection, etc. must be considered with attention and care as they relate to potential site alteration.

The geotechnical engineering advice provided in this report is based on the factual observations made from the site investigations as reported. It is intended for use by the owner and their retained design team. If there are changes to the features of the development or to the scope, the interpreted subsurface information, geotechnical engineering design parameters, advice, and discussion on construction considerations may not be relevant or complete for the project. Grounded should be retained to review the implications of such changes with respect to the contents of this report.

6.3 Report Use

The authorized users of this report are 1315 Bough Beeches Boulevard Limited and their design team, for whom this report has been prepared. Grounded Engineering Inc. maintains the copyright and ownership of this document. Reproduction of this report in any format or medium requires explicit prior authorization from Grounded Engineering Inc.

The City of Mississauga may also make use of and rely upon this report, subject to the limitations as stated.

The regional governing bodies may also make use of and rely upon this report, subject to the limitations as stated.

7 Closure

If the design team has any questions regarding the discussion and advice provided, please do not hesitate to have them contact our office. We trust that this report meets your requirements at present.

For and on behalf of our team,

GROUNDED
ENGINEERING

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Deepak Kanraj, M.A.Sc., P.Eng.
Project Engineer

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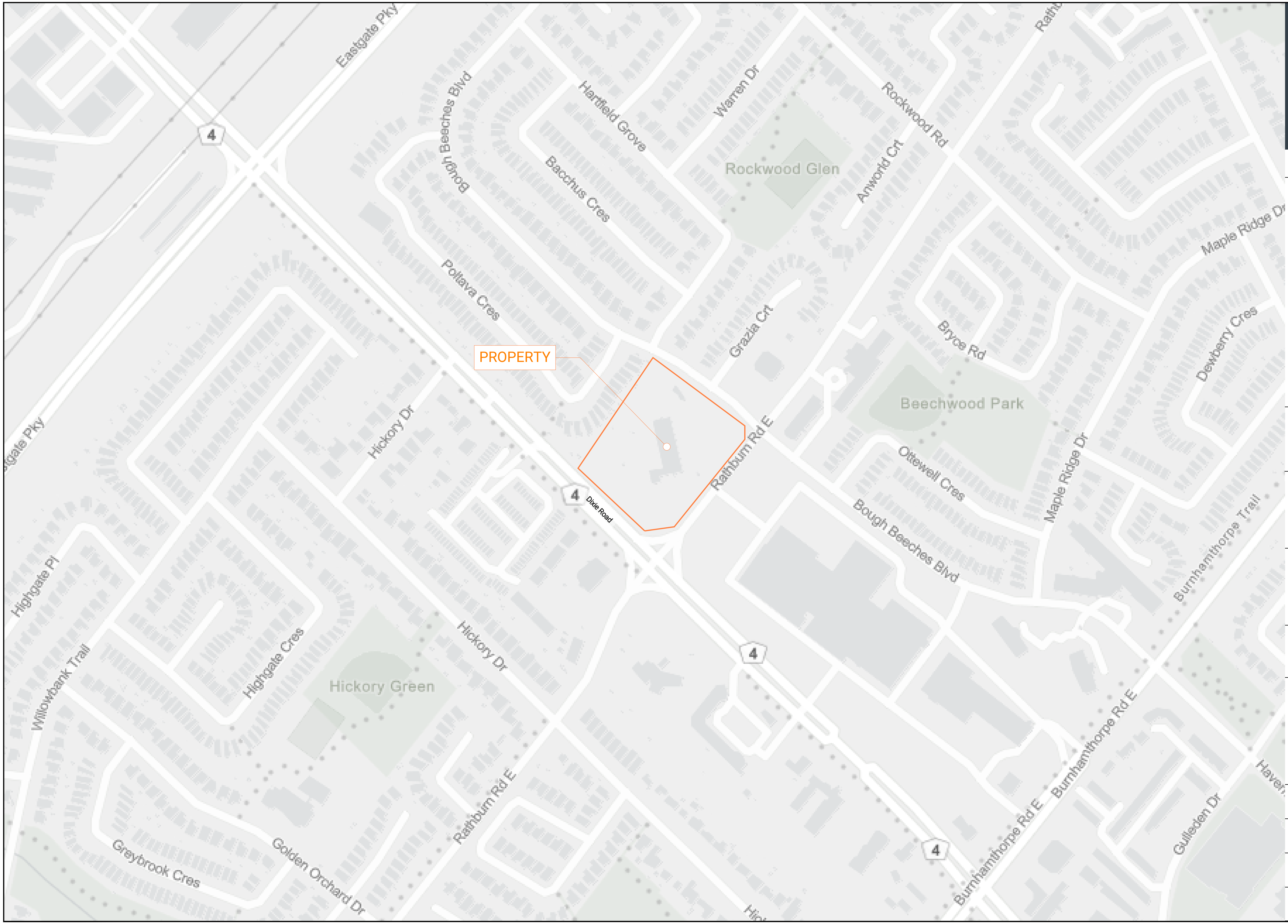
Associate

Jason Crowder, Ph.D., P.Eng.
Principal

FIGURES



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LEGEND

— APPRXIMATE PROPERTY BOUNDARY

Note

Reference

ArcGIS Online Maps

Project

1315 BOUGH BEECHES BLVD, MISSISSAUGA, ONTARIO

Figure Title

SITE LOCATION PLAN

North

TRUE

PROJECT

Date

MARCH 2026

Scale

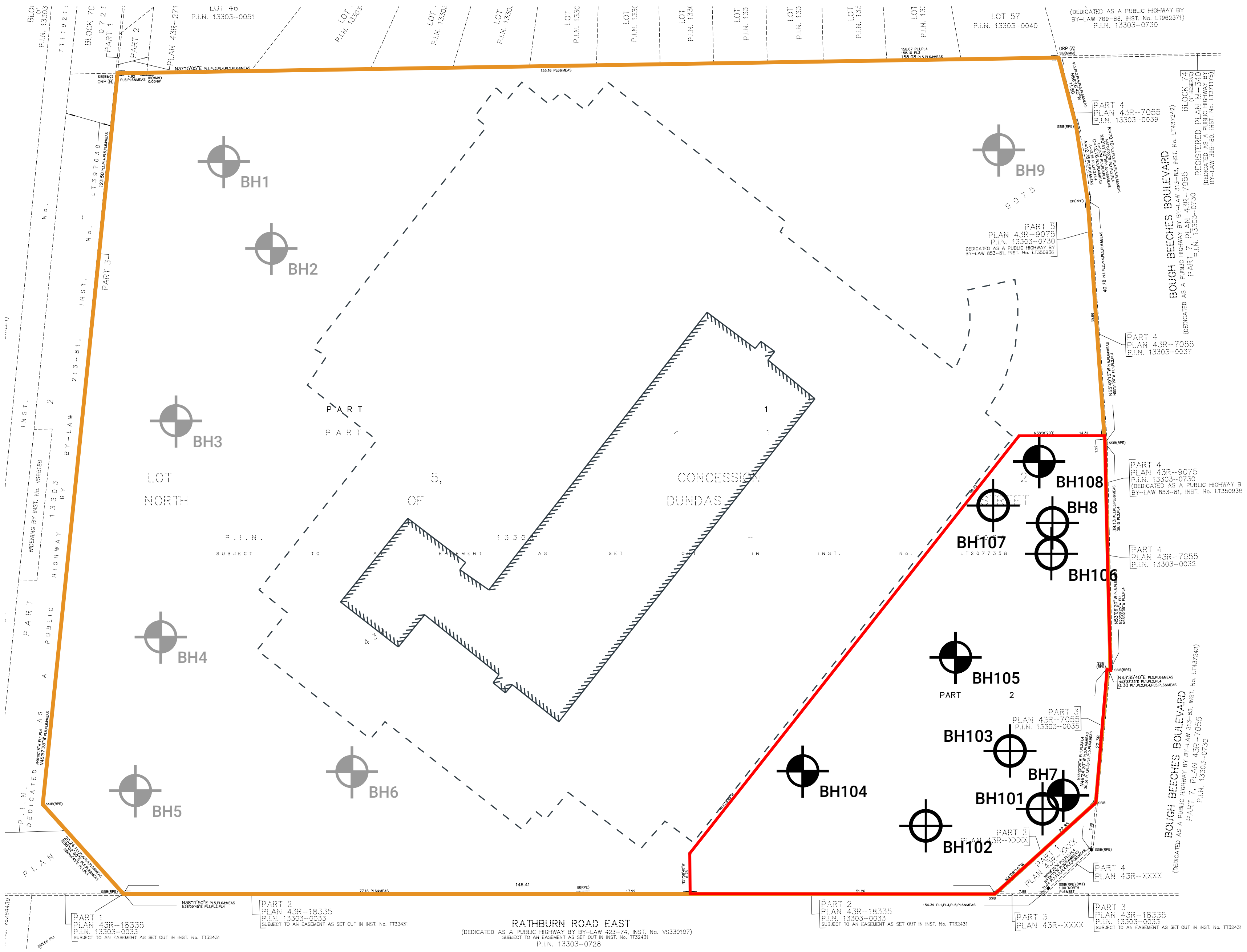
0m 75m 150m

Job No

25-122

Figure No

FIGURE 1



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LEGEND

🟧	APPRXIMATE PROPERTY BOUNDARY
🟥	APPRXIMATE EXTENT OF PROPOSED DEVELOPMENT
▨	EXISTING BUILDING
⬚	EXISTING UNDERGROUND GARAGE
⌖ ⌖	RELEVANT MONITORING WELL/BOREHOLE BY GROUNDED FOR PROPOSED DEVELOPMENT
⌖	MONITORING WELL/BOREHOLE BY GROUNDED PREVIOUSLY DRILLED AT OTHER PORTIONS OF THE PROPERTY

Note

Reference

Survey Drawing job no. 25-080
Dated July 03, 2025
Prepared by R-PE SURVEYING LTD.
Received on July 07, 2025.

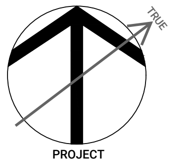
Project

1315 BOUGH BEECHES BLVD, MISSISSAUGA, ONTARIO

Figure Title

BOREHOLE AND MONITORING WELL LOCATION PLAN - EXISTING SITE CONDITION

North



Date

MARCH 2026

Scale

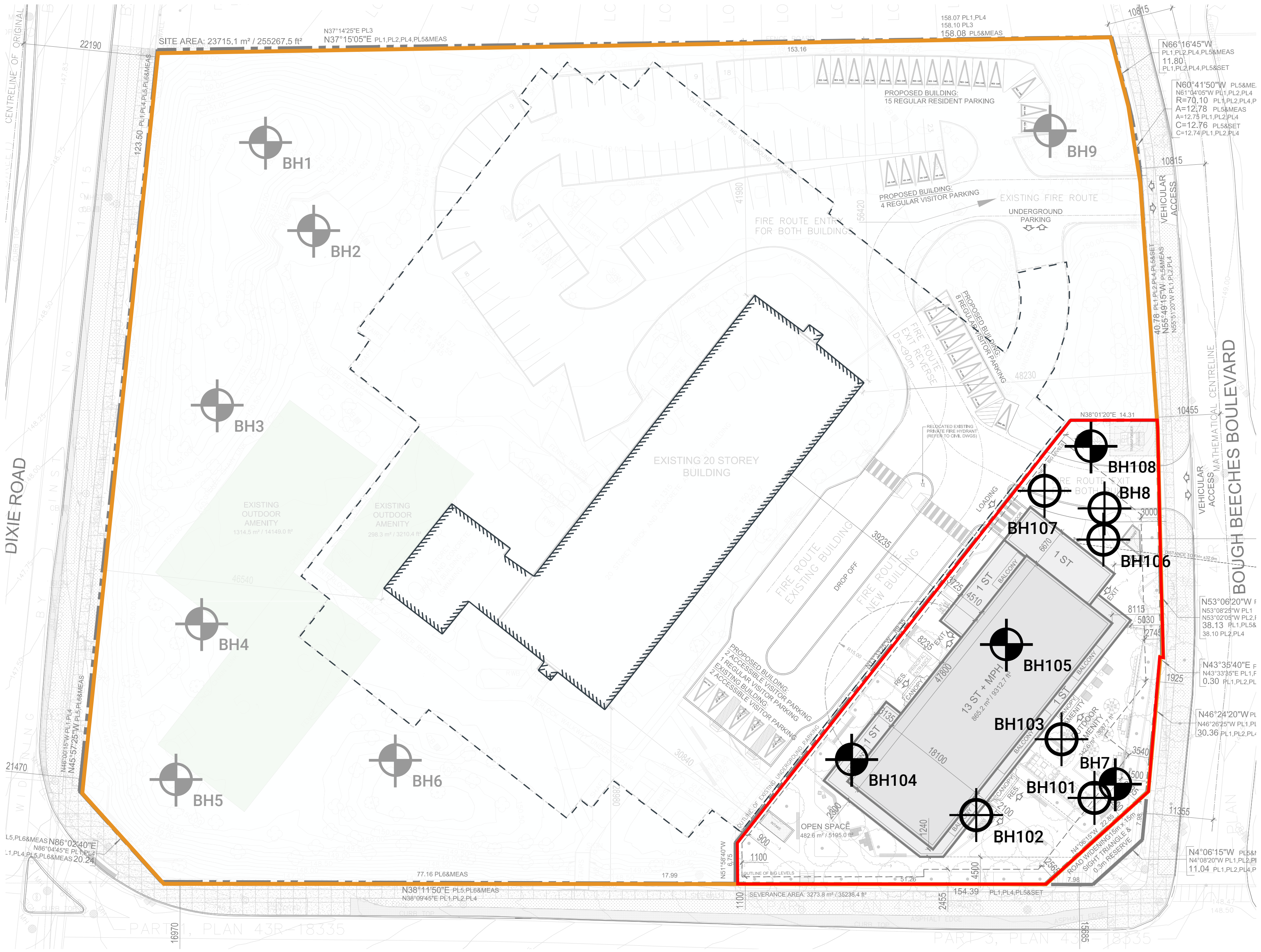
0m 15m 30m

Job No

25-122

Figure No

FIGURE 2



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LEGEND

—	APPRXIMATE PROPERTY BOUNDARY
—	APPRXIMATE EXTENT OF PROPOSED DEVELOPMENT
▨	EXISTING BUILDING
⬚	EXISTING UNDERGROUND GARAGE
⊕ ⊕	RELEVANT MONITORING WELL/BOREHOLE BY GROUNDED FOR PROPOSED DEVELOPMENT
⊕	MONITORING WELL/BOREHOLE BY GROUNDED PREVIOUSLY DRILLED AT OTHER PORTIONS OF THE PROPERTY

Note

Reference

Architectural Drawing, "1315 Bough Beeches Boulevard", job no. 30280666, dated February 12, 2026 Prepared by Arcadis.

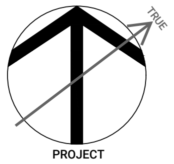
Project

1315 BOUGH BEECHES BLVD, MISSISSAUGA, ONTARIO

Figure Title

BOREHOLE AND MONITORING WELL LOCATION PLAN - PROPOSED SITE CONDITIONS

North



Date

MARCH 2026

Scale

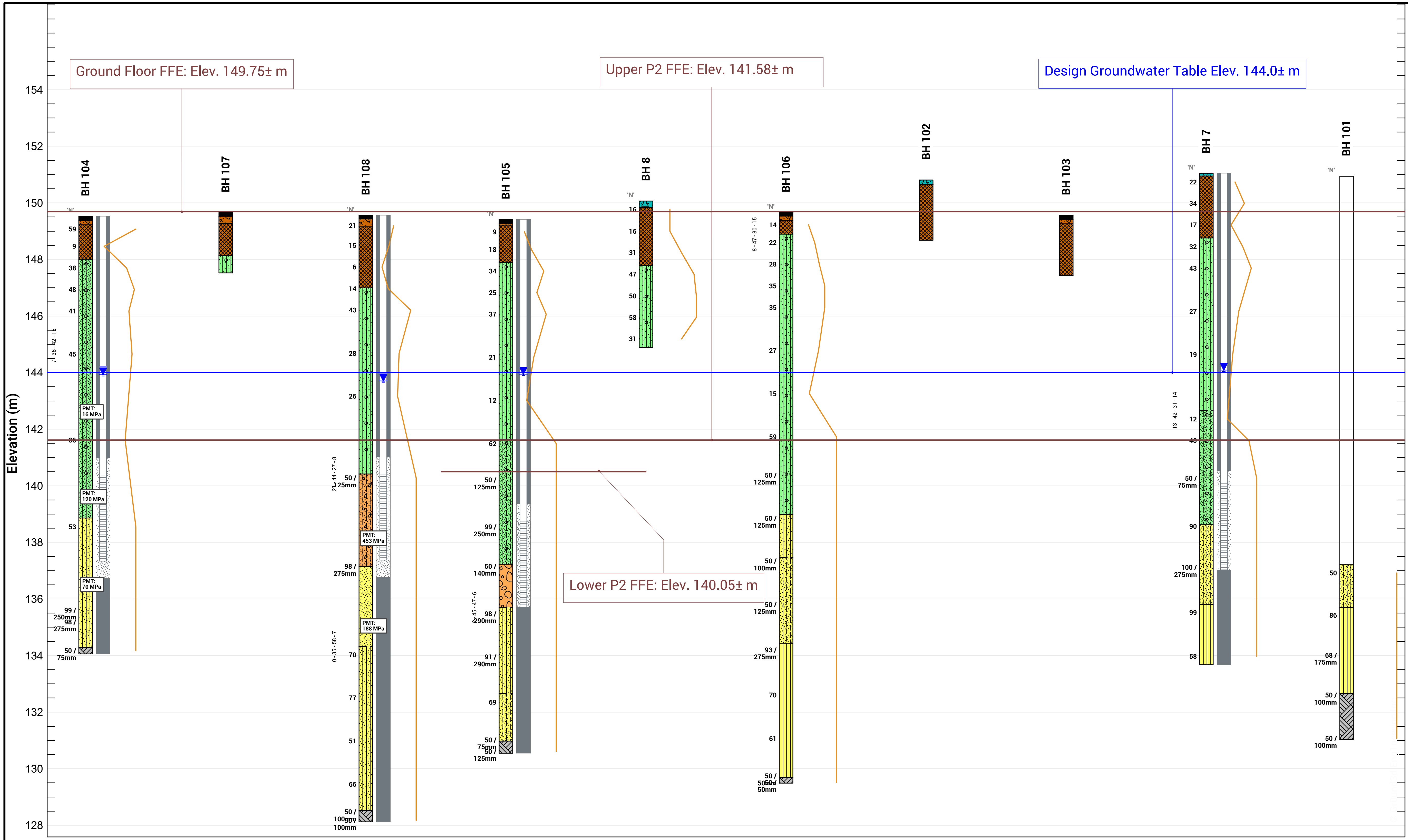
0m 15m 30m

Job No

25-122

Figure No

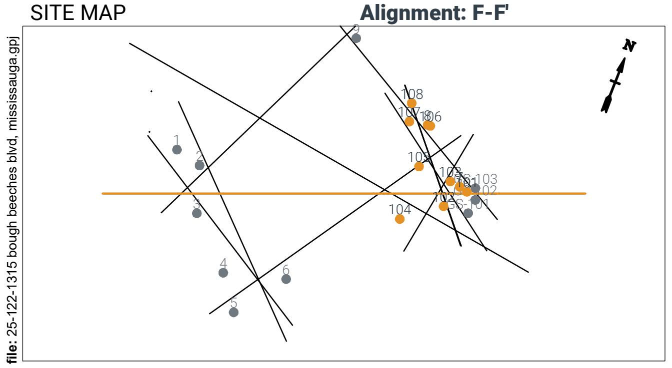
FIGURE 3



Boreholes Equally Spaced

SITE MAP

Alignment: F-F'



file: 25-122-1315 bough beeches blvd. mississauga.gpj

BOREHOLE STRATIGRAPHY LEGEND

- Topsoil
- Fill
- Sandy Silt Till
- Silty Sand Till
- Silty Sand
- Silt
- Blank
- Bedrock (inferred)
- Asphalt
- Aggregate
- Silt and Sand Till
- Sand and Silt
- Gravel
- Silt and Sand
- Gravelly Silty Sand
- Sand

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LEGEND

- 🟧 FILL
- 🟧 GRAVELS (gravel to gravelly sand)
- 🟨 SILT TO SAND (not till)
- 🟩 COHESIONLESS TILLS
- 🟦 COHESIVE SOILS (clayey silt to clay, incl. tills)
- 🟦 DISTURBED/REWORKED/ORGANIC

BH 101 BOREHOLES BY GROUNDED

T-BH7 BOREHOLES BY OTHERS

▽ water level, unstabilized
▼ water level, stabilized (latest)
▼ water level, stabilized (highest)

Project
1315 BOUGH BEECHES BVLD
MISSISSAUGA, ON

Figure Title
SUBSURFACE PROFILE
F-F'

Date
MARCH 2026

Scale
AS INDICATED

Job No
25-122

Figure No
FIGURE 4

TABLE 1

TABLE 1:
GROUNDWATER LEVEL MONITORING SUMMARY
1315 BOUGH BEECHES BOULEVARD, MISSISSAUGA

					Grounded Engineering					
Well ID	**Ground Surface Elev. (masl)**	**Well Screen Interval**		**Soil Strata**	**August 21, 2025**		**September 18, 2025**		**October 17, 2025**	
		(mbgs)	**(masl)**		**(mbgs)**	**(masl)**	**(mbgs)**	**(masl)**	**(mbgs)**	**(masl)**
BH7	151.0	11.0 - 14.0	140.1 - 137.0	Glacial Till	7.42	143.58	7.53	143.47	7.54	143.46
BH104	149.5	9.1 - 12.2	140.4 - 137.3	Sand and Silt	NA	-	NA	-	NA	-
BH105	149.4	10.7 - 13.7	138.7 - 135.7	Glacial Till	NA	-	NA	-	NA	-
BH108	149.6	9.1 - 12.2	140.4 - 137.4	Sand and Silt	NA	-	NA	-	NA	-

mbgs = metres below existing ground surface
masl = metres above sea level
NA = not available, unable to access monitoring well

TABLE 1:
GROUNDWATER LEVEL MONITORING SUMMARY
1315 BOUGH BEECHES BOULEVARD, MISSISSAUGA

					Grounded Engineering							
Well ID	**Ground Surface Elev. (masl)**	**Well Screen Interval**		**Soil Strata**	**November 14, 2025**		**January 5, 2026**		**January 27, 2026**		**February 27, 2026**	
		(mbgs)	**(masl)**		**(mbgs)**	**(masl)**	**(mbgs)**	**(masl)**	**(mbgs)**	**(masl)**	**(mbgs)**	**(masl)**
BH7	151.0	11.0 - 14.0	140.1 - 137.0	Glacial Till	7.30	143.70	7.44	143.56	7.32	143.68	7.32	143.68
BH104	149.5	9.1 - 12.2	140.4 - 137.3	Sand and Silt	NA	-	5.94	143.56	5.58	143.92	5.69	143.81
BH105	149.4	10.7 - 13.7	138.7 - 135.7	Glacial Till	NA	-	5.79	143.61	5.69	143.71	5.69	143.71
BH108	149.6	9.1 - 12.2	140.4 - 137.4	Sand and Silt	NA	-	5.96	143.64	5.87	143.73	6.10	143.50

mbgs = metres below existing ground surface
masl = metres above sea level
NA = not available, unable to access monitoring well

TABLE 1:
GROUNDWATER LEVEL MONITORING SUMMARY
1315 BOUGH BEECHES BOULEVARD, MISSISSAUGA

Well ID	Ground Surface Elev. (masl)	Well Screen Interval		Soil Strata	March 13, 2026		Minimum Elev. (Lowest)		Maximum Elev. (Highest)		Seasonal Fluctuation
		(mbgs)	(masl)		(mbgs)	(masl)	(mbgs)	(masl)	(mbgs)	(masl)	(±m)
BH7	151.0	11.0 - 14.0	140.1 - 137.0	Glacial Till	7.00	144.00	7.54	143.46	7.00	144.00	0.54
BH104	149.5	9.1 - 12.2	140.4 - 137.3	Sand and Silt	5.70	143.80	5.94	143.56	5.58	143.92	0.36
BH105	149.4	10.7 - 13.7	138.7 - 135.7	Glacial Till	5.50	143.90	5.79	143.61	5.50	143.90	0.29
BH108	149.6	9.1 - 12.2	140.4 - 137.4	Sand and Silt	5.90	143.70	6.10	143.50	5.87	143.73	0.23

mbgs = metres below existing ground surface
masl = metres above sea level
NA = not available, unable to access monitoring well

APPENDIX A

BOREHOLE LOG TERMINOLOGY

GROUNDED ENGINEERING

SAMPLING/TESTING METHODS

SS: split spoon sample

AS: auger sample

GS: grab sample

FV: shear vane

DP: direct push

PMT: pressuremeter test

ST: shelby tube

CORE: soil coring

RUN: rock coring

SYMBOLS & ABBREVIATIONS

MC: moisture content

LL: liquid limit

PL: plastic limit

NP: non-plastic

γ: soil unit weight (bulk)

G_s: specific gravity

S_u: undrained shear strength

▽ unstabilized water level

▼ water level measurement

▼ highest water level measurement

ENVIRONMENTAL SAMPLES

M&I: metals and inorganic parameters

PAH: polycyclic aromatic hydrocarbon

PCB: polychlorinated biphenyl

VOC: volatile organic compound

PHC: petroleum hydrocarbon

BTEX: benzene, toluene, ethylbenzene and xylene

PPM: parts per million

FIELD MOISTURE (based on tactile inspection)

DRY: no observable pore water

MOIST: inferred pore water, not observable (i.e. grey, cool, etc.)

WET: visible pore water

COHESIONLESS

Relative Density	N-Value
Very Loose	<4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very Dense	>50

COHESIVE

Consistency	N-Value	Su (kPa)
Very Soft	<2	<12
Soft	2 - 4	12 - 25
Firm	4 - 8	25 - 50
Stiff	8 - 15	50 - 100
Very Stiff	15 - 30	100 - 200
Hard	>30	>200

COMPOSITION

Term	% by weight
trace silt	<10
some silt	10 - 20
silt***y***	20 - 35
sand ***and*** silt	>35

ASTM STANDARDS

ASTM D1586 Standard Penetration Test (SPT)
Driving a 51 mm O.D. split-barrel sampler ("split spoon") into soil with a 63.5 kg weight free falling 760 mm. The blows required to drive the split spoon 300 mm ("bpf") after an initial penetration of 150 mm is referred to as the N-Value.

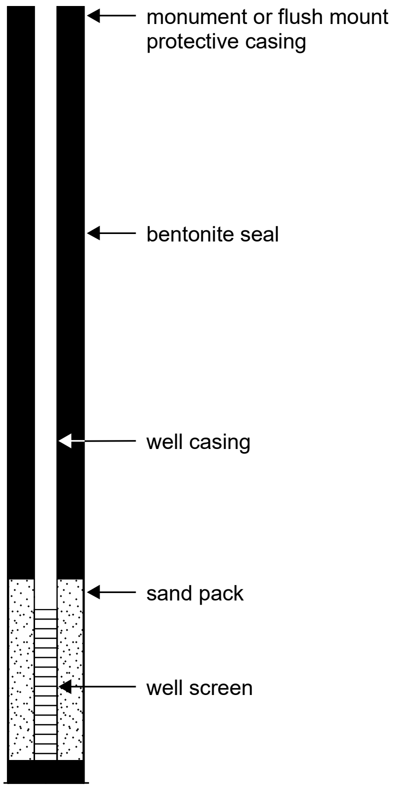
ASTM D3441 Cone Penetration Test (CPT)
Pushing an internal still rod with a outer hollow rod ("sleeve") tipped with a cone with an apex angle of 60° and a cross-sectional area of 1000 mm^2 into soil. The resistance is measured in the sleeve and at the tip to determine the skin friction and the tip resistance.

ASTM D2573 Field Vane Test (FVT)
Pushing a four blade vane into soil and rotating it from the surface to determine the torque required to shear a cylindrical surface with the vane. The torque is converted to the shear strength of the soil using a limit equilibrium analysis.

ASTM D1587 Shelby Tubes (ST)
Pushing a thin-walled metal tube into the in-situ soil at the bottom of a borehole, removing the tube and sealing the ends to prevent soil movement or changes in moisture content for the purposes of extracting a relatively undisturbed sample.

ASTM D4719 Pressuremeter Test (PMT)
Place an inflatable cylindrical probe into a pre-drilled hole and expanding it while measuring the change in volume and pressure in the probe. It is inflated under either equal pressure increments or equal volume increments. This provides the stress-strain response of the soil.

WELL LEGEND



GROUNDED
ENGINEERING

Date Started : Aug 11, 2025
Position : E: 612443, N: 4831462 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 7

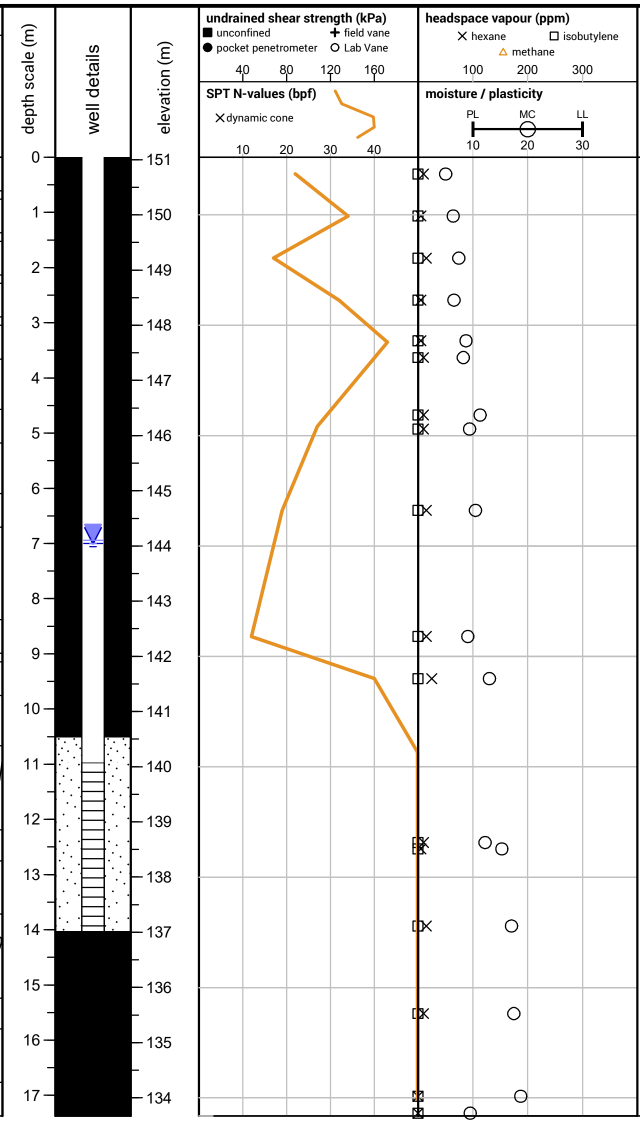
File No. : 25-122 **Project** : 1315 Bough Beeches Bvld, Mississauga, ON **Client** : 1315 Bough Beeches Boulevard Limited

drill method : CME 55

- hollow stem augers OD=215 mm (0 to 3.0 m)
- mud rotary med OD=125 mm (3.0 to 17.4 m)

elev depth (m)	stratigraphy description
151.0	**GROUND SURFACE**
	100mm **TOPSOIL**
	FILL, silty sand, trace clay, trace gravel, occassional rootlets, trace glass fragments, compact to dense, light brown with orange, moist
148.7 2.3	**SANDY SILT**, gravelly, trace clay, trace gravel, dense to compact, light brown with orange, moist (GLACIAL TILL) ...at 2.9 m, trace gravel ...at 4.6 m, wet ...at 4.8 m, grey ...at 6.1 m, clayey
142.6 8.4	**SILTY SAND**, some clay, some gravel, compact, grey, wet (GLACIAL TILL)
138.6 12.4	**SILTY SAND**, some gravel, very dense, grey, wet ...at 13.7 m, trace gravel
135.8 15.2	**SILT**, trace clay, trace sand, very dense, grey, wet
133.6 17.4	...at 17.3 m, sandy silt, trace shale fragments

number	type	SPT N-value
1	SS	22
2	SS	34
3	SS	17
4	SS	32
5A	SS	43
5B	SS	
6A	SS	27
6B	SS	
7	SS	19
8	SS	12
9	SS	40
10	SS	50 / 75mm
11A	SS	90
11B	SS	
12	SS	100 / 275mm
13	SS	99
14A	SS	58
14B	SS	



lab data and comments

SS1: PAHs, OCs

2.3m: spoon bouncing

SS4: BTEX, OCs, PHCs, VOCs

SS5A: H-Ms, Metals, ORPs

SS7: PAHs, OCs

grain size distribution (%) (MIT)

GR	SA	SI	CL
13	42	31	14

SS9: BTEX, H-Ms, Metals, ORPs, PHCs
9.8m: 10 mins of auger grinding **to 10.7m**

SS12: VOCs
13.7m: spoon bouncing

END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

100 mm dia. monitoring well installed.
No. 10 screen

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Aug 20, 2025	7.4	143.6
Sep 18, 2025	7.5	143.5
Oct 17, 2025	7.5	143.5
Nov 14, 2025	7.3	143.7
Jan 5, 2026	7.4	143.6
Jan 27, 2026	7.3	143.7
Feb 27, 2026	7.3	143.7
Mar 13, 2026	7.0	144.0

file: 25-122-1315 bough beeches blvd, mississauga.gpj

Tech : EB | **PM** : DK | **Rev** : NN

Date Started : Aug 11, 2025
Position : E: 612410, N: 4831491 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 8

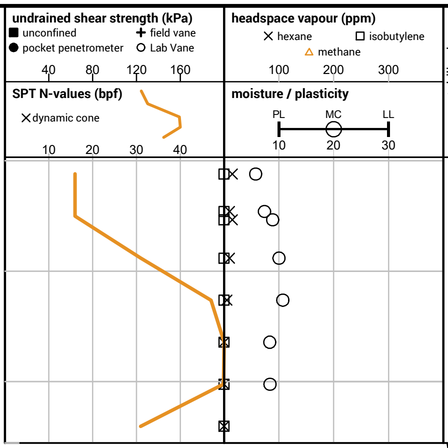
File No. : 25-122 **Project** : 1315 Bough Beeches Bvld, Mississauga, ON **Client** : 1315 Bough Beeches Boulevard Limited

drill method : *CME 55*

hollow stem augers OD=215 mm

elev / depth (m)	description	number	type	SPT N-value	depth scale (m)	elevation (m)	lab data and comments
150.1	**GROUND SURFACE**				0	150	
	225mm **TOPSOIL**	1	SS	16			SS1: PAHs, OCs
	FILL, sandy silt, trace clay, trace gravel, occassional rootlets, compact to dense, light brown with orange, moist ...at 0.8 m, sand seam to 1.5 m ...at 1.5 m, trace rock fragments, cobbles inferred, dense	2A 2B	SS SS	16	1	149	
		3	SS	31	2	148	SS3: H-Ms, Metals, ORPs
147.8 2.3	**SANDY SILT**, trace clay, trace gravel, dense to very dense, light brown with orange, moist (GLACIAL TILL)	4	SS	47			SS4: BTEX, OCs, PHCs, VOCs
		5	SS	50	3	147	
		6	SS	58	4	146	
144.9 5.2		7	SS	31	5	145	SS7: BTEX, H-Ms, Metals, ORPs, PAHs, OCs, PHCs, VOCs

well details



unstabilized water level

grain size distribution (%) (MIT)
GR SA SI CL

END OF BOREHOLE

Borehole was dry upon completion of drilling.

file: 25-122-1315 bough beeches blvd, mississauga.gpj

Tech : EB | **PM** : DK | **Rev** : NN

Date Started : Dec 15, 2025
Position : E: 612448, N: 4831461 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 101

File No. : 25-122 **Project** : 1315 Bough Beeches Bvld, Mississauga, ON **Client** : 1315 Bough Beeches Boulevard Limited

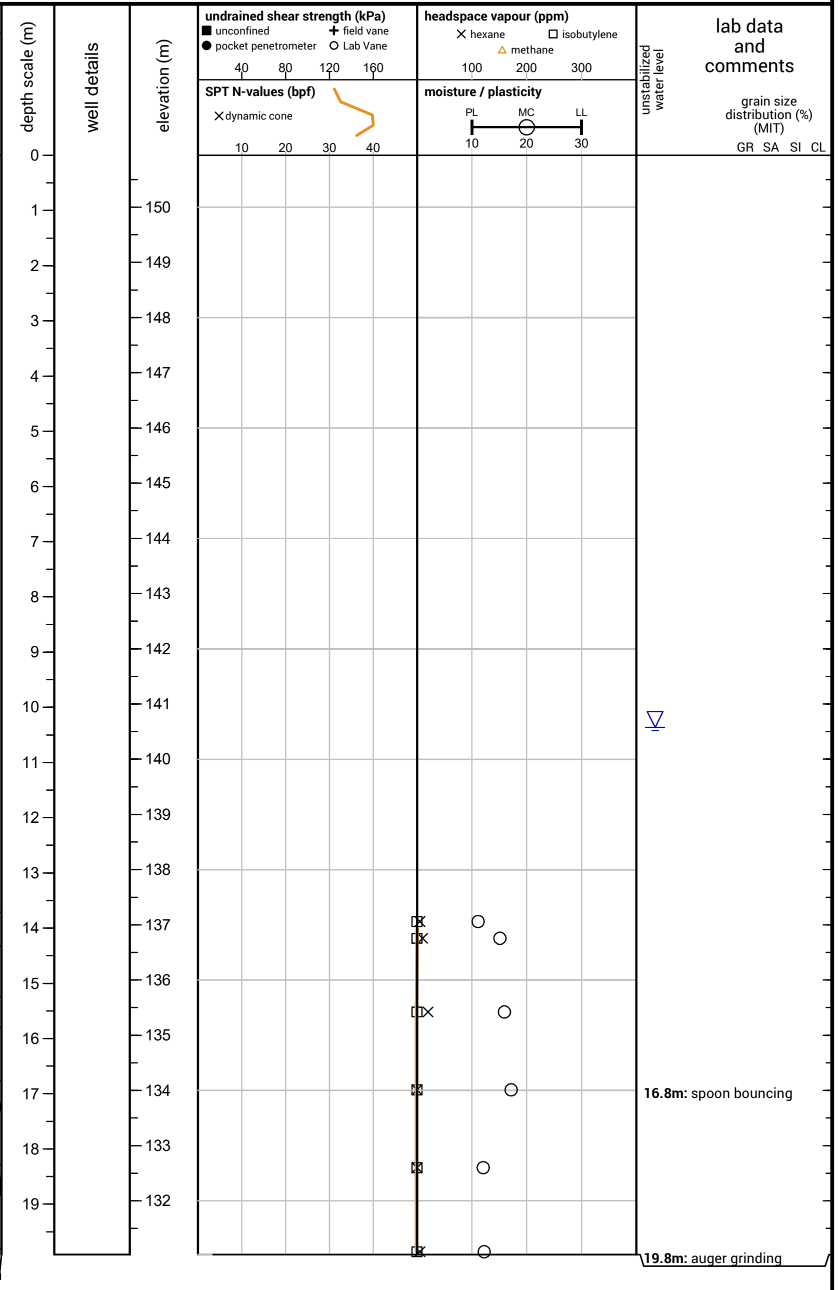
drill method : *LC rig*

hollow stem augers OD=215 mm

elev depth (m)	stratigraphy description	graphic log	samples number	type	SPT N-value	lab data and comments
150.9	**GROUND SURFACE**					
	Straight drill to 13.7m. Refer to Borehole 7 for stratigraphy					
137.2 13.7	**SILTY SAND**, trace gravel, very dense, grey, wet		1A 1B	SS	50	
135.7 15.2	**SILT**, trace sand, trace clay, very dense, grey, moist		2	SS	86	
			3	SS	68 / 175mm	**16.8m:** spoon bouncing
132.6 18.3	**INFERRED BEDROCK**, weathered shale fragments, grey, moist		4	SS	50 / 100mm	
131.0 19.9			5	SS	50 / 100mm	**19.8m:** auger grinding

END OF BOREHOLE

Unstabilized water level measured at 10.4 m below ground surface upon completion of drilling.

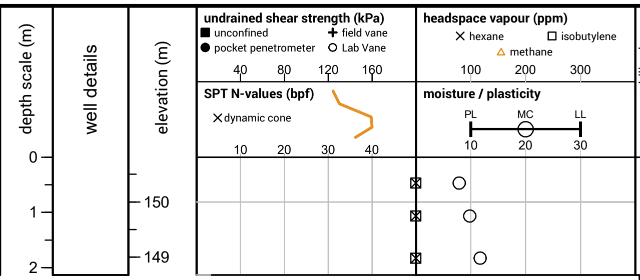


Date Started : Dec 15, 2025
Position : E: 612438, N: 4831447 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 102

File No. : 25-122 **Project** : 1315 Bough Beeches Bvld, Mississauga, ON **Client** : 1315 Bough Beeches Boulevard Limited

drill method : *LC rig*	elev depth (m)	stratigraphy description	graphic log	samples number	type	SPT N-value
hollow stem augers OD=215 mm	**150.8**	**GROUND SURFACE**				
		165mm **TOPSOIL**		1	SS	
		FILL, sandy silt, trace clay, trace gravel, trace rootlets, trace glass, brown, moist		2	SS	
	148.7 2.1			3	SS	



END OF BOREHOLE

Borehole was dry and caved to 1.4 m below ground surface upon completion of drilling.

lab data and comments
grain size distribution (%) (MIT)
GR SA SI CL

SS1: BTEX, PAHs, OCs, PHCs, VOCs

SS2: EC/SAR, H-Ms, Metals, ORPs, pH

SS3: BTEX, PAHs, OCs, PHCs, VOCs

file: 25-122-1315 bough beeches blvd, mississauga.gpj

Tech : EB/SA | **PM** : DK | **Rev** : KB

Date Started : Dec 16, 2025
Position : E: 612436, N: 4831463 (UTM 17T)
Elev. Datum : Geodetic

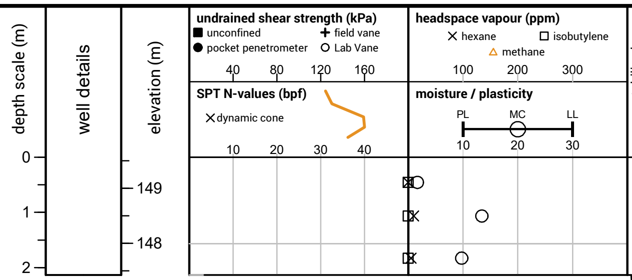
BOREHOLE LOG 103

File No. : 25-122 **Project** : 1315 Bough Beeches Bvld, Mississauga, ON **Client** : 1315 Bough Beeches Boulevard Limited

drill method : *LC rig*	elev depth (m)	stratigraphy description	samples number	samples type	SPT N-value
hollow stem augers OD=215 mm	149.6	**GROUND SURFACE**			
	149.3 0.3	150mm **ASPHALT**	1	SS	
		150mm **AGGREGATE**			
		FILL, sandy silt, trace to some clay, trace gravel, trace rootlets, brown, moist	2	SS	
	147.5 2.1		3	SS	

END OF BOREHOLE

Borehole was dry upon completion of drilling.



lab data and comments

unstabilized water level

grain size distribution (%) (MIT)
GR SA SI CL

SS1: PAHs, OCs

SS2: BTEX, EC/SAR, H-Ms, Metals, ORPs, pH, PHCs, VOCs

SS3: BTEX, PAHs, OCs, PHCs, VOCs

Date Started : Dec 16, 2025
Position : E: 612415, N: 4831430 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 104

File No. : 25-122 **Project** : 1315 Bough Beeches Bvld, Mississauga, ON **Client** : 1315 Bough Beeches Boulevard Limited

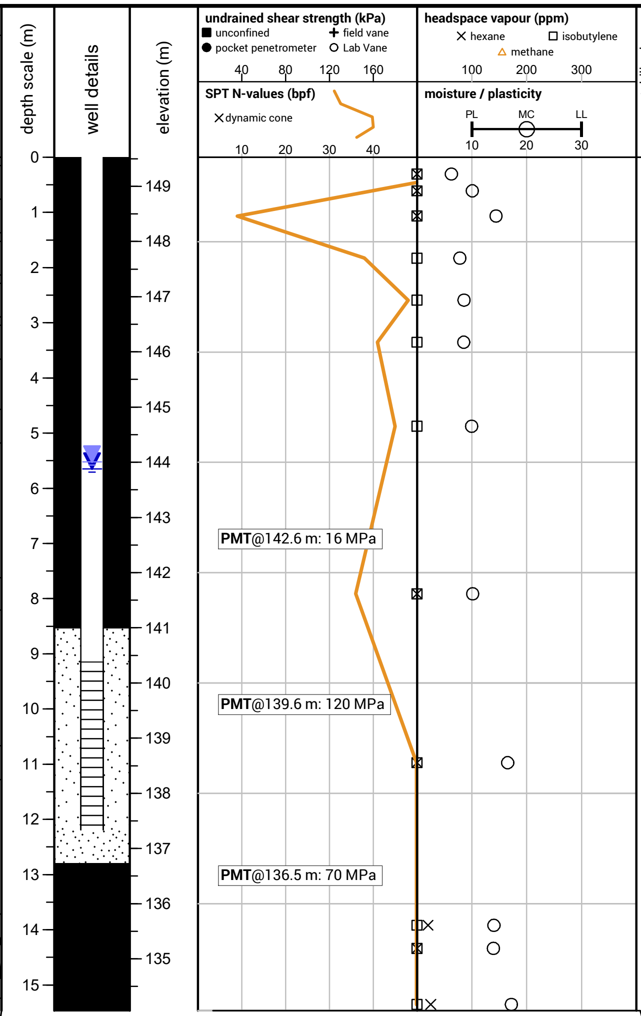
drill method : *CME 75*
hollow stem augers OD=215 mm
mud rotary small OD=100 mm

stratigraphy

elev depth (m)	description
149.5	**GROUND SURFACE**
149.2 0.3	150mm **ASPHALT**
	150mm **AGGREGATE**
148.0 1.5	**FILL**, sandy silt, trace gravel, trace clay, very dense, dark brown, moist ...at 0.8 m, clayey silt, trace sand, stiff, dark brown, moist
	SILT AND SAND, trace to some clay, trace gravel, dense, light brown with orange, moist (GLACIAL TILL) ...at 4.6 m, grey
138.8 10.7	**SAND AND SILT**, trace gravel, very dense, grey, wet ...at 13.7 m, clayey, weathered shale fragments
134.3 15.2	**INFERRED BEDROCK**, weathered shale and limestone fragments, grey, wet
134.0 15.5	

samples

number	type	SPT N-value
1A 1B	SS	59
2	SS	9
3	SS	38
4	SS	48
5	SS	41
6	SS	45
1	PMT	
7	SS	36
2	PMT	
8	SS	53
3	PMT	
9	SS	99 / 250mm
10	SS	98 / 275mm
11	SS	50 / 75mm



lab data and comments

grain size distribution (%) (MIT)

	GR	SA	SI	CL
<u>1B:</u> BTEX, PAHs, OCs, PHCs, VOCs				
<u>SS2:</u> EC/SAR, H-Ms, Metals, ORPs, pH				
<u>SS3:</u> BTEX, PAHs, OCs, PHCs, VOCs				
<u>SS4:</u> EC/SAR, H-Ms, Metals, ORPs, pH				
<u>SS5:</u> BTEX, EC/SAR, H-Ms, Metals, ORPs, OCs, pH, PHCs, VOCs	7	36	42	15
<u>SS6:</u> BTEX, EC/SAR, H-Ms, Metals, ORPs, PAHs, pH, PHCs, VOCs				

15.3m: spoon bouncing
15.4m: spoon bouncing

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jan 5, 2026	5.9	143.6
Jan 27, 2026	5.6	143.9
Feb 27, 2026	5.7	143.8
Mar 13, 2026	5.7	143.8

END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

50 mm dia. monitoring well installed.
No. 10 screen

file: 25-122-1315 bough beeches blvd, mississauga.gpj

Tech : CSH/SA | **PM** : DK | **Rev** : KB

Date Started : Dec 22, 2025
Position : E: 612415, N: 4831465 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 105

File No. : 25-122 **Project** : 1315 Bough Beeches Bvld, Mississauga, ON **Client** : 1315 Bough Beeches Boulevard Limited

drill method : CME 75
hollow stem augers OD=215 mm
mud rotary small OD=100 mm

stratigraphy

elev depth (m)	description
149.4	**GROUND SURFACE**
	140mm **ASPHALT**
	65mm **AGGREGATE**
147.9 1.5	**FILL**, sandy silt, trace gravel, loose to compact, light brown, moist
	SANDY SILT, trace to some clay, trace gravel, dense to compact, orangey brown with grey, moist (GLACIAL TILL) ...at 3.0 m, grey with orange ...at 4.6 m, grey ...at 6.3 m, clayey to 6.7 m
141.6 7.8	...at 7.6 m, silty clay seams to 7.8 m
	SILTY SAND, trace to some clay, trace gravel, very dense, grey, wet (GLACIAL TILL)
137.2 12.2	**GRAVEL**, trace sand, trace silt, very dense, grey, wet
135.7 13.7	**SILT AND SAND**, trace clay, trace gravel, occasional rock fragments, very dense, grey, wet
132.6 16.8	**SILTY SAND**, trace clay, very dense, grey, wet
131.0 18.4	...at 18.3 m, weathered rock fragments
130.5 18.9	**INFERRED BEDROCK**, weathered shale and limestone fragments, grey, moist to wet

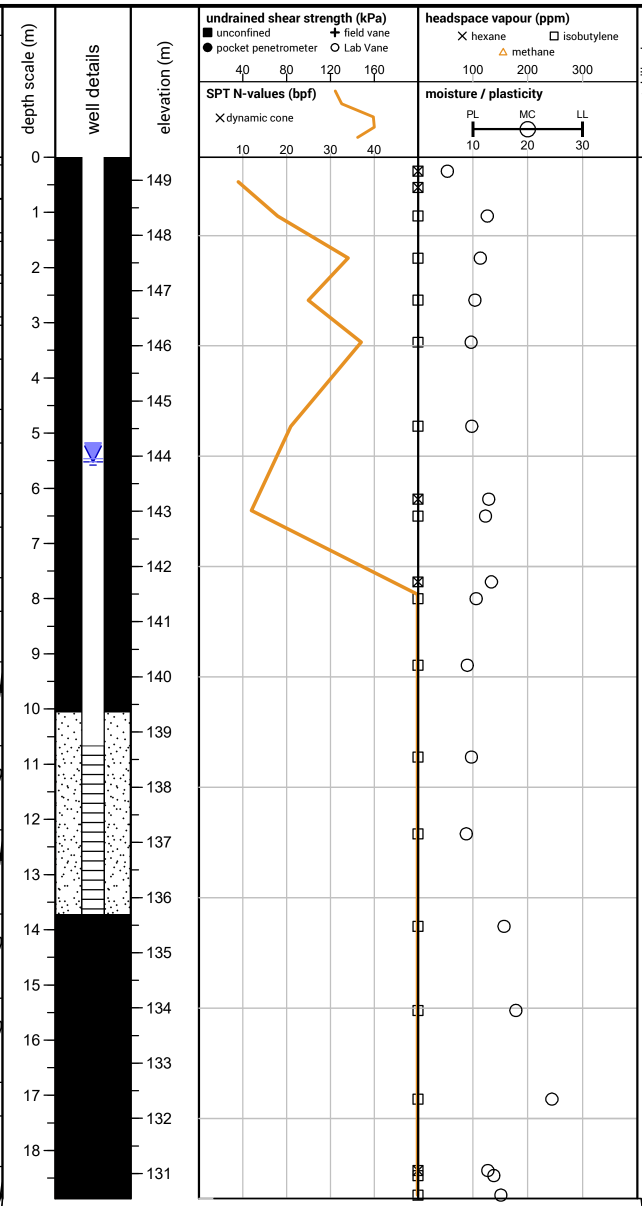
END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

50 mm dia. monitoring well installed.
No. 10 screen

samples

number	type	SPT N-value
1A 1B	SS	9
2	SS	18
3	SS	34
4	SS	25
5	SS	37
6	SS	21
7A 7B	SS	12
8A 8B	SS	62
9	SS	50 / 125mm
10	SS	99 / 250mm
11	SS	50 / 140mm
12	SS	98 / 290mm
13	SS	91 / 290mm
14	SS	69
15A 15B	SS SS	50 / 75mm
16		50 / 125mm



unstabilized water level

lab data and comments

grain size distribution (%) (MIT)

GR	SA	SI	CL
2	45	47	6

1B: OCs

SS2: BTEX, EC/SAR, H-Ms, Metals, ORPs, PAHs, pH, PHCs, VOCs

SS3: BTEX, PAHs, OCs, PHCs, VOCs

SS4: EC/SAR, H-Ms, Metals, ORPs, pH

SS5: BTEX, EC/SAR, H-Ms, Metals, ORPs, OCs, pH, PHCs, VOCs

SS6: BTEX, PAHs, PHCs, VOCs

7A: EC/SAR, H-Ms, Metals, ORPs, pH

13.9m: spoon bouncing **to 14.2m**

15.6m: spoon bouncing **to 15.7m**

18.4m: auger grinding **to 18.7m**
18.4m: spoon bouncing **to 18.5m**
18.7m: auger refusal
18.8m: spoon bouncing **to 18.9m**

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jan 5, 2026	5.8	143.6
Jan 27, 2026	5.7	143.7
Feb 27, 2026	5.7	143.7
Mar 13, 2026	5.5	143.9

file: 25-122-1315 bough beeches blvd, mississauga.gpj

GROUNDED
ENGINEERING

Date Started : Dec 23, 2025
Position : E: 612412, N: 4831491 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 106

File No. : 25-122 **Project** : 1315 Bough Beeches Bvld, Mississauga, ON **Client** : 1315 Bough Beeches Boulevard Limited

drill method : *CME 75*

hollow stem augers OD=215 mm

mud rotary small OD=100 mm

stratigraphy

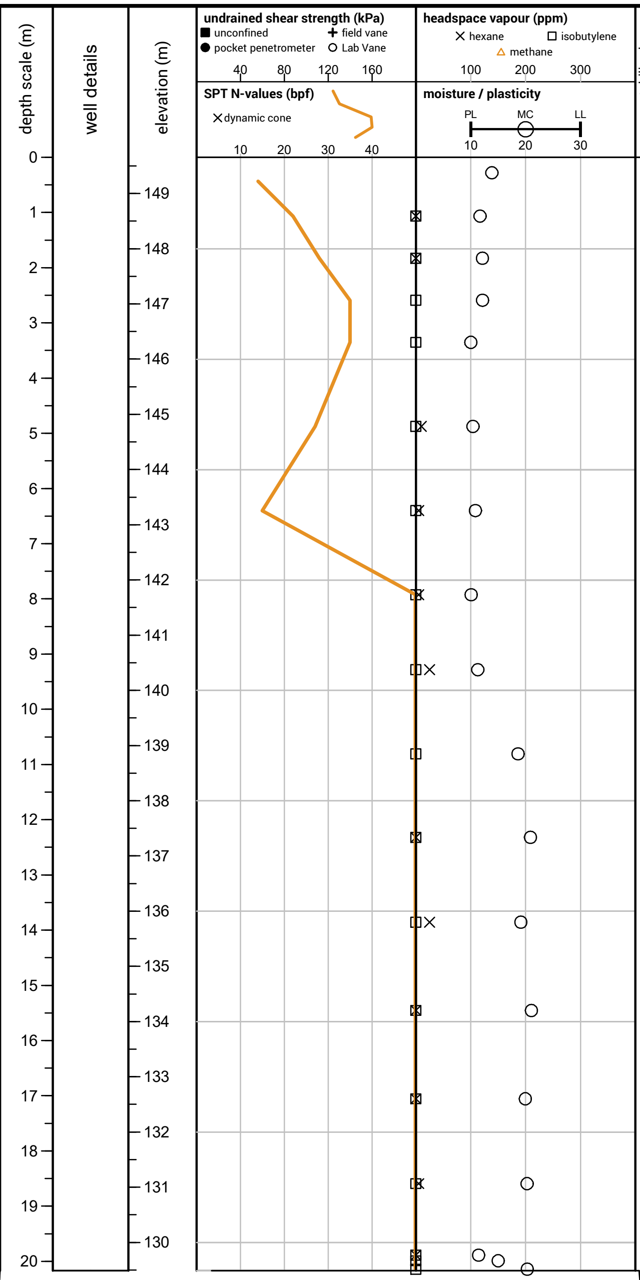
elev depth (m)	description
149.7	**GROUND SURFACE**
	125mm **ASPHALT**
148.9 0.8	150mm **AGGREGATE**
	FILL, sandy silt, trace gravel, compact, dark brown, moist
	SANDY SILT, trace to some clay, trace gravel, compact, orangey brown with grey, moist (GLACIAL TILL) ...at 2.3 m, dense ...at 4.6 m, compact, grey, wet ...at 7.6 m, very dense
139.0 10.7	**SAND AND SILT**, trace clay, trace gravel, very dense, grey, wet
137.5 12.2	**SILTY SAND**, trace gravel, very dense, grey, wet
134.5 15.2	**SILT**, some clay, trace sand, trace gravel, very dense, grey, wet
129.7 20.0	...at 19.8 m, weathered shale fragments
129.5 20.2	**INFERRED BEDROCK**, weathered shale and limestone fragments, grey, wet, moist

END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

samples

number	type	SPT N-value
1A 1B	SS	14
2	SS	22
3	SS	28
4	SS	35
5	SS	35
6	SS	27
7	SS	15
8	SS	59
9	SS	50 / 125mm
10	SS	50 / 125mm
11	SS	50 / 100mm
12	SS	50 / 125mm
13	SS	93 / 275mm
14	SS	70
15	SS	61
16A	SS	50 / 50mm
16B	SS	
17	SS	50 / 50mm



unstabilized water level

lab data and comments

grain size distribution (%) (MIT)

	GR	SA	SI	CL
1B: BTEX, PAHs, OCs, PHCs, VOCs	8	47	30	15

SS2: EC/SAR, H-Ms, Metals, ORPs, pH

SS3: EC/SAR, H-Ms, Metals, ORPs, pH

SS4: EC/SAR, H-Ms, Metals, ORPs, OCs, pH

SS5: BTEX, PAHs, PHCs, VOCs

SS6: BTEX, EC/SAR, H-Ms, Metals, ORPs, pH, PHCs, VOCs

SS7: EC/SAR, H-Ms, Metals, ORPs, PAHs, pH

19.5m: auger grinding
20.0m: spoon bouncing **to 20.0m**
20.1m: spoon bouncing **to 20.2m**

file: 25-122-1315 bough beeches blvd, mississauga.gpj

Tech : CSH/SA | **PM** : DK | **Rev** : KB

Date Started : Dec 16, 2025
Position : E: 612399, N: 4831488 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 107

File No. : 25-122 **Project** : 1315 Bough Beeches Bvld, Mississauga, ON **Client** : 1315 Bough Beeches Boulevard Limited

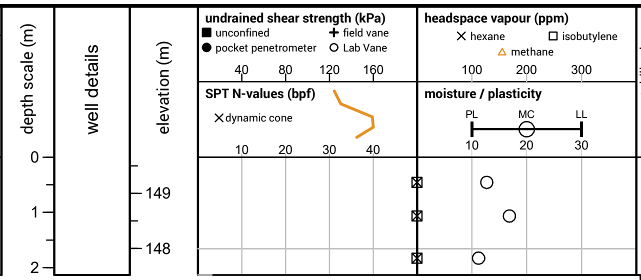
drill method : *CME 75*
hollow stem augers OD=215 mm

elev depth (m)	description
149.7	**GROUND SURFACE**
149.3 0.4	125mm **ASPHALT**
	250mm **AGGREGATE**
148.2 1.5	**FILL**, sandy silt, trace clay, trace gravel, dark brown with orange, moist
147.6 2.1	**SANDY SILT**, trace clay, trace gravel, orangey brown, moist (GLACIAL TILL)

number	type	SPT N-value
1	SS	
2	SS	
3	SS	

END OF BOREHOLE

Borehole was dry upon completion of drilling.



lab data and comments
grain size distribution (%) (MIT)
GR SA SI CL

SS1: BTEX, PAHs, OCs, PHCs, VOCs

SS2: EC/SAR, H-Ms, Metals, ORPs, pH

SS3: BTEX, PAHs, OCs, PHCs, VOCs

Tech : CSH/SA | **PM** : DK | **Rev** : KB

GROUNDED ENGINEERING

Date Started : Dec 18, 2025
Position : E: 612396, N: 4831500 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 108

File No. : 25-122 **Project** : 1315 Bough Beeches Bvld, Mississauga, ON **Client** : 1315 Bough Beeches Boulevard Limited

drill method : *CME 75*

hollow stem augers OD=215 mm (0 to 2.6 m); mud rotary small OD=100 mm (2.6 to 21.4 m)

stratigraphy

elev depth (m)	description
149.6	**GROUND SURFACE**
149.2 0.4	125mm **ASPHALT**
	280mm **AGGREGATE**
	FILL, gravelly sand, some silt, compact, brown, moist
147.0 2.6	...at 2.3 m, clayey ...at 2.6 m, some clay, light brown with orange
	SANDY SILT, trace to some clay, trace gravel, compact to loose, dark brown with orange, moist (GLACIAL TILL) ...at 3.0 m, dense ...at 4.6 m, compact, grey
140.5 9.1	**SILTY SAND**, gravelly, trace clay, very dense, grey, wet
137.2 12.4	**SAND**, trace silt, trace gravel, very dense, grey, wet
134.4 15.2	**SILT AND SAND**, trace clay, very dense, grey, wet
128.6 21.0	**INFERRED BEDROCK**, weathered shale and limestone fragments, grey, wet, moist
128.2 21.4	

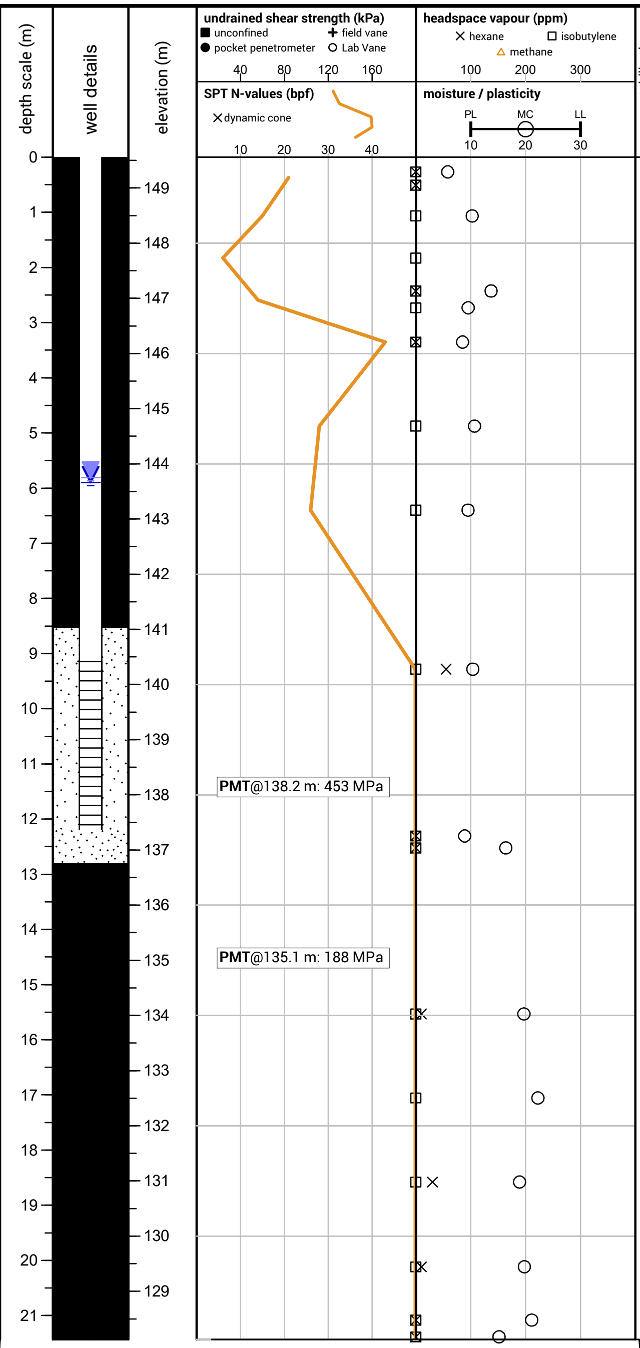
END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

50 mm dia. monitoring well installed.
No. 10 screen

samples

number	type	SPT N-value
1A 1B	SS	21
2	SS	15
3	SS	6
4A 4B	SS	14
5	SS	43
6	SS	28
7	SS	26
1	PMT	
8	SS	50 / 125mm
2	PMT	
9A 9B	SS	98 / 275mm
3	PMT	
10	SS	70
11	SS	77
12	SS	51
13	SS	66
14	SS	50 / 100mm
15	SS	50 / 100mm



lab data and comments

grain size distribution (%) (MIT)

1B: OCs

SS2: BTEX, EC/SAR, H-Ms, Metals, ORPs, PAHs, OCs, pH, PHCs, VOCs

SS3: EC/SAR, H-Ms, Metals, ORPs, PAHs, pH

4B: BTEX, PAHs, OCs, PHCs, VOCs

SS5: EC/SAR, H-Ms, Metals, ORPs, pH

SS6: BTEX, PAHs, PHCs, VOCs

SS7: EC/SAR, H-Ms, Metals, ORPs, pH
7.6m: PMT probe ruptured during testing - PMT test unsuccessful

GR	SA	SI	CL
21	44	27	8
0	35	58	7

12.6m: spoon bouncing **to 12.6m**

15.3m: spoon bouncing **to 15.4m**

21.1m: spoon bouncing **to 21.1m**
21.4m: spoon bouncing **to 21.4m**

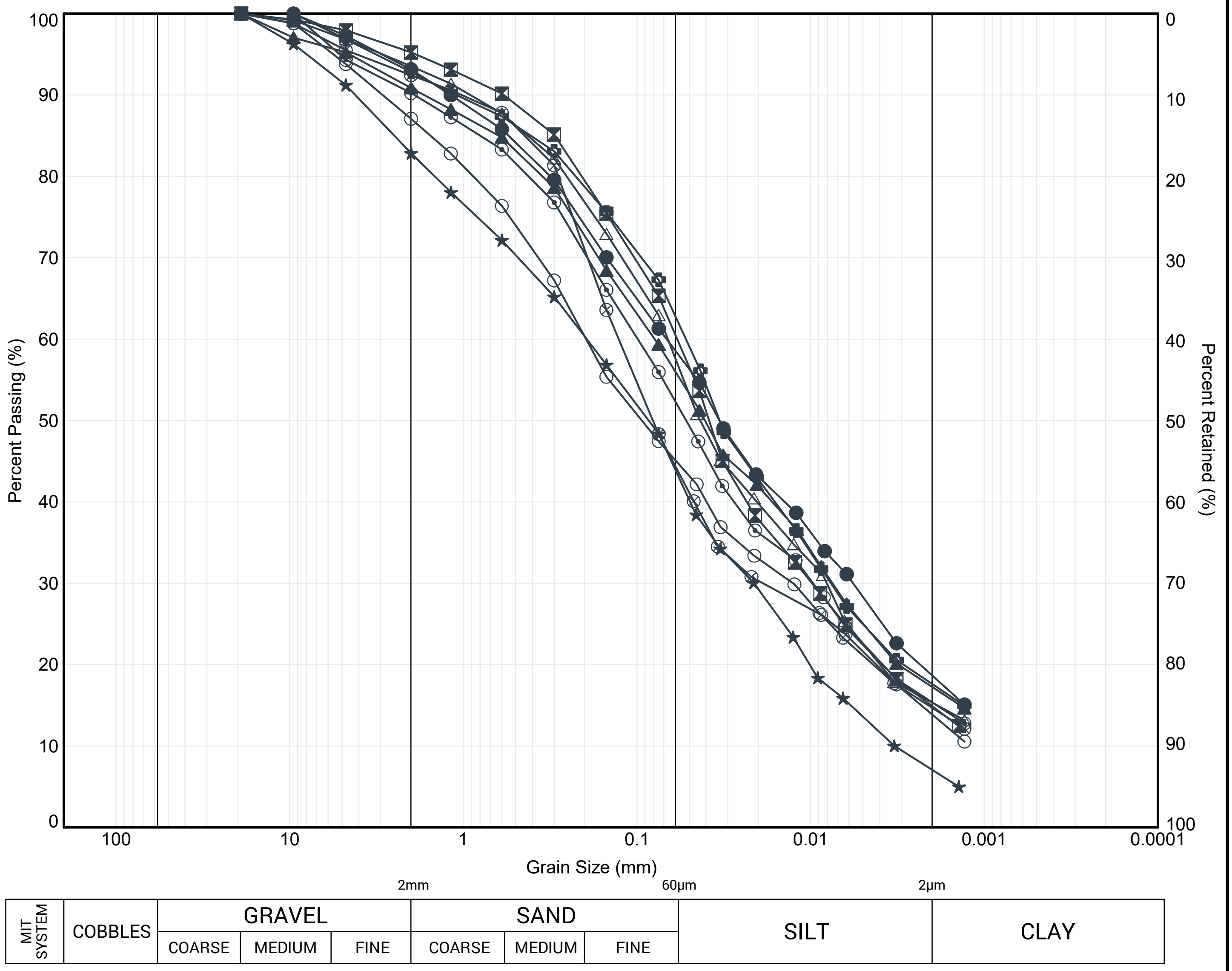
GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jan 5, 2026	6.0	143.6
Jan 27, 2026	5.9	143.7
Feb 27, 2026	6.1	143.5
Mar 13, 2026	5.9	143.7

file: 25-122-1315 bough beeches blvd, mississauga.gpj

Tech : CSH/SA | **PM** : DK | **Rev** : KB

APPENDIX B



MIT SYSTEM

	Location	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	**BH 1**	SS8	5.6	146.7	7	34	40	19
☒	**BH 3**	SS10	9.4	141.9	5	34	46	15
▲	**BH 4**	SS8	7.9	141.7	9	35	39	17
★	**BH 4**	SS10	11.0	138.7	17	39	37	7
⊙	**BH 5**	SS7	4.9	145.2	10	38	37	15
✪	**BH 6**	SS10	9.4	141.8	7	30	45	18
○	**BH 7**	SS8	8.7	142.4	13	42	31	14
△	**BH 104**	SS6	4.9	144.7	7	36	42	15
⊗	**BH 106**	SS2	1.1	148.6	8	47	30	15

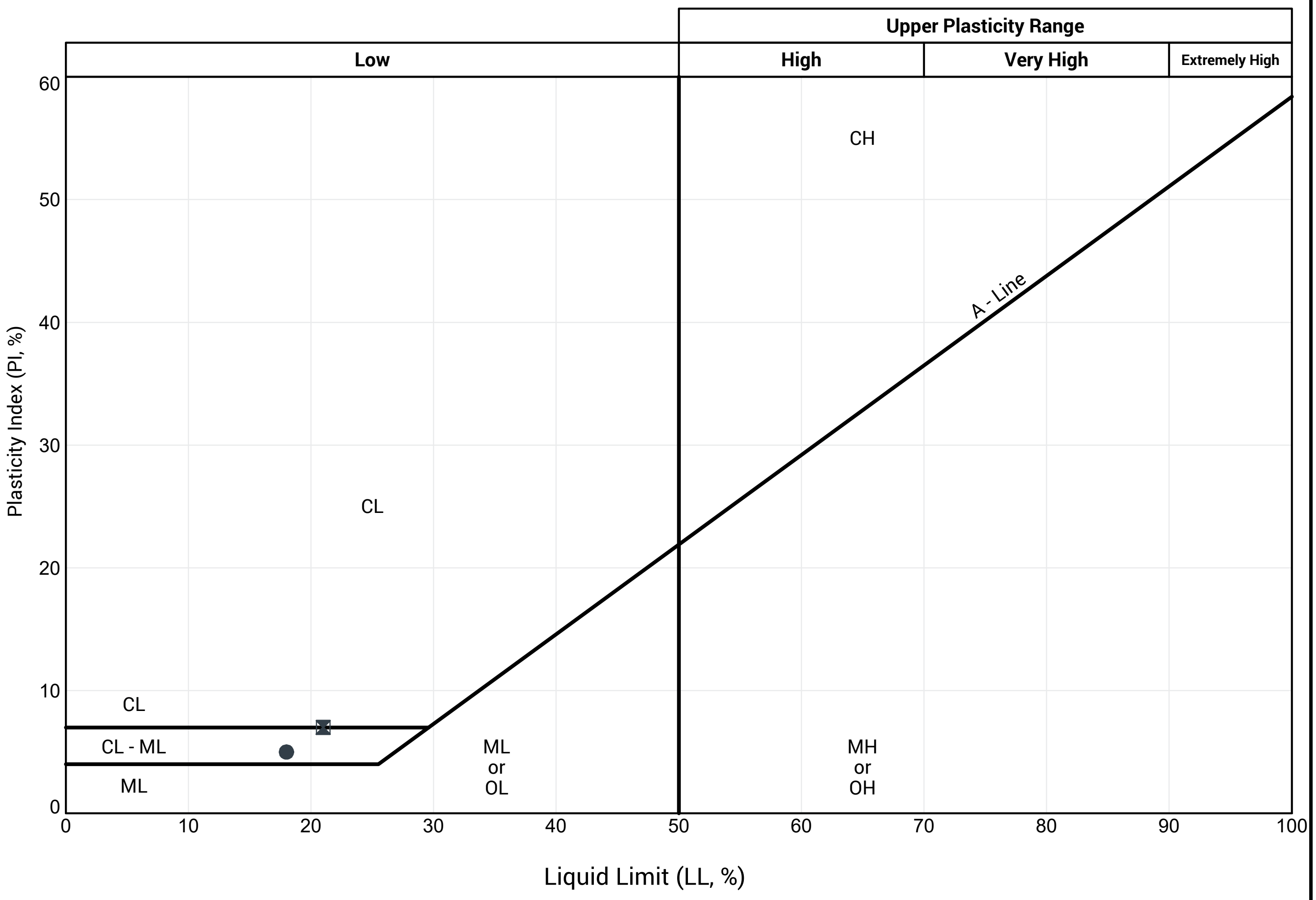
GROUNDED
ENGINEERING

Title:

GRAIN SIZE DISTRIBUTION
GLACIAL TILL

File No.: **25-122**

file: 25-122-1315 bough beeches blvd, mississauga.gpj



	Location	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)
●	**BH 4**	SS8	7.9	141.7	18	13	5
☒	**BH 5**	SS7	4.9	145.2	21	14	7

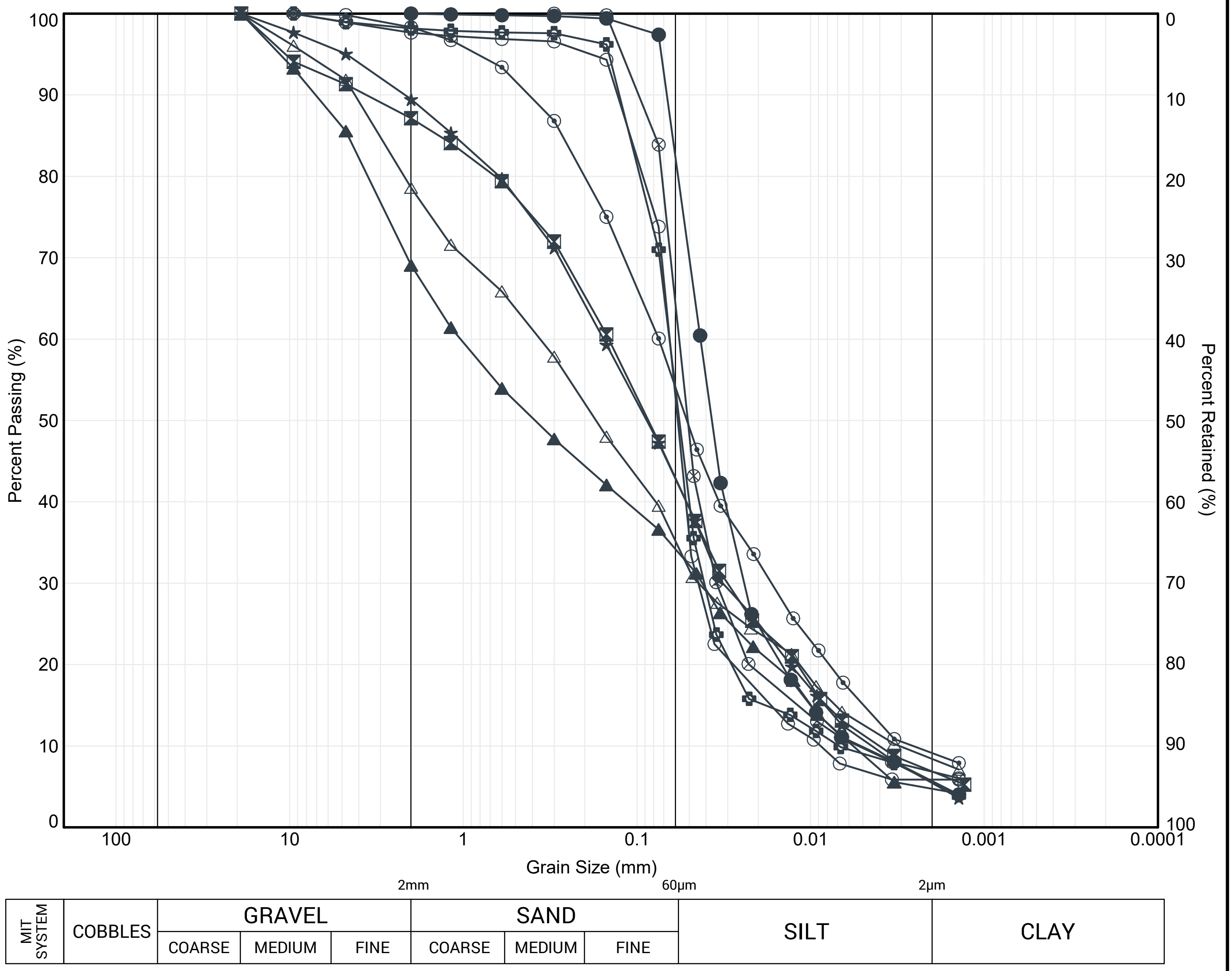
GROUNDED
ENGINEERING

Title:

ATTERBERG LIMITS CHART
GLACIAL TILL

File No.: **25-122**

file: 25-122-1315 bough beeches blvd, mississauga.gpj



MIT SYSTEM

	Location	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	**BH 1**	SS18	20.0	132.4	0	17	77	6
☒	**BH 2**	SS11	12.3	137.5	13	44	36	7
▲	**BH 3**	SS12	15.5	135.8	31	34	30	5
★	**BH 5**	SS11	11.6	138.4	11	46	37	6
⊙	**BH 6**	SS11	12.4	138.8	2	44	45	9
✚	**BH 9**	SS12	13.9	135.7	2	44	48	6
○	**BH 105**	SS12	13.9	135.5	2	45	47	6
△	**BH 108**	SS8	9.3	140.3	21	44	27	8
⊗	**BH 108**	SS10	15.5	134.0	0	35	58	7

GROUNDED
ENGINEERING

Title:

GRAIN SIZE DISTRIBUTION
SAND AND SILT

File No.:

25-122

file: 25-122-1315 bough beeches blvd, mississauga.gpj

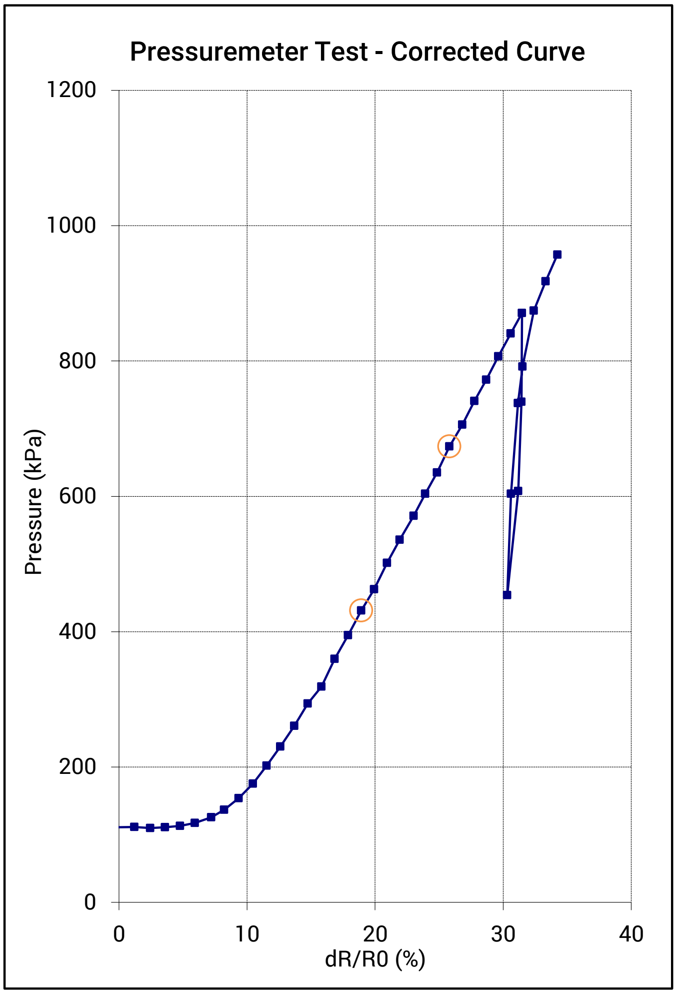
APPENDIX C

TEXAM Pressuremeter Test Results

GROUNDED
ENGINEERING

Project name:	**1315 Bough Beeches Blvd, Mississauga**	Drilling Method:	Mud Rotary Drilling
Borehole name:	**BH104**	Test depth:	6.9 m
Test date: (dd/mm/yyyy)	16/12/2025	Test Elev:	142.6 m
Test number:	25-122 BH104 22	Poisson's ratio:	0.35
Probe Designation	N Probe (76 mm OD)	Probe initial volume:	1632 cm^3

Raw Readings		**Corrected Readings**			
Pressure kPa	**Volume cm³**	**Pressure kPa**	**Volume cm³**	**DR/R_0 %**	
39	-3.0	111	-3.5	-0.11	
46	40.1	111	39.5	1.20	
51	81.6	110	80.9	2.45	
55	120.1	111	119.3	3.59	
60	160.4	113	159.6	4.78	
67	200.3	117	199.5	5.94	
78	244.6	126	243.5	7.20	
91	280.1	137	278.9	8.21	
110	320.9	154	319.4	9.35	
133	361.0	175	359.2	10.46	
161	399.9	202	397.8	11.53	
191	439.6	230	437.1	12.60	
223	480.7	261	477.8	13.70	
257	520.2	294	516.9	14.75	
283	560.6	319	556.9	15.81	
325	600.2	360	595.9	16.84	
361	641.1	395	636.4	17.90	
398	680.9	431	675.7	18.92	◄
430	720.6	463	714.9	19.92	
470	761.2	502	755.1	20.94	
505	800.6	536	794.0	21.93	
541	844.5	571	837.4	23.01	
574	881.7	604	874.3	23.93	
606	919.4	635	911.5	24.84	
645	959.4	674	951.0	25.81	◄
677	1000.8	706	992.0	26.80	
712	1041.3	741	1032.0	27.77	
744	1079.9	772	1070.1	28.68	
779	1120.1	807	1109.9	29.62	
813	1161.5	841	1150.9	30.59	
843	1199.8	871	1188.8	31.47	
712	1196.7	740	1187.4	31.44	
580	1183.6	608	1176.0	31.18	
426	1145.5	454	1139.9	30.33	◄Er
576	1159.4	604	1151.8	30.61	
710	1184.9	738	1175.6	31.17	
764	1200.2	792	1190.2	31.51	
847	1239.9	874	1228.9	32.40	
891	1280.5	918	1268.8	33.33	
931	1320.8	957	1308.7	34.24	



Interpreted Test Results

Epmt:	5,816 kPa
Ep-ur	48,861 kPa
Ey:	**16 MPa**
Ey-ur:	**132 MPa**
Pl:	1,202 kPa
Ep / Pl:	4.8
Py:	674 kPa
Poh (est.) :	**111 kPa**
K_0 (est):	**0.83**

Time before recording readings : 15 sec.

Method for estimating Pl : 1/V vs P as per ASTM D4719

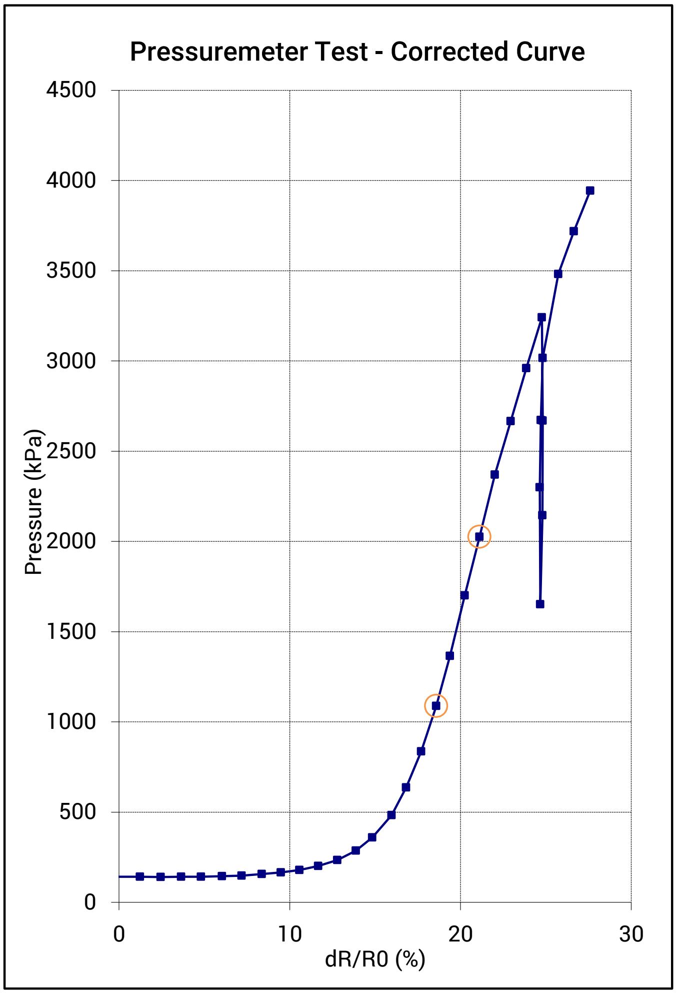
TEXAM COMPANION V.3.4.25

TEXAM Pressuremeter Test Results

GROUNDED
ENGINEERING

Project name:	**1315 Bough Beeches Blvd, Mississauga**	Drilling Method:	Mud Rotary Drilling
Borehole name:	**BH104**	Test depth:	9.9 m
Test date: (dd/mm/yyyy)	17/12/2025	Test Elev:	139.6 m
Test number:	25-122 BH104 32	Poisson's ratio:	0.35
Probe Designation	N Probe (76 mm OD)	Probe initial volume:	1602 cm^3

Raw Readings		**Corrected Readings**			
Pressure kPa	**Volume cm³**	**Pressure kPa**	**Volume cm³**	**DR/R_0 %**	
43	-2.8	142	-4.0	-0.13	
49	41.3	141	39.8	1.24	
54	81.1	140	79.5	2.45	
58	120.7	141	119.0	3.65	
62	159.6	142	157.7	4.81	
67	201.2	145	199.2	6.03	
73	240.9	148	238.8	7.19	
84	281.8	157	279.3	8.37	
95	321.2	166	318.4	9.48	
110	359.9	180	356.6	10.57	
133	399.9	201	396.0	11.67	
168	441.0	235	436.0	12.79	
221	481.9	286	475.4	13.87	
295	519.3	359	510.6	14.83	
420	564.9	483	552.5	15.97	
574	601.2	636	584.3	16.82	
775	640.3	836	617.4	17.70	
1028	681.2	1089	650.9	18.58	◄
1305	719.9	1365	681.3	19.38	
1642	761.3	1701	714.6	20.25	
1967	800.3	2025	748.2	21.12	◄
2313	840.2	2371	783.3	22.02	
2610	880.4	2667	819.7	22.95	
2903	919.6	2960	855.5	23.85	
3186	959.3	3242	892.0	24.77	
2613	955.0	2669	894.2	24.82	
2089	947.0	2145	893.1	24.79	
1595	934.4	1651	888.5	24.68	◄Er
2244	943.1	2300	887.2	24.65	
2616	950.3	2672	889.5	24.70	
2961	958.9	3017	894.1	24.82	◄Er
3426	1000.4	3482	930.7	25.73	
3663	1039.8	3719	967.9	26.65	
3888	1080.6	3944	1006.5	27.60	



Time before recording readings : 15 sec.
Method for estimating Pl : 1/V vs P as per ASTM D4719

Interpreted Test Results

Epmt:	59,820 kPa
Ep-ur	585,750 kPa
Ey:	**120 MPa**
Ey-ur:	**1172 MPa**
Pl:	7,005 kPa
Ep / Pl:	8.5
Py:	2,025 kPa
Poh (est.) :	**142 kPa**
K_0 (est):	**0.84**

TEXAM COMPANION V.3.4.25

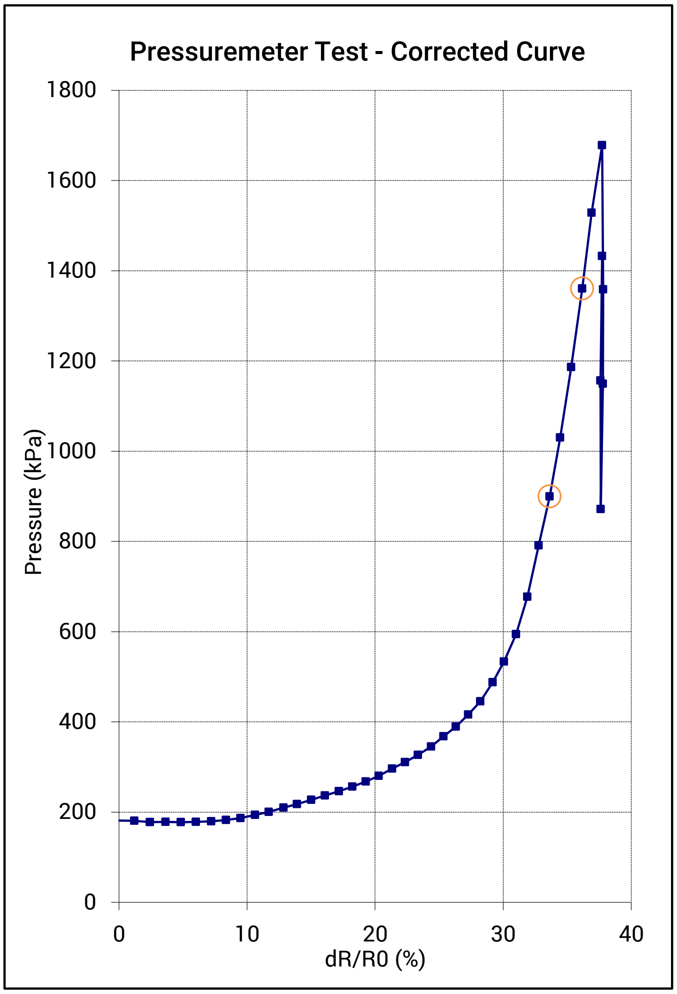
TEXAM Pressuremeter Test Results

GROUNDED ENGINEERING

Project name:	**1315 Bough Beeches Blvd, Mississauga**	Drilling Method:	Mud Rotary Drilling
Borehole name:	**BH104**	Test depth:	13.0 m
Test date: (dd/mm/yyyy)	17/12/2025	Test Elev:	136.5 m
Test number:	25-122 BH104 42	Poisson's ratio:	0.33
Probe Designation	N Probe (76 mm OD)	Probe initial volume:	1599 cm^3

Raw Readings		Corrected Readings			
Pressure kPa	**Volume cm³**	**Pressure kPa**	**Volume cm³**	**DR/R_0 %**	
55	-2.6	182	-4.2	-0.13	
61	40.4	181	38.6	1.20	
64	80.4	178	78.5	2.42	
68	120.6	179	118.6	3.64	
70	160.6	178	158.5	4.84	
73	199.9	178	197.8	6.00	
77	241.1	180	238.8	7.21	
82	280.9	183	278.4	8.36	
88	320.7	186	318.1	9.49	
97	360.8	194	357.9	10.63	
105	399.4	201	396.3	11.70	
116	441.2	210	437.7	12.86	
125	479.3	218	475.6	13.90	
136	520.5	228	516.5	15.02	
146	560.1	237	555.8	16.08	
157	601.0	247	596.3	17.17	
168	641.3	256	636.3	18.23	
180	681.8	268	676.4	19.29	
193	720.1	281	714.4	20.28	
210	761.3	296	755.1	21.33	
225	801.5	310	794.8	22.35	
242	841.4	327	834.2	23.35	
261	882.5	345	874.8	24.38	
284	921.8	368	913.4	25.34	
306	961.2	390	952.2	26.31	
333	1001.0	417	991.2	27.27	
362	1041.1	446	1030.4	28.23	
405	1081.7	488	1069.7	29.19	
451	1119.4	534	1106.0	30.06	
512	1161.2	595	1146.1	31.02	
595	1200.8	678	1183.2	31.90	
709	1240.6	791	1219.7	32.77	
818	1280.6	900	1256.5	33.63	◄
949	1320.2	1030	1292.1	34.46	
1106	1361.8	1187	1329.1	35.32	
1280	1403.8	1361	1366.0	36.17	◄
1448	1441.6	1529	1398.8	36.92	
1598	1479.9	1679	1434.0	37.72	
1278	1475.6	1359	1437.9	37.81	
1069	1469.1	1150	1437.6	37.80	
791	1453.5	872	1430.2	37.63	
1076	1460.8	1157	1429.0	37.61	
1352	1474.8	1433	1434.8	37.74	
1449	1481.3	1530	1438.5	37.82	
1768	1520.5	1849	1471.7	38.57	
1963	1561.0	2044	1508.9	39.41	
2144	1600.8	2225	1546.2	40.24	

Time before recording readings : 15 sec.
Method for estimating Pl : 1/V vs P as per ASTM D4719



Interpreted Test Results

Epmt:	32,596 kPa
Ep-ur	257,847 kPa
Ey:	**70 MPa**
Ey-ur:	**558 MPa**
Pl:	5,358 kPa
Ep / Pl:	6.1
Py:	1,361 kPa
Poh (est.) :	**180 kPa**
K_0 (est):	**0.89**

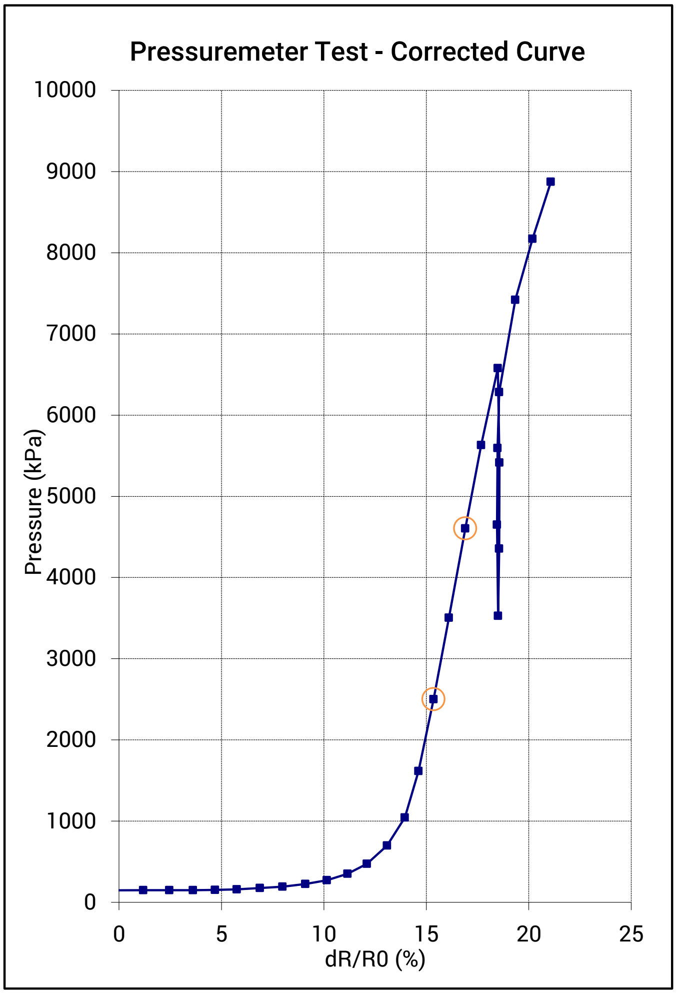
TEXAM COMPANION V.3.4.25

TEXAM Pressuremeter Test Results

GROUNDED
ENGINEERING

Project name:	**1315 Bough Beeches Blvd, Mississauga**	Drilling Method:	Mud Rotary Drilling
Borehole name:	**BH108**	Test depth:	11.4 m
Test date: (dd/mm/yyyy)	19/12/2025	Test Elev:	138.1 m
Test number:	25-122 - BH 108 - 37	Poisson's ratio:	0.33
Probe Designation	N Probe (76 mm OD)	Probe initial volume:	1667 cm^3

Raw Readings		Corrected Readings			
Pressure kPa	**Volume cm³**	**Pressure kPa**	**Volume cm³**	**DR/R_0 %**	
63	-2.3	148	-3.8	-0.11	
71	41.6	149	39.9	1.19	
77	84.9	148	83.0	2.46	
82	124.8	150	122.9	3.62	
88	161.9	152	159.8	4.68	
99	199.6	161	197.3	5.75	
116	240.2	175	237.4	6.88	
137	279.8	193	276.5	7.98	
171	321.3	225	317.2	9.10	
220	360.7	272	355.4	10.15	
303	399.9	353	392.6	11.15	
425	438.2	473	428.0	12.10	
653	480.3	700	464.6	13.08	
997	521.3	1042	497.4	13.95	
1573	560.1	1617	522.9	14.62	
2458	600.7	2501	551.1	15.35	◄
3463	640.3	3505	580.0	16.10	
4562	681.3	4603	611.0	16.90	◄
5589	720.5	5630	641.5	17.68	
6538	760.3	6578	673.6	18.49	
5375	753.7	5415	676.5	18.57	
4316	744.4	4356	676.2	18.56	
3487	734.5	3527	673.9	18.50	◄Er
4609	742.6	4649	671.9	18.45	
5553	751.4	5593	672.8	18.47	
6244	760.4	6284	676.1	18.56	◄Er
7381	800.3	7421	707.1	19.34	
8131	839.8	8170	740.8	20.18	
8834	881.0	8873	776.8	21.08	



Interpreted Test Results

Epmt:	209,589 kPa
Ep-ur	2,224,448 kPa
Ey:	**453 MPa**
Ey-ur:	**4810 MPa**
Pl:	19,979 kPa
Ep / Pl:	10.5
Py:	4,603 kPa
Poh (est.) :	**150 kPa**
K_0 (est):	**0.80**

Time before recording readings : 15 sec.

Method for estimating Pl : 1/V vs P as per ASTM D4719

TEXAM COMPANION V.3.4.25

TEXAM Pressuremeter Test Results

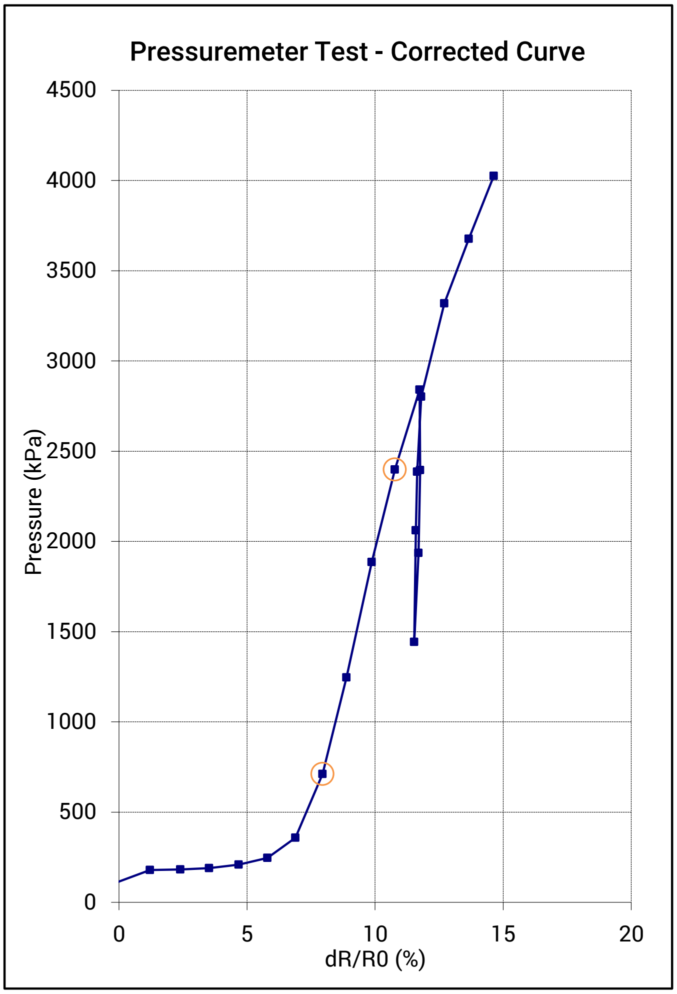
GROUNDED ENGINEERING

Project name:	**1315 Bough Beeches Blvd, Mississauga**	Drilling Method:	Mud Rotary Drilling
Borehole name:	**BH108**	Test depth:	14.5 m
Test date: (dd/mm/yyyy)	19/12/2025	Test Elev:	135.1 m
Test number:	25-122 - BH 108 - 47	Poisson's ratio:	0.33
Probe Designation	N Probe (76 mm OD)	Probe initial volume:	1664 cm^3

Raw Readings		Corrected Readings			
Pressure kPa	Volume cm³	Pressure kPa	Volume cm³	DR/R_0 %	
-1	-2.3	111	-2.3	-0.07	
74	41.3	180	40.5	1.21	
83	81.7	182	80.9	2.40	
94	120.2	190	119.3	3.52	
117	160.3	209	159.2	4.68	
157	200.2	246	198.7	5.80	
272	239.9	358	237.3	6.89	
628	281.1	711	275.1	7.95	◄
1165	319.8	1246	308.7	8.88	
1806	361.7	1885	344.5	9.86	
2321	399.9	2398	377.8	10.77	◄
2765	440.0	2840	413.6	11.74	
2320	436.3	2396	414.2	11.75	
1860	429.7	1936	412.0	11.69	
1367	419.0	1443	405.9	11.53	◄Er
1986	427.1	2062	408.2	11.59	
2311	432.3	2387	410.2	11.65	
2727	441.8	2802	415.8	11.80	◄Er
3245	480.7	3319	449.7	12.71	
3604	519.9	3677	485.5	13.66	
3955	560.4	4026	522.6	14.63	

Time before recording readings : 15 sec.

Method for estimating Pl : 1/V vs P as per ASTM D4719



Interpreted Test Results

Epmt:	86,956 kPa
Ep-ur	541,353 kPa
Ey:	**188 MPa**
Ey-ur:	**1171 MPa**
Pl:	7,103 kPa
Ep / Pl:	12.2
Py:	2,398 kPa
Poh (est.) :	**180 kPa**
K_0 (est):	**0.81**

TEXAM COMPANION V.3.4.25

APPENDIX D

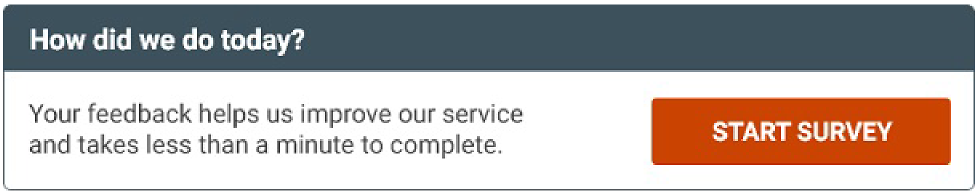
CORROSIVITY (SGS)

Report No.	**CA40000-JAN26**
Customer	**Grounded Engineering Inc.**
Attention	**Deepak Kanraj**
Reference	**25-122, Deepak Kanraj**
Works#	
Title	**Final Report**

Sample ID		Analysis Start Date	Analysis Start Time	Analysis Completed Date	Analysis Completed Time	BH 104 SS3	BH 108 SS5	BH 105 SS4
Sample Date / Time						10-Dec-25 10:00	18-Dec-25 13:00	22-Dec-25 12:00
Analysis	**Units**							
Corrosivity Index	none	09-Jan-26	12:23	09-Jan-26	12:23	13	10	11
Soil Redox Potential	mV	08-Jan-26	08:14	08-Jan-26	15:07	292	277	273
Sulphide (Na2CO3)	%	06-Jan-26	14:00	07-Jan-26	09:08	< 0.01	0.01	0.01
Moisture Index	No Units	09-Jan-26	12:07	09-Jan-26	12:22	0.0	0.0	1.0
pH	pH Units	06-Jan-26	14:00	07-Jan-26	09:08	8.84	8.14	8.06
Chloride	µg/g	07-Jan-26	11:06	09-Jan-26	15:01	580	460.0	670
Sulphate	µg/g	07-Jan-26	11:06	09-Jan-26	15:01	29	21.0	47
Conductivity	uS/cm	08-Jan-26	08:14	08-Jan-26	15:07	1180	846	1140
Resistivity (calculated)	ohms.cm	08-Jan-26	08:14	08-Jan-26	15:08	847	1180	881

INTERPRETATION

AWWA C-105 Standard		**Points**	**Points**	**Points**
% Moisture		**1**	**1**	**1**
pH				
Is pH bet 6.5-7.5 ?		NO	NO	NO
Is Redox Potential < 100 mv?		NO	NO	NO
Are Sulphides present ?		NO	YES	YES
If above three conditions are met, pH is assigned 3 points				
pH - Score		**3**	**0**	**0**
Redox Potential		**0**	**0**	**0**
Resistivity		**10**	**10**	**10**
Acid Volatile Sulphides		**0**	**0**	**0**
TOTAL SCORE (AWWA C-105)		**14**	**11**	**11**
Sample		**BH 104 SS3**	**BH 108 SS5**	**BH 105 SS4**
Corrosion Protection Recommended?		**YES**	**YES**	**YES**
Sulphate	**%**	0.003%	0.002%	0.005%
CLASS OF EXPOSURE		**Negligible**	**Negligible**	**Negligible**



How did we do today?

Your feedback helps us improve our service and takes less than a minute to complete.

FINAL REPORT

CA40000-JAN26 R1

25-122, 1315 Bough Beechas Blvd

Prepared for

Grounded Engineering Inc.

FINAL REPORT

First Page

CLIENT DETAILS

Client	Grounded Engineering Inc.
Address	49 Mobile Drive Toronto, Ontario M4A 1H5. Canada
Contact	Deepak Kanraj
Telephone	647-264-7909
Facsimile	
Email	dkanraj@groundedeng.ca
Works #	
Project	25-122, 1315 Bough Beechas Blvd
Reference	
Batch	
Samples	SOIL (3)

LABORATORY DETAILS

Project Specialist	Brad Moore Hon. B.Sc
Laboratory	SGS Canada Inc.
Address	185 Concession St., Lakefield ON, K0L 2H0
Telephone	705-652-2143
Facsimile	705-652-6365
Email	brad.moore@sgs.com
SGS Reference	CA40000-JAN26
Received	2026-01-05
Approved	01/09/2026
Report Number	CA40000-JAN26 R1
Date Reported	01/09/2026

COMMENTS

Temperature of Sample upon Receipt: 6 degrees C
Cooling Agent Present:Yes
Custody Seal Present:Yes

Chain of Custody Number:046043

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Brad Moore Hon. B.Sc

TABLE OF CONTENTS

MATRIX: SOIL

Sample Number	5	6	7
Sample Name	BH 104 SS3	BH 108 SS5	BH 105 SS4
Sample Matrix	Soil	Soil	Soil
Sample Date	2025-12-10 00:00	2025-12-18 00:00	2025-12-22 00:00

Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
Corrosivity Index						
Corrosivity Index	none	1		13	10	11
pH	pH Units	0.05		8.84	8.14	8.06
Soil Redox Potential	mV	no		292	277	273
Sulphide (Na2CO3)	%	0.01		< 0.01	0.01	0.01
Resistivity (calculated)	ohms.cm	-9999		847	1180	881
General Chemistry						
Conductivity	uS/cm	2		1180	846	1140
Metals and Inorganics						
Sulphate	µg/g	0.4		29	21	47
Other (ORP)						
Chloride	µg/g	0.4		580	460	670

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0122-JAN26	µg/g	0.4	<0.4	0	35	100	80	120	98	75	125
Sulphate	DIO0122-JAN26	µg/g	0.4	<0.4	24	35	96	80	120	96	75	125

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide (Na2CO3)	ECS0015-JAN26	%	0.01	< 0.01								

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0103-JAN26	uS/cm	2	< 2	0	20	99	90	110	NA		

FINAL REPORT

QC SUMMARY

pH

Method: SM 4500 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0103-JAN26	pH Units	0.05	NA	1		101			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS	Insufficient sample for analysis.
RL	Reporting Limit.
↑	Reporting limit raised.
↓	Reporting limit lowered.
NA	The sample was not analysed for this analyte
ND	Non Detect

-- End of Analytical Report --

SGS Industries & Environment - Lakefield: 185 Concession St., Lakefield, ON K0L 2H0 Phone: 705-652-2000 Fax: 705-652-6365 Web: www.sgs.com/environment
- London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361

Request for Laboratory Services and CHAIN OF CUSTODY

No: 046043

Page 1 of 1

Laboratory Information Section - Lab use only

Received By: EO
Received Date: 1 5 26 (mm/dd/yy)
Received Time: 1030 (hr : min)

Received By (signature):
Custody Seal Present: Yes ☑ No ☐
Custody Seal Intact: Yes ☑ No ☐

Cooling Agent Present: Yes ☑ No ☐ Type: ICE
Temperature Upon Receipt (°C) 6X3 ,

LAB LIMS #: CA40000Jn25

REPORT INFORMATION

Company: Grounded Engineering
Contact: Deepak Kanraj
Address: 49 Mobile Dr
NorthYork ON M4A 1H5
Phone: (647) 265-0912
Fax:
Email: dkanraj@groundedeng.ca

INVOICE INFORMATION

☒ (same as Report Information)
Company:
Contact:
Address:
Phone:
Email:

Quotation #:
P.O. #:
Project #: 25-122
Site Location/ID: 1315 Bough Beeches Blvd

TURNAROUND TIME (TAT) REQUIRED

☐ Client Regular TAT ☒ Regular TAT (5-7days)

TAT's are quoted in business days (exclude statutory holidays & weekends). Samples received after 6pm or on weekends: TAT begins next business day

RUSH TAT (Additional Charges May Apply): ☐ 1 Day ☐ 2 Days ☐ 3 Days ☐ 4 Days

PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION

Specify Due Date:

*NOTE: DRINKING (POTABLE) WATER SAMPLES FOR HUMAN CONSUMPTION MUST BE SUBMITTED WITH SGS DRINKING WATER CHAIN OF CUSTODY

REGULATIONS

☐ O.Reg 153/04
☐ Table 1 ☐ Res/Park
☐ Table 2 ☐ Ind/Com
☐ Table 3 ☐ Agri/Other
☐ Table ______ Appx. ______
Soil Texture: ☐ Coarse ☐ Medium/Fine
Soil Volume ☐ <350m3 ☐ >350m3

☐ O.Reg 406/19

Other Regulations:
☐ Reg 347/558 (3 Day min TAT)
☐ PWQO ☐ MMER
☐ CCME ☐ Other:
☐ MISA
☐ ODWS Not Reportable *See note

Sewer By-Law:
☐ Sanitary
☐ Storm
Municipality:

RECORD OF SITE CONDITION (RSC) ☒ YES ☐ NO

ANALYSIS REQUESTED

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	# OF BOTTLES	MATRIX	M & I: Field Filtered (Y/N)	M & I: Metals & Inorganics incl CrVI, CN,Hg pH,(B(HWS),EC,SAR-soil) (Cl, Na-water)	M & I: Full Metals Suite ICP metals plus B(HWS-soil only) Hg, CrVI	M & I: ICP Metals only Sb,As,Ba,Be,B,Cd, Cr,Co,Cu,Pb,Mo,Ni,Se,Ag,Tl,U,V,Zn	SVOC: PAHs only	SVOC: SVOCs all incl PAHs, ABNs, CPs	PCB: PCBs ☐ Total ☐ Aroclor	PHC: F1-F4 + BTEX	PHC: F1-F4 only no BTEX	VOC: VOCs all incl BTEX	VOC: BTEX only	Pest: Pesticides Organochlorine or specify other	Other (please specify)	Other: Corrosivity	Other	Other	Sewer Use: Specify pkg:	Water Characterization Pkg ☐ General ☐ Extended	SPLP Specify tests ☐ Metals ☐ VOC ☐ 1,4-Dioxane ☐ OCP ☐ ABN	TCLP Specify tests ☐ M&I ☐ VOC ☐ PCB ☐ B(a)P ☐ ABN ☐ Ignit.	COMMENTS:
1	BH 104 SS3	12-10-25	1100	1	SOIL														X							
2	BH 108 SS5	12-18-25	1300	↓	↓														X							
3	BH 105 SS4	12-22-25	1200	↓	↓														X							
4																										
5																										
6																										
7																										
8																										
9																										
10																										
11																										
12																										

Observations/Comments/Special Instructions

Sampled By (NAME): Christopher Hartley Signature: Date: 01/02/26 (mm/dd/yy) Pink Copy - Client

Relinquished by (NAME): Christopher Hartley Signature: Date: 01/02/26 (mm/dd/yy) Yellow & White Copy - SGS

Revision #: 1.8
Date of Issue: 06 SEP 2024

APPENDIX E

GEOPHYSICS GPR
INTERNATIONAL INC.

March 9th , 2026 Transmitted by email:kbyckalo@groundedeng.ca
Ref.: GPR-26-6926

Kyle Byckalo, PEng
Associate
Geotechnical Engineering Services
Grounded Engineering Inc.
49 Mobile Drive,
Toronto, ON,
M4A 1H5

Subject: Shear Wave Velocity Survey for Seismic Site Class Determination at 1315 Bough Beeches Blvd, Mississauga, Ontario

Dear Kyle

Geophysics GPR International Inc. has been requested by Grounded Engineering Inc. to carry out seismic shear-wave velocity (V_s) measurements at 1315 Bough Beeches Blvd, Mississauga, Ontario (Figure 1/ Figure 2).

The data collection was carried out February 17, 2026, by the GPR Field crew led by Duro Zeljkovic. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spread. Figures are presented in the Appendix.

The geophysical investigation used the Multi-channel Analysis of Surface Waves (MASW), the Spatial AutoCorrelation (SPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocity values were calculated for the soil and/or rock, to determine the seismic site classification and provide depth to bedrock information.

The following report describes the survey design, the principles of the test methods, the methodology for interpreting the data, and a culmination of the results in table and chart formats.

MASW Principle

The Multi-channel Analysis of Surface Waves (MASW) and the SPatial AutoCorrelation (SPAC or MAM for Microtremors Array Method) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface wave. MASW is considered an "active" method, as the seismic signal is induced at known location and time in the vibration sensors' (geophones) array axis. Conversely, the SPAC method is considered a "passive" method, using the low frequency "signals" produced far away. The method can also be used with "active" seismic source records. The SPAC method generally allows deeper V_s soundings. Its dispersion curve can then be merged with the one of higher frequency from the MASW analysis to calculate a more complete inversion. The dispersion properties are expressed as a change of velocities with respect to frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_s) velocity depth profile (sounding).

Figure 3 outlines the basic operating procedure for the MASW method. Figure 4 illustrates an example of one of the MASW/SPAC records, the corresponding spectrogram analysis and resulting 1D V_s model.

Survey Design

The seismic investigation consisted of a linear array of 24 x 4.5Hz geophones connected to an ABEM Terraloc Pro2 or equivalent seismograph. An elastic accelerated weight-drop and/or sledgehammer was used as the primary energy source with traces typically being recorded at 6 locations: approximately 6 m off both ends, 25 to 30 m off both ends, and in the middle of the spread. Data were collected using arrays with geophone spacings of 3m and 1m for a total of 10 shot records per sounding.

Unlike the seismic refraction method, which produces a data point beneath each geophone, the shear-wave depth profile is the average of the bulk area within the middle third of the geophone spread.

The theoretical maximum depth of penetration (34.5m) for the MASW method is half of the maximum seismic array length (69 m), in practice the maximum depth of penetration is often influenced by the geology. The SPAC method in some cases can resolve greater depths.

Shear-wave Velocity Interpretation Method and Accuracy of the Results

The main processing sequence involved data inspection and editing when required; spectral analysis ("phase shift" for MASW, and "cross-correlation" for SPAC); picking the fundamental mode; and 1D inversion of the MASW and SPAC shot records using the SeisImagerSW™ software and/or MASwAI from Geophysics GPR.

Assuming all layers are flat, horizontal, and laterally homogeneous, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, local surface seismic velocities variations, and/or dipping of overburden layers or rock. In general, the precision of the calculated seismic shear wave velocities (V_s) is of the order of 15% or better.

The results of the inversion process are inherently non-unique, and the final model must be judged to be geologically realistic. Additionally, the inversion model is interpreted as a lateral average of the studied profiles, represented as a single column located at the centre of the survey area.

More detailed descriptions of these methods are presented in Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock, Geological Surveys of Canada, General Information Product 110, 2015.

Seismic Refraction

The seismic refraction method relies on measuring the transit time of the wave that takes the shortest time to travel from the shot-point to each geophone. The fastest seismic waves are the compressional (P) or acoustic waves, where displaced particles oscillate in the direction of wave propagation.

The seismic refraction data used the same survey design as described above with minor variations in shot locations and recording parameters. The analysis was completed using the Hawkins' and critical distance methods.

The Hawkins' method allows the computation of the rock depth to every geophone. This method provides information on the thickness of the various overburden layers, depth to bedrock and rock quality. It is based on the closure times of the inner shots. It can calculate the true velocities of the rock using the apparent velocities, measured with information provided by the outer shots. A basic description of the Hawkins' method can also be found in the article Seismic Refraction Surveys for Civil Engineering by L. Hawkins (1961).

The accuracy of the method is typically on the order of +/-10%. The presence of velocity inversions (lower velocities underlying higher velocity layers due to softer/looser material at depth) and/or hidden layers can impact this accuracy.

Results

The results of this investigation are presented as 1D shear-wave velocity profiles in Figure 5 and in Table 1. The interpreted bedrock profile along the alignment is presented in Figure 6.

The chart in Figure 5 plots the average V_s values versus depth along with the minimum and maximum modelled envelopes. The spread of the minimum to maximum envelopes provides a visual representation of the confidence and variability in the results.

The V_{s30} value is calculated from the harmonic mean of the shear wave velocities, between the surface to 30 m below grade. It is calculated by dividing the total depth of interest (30 m) by the sum of the time spent in each velocity layer from the surface down to 30 m, as:

$$\bar{V}_{\mathrm{S30}} = \frac{\sum_{i=1}^{\mathrm{N}} \mathrm{H}_i}{\sum_{i=1}^{\mathrm{N}} {}^{H_i}/_{V_i}} \;\; | \;\; \sum_{i=1}^{\mathrm{N}} \mathrm{H}_i \; = \; 30 \text{ m}$$

(N: number of layers; H_i : thickness of layer "*i*" ; V_i : V_S of layer "*i*")

Thus, the V_{s30} value represents the seismic shear wave velocity of an equivalent homogeneous single layer response for the upper 30 m.

Based on the above formula, the average Vs30 value at this test location is calculated as 529 m/s over the depth interval of 0 to 30 m below grade (as determined through the MASW/SPAC and/or MAM methods). The minimum and maximum envelopes of the calculation over the same depth interval are 462 m/s and 583 m/s respectively.

Conclusions

A non-invasive geophysical survey was carried out to measure shear-wave velocities for seismic site classification at 1315 Bough Beeches Blvd, Mississauga, Ontario (Figure 2). The seismic survey used the MASW, MAM and the SPAC analysis methods to model the shear-wave velocities used in the calculation of the V_{s30} value. Its calculation is presented in Table 1 and summarized below.

The seismic refraction analysis indicates bedrock along the seismic alignment dipping from approximately 17m at the south end to approximately 27m at the north end. The compressional wave velocities suggest the material is predominately hard/dense. Weathered bedrock may be present above the indicated depths. The interpreted bedrock profile is presented in Figure 6. Borehole information provided by the client indicates shale bedrock at depths between 14m and 24m below grade.

Sounding	V_{s30}	Site Class	Site Designation
#1	529 m/s	C	X_{529}

The site classification provided in this report is based solely on the V_{s30} value as derived from non-invasive surface seismic methods and can be superseded by other geotechnical information. This geotechnical information includes, but is not limited to, variations in the thickness of the overburden within the building footprint, the presence of sensitive and/or liquefiable soils, more than 3m of soft clays, high moisture content, etc. The reader is referred to Table 4.1.8.4.A/B of the NBC, the 2020 NBC Structural Commentaries and the Ontario 2024 Building Code Compendium.

The V_S value calculated is representative of the in-situ materials and are not corrected for the total and effective stresses.

This report has been prepared by Ben McClement, P.Eng.

Ben McClement, P.Eng.

LICENSED PROFESSIONAL ENGINEER
R.B. McCLEMENT
2026-03-09
PROVINCE OF ONTARIO

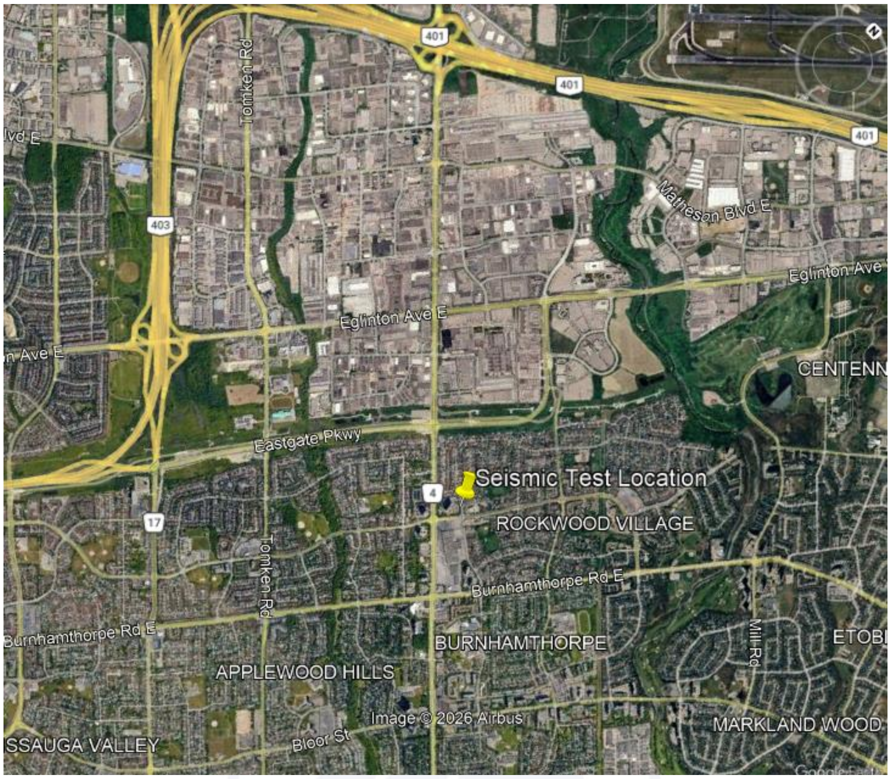


Figure 1: Regional location of the Site
(Source: *Google Earth™*)

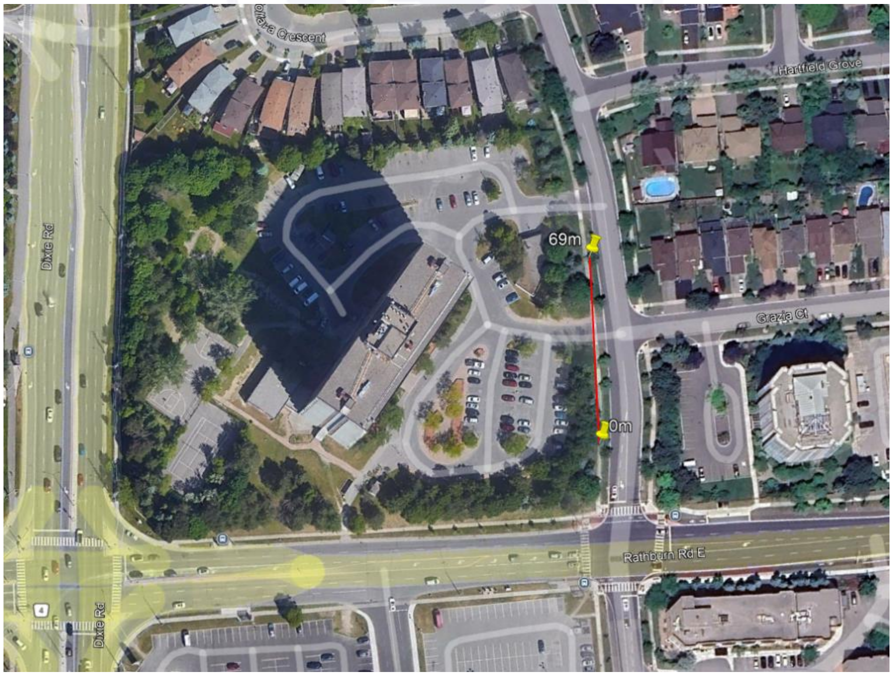


Figure 2: Location of the seismic spread
(Source: *Google Earth™*)

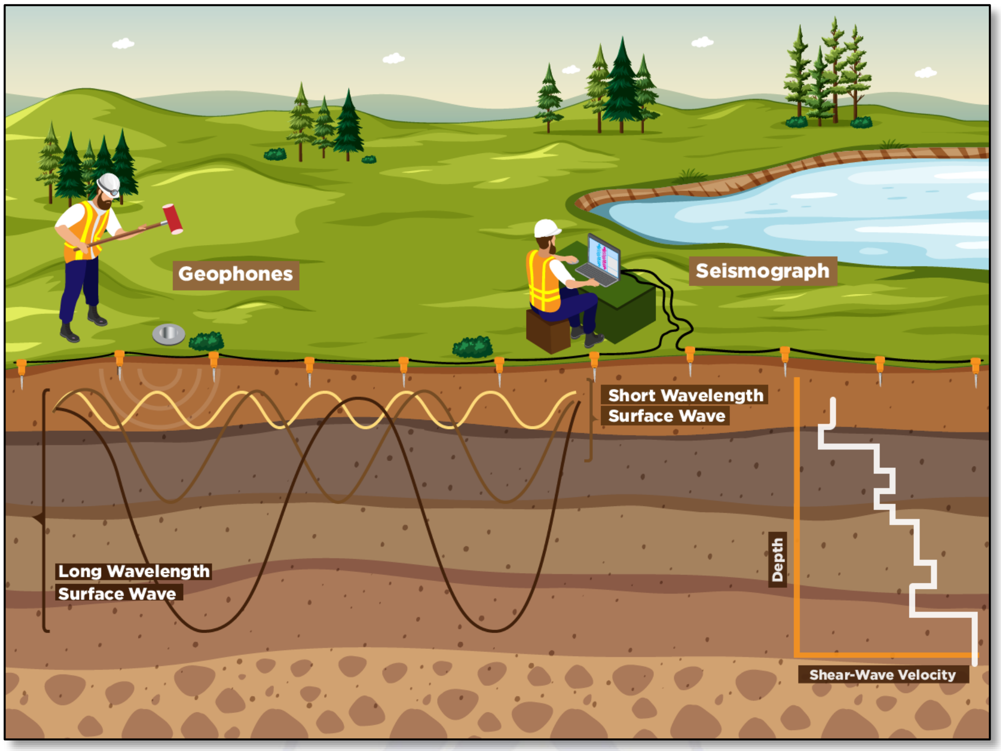


Figure 3: MASW Operating Principle

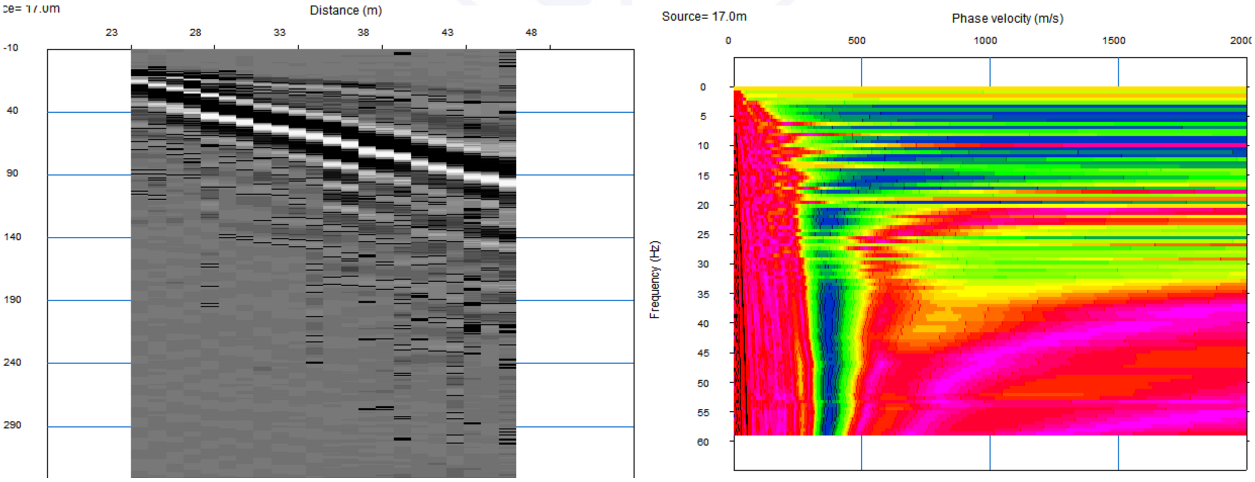


Figure 4: Example

of a MASW/SPAC record and Phase Velocity - Frequency curve of the Rayleigh wave

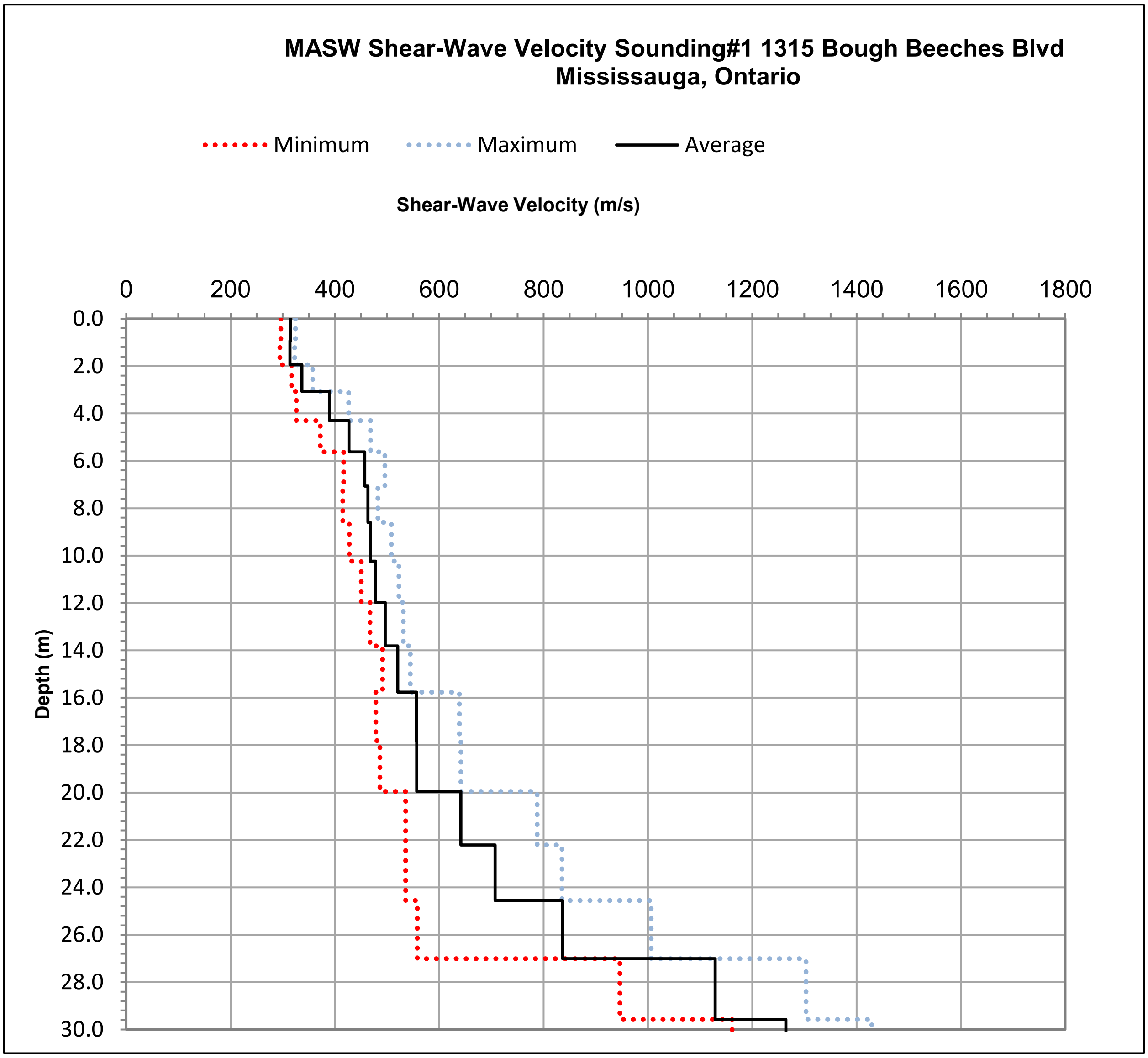


Figure 5: Shear-Wave Velocity Inversion Model from MASW/SPAC

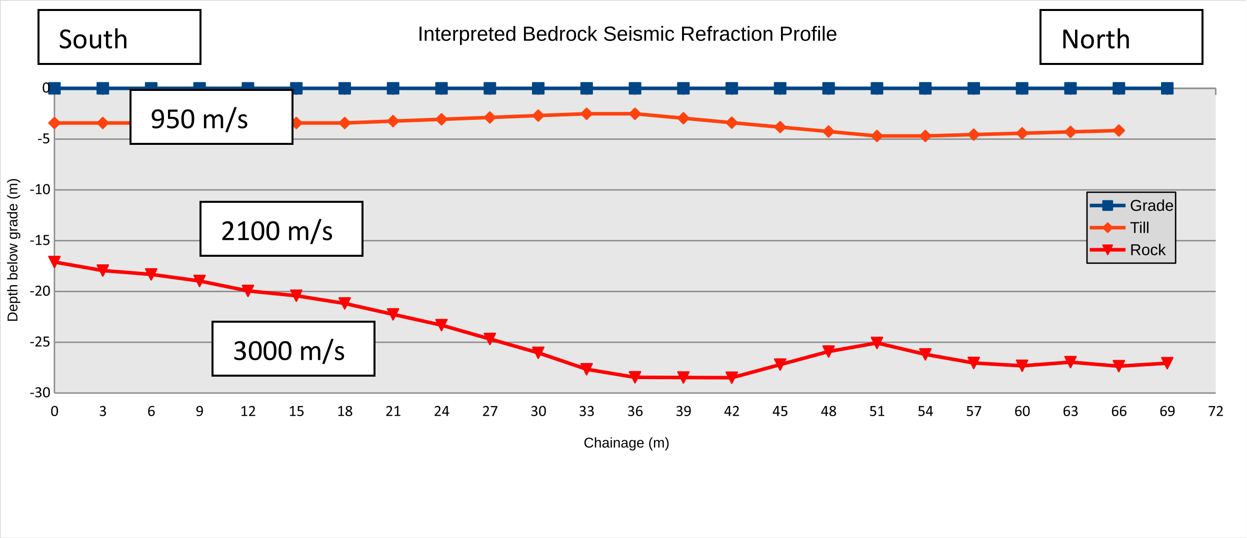
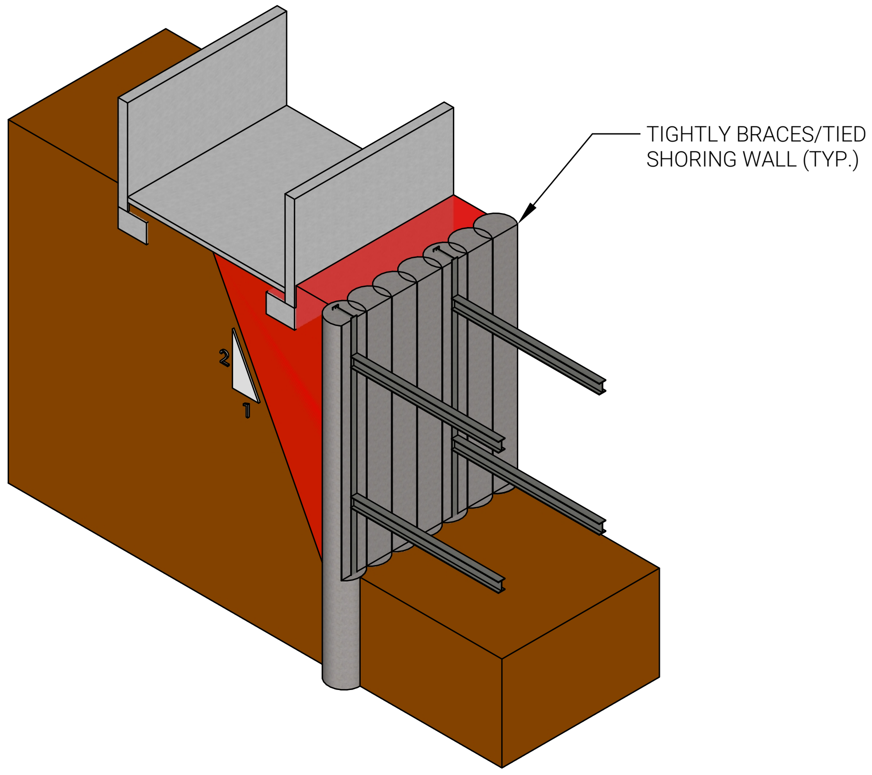


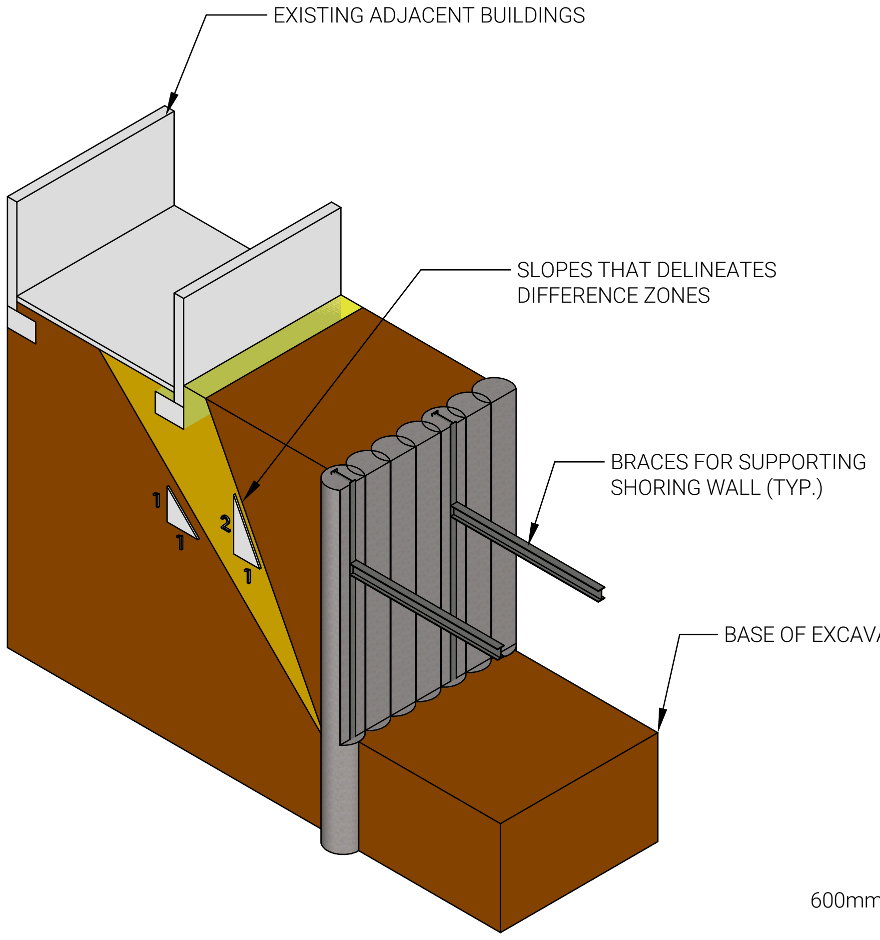
Figure 6: Interpreted bedrock profile from seismic refraction analysis with P-wave Velocities

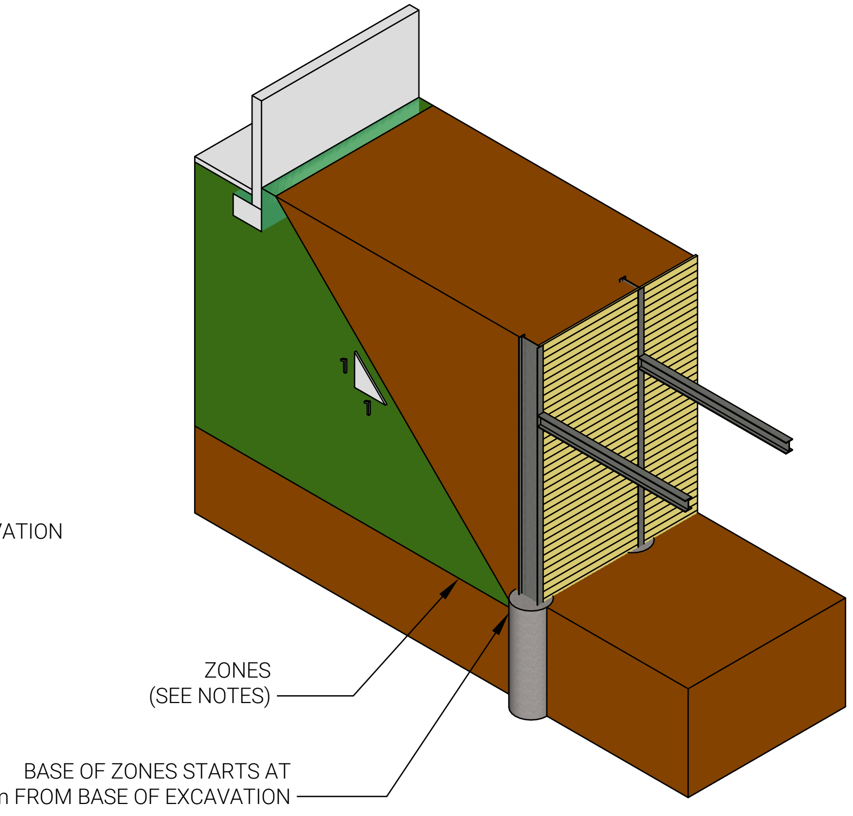
TABLE 1: V$_{S30}$ Calculation for the Site Class

Depth	Vs			Thickness	Cumulative Thickness	Delay for Vs Mean	Cumulative Delay	Vs Mean at given Depth
	Min.	Mean	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0.0	296	315	324	Grade Level				
0.9	295	314	323	0.9	0.9	0.002924	0.002924	315
1.9	317	337	357	1.0	1.9	0.003260	0.006184	314
3.1	326	390	426	1.1	3.1	0.003345	0.009529	322
4.3	372	427	468	1.2	4.3	0.003152	0.012681	339
5.6	417	457	496	1.3	5.6	0.003116	0.015797	356
7.1	415	463	482	1.4	7.1	0.003134	0.018931	373
8.6	428	468	508	1.5	8.6	0.003313	0.022244	386
10.2	451	478	523	1.6	10.2	0.003498	0.025742	398
12.0	467	496	531	1.7	12.0	0.003642	0.029383	407
13.8	491	521	545	1.8	13.8	0.003712	0.033095	417
15.8	479	556	639	1.9	15.8	0.003736	0.036831	428
17.8	487	557	642	2.0	17.8	0.003680	0.040511	440
20.0	536	642	788	2.1	20.0	0.003858	0.044369	450
22.2	536	707	835	2.3	22.2	0.003508	0.047877	464
24.6	558	836	1006	2.4	24.6	0.003328	0.051205	480
27.0	946	1129	1303	2.5	27.0	0.002937	0.054143	499
29.6	1161	1264	1429	2.6	29.6	0.002267	0.056409	524
30.0				0.4	30.0	0.000335	0.056745	529

APPENDIX F







ZONE A (RED)

FOUNDATIONS WITHIN THIS ZONE OFTEN REQUIRE UNDERPINNING OR SHORING SYSTEM. HORIZONTAL AND VERTICAL PRESSURES ON EXCAVATION WALL OF NON-UNDERPINNED FOUNDATION MUST BE CONSIDERED

ZONE B (YELLOW)

FOUNDATIONS WITHIN THIS ZONE OFTEN DO NOT REQUIRE UNDERPINNING BUT MAY REQUIRE SHORING SYSTEM. HORIZONTAL AND VERTICAL PRESSURES ON EXCAVATION WALL OF NON-UNDERPINNED FOUNDATION MUST BE CONSIDERED

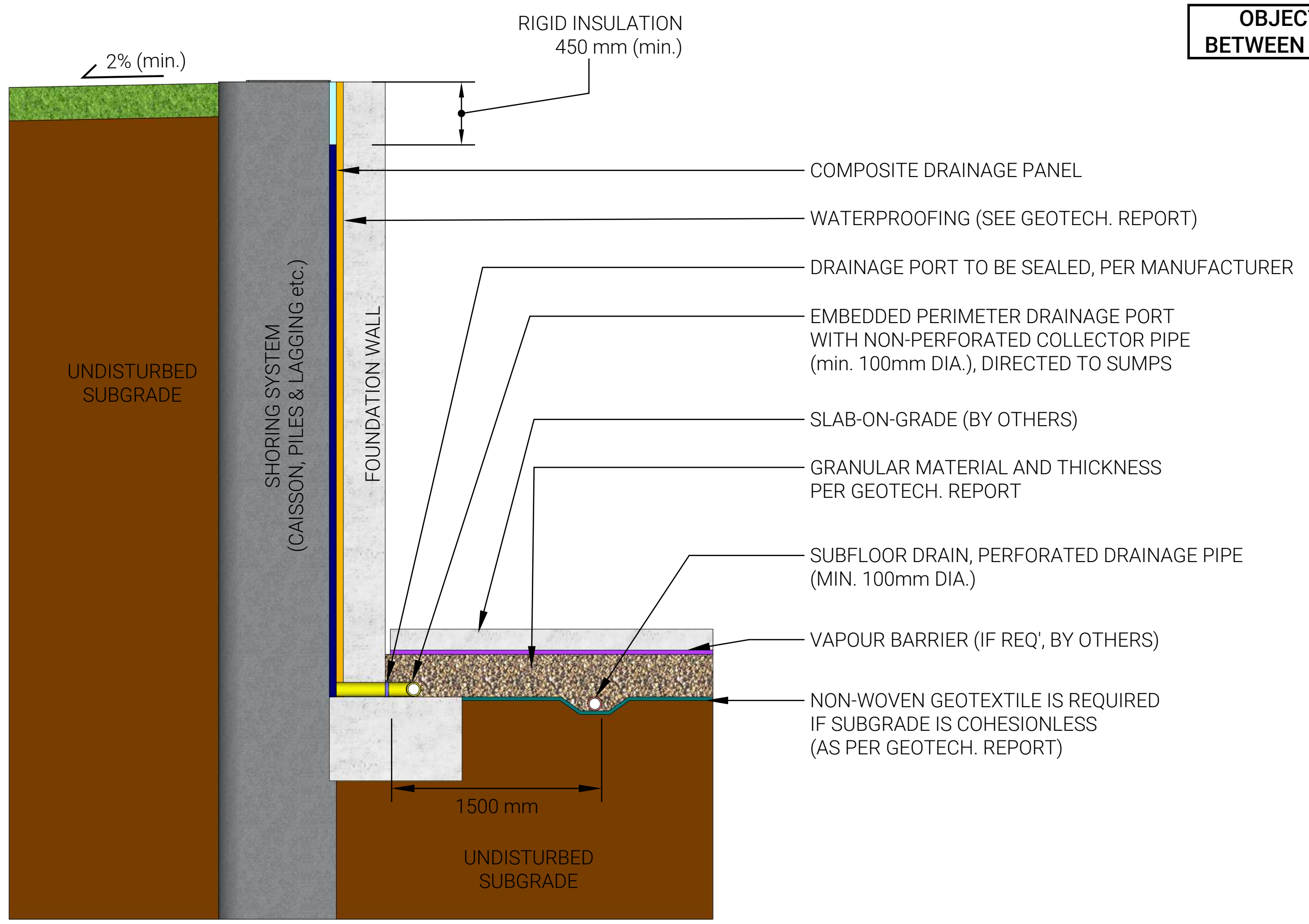
ZONE C (GREEN)

FOUNDATIONS WITHIN THIS ZONE USUALLY DO NOT REQUIRE UNDERPINNING OR SHORING SYSTEM

NOTES:
1. USER'S GUIDE - NBC 2005 STRUCTURAL COMMENTARIES (PART 4 OF DIVISION B) - COMMENTARY K.

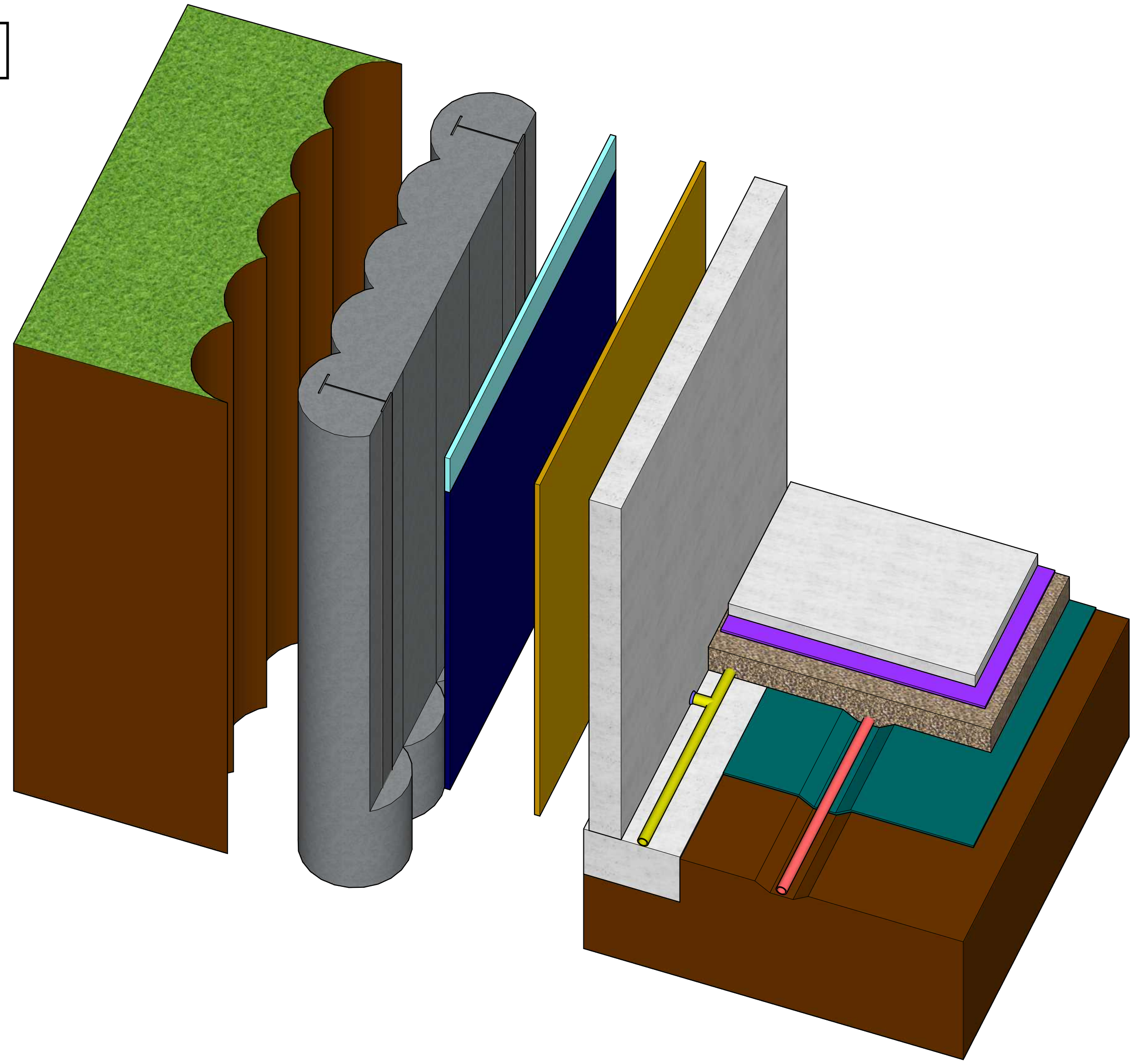
GROUNDED
ENGINEERING

Title
EXCAVATION ZONE OF INFLUENCE GUIDELINES



SECTIONAL VIEW

OBJECTS ARE COLOR-CODED BETWEEN TWO VIEWS FOR CLARITY



ISOMETRIC VIEW

SUBFLOOR DRAINAGE SYSTEM

1. THE SUBFLOOR DRAINS SHOULD BE SET IN PARALLEL ROWS, IN ONE DIRECTION, AND SPACED AS PER THE GEOTECHNICAL REPORT.
2. THE INVERT OF THE PIPES SHOULD BE A MINIMUM OF 300mm BELOW THE UNDERSIDE OF THE SLAB-ON-GRADE.
3. A CAPILLARY MOISTURE BARRIER (I.E. DRAINAGE LAYER) CONSISTING OF A MINIMUM 200 mm LAYER OF CLEAR STONE (OPSS MUNI 1004) COMPACTED TO A DENSE STATE (OR AS PER THE GEOTECHNICAL REPORT). WHERE VEHICULAR TRAFFIC IS REQUIRED, THE UPPER 50 mm OF THE CAPILLARY MOISTURE BARRIER MAY BE REPLACED WITH GRANULAR A (OPSS MUNI 1010) COMPACTED TO A MINIMUM 98% SPMDD.
4. A NON-WOVEN GEOTEXTILE MUST SEPARATE THE SUBGRADE FROM THE SUBFLOOR DRAINAGE LAYER IF THE SUBGRADE IS COHESIONLESS. THE NON-WOVEN GEOTEXTILE MAY CONSIST OF TERRAFIX 360R OR AN APPROVED EQUIVALENT.

PERIMETER DRAINAGE SYSTEM

1. FOR A DISTANCE OF 1.2m FROM THE BUILDING, THE GROUND SURFACE SHOULD HAVE A MINIMUM 2% GRADE.
2. PREFABRICATED COMPOSITE DRAINAGE PANEL (CONTINUOUS COVER, AS PER MANUFACTURER'S REQUIREMENTS) IS RECOMMENDED BETWEEN THE BASEMENT WALL AND RIGID SHORING WALL. THE DRAINAGE PANEL MAY CONSIST OF MIRADRAIN 6000 OR AN APPROVED EQUIVALENT.
3. PERIMETER DRAINAGE IS TO BE COLLECTED IN NON-PERFORATED PIPES AND CONVEYED DIRECTLY TO THE BUILDING SUMPS.
4. PERIMETER DRAINAGE PORTS SHOULD BE SPACED A MAXIMUM 3m ON-CENTRE. EACH PORT SHOULD HAVE A MINIMUM CROSS-SECTIONAL AREA OF 1500 mm2.

GENERAL NOTES

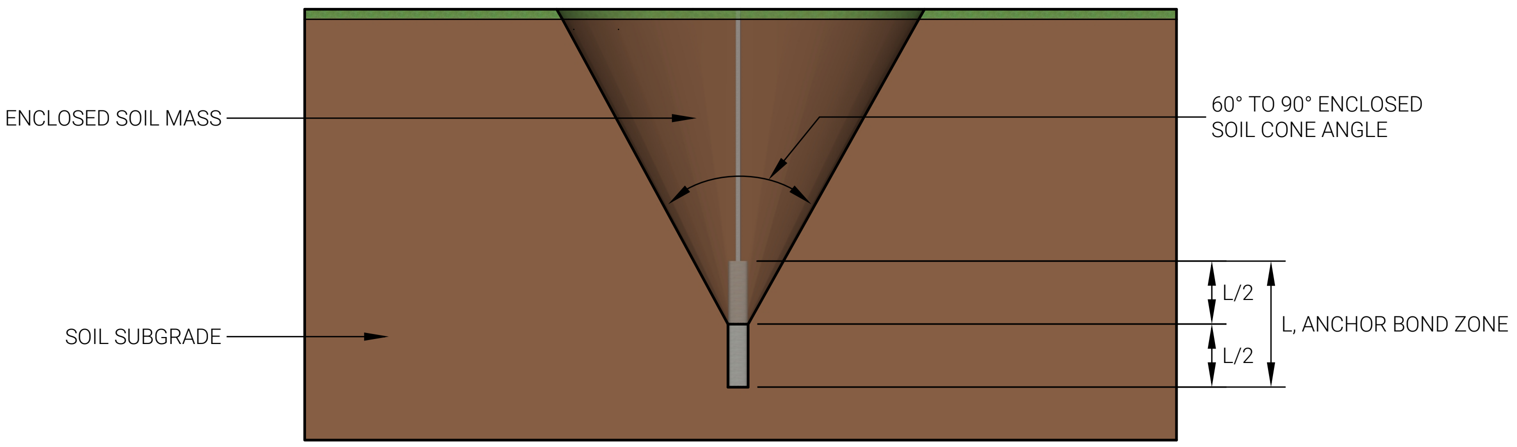
1. THERE SHOULD BE NO STRUCTURAL CONNECTION BETWEEN THE SLAB-ON-GRADE AND THE FOUNDATION WALL OR FOOTING.
2. THERE SHOULD BE NO CONNECTION BETWEEN THE SUBFLOOR AND PERIMETER DRAINAGE SYSTEMS.
3. THIS IS ONLY A TYPICAL BASEMENT DRAINAGE DETAIL. THE GEOTECHNICAL REPORT SHOULD BE CONSULTED FOR SITE SPECIFIC RECOMMENDATIONS.
4. THE FINAL BASEMENT DRAINAGE DESIGN SHOULD BE REVIEWED BY THE GEOTECHNICAL ENGINEER TO CONFIRM THE DESIGN IS ACCEPTABLE.

GROUNDED
ENGINEERING

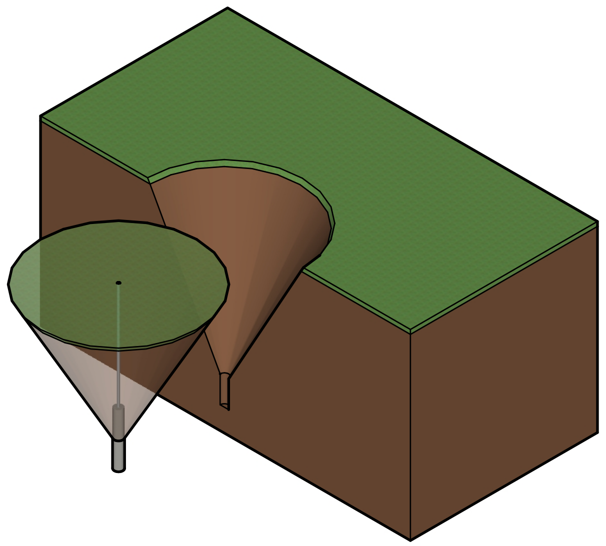
Title

BASEMENT DRAINAGE SHORING SYSTEM TYPICAL DETAILS

INDIVIDUAL SOIL TIEDOWN ANCHOR

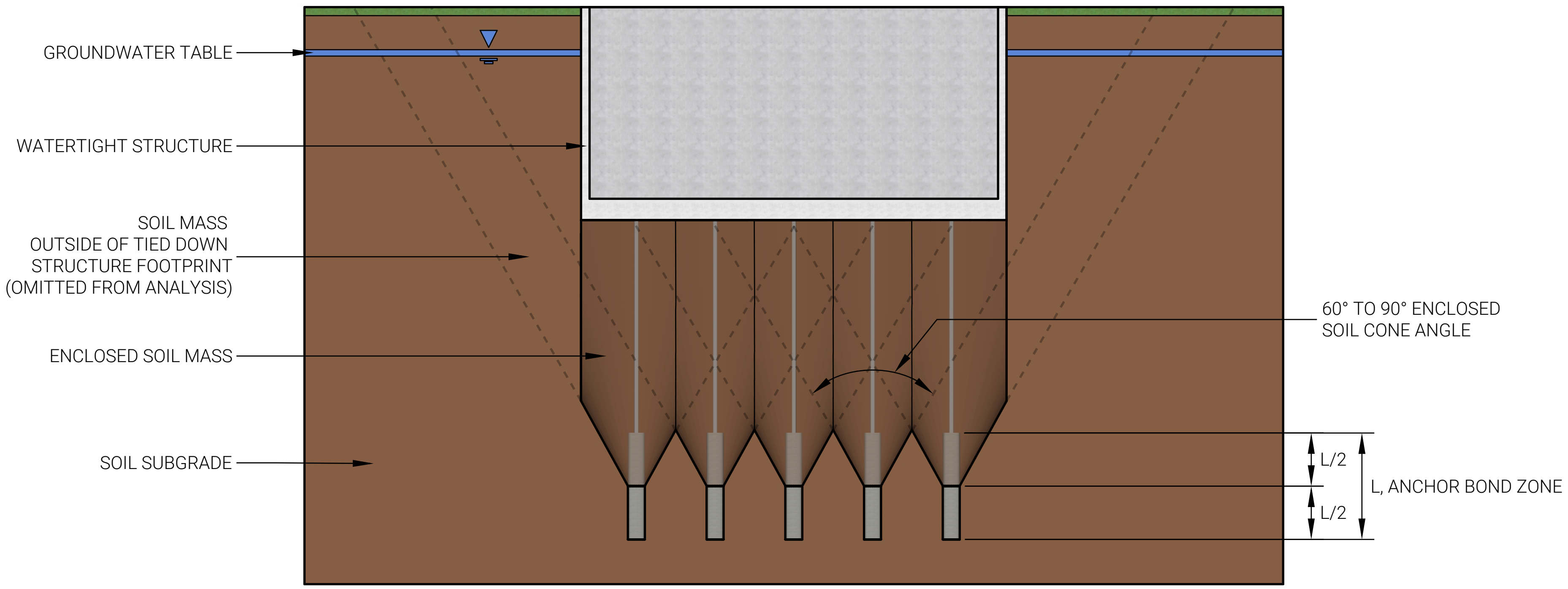


SECTIONAL VIEW

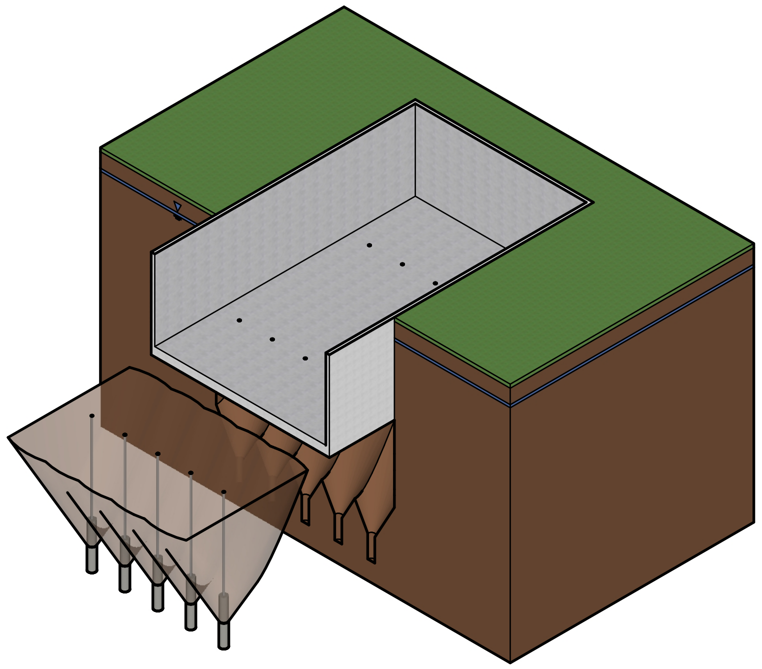


ISOMETRIC VIEW

GRID OF SOIL TIEDOWN ANCHORS WITH STRUCTURE



SECTIONAL VIEW



ISOMETRIC VIEW

NOTES:

1. UNFACTORED EQUILIBRIUM BETWEEN A STRUCTURE AND UPLIFT IS ESTABLISHED WHEN THE TOTAL WEIGHT OF THE STRUCTURE AND THE EFFECTIVE WEIGHT (CALCULATED USING γ') OF THE ENCLOSED SOIL MASS BELOW THE STRUCTURE IS EQUAL TO THE TOTAL UPLIFT PRESSURE (FHWA GEOTECHNICAL ENGINEERING CIRCULAR NO. 4 - GROUND ANCHORS AND ANCHORED SYSTEMS, 1999).
2. THE WEIGHT OF OVERLAPPING ENCLOSED SOIL MASSES MUST ONLY BE ACCOUNTED FOR ONCE.
3. THE WEIGHT OF SOIL OUTSIDE OF THE FOOTPRINT OF THE TIED DOWN STRUCTURE SHOULD BE NEGLECTED.

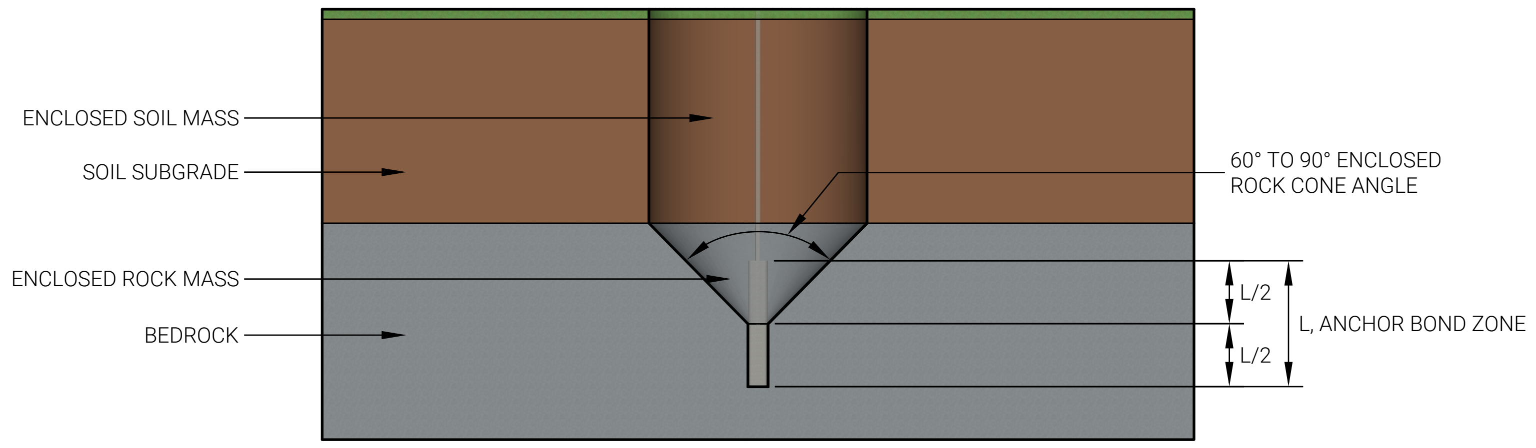
NOT TO SCALE. FEATURES ARE EXAGGERATED FOR DEMOSTRATION PURPOSES.

GROUNDED
ENGINEERING

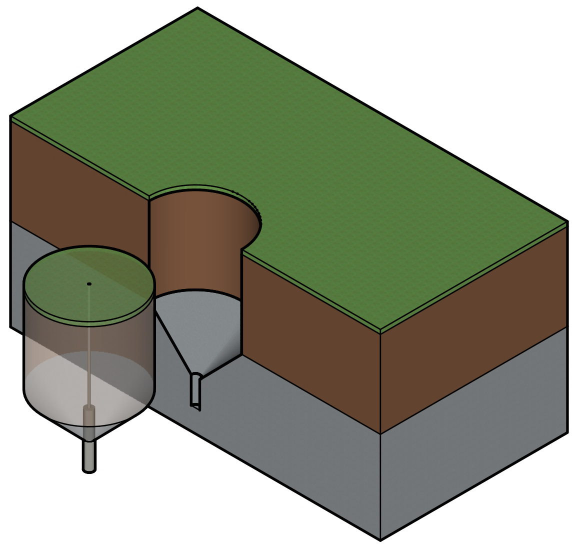
Title

SOIL TIEDOWN ANCHOR GLOBAL STABILITY

INDIVIDUAL ROCK TIEDOWN ANCHOR

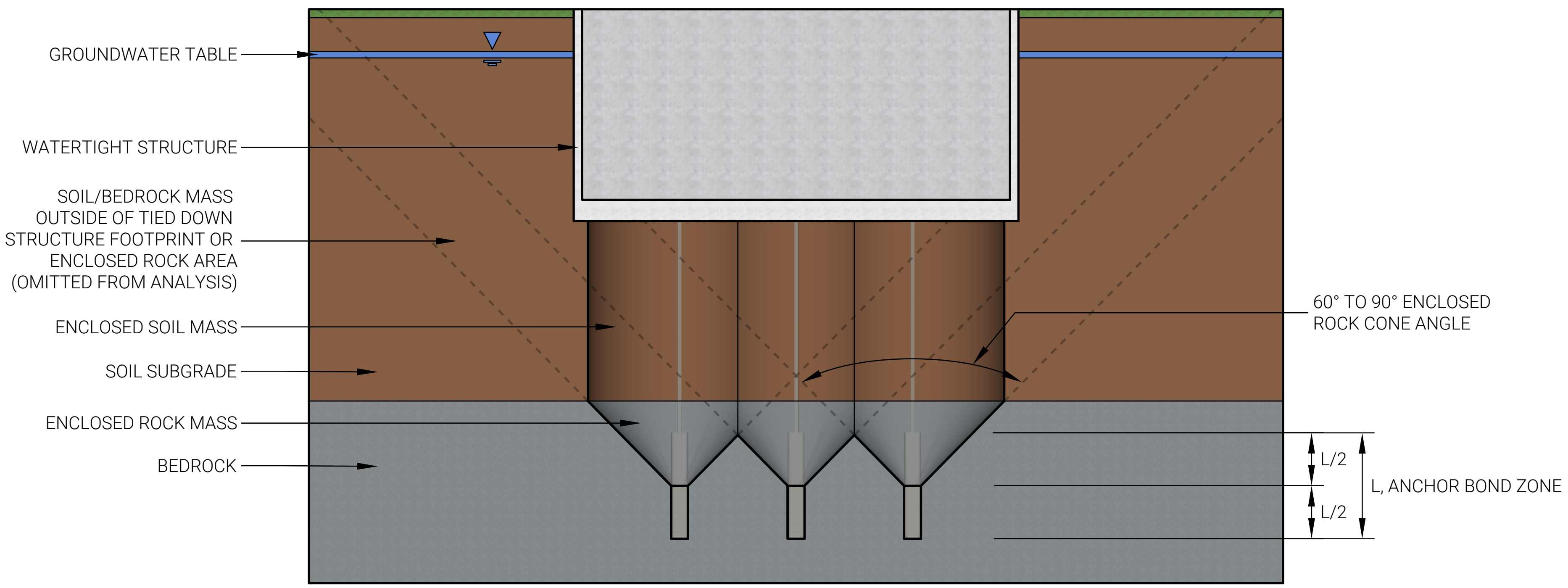


SECTIONAL VIEW

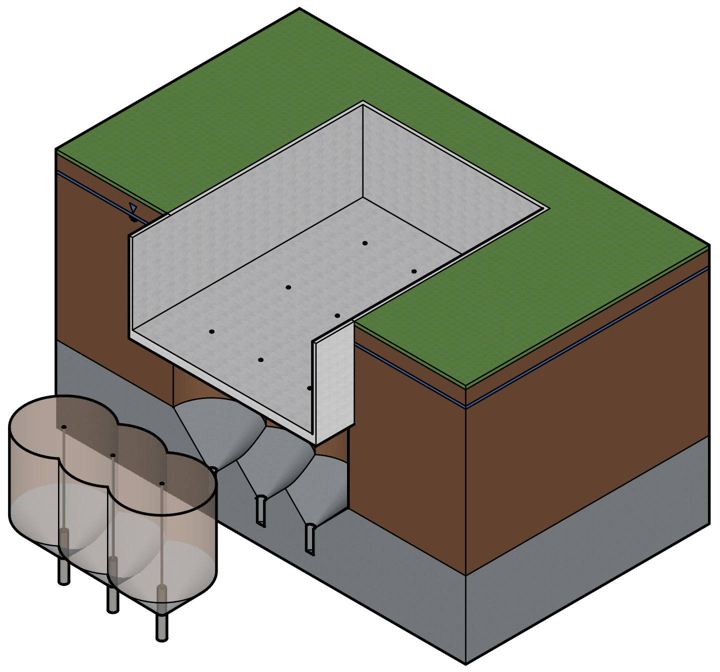


ISOMETRIC VIEW

GRID OF ROCK TIEDOWN ANCHORS WITH STRUCTURE



SECTIONAL VIEW



ISOMETRIC VIEW

NOTES:

1. UNFACTORED EQUILIBRIUM BETWEEN A STRUCTURE AND UPLIFT IS ESTABLISHED WHEN THE TOTAL WEIGHT OF THE STURCTURE AND THE EFFECTIVE WEIGHT (CALCULATED USING γ') OF THE ENCLOSED SOIL/ROCK MASS BELOW THE STRUCTURE IS EQUAL TO THE TOTAL UPLIFT PRESSURE (FHWA GEOTECHNICAL ENGINEERING CIRCULAR NO. 4 - GROUND ANCHORS AND ANCHORED SYSTEMS, 1999).
2. WHERE SOIL AND ROCK IS PRESENT, THE ENCLOSED ROCK MASS CONE EXTENDS FROM THE ANCHOR ZONE TO THE ROCK-SOIL TRANSITION. ENCLOSED SOIL MASS ONLY INCLUDES SOIL ABOVE THE RESULTING ROCK CONE.
3. THE WEIGHT OF OVERLAPPING ENCLOSED SOIL/ROCK MASSES MUST ONLY BE ACCOUNTED FOR ONCE.
4. THE WEIGHT OF SOIL/ROCK OUTSIDE OF THE FOOTPRINT OF THE TIED DOWN STRUCTURE SHOULD BE NEGLECTED.

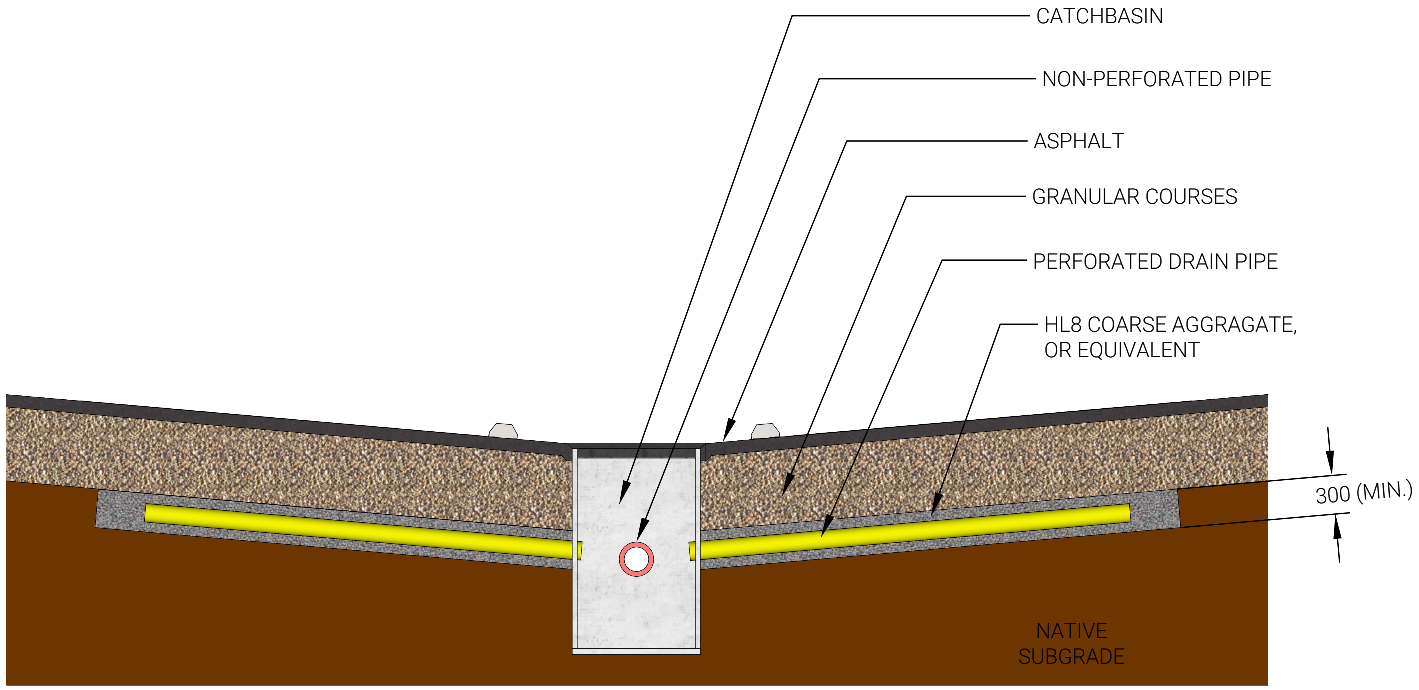
NOT TO SCALE. FEATURES ARE EXAGGERATED FOR DEMOSTRATION PURPOSES.

GROUNDED ENGINEERING

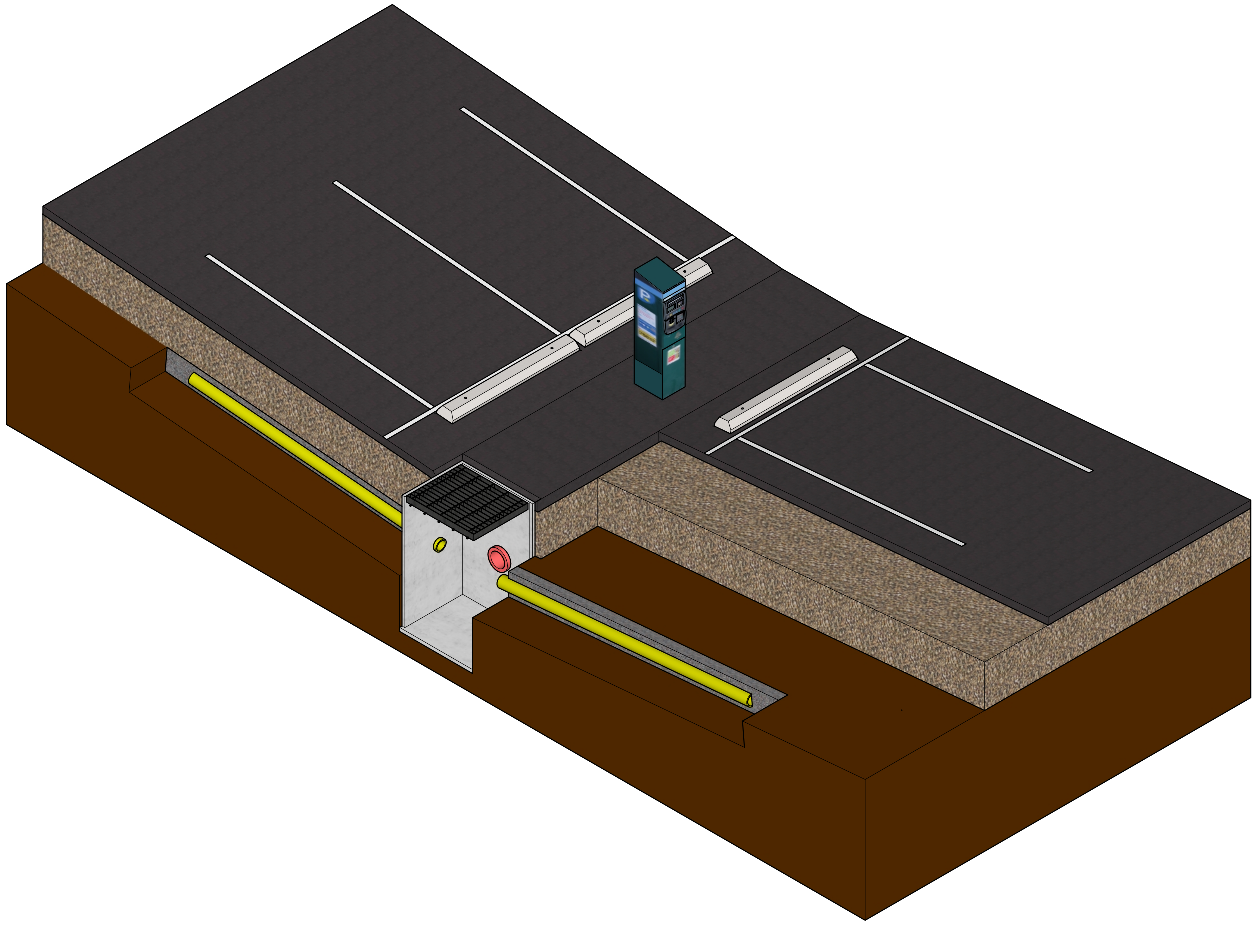
Title

ROCK TIEDOWN ANCHOR GLOBAL STABILITY

OBJECTS ARE COLOR-CODED
BETWEEN TWO VIEWS FOR CLARITY



SECTIONAL VIEW



ISOMETRIC

GROUNDED
ENGINEERING

Title

LONGITUDINAL SUBDRAIN CONNECTION TO CATCHBASIN