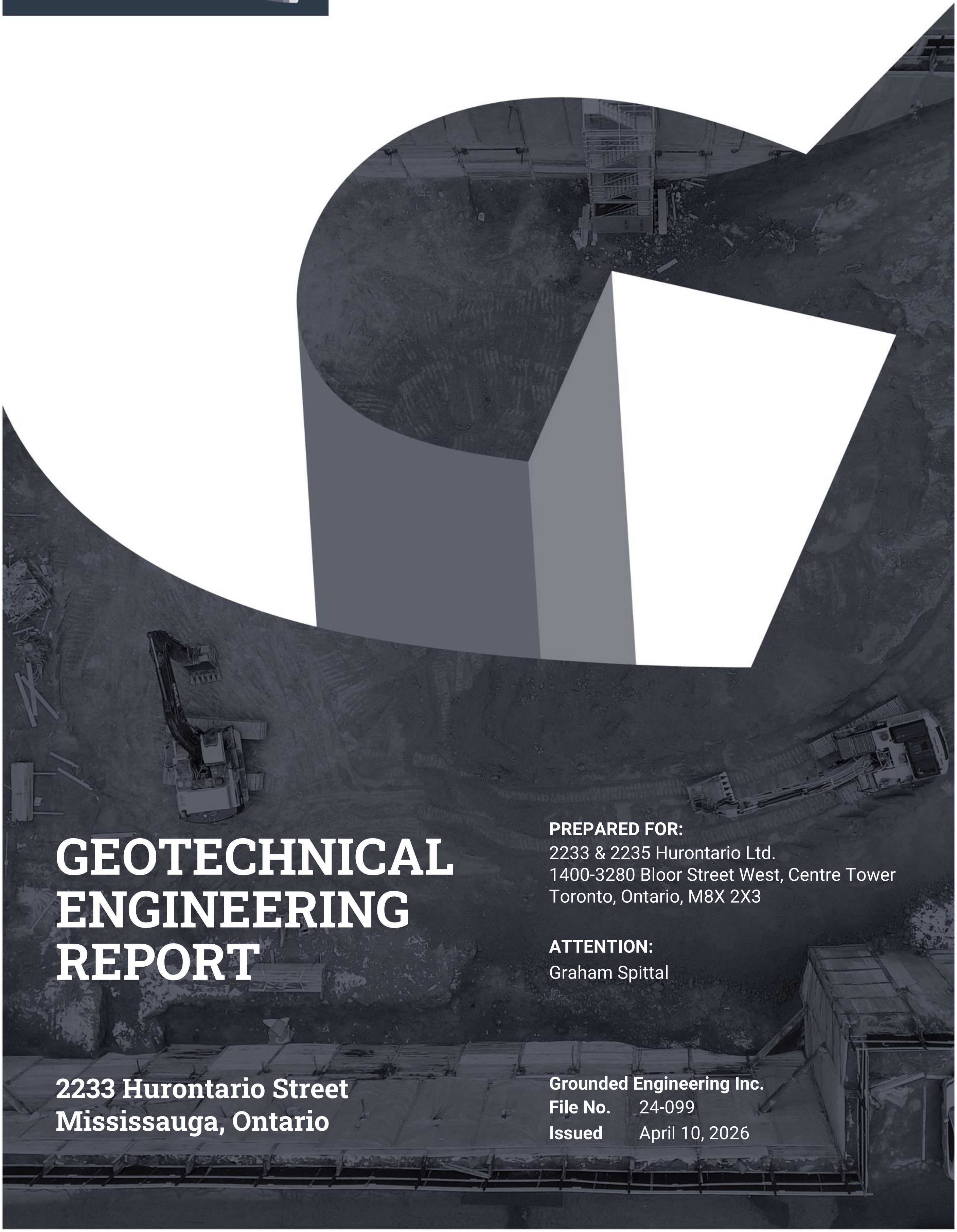

GEOTECHNICAL ENGINEERING REPORT

PREPARED FOR:
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ATTENTION:
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Grounded Engineering Inc.
File No. 24-099
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1 Introduction

2233 & 2235 Hurontario Ltd. has retained Grounded Engineering Inc. to provide geotechnical engineering design advice for their proposed development at 2233 Hurontario Street, in Mississauga, Ontario.

The subject site is occupied by two (2) existing apartment buildings (13- and 12-storeys) and a 1-storey commercial building; the existing structures are resting on a single below-grade level consisting of basements (below the respective buildings) and underground parking across the site. The site is also surfaced with landscaped and asphalt parking areas. The proposed project includes demolishing the 1-storey building and constructing two (2) new infilled 35-storey residential towers extending from a low-rise podium structure. The new structures will rest on three levels of underground parking (P3), with a typical finished floor elevation (FFE) of 94.7± m.

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

- Architectural Drawings, "2233 Hurontario Street"; dated April 1, 2026, prepared by BDP Quadrangle.
- Site Survey, “Part of Lot 15, Concession 1 South of Dundas Street”; dated July 29, 2024, prepared by J.D. Barnes Ltd.

Grounded’s subsurface investigation of the site to date includes five (5) boreholes (Boreholes 1 to 3 and 201 to 202) which were advanced on June 26th and 27th, 2024 (for Boreholes 1 to 3), February 25th, 2026 (for Borehole 201), and March 2nd to 4th, 2026 (for Borehole 202). Boreholes 1 to 3 were completed for engineering feasibility purposes (ref. *“24-099 Feasibility Letter, 2233 & 2235 Hurontario St, Mississauga ON”; dated August 16, 2024*), and Boreholes 201 and 202 were completed for ZBA/SPA submission and detailed design purposes.

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.

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 (City of Mississauga Benchmark No. 075033035). The horizontal coordinates are provided relative to the Universal Transverse Mercator (UTM) geographic coordinate system. Boreholes 1 to 3 and 201 were drilled at existing grade elevation, and Borehole 202 was drilled from within the existing P1 parking garage level.

2.1 Stratigraphy

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

A subsurface profile showing stratigraphy and engineering units is appended.

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 1, 2, and 201 encountered 100 to 610 mm of topsoil at ground surface. Borehole 3 observed a 140 mm of asphalt pavement overlying a 125 mm thick aggregate layer. Borehole 202, advanced from within the existing P1 level, observed a 165 mm thick concrete slab overlying a 140 mm aggregate layer.

Underlying the surficial materials, the boreholes (excluding Borehole 202) observed a layer of earth fill that extends to depths of 1.5 to 3.8 m below grade (Elev. 103.3 to 101.3 m). The earth fill varies in composition but generally consists of sand, to silty sand. It contains trace amounts of clay, gravel, and deleterious materials (incl. asphalt and rootlets). The earth fill ranges in colour from dark brown to brown with orange staining and is moist.

The Standard Penetration Test (SPT) results (N-Values) measured in the fill range from 2 to 24 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 Sands

Underlying the fill materials and existing P1 level slab, the boreholes encountered an undisturbed native sand deposit at depths of 0.4 to 3.8 m below grade (Elev 103.3 to 101.2 m) extending to depths of 0.8 to 4.9 m below grade (Elev 100.7 to 100.1 m). The sand deposit contains a variable amount of silt (ranging from trace silt to sand and silt) and trace clay. The sand typically transitions from brown to grey with increasing depth, and is generally moist.

The SPT N-Values measured in the deposit range from 9 to 45 bpf, indicating a loose to dense relative density.

2.1.3 Glacial Till

Underlying the sand deposit, the boreholes encountered a glacial till at depths of 0.8 to 4.9 m below grade (Elev. 100.7 to 100.1 m), extending to depths of 7.6 to 11.1 m below grade (Elev. 94.4 to 93.9 m). The glacial till varies in composition from cohesive clayey silt (sandy) to cohesionless sandy silt (some clay) matrices, contains trace gravel, ranges in colour from greyish brown to grey, and is generally moist. The glacial till also contains trace to some rock and shale fragments with increasing depth; Boreholes 1 and 2 also observed layers of gravelly sand within the glacial till at depths of 10.9 and 9.1 m below grade (Elev. 96.0 to 94.6 m), respectively.

The SPT N-Values measured range from 17 to greater than 30 bpf in the clayey silt glacial till, suggesting a very stiff to hard consistency, and 26 to greater than 50 bpf in the sandy silt glacial till, indicating a compact to very dense consistency.

2.1.4 Bedrock

Bedrock was confirmed by rock cores recovered in all boreholes extending to depths of 11.9 to 15.7 m below grade (Elev. 89.9 to 89.3 m).

Detailed core logs are included with the corresponding borehole logs. Photographs of the recovered rock core and a guide of rock core terminology are appended. The rock core terminology sheet defines many of the descriptive terms used below.

The bedrock beneath the site 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”. The percentage of strong limestone beds in each run is reported on the rock core logs. The percentage of limestone found in each run of rock core ranged from 0 to 31%.

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

Rock core samples were submitted for testing of unconfined compressive strength (ASTM D7012) and elastic moduli in uniaxial compression (ASTM D7012). The detailed rock laboratory testing results are appended. The test results are summarized as follows:

Borehole ID	Core ID	Depth (m)	Bulk Density (kg/m^3)	UCS (MPa)	Young's Modulus, E (GPa)	Lithology
BH1	CS1	13.7 to 13.8	2585	12.1	2.9	Shale
BH3	CS2	14.2 to 14.4	2615	17.0	3.0	Shale
BH2	CS3	13.1 to 13.3	2617	16.1	3.4	Shale

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

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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.

	Zone	Description	Notes
	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.

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)
1	105.5	11.1	94.4	11.7	93.8
2	105.1	11.1	94.0	11.4	93.7
3	104.8	10.8	94.0	11.0	93.8
201	104.7	10.8	93.9	11.7	93.0
202	101.5	7.6*	93.9	9.7*	91.8

*Borehole 202 was advanced from within the existing P1 level.

Borehole 202 noted extensive zones of core loss (inferred washout due to rock core tooling issues) during advancement, which was subsequently interpreted as a lower sound bedrock elevation at 91.8 m compared to the remaining boreholes advanced across the site.

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 70% in the weathered bedrock, and between 50% and 80% 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 is recognized as a design and constructability issue to be addressed.

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. Monitoring wells were installed in each of the boreholes, and stabilized groundwater levels were measured in each of the installed monitoring wells. The groundwater observations are shown on the Borehole Logs and are summarized as follows.

Well ID	Well Diameter (mm)	Ground Surface (masl)	Top of Screen (masl)	Bottom of Screen (masl)	Screened Geological Unit
BH1	50	105.5	92.9	89.8	Bedrock
BH2	50	105.1	97.5	94.4	Glacial Till
BH3	50	104.8	92.3	89.3	Bedrock
BH201	50	104.7	99.8	96.8	Glacial Till
BH202	50	101.5	98.8	95.7	Glacial Till

A detailed table of monitoring well observation data is appended.

Well ID	Groundwater Elevation (masl)				
	July 4, 2024	July 12, 2024	March 4, 2026	April 1, 2026	Maximum
BH1	99.4	99.5	95.0	95.1	**99.5**
BH2	100.7	99.5	95.1	95.0	**100.7**
BH3	99.2	99.3	95.4	95.4	**99.3**
BH201	NA	NA	97.5	97.0	**97.5**
BH202	NA	NA	NA	95.8	**95.8**

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. 97.5± m.

Within the zone of excavation, the earth fill and sand will permit the free-flow of water when wet, whereas the glacial till contains a sufficient amount of fines to be considered a low permeability soil and would yield only minor seepage in the long-term. It can be expected that the weathered bedrock and fractures in the sound bedrock, as well as cohesionless zones within the glacial till, will produce groundwater seepage. 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. 24-099).

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].

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

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

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.

3.1 Foundation Design Parameters

The proposed project includes demolishing the 1-storey building and constructing two (2) new infilled 35-storey residential towers extending from a low-rise podium structure. The new structures will rest on three levels of underground parking (P3), with a typical FFE of 94.7± m. Spread footings bearing on bedrock of the Georgian Bay Formation have been considered in our analysis.

3.1.1 Spread Footings

Spread footings made below the proposed P3 level (FFE of 94.7± m) may bear on bedrock of the Georgian Bay Formation. Footings bearing on the weathered bedrock at Elev. 94.0± m may be designed using a maximum factored geotechnical resistance at ultimate limit state (ULS) of 5 MPa. The geotechnical reaction at serviceability limit state (SLS) is 3 MPa, for an estimated total settlement of 25 mm.

Alternatively, if higher capacities are required by the Structural Engineer, the footings can be deepened (or constructed as drilled piers) to bear on sound bedrock at Elev. 93.8 to 91.8± m, and may be designed using a maximum factored geotechnical resistance at ULS of 10 MPa. The geotechnical reaction at SLS is 6 MPa, for an estimated total settlement of 15 to 20 mm.

If this higher capacity spread footing design is preferred, additional boreholes/rock coring is recommended to delineate the potentially lower sound bedrock elevation within the vicinity of Borehole 202.

The capacities provided above are based on individual spread footing foundations that are 1 to 3 m wide, spaced one footing width apart, and embedded a minimum of 1 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.

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.

In the rock, there must be a minimum of 300 mm between the edge of any footing and the top of a sloped 2V:1H sound rock cut down to another footing. This requirement exists to avoid undermining adjacent footings.

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.

The bedrock surface can weather and deteriorate on exposure to the atmosphere or surface water; hence, foundation bases which remain open for an extended period of time should be protected by a skim coat of lean concrete.

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.

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 542 m/s was assessed from grade. Based on the measured shear wave velocities, the site designation for seismic analysis is **X_{542}**.

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
Sands	21	32	0.31	0.47	3.25
Glacial Till	22	32	0.31	0.47	3.25
Sound 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:

$$\boldsymbol{P = K[\gamma h + q]}$$

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.

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.

The earth pressure design approach for foundation walls below the top of bedrock is empirical and assumes a uniform pressure distribution below the top of bedrock elevation equal to the maximum earth pressure calculated for the lowest level of soil overtop. This approach is conventional and likely conservative, but it is practical insofar as it acknowledges the requirement of having a foundation wall of a consistent width at the lower levels.

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:

$$\boldsymbol{R_f = \Phi N \tan\varphi}$$

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

At the proposed P3 level (FFE of 94.7± m), the undisturbed native soils will provide adequate subgrade for the support of a conventional slab on grade. The modulus of subgrade reaction appropriate for slab-on-grade design supported by the undisturbed soils is 60,000 kPa/m.

As this basement structure is to be 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 19 mm clear stone (OPSS.MUNI 1004) vibrated to a dense state.

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.

The use of excavated bedrock spoil to restore subgrade elevations is to be specifically prohibited. This bedrock spoil cannot be adequately compacted to provide support for the slab on grade and is not to be reused below any settlement sensitive areas.

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.

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 (min. 100 mm dia.) are to be spaced at a maximum 6 m (measured on centre).

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.

Although the basement will be made as a drained 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 are provided in Grounded’s Hydrogeological Report (File No. 24-099).

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.

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 Underground Parking Structure

It is expected that all new 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
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.

Should pavement be placed on soil subgrade, additional recommendations may be provided upon request.

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 (subject to on-site inspection and approval by Grounded during below-grade construction):

- The earth fill is a Type 3 soil
- The sands are Type 3 soils, or Type 4 soils if wet
- The glacial till is a Type 2 soil

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.

Bedrock is not considered a soil under the Act, per clause 234(2)(d). Vertical excavations made in sound bedrock are generally self-supporting provided the rock bedding is horizontally oriented. If deemed necessary, rock bolts and welded wire mesh/shotcrete (or other means and methods) can be used to protect workers from rock spalling from the face of excavation. The rock face must be inspected by Grounded to determine that no other support system is required to prevent the spalling of loose rock, and to confirm that all loose spall material at risk of falling upon a worker is removed (clause 234(2)(h) of the above noted regulations).

The exposed vertical bedrock face deteriorates with time and exposure. Exposed excavation faces have been found to flake and recede as much as 300 mm within a 12-month exposure. This recession generally takes the form of coin-sized shale particles dropping from the face on a constant basis. If deemed necessary, debris netting draped over the rock face can be used to contain and collect these coin-sized shale particles.

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.

Excavations for foundations and other deeper protrusions will penetrate weathered and sound bedrock. Georgian Bay Formation bedrock is a rippable rock that can be removed with conventional excavation equipment once it has been broken by ripper tooth or hoe ram. Creating detailed excavation shapes for foundations etc. is normally accomplished by hoe ram. The removal of rock from a vertical face without over-excavation, which can happen inadvertently by dislodging additional rock, is largely dependent on machine operator skill. The contractor shall exercise caution and implement the appropriate techniques to reduce the amount of disturbance to the rock mass (rock fracturing) with the excavator.

If excavation faces must be made neat (such as beside an existing footing), a line of excavation can be provided by line drilling the rock a series of closely-spaced vertical holes (100 mm diameter, spaced at 300 mm on centre) to provide a preferential vertical break path for the excavation face.

Georgian Bay Formation bedrock contains beds of harder calcareous beds (e.g. limestone). When excavating this bedrock, it should be expected that these harder layers will be encountered. Hard layers interbedded within the shale are normally broken with hoe mounted hydraulic rams before excavation.

Limestone beds may also be found to straddle the founding elevation, in which case the entire thickness of the hard limestone layer must be removed to expose founding subgrade as it is not possible to remove part of one of these layers. This will in turn result in excess rock removal not intrinsic to the project requirements. The risk and responsibility for the excess rock removal under these circumstances, and the supply and placement of the extra concrete to restore the foundation grade, must be addressed in the contract documents for foundations, excavation, and shoring contractors.

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. The potential for gas emissions from this formation is recognized as a design and constructability issue to be addressed.

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 design groundwater table for engineering purposes is at Elev. 97.5± m.

Within the zone of excavation, the earth fill and sand will permit the free-flow of water when wet, whereas the glacial till contains a sufficient amount of fines to be considered a low permeability soil and would yield only minor seepage in the long-term. It can be expected that the weathered bedrock and fractures in the sound bedrock will produce groundwater seepage. There is also infiltrated stormwater perched in the earth fill which is flowing down towards the groundwater table.

On this basis, seepage into excavations may be allowed to drain into the excavation and then controlled by a conventional sump pump arrangement. Nevertheless, delays in excavation will occur as the seepage is controlled and these delays should be anticipated in the construction schedule.

The City of Mississauga will require a Discharge Agreement in the short term, if any water is to be discharged to the storm or sanitary sewers during 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:

$\boldsymbol{P = 0.8\,K[\gamma H + q] + \gamma_w h_w}$ … in cohesive soils
$\boldsymbol{P = 0.65\,K[\gamma H + q] + \gamma_w h_w}$ … 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/m^3)
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), h_w is equal to zero. For the design of impermeable shoring, a design groundwater table at Elev. 97.5± 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 cohesive soils, the lateral earth pressure distribution is trapezoidal, uniformly increasing from zero to the maximum pressure defined in the equation above over the top and bottom quarter (H/4) of the shoring. In cohesionless soils, the lateral earth pressure distribution is rectangular.

Where the excavation penetrates the bedrock, the rock excavation is nominally self-supporting in a vertical face, provided the rock bedding is horizontally oriented. The requirement for extending lagging into partially weathered rock depends on the quality of the excavation cut and the degree of weathering.

5.3.2 Soldier Pile Toe Embedment

Soldier pile toes will be made below the proposed P3 level in the bedrock of the Georgian Bay Formation. Soldier pile toes resist horizontal movement due to the passive earth pressure acting on the toe below the base of excavation. The maximum factored vertical geotechnical resistance at ULS for the design of a pile embedded in the sound bedrock is 10 MPa. The maximum factored lateral geotechnical resistance at ULS of the undisturbed rock is 1 MPa.

There are zones of soil in the subgrade that are wet, cohesionless, 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.

Exposed bedrock of the Georgian Bay Formation deteriorates with time. Within 12 months of exposure, excavation faces made within this bedrock flake and recede as much as 300 mm, generally in the form of coin-size shale particles dropping from the face on a constant basis. The deteriorated rock loses internal integrity and bearing capability. Solider piles for the shoring system are typically advanced at least 1 metre below the base of the excavation (to be confirmed by the geostructural engineer) to accommodate this weathering and still ensure that the required lateral and vertical bearing resistances can be utilized.

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 compact to dense/very stiff to hard subgrade, post-grouted micropile ground anchors in tension can be designed using a maximum factored geotechnical resistance at ULS of 70 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.

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.

Raker footings established on weathered bedrock at an inclination of 45 degrees can be designed using a maximum factored geotechnical resistance at ULS 2500 kPa.

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 soils 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.

Deteriorating bedrock loses internal integrity and bearing capability. If bedrock subgrade is to be exposed for prolonged periods of time, it is recommended that a skim coat of concrete be used to protect the bedrock subgrade from slaking and other degradation resulting from weathering.

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.

Fill placement is typically measured relative to Standard Proctor Maximum Dry Density (SPMDD). A third-party testing agency can be retained to provide in situ density measurements on site to confirm that the specified density is achieved during fill or asphalt placement. Grounded can provide oversight and review services for this testing as needed.

6 Limitations and Restrictions

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.

6.1 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.

6.2 Report Use

The authorized users of this report are 2233 & 2235 Hurontario Ltd. 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.

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The City of Mississauga 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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FIGURES

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49 MOBILE DRIVE, TORONTO, ONT., M4A 1H5
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LEGEND

APPROXIMATE SITE BOUNDARY

Note

Reference

ArcGIS Online 2026

Project

2233 HURONTARIO ST, MISSISSAUGA, ONTARIO

Figure Title

SITE LOCATION PLAN

North

TRUE
PROJECT

Date

MARCH 2026

Scale

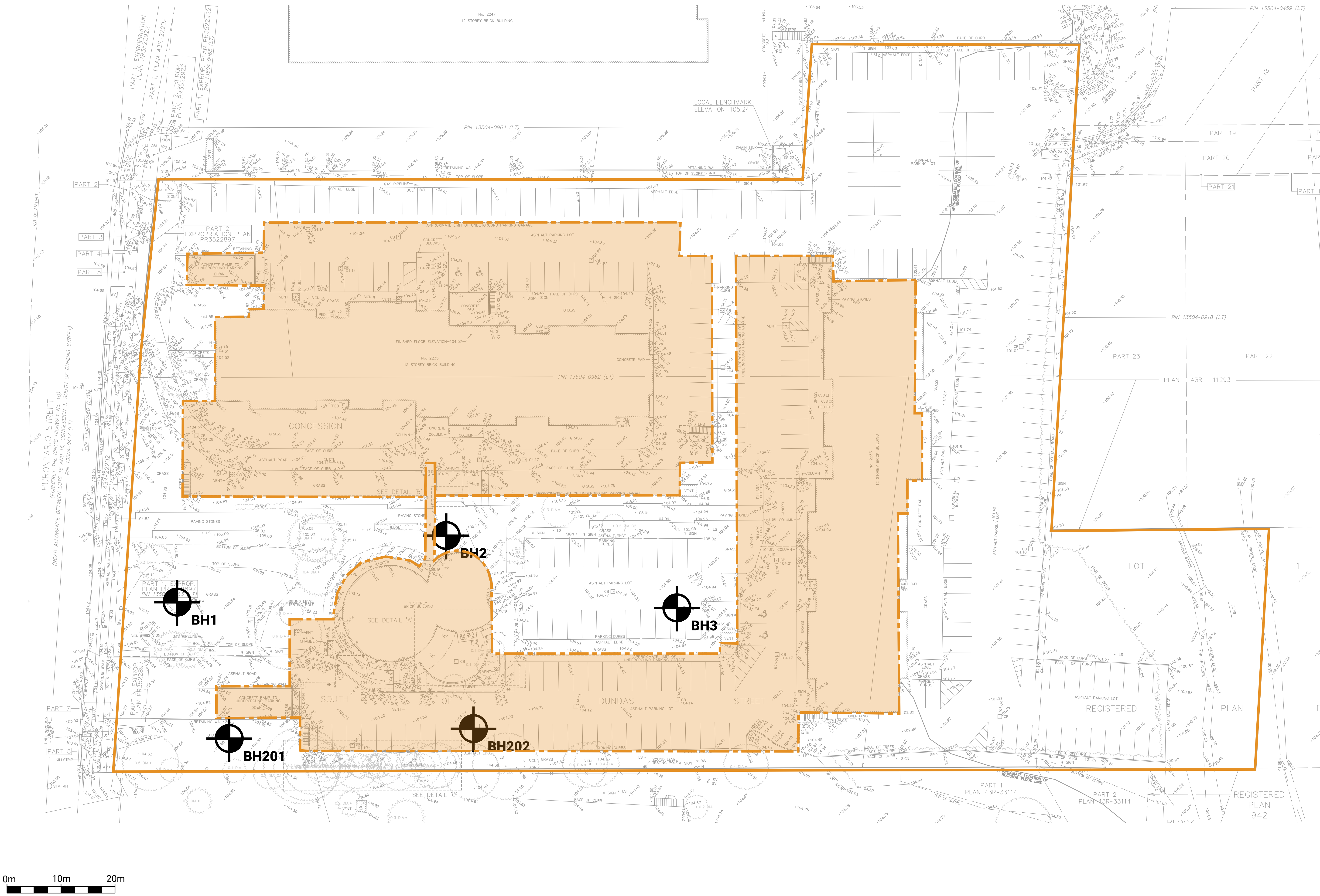
AS INDICATED

Job No

24-099

Figure No

FIGURE 1



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LEGEND

—— APPROXIMATE SITE BOUNDARY

– – APPROXIMATE EXTENT OF EXISTING BASEMENT AND UNDERGROUND PARKING LEVEL

⊕ ⊕ APPROXIMATE MONITORING WELL/BOREHOLE LOCATION BY GROUNDED

Note

Borehole 202 was advanced within the existing underground parking level.

Reference

Site Survey, "Part of Lot 15, Concession 1 South of Dundas Street"; dated July 29, 2024, prepared by J.D. Barnes Ltd.

Project

2233 HURONTARIO ST, MISSISSAUGA, ONTARIO

Figure Title

BOREHOLE LOCATION PLAN - EXISTING CONDITION

North

TRUE

PROJECT

Date

MARCH 2026

Scale

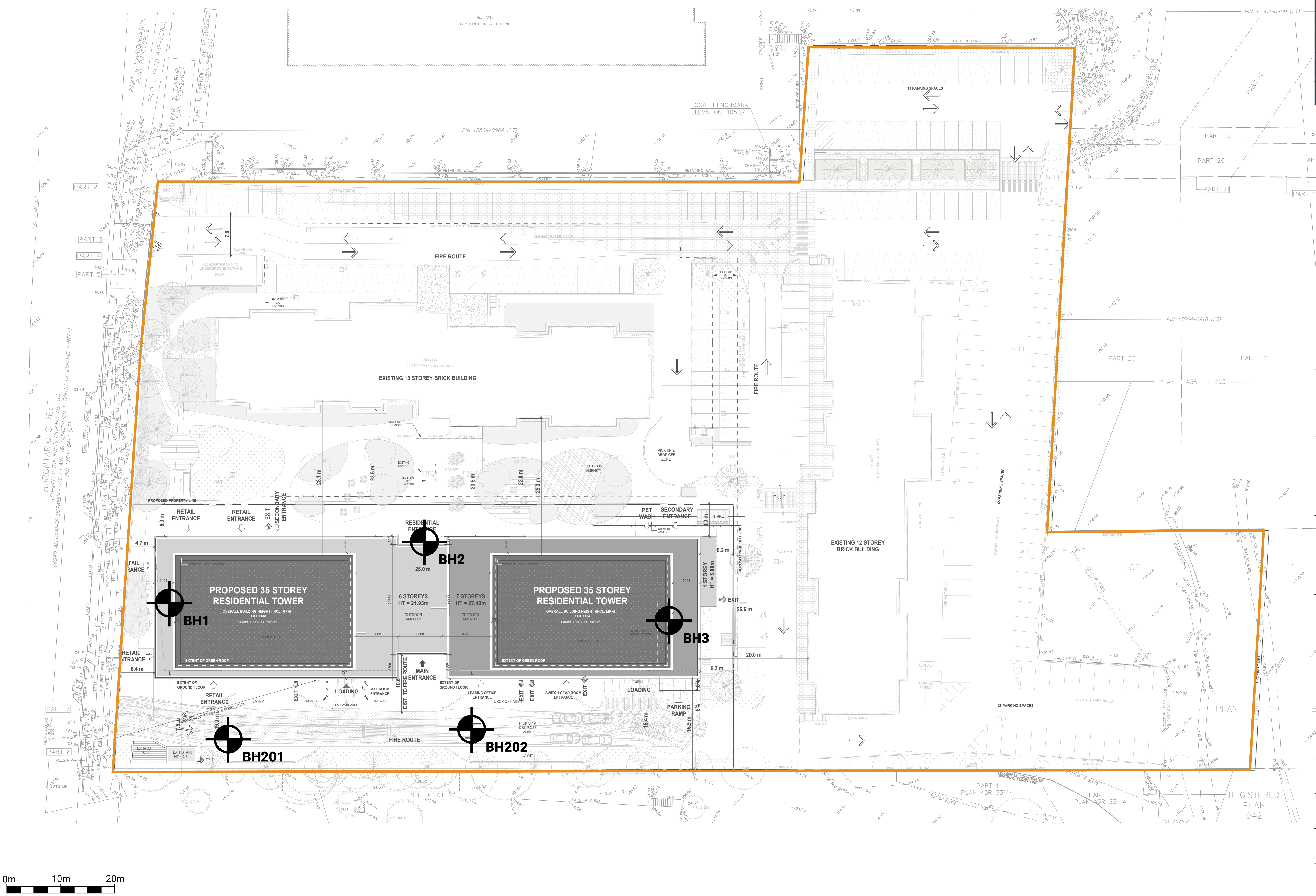
AS INDICATED

Job No

24-099

Figure No

FIGURE 2



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LEGEND

— APPROXIMATE SITE BOUNDARY

⊕ APPROXIMATE MONITORING WELL/BOREHOLE LOCATION BY GROUNDED

Note

Reference

Architectural Drawings, "2233 Hurontario Street"; dated April 1, 2026, prepared by BDP Quadrangle.

Project

2233 HURONTARIO ST, MISSISSAUGA, ONTARIO

Figure Title

BOREHOLE LOCATION PLAN - PROPOSED CONDITION - SITE PLAN

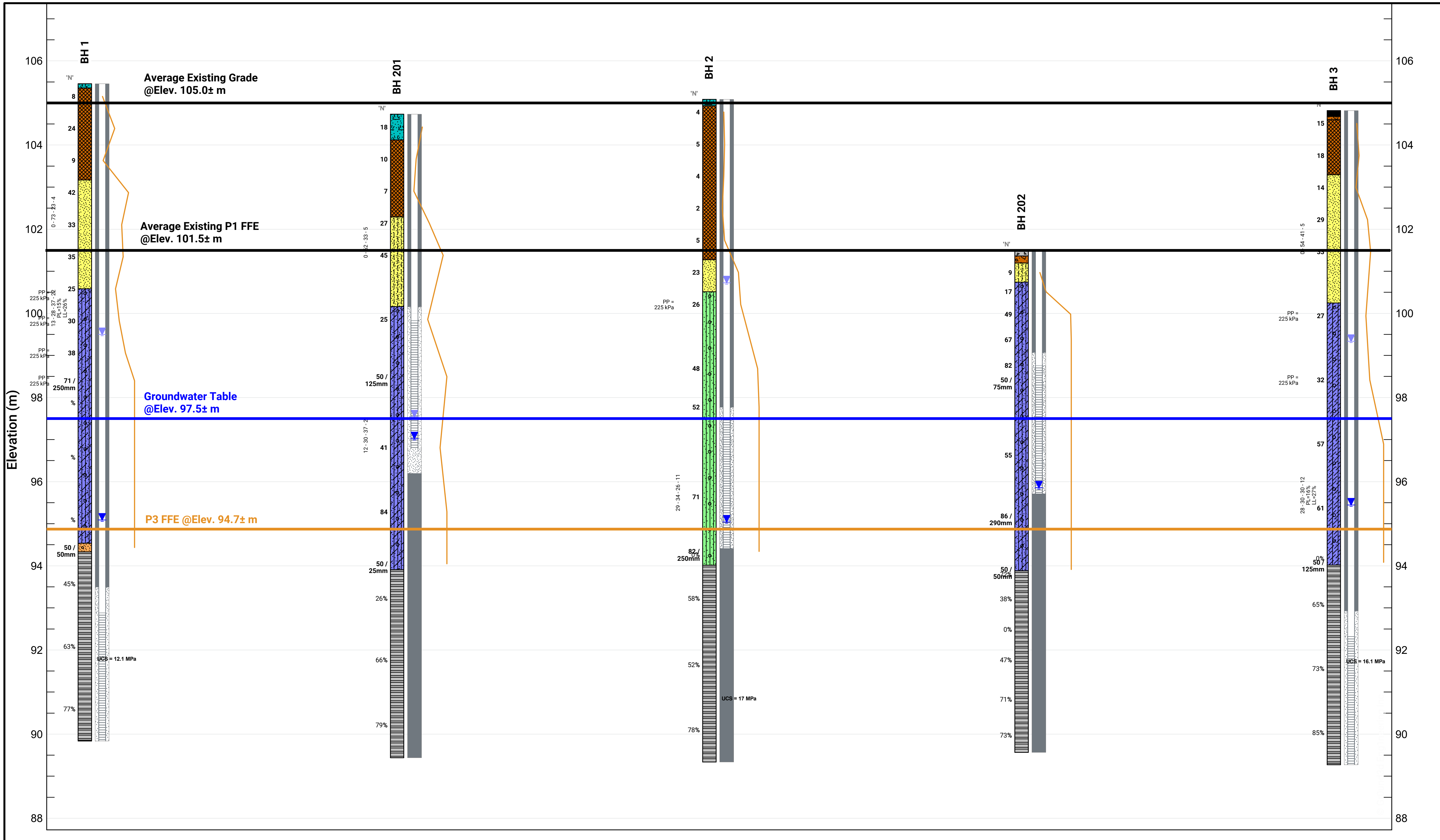
North

Date
MARCH 2026

Scale
AS INDICATED

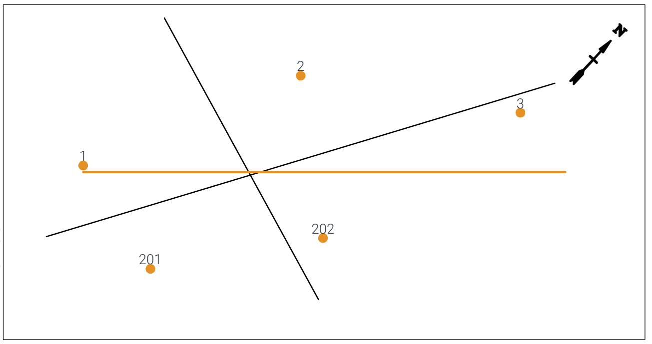
Job No
24-099

Figure No
FIGURE 3



Boreholes Equally Spaced

SITE MAP



BOREHOLE STRATIGRAPHY LEGEND

▨	Topsoil	▨	Gravelly Sand	▨	Aggregate
▨	Fill	▤	Bedrock (cored)	▨	Clayey Silt Till
▨	Sand	▨	Sandy Silt Till	▨	Silty Sand
▨	Clayey Silt Till (sandy)	■	Asphalt	▨	Concrete

file: 24-099 gint.gpj

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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
2233 & 2235 HURONTARIO ST
MISSISSAUGA

Figure Title
SUBSURFACE PROFILE

Date
APRIL 2026

Scale
AS INDICATED

Job No
24-099

Figure No
FIGURE 4

APPENDIX A

BOREHOLE LOG TERMINOLOGY

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.

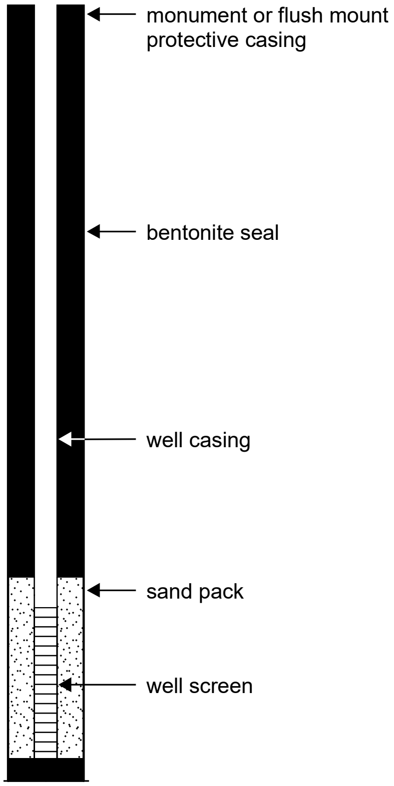
ASTM D3441 Cone Penetration Test (CPT)
Pushing an internal still rod with a outer hollow rod ("sleeve") tipped with a cone with an apex angle of 60° and a cross-sectional area of 1000 mm^2 into soil. The resistance is measured in the sleeve and at the tip to determine the skin friction and the tip resistance.

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



ROCK CORE TERMINOLOGY (MTO SHALE)

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TCR	**Total Core Recovery** the total length of recovery (soil or rock) per run, as a percentage of the drilled length
SCR	**Solid Core Recovery** the total length of sound full-diameter rock core pieces per run, as a percentage of the drilled length
RQD	**Rock Quality Designation** the sum of all pieces of sound rock core in a run which are 10 cm or greater in length, as a percentage of the drilled length

Natural Fracture Frequency (typically per 0.3 m) The number of natural discontinuities (joints, faults, etc.) which are present per 0.3m. Ignores mechanical or drill-induced breaks, and closed discontinuities (e.g. bedding planes).

LOGGING DISCONTINUITIES

Discontinuity Type

BP	bedding parting
CL	cleavage
CS	crushed seam
FZ	fracture zone
MB	mechanical break
IS	infilled seam
JT	Joint
SS	shear surface
SZ	shear zone
VN	vein
VO	void

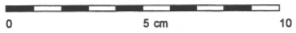
Coating

CN	Clean
SN	Stained
OX	Oxidized
VN	Veneer
CT	Coating (>1 mm)

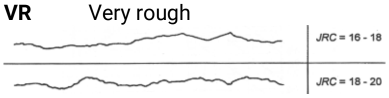
Dip Inclination

H	horizontal/flat	0 - 20°
D	dipping	20 - 50°
SV	sub-vertical	50 - 90°
V	vertical	90±°

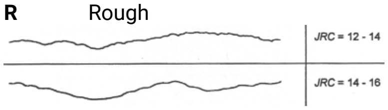
Roughness (Barton et al.)



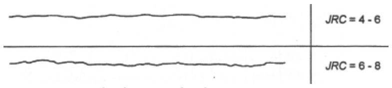
VR Very rough



R Rough

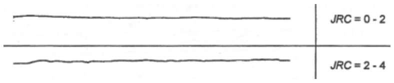


S Smooth



SL Slickensided
(visually assessed)

POL Polished



Spacing in Discontinuity Sets
(*ISRM 1981*)

VC	very close	< 60 mm
C	close	60 – 200 mm
M	mod. close	0.2 to 0.6 m
W	wide	0.6 to 2 m
VW	very wide	> 2 m

Aperture Size

T	closed / tight	< 0.5 mm
GA	gapped	0.5 to 10 mm
OP	open	> 10 mm

Planarity

PR	Planar
UN	Undulating
ST	Stepped
IR	Irregular
DIS	Discontinuous
CU	Curved

GENERAL

Degree of Weathering (*after MTO, RR229 Evaluation of Shales for Construction Projects*)

Zone	**Degree**	**Description**
Z1	unweathered	shale, regular jointing
Z2	partially weathered	angular blocks of unweathered shale, no matrix, with chemically weathered but intact shale
Z3		soil-like matrix with frequent angular shale fragments < 25mm diameter
Z4a		soil-like matrix with occasional shale fragments < 3mm diameter
Z4b	fully weathered	soil-like matrix only

Strength classification (*after Marinos and Hoek, 2001; ISRM 1981b*)

Grade		**UCS (MPa)**	**Field Estimate (Description)**
R6	extremely strong	> 250	can only be chipped by geological hammer
R5	very strong	100 - 250	requires many blows from geological hammer
R4	strong	50 - 100	requires more than one blow from geological hammer
R3	medium strong	25 - 50	can't be scraped, breaks under one blow from geological hammer
R2	weak	5 - 25	can be peeled / scraped with knife with difficulty
R1	very weak	1 - 5	easily scraped / peeled, crumbles under firm blow of geo. hammer
R0	extremely weak	< 1	indented by thumbnail

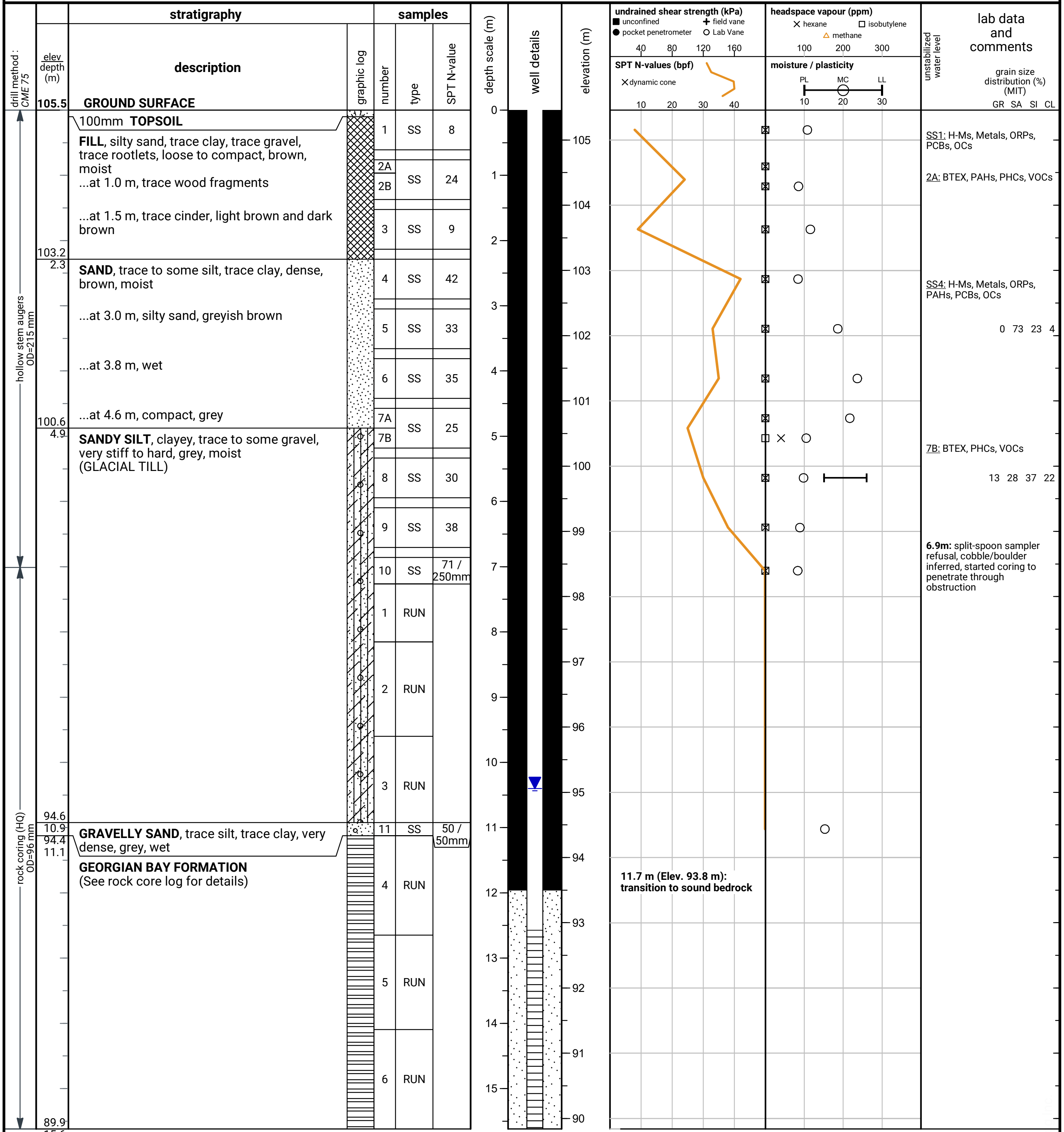
Bedding Thickness (*Q. J. Eng. Geology, Vol 3, 1970*)

Very thickly bedded	> 2 m
Thickly bedded	0.6 – 2m
Medium bedded	200 – 600mm
Thinly bedded	60 – 200mm
Very thinly bedded	20 – 60mm
Laminated	6 – 20mm
Thinly Laminated	< 6mm

Date Started : Jun 26, 2024
Position : E: 612593, N: 4825406 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 1

File No. : 24-099 **Project** : 2233 & 2235 Hurontario St, Mississauga **Client** : Starlight Development



elev depth (m)	description
105.5	**GROUND SURFACE**
	100mm **TOPSOIL**
	FILL, silty sand, trace clay, trace gravel, trace rootlets, loose to compact, brown, moist ...at 1.0 m, trace wood fragments ...at 1.5 m, trace cinder, light brown and dark brown
103.2 2.3	**SAND**, trace to some silt, trace clay, dense, brown, moist ...at 3.0 m, silty sand, greyish brown ...at 3.8 m, wet ...at 4.6 m, compact, grey
100.6 4.9	**SANDY SILT**, clayey, trace to some gravel, very stiff to hard, grey, moist (GLACIAL TILL)
94.6 10.9	**GRAVELLY SAND**, trace silt, trace clay, very dense, grey, wet
94.4 11.1	**GEORGIAN BAY FORMATION** (See rock core log for details)
89.9 15.6	

number	type	SPT N-value
1	SS	8
2A 2B	SS	24
3	SS	9
4	SS	42
5	SS	33
6	SS	35
7A 7B	SS	25
8	SS	30
9	SS	38
10	SS	71 / 250mm
1	RUN	
2	RUN	
3	RUN	
11	SS	50 / 50mm
4	RUN	
5	RUN	
6	RUN	

lab data and comments

grain size distribution (%) (MIT)

	GR	SA	SI	CL
SS5 (3.2 m)	0	73	23	4
SS8 (5.7 m)	13	28	37	22

SS1: H-Ms, Metals, ORPs, PCBs, OCs

2A: BTEX, PAHs, PHCs, VOCs

SS4: H-Ms, Metals, ORPs, PAHs, PCBs, OCs

7B: BTEX, PHCs, VOCs

6.9m: split-spoon sampler refusal, cobble/boulder inferred, started coring to penetrate through obstruction

END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jul 4, 2024	6.1	99.4
Jul 12, 2024	6.0	99.5
Mar 4, 2026	10.5	95.0
Apr 1, 2026	10.4	95.1

Tech : IH | **PM** : JAW/VT | **Rev** : NN

GROUNDED ENGINEERING

Date Started : Jun 26, 2024
Position : E: 612593, N: 4825406 (UTM 17T)
Elev. Datum : Geodetic

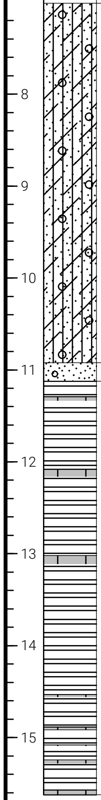
ROCK CORE LOG 1

File No. : 24-099

Project : 2233 & 2235 Hurontario St, Mississauga

Client : Starlight Development

Rock coring started at 7.0m below grade



stratigraphy

SANDY SILT, clayey, trace to some gravel, very stiff to hard, grey, moist
(GLACIAL TILL)

GRAVELLY SAND, trace silt, trace clay, very dense, grey, wet

GEORGIAN BAY FORMATION
Shale, dark grey, laminated to thinly bedded, weak; joints are horizontal, gapped, clean to unaltered;

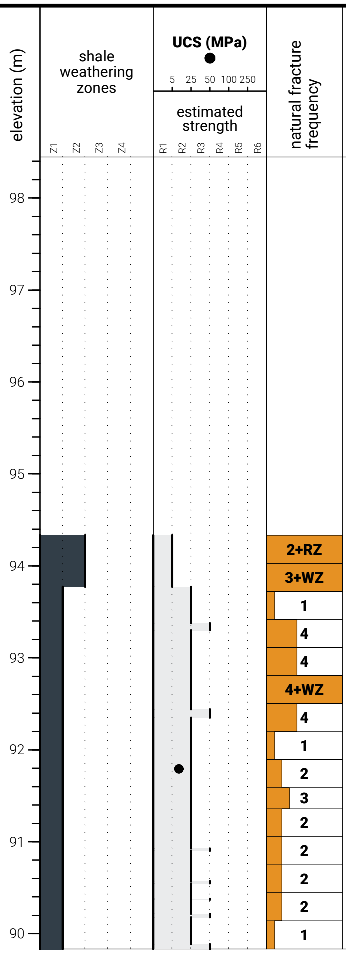
interbedded with **limestone**, light grey, laminated to thinly bedded, strong

Overall shale: 92%, limestone: 8%
... at 11.7 m (Elev. 93.8 m), transition to sound rock

Run 4 :	8% limestone 92% shale	
Run 5 :	6% limestone 94% shale	
Run 6 :	8% limestone 92% shale	

END OF COREHOLE

Run	elev / depth (m)	recovery
	98.5	
R1	7.0 97.3	TCR = **93%**
R2	8.2 95.9	TCR = **98%**
R3	9.6 94.4	TCR = **0%**
R4	11.1 92.9	TCR = **100%** SCR = **83%** RQD = **45%**
R5	12.6 91.4	TCR = **100%** SCR = **95%** RQD = **63%**
R6	14.1 89.9	TCR = **100%** SCR = **100%** RQD = **77%**
	15.6m	



laboratory testing

El. 91.8m:
UCS = 12.1 MPa
E = 2.90 GPa
γ = 25.4 kN/m^3

notes and comments

9.6 / 95.9 - 10.9 / 94.5m: core loss (no recovery)
11.1 / 94.3 - 11.3 / 94.2m: rubbilized zone
11.4 / 94.1m: 75mm weathered zone
11.5 / 94.0m: 25mm FC SV PR S GA CN
11.6 / 93.8m: 13mm IS clay
12.9 / 92.6m: 38mm IS clay
12.9 / 92.6 - 13.0 / 92.5m: weathered zone
13.1 / 92.4m: 25mm IS clay
13.9 / 91.6m: 25mm weathered zone
14.0 / 91.5m: 38mm rubbilized zone
15.5 / 90.0m: 25mm FC SV PR S T CN

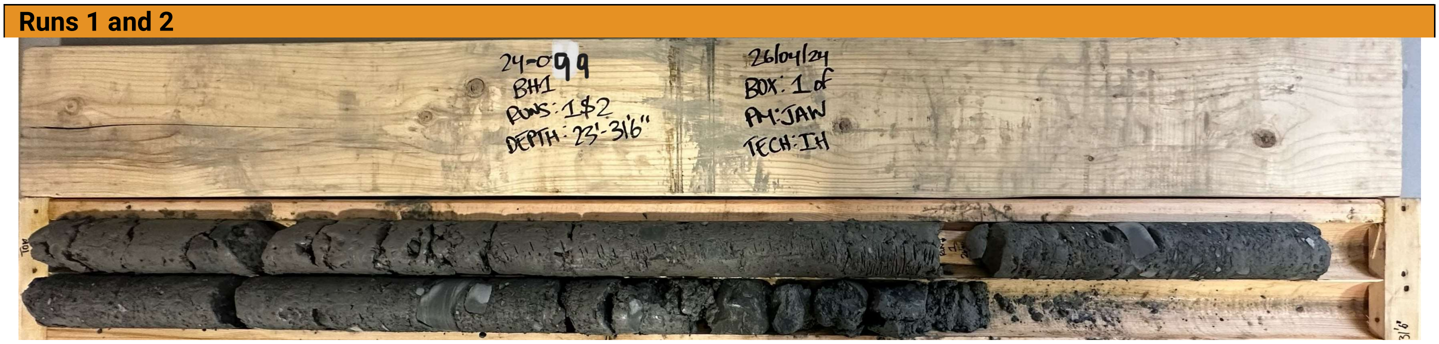
elevation (m): 98, 97, 96, 95, 94, 93, 92, 91, 90

Tech : IH | **PM** : JAW/VT | **Rev** : NN

Borehole 1

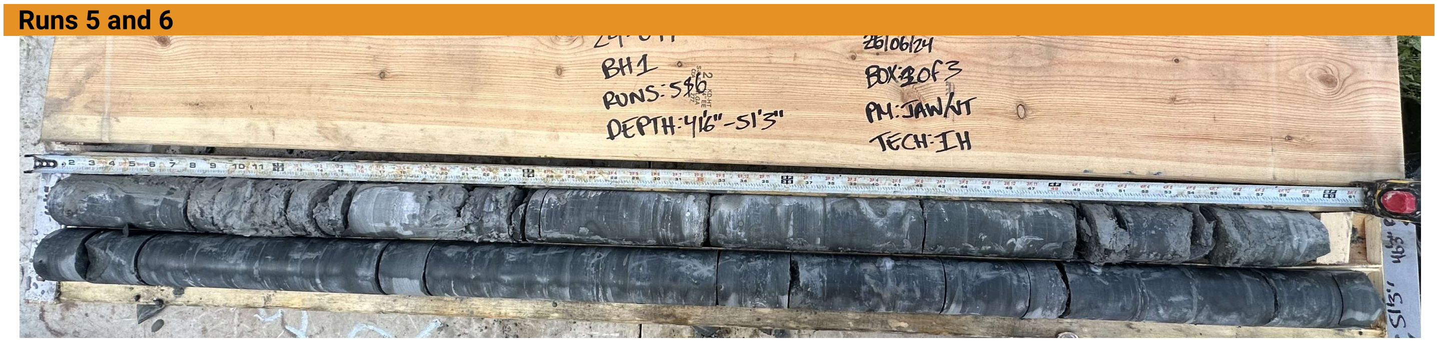
Run #	Depth (m)	Elevation (m)
1	7.0 – 8.2	98.5 – 97.3
2	8.2 – 9.6	97.3 – 95.9
3	9.6 – 11.1	95.9 – 94.4
4	11.1 – 12.6	94.4 – 92.9
5	12.6 – 14.1	92.9 – 91.4
6	14.1 – 15.6	91.4 – 89.9

Runs 1 and 2



Runs 3 and 4

Runs 5 and 6

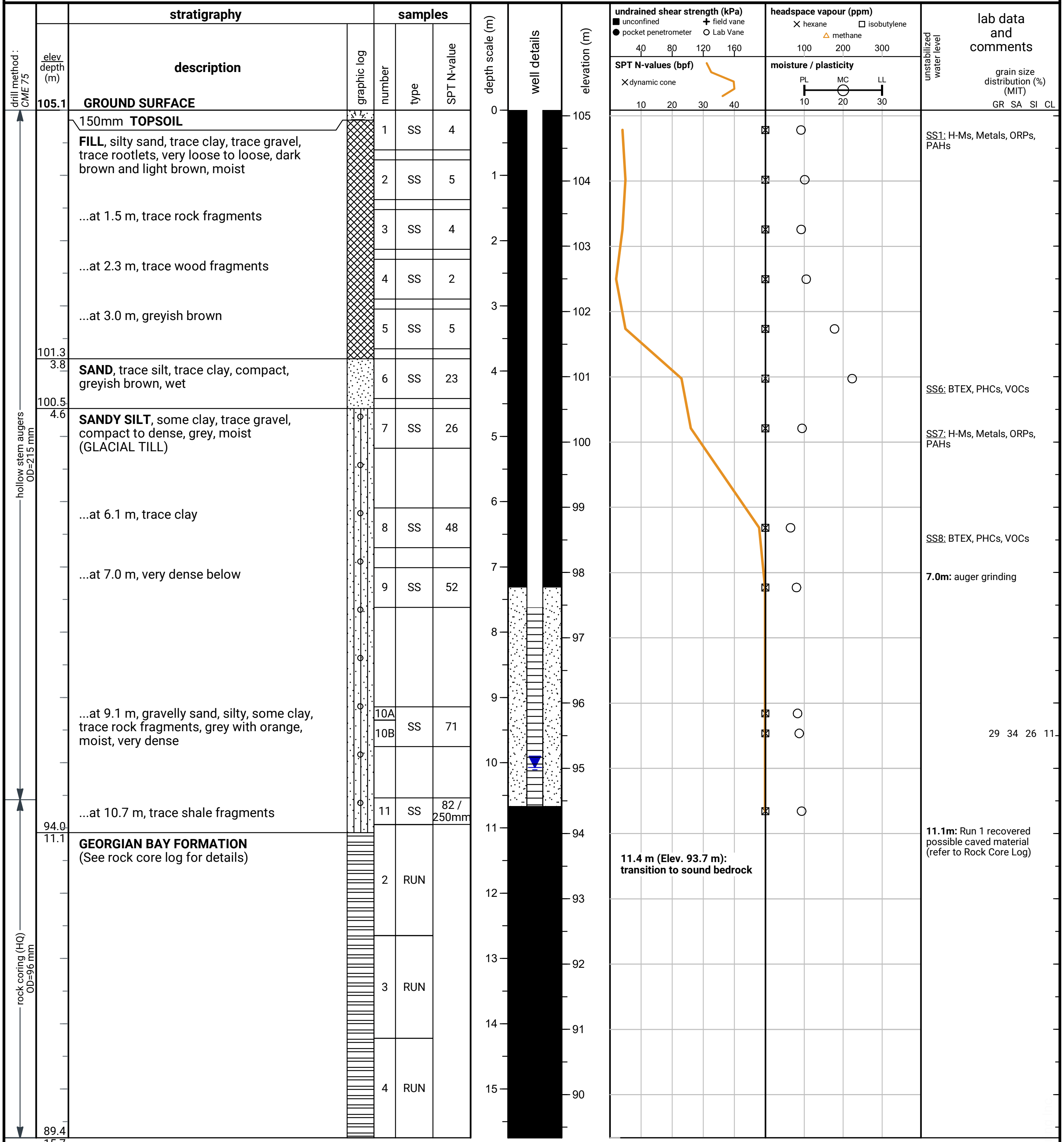


GROUNDED ENGINEERING

Date Started : Jun 27, 2024
Position : E: 612614, N: 4825451 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 2

File No. : 24-099 **Project** : 2233 & 2235 Hurontario St, Mississauga **Client** : Starlight Development



elev depth (m)	description
105.1	**GROUND SURFACE**
	150mm **TOPSOIL**
	FILL, silty sand, trace clay, trace gravel, trace rootlets, very loose to loose, dark brown and light brown, moist
	...at 1.5 m, trace rock fragments
	...at 2.3 m, trace wood fragments
	...at 3.0 m, greyish brown
101.3 3.8	**SAND**, trace silt, trace clay, compact, greyish brown, wet
100.5 4.6	**SANDY SILT**, some clay, trace gravel, compact to dense, grey, moist (GLACIAL TILL)
	...at 6.1 m, trace clay
	...at 7.0 m, very dense below
	...at 9.1 m, gravelly sand, silty, some clay, trace rock fragments, grey with orange, moist, very dense
	...at 10.7 m, trace shale fragments
94.0 11.1	**GEORGIAN BAY FORMATION** (See rock core log for details)
89.4 15.7	**END OF BOREHOLE**

number	type	SPT N-value	lab data and comments
1	SS	4	SS1: H-Ms, Metals, ORPs, PAHs
2	SS	5	
3	SS	4	
4	SS	2	
5	SS	5	
6	SS	23	SS6: BTEX, PHCs, VOCs
7	SS	26	SS7: H-Ms, Metals, ORPs, PAHs
8	SS	48	SS8: BTEX, PHCs, VOCs
9	SS	52	**7.0m:** auger grinding
10A 10B	SS	71	29 34 26 11
11	SS	82 / 250mm	**11.1m:** Run 1 recovered possible caved material (refer to Rock Core Log)
2	RUN		
3	RUN		
4	RUN		

END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

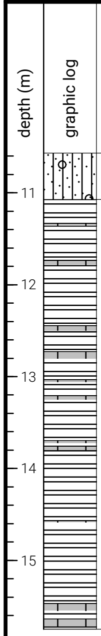
date	depth (m)	elevation (m)
Jul 4, 2024	4.4	100.7
Jul 12, 2024	5.6	99.5
Mar 4, 2026	10.0	95.1
Apr 1, 2026	10.1	95.0

GROUNDED ENGINEERING

Date Started : Jun 27, 2024
Position : E: 612614, N: 4825451 (UTM 17T)
Elev. Datum : Geodetic

ROCK CORE LOG 2

File No. : 24-099 **Project** : 2233 & 2235 Hurontario St, Mississauga **Client** : Starlight Development



stratigraphy

Rock coring started at 10.6m below grade

SANDY SILT, some clay, trace gravel, compact to dense, grey, moist
(GLACIAL TILL)

GEORGIAN BAY FORMATION
Shale, dark grey, laminated to thinly bedded, weak; joints are horizontal, closed to gapped, clean to unaltered;

interbedded with **limestone**, light grey, laminated to thinly bedded, strong

Overall shale: 90%, limestone: 10%
... at 11.4 m (Elev. 93.7 m), transition to sound rock

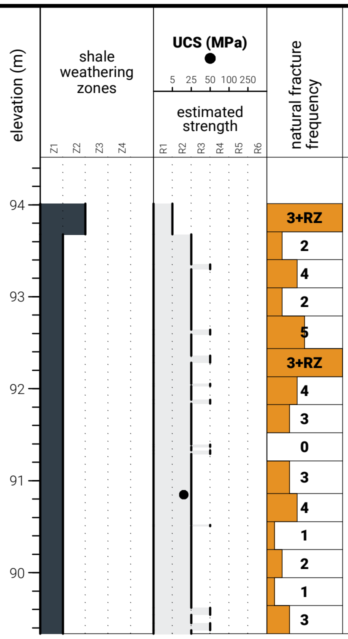
Run 2 : 8% limestone
92% shale

Run 3 : 13% limestone
87% shale

Run 4 : 11% limestone
89% shale

END OF COREHOLE

Run	elev / depth (m)	recovery
	94.5	
R1	10.6	TCR = **45%** SCR = **0%** RQD = **0%**
R2	11.1	TCR = **100%** SCR = **89%** RQD = **58%**
	92.5	
R3	12.6	TCR = **100%** SCR = **95%** RQD = **52%**
	90.9	
R4	14.2	TCR = **100%** SCR = **100%** RQD = **78%**
	89.4	
	15.7m	



laboratory testing

El. 90.8m:
UCS = 17 MPa
E = 3.00 GPa
γ = 25.7 kN/m^3

notes and comments

10.6 / 94.5 - 10.9 / 94.1m: possible caved material

10.9 / 94.1 - 11.1 / 94.0m: core loss (no recovery)

11.3 / 93.8 - 11.4 / 93.7m: rubbilized zone

12.0 / 93.1 - 12.0 / 93.0m: 25mm rubbilized zone

12.6 / 92.4 - 12.7 / 92.4m: 75mm rubbilized zone

elevation (m): 94, 93, 92, 91, 90

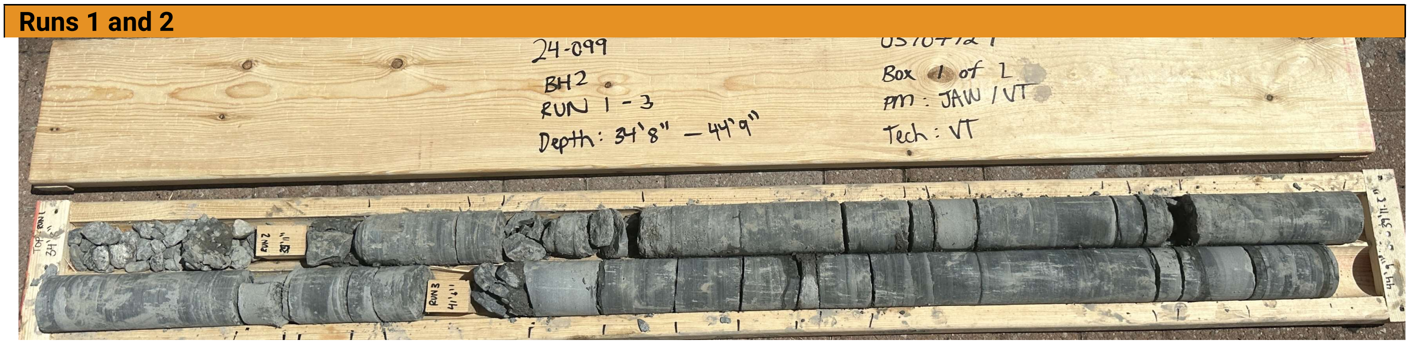
file: 24-099 gint.gpj

Tech : IH | **PM** : JAW/VT | **Rev** : NN

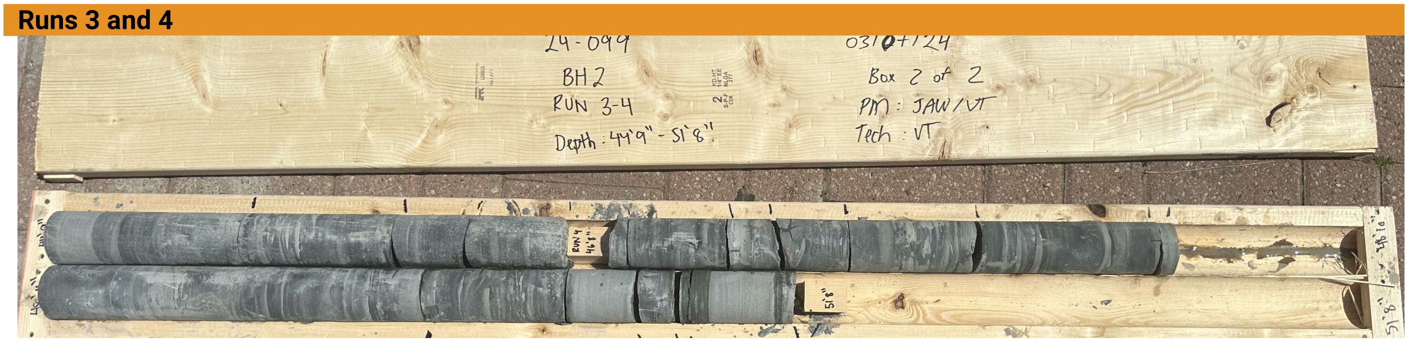
Borehole 2

Run #	Depth (m)	Elevation (m)
1	10.6 – 11.1	94.5 – 94.0
2	11.1 – 12.6	94.0 – 92.5
3	12.6 – 14.2	92.5 – 90.9
4	14.2 – 15.7	90.9 – 89.4

Runs 1 and 2



Runs 3 and 4

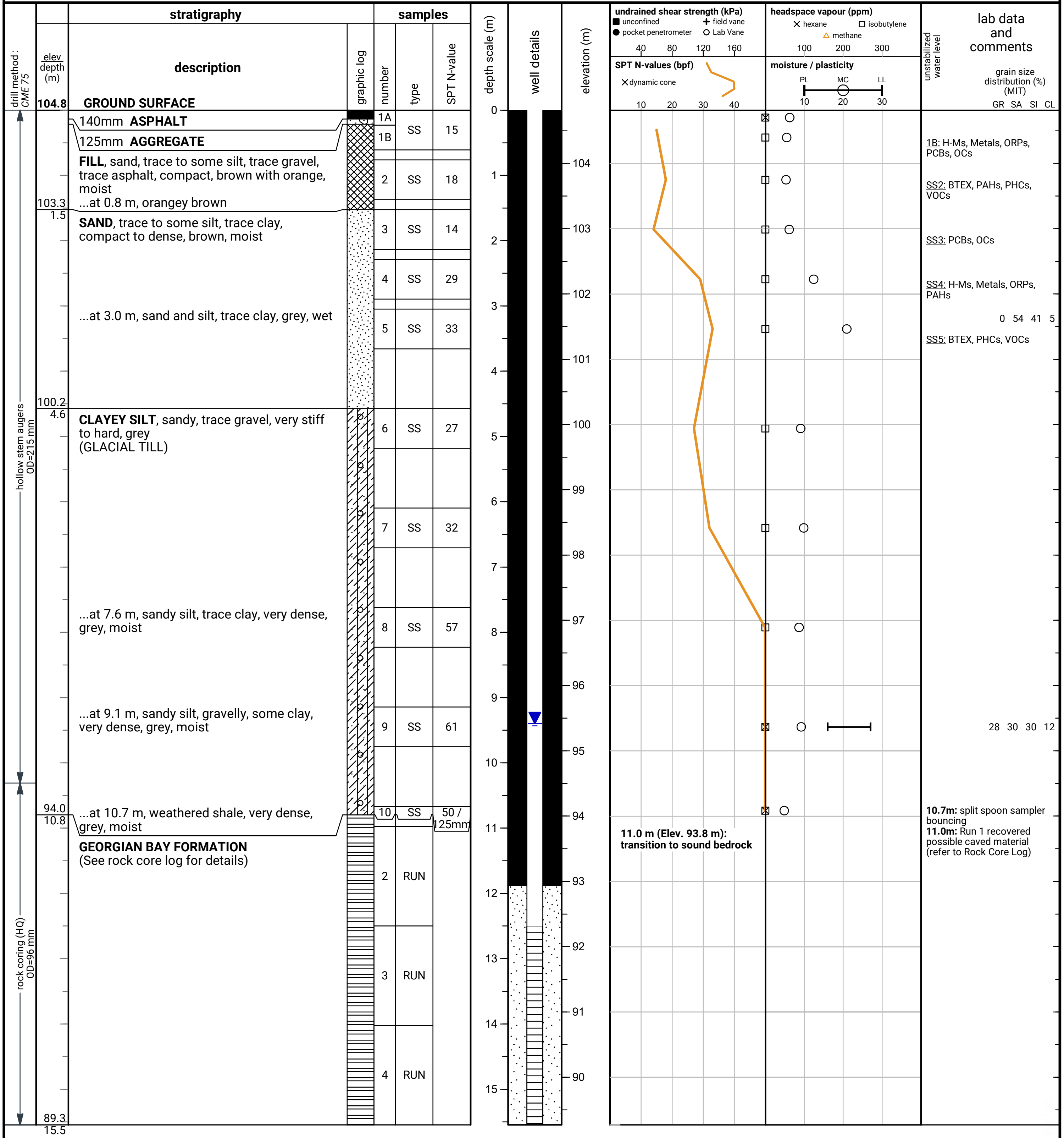


GROUNDED ENGINEERING

Date Started : Jun 27, 2024
Position : E: 612652, N: 4825476 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 3

File No. : 24-099 **Project** : 2233 & 2235 Hurontario St, Mississauga **Client** : Starlight Development



elev depth (m)	description	number	type	SPT N-value
104.8	**GROUND SURFACE**			
	140mm **ASPHALT**	1A	SS	15
	125mm **AGGREGATE**	1B		
	FILL, sand, trace to some silt, trace gravel, trace asphalt, compact, brown with orange, moist	2	SS	18
	...at 0.8 m, orangey brown			
103.3 1.5	**SAND**, trace to some silt, trace clay, compact to dense, brown, moist	3	SS	14
		4	SS	29
	...at 3.0 m, sand and silt, trace clay, grey, wet	5	SS	33
100.2 4.6	**CLAYEY SILT**, sandy, trace gravel, very stiff to hard, grey (GLACIAL TILL)	6	SS	27
		7	SS	32
	...at 7.6 m, sandy silt, trace clay, very dense, grey, moist	8	SS	57
	...at 9.1 m, sandy silt, gravelly, some clay, very dense, grey, moist	9	SS	61
94.0 10.8	...at 10.7 m, weathered shale, very dense, grey, moist	10	SS	50 / 125mm
	GEORGIAN BAY FORMATION (See rock core log for details)	2	RUN	
		3	RUN	
		4	RUN	
89.3 15.5				

11.0 m (Elev. 93.8 m): transition to sound bedrock

Lab data and comments:

1B: H-Ms, Metals, ORPs, PCBs, OCs

SS2: BTEX, PAHs, PHCs, VOCs

SS3: PCBs, OCs

SS4: H-Ms, Metals, ORPs, PAHs

GR	SA	SI	CL
0	54	41	5
28	30	30	12

SS5: BTEX, PHCs, VOCs

10.7m: split spoon sampler bouncing
11.0m: Run 1 recovered possible caved material (refer to Rock Core Log)

END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jul 4, 2024	5.6	99.2
Jul 12, 2024	5.5	99.3
Mar 4, 2026	9.4	95.4
Apr 1, 2026	9.4	95.4

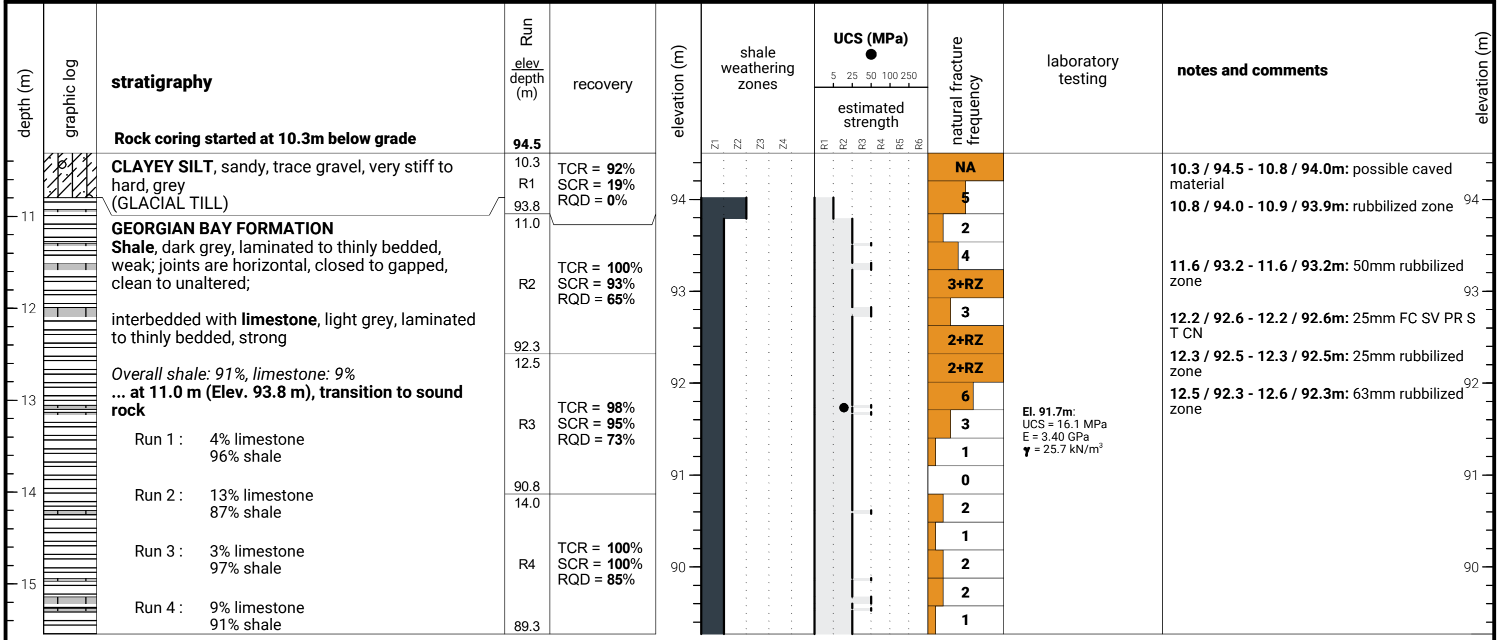
file: 24-099 gint.gpj

Tech : IH | **PM** : JAW/VT | **Rev** : NN

Date Started : Jun 27, 2024
Position : E: 612652, N: 4825476 (UTM 17T)
Elev. Datum : Geodetic

ROCK CORE LOG 3

File No. : 24-099 **Project** : 2233 & 2235 Hurontario St, Mississauga **Client** : Starlight Development



stratigraphy

Rock coring started at 10.3m below grade

CLAYEY SILT, sandy, trace gravel, very stiff to hard, grey
(GLACIAL TILL)

GEORGIAN BAY FORMATION
Shale, dark grey, laminated to thinly bedded, weak; joints are horizontal, closed to gapped, clean to unaltered;

interbedded with **limestone**, light grey, laminated to thinly bedded, strong

Overall shale: 91%, limestone: 9%
... at 11.0 m (Elev. 93.8 m), transition to sound rock

Run 1 : 4% limestone
96% shale

Run 2 : 13% limestone
87% shale

Run 3 : 3% limestone
97% shale

Run 4 : 9% limestone
91% shale

END OF COREHOLE

Run	elev / depth (m)	recovery
	94.5	
R1	10.3 – 11.0 (93.8)	TCR = **92%** SCR = **19%** RQD = **0%**
R2	11.0 – 12.5 (92.3)	TCR = **100%** SCR = **93%** RQD = **65%**
R3	12.5 – 14.0 (90.8)	TCR = **98%** SCR = **95%** RQD = **73%**
R4	14.0 – (89.3)	TCR = **100%** SCR = **100%** RQD = **85%**
	15.5m	

laboratory testing

El. 91.7m:
UCS = 16.1 MPa
E = 3.40 GPa
γ = 25.7 kN/m^3

notes and comments

10.3 / 94.5 - 10.8 / 94.0m: possible caved material

10.8 / 94.0 - 10.9 / 93.9m: rubbilized zone

11.6 / 93.2 - 11.6 / 93.2m: 50mm rubbilized zone

12.2 / 92.6 - 12.2 / 92.6m: 25mm FC SV PR S T CN

12.3 / 92.5 - 12.3 / 92.5m: 25mm rubbilized zone

12.5 / 92.3 - 12.6 / 92.3m: 63mm rubbilized zone

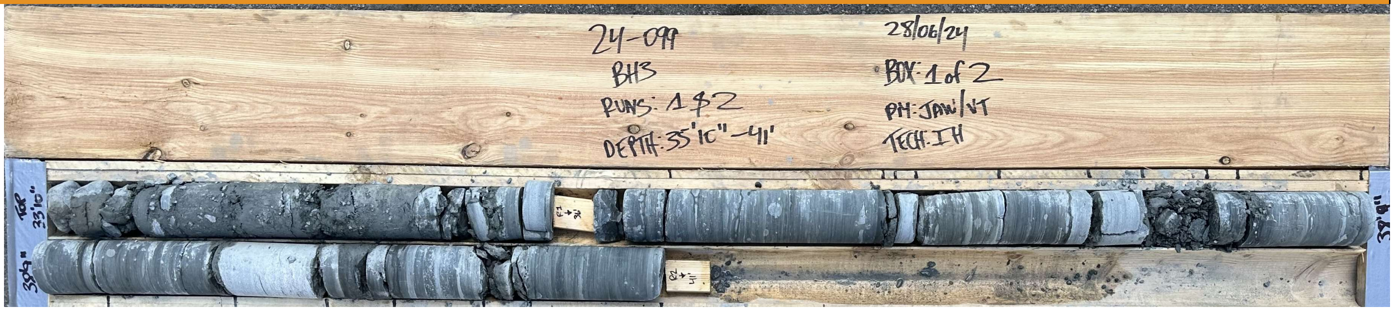
file: 24-099 gint.gpj

Tech : IH | **PM** : JAW/VT | **Rev** : NN

Borehole 3

Run #	Depth (m)	Elevation (m)
1	10.3 – 11.0	94.5 – 93.8
2	11.0 – 12.5	93.8 – 92.3
3	12.5 – 14.0	92.3 – 90.8
4	14.0 – 15.5	90.8 – 89.3

Runs 1 and 2



Runs 3 and 4

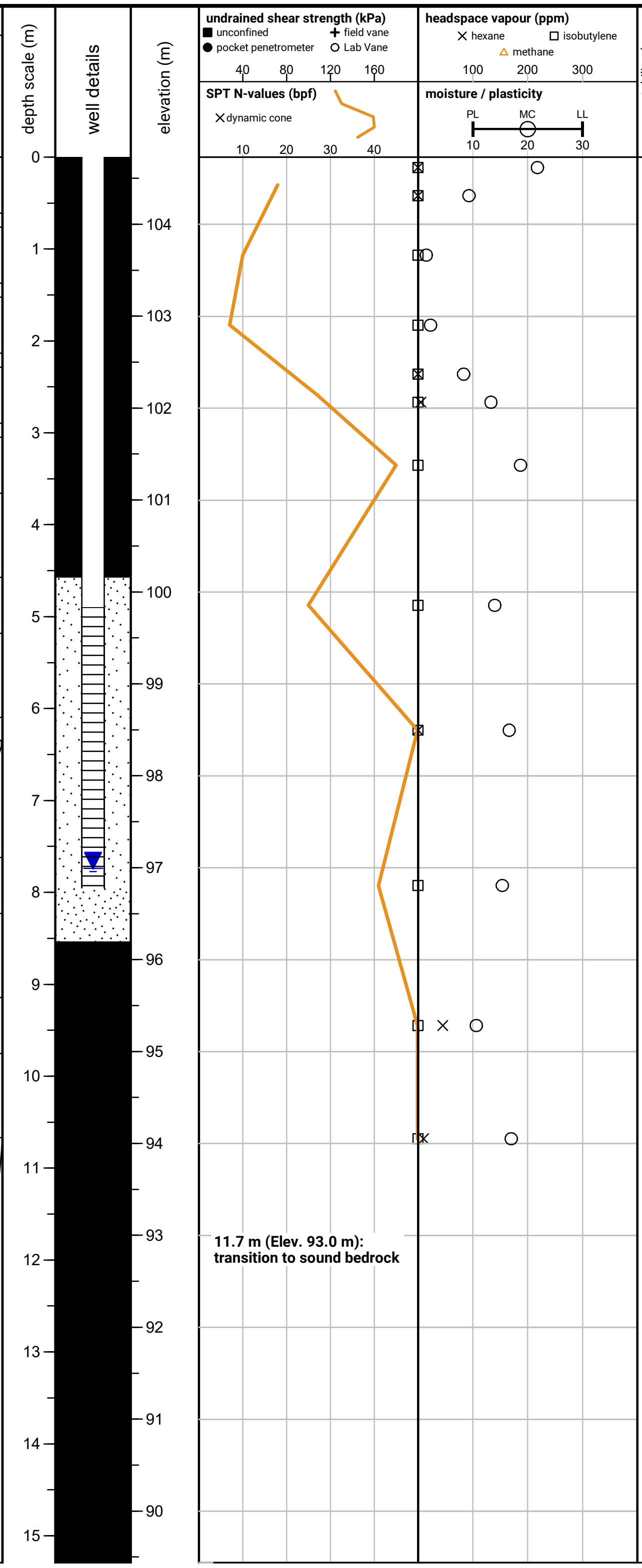
Date Started : Feb 25, 2026
Position : E: 612618, N: 4825400 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 201

File No. : 24-099 **Project** : 2233 & 2235 Hurontario St, Mississauga **Client** : Starlight Development

drill method : Geoprobe
- hollow stem augers OD=215 mm
- mud rotary med OD=125 mm
- Rock coring (HQ) OD=96 mm

elev depth (m)	stratigraphy: description	samples: number	samples: type	samples: SPT N-value	lab data and comments / grain size distribution (%) (MIT) GR SA SI CL
104.7	**GROUND SURFACE**				
	610mm **TOPSOIL**	1A 1B	SS	18	1B: PAHs, PCBs, OCs
104.1 0.6	**FILL**, sand, trace silt, trace gravel, loose, brown, moist	2	SS	10	SS2: BTEX, PCBs, OCs, PHCs, VOCs
		3	SS	7	SS3: H-Ms, Metals, ORPs
	...at 2.3 m, some silt, trace clay, trace rootlets	4A			
102.3 2.4	**SILTY SAND**, trace clay, compact to dense, brown, moist ...at 3.0 m, wet	4B	SS	27	
		5	SS	45	0 62 33 5 SS5: BTEX, PHCs
100.1 4.6	**SANDY SILT**, clayey, trace gravel, trace rock fragments, very stiff to compact, grey, moist (GLACIAL TILL)	6	SS	25	SS6: H-Ms, Metals, ORPs, PAHs, PCBs, OCs
	...at 6.1 m, some rock fragments	7	SS	50 / 125mm	**6.9m:** audible grinding
	...at 7.6 m, trace rock fragments	8	SS	41	12 30 37 21 SS8: VOCs
		9	SS	84	
	...at 10.7 m, some rock fragments, shale	10	SS	50 / 25mm	**10.7m:** spoon bouncing
93.9 10.8	**GEORGIAN BAY FORMATION** (See rock core log for details)	1	RUN		
		2	RUN		
		3	RUN		
89.4 15.3	**END OF BOREHOLE**				



Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Mar 4, 2026	7.2	97.5
Apr 1, 2026	7.7	97.0

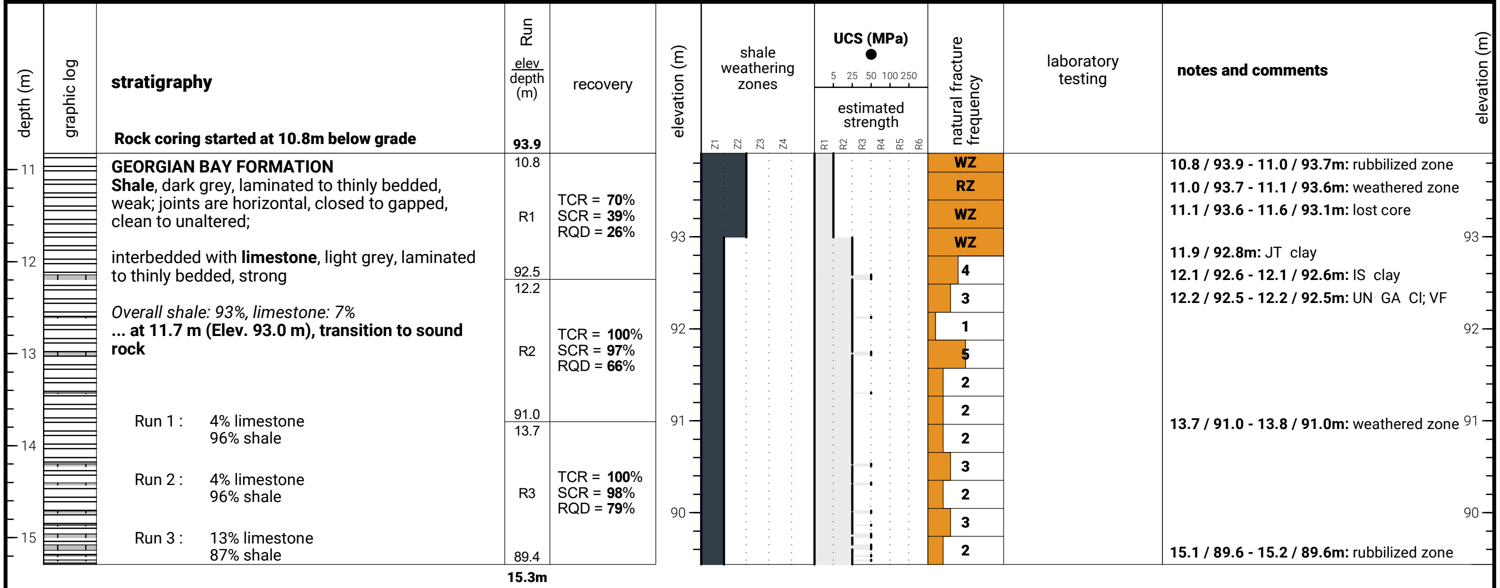
file: 24-099 gint.gpj

Tech : AJ | **PM** : RS | **Rev** : NN

Date Started : Feb 25, 2026
Position : E: 612618, N: 4825400 (UTM 17T)
Elev. Datum : Geodetic

ROCK CORE LOG 201

File No. : 24-099 **Project** : 2233 & 2235 Hurontario St, Mississauga **Client** : Starlight Development



stratigraphy

Rock coring started at 10.8m below grade

GEORGIAN BAY FORMATION
Shale, dark grey, laminated to thinly bedded, weak; joints are horizontal, closed to gapped, clean to unaltered;

interbedded with **limestone**, light grey, laminated to thinly bedded, strong

Overall shale: 93%, limestone: 7%
... at 11.7 m (Elev. 93.0 m), transition to sound rock

Run 1 : 4% limestone
96% shale

Run 2 : 4% limestone
96% shale

Run 3 : 13% limestone
87% shale

END OF COREHOLE

Run	elev / depth (m)	recovery
	93.9	
R1	10.8 – 92.5	TCR = **70%** SCR = **39%** RQD = **26%**
R2	12.2 – 91.0	TCR = **100%** SCR = **97%** RQD = **66%**
R3	13.7 – 89.4	TCR = **100%** SCR = **98%** RQD = **79%**
	15.3m	

natural fracture frequency: WZ, RZ, WZ, WZ, 4, 3, 1, 5, 2, 2, 2, 3, 2, 3, 2

notes and comments

10.8 / 93.9 - 11.0 / 93.7m: rubbilized zone
11.0 / 93.7 - 11.1 / 93.6m: weathered zone
11.1 / 93.6 - 11.6 / 93.1m: lost core
11.9 / 92.8m: JT clay
12.1 / 92.6 - 12.1 / 92.6m: IS clay
12.2 / 92.5 - 12.2 / 92.5m: UN GA Cl; VF
13.7 / 91.0 - 13.8 / 91.0m: weathered zone
15.1 / 89.6 - 15.2 / 89.6m: rubbilized zone

file: 24-099 gint.gpj

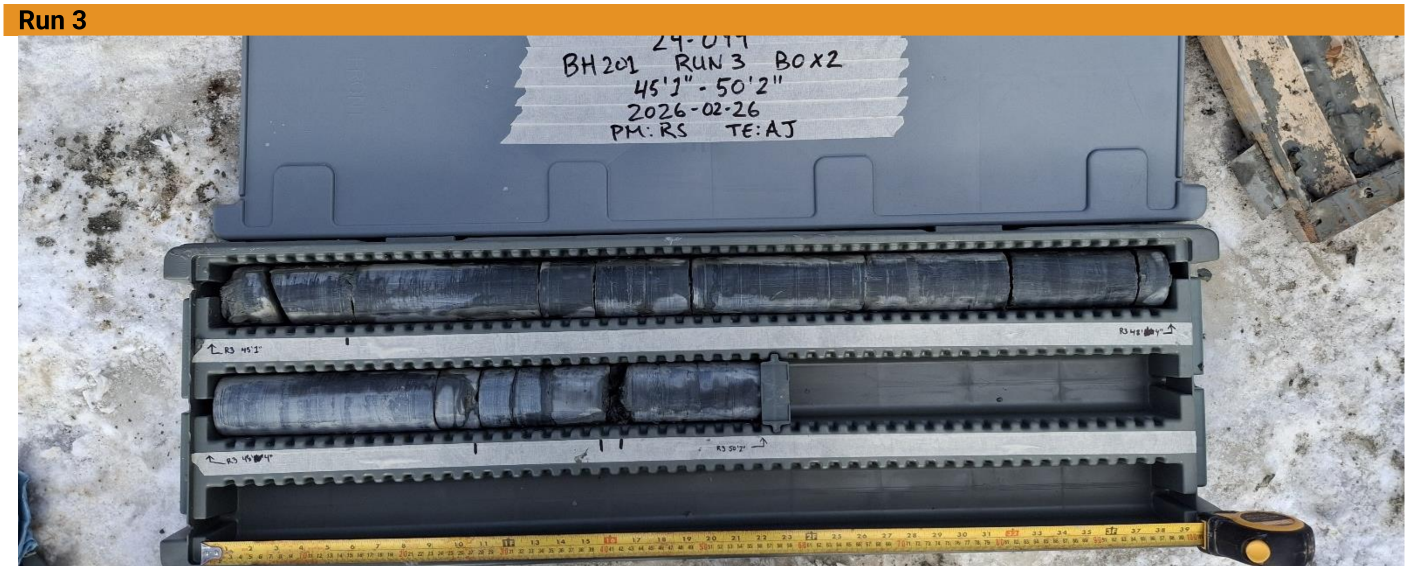
Tech : AJ | **PM** : RS | **Rev** :

Borehole 201

Run #	Depth (m)	Elevation (m)
1	10.8 – 12.2	93.9 – 92.5
2	12.2 – 13.7	92.5 – 91.0
3	13.7 – 15.3	91.5 – 89.4

Runs 1 and 2

Run 3



GROUNDED ENGINEERING

Date Started : Mar 2, 2026
Position : E: 612640, N: 4825429 (UTM 17T)
Elev. Datum : Geodetic

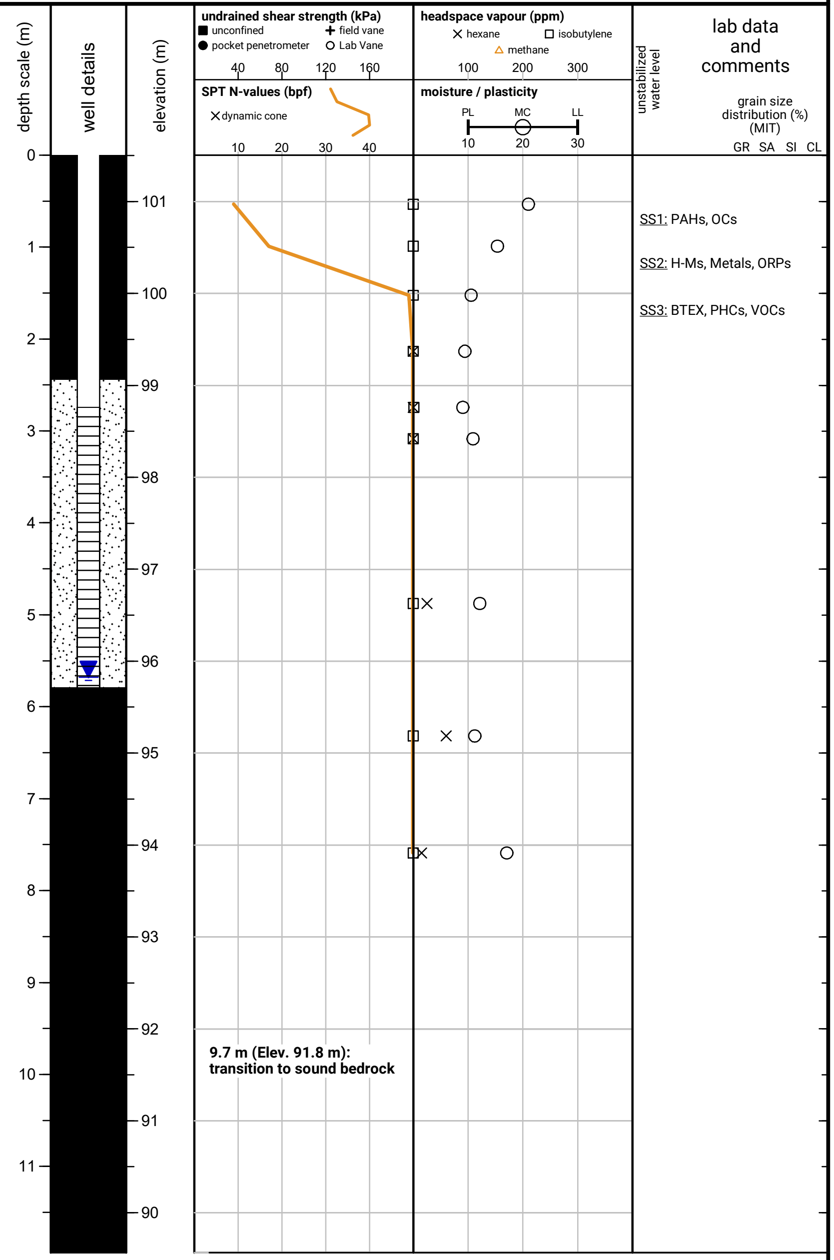
BOREHOLE LOG 202

File No. : 24-099 **Project** : 2233 & 2235 Hurontario St, Mississauga **Client** : Starlight Development

drill method : *Hilti DD350*

elev depth (m)	description (stratigraphy)	drill method
101.5	**TOP OF SLAB**	
101.2 0.3	165mm **CONCRETE**	mud rotary small OD=100 mm
	140mm **AGGREGATE**	
100.7 0.8	**SILTY SAND**, loose, brown, moist	
	SANDY SILT, clayey, trace gravel, very stiff, greyish brown, moist (GLACIAL TILL) ...at 1.2 m, grey, hard ...at 6.1 m, layer of sandy silt till, some clay, some shale fragments, grey, moist, very dense	
93.9 7.6	**GEORGIAN BAY FORMATION** (See rock core log for details)	Rock coring (NQ) OD=76 mm
89.6 11.9		

number	type	SPT N-value
1	SS	9
2	SS	17
3	SS	49
4	SS	67
5	SS	82
6	SS	50 / 75mm
7	SS	55
8	SS	86 / 290mm
9	SS	50 / 50mm
1	RUN	
2	RUN	
3	RUN	
4	RUN	
5	RUN	
6	RUN	



9.7 m (Elev. 91.8 m): transition to sound bedrock

SS1: PAHs, OCs

SS2: H-Ms, Metals, ORPs

SS3: BTEX, PHCs, VOCs

END OF BOREHOLE

Dry and open upon completion of drilling.

GROUNDWATER LEVELS

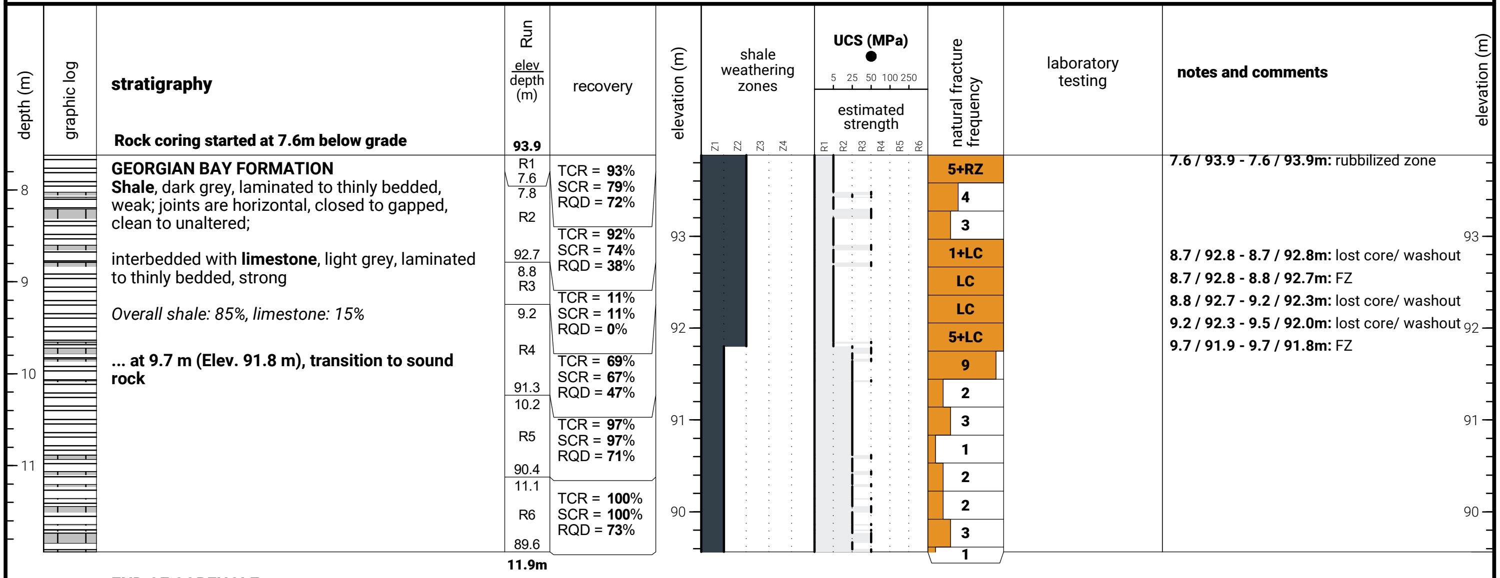
date	depth (m)	elevation (m)
Apr 1, 2026	5.7	95.8

file: 24-099 gint.gpj

Date Started : Mar 2, 2026
Position : E: 612640, N: 4825429 (UTM 17T)
Elev. Datum : Geodetic

ROCK CORE LOG 202

File No. : 24-099   **Project** : 2233 & 2235 Hurontario St, Mississauga   **Client** : Starlight Development



stratigraphy

Rock coring started at 7.6m below grade

GEORGIAN BAY FORMATION
Shale, dark grey, laminated to thinly bedded, weak; joints are horizontal, closed to gapped, clean to unaltered;

interbedded with **limestone**, light grey, laminated to thinly bedded, strong

Overall shale: 85%, limestone: 15%

... at 9.7 m (Elev. 91.8 m), transition to sound rock

Run	elev / depth (m)	recovery
	93.9	
R1	7.6 – 7.8	TCR = **93%** SCR = **79%** RQD = **72%**
R2	92.7 / 8.8	TCR = **92%** SCR = **74%** RQD = **38%**
R3	9.2	TCR = **11%** SCR = **11%** RQD = **0%**
R4	91.3 / 10.2	TCR = **69%** SCR = **67%** RQD = **47%**
R5	90.4 / 11.1	TCR = **97%** SCR = **97%** RQD = **71%**
R6	89.6 / **11.9m**	TCR = **100%** SCR = **100%** RQD = **73%**

notes and comments

7.6 / 93.9 - 7.6 / 93.9m: rubbilized zone
8.7 / 92.8 - 8.7 / 92.8m: lost core/ washout
8.7 / 92.8 - 8.8 / 92.7m: FZ
8.8 / 92.7 - 9.2 / 92.3m: lost core/ washout
9.2 / 92.3 - 9.5 / 92.0m: lost core/ washout
9.7 / 91.9 - 9.7 / 91.8m: FZ

END OF COREHOLE

Run 1 : 0% limestone
100% shale

Run 2 : 19% limestone
81% shale

Run 3 : 8% limestone
92% shale

Run 4 : 12% limestone
88% shale

Run 5 : 8% limestone
92% shale

Run 6 : 31% limestone
69% shale

file: 24-099 gint.gpi

Tech : LB | **PM** : RS | **Rev** : NN

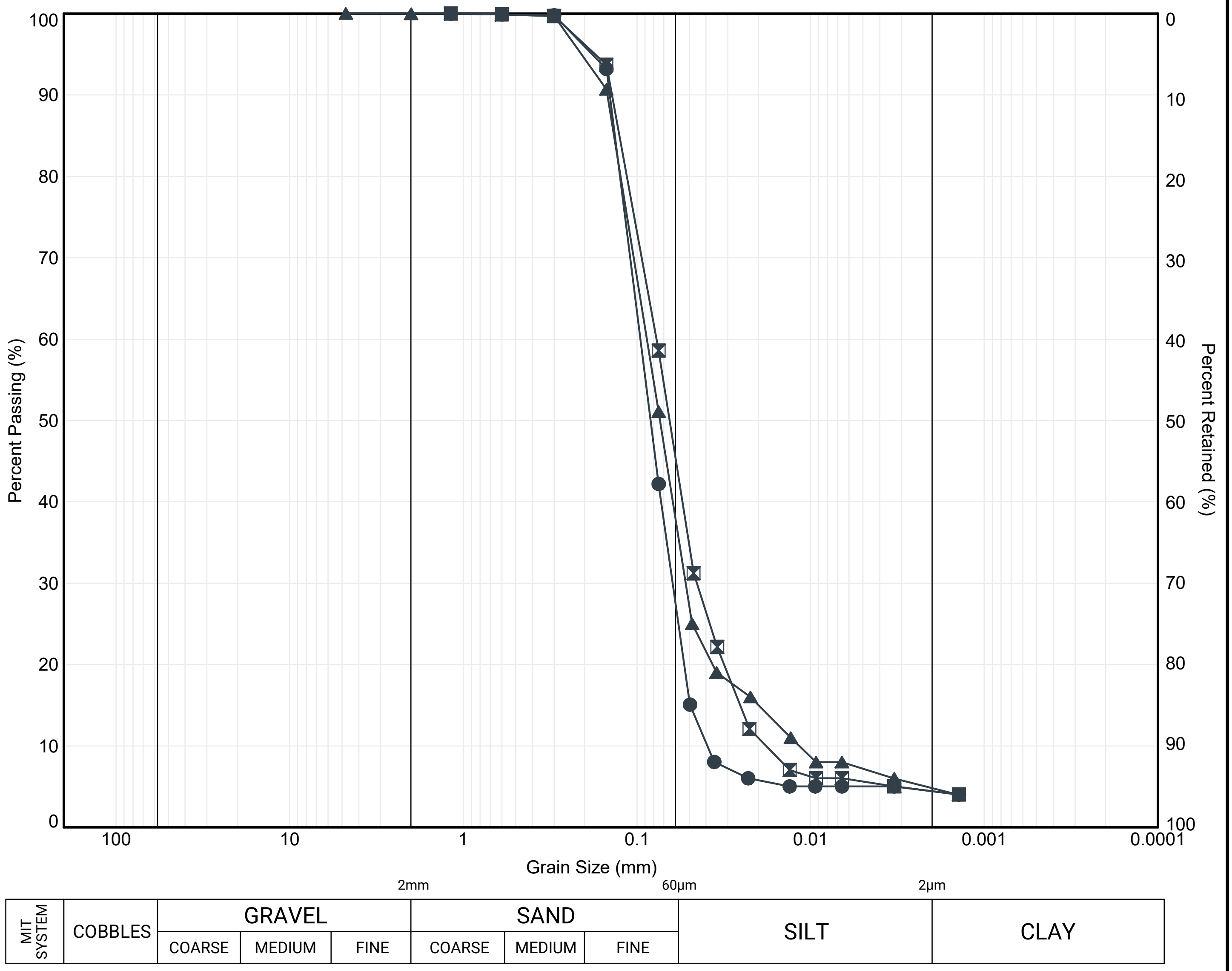
Borehole 202

Run #	Depth (m)	Elevation (m)
1	7.6 – 7.8	93.9 – 93.7
2	7.8 – 8.8	93.7 – 92.7
3	8.8 – 9.2	92.7 – 92.3
4	9.2 – 10.2	92.3 – 91.3
5	10.2 – 11.1	91.3 – 90.4
6	11.1 – 11.9	90.4 – 89.6

Runs 1 to 5

Run 6

APPENDIX B



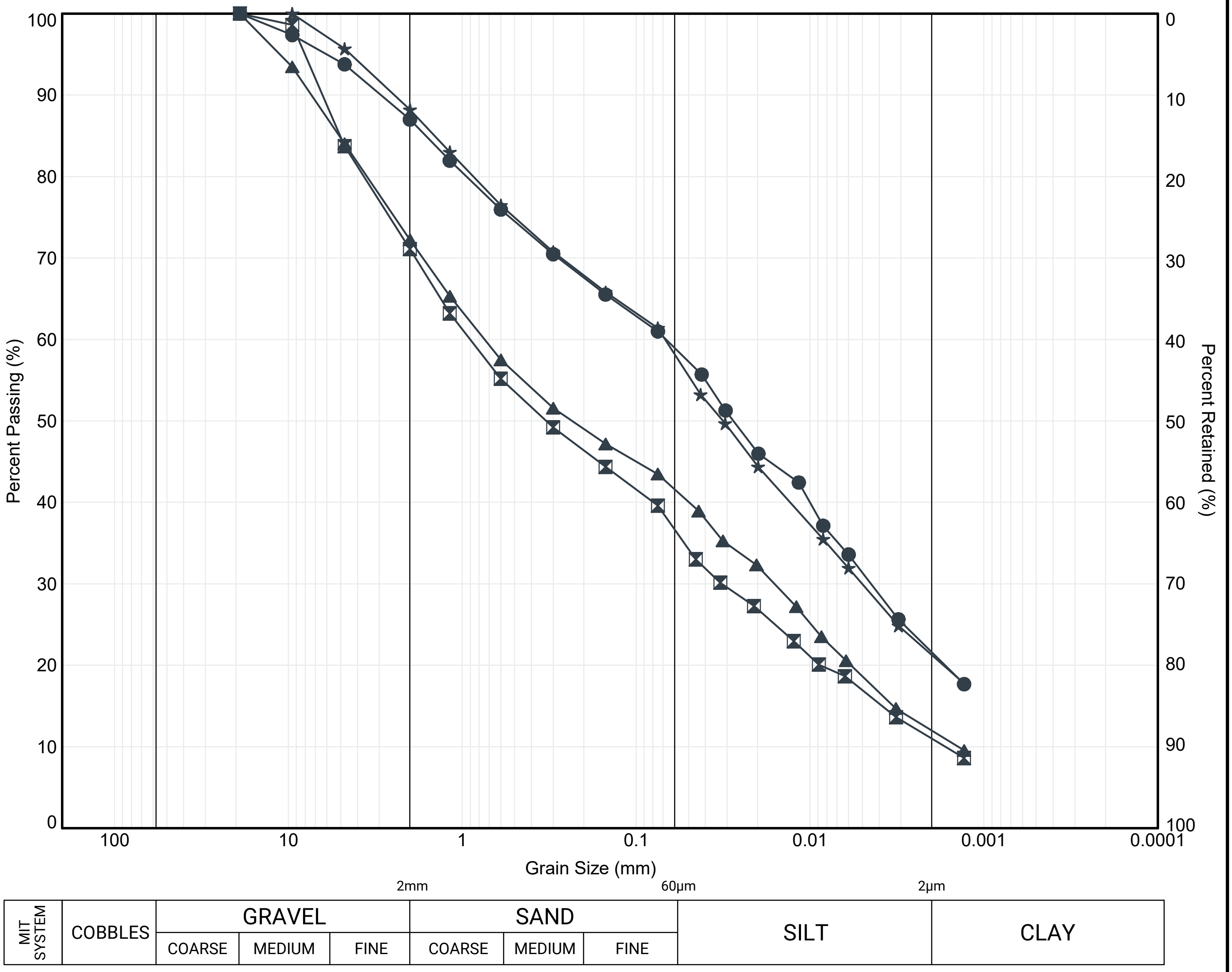
MIT SYSTEM

	Borehole	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	**1**	SS5	3.4	102.1	0	73	23	4
☒	**3**	SS5	3.4	101.5	0	54	41	5
▲	**201**	SS5	3.4	101.4	0	62	33	5

GROUNDED
ENGINEERING

Title: **GRAIN SIZE DISTRIBUTION**
SANDS

File No.: **24-099**



MIT SYSTEM

	Borehole	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	**1**	SS8	5.6	99.8	13	28	37	22
☒	**2**	10B	9.6	95.5	29	34	26	11
▲	**3**	SS9	9.4	95.4	28	30	30	12
★	**201**	SS8	7.9	96.8	12	30	37	21

GROUNDED
ENGINEERING

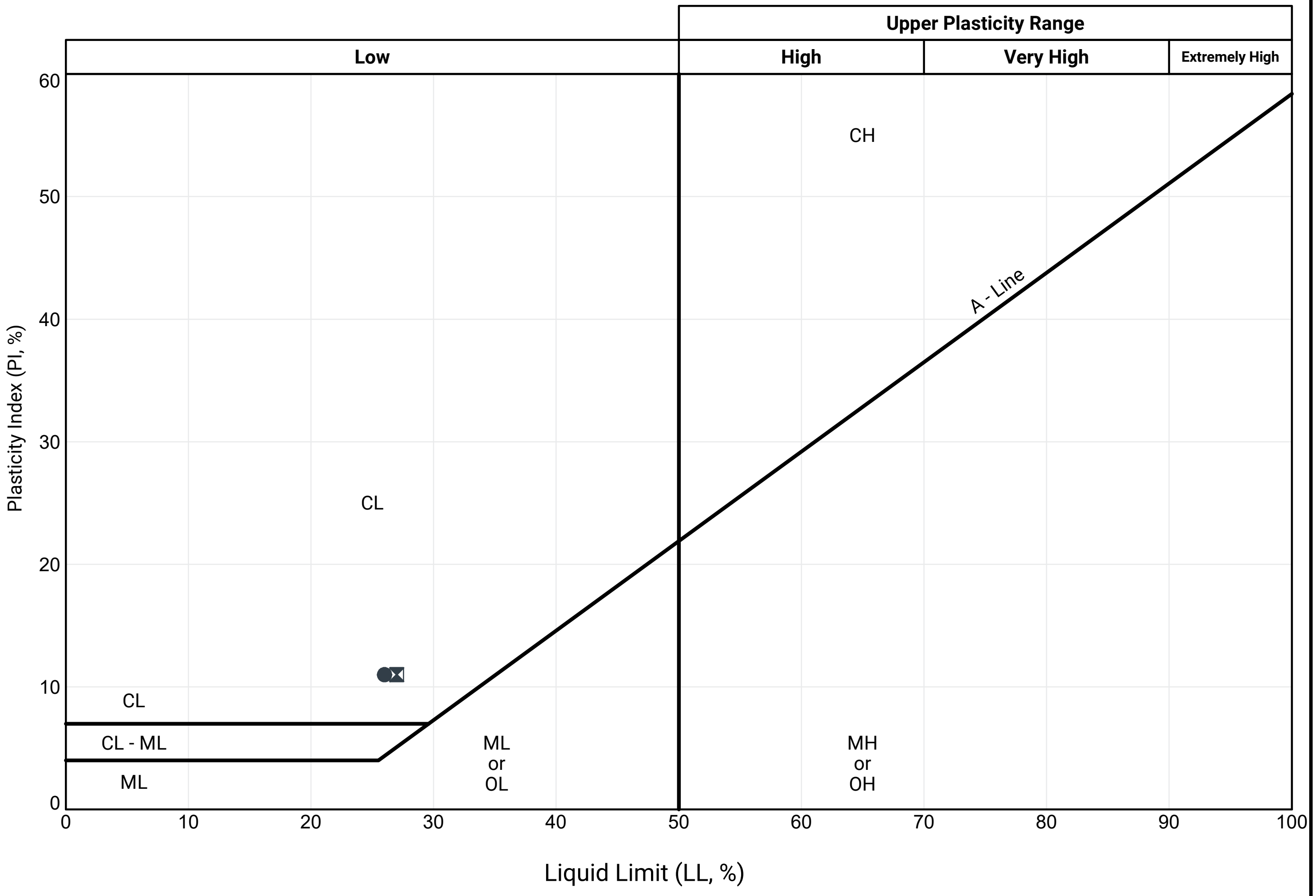
Title:

GRAIN SIZE DISTRIBUTION
GLACIAL TILL

File No.:

24-099

file: 24-099 gint.gpj



	Borehole	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)
●	**1**	SS8	5.6	99.8	26	15	11
☒	**3**	SS9	9.4	95.4	27	16	11

file: 24-099 gint.gpj

GROUNDED ENGINEERING

Title:

ATTERBERG LIMITS CHART

File No.: **24-099**

APPENDIX C

Geomechanica Inc.
#14-1240 Speers Rd.
Oakville ON
L6L 2X4 Canada

July 10, 2024

James Wagner
Grounded Engineering Inc.
1 Banigan Drive,
Toronto, Ontario
Canada, M4H 1G3

Re: UCS Testing (Grounded Engineering Inc. Project No. 24-099)

Dear James Wagner:

On July 04, 2024 a series of three (3) core samples (HQ-sized) were received by Geomechanica Inc. via drop-off by Grounded personnel. These samples were identified as being from Grounded Engineering Inc. Project No. 24-099. From these samples, 3 Uniaxial Compressive Strength (UCS) tests were completed.

Details regarding the steps of specimen preparation and testing along with the test results are presented in the accompanying laboratory report and summary spreadsheet.

Sincerely,

Bryan Tatone, PhD, PEng

Geomechanica Inc.
Tel: +1-647-478-9767
lab@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:
James Wagner
Grounded Engineering Inc.
1 Banigan Drive,
Toronto, Ontario
Canada, M4H 1G3

Prepared by:
Bryan Tatone, PhD, PEng

Omid Mahabadi, PhD, PEng
Geomechanica Inc.
#14-1240 Speers Rd.
Oakville ON
L6L 2X4 Canada
Tel: +1-647-478-9767
lab@geomechanica.com

July 10, 2024
Project number: 24-099

Abstract

This document summarizes the results of rock laboratory testing of 3 Uniaxial Compressive Strength (UCS) tests. The UCS values and Young's modulus values along with photographs of specimens before and after testing are presented herein.

In this document:

1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing. The testing was performed in Geomechanica Inc.'s rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.150 mm/min (Figure 1). The preparation and testing procedure for each specimen included the following:

1. Unwrapping the core sample, inspecting it for damage, and re-wrapping it in electrical tape to minimize exposure to moisture during subsequent specimen preparation.
2. Diamond cutting the core sample to obtain a cylindrical specimen with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding the specimen to obtain flat (within ±0.025 mm) and parallel end faces (within 0.25°).
4. Placing the specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axially loading the specimen to rupture while continuously recording axial force to determine the peak strength (UCS) and the axial deformation to determine the tangent Young's modulus.

Figure 1: Forney loading frame setup for UCS testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-19. The side straightness criteria, as checked with a feeler gauge, and the minimum length:diameter criteria were met for all specimens unless noted otherwise in Table 1. Testing of the specimens included the measurement of the UCS and elastic modulus, but not the Poisson's ratio. This represents a hybrid between Methods C and D of ASTM D7012-14.

1.2 Results

The results of UCS testing are summarized in Table 1. The corresponding stress-strain curves are presented in Figure 2. The Young's modulus is the tangent modulus calculated as the slope of the best-fit line through a selection of data points defining the stress-strain curve. Typically the modulus is defined at 50% of the UCS strength. However, due to non-linear pre-peak stress-strain behaviour of some specimens, a custom stress range (where the specimen deformed linearly) was selected for modulus determination. These stress ranges along with additional specimen details and measurements are provided in the summary spreadsheet that accompanies this report.

Table 1: Summary of UCS test results.

Sample	Depth (ft' in")	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's modulus E (GPa)	Lithology	Failure description
BH1, CS1	44'10" - 45'4.5"	2.585	12.1	2.9	Shale	1, 2, 3
BH3, CS2	46'8.5" - 47'3"	2.615	17.0	3.0	Shale	4, 3
BH2, CS3	42'11" - 43'7"	2.617	16.1	3.4	Shale	1, 3

[1] Inclined shear failure

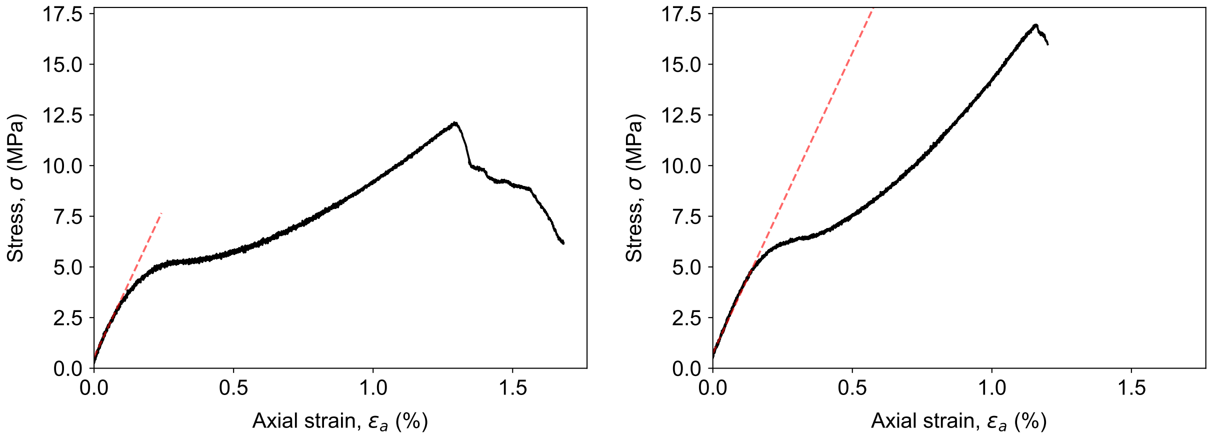
[2] Partial hourglass failure

[3] Specimen emitted pore water upon loading

[4] Inclined shear fracture and axial splitting failure

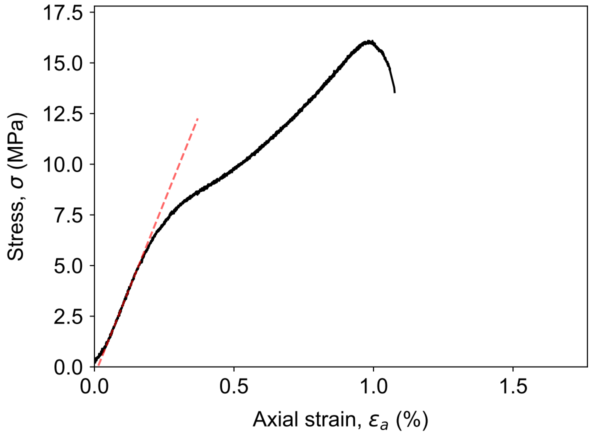
1.3 Specimen photographs

Photographs of the specimens before and after testing are presented in the Appendix of this report.



(a) BH1-CS1

(b) BH3-CS2



(c) BH2-CS3

Figure 2: Measured stress-strain curves.

Appendices

Specimen sheets

- BH1, CS1
- BH3, CS2
- BH2, CS3

Uniaxial Compression Test

Client	Grounded Engineering Inc.	**Project**	24-099
Sample	BH1, CS1	**Depth**	44’10” - 45’4.5”

Specimen parameters

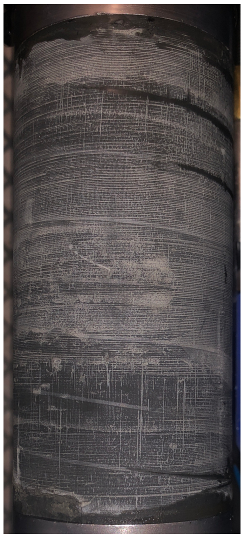
Diameter (mm) [a]	63.18
Length (mm) [a]	125.69
Bulk density ρ (g/cm^3)	2.585
UCS (MPa)	12.1
Young’s modulus E (GPa) [b]	2.9
Lithology	Shale
Failure description [c]	1, 2, 3

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.

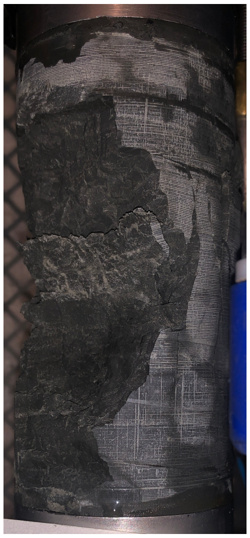
[b] Tangent modulus, calculated as the slope of the best fit line through ±43 data points on either side of the point representing 15.0% of the peak strength.

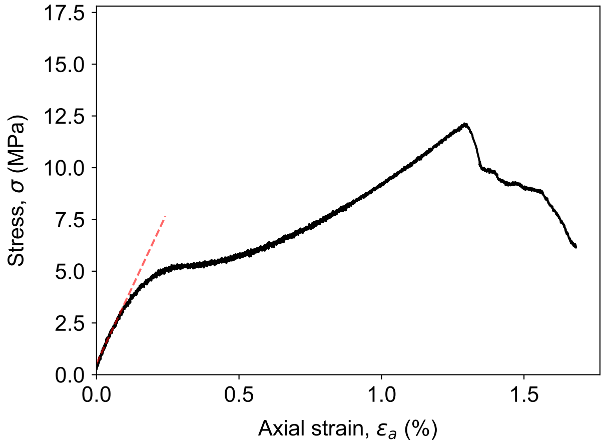
[c] Failure description: [1] Inclined shear failure; [2] Partial hourglass failure; [3] Specimen emitted pore water upon loading;

Prior to testing



After testing





Remarks: Loading Rate: 0.15 mm/min.

Performed by	AB	**Date**	2024-07-08

Uniaxial Compression Test

Client	Grounded Engineering Inc.	**Project**	24-099
Sample	BH3, CS2	**Depth**	46’8.5” - 47’3”

Specimen parameters

Diameter (mm) [a]	63.11
Length (mm) [a]	125.81
Bulk density ρ (g/cm^3)	2.615
UCS (MPa)	17.0
Young’s modulus E (GPa) [b]	3.0
Lithology	Shale
Failure description [c]	4, 3

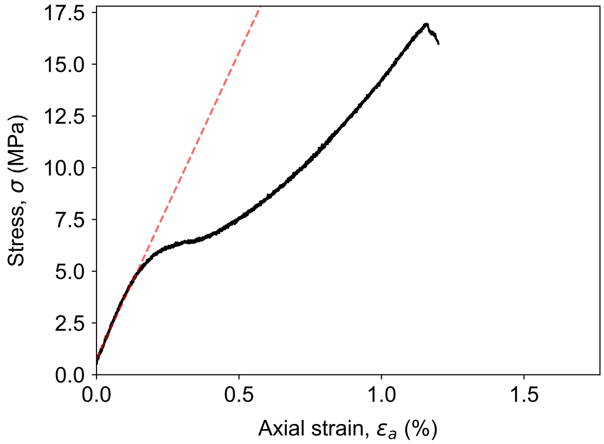
[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.

[b] Tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 15.0% of the peak strength.

[c] Failure description: [4] Inclined shear fracture and axial splitting failure; [3] Specimen emitted pore water upon loading;

Prior to testing

After testing



Remarks: Loading Rate: 0.15 mm/min.

Performed by	AB	**Date**	2024-07-08

Uniaxial Compression Test

Client	Grounded Engineering Inc.	**Project**	24-099
Sample	BH2, CS3	**Depth**	42’11” - 43’7”

Specimen parameters

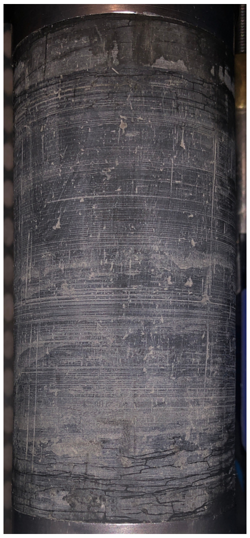
Diameter (mm) [a]	62.92
Length (mm) [a]	123.61
Bulk density ρ (g/cm^3)	2.617
UCS (MPa)	16.1
Young’s modulus E (GPa) [b]	3.4
Lithology	Shale
Failure description [c]	1, 3

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.

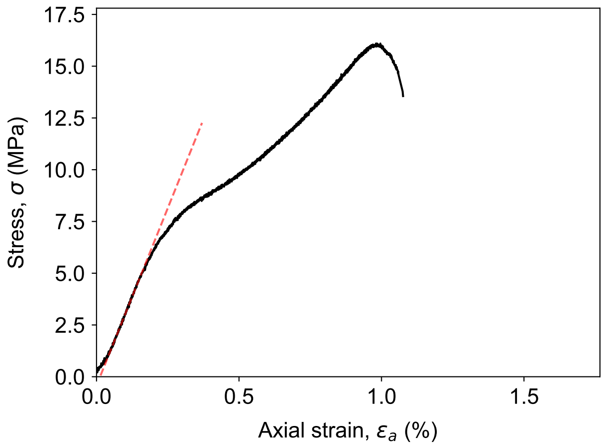
[b] Tangent modulus, calculated as the slope of the best fit line through ±68 data points on either side of the point representing 15.0% of the peak strength.

[c] Failure description: [1] Inclined shear failure; [3] Specimen emitted pore water upon loading;

Prior to testing



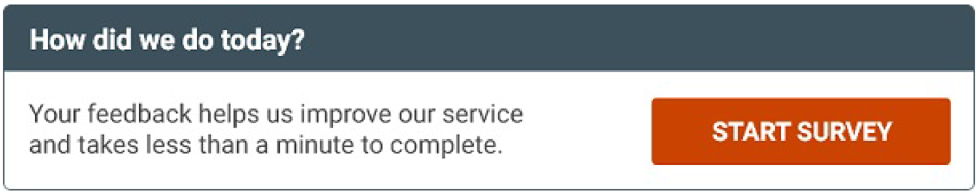
After testing



Remarks: Loading Rate: 0.15 mm/min.

Performed by	AB	**Date**	2024-07-08

APPENDIX D


How did we do today?

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

FINAL REPORT

CA40055-MAR26 R1

24-099, 2233 Hurontario

Prepared for

Grounded Engineering Inc.

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Grounded Engineering Inc.	Project Specialist	Brad Moore Hon. B.Sc
Address	49 Mobile Drive Toronto, Ontario M4A 1H5. Canada	Laboratory	SGS Canada Inc.
		Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Ruth Schoenhardt		
Telephone		Telephone	705-652-2143
Facsimile		Facsimile	705-652-6365
Email	rschoenhardt@groundedeng.ca	Email	brad.moore@sgs.com
Works #		SGS Reference	CA40055-MAR26
Project	24-099, 2233 Hurontario	Received	2026-03-05
Reference		Approved	03/11/2026
Batch		Report Number	CA40055-MAR26 R1
Samples	SOIL (3)	Date Reported	03/11/2026

COMMENTS

Temperature of Sample upon Receipt: 3 degrees C
Cooling Agent Present: yes
Custody Seal Present: yes

Chain of Custody Number: 045489

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	BH201 SS4B	BH201 SS10	BH202 SS1
				Sample Matrix	Soil	Soil	Soil
				Sample Date	2026-03-03 00:00	2026-03-03 00:00	2026-03-05 00:00

Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
Corrosivity Index						
Corrosivity Index	none	1		14	11	14
pH	pH Units	0.05		9.35	9.23	8.60
Soil Redox Potential	mV	no		247	51	159
Sulphide (Na2CO3)	%	0.01		< 0.01	0.05	0.01
Resistivity (calculated)	ohms.cm	-9999		1060	3830	620
General Chemistry						
Conductivity	uS/cm	2		942	261	1610
Metals and Inorganics						
Sulphate	µg/g	0.4		30	50	92
Other (ORP)						
Chloride	µg/g	0.4		470	31	880

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	DIO0199-MAR26	µg/g	0.4	<0.4	1	35	98	80	120	100	75	125
Sulphate	DIO0199-MAR26	µg/g	0.4	<0.4	10	35	97	80	120	97	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)	ECS0024-MAR26	%	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	EWL0186-MAR26	uS/cm	2	< 2	1	20	100	90	110	NA		

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	EWL0186-MAR26	pH Units	0.05	NA	0		100			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

Test method information available upon request.
"Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

SGS Canada Inc. statement of conformity decision rule does not consider uncertainty when analytical results are compared to a specified standard or regulation.

-- End of Analytical Report --

SGS

Request for Laboratory Services and CHAIN OF CUSTODY

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

No: U45489

Page 1 of 1

Laboratory Information Section - Lab use only

Received By: ED
Received Date: 3 5 26 (mm/dd/yy)
Received Time: 10 30 (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) 3.0 x3
LAB LIMS #: CA 40055-Mar26 AK

REPORT INFORMATION

Company: GroundedEngineering
Contact: Ruth Schoenhardt
Address: 49 Mobile Drive
Toronto ON, M4A 1H5
Phone:
Fax:
Email: rschoenhardt@groundedeng.ca

INVOICE INFORMATION

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

Quotation #:
P.O. #:
Project #: 24-099
Site Location/ID: 2233 Hurontario

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 ☐ O.Reg 406/19
☐ Table 1 ☐ Res/Park Soil Texture:
☐ Table 2 ☐ Ind/Com ☐ Coarse
☐ Table 3 ☐ Agri/Other ☐ Medium/Fine
☐ Table ______ Appx.
Soil Volume ☐ <350m3 ☐ >350m3

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

M & I: Field Filtered (Y/N); Metals & Inorganics incl CrVI, CN,Hg pH,(B(HWS),EC,SAR-soil) (Cl, Na-water); Full Metals Suite ICP metals plus B(HWS-soil only) Hg, CrVI; ICP Metals only Sb,As,Ba,Be,B,Cd, Cr,Co,Cu,Pb,Mo,Ni,Se,Ag,Tl,U,V,Zn
SVOC: PAHs only; SVOCs all incl PAHs, ABNs, CPs
PCB: PCBs Total ☐ Aroclor ☐
PHC: F1-F4 + BTEX; F1-F4 only no BTEX
VOC: VOCs all incl BTEX; BTEX only
Pest: Pesticides Organochlorine or specify other
Other (please specify): Corrosivity
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.

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	# OF BOTTLES	MATRIX	Field Filtered (Y/N)	Metals & Inorganics	Full Metals Suite	ICP Metals only	PAHs only	SVOCs	PCBs	F1-F4 + BTEX	F1-F4 only	VOCs	BTEX only	Pesticides	Corrosivity				Sewer Use	Water Characterization Pkg	SPLP	TCLP	COMMENTS:
1	BH201 SS 4B	March 3 2026	1400	1	Soil													X								
2	BH201 SS10	March 3 2026	1400	1	Soil													X								
3	BH202 SS1	March 5 2026	8:30	1	Soil													X								
4																										
5																										
6																										
7																										
8																										
9																										
10																										
11																										
12																										

Observations/Comments/Special Instructions

Sampled By (NAME): Ruth Schoenhardt Signature: Date: 2026/05/03 ~~(mm/dd/yy)~~ YY/dd/MM
Relinquished by (NAME): Ruth Schoenhardt Signature: Date: 2026/05/03 (mm/dd/yy)

Pink Copy - Client
Yellow & White Copy - SGS

Revision #: 1.8
Date of Issue: 06 SEP 2024

Note: Submission of samples to SGS is acknowledgement that you have been provided direction on sample collection/handling and transportation of samples. {2} Submission of samples to SGS is considered authorization for completion of work. Signatures may appear on this form or be retained on file in the contract, or in an alternative format (e.g. shipping documents). {3} Results may be sent by email to an unlimited number of addresses for no additional cost. Fax is available upon request. This document is issued by the Company under its General Conditions of Service accessible at http://www.sgs.com/terms_and_conditions.htm. (Printed copies are available upon request.) Attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein.

CORROSIVITY (SGS)

Report No.	**CA40055-MAR26**
Customer	**Grounded Engineering Inc.**
Attention	**Ruth Schoenhardt**
Reference	**24-099, Ruth Schoenhardt**
Works#	
Title	**Final Report**

Sample ID		Analysis Start Date	Analysis Start Time	Analysis Completed Date	Analysis Completed Time	BH201 SS4B	BH201 SS10	BH202 SS1
Sample Date / Time						03-Mar-26 14:00	03-Mar-26 14:00	05-Mar-26 08:30
Analysis	**Units**							
Corrosivity Index	none	10-Mar-26	14:40	10-Mar-26	14:40	14	11	14
Soil Redox Potential	mV	06-Mar-26	14:23	09-Mar-26	15:14	247	51	159
Sulphide (Na2CO3)	%	10-Mar-26	10:12	10-Mar-26	11:01	< 0.01	0.05	0.01
Moisture Index	no unit	06-Mar-26	14:23	09-Mar-26	15:14	1	1	1
pH	pH Units	10-Mar-26	09:38	10-Mar-26	14:40	9.35	9.23	8.60
Chloride	µg/g	10-Mar-26	07:47	11-Mar-26	15:56	470	31	880
Sulphate	µg/g	10-Mar-26	07:47	11-Mar-26	15:56	30	50	92
Conductivity	uS/cm	10-Mar-26	09:38	10-Mar-26	14:39	942	261	1610
Resistivity (calculated)	ohms.cm	10-Mar-26	09:38	10-Mar-26	14:40	1060	3830	620

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	YES	NO
Are Sulphides present ?	NO	YES	YES
If above three conditions are met, pH is assigned 3 points			
pH - Score	**3**	**3**	**3**
Redox Potential	**0**	**3.5**	**0**
Resistivity	**10**	**0**	**10**
Acid Volatile Sulphides	**0**	**0**	**0**
TOTAL SCORE (AWWA C-105)	**14**	**7.5**	**14**

Sample		**BH201 SS4B**	**BH201 SS10**	**BH202 SS1**
Corrosion Protection Recommended?		**YES**	**No**	**YES**
Sulphate	**%**	0.003%	0.005%	0.009%
CLASS OF EXPOSURE		**Negligible**	**Negligible**	**Negligible**

APPENDIX E

Next Generation Geophysics

SIMCOE GEOSCIENCE
www.SimcoeGeoscience.com

GEOPHYSICAL REPORT

GROUNDED ENGINEERING INC.

GEOPHYSICAL TESTING FOR SEISMIC SITE CLASSIFICATION

2233 Hurontario Street, Mississauga, ON

February 23, 2026 **Project # SGL-#26522**

GENERAL OVERVIEW

Simcoe Geoscience Limited (herein referred to as Simcoe) was requested by Grounded Engineering Inc. to conduct a seismic site class testing at 2233 Hurontario Street in Mississauga, Ontario. A MASW/MAM and an HVSR tests were executed to obtain shear wave velocity values from surface. The location is shown in Figure 1.

The survey was carried out on February 12th, 2026. The survey was conducted in the morning along Hurontarion Street. The presence of the underground car park at this site limited the available space for the test profile. The cultural nose levels, mainly for road traffic, were substantial. Timing the shots and stacking minimized the overall impact of the noise on the data quality. The topography of the line was relatively flat. The site was covered with snow.

One MASW Sounding was acquired using a single string of 24 geophones with a standard receiver spacing of 3 meters and one HVSR test conducted simultaneously. Seven (7) active shots and twenty (20) passive records were measured and recorded for this sounding, and a 30-minute recording time for the HVSR test. The passive records were collected with a longer sampling rate and longer total recording length. The active records are predominantly intended for modeling the upper 20 meters while the passive records tend to be lower frequency components that are suited for modeling in the 15-to-40-meter range.

Figure 1: MASW/MAM Sounding and HVSR Location

FIELD PROCEDURE

The field setup of a MASW survey involves the layout 24 geophones in a linear array. There may be other array designs depending on the site and aim of the survey. Data acquisition involves generating an acoustic wave with a sledgehammer (commonly there are several other sources possible) and digitally recording the rolling surface waves from the moment of source impact for a full second as it passes the entire array of sensors (geophones).

For this study, data was collected with ABEM Terraloc Pro 2™ seismograph - 24 channels and 4.5 Hz geophones. A sledgehammer was used as the primary energy source with traces being recorded at 7 locations: one shot at centre, two shots between the first and last geophones and two shots at 6m and 25m offset beyond the ends of the line. The shot at the centre of the spread is used as a refraction reference for the operator and in processing to see the compression waves rather than shear waves. Figure 2 shows typical field setup for 3-meter receiver interval. Note that the shot locations are different in Figure 2 than in the executed survey. It is common to adjust shot locations depending on a few factors such as rock depth, field conditions (like noise) and intended requirements for the survey.

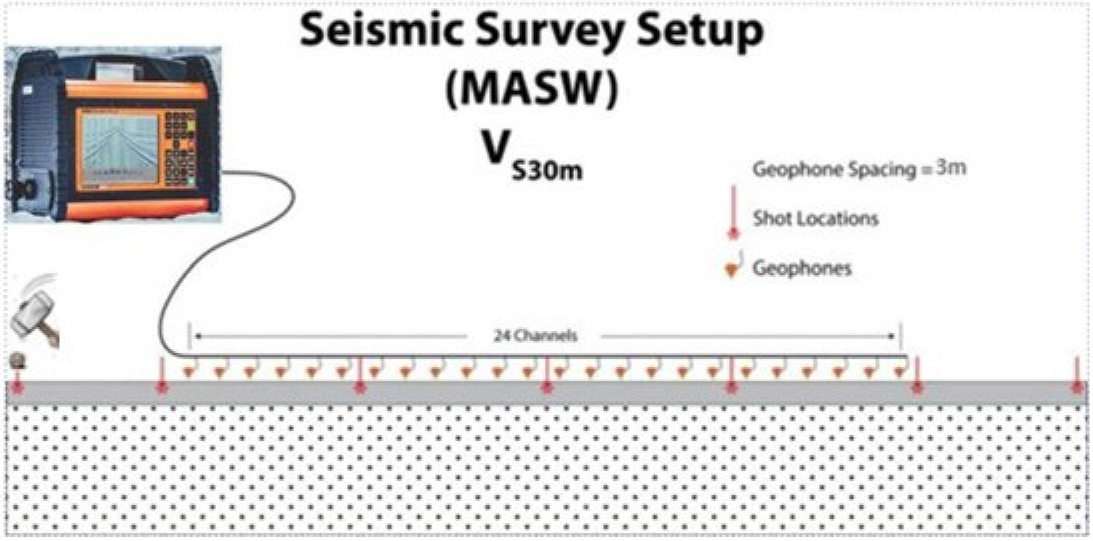


Figure 2 MASW 3-meter Spacing Field Setup, Geophones (orange), Shot Locations (red)

The passive survey (MAM) used the same geophone array set up as for the MASW survey. Unlike the MASW survey, the MAM method is considered a “passive source” method. There is no time break, and the motions recorded are from ambient energy generated by cultural noise such as traffic, wind, wave motion, etc. Data collection for the passive method involved recording approximately 10 minutes of background “noise” for this sounding.

The records generated by the MAM method contain lower frequency data, thus increasing the data modelling to greater depths. Typically, the MAM results help clarify the MASW results for depths greater than 20 m; however, the direction of noise propagation relative to the spread orientation can influence the results. In this survey, the cultural noise direction was parallel to the entire spread which is excellent for data collection.

An additional test was conducted near the centre of the MASW test called HVSR (Horizontal to Vertical Spectral Ratio). This uses a device called a Tromino™ seismograph with three component (three orientations) sensors. The principal objective of the HVSR method is to obtain the fundamental natural frequency of a site, which is often related to the depth of the bedrock or a significant velocity contrast layer. This method collects passive data for 30 minutes at an individual location.

DATA PROCESSING AND INTERPRETATION

The MASW data was processed and interpreted using SeisImager™ Surface Wave Analysis to generate a 1-D (depth) shear-wave velocity (Vs) profile. The active and passive data were post-processed, and individual dispersion curves were generated and stacked to generate one average dispersion image of the highest signal-to-noise (S/N) ratio. Two separate dispersion images were generated, i.e., active, and passive records.

The passive image was prepared by stacking all individual dispersion images processed from twenty (20) passive field records (30 seconds each). This indicates surface wave energy accumulation at relatively lower frequencies (e.g., ≤ 10 Hz) where the active image significantly lacks any meaningful energy trend.

Finally, both active and passive dispersion images were combined to generate one combined dispersion curve that has the highest resolution and the broadest bandwidth. In the overall dispersion trend, extracting the fundamental-mode dispersion curve (M0), which then indicates that the modal interpretation of M0 is more confident in the combined image. This also has the final 1-D velocity (Vs) profile which will have an increased confidence level at deeper depths (e.g., ≥ 20 m) because of the lower frequencies (e.g., ≤ 10 Hz) thus can then be extracted for the M0 curve. The M0 curve (Figure 3) was then used to generate the final 1-D shear-wave velocity (Vs) profile through the subsequent inversion process. The smoothing of the curve helped to minimize the noise of the data, which could produce extra layers in the 1D results.

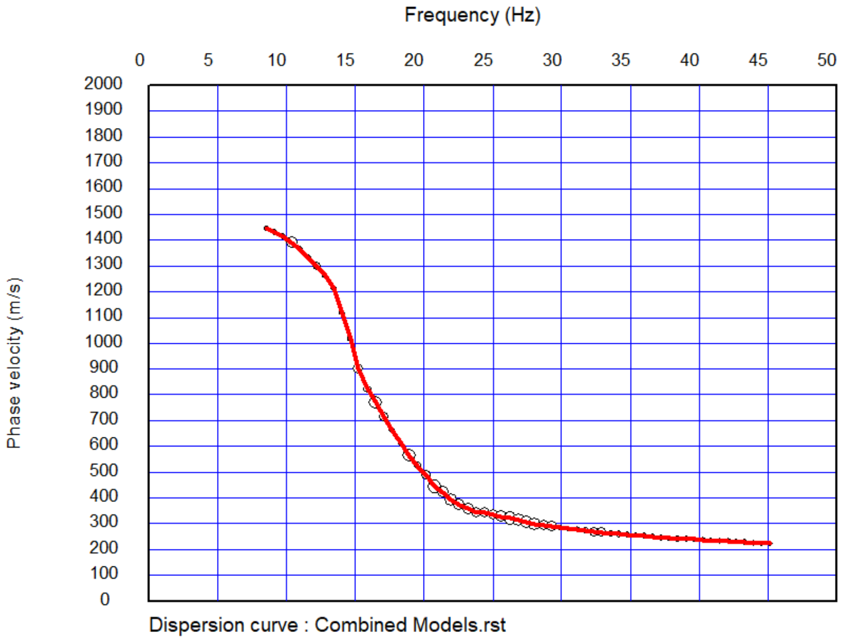


Figure 3: Combined Active & Passive Dispersion Curve

RESULTS

The Rayleigh dispersion curves obtained from the active and passive datasets were of good quality. There is a process called “stacking” which combines records after hitting the same location several times. This helps to maximize the signal to noise ratio.

The inverted shear wave velocity model (see Figure 5) indicates that the shale bedrock surface is encountered at approximately 11 m depth. Independent seismic refraction and HVSR analyses suggest a comparable bedrock depth in the range of 10-12 m. Available borehole records corroborate these findings, confirming bedrock at approximately 11 m depth.

The top 5 m has shear-wave velocities are characteristic of loose fill to compact silty sand. From 5 to 11 m, the velocity increase is consistent with compact to hard sand.

Inverted MASW models were used to calculate Vs30 with no constraint to the bedrock values.

Simple refraction analysis indicates that the shale bedrock exhibits a P-wave velocity (Vp) in the range of 3200-3300 m/s. Using representative Vp/Vs relationships for competent shale, this corresponds to estimated shear wave velocities (Vs) on the order of 1400-1500 m/s, which are considered more realistic values for competent shale bedrock.

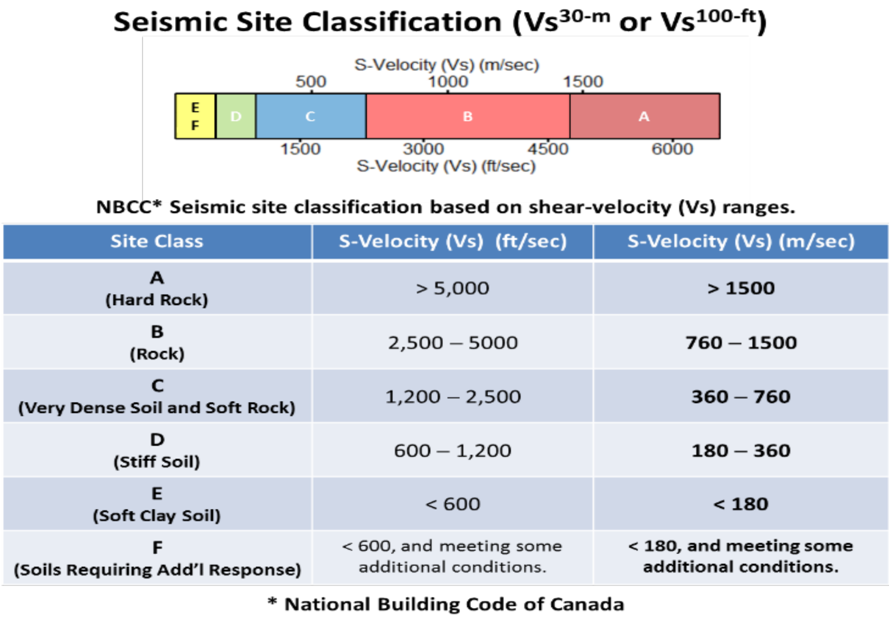
If the MASW models were constrained using the refraction-derived velocities, the calculated Vs_{30} would increase modestly, by approximately 40 m/s.

Option #1: Slab-on-Grade Design, 106 m asl

Profile	Depth (m)	Vs30 (m/s)	Seismic Site Class
MASW/MAM	0 to 30	542	C

This site can be classified as **Seismic Site Class C (“Very Dense Soil and Soft Rock”)** according to the seismic site classification provisions of the following codes, National Building Code of Canada 2020, Ontario Building Code 2024 and the International Building Code (IBC), see Figure 4.

If we apply the NBCC 2020 and the OBC 2024 codes, there is Table 4.1.8.4.A *Exceptions for Site Designation Using Vs30 Calculated from in Situ Measurements*. X_v = 542.



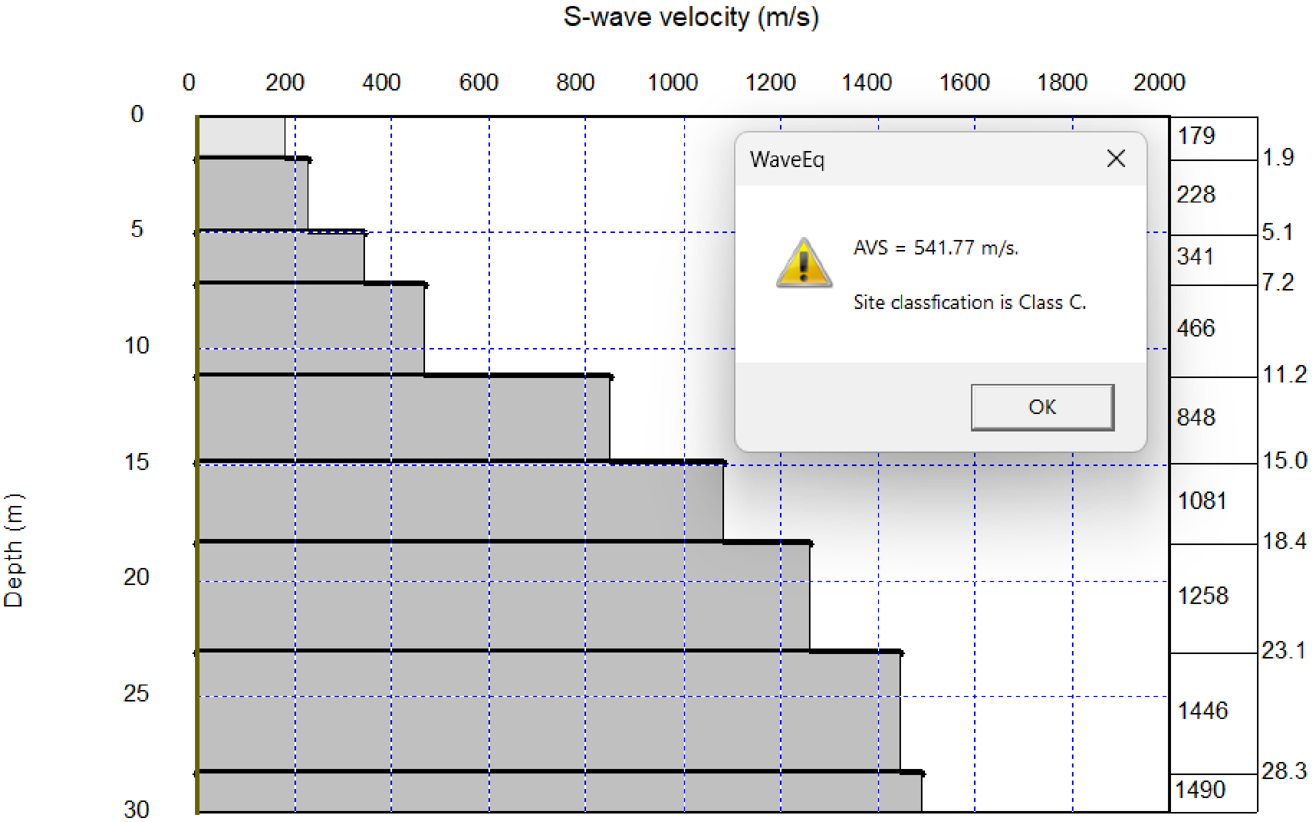
Seismic Site Classification ($Vs^{30\text{-}m}$ or $Vs^{100\text{-}ft}$)

NBCC* Seismic site classification based on shear-velocity (Vs) ranges.

Site Class	S-Velocity (Vs) (ft/sec)	S-Velocity (Vs) (m/sec)
A **(Hard Rock)**	> 5,000	**> 1500**
B **(Rock)**	2,500 – 5000	**760 – 1500**
C **(Very Dense Soil and Soft Rock)**	1,200 – 2,500	**360 – 760**
D **(Stiff Soil)**	600 – 1,200	**180 – 360**
E **(Soft Clay Soil)**	< 600	**< 180**
F **(Soils Requiring Add'l Response)**	< 600, and meeting some additional conditions.	**< 180, and meeting some additional conditions.**

*** National Building Code of Canada**

Figure 4: Summary of Site Class based upon Shear-wave velocities



WaveEq

AVS = 541.77 m/s.

Site classfication is Class C.

S-wave velocity model (inverted) : Combined Models.rst

Average Vs to 30m = 541.8 m/sec

Figure 5: MASW/MAM Sounding results

Figure 5 is the model from the MASW/MAM soundings. The strength of the MASW is its ability to measure soil strength. To find bedrock depth an additional test is performed.

Bedrock Depth Measurement

A useful rule-of-thumb for detecting the bedrock surface from the MASW/MAM results is that when shear wave velocity reaches 650 m/s. This is not the true shear-wave value for bedrock as the method underestimates rock velocities. However, it can be a very strong indicator of bedrock if the velocity continues to accelerate toward that threshold. In this case the average shear wave velocity of overburden was 365 m/s using the MASW/MAM methods. MASW/MAM is very good at obtaining the shear-wave model of the overburden, but it only provides an indication of the top of bedrock. Its weakness is measuring the actual velocity of the rock. It is best to have another method to confirm the top of rock with either a borehole or another geophysical method which would allow the modeler to insert a value for rock that is more realistic for the rock type. The confirmation geophysical test is HVSR.

One Horizontal to Vertical Spectral Ratio tests (HVSR) was employed to obtain the top of bedrock by measuring the natural frequency of any location. When on open soil the natural frequency is often governed by the top of bedrock. The device is the Tromino Blu™ which has a three-component sensor; horizontal, vertical and transverse. By using low frequency cultural noise, which was present, the passive HVSR tests (similar in practice to the MAM test) generated a natural frequency of 8.97 Hz, see frequency plot in Figure 6. We can employ a simple formula to calculate the approximate rock depth.

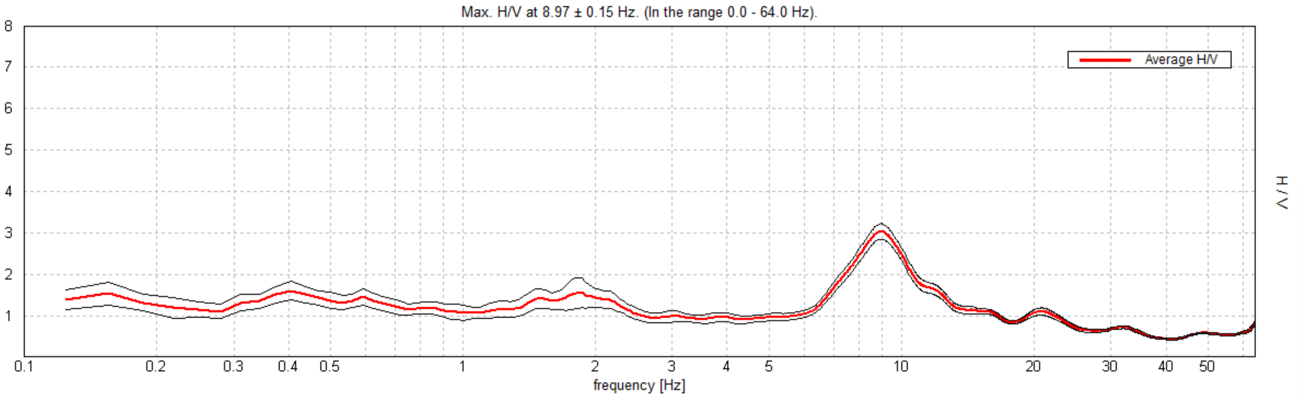


Figure 6: HVSR Sounding Results

h = Vs/4*f,* where h is depth, *f* is natural frequency (8.97 Hz) and Vs is 365 m/s, average of shear wave velocities near the surface (from the MASW/MAM testing). Vs is approximate because we are using the average Vs of overlaying layers in this case. Note, the Vs number used in these calculations is different from the Vs used in the calculation of the Vs30 site class, as this calculation is consistent only of overburden soils.

The approximate depth of the bedrock is 10-11 meters.

CONCLUSIONS

The data quality was good and the dispersion curves were well defined.

The Vs30 with from surface to 30 meters is 542 m/s. MASW models were not constrained to optimize the bedrock shear-wave values.

The seismic site class is “C”. At this site, the bedrock depth is approximately 11 m below grade.

It is important to note that data analysis and seismic site classification described in this report is based on seismic methods only. The results of MASW sounding can be superseded by other geotechnical information such as the presence of sensitive and/or liquefiable soils, more than 3 meters of soft clays, high moisture content, etc. It is important to consider other geotechnical information prior to further investigations on site. For more details about seismic site classification, the reader is referred to section 4.1.8.4 of the National Building Code of Canada, 2020 Edition.

We are committed to providing the next-generation ground and marine geophysical technologies and expertise to apply to all forms of engineering applications.

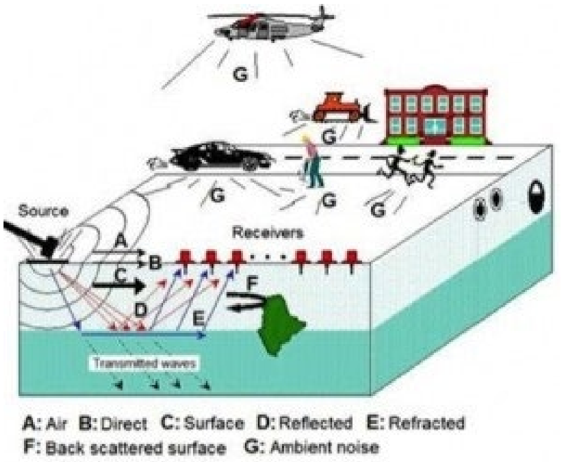
Respectfully submitted,

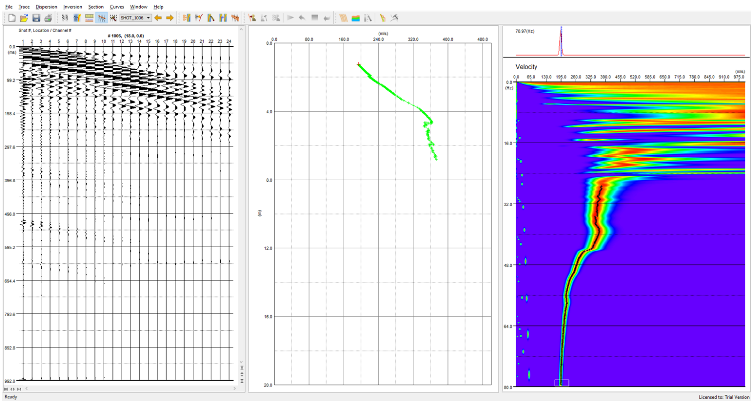
Lhoucin Taghya, P. Geo.
Senior Geophysicist
Simcoe Geoscience Limited

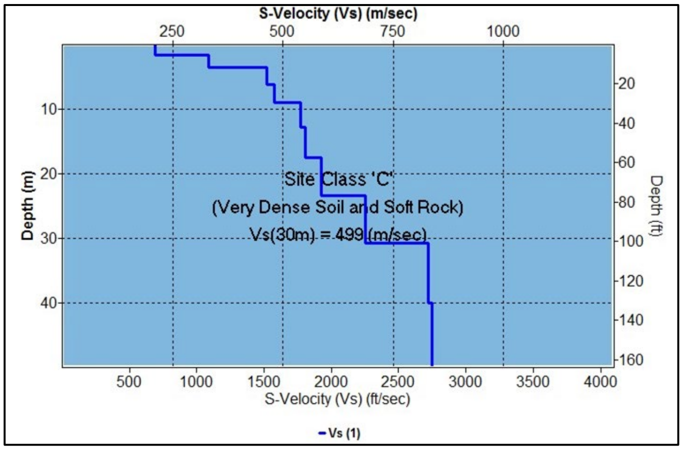
PROFESSIONAL GEOSCIENTIST
LHOUCIN TAGHYA (LT)
PRACTISING MEMBER
2872
ONTARIO

MASW METHOD

First introduced in GEOPHYSICS (1999), the Multichannel Analysis of Surface Waves (MASW) method is one of the seismic survey methods evaluating the elastic condition (stiffness) of the ground for geotechnical engineering purposes. MASW first measures seismic surface waves generated from various types of seismic sources—such as sledgehammer—analyzes the propagation velocities of those surface waves, and then finally deduces shear-wave velocity (Vs) variations below the surveyed area that is most responsible for the analyzed propagation velocity pattern of surface waves. Shear-wave velocity (Vs) is one of the elastic constants and closely related to young's modulus. Under most circumstances, Vs is a direct indicator of ground strength (stiffness) and therefore commonly used to derive load-bearing capacity. After a series of processing and modeling procedures, final Vs information is provided in 1D, 2D and 3D formats. Figures below outline the basic operating procedure for the MASW method and an example image of a typical MASW record and resulting 1D Vs model. The shear-wave depth profile is the average of the bulk area within the middle third of the geophone spread. The nominal maximum depth of penetration is half of the maximum seismic array length, which in practice is often influenced by the geology. A more detailed description of the method can be found in the paper *Multi-channel Analysis of Surface Waves,* Park, C.B., Miller, R.D. and Xia, J. Geophysics, Vol. 64, No. 3 (May-June 1999); P. 800–808.

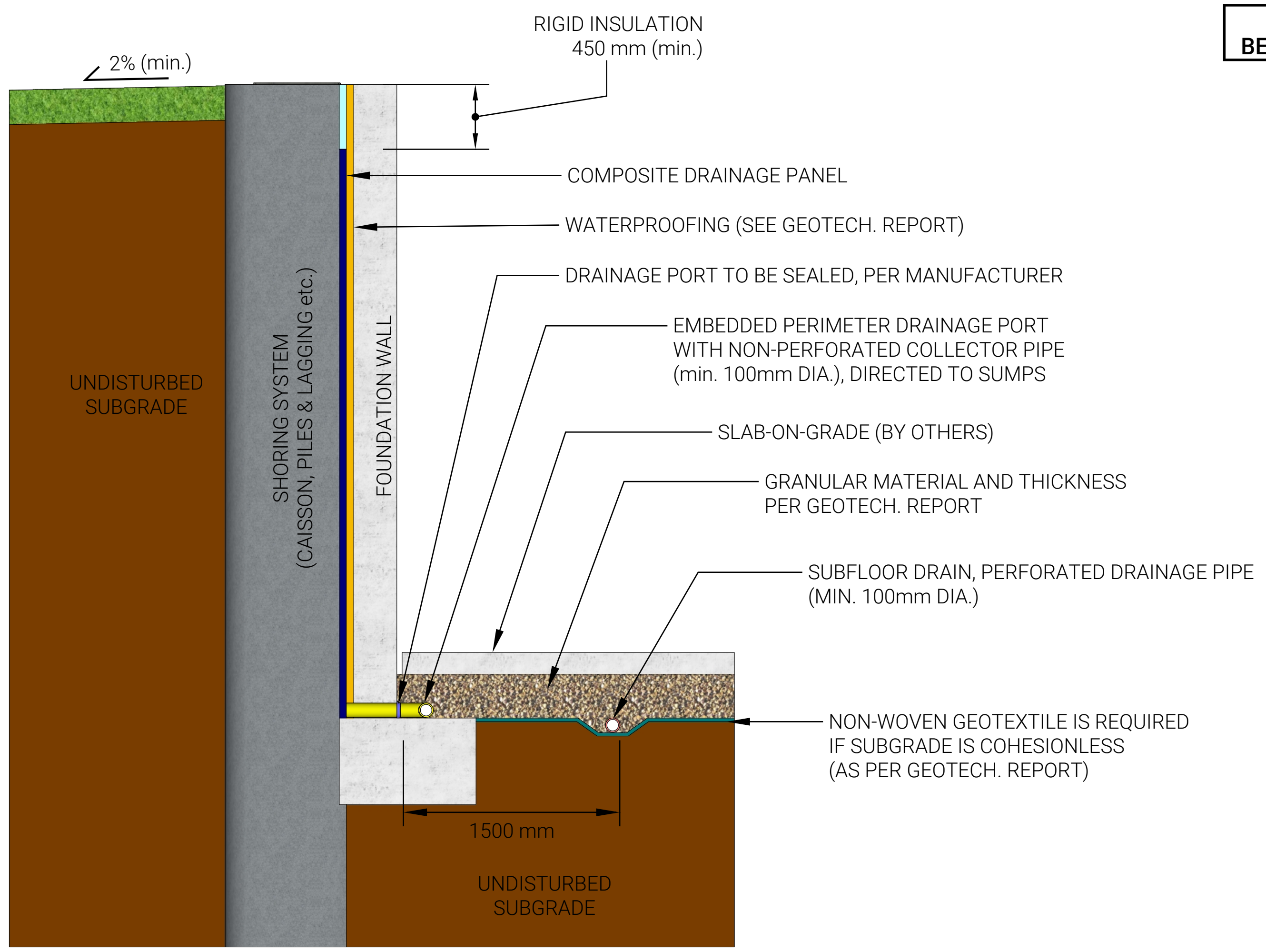




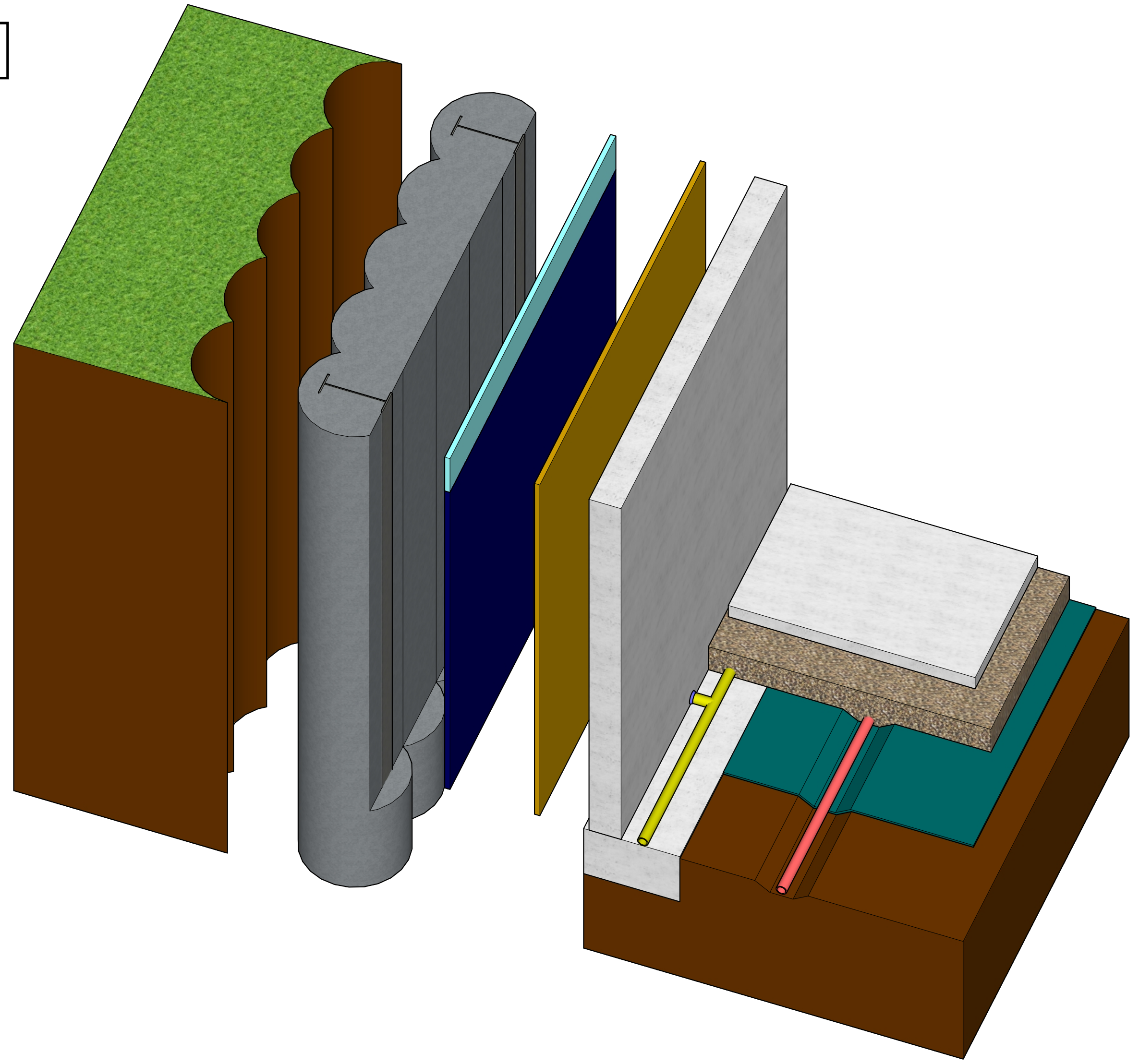


APPENDIX F

OBJECTS ARE COLOR-CODED BETWEEN TWO VIEWS FOR CLARITY



SECTIONAL VIEW



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. SEE THE GEOTECHNICAL REPORT FOR GEOTEXTILE REQUIREMENTS.

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

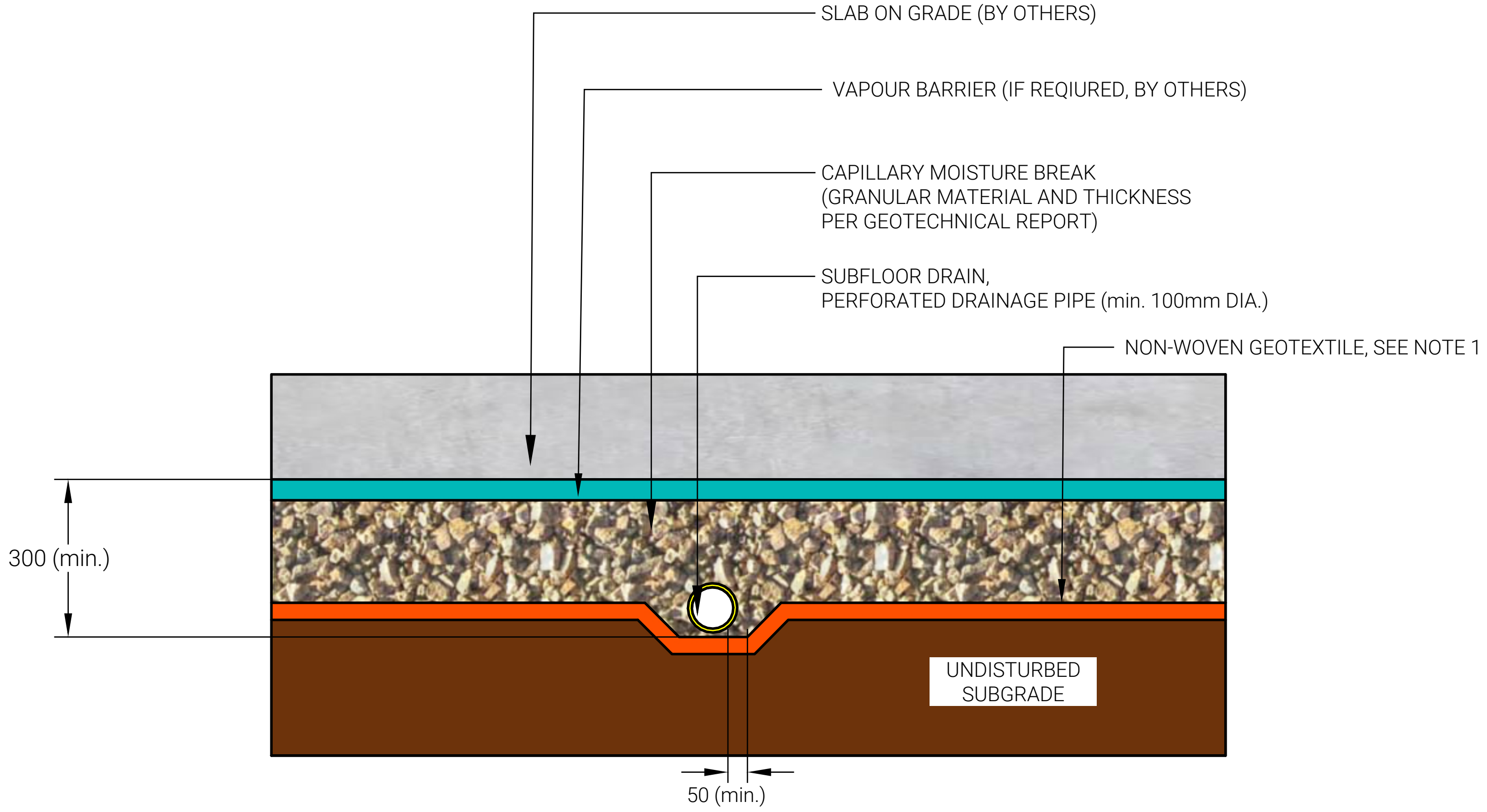
1. THERE SHOULD BE NO STRUCTURAL CONNECTION BETWEEN THE SLAB-ON-GRADE AND THE FOUNDATION WALL OR FOOTING.
2. THERE SHOULD BE NO CONNECTION BETWEEN THE SUBFLOOR AND PERIMETER DRAINAGE SYSTEMS.
3. THIS IS ONLY A TYPICAL BASEMENT DRAINAGE DETAIL. THE GEOTECHNICAL REPORT SHOULD BE CONSULTED FOR SITE SPECIFIC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

GROUNDED
ENGINEERING

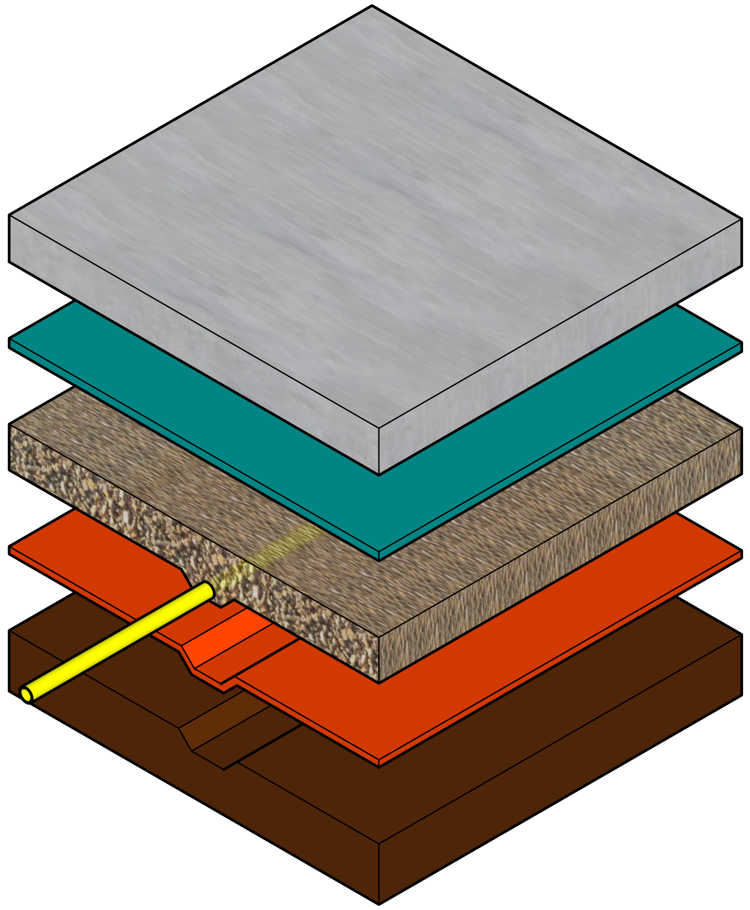
Title

BASEMENT DRAINAGE SHORING SYSTEM TYPICAL DETAILS

OBJECTS ARE COLOR-CODED
BETWEEN TWO VIEWS FOR CLARITY



SECTIONAL VIEW



ISOMETRIC VIEW

NOTES

1. WHEN THE SUBGRADE CONSISTS OF COHESIONLESS SOIL, IT MUST BE SEPARATED FROM THE SUBFLOOR DRAINAGE LAYER USING A NON-WOVEN GEOTEXTILE (WITH AN APPARENT OPENING SIZE OF < 0.250mm AND A TEAR RESISTANCE OF > 200 N).

2. TYPICAL SCHEMATIC ONLY. MUST BE READ IN CONJUNCTION WITH GEOTECHNICAL REPORT.

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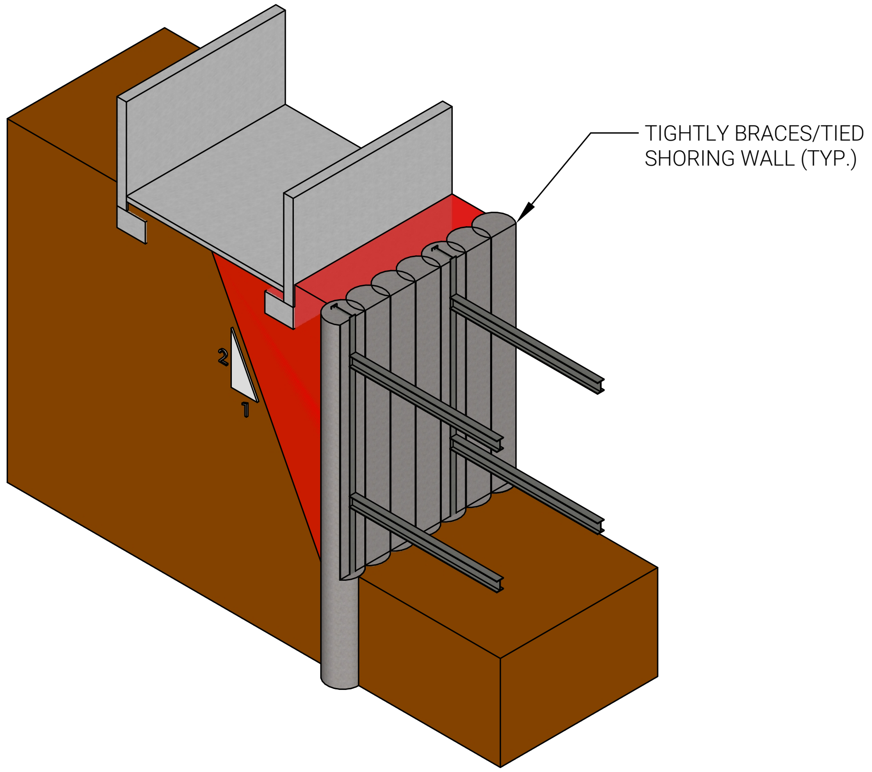
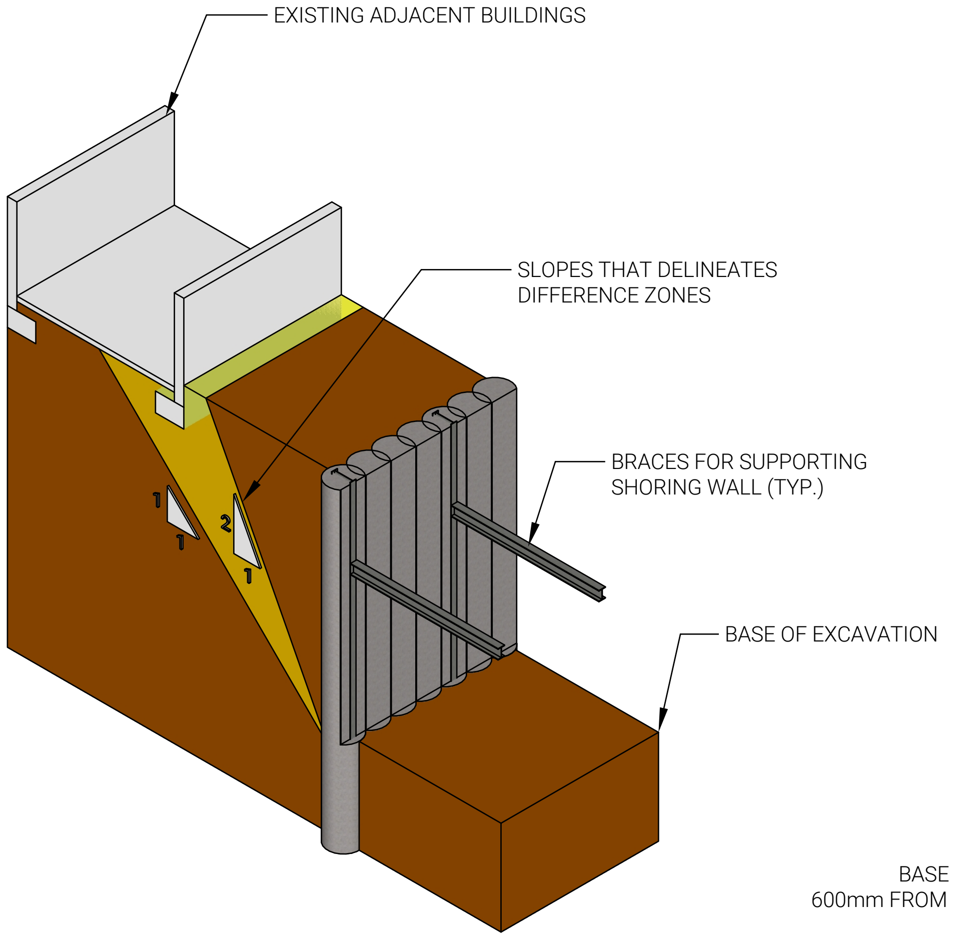
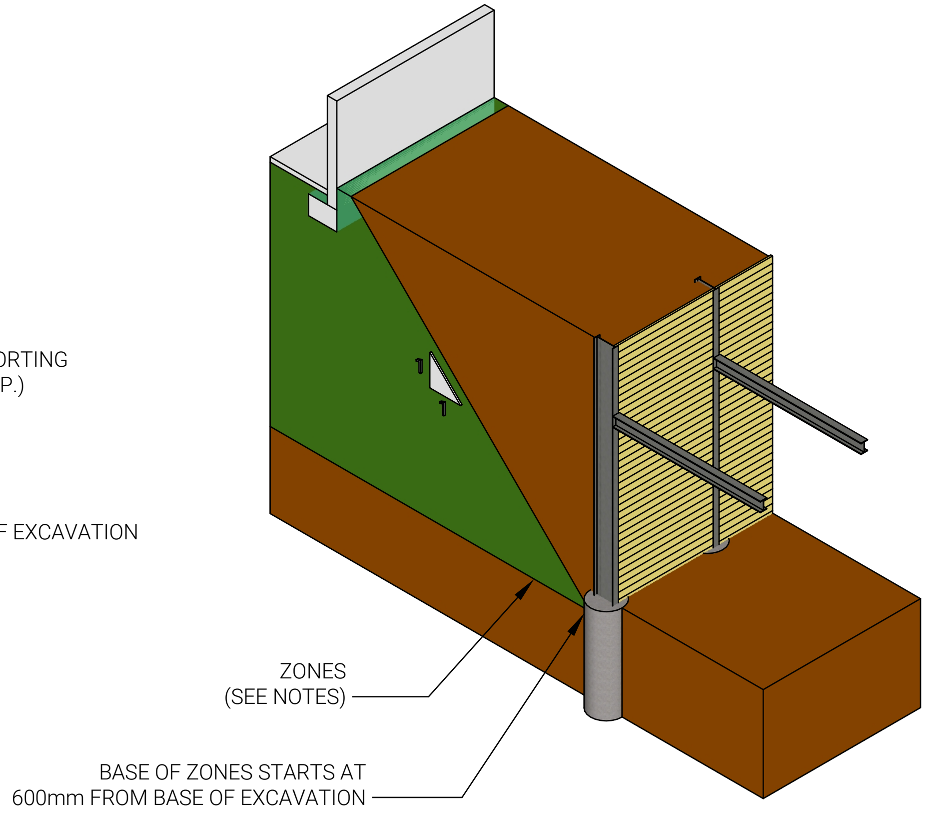
BASEMENT SUBDRAIN TYPICAL DETAIL







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.

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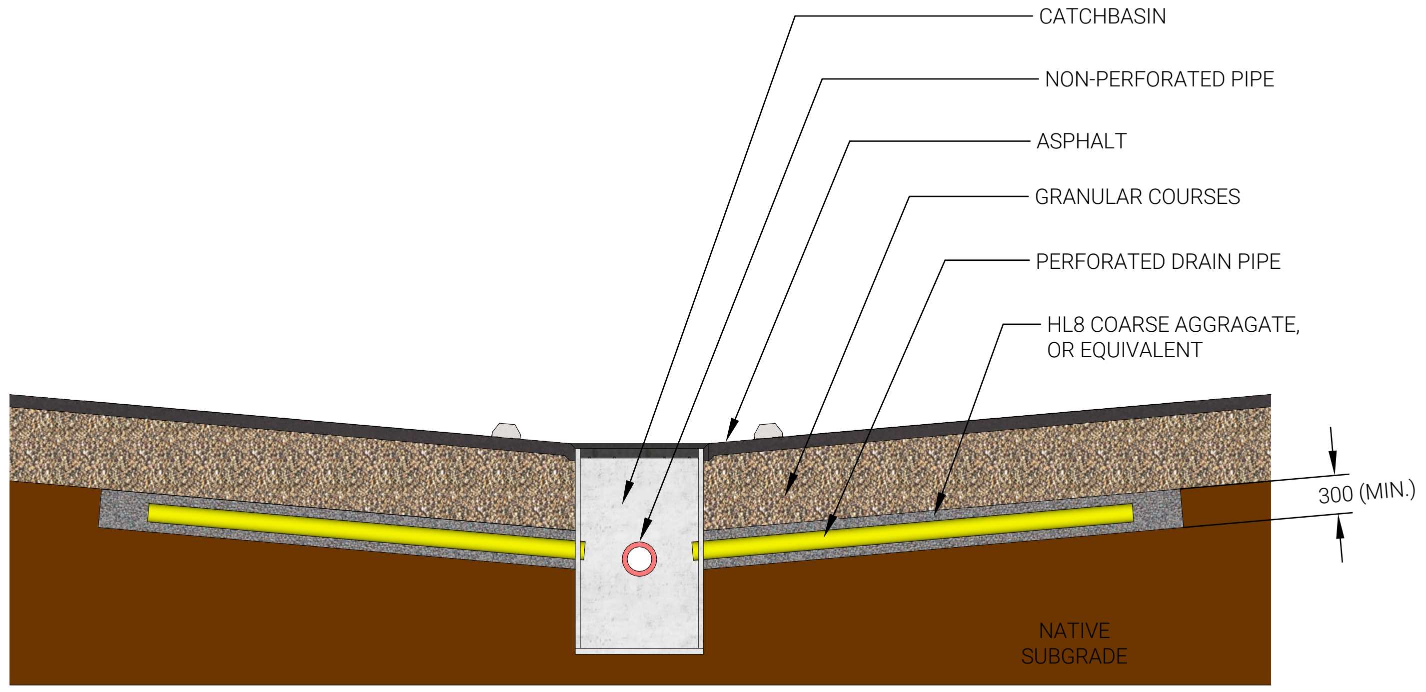
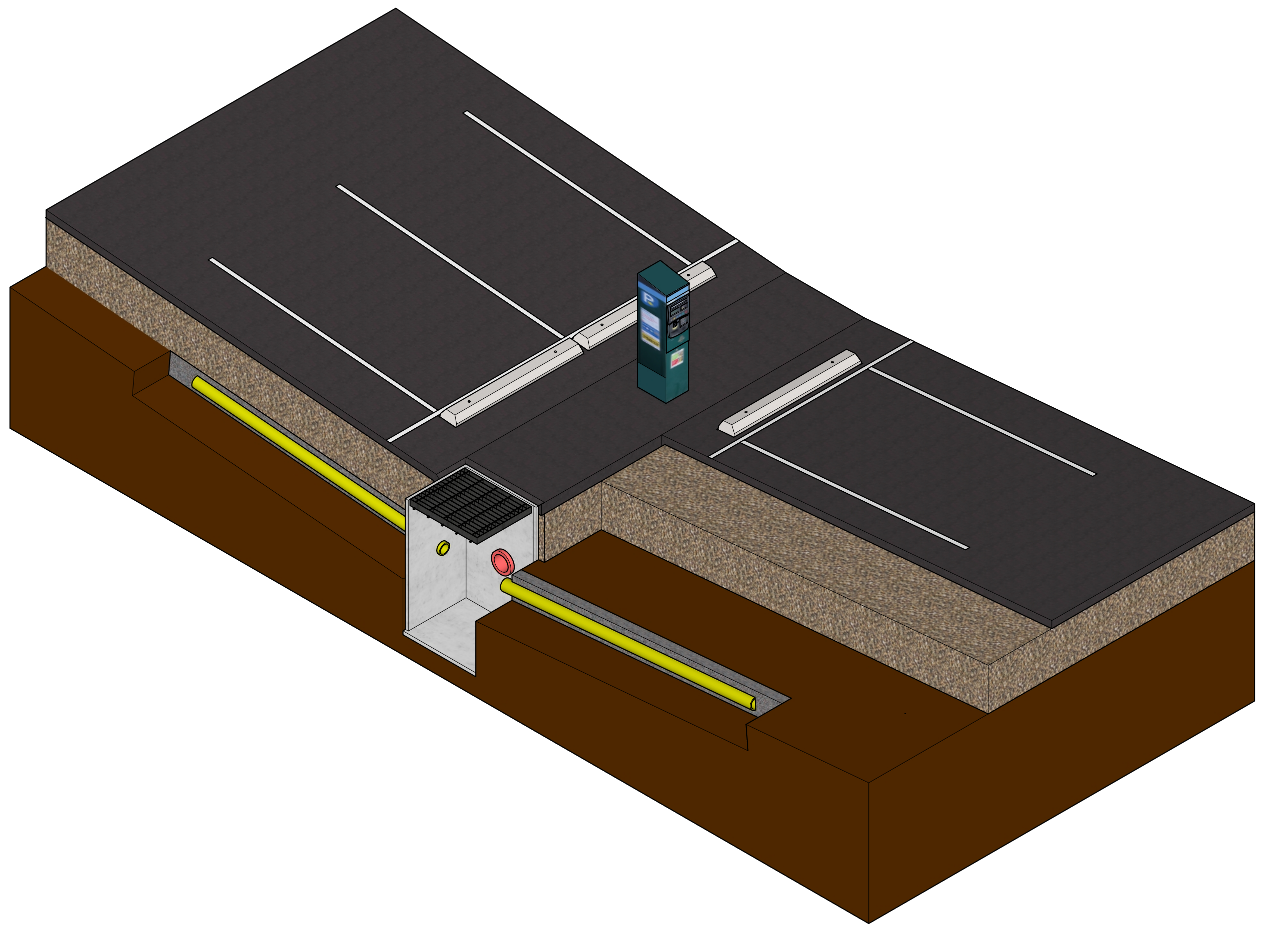
EXCAVATION ZONE OF INFLUENCE GUIDELINES

OBJECTS ARE COLOR-CODED
BETWEEN TWO VIEWS FOR CLARITY



SECTIONAL VIEW



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LONGITUDINAL SUBDRAIN CONNECTION TO CATCHBASIN