Proposed 26-Storey Building
12 & 26 Park Street East

Mississauga, Ontario

Canahahns Company Limited
Geotechnical Investigation Report

July 30, 2025
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Geotechnical Investigation Report

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1 Introduction

Englobe Corp. (Englobe) was retained by Canahahns Company Limited to conduct geotechnical investigations for a proposed 26 - storey building to be constructed at the project site located at 12 & 26 Park Street East, Mississauga, Ontario. The general location of the site is presented on Figure 1.

This report encompasses the results of the geotechnical investigations conducted for the proposed development site to determine the prevailing subsurface soil and groundwater conditions, and on this basis, provides geotechnical engineering design advice and recommendations for the building foundations, earthquake and earth pressure design parameters, basement floor and drainage and pavements. In addition, comments are also included on pertinent construction aspects including excavation, backfill and groundwater control.

Englobe has also conducted the hydrogeological and environmental studies for this site. The findings of the studies are reported under the separate covers.

2 Site and Project Description

The project site is in the northwest quadrant of the Elizabeth Street North and Park Street East, municipally described as 12 & 26 Park Street East, Mississauga. The original development was only limited within the property at 26 Park Street East, Mississauga. Englobe has completed the original geotechnical investigation for this property. Subsequent to the submission of the geotechnical report, the client has acquired the neighbouring property located at 12 Park Street East. The proposed development plan is therefore significantly updated to include the recently acquired property. As the previous investigation only covered the property at 26 Park Street East, a supplementary investigation is therefore conducted to reflect the updated design plan.

It is understood that the existing five and seven-storey buildings will remain, and the north portion of the site will be redeveloped to include a 26-storey residential building, including a three-storey above-grade parkade resting on one-level underground parking garage (P1) with a partial second underground parking garage (P2).

The following drawing set was provided for our review in preparation of this report,

- *A2 00, Level P1-P2 Floor Plan, 12, 26 Park Street East*, *Mississauga, Ontario,* Project No.: 21121, dated June 21, 2026, prepared by Kohn Partnership Architects Inc.
- *Site Grading Plan, 12 & 26 Park Street East, Mississauga, Ontario*, Project No.: 26-1247, dated June, 2026, prepared by Envision.

The above-noted design drawings indicate that the lowest finished floor elevation (FFE) for the P1 and P2 underground parking garage will be set at Elev. 73.2 m and Elev. 71.9 m, respectively.

3 Investigation Procedure

The field investigations were conducted on August 25, 26, 2025 and May 25, 26, 2026 and consisted of drilling and sampling a total of eight (8) geotechnical boreholes across the project site, extending to 4.6 to 10.9 m depth below grade with the monitoring wells installed in Boreholes 1 to 4, 6 and 7. In addition, a nested shallow well was installed at the Borehole 3 location. The approximate locations of the boreholes are shown on the enclosed Figures 2A and 2B - Borehole Location Plan.

The borings were advanced using continuous flight solid stem augers and HQ rock coring. The soil was sampled at 0.75 m (up to 3.0 m depth) and 1.5 m (below 3.0 m depth) intervals with a conventional 50 mm diameter split barrel sampler when the Standard Penetration Test (SPT) was carried out (ASTM D1586). Bedrock coring (using HQ bit size) was carried out in Boreholes 2, 3 and 6 to confirm and characterize bedrock. The field work (drilling, coring, sampling and testing) was observed and recorded by a member of our field engineering staff, who logged the borings and examined the samples as they were obtained.

All soil samples obtained during the investigation were sealed into clean plastic bags, and transported to our geotechnical testing laboratory for detailed inspection and testing. All borehole samples were examined (tactile) in detail by a geotechnical engineer, and classified according to visual and index properties. Laboratory tests consisted of water content determination on all samples; and a Sieve and Hydrometer analysis and Atterberg test on selected native soil samples. The measured natural water contents of individual samples, the results of the Sieve and Hydrometer analysis and Atterberg test are plotted on the enclosed Borehole Logs at respective sampling depths. The results of Sieve and Hydrometer analysis and Atterberg tests are also summarized in Section 4.2 of this report and appended.

The bedrock core samples were stored in the wood boxes and transported to our laboratory for the further visual inspection and laboratory test. The recovered rock cores were visually inspected and photographed by a geotechnical engineer. Photographs of the rock core are provided in Appendix C.

Water levels were measured in open boreholes upon completion of drilling. Monitoring wells comprising 50 mm diameter PVC pipes were installed in Boreholes 1 to 4, 6 and 7 with a nest well in Borehole 3 to facilitate groundwater monitoring and for the purpose of the Hydrogeological Study. The PVC tubing was fitted with a bentonite clay seal as shown on the accompanying Borehole Logs. Water levels in the monitoring wells were measured on September 18, 2025 and June 16, 2026. The results of groundwater monitoring are presented in Section 4.3 of this report.

Borehole surface elevations were surveyed by Englobe. Borehole elevations are provided relative to Geodetic Datum (NAD). The horizontal coordinates are reported relative to the Universal Transverse Mercator geographic coordinate system (UTM Zone 17T). The borehole elevations noted on the Borehole Logs are provided for the purpose of relating borehole soil stratigraphy and should not be used or relied on for other purposes.

4 Subsurface Conditions

The specific soil conditions encountered at each borehole location are described in greater detail on the Borehole Logs, with a summary of the general subsurface soil conditions outlined below. This summary is intended to correlate this data to assist in the interpretation of the subsurface conditions encountered at the site.

It should be noted that the subsurface conditions are confirmed at the borehole locations only, and may vary between and beyond the borehole locations. The boundaries between the various strata as shown in the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of geologic change.

4.1 Stratigraphy

The following stratigraphy is based on the borehole findings, as well as the geotechnical laboratory testing conducted on selected representative soil samples. The summary provided below is for general guidance only. Detailed depths and elevations are given in the following subsections and appended borehole logs. In general, four (4) main stratigraphic units are as follows:

- Asphalt pavement structure underlain by an earth fill zone, extending to Elev. 75.9 to 78.6 m (0.2 to 2.3 m depth below grade), overlying;
- Sandy silt to silt, extending to Elev. 76.5 to 76.9 m (1.5 to 2.3 m depth below grade), overlying
- Silty clay till, extending to Elev. 73.6 to 74.2 m (4.6 to 5.0 m depth below grade), overlying; and
- Shale bedrock of Georgian Bay Formation.
- The stabilized groundwater levels ranged from Elev. 72.7 to 77.4 m (1.0 to 5.9 m depth below grade).

4.1.1 Asphalt Pavement

Asphalt pavement structure was encountered at ground surface in all boreholes and consisted of 80 to 100 mm thick asphaltic concrete underlain by 100 to 520 mm thick aggregate layer.

4.1.2 Earth Fill

Earth fill materials, consisting of silty sand to sandy silt with trace amounts of gravel, clay, rootlets and organic were encountered beneath the asphalt pavement structure in Boreholes 1, 3, 4 and 7, and extended depths varying from 0.8 to 1.8 m below grade depth below grade.

Standard Penetration Test results (N-values) obtained from cohesionless earth fill zone ranged from 5 to 17 blows per 300 mm of penetration, indicating a loose to compact relative density. The in-situ moisture contents of the cohesionless earth fill samples ranged from 14 to 31 percent by mass, indicating a moist condition.

Earth fill materials, consisting of clayey silt to silty clay with varying amounts of sand (trace sand to sandy) and trace amounts of gravel and trace rootlets and organics were encountered beneath the sandy silt fill zone in Borehole 3, and beneath the aggregate layer in Boreholes 5, 6 and 8 extended to about 2.3 m depth below grade.

Standard Penetration Test result (N-values) obtained from the cohesive earth fill zone ranged from 4 to 23 blows per 300 mm of penetration, indicating a soft to very stiff consistency. The in-situ moisture content of the cohesive earth fill samples ranged from 12 to 33 percent by mass, indicating a moist condition.

4.1.3 Sandy Silt to Silt

The native sandy silt to silt deposit with trace to some clay was encountered beneath earth fill zone or pavement structure in Boreholes 2 and 4 and extended to 1.5 m and 2.3 m depth below grade. Standard Penetration Test results (N-values) obtained from the sandy silt to silt deposit ranged from 18 to 33 blows per 300 mm, indicating a compact to dense relative density. The in-situ moisture contents of the sandy silt to silt samples ranged from 6 to 16 percent by mass, indicating a moist condition.

4.1.4 Glacial Till

Cohesive glacial till includes silty clay, with varying amounts of sand (some sand to sandy) and trace to some gravel was encountered beneath the fill zone or the native sandy silt to silt deposit in all boreholes extended to 4.6 to 5.0 m depth below grade.

Standard Penetration Test results (N-values) obtained from the silty clay till are ranged from 7 to 48 blows per 300 mm, indicating a firm to hard consistency. The typical in-situ moisture contents of the silty clay till samples ranged from 8 to 17 percent by mass, indicating a moist condition.

It must be noted that glacial till deposit may contain larger size particles (cobbles and boulders) that are not specifically identified in the boreholes. The size and distribution of such obstructions cannot be predicted with borings, because the borehole sampler size is insufficient to secure representative samples of the particles of this size.

4.1.5 Bedrock

The partially weathered shale (inferred Bedrock of Georgian Bay Formation) was identified in all boreholes at depths varying from 4.6 to 5.0 m below grade. Rock coring was carried out in Boreholes 2, 3 and 6 and extending from 9.6 to 10.9 m below grade. Borehole Logs and Rock Core Logs are provided in Appendix A. Photographs of the recovered rock core samples are provided in Appendix C. Based on the results of the visual inspection of the recovered rock cores and field observations, the bedrock depth and elevations are tabulated as follows:

Borehole	Ground Surface or Top of Ties Elevation	Bedrock Depth below Grade	Top of Bedrock Elevation	Method of Observation
BH-1	Elev. 78.6 m	4.7 m	Elev. 73.9 m	Augering
BH-2	Elev. 78.8 m	5.0 m	Elev. 73.8 m	Rock Coring
BH-3	Elev. 78.2 m	4.6 m	Elev. 73.6 m	Rock Coring
BH-4	Elev. 78.4 m	4.6 m	Elev. 73.8 m	Augering
BH-5	Elev. 78.3 m	4.6 m	Elev. 73.7 m	Augering
BH-6	Elev. 78.3 m	4.6 m	Elev. 73.7 m	Rock Coring
BH-7	Elev. 78.4 m	4.6 m	Elev. 73.8 m	Augering
BH-8	Elev. 78.8 m	4.6 m	Elev. 74.2 m	Augering

The bedrock beneath the site consists of the Georgian Bay Formation, which is a deposit predominantly comprising thin to medium bedded grey shale of Ordovician age. The shale contains interbedded grey calcareous shale, limestone/dolostone and calcareous sandstone (conventionally grouped together as "limestone") which are discontinuous and nominally 25 to 175 mm thick. The strength of the bedrock ranged from weak to medium strong based on the field estimate method.

There is typically a zone of weathering at the contact between the rock of the Georgian Bay Formation and the glacial soil overburden. In the Ontario Ministry of Transportation and Communications document RR229, there is reproduced from Skempton, Davis and Chandler, a typical weathering profile of a low durability shale, that characterizes the shale surface into three grades of weathering and four zones described as follows:

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

Rock Quality Designation (RQD) refers to the total length of those pieces of sound core which are 100 mm or greater in length in a core run, expressed as a percentage of the total length of that core run. Sound pieces of rock are those pieces separated by natural fractures or bedding, and not machine induced or subsequent artificial breaks. The RQD of the recovered rock cores varied between 28 percent and 90 percent. The details of RQD are summarized in the following table,

Borehole	Rock Coring Run No.	RQD	Rock Mass Quality	Depth/Elevation Top of Sound Bedrock
BH-2	R1	28	Poor	5.9 m depth below grade Elev.72.9 m
	R2	48	Poor	
	R3	78	Good	
	R4	75	Good	
BH-3	R1	44	Poor	5.7 m depth below grade Elev.72.5 m
	R2	62	Fair	
	R3	67	Fair	
	R4	62	Fair	
BH-6	R1	68	Fair	5.4 m depth below grade Elev.72.9 m
	R2	40	Poor	
	R3	90	Good	

4.2 Geotechnical Laboratory Test Results

The geotechnical laboratory testing consisted of natural water content determination for all samples, while a Sieve and Hydrometer analysis and Atterberg tests were conducted on selected soil samples. The test results are plotted on the enclosed Borehole Logs at respective sampling depths. The results (graphs) of the Sieve and Hydrometer (grain size) analysis and Atterberg tests are appended and a summary of these results are presented as follows:

Borehole No.	Sample No.	Sampling Depth Below Grade (m)	Percentage (by mass)			Atterberg Limits			Descriptions (USCS)
			Gravel	Sand	Silt & Clay	Liquid Limit	Plastic Limit	Plasticity Index	
BH-1	SS-5	3.4	7	36	57	-	-	-	SILTY CLAY sandy, trace gravel
BH-2	SS-3	1.8	0	4	96	-	-	-	SILT some clay trace sand
BH-5	SS-5	3.4	6	30	64	30	18	12	SILTY CLAY sandy, trace gravel
BH-8	SS-4	2.6	5	27	68	29	17	12	SILTY CLAY sandy, trace gravel

4.3 Groundwater

Observations pertaining to the depth of caving were made in boreholes immediately after completion of drilling and are noted on the enclosed Borehole Logs. Monitoring wells were installed in Boreholes 1 to 4, 6 and 7 to facilitate groundwater level monitoring and for the purpose of hydrogeological study. The groundwater level measurements in the monitoring wells were taken during the subsequent site visits and are noted on the enclosed Borehole Logs. A summary of these observations is provided as follows:

Borehole No.	Depth of Boring below Grade (m)	Strata Screened (m)	Water Level in Well, Depth/Elev. (m)	
			September 18, 2025	June 16, 2026
BH-1	7.7	Silty Clay Till and Shale Bed Rock (Elev. 72.6 to 75.6)	1.2 / 77.4	2.3 / 76.3
BH-2	10.9	Shale Bed Rock (Elev. 67.9 to 70.9)	5.8 / 73.0	5.9 / 72.9
BH-3D	10.7	Shale Bed Rock (Elev. 69.1 to 72.1)	5.6 / 72.7	2.0 / 76.2
BH-3S	10.7	Silty Clay Till and Shale Bed Rock (Elev. 72.1 to 75.1)	1.0 / 77.2	2.0 / 76.2
BH-4	5.5	Silty Clay Till and Shale Bed Rock (Elev. 72.9 to 75.9)	1.7 / 76.8	2.0 / 76.4
BH-6	9.6	Silty Clay Till and Shale Bed Rock (Elev. 72.3 to 75.2)	-	2.9 / 75.4
BH-7	4.6	Silty Clay Till (Elev. 73.9 to 75.4)	-	2.8 / 75.6

Groundwater levels may fluctuate with time, and seasonally, depending on the amount of precipitation and surface runoff. Additional information pertaining to groundwater at the site is discussed in the hydrogeological report by Englobe under a separate cover (File No. 02211043.006).

5 Discussions and Recommendations

The following discussion and recommendations are based on the factual data obtained from this investigation and are intended for the use of the owner and the design engineer. Contractors bidding or providing services on this project should review the factual data and determine their own conclusions regarding construction methods and scheduling.

This report is provided on the basis of these terms of reference and on the assumption that the design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards and guidelines of practice. The Ontario Building Code may require additional considerations beyond the recommendations provided in this report, and must be followed. 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 Englobe should be retained to review the implications of these changes with respect to the contents of this report.

5.1 Foundation

The subsurface conditions encountered at the borehole locations are summarized below,

- Asphalt pavement structure underlain by an earth fill zone, extending to Elev. 75.9 to 78.6 m (0.2 to 2.3 m depth below grade), overlying;
- Sandy silt to silt, extending to Elev. 76.5 to 76.9 m (1.5 to 2.3 m depth below grade), overlying
- Silty clay till, extending to Elev. 73.6 to 74.2 m (4.6 to 5.0 m depth below grade), overlying; and
- Shale bedrock of Georgian Bay Formation.
- The stabilized groundwater levels ranged from Elev. 72.7 to 77.4 m (1.0 to 5.9 m depth below grade.

The proposed development will include a 26-storey tower resting on a one-level underground parking garage with a partial second storey underground parking garage. The lowest P1 and P2 FFE will be set at Elev. 73.1 m and Elev. 71.9 m, respectively.

5.1.1 Foundation Design Parameters

The top of the partially weathered shale is expected to be encountered at Elev. 73.6 to 74.2 m, while the sound/unweathered bedrock may be encountered at Elev. 72.5 to 72.9 m.

Given the anticipated bedrock profile and proposed excavation depths, it is recommended that the tower foundations be supported on conventional spread footings uniformly founded on sound/unweathered shale bedrock. The foundations may be designed for Serviceability Limit State (SLS) bearing resistance of 6.0 MPa and Ultimate Limit State (ULS) bearing resistance of 10.0 MPa.

Footings designed based on the specified bearing capacity at the serviceability limit states (SLS) are expected to settle less than 25 mm total settlement and less than 19 mm differential settlement.

5.1.2 Foundation Installation

The foundation installations must be reviewed in the field by Englobe. The on-site review of the condition of the foundation subgrade as the foundations are constructed is an integral part of the geotechnical engineering design function and is not to be considered as third-party inspection services. If Englobe is not retained to carry out all of the foundation evaluations during construction, then Englobe accepts no responsibility for the performance of the foundations.

The underside of footing elevations must be designed to provide a minimum of 1.2 m of soil cover or equivalent insulation to the foundation subgrade for frost protection considerations in unheated areas.

Prior to pouring concrete for the footings, the footing subgrade must be cleaned of all deleterious materials such as softened, disturbed or caved materials, as well as any weathered rock or standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided. It should be noted that 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.

Depending on the foundation design, some of the footings will need to extend deeper than the nominal founding elevation to be founded on the bedrock. The over-excavation required for the foundations in these areas may be filled with lean mix concrete (strength to be provided by the structural engineer) up to the normal design foundation level, and the foundations may be supported on this lean mix concrete pad. The lean mix concrete pad must extend a minimum of 300 mm beyond the edge of the foundation in every direction.

Footings stepped from one level to another supported on the bedrock should be designed at a slope not exceeding 1 vertical to 1 horizontal in conjunction with the above bearing pressures. There must be a minimum of 500 mm separation between the edge of any footing and the top of a sloped/vertical rock cut down to another footing.

Foundations placed directly on bedrock should be established on a relatively level rock surface, i.e. generally sloping at an angle of less than approximately 10 degrees from the horizontal. In some instances, foundation bases can be placed on bedrock sloping at angles up to 25 to 30 degrees from the horizontal, provided dowels are incorporated to resist shear. Where rock slopes are at steeper angles, the rock surface is to be levelled to provide a stepped footing base. As an alternative to levelling the bedrock, where the bedrock surface is irregular and jagged, it may be more practical to provide level benching over these areas by pouring lean concrete (minimum 10 MPa) prior to constructing the foundations. This determination is made on site based on site specific bedrock conditions.

5.2 Earth Pressure Design Parameters

The lowest P1 and P2 FFE will be set at Elev. 73.1 m and Elev. 71.9 m, respectively. The top of the partially weathered shale is expected to be encountered at Elev. 73.6 to 74.2 m, while the sound/unweathered bedrock may be encountered at Elev. 72.5 to 72.9 m.

Given the anticipated bedrock profile and the proposed underground parking structure depths, the P1 foundation wall would be subjected to lateral earth pressure and possibly the pressure from the shale swell at the lower portion while the P2 excavation would likely extend into the bedrock and possibly be subjected to pressure from the shale swell.

5.2.1 Earth Pressure

Walls or bracings subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

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

Where:

P = the horizontal pressure (kPa)
K = the earth pressure coefficient
h = the depth below the ground surface (m)
h_w = the depth below the groundwater level (m)
γ = the bulk unit weight of soil (kN/m^3)
γ_w = the bulk unit weight of water (9.8 kN/m^3)
γ' = the submerged unit weight of the exterior soil, (γ_{sat} - γ_w)
q = the complete surcharge loading (kPa)

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall, this equation can be simplified to:

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

This equation assumes that free-draining granular backfill is used and positive drainage is provided to ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure.

Resistance to sliding of retaining structures is developed by friction between the base of the footing and the soil. This friction (**R**) depends on the normal load on the soil contact (N) and the frictional resistance of the soil (**tan ϕ**) expressed as **R = N tan ϕ**. The factored geotechnical resistance at ULS is 0.8 R.

Passive earth pressure resistance is generally not considered as a resisting force against sliding for conventional retaining structure design because a structure must deflect significantly to develop the full passive resistance.

The average values for use in the design of walls subjected to unbalanced earth pressures at this site are tabulated as follow:

Parameter	Definition	Units
ϕ	angle of internal friction	degrees
γ	bulk unit weight of soil	kN/ m^3
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

Stratum/Parameter	γ	Φ	K_a	K_o	K_p
Fill/Disturbed Soil	18.0	28	0.36	0.53	2.77
Sandy Silt/Silt/Silty Clay Till	21.0	32	0.31	0.47	3.25
Georgian Bay Formation Bedrock	25.0	26	N/A	N/A	N/A

The above values of the earth pressure coefficients are for the horizontal backfill grade behind the wall. The earth pressure coefficients for inclined grade will vary based on the inclination of the retained ground surface.

5.2.2 Rock Pressure

As previously noted, the partial P1 and full P2 excavation would likely extend into the bedrock and possibly be subjected to pressure from the shale swell.

The empirical approach for the design of foundation walls below bedrock level has been to use a uniform pressure distribution for the design of the basement walls below the top of bedrock elevation, which is consistent with the maximum earth pressure calculated for the lowest level of soil in the profile. This approach is likely conservative but it recognizes the practical requirement to have a foundation wall of a consistent width through the lower reach of the building.

This approach does not recognize the potential for pressures on the basement wall due to time dependant rock swell that results when locked in horizontal stresses are released. It presupposes that there is sufficient time between the cutting of the rock face and the construction of the building structure to allow the rock to de-stress and swell. Experience suggests that if there is a 120-day period after the rock cut, before the rock is restrained by the structure, that there has been sufficient swell and no significant stresses are imposed on the structural wall. Depending on the building construction sequence some provision for compressible material at the excavation perimeter (particularly for excavations extending deeper into the rock) may be necessary, which should be assessed during the detailed design stage.

For the lower foundation walls and where pits are made for sumps, elevators or other such features are cast directly against the rock face, there must be careful consideration of the potential for rock squeeze effects. To accommodate the rock squeeze effect, a compressible layer can be placed between the rock and the concrete. A 50 mm thick 220 Ethafoam Polyethylene Foam planks are typically used in this application. Foundation walls are typically designed for the strength of the foam at the 50 percent compressive deflection. At 50 percent compressive deflection, 220 Ethafoam plank material will provide a resistance of 18 psi (124 kPa). The 10 percent deflection compressive strength of this material is 7 psi (50 kPa), which will allow for concrete placement.

In the case of sumps, elevators, etc., if the rock is over excavated by at least 600 mm and the pits and sumps are backfilled with 19 mm clear stone (OPSS.MUNI 1004), then there is sufficient give in the backfill to accommodate the rock swell.

5.2.3 Sliding Resistance

Resistance to sliding of the structures is developed by friction between the base of the footing and the bedrock. This friction **(R)** depends on the normal load on the soil contact (**N**) and the frictional resistance of the soil (**tan ϕ**) expressed as **R = N tan ϕ**. The factored geotechnical resistance at ULS is **0.8 R**. The coefficient of friction angle between the underside of a cast-in-place concrete footing and a clean sound bedrock surface may be taken as **tan 26° (0.5).**

5.3 Site Classification for Seismic Site Response

The Ministry of Municipal Affairs and Housing (MMAH) announced 2024 Ontario Building Code (OBC) that came into effect on January 1, 2025. The 2024 OBC is further harmonized with 2020 National Building Code (NBC) of Canada for the earthquake design analysis. The determination of the type of analysis is predicated on Earthquake Importance Factor (I_E), the Peak Ground Acceleration (PGA(X)), Peak Ground Velocity (PGV(X)), Design Spectral Acceleration (S(T)) and Site Designation (X_V or V_S).

Seismic hazard is defined by 5%-damped spectral acceleration ($S_a(T,X)$) at periods T of 0.2 s, 0.5 s, 1.0 s, 2.0 s, 5.0 s and 10.0 s corresponding to a 2% probability of exceedance in 50 years.

Site Designation (X) shall be X_V, where V is the value of average in situ shear wave velocity in top 30 m soil or rock ($\overline{V}_{S30}$) measured by a Multi-Channel Analysis of Shear Wave (MASW) test. Four (4) ground profiles as set out in the Table 4.1.8.4-A of 2024 OBC are except for determining Site Designation using V_{S30} calculated from the in suite measurements. The Site Designation shall be determined in accordance with the above table instead.

The 2024 OBC also provided alternative methods to determine the Site Designation (X_S), if V_{S30} calculated from in situ measurements is not available. The Site Designation shall be X_S, where S is the original Site Class determined using energy-corrected average standard penetration resistance($\overline{N}_{60}$) or average undrained shear strength ($\overline{S}_u$) in the top 30 m of the site stratigraphy below the foundation level. There are 6 site classes from A to F, decreasing in ground stiffness from A, hard rock, to E, soft soil; with site class F used to denote problematic soils (e.g., sites underlain by thick peat deposits and/or liquefiable soils).

The Site Designation is then used to obtain Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), and site coefficients (F_a and F_v), and Seismic Category (SC), respectively, used to modify the Uniform Hazard Spectra (UHS) to account for the effects of site-specific soil conditions.

The foundations are made to uniformly bear in the sound/unweathered shale bedrock, it is recommended that the Site Designation for seismic analysis be $\mathbf{X_B}$, as per the Ontario Building Code.

Note that it is preferable to determine the Site Designation as X_V on the basis of the average shear wave velocity (V_{S30}). This Site Designation will typically result in a lower seismic demand than a Site Designation X_S determined using the energy-corrected average standard penetration resistance($\overline{N}_{60}$) or average undrained shear strength ($\overline{S}_u$). Therefore, consideration may be given to conducting a site-specific Multichannel Analysis of Surface Waves (MASW) at this site to confirm the average shear wave velocity in the top 30 metres of the site stratigraphy.

5.4 Basement Floor Slab

The proposed floor would include a one-level underground parking garage with a partial second level underground parking garage. The lowest P1 and P2 FFE will be set at Elev. 73.2 m and Elev. 71.9 m. Based on the borehole information, the floor slab could be supported on the sound/unweathered shale bedrock, partially weathered shale bedrock and silty clay till. The modulus of subgrade reaction appropriate for the slab design is provided as follows,

- Clayey silt till: K_s =40,000 kPa/m
- Partially weathered shale bedrock: K_s =60,000 kPa/m
- Sound/unweathered shale bedrock: K_s =80,000 kPa/m

Prior to the construction of the slab, it is recommended that the subgrade be cut-neat, approved and inspected under the supervision of Englobe for obvious loose or disturbed areas as exposed, or for areas containing excessively deleterious materials or moisture. All sub-excavated areas shall be replaced with OPSS.MUNI 1010 Granular B placed as compacted fill (in lifts 150 mm thick or less and compacted to a minimum of 98 percent Standard Proctor Maximum Dry Density, SPMDD).

It is necessary that building basement slab be provided with a capillary moisture barrier and drainage layer. This is made by placing the slab on a minimum 200 mm layer of 19 mm stone (OPSS.MUNI 1004) compacted by vibration to a dense state. This material also serves as the drainage media for the subfloor drainage system. Provision of subfloor drainage is required in conjunction with the perimeter drainage of the structure.

The subfloor drainage system is an important building element, as such the storm sump which ensures the performance of this system must have a duplexed pump arrangement for 100% pumping redundancy and this pump must be provided with emergency power as needed. Basement and subfloor drainage provisions are further discussed in Section 5.5 of this report.

5.5 Basement Drainage

The groundwater level measurements in the monitoring wells were taken during the subsequent site visits ranged from about Elev. 72.7 to 77.4 m.

The exterior grade around the building should be sloped away at a 2 percent gradient or more for a distance of at least 1.2 m to assist in maintaining basement dry from seepage.

The basement wall (for basement) must be provided with damp-proofing provisions in conformance to the Section 9.13.2 of the Ontario Building Code (2024).

Based on the design information, the south excavation wall for the underground parking garage is likely to be adequately sloped while due to the limited setback available along the east, north and west property lines, it is not feasible to construct the excavation wall at this slope. Consequently, a shoring system will be required along these excavation limits.

In case of open excavation along the south limit, the basement wall backfill for a minimum lateral distance of 0.6 m out from the wall should consist of free-draining granular material (OPSS.MUNI 1010 Granular B), or provided with a prefabricated drain material (for instance, CCW Miradrain 6000 series, Terrafix Terradrain 200 or Delta-Drain 6000 HI-X), see Figure 3 Basement Drainage Detail. The perimeter drain installation and outlet provisions must conform to the plumbing code requirements.

Where the structure is made directly against a shored excavation, drainage is provided by forming a drained cavity with prefabricated drainage composites, such as CCW Miradrain 6000 series, Terrafix Terradrain 200 or Delta-Drain 6000 HI-X, should be incorporated between the shoring wall or rock face and the cast-in-place concrete foundation wall to make a drained cavity. Drainage from the cavity must be collected at the base of the wall in non-perforated pipes and conveyed directly to the sumps. The flow to the building storm water sump from the subsurface drainage will be governed largely by the building perimeter drainage collection during rainfall and runoff events. Typical shored excavation drainage details and Basement Subfloor Drainage details are provided in Figure 4 Schematic Drainage Detail - Soldier Pile & Lagging Shoring System.

The sub-floor drainage system should consist of perforated pipes (minimum 100 mm diameter) located at a spacing of about 5.0 m centre to centre (Refer to Figure 5 Basement Floor Subdrain Detail). The subdrain system should be outlet to a suitable discharge point under gravity flow or connected to a sump located in the lowest level of the basement. The water from the sump must be pumped out to a suitable discharge point/positive outlet. The installation of the drains as well as the outlet must conform to the applicable plumbing code requirements.

The drainage system is a critical structural element, since it keeps water pressure from acting on the basement floor slab. As such, the sump that ensures the performance of this system must have a duplexed pump arrangement for 100 percent pumping redundancy and these pumps must be on emergency power. The size of the sump should be adequate to accommodate the water seepage. It is anticipated the amount of seepage can be controlled with typical, widely available, commercial sump pumps.

A separate hydrogeological study has been prepared by Englobe for this site, which provides the approximate amount of daily temporary (construction) and permanent groundwater collection and discharge.

5.6 Pavement

The pavement design was provided for the parking lots, fire route and driveways in the following sections.

5.6.1 Pavement Design

The following flexible pavement thickness design is provided in the table below.

Pavement Structural Layers	Light Duty Pavement	Heavy Duty Pavement
Hot Mix Asphalt Surface Course, OPSS.MUNI 1150 HL 3	40 mm	40 mm
Hot Mix Asphalt Binder Course, OPSS.MUNI 1150 HL 8	50 mm	85 mm
Base Course, OPSS.MUNI 1010 Granular A	150 mm	150 mm
Subbase Course, OPSS.MUNI 1010 Granular B Type I	300 mm	350 mm
Total Thickness	540 mm	625 mm

Alternatively, consideration may also be given to the use of Portland cement concrete pavement where there is intense truck use, and turning of transport vehicles in conjunction with the waste handling, loading docks or delivery facilities. The following table provides the minimum recommended rigid pavement structures:

Pavement Structural Layers	Light Duty Pavement	Loading Dock
Portland Cement Concrete, CAN/CSA A23.1, Class C-2	150 mm	200 mm
Subbase Course, OPSS.MUNI 1010 Granular A	300 mm	300 mm
Total Thickness	450 mm	500 mm

It should be noted that in addition to the adherence to the above pavement design recommendations, a close control on the pavement construction process will also be required in order to obtain the desired pavement life. It is recommended that regular inspection and testing be conducted during the pavement construction to confirm material quality, thickness, and to ensure adequate compaction.

5.6.2 Drainage

Control of water is an important factor in achieving a good pavement life. The need for adequate subgrade drainage cannot be over-emphasized. The subgrade must be free of depressions and sloped (preferably at a minimum grade of 3%) to provide effective drainage toward subgrade drains. Grading adjacent to the pavement areas should be designed to ensure that water is not allowed to pond adjacent to the outside edges of the pavement.

Continuous pavement subdrains should be provided along both sides of the driveway and drained into respective catchbasins to facilitate drainage of the subgrade and granular materials. Continuous subdrains should be also provided for the parking lot/driveway pavement areas along the curb-lines/sidewalk and at all catchbasins within the parking areas. Two lengths of subdrain (each minimum of about 3 m long) should be installed at each catchbasin. The subdrain invert should be maintained at least 0.3 m below subgrade level. All subdrain arrangements should comply with the City of Mississauga Standard Drawing No. 2220.040.

5.6.3 General Pavement Recommendations

HL 3 and HL 8 hot mix asphalt (HMA) mixes should be designed, produced and placed in conformance with OPSS.MUNI 1150 and OPSS.MUNI 310 requirements and the relevant City’s requirements.

Portland cement concrete should be designed in accordance with CAN/CSA-A23.1 to provide a minimum 28-day compressive strength of 32 MPa.

Both the Granular A and Granular B Type I materials should meet the requirements of OPSS.MUNI 1010 requirements and the relevant City’s standards. Granular materials should be compacted to 100% of Standard Proctor Maximum Dry Density (SPMDD).

HL3 HS HMA is recommended as padding. Padding should be placed in lifts not exceeding 50 mm.

Performance graded asphalt cement, PG 58-28, conforming to OPSS.MUNI 1101 requirements, should be used in both HMA binder and surface courses.

A tack coat (SS1) should be applied to all construction joints prior to placing hot mix asphalt to create an adhesive bond. SS1 tack coat should also be applied between hot mix asphalt binder and surface courses.

The joint layout should be carried out in accordance with OPSD 551.010. The joint construction, including dowel bar and tie bar layouts, and joint reservoirs should be carried out in accordance with OPSD 552.010.

A transition panel will be required wherever a concrete pavement is to be connected to an asphalt pavement. The transition should be constructed in accordance with OPSD 555.010.

5.6.4 Subgrade Preparation

All topsoil, organics, soft/loose and otherwise disturbed/weathered soils should be stripped from the subgrade areas. The exposed subgrade is expected to consist of the earth fill materials or native soils, which will be weakened by construction traffic when wet; especially if site work is carried out during the periods of wet weather. An adequate granular working surface would be likely required in order to minimize subgrade disturbance and protect its integrity during wet periods.

Immediately prior to placing the granular subbase, the exposed subgrade should be proof rolled with a heavy rubber-tired vehicle (such as a loaded gravel truck). The subgrade should be inspected for signs of rutting, distress and displacement. Areas displaying signs of rutting, distress and displacement should be recompacted and retested or, these materials should be locally excavated and replaced with well-compacted clean approved fill material.

The fill material may consist of either granular material or local inorganic soils provided that its moisture content is within ±2 percent of Optimum Moisture Content (OMC). Fill material should be placed and compacted in accordance with OPSS.MUNI 501 and the subgrade should be compacted to 98 percent of SPMDD. The final subgrade surface should be sloped at least 3 percent to provide positive drainage.

5.7 Excavations

The boreholes data indicate that the earth fill materials and undisturbed native soils would be encountered in the excavations. Excavations must be carried out in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety.

TYPE 1 SOIL

a. is hard, very dense and only able to be penetrated with difficulty by a small sharp object;
b. has a low natural moisture content and a high degree of internal strength;
c. has no signs of water seepage; and
d. can be excavated only by mechanical equipment.

TYPE 2 SOIL

a. is very stiff, dense and can be penetrated with moderate difficulty by a small sharp object;
b. has a low to medium natural moisture content and a medium degree of internal strength; and
c. has a damp appearance after it is excavated.

TYPE 3 SOIL

a. is stiff to firm and compact to loose in consistency or is previously-excavated soil;
b. exhibits signs of surface cracking;
c. exhibits signs of water seepage;
d. if it is dry, may run easily into a well-defined conical pile; and
e. has a low degree of internal strength

TYPE 4 SOIL

a. is soft to very soft and very loose in consistency, very sensitive and upon disturbance is significantly reduced in natural strength;
b. runs easily or flows, unless it is completely supported before excavating procedures;
c. has almost no internal strength;
d. is wet or muddy; and
e. exerts substantial fluid pressure on its supporting system.

The earth fill materials encountered in the boreholes are classified as Type 3 Soil, while the undisturbed native soils would be classified as Type 2 Soil above and Type 3 Soil below prevailing groundwater level under these regulations.

Where workmen must enter excavations advanced deeper than 1.2 m, the trench walls should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The regulation stipulates the steepest slopes of excavation by soil type as follows:

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 the Occupational Health and Safety Act and Regulations for Construction Projects, and include provisions for timbering, shoring and moveable trench boxes.

The excavations will be carried out through the fill zone overlying stiff to hard till deposit followed by weathered and sound rock. Obstructions and boulders should be expected in the overburden. Excavation of the shale, if required, can be carried out using heaviest available single tooth ripper equipment. The stronger limestone and siltstone beds are frequent and therefore be necessary to utilize hoe rams to "open" the hard layers of limestone/siltstone for the ripper. The removal of the underlying fresh and stronger rock and especially the interbedded limestone and siltstone layers and zones where rock quality is "fair", "good" or "excellent" (i.e. RQD > 50%), will be arduous and time consuming.

It should be noted that the native deposits may contain larger particles (cobbles and boulders) that are not specifically identified in the Borehole Logs. The size and distribution of such obstructions cannot be predicted with borings, because the borehole sampler size is insufficient to secure representative samples of the particles of this size. Provision should be made in excavation contracts to allocate risks associated with time spent and equipment utilized to remove or penetrate such obstructions when encountered.

5.8 Groundwater Control

Englobe has completed Hydrogeological Report (File No.02211043.006) for this site to provide groundwater control measures and estimate ground water discharge volume (Refer to this report for detailed information about ground water volumes, quality and control provisions).

The groundwater level measurements taken on September 18, 2025, and June 16, 2026, in the monitoring wells indicated that the groundwater levels ranged from Elev. 72.7 to 77.4 m. The groundwater levels may fluctuate seasonally depending upon the amount of precipitation and surface runoff.

The site is generally underlain by relatively low permeability glacial till deposit (silty clay till) which should preclude significant amounts of free-flowing groundwater seepage into the excavation in the short-term. However, perched water seepage will likely be encountered during the excavations primarily emanating from the earth fill zone, and silt/sand lenses typically found in the glacial till deposit due to its mode of deposition. The identification and delineation of these lenses in the boreholes are difficult due to the standard interval sampling method.

Based on the borehole information and design information provided, it is likely that perched water seepage will be encountered in the underground parking excavation. The perched water seepage emanating should diminish slowly and can be controlled by continuous pumping from filtered sumps at the base of the excavation.

For excavations extending below the prevailing ground water level, it will be necessary to lower the ground water level and maintain it below the excavation base (at least 1.0 m) prior to and during the subsurface construction. A professional dewatering expert should review the subsurface information to assess the potential requirement of dewatering and establish appropriate dewatering methodology which will be responsibility of the dewatering contractor. Consideration should be given to install a skim coat of lean concrete (mud-slab) to preserve the subgrade integrity, and to provide a working platform as required based on site conditions.

5.9 Backfill

The earth fill materials containing excessive amounts of organic inclusion should not be reused as backfill in settlement sensitive areas, such as beneath the floor slabs, trench backfill and pavement areas. However, these materials may be stockpiled and reused for landscaping purposes.

The existing earth fill materials are considered suitable (with selection and sorting as required) for backfill provided the moisture content of these soils is within ±2 percent of the OMC. Any soil material with ±2 percent or higher in-situ moisture content than its OMC, could be put aside to dry or be tilled to reduce the moisture content so that it can be effectively compacted. Alternatively, materials of higher moisture content could be wasted and be replaced with imported material which can be readily compacted.

The existing earth fill soil will likely require selection and sorting to be reused as backfill. The selection and sorting must be conducted under the supervision of a geotechnical engineer. The site soils will be best compacted with a heavy sheep foot type roller.

The backfill should consist of clean earth and be placed in lifts of 150 mm thickness or less, and heavily compacted to a minimum of 95 percent SPMDD at a water content close to optimum (within 2 percent). The upper 600 mm of the pavement subgrade (at driveways outside of the basement roof deck) must be compacted to a minimum of 98 percent SPMDD.

It should be noted that the soils encountered on the site are generally not free draining, and will be difficult to handle and compact should they become wetter as a result of inclement weather or seepage.

5.10 Shoring Design

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Temporary shoring should be carried out by a licensed Professional Engineer experienced in shoring design.

The east, north and west excavation walls for the underground parking garage will likely have to be shored to preserve the integrity of the boundary conditions. The south excavation wall can be adequately sloped. No excavation shall extend below a line cast as one vertical to one horizontal from foundations of the existing structures without adequate alternate support being provided. Where the adjacent building foundations are removed from the excavation, a foundation which lies above a line drawn upward at 10 horizontal to 7 vertical from the closest excavation edge is within the zone of potential influence of the excavation, and support for the existing foundations must be carefully assessed and possibly augmented. Underpinning guidelines are provided in Figure 6.

The shoring requirements for the site will have to be examined in detail with respect to the site boundary constraints, once the development details and the building footprint are finalized. Based on the boundary conditions and structures located in the vicinity, groundwater condition and dewatering details, consideration may be given to a permeable shoring system (a steel soldier piles and timber lagging).

5.10.1 Earth Pressure Distribution

If a single level of support will be required for shoring system, a triangular earth pressure distribution similar to that used for the basement wall design, is appropriate for this case,

$$P = K(\gamma H + q)$$

Where:
P = the horizontal pressure (kPa)
K = the earth pressure coefficient
H = the total depth of excavation (m)
γ = the bulk unit weight of soil (kN/m^3)
q = the complete surcharge loading (kPa)

Applicable soil parameters are included in the Earth Pressure Design Parameters Section (Section 5.2).

5.10.2 Pile Toe Design

Pile toes will be made in sound/unweathered bedrock of the Georgian Bay Formation. The maximum factored geotechnical resistance at ULS for the design of a pile, embedded in the sound bedrock is

10 MPa. The maximum factored ultimate lateral geotechnical resistance of the sound rock at ULS is 1 MPa.

It should be noted that possibility exists that there can be zones of material in the subsurface soils which could be sufficiently wet and permeable (specially fill materials) such that augered borings for soldier piles made through these soils could be unstable. In these cases, it will be necessary to advance temporarily cased holes to maintain sidewall support and to prevent the ingress of water during soldier pile (and caisson wall fillers, if applicable) installation. Slurry, polymer or other drilling methods may be required.

The exposed Georgian Bay Formation deteriorates with time. Exposed excavation faces have been found to flake and recede as much as 300 mm with 12 months exposure. This recession generally takes the form of coin-size shale particles dropping from the face on a constant basis. The deteriorated rock loses internal integrity and bearing capability. Typically, the soldier piles are advanced at least 2 m into the sound bedrock to accommodate this weathering to ensure the lateral and vertical bedrock capacity in the design can be achieved.

5.10.3 Lateral Bracing Elements

It will be necessary to secure encroachment agreements from the adjacent land owners, in order to use soil anchors on the adjoining properties. Pre-construction condition surveys should be carried out for the adjacent structures to establish existing conditions prior to excavation and mitigate the possibility of spurious claims for excavation induced damages. Access to the properties for such surveys must be part of any encroachment agreements.

A careful evaluation of the subsurface soil conditions is required by the shoring designer to establish appropriate levels/elevations and design the soil anchors. The anchor design will be governed by the weakest material in the profile. It is imperative that a detailed design is carried out at different anchor levels and locations, and the anchors must be tested at each level.

The optimal anchor design at this site will consist of rock anchors made in the Bedrock of Georgian Bay Formation. Conventional earth anchors made in bedrock may be designed using a working adhesion of 620 kPa. One or more proto-type anchor must be performance tested to 200% of the design load at each anchor level to demonstrate the anchor capacity and validate the design assumptions. This test must be completed before production anchors are made. All production anchors must be proof-tested to 133% of the design load.

Regardless, the subsurface soil information should be reviewed by the shoring designer to decide on the suitable type of earth anchors and anchor capacity to be employed at this site.

Where anchors cannot be used, internal bracing or rakers would be necessary. The footings for the rakers would be made on the sound bedrock where they could be designed using a maximum geotechnical resistance at ULS of 2,000 kPa when inclined at 45 degrees.

5.11 Quality Control

All foundations must be monitored by the geotechnical engineer on a continuous basis as they are constructed. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.2 of the Ontario Building Code. If Englobe is not retained to carry out foundation evaluations during construction, then Englobe accepts no responsibility for the performance or non-performance of the foundations, even if they are ostensibly constructed in accordance with the conceptual design advice provided in this report.

Concrete for this structure will be specified in accordance with the requirements of CAN3 - CSA A23.1. Englobe maintains a CSA certified concrete laboratory and can provide concrete sampling and testing services for the project as necessary.

The requirements for fill placement on this project should be stipulated relative to SPMDD, as determined by ASTM D698. In-situ determinations of density during fill placement by Procedure Method B of ASTM D2922 are recommended to demonstrate that the contractor is achieving the specified soil density. Englobe is a CNSC licensed operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary.

Englobe can provide thorough in house resources, quality control services for Building Envelope, Roofing, as well as Structural Steel in accordance with CSA W178, as necessary, for the Structural and Architectural quality control requirements of the project. Englobe is certified by the Canadian Welding Bureau under W178.1-1996.

6 Limitations and Risk

6.1 Procedures

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Englobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained by Englobe.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Englobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Englobe has interpreted to exist between sampling points can differ from those that actually exist.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities so that they may draw their own conclusions as to how the subsurface conditions may affect them.

6.2 Changes in Site and Scope

It must also be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. Groundwater levels are particularly susceptible to seasonal fluctuations.

The discussion and recommendations are based on the factual data obtained from this investigation conducted at the site by Englobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructability issues and quality control may not be relevant or complete for the revised project. Englobe should be retained to review the implications of such changes with respect to the contents of this report.

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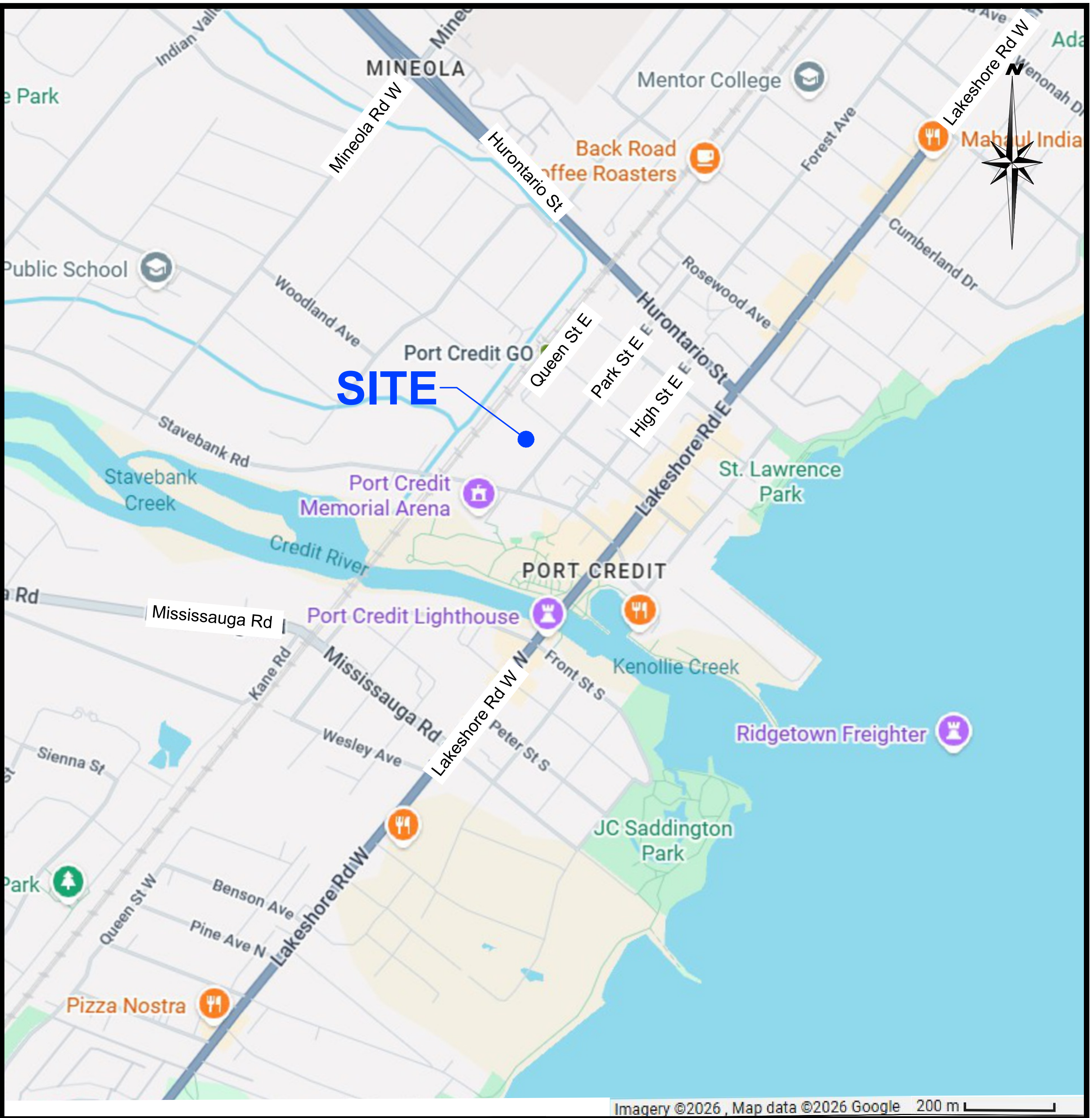
It is recognized that the regulatory agencies in their capacities as the planning and building authorities under Provincial statues, will make use of and rely upon this report, cognizant of the limitations thereof, both expressed and implied.

We trust the foregoing information is sufficient for your present requirements. If you have any questions, or if we can be of further assistance, please do not hesitate to contact us.

Figures



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Client

Canahahns Company Limited

Project

26 Park St East

Mississauga, Ontario

Title

Site Location Plan

ENGLOBE

Discipline: Geosciences		Prepared by: KC	Checked by: JZ
Scale: As Shown		Drawn by: KC	Approved by: SZ
Date: July 2026		Figure N°: ----	
Page setup: Figure 1	Paper Format: ANSI full bleed A (8.50 x 11.00 Inches)	Register N°:	

Resp.	Project	Phase	Disc.	Type	Drawing N°	Rev.
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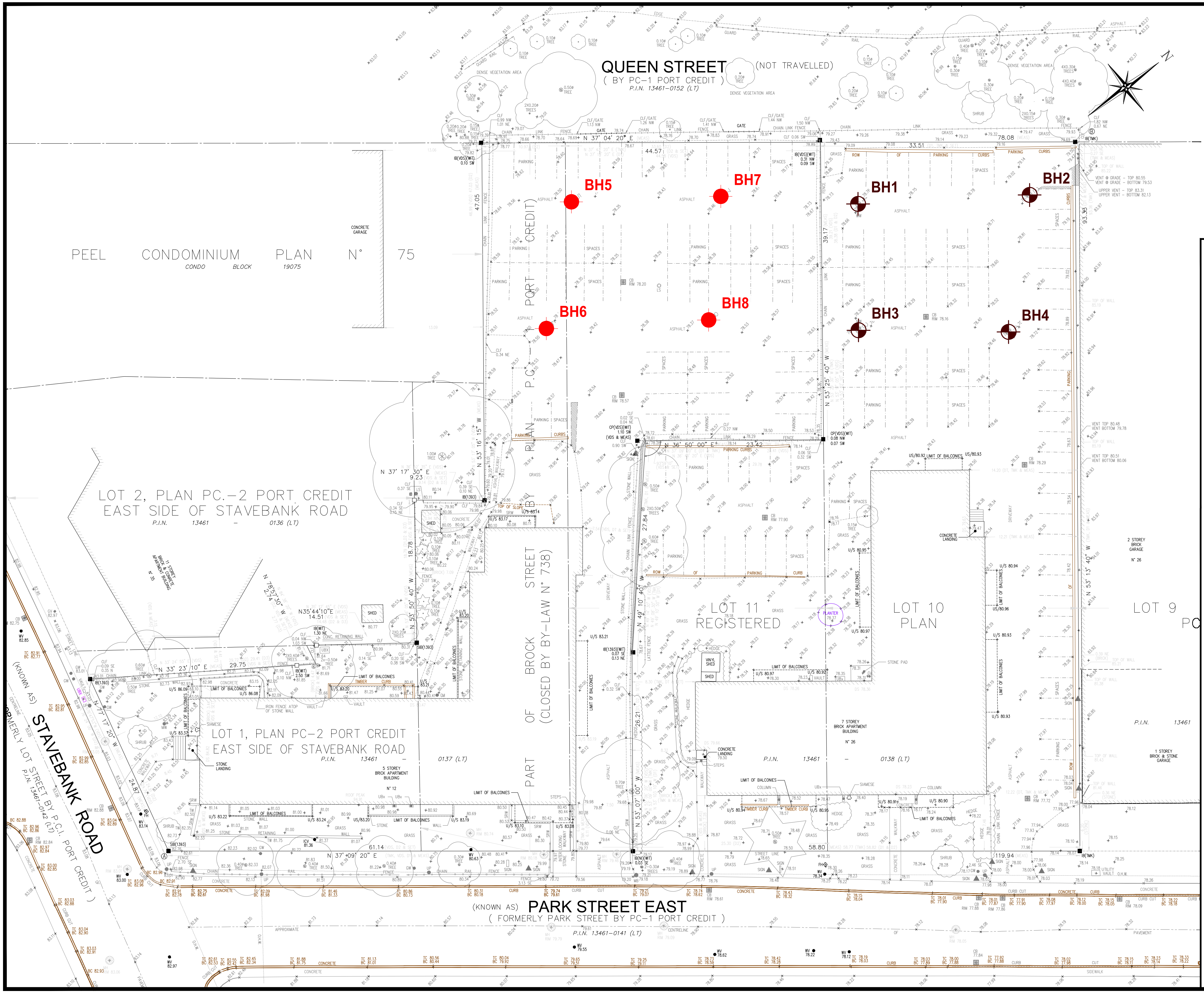

This document must be used jointly with the recommendations formulated in the geotechnical study report

XX	REVISION	JJ/MM/AA	XX	XX	XX
No.	Version	Date	By	Check	Appr.

REFERENCES:
Topographic Survey
Project No.: GLS-2786,
Date: June 16, 2026
By: Genesis Land Surveying Inc.

LEGEND:

●	Approx. Borehole Location (2026)
⊕	Approx. Borehole Location (2025)

Scale

0 2.5 5 7.5 10 15 20 25 m

1:500

Client

Canahahns Company Limited

ENGLOBE

Englobe Corp.
20 Carlson Court
Etobicoke, ON M9W 7K6
T 416 213-1060

Project

26 Park St East

Mississauga, Ontario

Title

Borehole Location Plan
Existing Condition

Discipline: Geotechnical		Prepared by: KC	Checked by: JZ
Scale: 1:500		Drawn by: KC	Approved by: JZ
Date: Julyl 2026		Figure N°: ---	
Page setup: Figure 2A	Paper format: ANSI full bleed B (17.00 x 11.00 Inches)	Register N°:	

Resp.	Project	Phase	Disc.	Type	Drawing N°	Rev.
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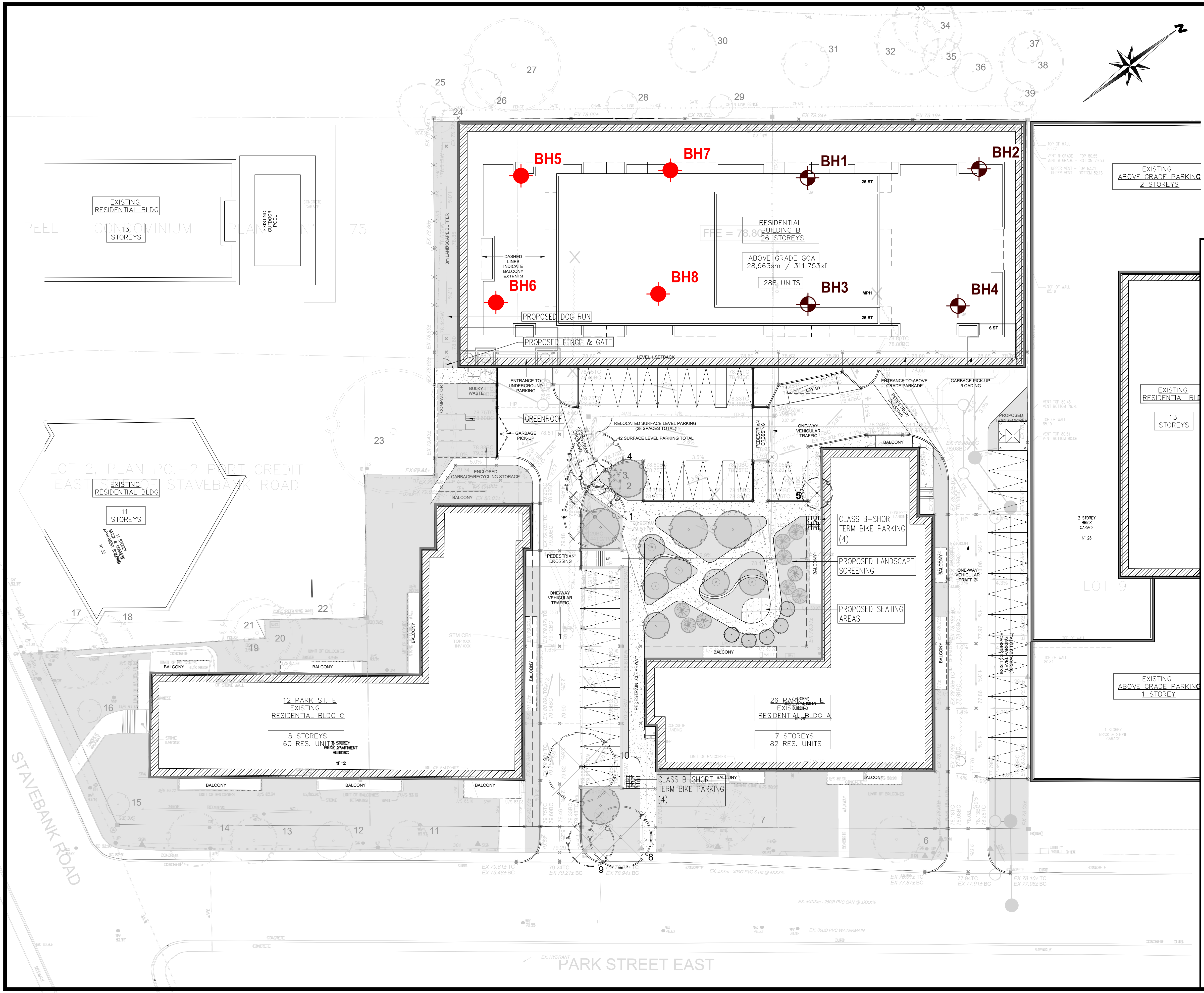

Y:\SHARED\PROJ\ON\2026\02211043.005_CANAHAHNS_GEO_26_PARK_ST_E_MISSISSAUGA\4_CAD\02211043.005 (GLS-2786-2) FIGURE 2B 2026-07-30.DWG

This document must be used jointly with the recommendations formulated in the geotechnical study report

XX	REVISION	JJ/MM/AA	XX	XX	XX
No.	Version	Date	By	Check	Appr.

REFERENCES:

Drawing provided in digital format by Ron Kaudys Landscape Architects Inc., received July 30, 2026 via email.

LEGEND:

- 🔴 Approx. Borehole Location (2026)
- ⊕ Approx. Borehole Location (2025)

Scale

0 2.5 5 7.5 10 15 20 25 m

1:500

Client

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Project

26 Park St East

Mississauga, Ontario

Title

Borehole Location Plan
Proposed Condition

Discipline: Geotechnical

Scale: 1:500

Date: Julyl 2026

Page setup: Layout1

Paper format: ANSI full bleed B (17.00 x 11.00 Inches)

Prepared by: KC

Checked by: JZ

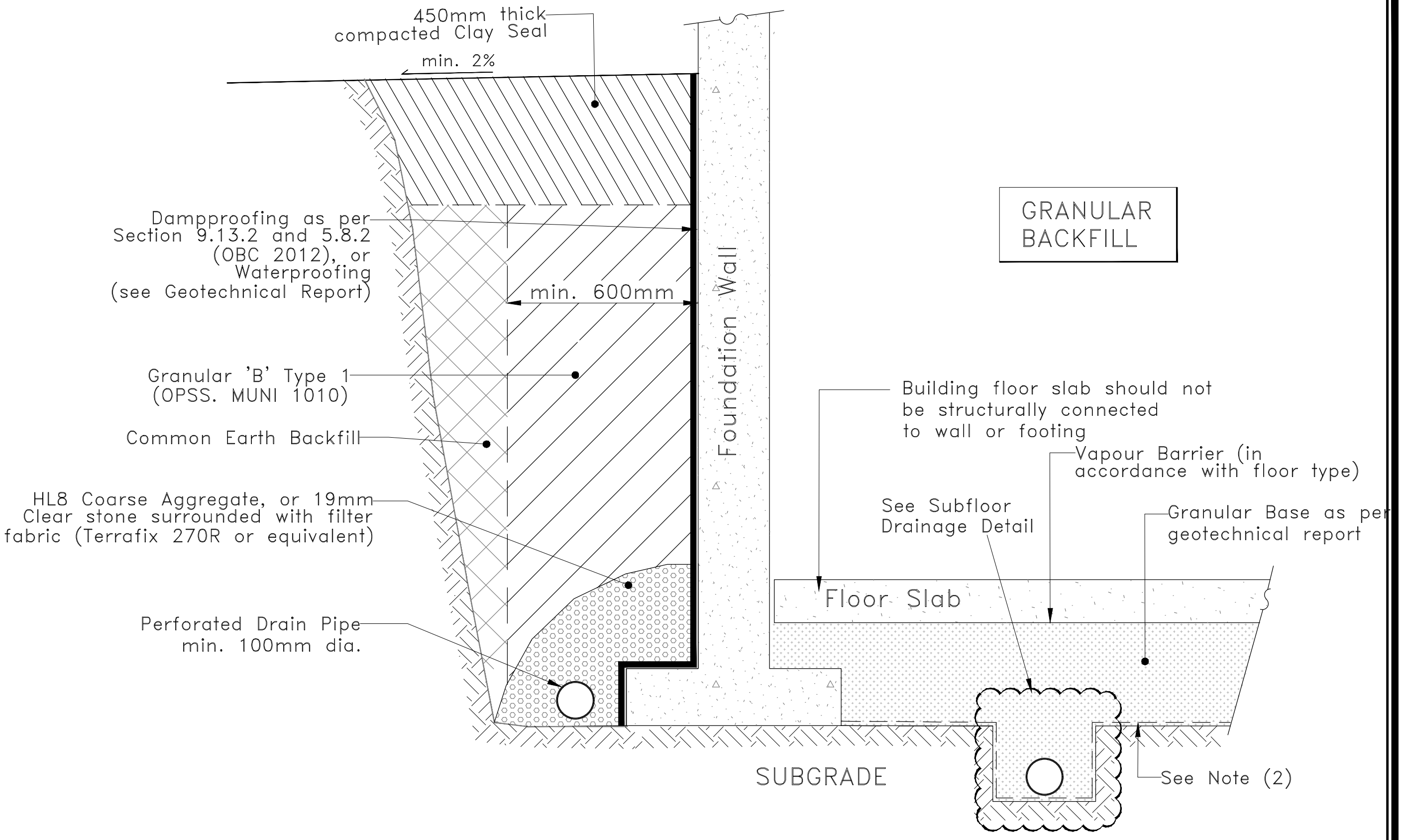
Drawn by: KC

Approved by: JZ

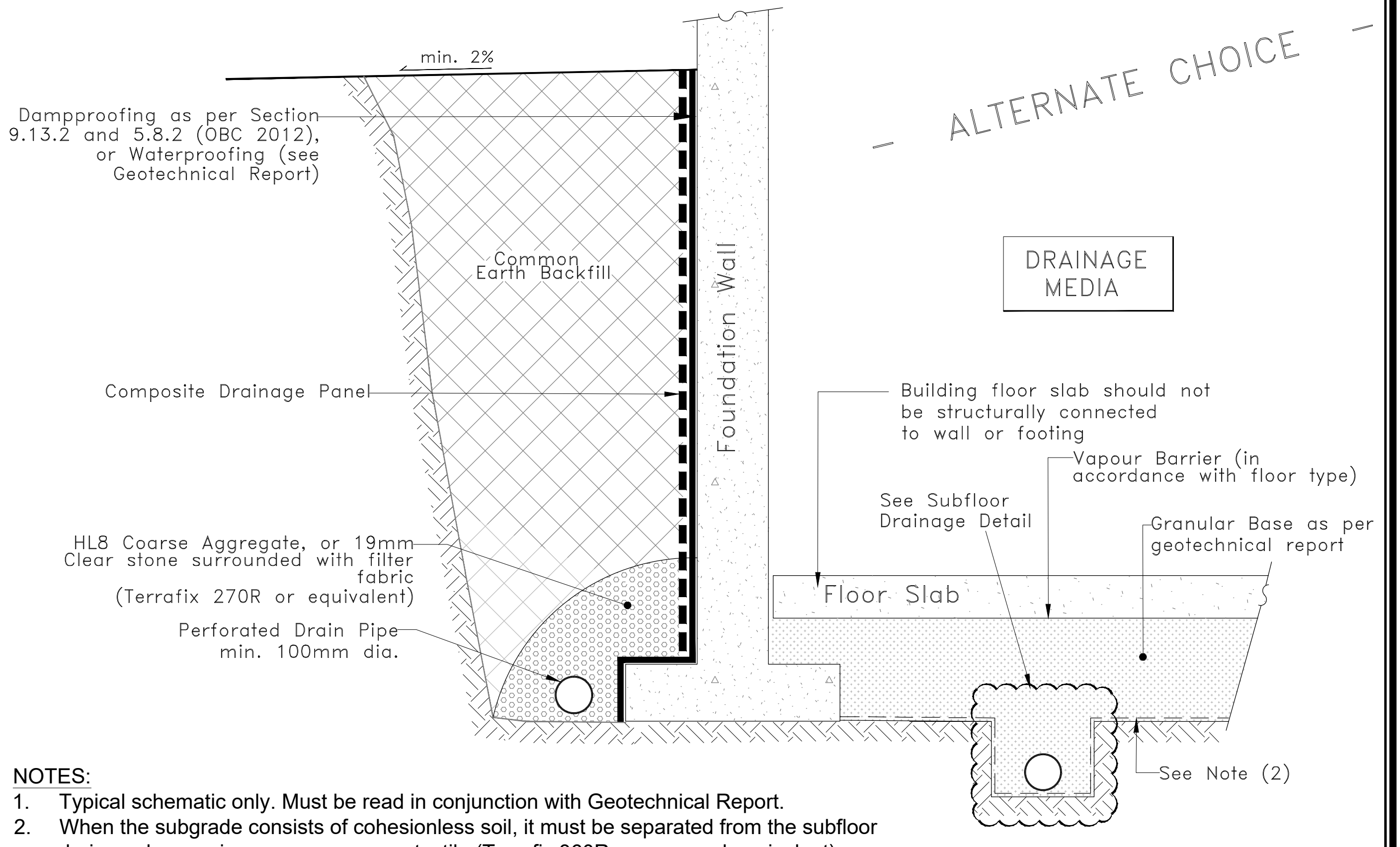
Figure N°: ---

Register N°:

Resp.	Project	Phase	Disc.	Type	Drawing N°	Rev.
00	02211043.005	0000	GS	D	2B	00



— ALTERNATE CHOICE —



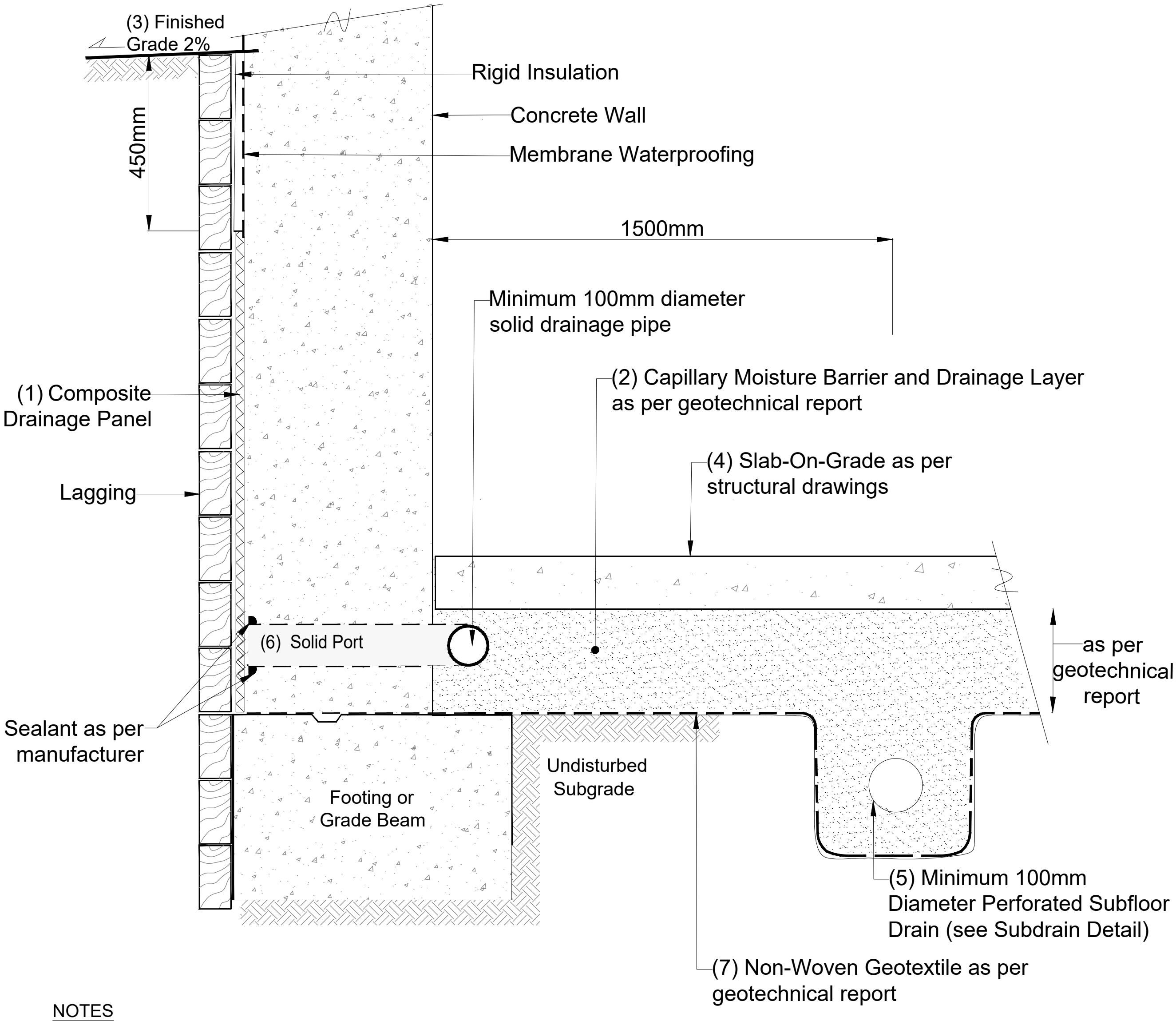
NOTES:

1. Typical schematic only. Must be read in conjunction with Geotechnical Report.
2. When the subgrade consists of cohesionless soil, it must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).
3. Not to Scale

englobe

Title:

TYPICAL BASEMENT DRAINAGE SCHEMATIC



NOTES

1) Prefabricated composite drainage panels to consist of Miradrain 6000, or approved equivalent. Panels should provide continuous cover as per manufacturer's requirements.

2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS. MUNI 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular “A” (OPSS. MUNI 1010) compacted to 98% SPMDD where vehicular traffic is required. A vapour barrier may be required depending on floor type.

3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.

4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.

5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report. Don't connect subfloor drains to perimeter drains.

6) Embedded ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in solidpipe.

7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

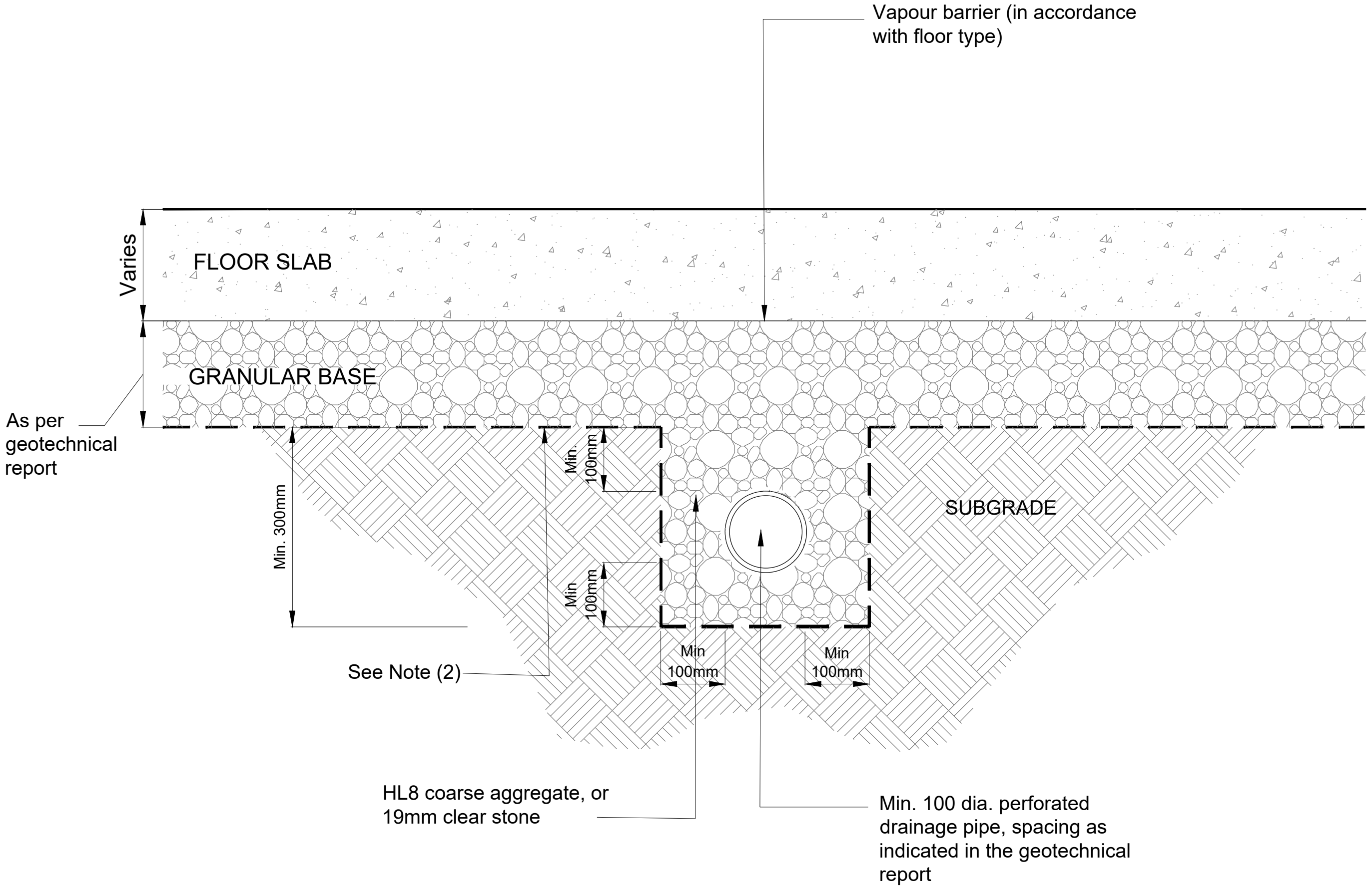
8) Geotechnical report contains specific details. Final detail must be reviewed before system is considered acceptable to use.

N.T.S.

ENGLOBE

Title:

SCHEMATIC DRAINAGE DETAIL
SOLDIER PILE & LAGGING SHORING SYSTEM
(DRAINED BASEMENT CONDITION)



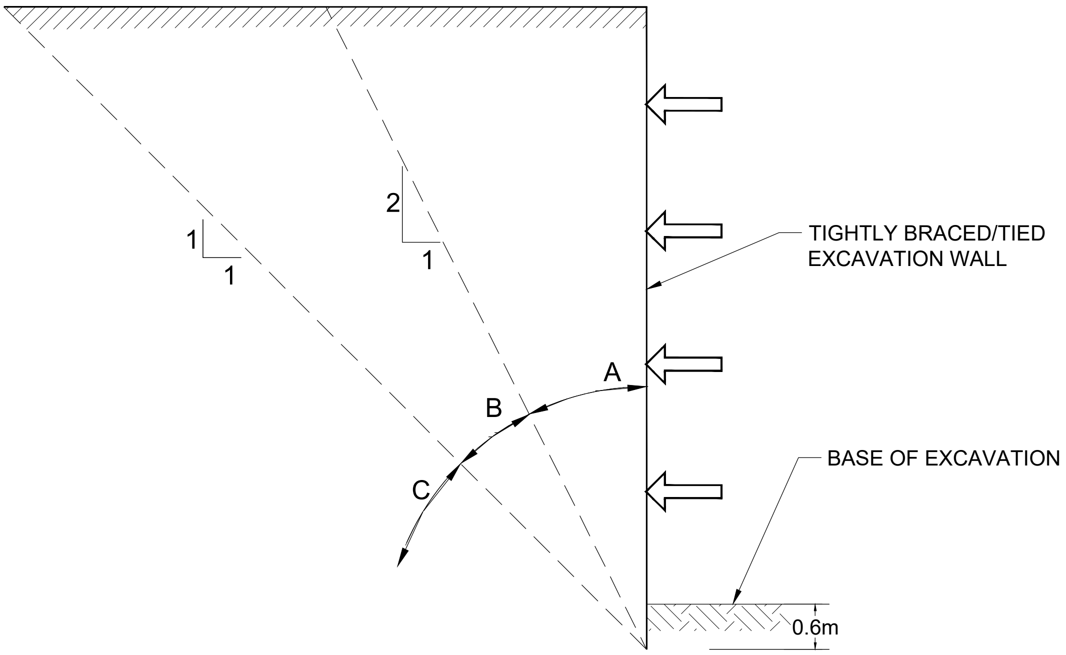
NOTES:

1. Typical schematic only. Must be read in conjunction with Geotechnical Report.
2. When the subgrade consists of cohesionless soil, it must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).
3. Not to Scale

ENGLOBE

Title:

TYPICAL BASEMENT SUBDRAIN DETAIL



Zone A: Foundations within this zone often require underpinning. Horizontal and vertical pressures on excavation wall of non-underpinned foundations must be considered.

Zone B: Foundation within this zone often do not require underpinning. Horizontal and vertical pressures on excavation wall of non-underpinned foundations must be considered.

Zone C: Foundations within this zone usually do not require underpinning.

REFERENCE

User's Guide - NBC 2005 Structural Commentaries

(Part 4 of Division B) - Commentary K

Title:

GUIDELINES FOR UNDERPINNING SOILS

Appendix A
Borehole Logs

englobe

ABBREVIATIONS AND TERMINOLOGY

SAMPLING METHODS

AS	auger sample
CORE	cored sample
DP	direct push
FV	field vane
GS	grab sample
SS	split spoon
ST	shelby tube
WS	wash sample

PENETRATION RESISTANCE

Standard Penetration Test (SPT) resistance ('N' values) is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a standard 50 mm (2 in.) diameter split spoon sampler for a distance of 0.3 m (12 in.).

Dynamic Cone Test (DCT) resistance is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a conical steel point of 50 mm (2 in.) diameter and with 60° sides on 'A' size drill rods for a distance of 0.3 m (12 in.)."

COHESIONLESS SOILS

Compactness	**'N' value**
very loose	< 4
loose	4 – 10
compact	10 – 30
dense	30 – 50
very dense	> 50

COHESIVE SOILS

Consistency	**'N' value**	**Undrained Shear Strength (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 (e.g)	**% by weight**
trace silt	< 10
some silt	10 – 20
silty	20 – 35
sand *and* silt	> 35

TESTS AND SYMBOLS

MH	mechanical sieve and hydrometer analysis
w, w_c	water content
w_L, LL	liquid limit
w_P, PL	plastic limit
I_P, PI	plasticity index
k	coefficient of permeability
γ	soil unit weight, bulk
G_s	specific gravity
φ'	internal friction angle
c'	effective cohesion
c_u	undrained shear strength

▽	Unstabilized water level
⧨	1st water level measurement
⧩	2nd water level measurement
▼	Most recent water level measurement
3.0 +	Undrained shear strength from field vane (with sensitivity)
C_c	compression index
c_v	coefficient of consolidation
m_v	coefficient of compressibility
e	void ratio

FIELD MOISTURE DESCRIPTIONS

Damp	refers to a soil sample that does not exhibit any observable pore water from field/hand inspection.
Moist	refers to a soil sample that exhibits evidence of existing pore water (e.g. sample feels cool, cohesive soil is at plastic limit) but does not have visible pore water
Wet	refers to a soil sample that has visible pore water

ROCK CORE TERMINOLOGY

RECOVERY

TCR	**Total Core Recovery** is the total length of core pieces, irrespective of their individual lengths obtained in a core run, and expressed as a percentage of the length of that core run.
SCR	**Solid Core Recovery** is the total length of sound full-diameter core pieces obtained in a core run, expressed as a percentage of the length of that core run .
RQD	**Rock Quality Designation** pertains to the sum of those pieces of sound core which are 10 cm or greater in length obtained in a core run, expressed as a percentage of the length of that core run.

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
QUALITY	**very poor**	**poor**	**fair**	**good**	**excellent**

JOINT CHARACTERISTICS

Joint Spacing (adapted from *Bieniawski 1989*, *ISRM 1981*)

Classification	Spacing
very close	< 60 mm
close	60 – 200 mm
moderately close	0.2 to 0.6 m
wide	0.6 to 2 m
very wide	> 2 m

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

Orientation

Orientation	Angle from horiz.
horizontal/flat	0 - 20°
dipping	20 - 50°
vertical	50 - 90°

Joint Filling

Description	Approx. φ`
tight, hard, non-softening	25 - 35
oxidation, surface staining only	25 - 30
slightly altered, clay-free	25 - 30
sandy particles, clay-free	2 - 25
sandy and silty, minor clay	1 - 24
non-softening clays	6 - 12
swelling clay fillings	n/a

Joint Aperture

Classification	Aperture
closed / tight	< 0.5 mm
gapped	0.5 to 10 mm
open	> 10 mm

Planarity

- Planar
- Undulating
- Stepped
- Irregular
- Discontinuous

Roughness

- Very rough
- Rough
- Smooth
- Slickensided
- Polished

Coating	Description
clean	no filling
veneer	< 1 mm filling
coating / infill	> 1 mm filling

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*)

Grade	Term	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 (*Quarterly Journal of Engineering Geology, Vol 3, 1970*)

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

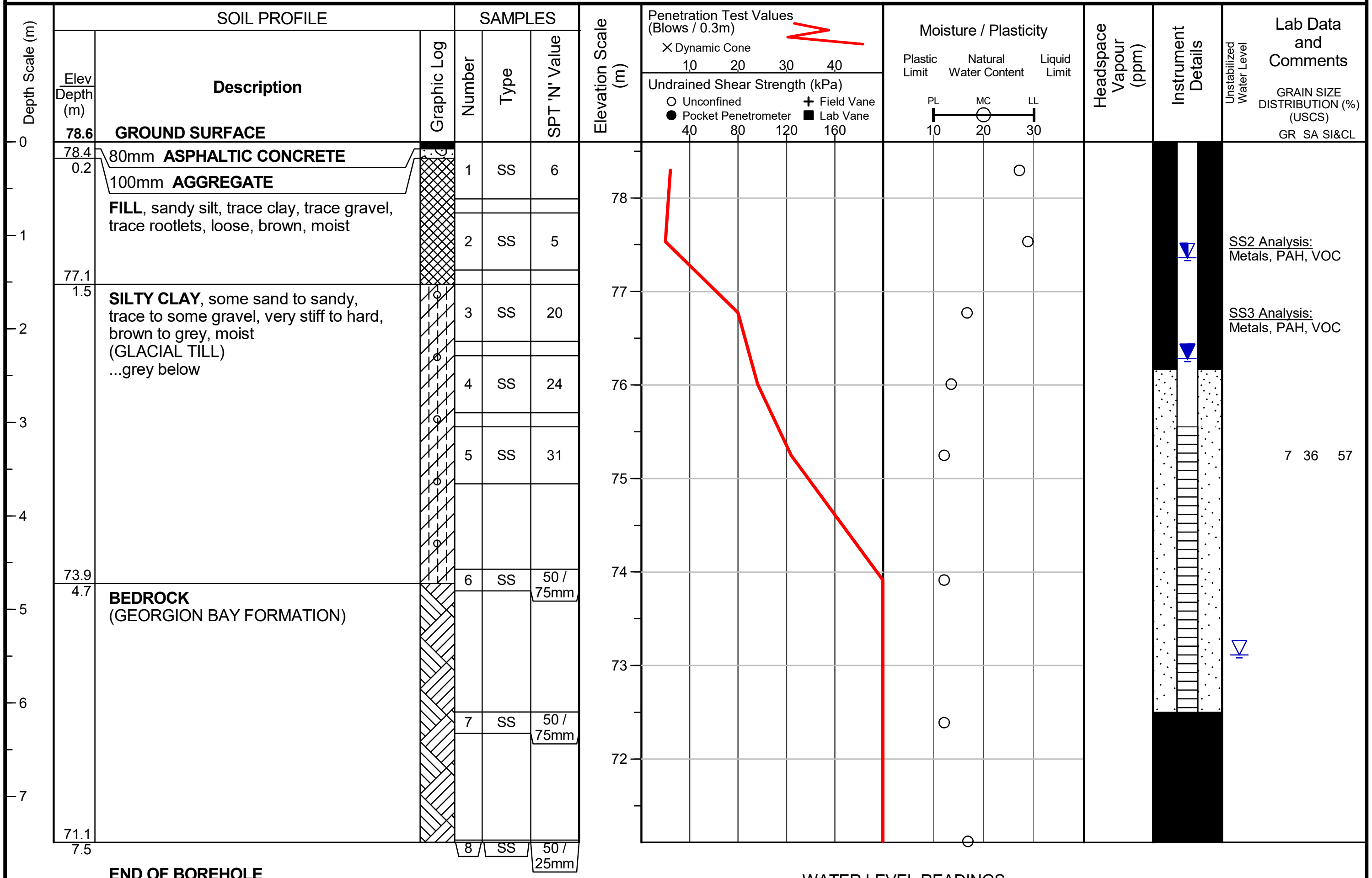
LOG OF BOREHOLE 1

Project No. : 02211043.005
Date started : August 25, 2025
Sheet No. : 1 of 1

Client : Canahahns Company Limited
Project : 26 Park St E
Location : Mississauga, Ontario

Originated by : SM
Compiled by : JZ
Checked by : SZ

Position : E: 614051, N: 4823353 (UTM 17T)
Rig type : Truck-mounted
Elevation Datum : Geodetic
Drilling Method : Solid stem augers



Elev Depth (m)	SOIL PROFILE Description	SAMPLES Number	Type	SPT 'N' Value	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (USCS) GR SA SI&CL
78.6	**GROUND SURFACE**				
78.4 0.2	80mm **ASPHALTIC CONCRETE** 100mm **AGGREGATE**				
	FILL, sandy silt, trace clay, trace gravel, trace rootlets, loose, brown, moist	1	SS	6	
		2	SS	5	SS2 Analysis: Metals, PAH, VOC
77.1 1.5	**SILTY CLAY**, some sand to sandy, trace to some gravel, very stiff to hard, brown to grey, moist (GLACIAL TILL) ...grey below	3	SS	20	SS3 Analysis: Metals, PAH, VOC
		4	SS	24	
		5	SS	31	7 36 57
73.9 4.7	**BEDROCK** (GEORGION BAY FORMATION)	6	SS	50 / 75mm	
		7	SS	50 / 75mm	
71.1 7.5		8	SS	50 / 25mm	

END OF BOREHOLE

Unstabilized water level measured at 5.5 m below ground surface; borehole was open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Sep 18, 2025	1.2	77.4
Jun 16, 2026	2.3	76.3

file: 02211043.005 26_park_st_e_ bh logs.gpj

LOG OF BOREHOLE 2

Project No. : 02211043.005
Date started : August 25, 2025
Sheet No. : 1 of 1

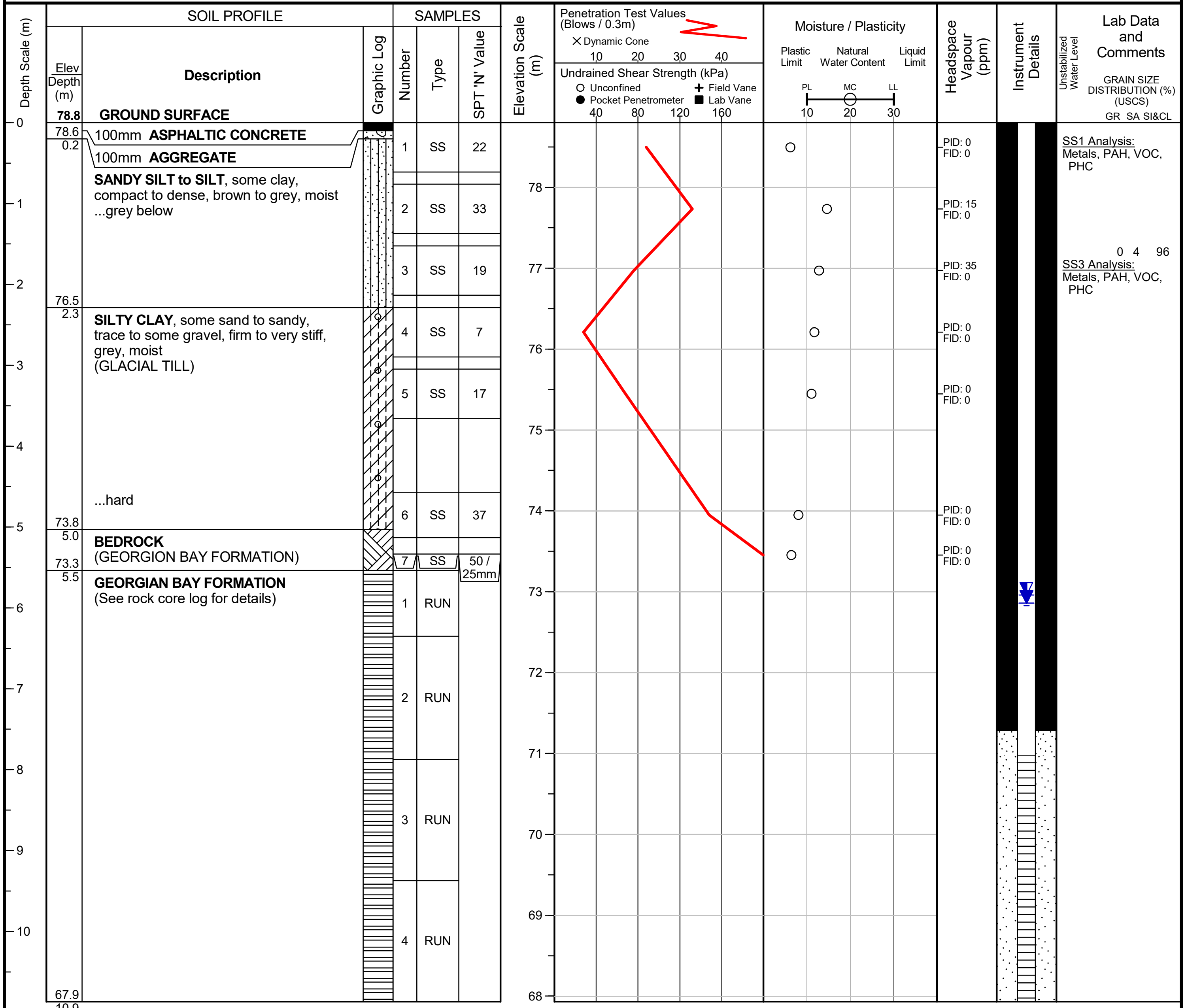
Client : Canahahns Company Limited
Project : 26 Park St E
Location : Mississauga, Ontario

Originated by : SM
Compiled by : JZ
Checked by : SZ

Position : E: 614063, N: 4823371 (UTM 17T)
Rig type : Truck-mounted

Elevation Datum : Geodetic
Drilling Method : Solid stem augers, HQ rock coring

Core Diameter : **HQ**, OD=96mm, ID=64mm



SOIL PROFILE		SAMPLES				
Elev Depth (m)	Description	Number	Type	SPT 'N' Value	Headspace Vapour (ppm)	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (USCS) GR SA SI&CL
78.8	**GROUND SURFACE**					
78.6 0.2	100mm **ASPHALTIC CONCRETE** 100mm **AGGREGATE**					
	SANDY SILT to SILT, some clay, compact to dense, brown to grey, moist ...grey below	1	SS	22	PID: 0 FID: 0	SS1 Analysis: Metals, PAH, VOC, PHC
		2	SS	33	PID: 15 FID: 0	
		3	SS	19	PID: 35 FID: 0	0 4 96 SS3 Analysis: Metals, PAH, VOC, PHC
76.5 2.3	**SILTY CLAY**, some sand to sandy, trace to some gravel, firm to very stiff, grey, moist (GLACIAL TILL)	4	SS	7	PID: 0 FID: 0	
		5	SS	17	PID: 0 FID: 0	
	...hard	6	SS	37	PID: 0 FID: 0	
73.8 5.0	**BEDROCK** (GEORGION BAY FORMATION)	7	SS	50 / 25mm	PID: 0 FID: 0	
73.3 5.5	**GEORGIAN BAY FORMATION** (See rock core log for details)	1	RUN			
		2	RUN			
		3	RUN			
		4	RUN			
67.9 10.9						

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Sep 18, 2025	5.8	73.0
Jun 16, 2026	5.9	72.9

ROCK CORE LOG 2

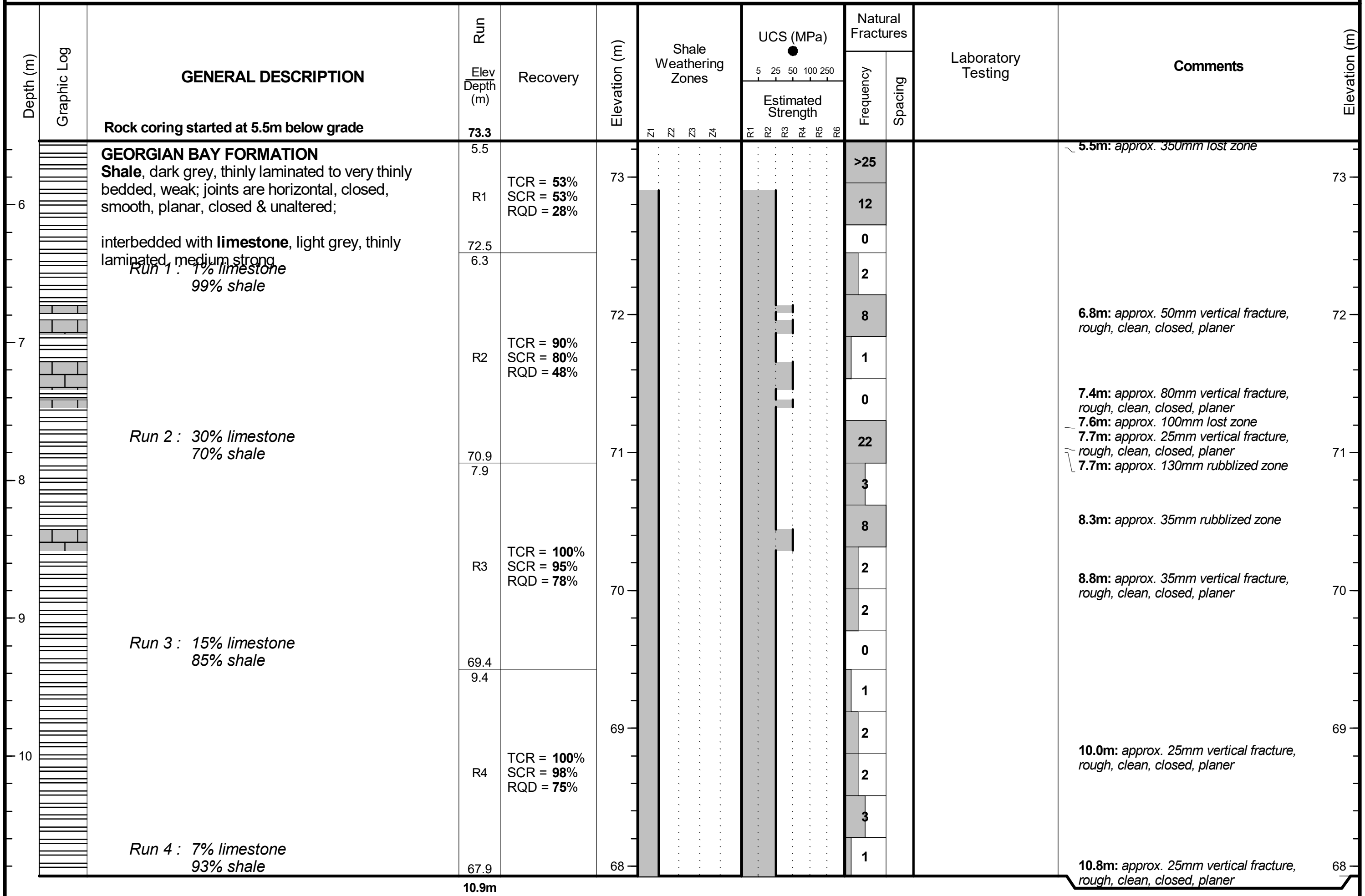
Project No. : 02211043.000
Client : Canahahns Company Limited
Originated by : SM

Date started : 2025 August 25
Project : 26 Park Street East
Compiled by : JZ

Sheet No. : 1 of 1
Location : Mississauga, Ontario
Checked by : SZ

Position : E: 614063, N: 4823371 (UTM 17T)
Elevation Datum : Geodetic
Core Diameter : **HQ**, OD=96mm, ID=64mm

Rig type : Truck-mounted
Drilling Method : Solid stem augers, HQ rock coring



GENERAL DESCRIPTION

Rock coring started at 5.5m below grade

GEORGIAN BAY FORMATION
Shale, dark grey, thinly laminated to very thinly bedded, weak; joints are horizontal, closed, smooth, planar, closed & unaltered;

interbedded with **limestone**, light grey, thinly laminated, medium strong

Run 1 : 1% limestone
99% shale

Run 2 : 30% limestone
70% shale

Run 3 : 15% limestone
85% shale

Run 4 : 7% limestone
93% shale

END OF COREHOLE

Run	Elev / Depth (m)	Recovery
	73.3	
R1	5.5 72.5	TCR = **53%** SCR = **53%** RQD = **28%**
R2	6.3 70.9	TCR = **90%** SCR = **80%** RQD = **48%**
R3	7.9 69.4	TCR = **100%** SCR = **95%** RQD = **78%**
R4	9.4 67.9	TCR = **100%** SCR = **98%** RQD = **75%**
	10.9m	

Natural Fractures – Frequency: >25, 12, 0, 2, 8, 1, 0, 22, 3, 8, 2, 2, 0, 1, 2, 2, 3, 1

Comments

5.5m: *approx. 350mm lost zone*

6.8m: *approx. 50mm vertical fracture, rough, clean, closed, planer*

7.4m: *approx. 80mm vertical fracture, rough, clean, closed, planer*
7.6m: *approx. 100mm lost zone*
7.7m: *approx. 25mm vertical fracture, rough, clean, closed, planer*
7.7m: *approx. 130mm rubblized zone*

8.3m: *approx. 35mm rubblized zone*

8.8m: *approx. 35mm vertical fracture, rough, clean, closed, planer*

10.0m: *approx. 25mm vertical fracture, rough, clean, closed, planer*

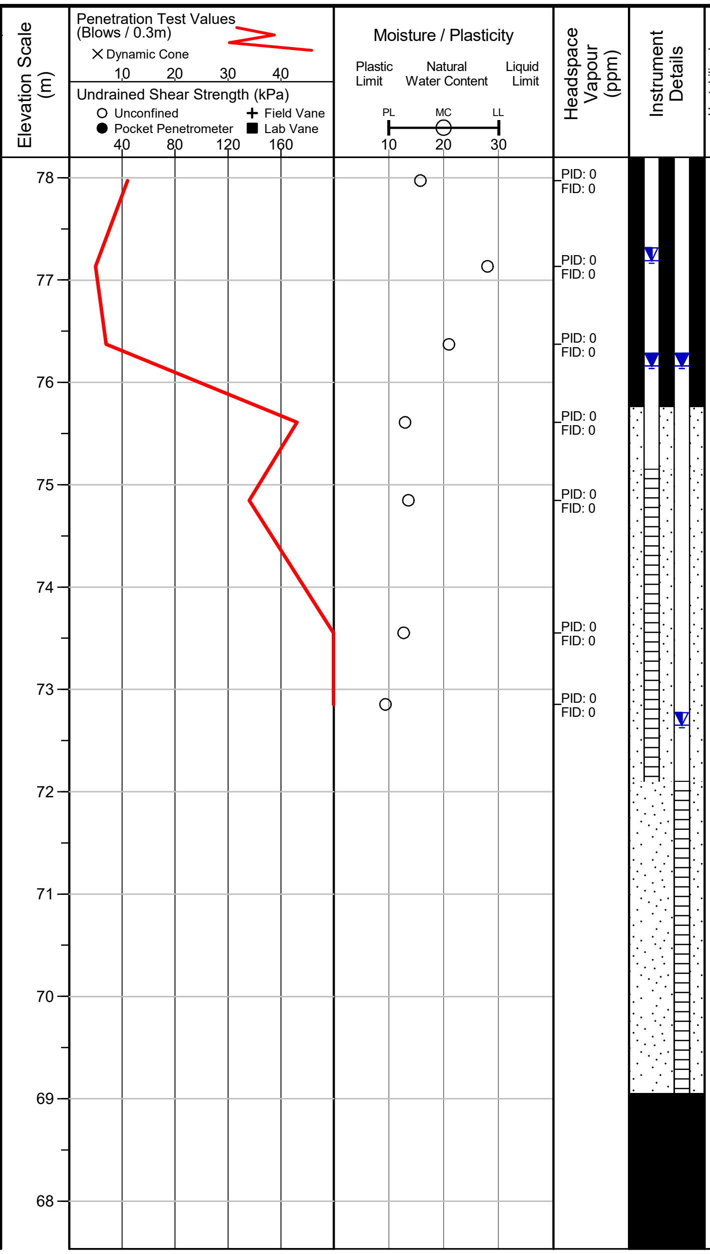
10.8m: *approx. 25mm vertical fracture, rough, clean, closed, planer*

LOG OF BOREHOLE 3

Project No. : 02211043.005
Client : Canahahns Company Limited
Originated by : SM
Date started : August 26, 2025
Project : 26 Park St E
Compiled by : JZ
Sheet No. : 1 of 1
Location : Mississauga, Ontario
Checked by : SZ

Position : E: 614064, N: 4823343 (UTM 17T)
Elevation Datum : Geodetic
Core Diameter : **HQ**, OD=96mm, ID=64mm
Rig type : Truck-mounted
Drilling Method : Solid stem augers, HQ rock coring

Elev Depth (m)	Description	Number	Type	SPT 'N' Value	Headspace Vapour (ppm)	Lab Data and Comments
78.2	**GROUND SURFACE**					
78.0 0.2	100mm **ASPHALTIC CONCRETE**	1	SS	11	PID: 0 FID: 0	
	100mm **AGGREGATE**					
	FILL, sandy silt, trace clay, trace organics, loose to compact, brown to dark grey, moist	2	SS	5	PID: 0 FID: 0	SS2 Analysis: Metals, PAH
76.7 1.5	**FILL**, clayey silt, some sand, trace gravel, trace rootlets, firm, brown, moist	3	SS	7	PID: 0 FID: 0	SS3 Analysis: VOC, PHC
75.9 2.3	**SILTY CLAY**, some sand to sandy, trace to some gravel, hard, brown to grey, moist (GLACIAL TILL) ...grey below	4	SS	43	PID: 0 FID: 0	SS4 Analysis: Metals, PAH, VOC, PHC
		5	SS	34	PID: 0 FID: 0	
73.6 4.6	**BEDROCK** (GEORGION BAY FORMATION)	6	SS	50 / 150mm	PID: 0 FID: 0	
		7	SS	50 / 25mm	PID: 0 FID: 0	
72.6 5.6	**GEORGIAN BAY FORMATION** (See rock core log for details)	1	RUN			
		2	RUN			
		3	RUN			
		4	RUN			
67.5 10.7						



Depth Scale (m): 0 – 10

GRAIN SIZE DISTRIBUTION (%) (USCS)
GR SA SI&CL

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

W1 WATER LEVELS

Date	Water Depth (m)	Elevation (m)
Sep 18, 2025	1.0	77.2
Jun 16, 2026	2.0	76.2

W2 WATER LEVELS

Date	Water Depth (m)	Elevation (m)
Sep 18, 2025	5.6	72.7
Jun 16, 2026	2.0	76.2

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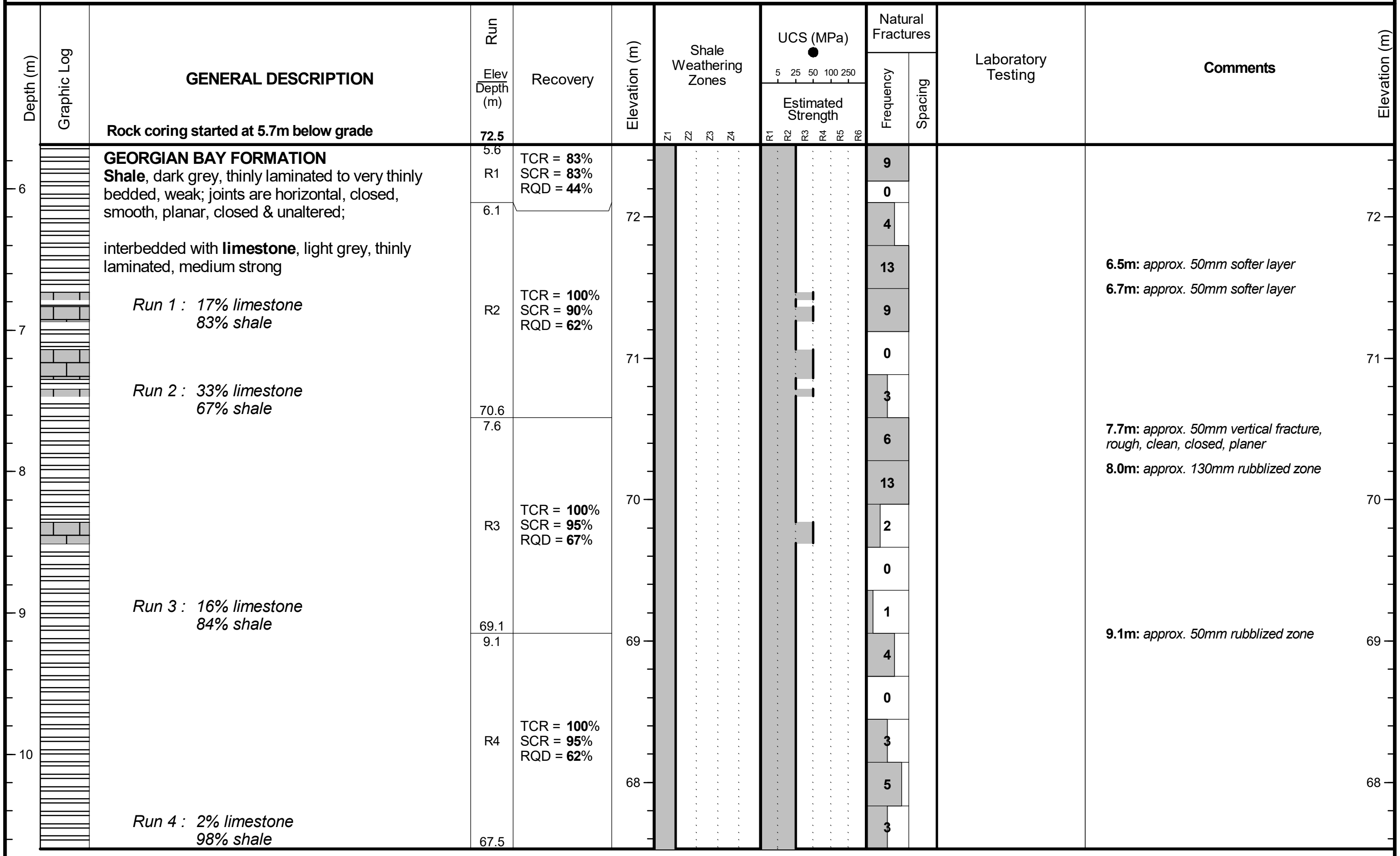
ROCK CORE LOG 3

Project No. : 02211043.000	Client : Canahahns Company Limited	Originated by : SM
Date started : 2025 August 26	Project : 26 Park Street East	Compiled by : JZ
Sheet No. : 1 of 1	Location : Mississauga, Ontario	Checked by : SZ

Position : E: 614064, N: 4823343 (UTM 17T)	Elevation Datum : Geodetic	Core Diameter : **HQ**, OD=96mm, ID=64mm
Rig type : Truck-mounted	Drilling Method : Solid stem augers, HQ rock coring	



GENERAL DESCRIPTION

Rock coring started at 5.7m below grade

GEORGIAN BAY FORMATION

Shale, dark grey, thinly laminated to very thinly bedded, weak; joints are horizontal, closed, smooth, planar, closed & unaltered;

interbedded with **limestone**, light grey, thinly laminated, medium strong

Run 1 : 17% limestone
83% shale

Run 2 : 33% limestone
67% shale

Run 3 : 16% limestone
84% shale

Run 4 : 2% limestone
98% shale

Run	Elev Depth (m)	Recovery
	72.5 5.6	
R1		TCR = **83**% SCR = **83**% RQD = **44**%
	6.1	
R2		TCR = **100**% SCR = **90**% RQD = **62**%
	70.6 7.6	
R3		TCR = **100**% SCR = **95**% RQD = **67**%
	69.1 9.1	
R4		TCR = **100**% SCR = **95**% RQD = **62**%
	67.5 **10.7m**	

Comments

6.5m: *approx. 50mm softer layer*

6.7m: *approx. 50mm softer layer*

7.7m: *approx. 50mm vertical fracture, rough, clean, closed, planer*

8.0m: *approx. 130mm rubblized zone*

9.1m: *approx. 50mm rubblized zone*

END OF COREHOLE

file: 02211043.001 - 26 park st e, mississauga bh logs.gpj

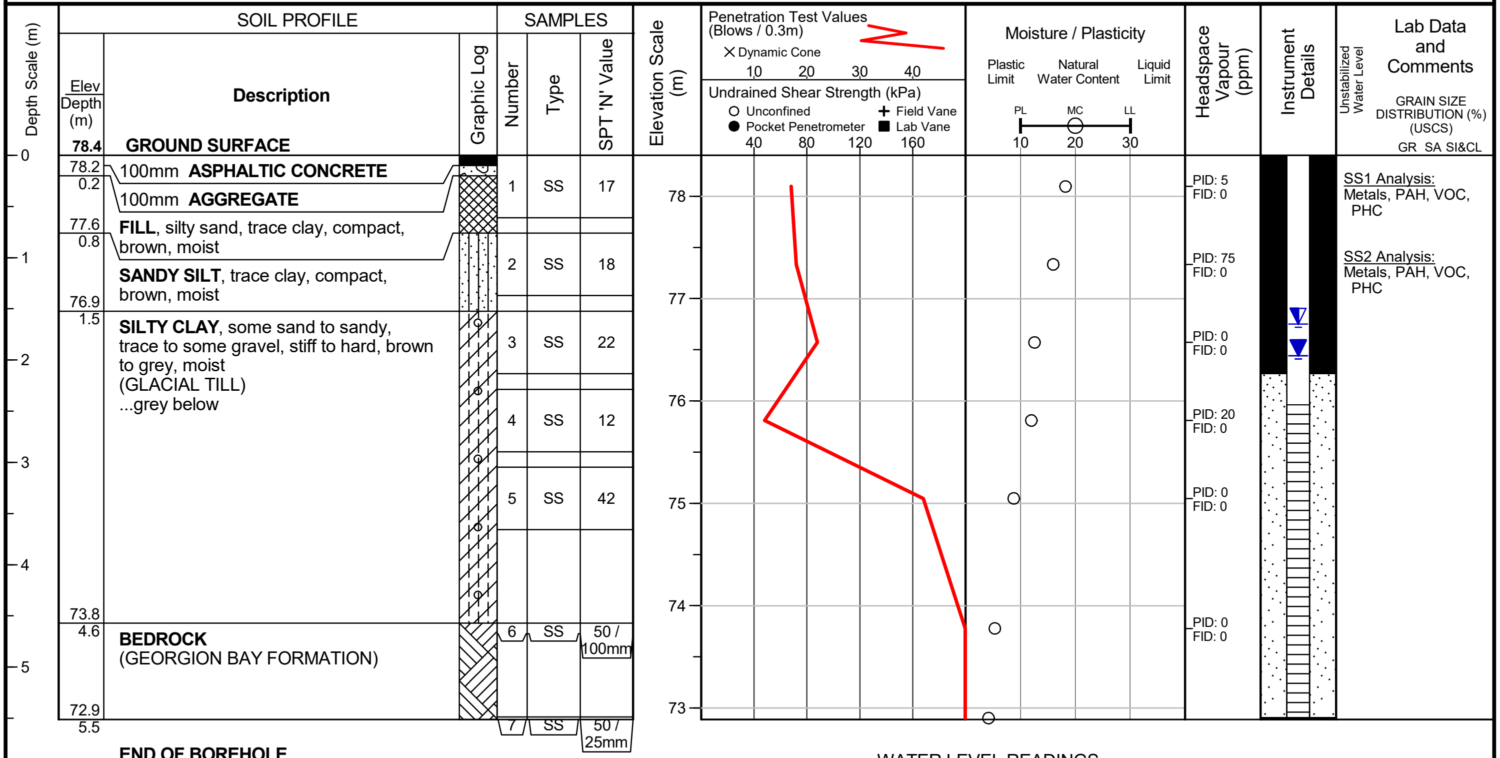
LOG OF BOREHOLE 4

Project No. : 02211043.005
Date started : August 26, 2025
Sheet No. : 1 of 1

Client : Canahahns Company Limited
Project : 26 Park St E
Location : Mississauga, Ontario

Originated by : SM
Compiled by : JZ
Checked by : SZ

Position : E: 614076, N: 4823358 (UTM 17T)
Rig type : Truck-mounted
Elevation Datum : Geodetic
Drilling Method : Solid stem augers



SOIL PROFILE

Elev Depth (m)	Description
78.4	**GROUND SURFACE**
78.2 0.2	100mm **ASPHALTIC CONCRETE**
	100mm **AGGREGATE**
77.6 0.8	**FILL**, silty sand, trace clay, compact, brown, moist
	SANDY SILT, trace clay, compact, brown, moist
76.9 1.5	**SILTY CLAY**, some sand to sandy, trace to some gravel, stiff to hard, brown to grey, moist (GLACIAL TILL) ...grey below
73.8 4.6	**BEDROCK** (GEORGION BAY FORMATION)
72.9 5.5	**END OF BOREHOLE**

SAMPLES

Number	Type	SPT 'N' Value	Headspace Vapour (ppm)
1	SS	17	PID: 5 FID: 0
2	SS	18	PID: 75 FID: 0
3	SS	22	PID: 0 FID: 0
4	SS	12	PID: 20 FID: 0
5	SS	42	PID: 0 FID: 0
6	SS	50 / 100mm	PID: 0 FID: 0
7	SS	50 / 25mm	

Lab Data and Comments

GRAIN SIZE DISTRIBUTION (%) (USCS)
GR SA SI&CL

SS1 Analysis:
Metals, PAH, VOC, PHC

SS2 Analysis:
Metals, PAH, VOC, PHC

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Sep 18, 2025	1.7	76.8
Jun 16, 2026	2.0	76.4

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

LOG OF BOREHOLE 5

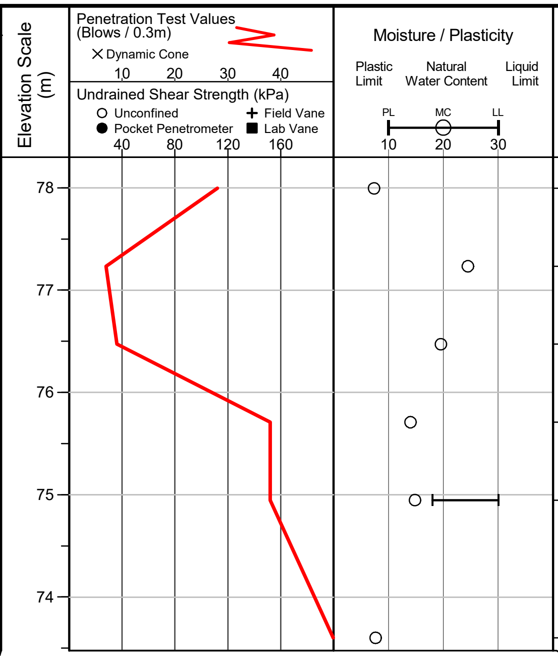
Project No. : 02211043.005
Client : Canahahns Company Limited
Originated by : SM
Date started : May 25, 2026
Project : 26 Park St E
Compiled by : JZ
Sheet No. : 1 of 1
Location : Mississauga, Ontario
Checked by : JZ

Position : E: 614028, N: 4823323 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers

Elev Depth (m)	Description
78.3	**GROUND SURFACE**
	80mm **ASPHALTIC CONCRETE**
77.7 0.6	520mm **AGGREGATE**
76.0 2.3	**FILL**, clayey silt to silty clay, some sand to sandy, trace gravel, firm to stiff, brown, moist
73.7 4.6	**SILTY CLAY**, some sand to sandy, trace gravel, hard, brown, moist (GLACIAL TILL)
73.5 4.8	**INFERRED BEDROCK (Bedrock of Georgian Bay Formation)**

Number	Type	SPT 'N' Value	Headspace Vapour (ppm)	Lab Data and Comments
1	SS	28	PID: 0 FID: 0	∇
2	SS	7	PID: 0 FID: 0	SS2 Analysis: Metals, PAH, VOC, PHC
3	SS	9	PID: 0 FID: 0	
4	SS	38	PID: 0 FID: 0	SS4 Analysis: Metals, PAH, VOC, PHC
5	SS	38	PID: 0 FID: 0	6 30 64
6	SS	50 / 100mm	PID: 0 FID: 0	

Lab Data and Comments: GRAIN SIZE DISTRIBUTION (%) (USCS) GR SA SI&CL



END OF BOREHOLE

Unstabilized water level measured at 0.6 m below ground surface; borehole was open upon completion of drilling.

LOG OF BOREHOLE 6

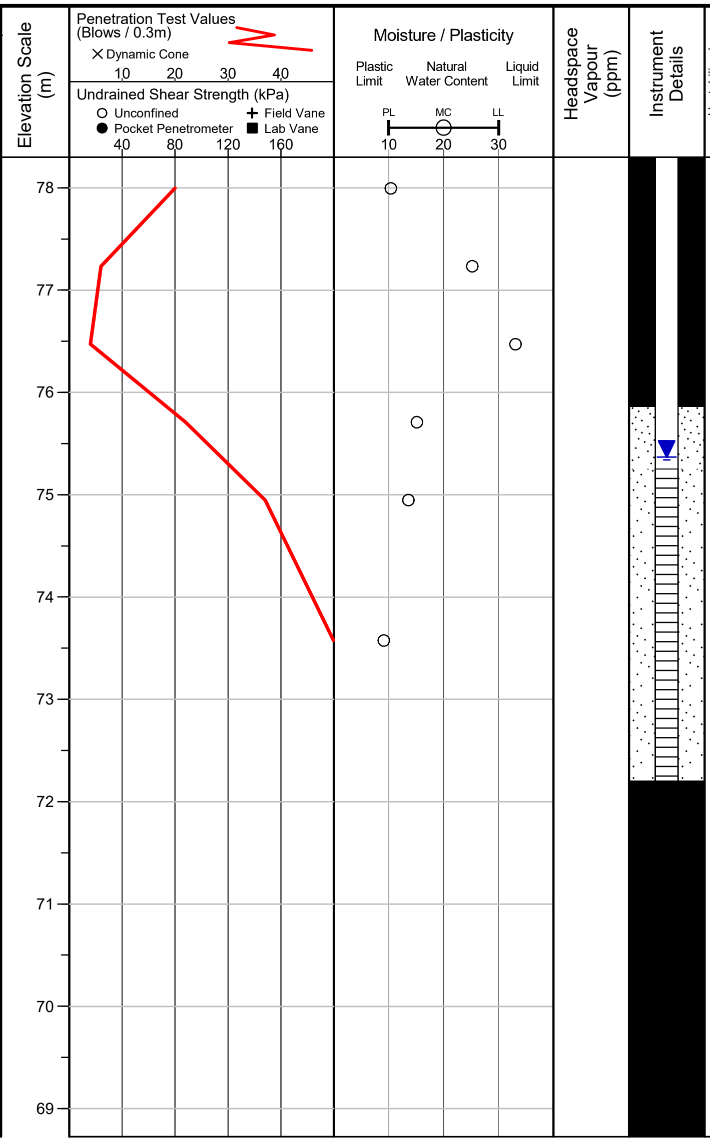
Project No. : 02211043.005
Date started : May 25, 2026
Sheet No. : 1 of 1

Client : Canahahns Company Limited
Project : 26 Park St E
Location : Mississauga, Ontario

Originated by : SM
Compiled by : JZ
Checked by : JZ

Position : E: 614039, N: 4823310 (UTM 17T)
Rig type : Track-mounted
Elevation Datum : Geodetic
Drilling Method : Solid stem augers

Elev Depth (m)	Description	Number	Type	SPT 'N' Value	Lab Data and Comments
78.3	**GROUND SURFACE**				
0.0	100mm **ASPHALTIC CONCRETE**	1	SS	20	
77.9 0.4	300mm **AGGREGATE**				
	FILL, silty clay, some sand, trace gravel, trace rootlets, soft to firm, dark grey, moist	2	SS	6	SS2 Analysis: PAH, VOC, PHC
		3	SS	4	
76.0 2.3	**SILTY CLAY**, some sand to sandy, trace gravel, very stiff to hard, brown, moist (GLACIAL TILL)	4	SS	22	SS4 Analysis: PAH, VOC, PHC
		5	SS	37	
73.7 4.6	**BEDROCK** **(Georgian Bay Formation)**	6	SS	50 / 150mm	
73.3 5.0	**GEORGIAN BAY FORMATION** (See rock core log for details)	1	RUN		
		2	RUN		
		3	RUN		
68.7 9.6					



GRAIN SIZE DISTRIBUTION (%) (USCS)
GR SA SI&CL

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Jun 16, 2026	2.9	75.4

ROCK CORE LOG 6

Project No. : 02211043.005
Client : Canahahns Company Limited
Originated by : SM

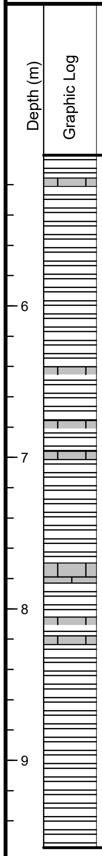
Date started : May 25, 2026
Project : 26 Park St E
Compiled by : JZ

Sheet No. : 1 of 1
Location : Mississauga, Ontario
Checked by : JZ

Position : E: 614039, N: 4823310 (UTM 17T)
Elevation Datum : Geodetic

Rig type : Track-mounted
Drilling Method : Solid stem augers

Rock coring started at 5.0m below grade



GEORGIAN BAY FORMATION
Shale, dark grey, thinly laminated to very thinly bedded, weak; joints are horizontal, closed, smooth, planar, closed & unaltered;

interbedded with **limestone**, light grey, thinly laminated, medium strong

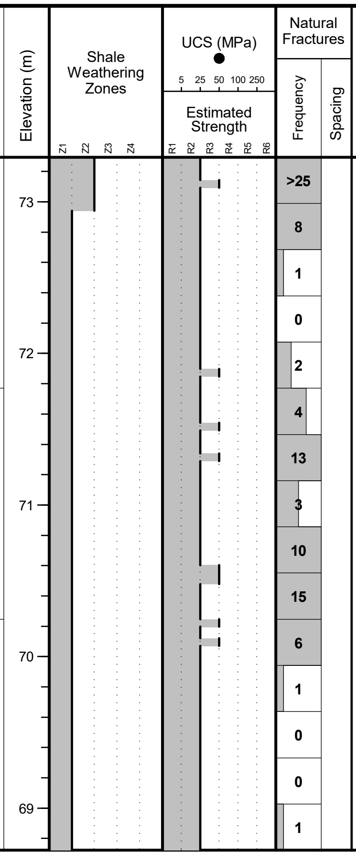
Run 1 : 7% limestone
93% shale

Run 2 : 25% limestone
75% shale

Run 3 : 8% limestone
92% shale

Run	Elev / Depth (m)	Recovery
	73.3 5.0	
R1	71.8 6.5	TCR = **100**% SCR = **80**% RQD = **68**%
R2	70.2 8.1	TCR = **100**% SCR = **68**% RQD = **40**%
R3	68.7 **9.6m**	TCR = **100**% SCR = **97**% RQD = **90**%

END OF COREHOLE



Laboratory Testing

Comments

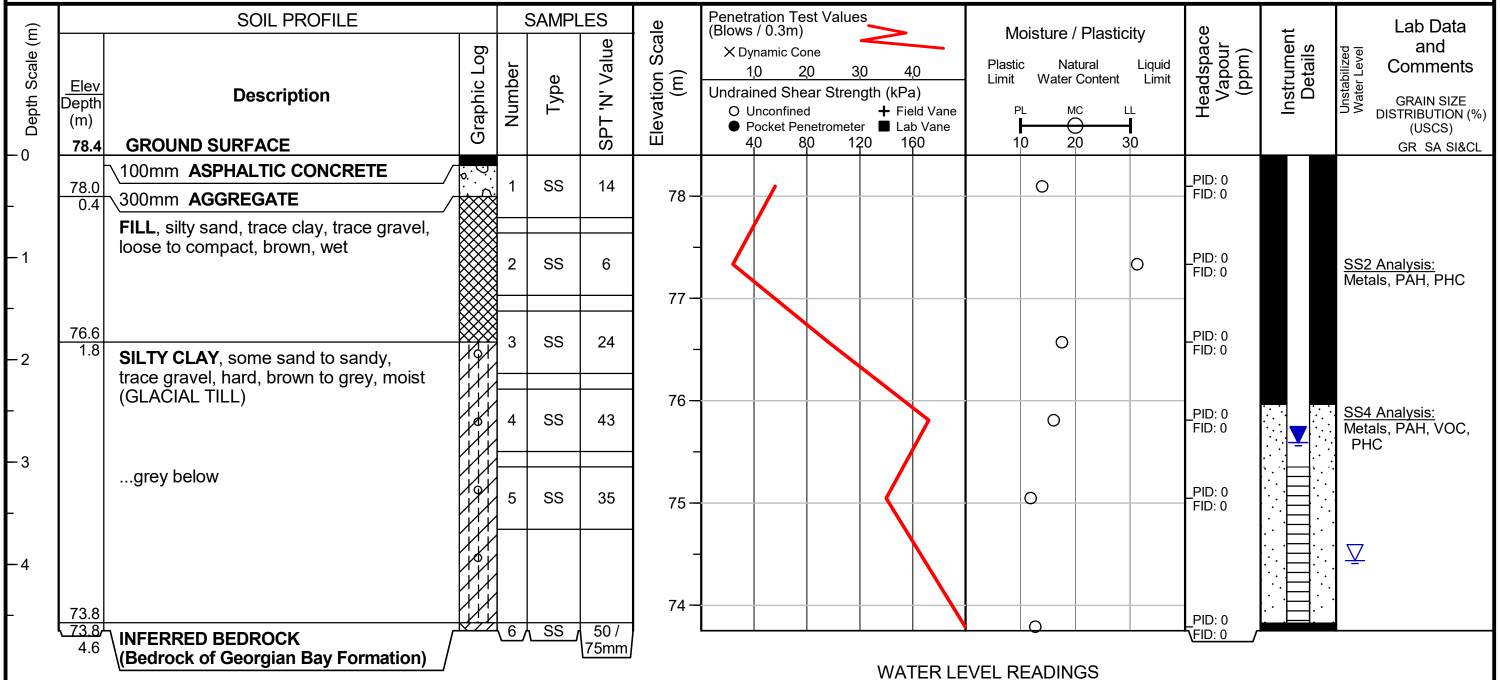
5.0m: *approx. 150mm long rubblized zone*
5.2m: *approx. 150mm long rubblized zone*
6.4m: *approx. 50mm long vertical fracture, rough, clean, closed, planer*
6.9m: *approx. 80mm long soft layer*
7.0m: *approx. 100mm long rubblized zone*
7.1m: *approx. 50mm long vertical fracture, rough, clean, closed, planer*
7.1m: *approx. 25mm long vertical fracture, rough, clean, closed, planer*
7.6m: *approx. 80mm long soft layer*
7.8m: *approx. 25mm long vertical fracture, rough, clean, closed, planer*
7.8m: *approx. 130mm long rubblized zone*
8.2m: *approx. 50mm long vertical fracture, rough, clean, closed, planer*
9.5m: *approx. 25mm long vertical fracture, rough, clean, closed, planer*

Elevation (m): 73, 72, 71, 70, 69

LOG OF BOREHOLE 7

Project No. : 02211043.005
Client : Canahahns Company Limited
Originated by : SM
Date started : May 26, 2026
Project : 26 Park St E
Compiled by : JZ
Sheet No. : 1 of 1
Location : Mississauga, Ontario
Checked by : JZ

Position : E: 614039, N: 4823339 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers



Elev Depth (m)	Description
78.4	**GROUND SURFACE**
	100mm **ASPHALTIC CONCRETE**
78.0 0.4	300mm **AGGREGATE**
	FILL, silty sand, trace clay, trace gravel, loose to compact, brown, wet
76.6 1.8	**SILTY CLAY**, some sand to sandy, trace gravel, hard, brown to grey, moist (GLACIAL TILL) ...grey below
73.8 73.8 4.6	**INFERRED BEDROCK (Bedrock of Georgian Bay Formation)**

Number	Type	SPT 'N' Value	Headspace Vapour (ppm)	Lab Data and Comments
1	SS	14	PID: 0 FID: 0	
2	SS	6	PID: 0 FID: 0	SS2 Analysis: Metals, PAH, PHC
3	SS	24	PID: 0 FID: 0	
4	SS	43	PID: 0 FID: 0	SS4 Analysis: Metals, PAH, VOC, PHC
5	SS	35	PID: 0 FID: 0	
6	SS	50 / 75mm	PID: 0 FID: 0	

END OF BOREHOLE

Unstabilized water level measured at 4.0 m below ground surface; borehole was open upon completion of drilling.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Jun 16, 2026	2.8	75.6

file: 02211043.005 26_park_st_e_ bh logs.gpj

LOG OF BOREHOLE 8

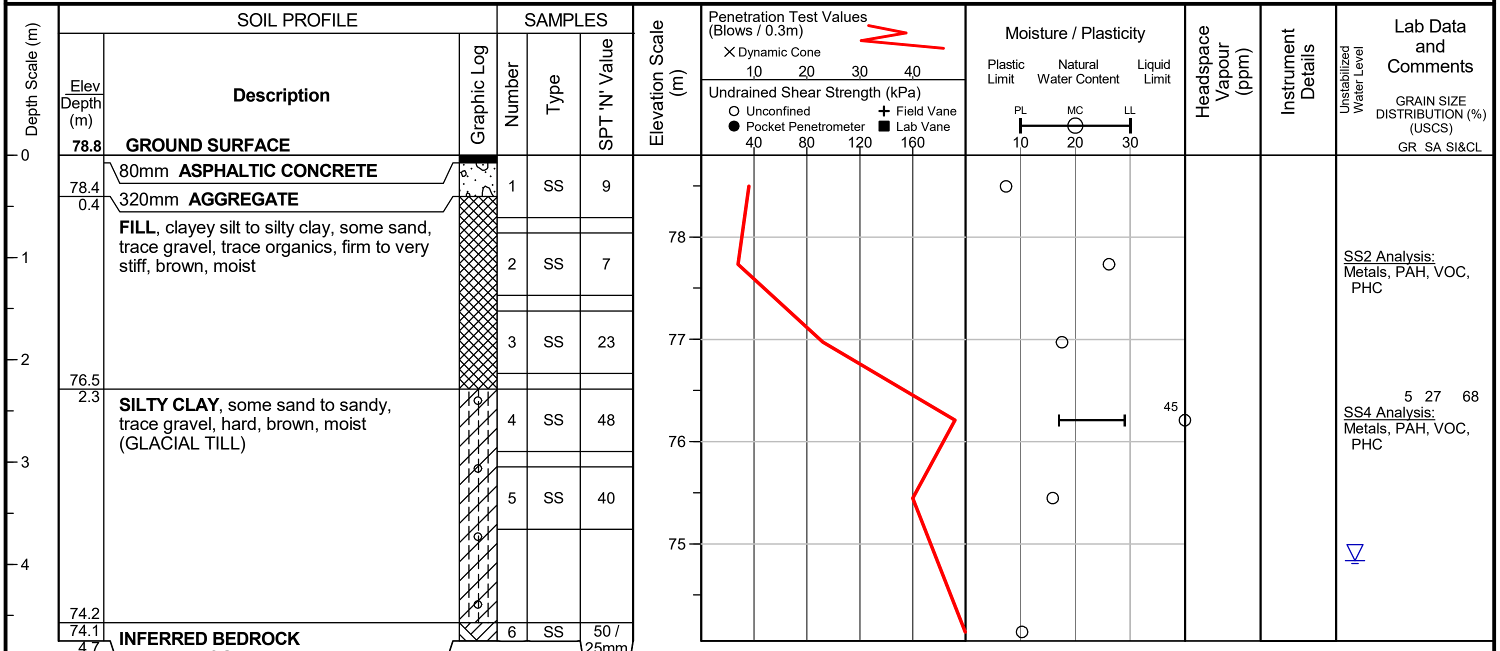
Project No. : 02211043.005
Client : Canahahns Company Limited
Originated by : SM
Date started : May 26, 2026
Project : 26 Park St E
Compiled by : JZ
Sheet No. : 1 of 1
Location : Mississauga, Ontario
Checked by : JZ

Position : E: 614051, N: 4823328 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers

Elev Depth (m)	SOIL PROFILE Description	SAMPLES Number	Type	SPT 'N' Value	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (USCS) GR SA SI&CL
78.8	**GROUND SURFACE**				
	80mm **ASPHALTIC CONCRETE**	1	SS	9	
78.4 0.4	320mm **AGGREGATE**				
	FILL, clayey silt to silty clay, some sand, trace gravel, trace organics, firm to very stiff, brown, moist	2	SS	7	SS2 Analysis: Metals, PAH, VOC, PHC
		3	SS	23	
76.5 2.3	**SILTY CLAY**, some sand to sandy, trace gravel, hard, brown, moist (GLACIAL TILL)	4	SS	48	5 27 68 SS4 Analysis: Metals, PAH, VOC, PHC
		5	SS	40	
74.2					
74.1 4.7	**INFERRED BEDROCK** **(Bedrock of Georgian Bay Formation)**	6	SS	50 / 25mm	

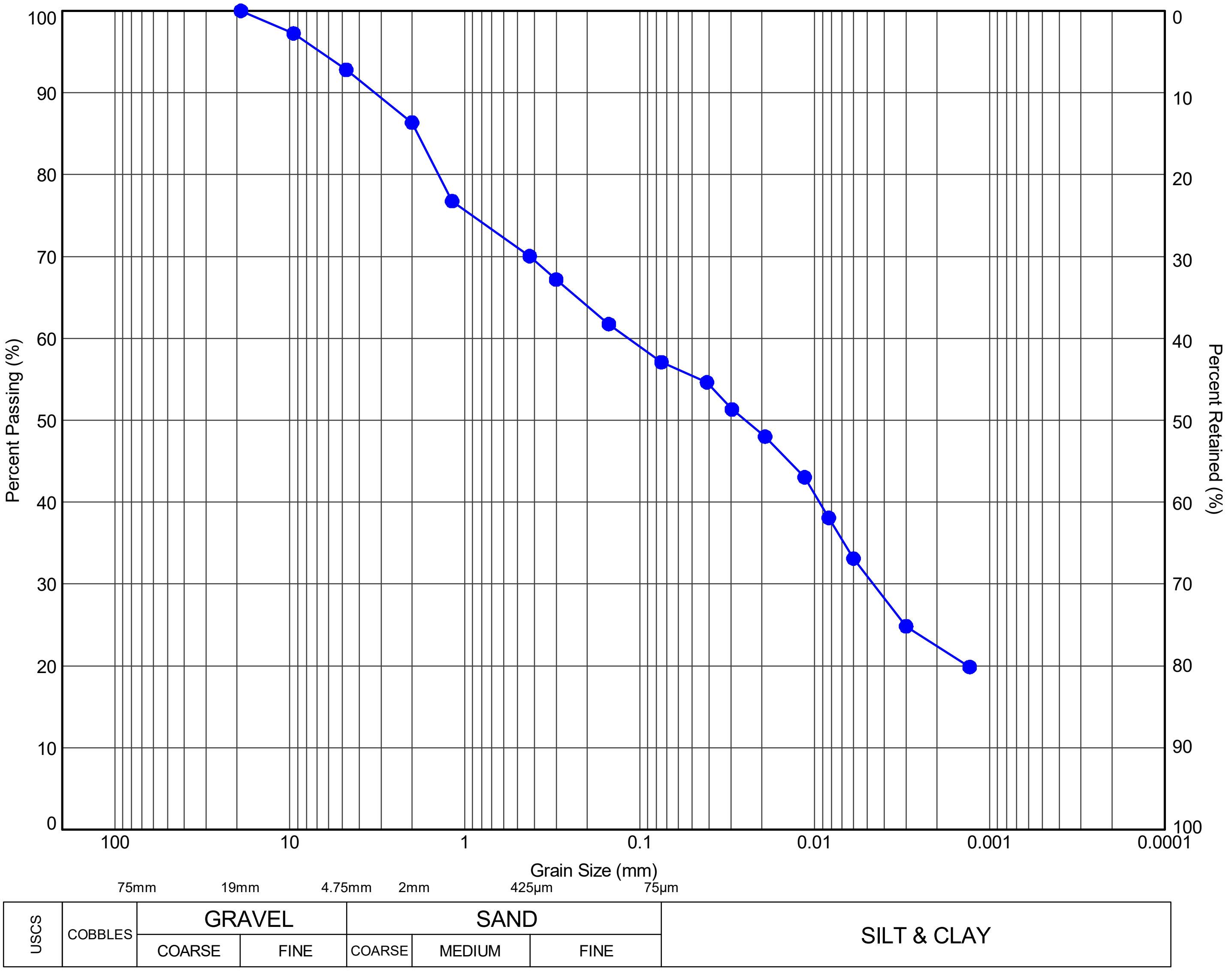
END OF BOREHOLE

Unstabilized water level measured at 4.0 m below ground surface; borehole was open upon completion of drilling.



Appendix B
Geotechnical Laboratory Test Results

englobe



UNIFIED SOIL CLASSIFICATION SYSTEM (%)

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel	Sand	Silt & Clay	USCS Description
●	**1**	SS5	3.4	75.2	7	36	57	

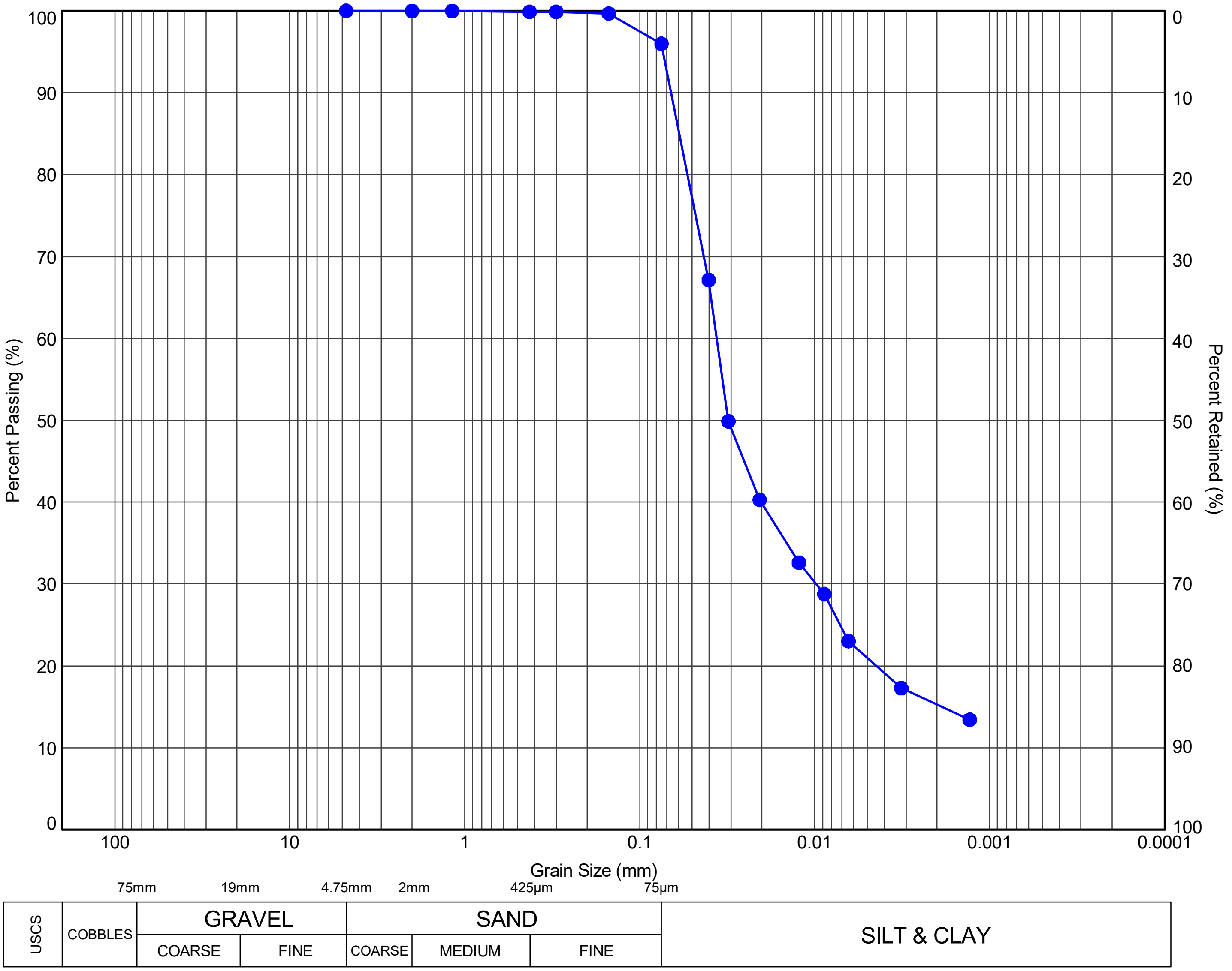
ENGLOBE

Title:

GRAIN SIZE DISTRIBUTION
SILTY CLAY, SANDY, TRACE GRAVEL

File No.:

02211043.000



UNIFIED SOIL CLASSIFICATION SYSTEM (%)

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel	Sand	Silt & Clay	USCS Description
●	**2**	SS3	1.8	77.0	0	4	96	

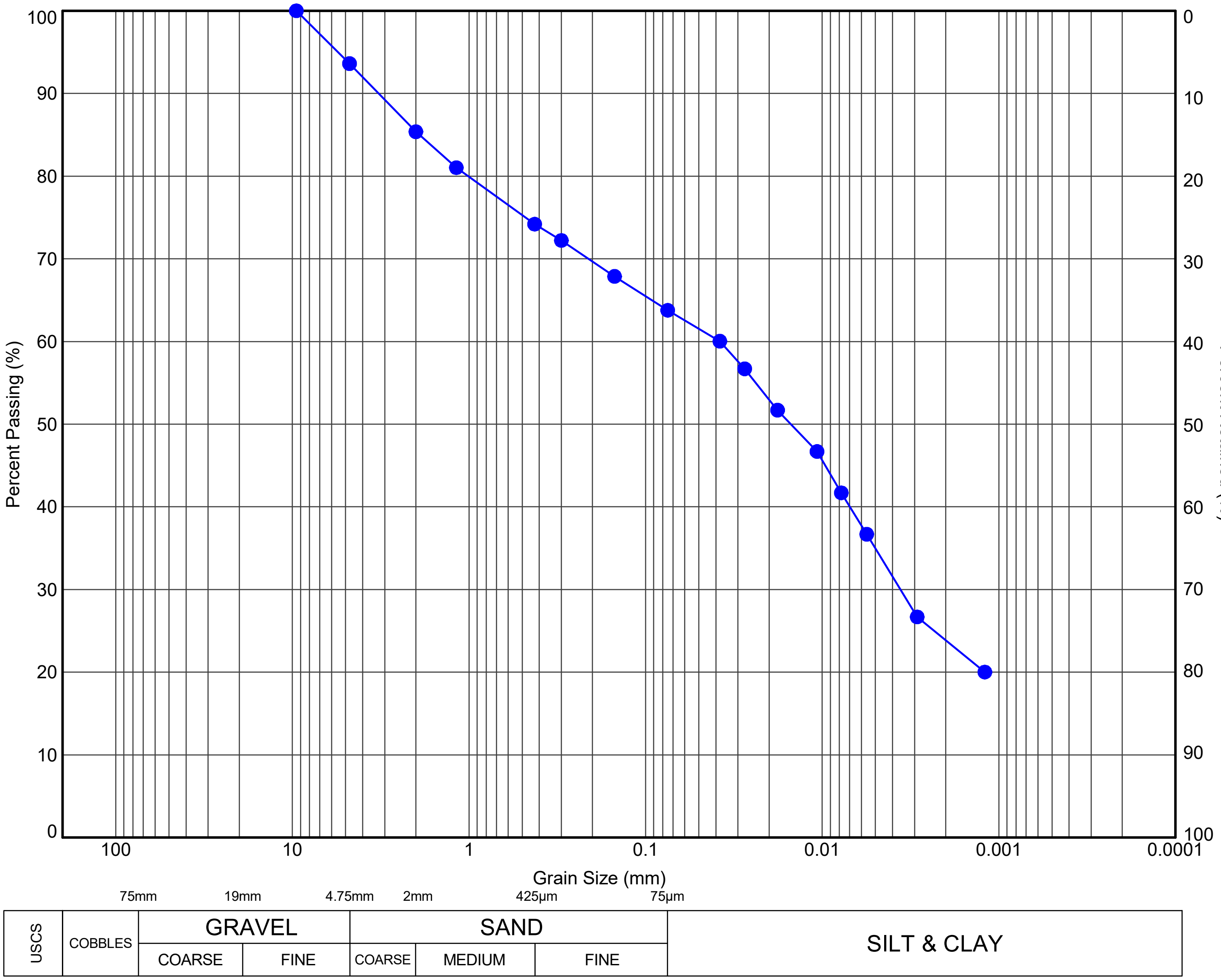
englobe

Title:

GRAIN SIZE DISTRIBUTION
SILT, SOME CLAY, TRACE SAND

File No.:

02211043.000



UNIFIED SOIL CLASSIFICATION SYSTEM (%)

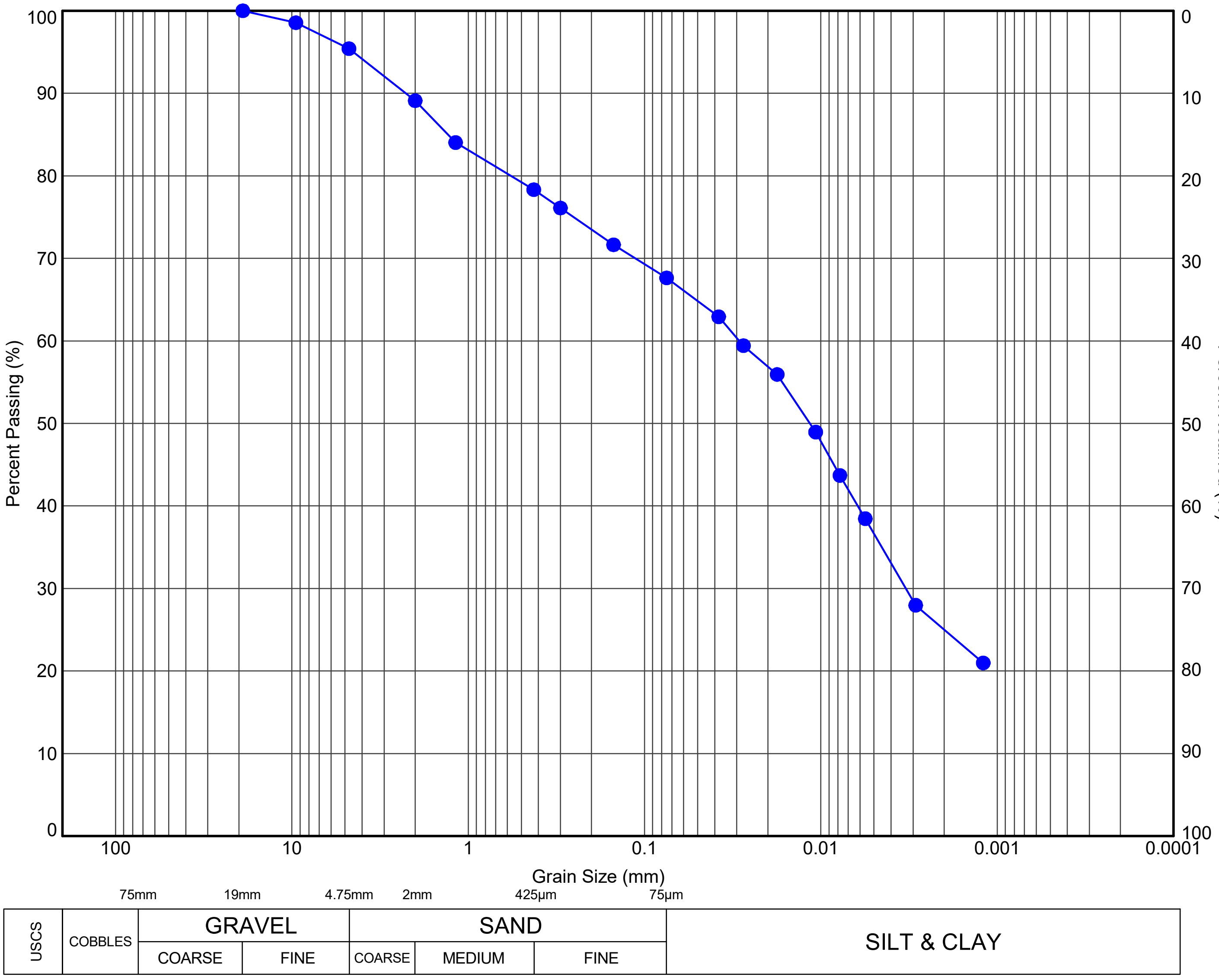
	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel	Sand	Silt & Clay
●	**5**	SS5	3.4	74.9	6	30	64

Title:

GRAIN SIZE DISTRIBUTION
SILTY CLAY, SANDY, TRACE GRAVEL

File No.:

02211043.005



UNIFIED SOIL CLASSIFICATION SYSTEM (%)

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel	Sand	Silt & Clay
●	**8**	SS4	2.6	76.2	5	27	68

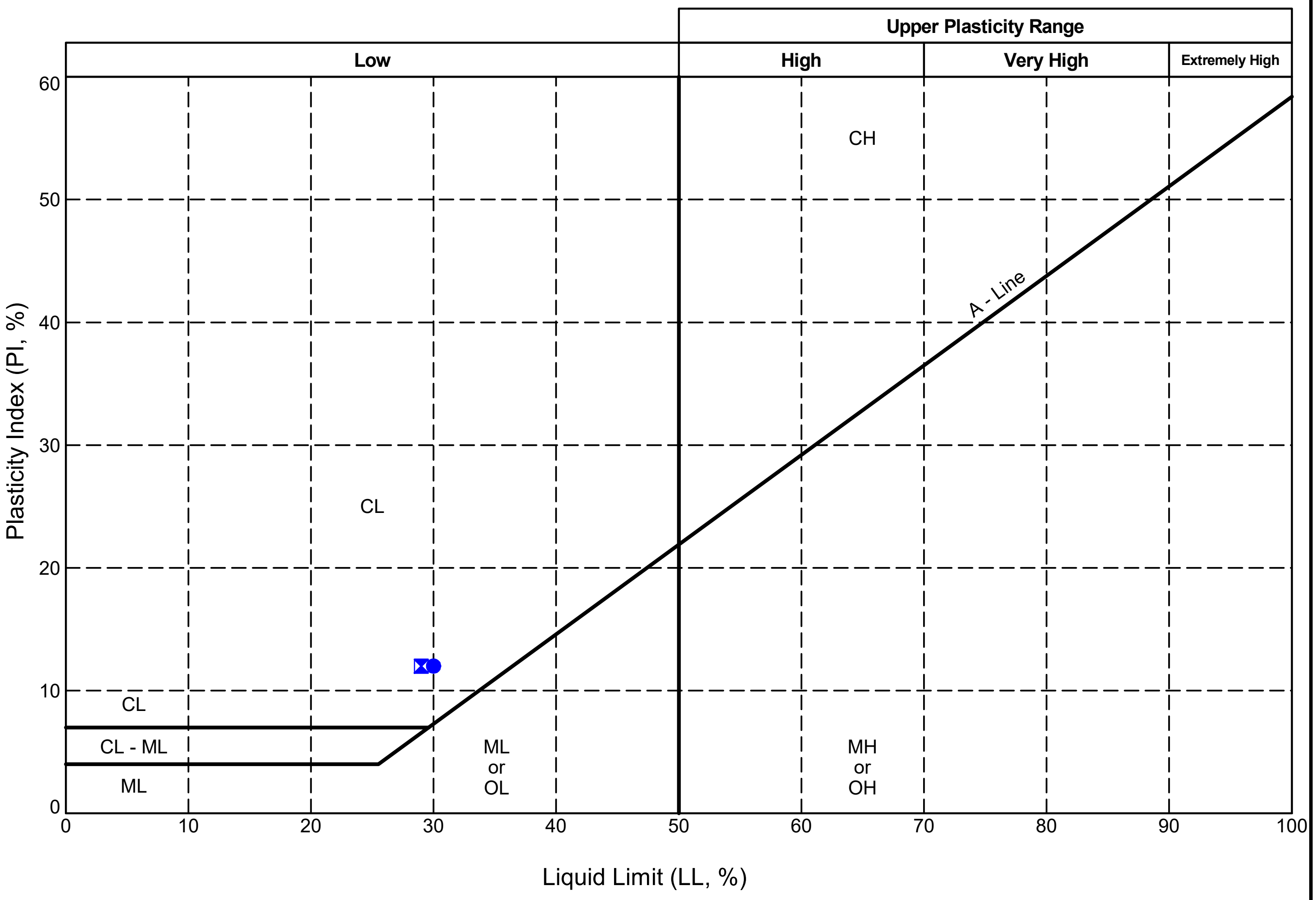
ENGLOBE

Title:

GRAIN SIZE DISTRIBUTION
SILTY CLAY, SANDY, TRACE GRAVEL

File No.:

02211043.005



	Borehole	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)	USCS Description
●	5	SS5	3.4	74.9	30	18	12	SLIGHTLY PLASTIC,
☒	8	SS4	2.6	76.2	29	17	12	SLIGHTLY PLASTIC,

ENGLOBE

Title:

ATTERBERG LIMITS CHART

File No.:

02211043.005

Appendix C
Rock Core Photographs

Project: 02211043.001 Borehole 2

Bedrock Core Sample (Box 1)
Borehole 2
Runs: 1, 2
Depth: 5.5 to 7.9 m below grade (Elev. 70.9 to Elev. 73.3 m)

Bedrock Core Sample (Box 2)
Borehole 2
Runs: 3, 4
Depth: 7.9 to 10.9 m below grade (Elev. 67.9 to Elev. 70.9 m)

Project: 02211043.001 Borehole 3

Bedrock Core Sample (Box 1)
Borehole 3
Runs: 1, 2
Depth: 5.6 to 7.6 m below grade (Elev. 70.6 to Elev. 72.6 m)

Bedrock Core Sample (Box 2)
Borehole 3
Runs: 3, 4
Depth: 7.6 to 10.7 m below grade (Elev. 67.5 to Elev. 70.6 m)

Project: 02211043.005 Borehole 6

Bedrock Core Sample (Box 1)
Borehole 6
Runs: 1, 2
Depth: 5.0 to 8.1 m below grade (Elev. 70.2 to Elev. 73.3 m)

Bedrock Core Sample (Box 2)
Borehole 6
Runs: 3
Depth: 8.1 to 9.6 m below grade (Elev. 68.7 to Elev. 70.2 m)