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A REPORT TO
CHELSEA ON THE GREEN I LIMITED PARTNERSHIP

A GEOTECHNICAL INVESTIGATION FOR PROPOSED CONDOMINIUM DEVELOPMENT WITH 2 TO 3-LEVEL UNDERGROUND PARKING

4100 PONYTRAIL DRIVE

CITY OF MISSISSAUGA

REFERENCE NO. 2505-S117

AUGUST 2025
(REVISED)

DISTRIBUTION

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1.0 INTRODUCTION

In accordance with the written authorization from Mr. Colin Eden of Chelsea on the Green I Limited Partnership, a geotechnical investigation was carried out in 4100 Ponytail Drive, in the City of Mississauga.

The purpose of the investigation was to reveal the subsurface conditions and to determine the engineering properties of the disclosed soils for the design and construction of multiple residential condominiums with 2 to 3-level underground parking structures. The findings and resulting geotechnical recommendations are presented in this report.

2.0 SITE AND PROJECT DESCRIPTION

The City of Mississauga is situated on Halton till plain where drift beds onto a shale bedrock (Georgian Bay Formation) at shallow to moderate depths. In places, the drift has been partly eroded by Peel Ponding (glacial lake) and filled with lacustrine sand, silt and clay.

The subject property, encompasses an approximate area of 3.75 hectares, locates at the south side of Ponytail Drive and Rathburn Road East, in the City of Mississauga. The site currently consists of two 18-storey residential condominiums with a separate 2-level underground parking, constructed since 1981. The site gradient descends from west to east with minor undulation.

The proposed project consisted of 2 phases. According to the site plan, prepared by 4 Architecture Inc., dated May 2025, the Phase 1 of the development consists of a 25-storey high-rise condominium with a 2-level underground parking, connecting to the underground parking of the adjoining two existing buildings. From a conceptual level Phase 2 site plan, the Phase 2 of the development, which will be constructed in later dates, consists of three 20-storey residential condominiums with 3-level underground parking proposed at the southern portion of the site.

3.0 FIELD WORK

The field work, consisting of eight (8) sampled boreholes extending to depths of 10.7 to 15.3 m below grade, was completed between June 10 and 19, 2025. The locations of the boreholes are shown on Drawing No. 1.

The boreholes were advanced at intervals to the sampling depths by a track-mounted machine using solid augers and equipped with split spoon sampler for soil sampling and core barrels for rock coring. Standard Penetration Tests, using the procedures described on the enclosed “List of Abbreviations and Terms”, were performed at the sampling depths. The results are recorded as the Standard Penetration Resistance (or ‘N’ values) of the subsoil. The compactness of the cohesionless strata and the consistency of the cohesive strata are inferred from the ‘N’ values. Split-spoon samples were recovered for soil classification and laboratory testing.

‘HQ’ size (63 mm diameter) rock coring was carried out in four selected boreholes to assess the quality and soundness of the encountered bedrock. The quality of the rock has been assessed by applying the ‘Rock Quality Designation’ (RQD) classification, considering the total length of the recovered core pieces 10 cm or longer against the length of the core run. The results are expressed in percentages and are recorded on the Borehole Log.

Upon completion of drilling and sampling, monitoring wells were installed the selected boreholes to facilitate a hydrogeological assessment which will be presented in a separate cover. Three (3) pairs of nested wells were installed at Boreholes 1, 6 and 7. A suffix of ‘S’ or ‘D’, representing the shallow and deep wells, was used to differentiate the well depths at the nested well location. The depth and details of the monitoring wells are shown on the corresponding Borehole Logs.

The field work was supervised and the findings were recorded by a Geotechnical Technician. The geodetic elevation at each of the borehole locations was obtained using the Global Navigation Satellite System (GNSS).

4.0 SUBSURFACE CONDITIONS

The borehole findings indicate that beneath a topsoil veneer, pavement structure and earth fill, the site is underlain by the strata of glacial tills with a layer of sand in one of the boreholes, overlying the shale bedrock.

Detailed descriptions of the encountered subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 11, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2. The engineering properties of the disclosed soils are discussed herein.

4.1 Surface Cover

A veneer of topsoil, approximately 3 cm in thickness, was noted on Borehole 2. In Borehole 6, the ground surface is covered with 150 mm thick asphalt pavement, overlying a 200 mm thick granular fill. No topsoil or pavement structure was noted at the surface of the remaining boreholes.

4.2 Earth Fill

A layer of earth fill was contacted at the ground surface or underneath the pavement structure in all boreholes, extending to a depth of 0.8 to 4.0 m. The fill consists of silty clay or sand with gravel and occasional organic inclusions and cobbles.

The recorded ‘N’ values range from 7 to 55, with a median of 14 blows per 30 cm of penetration, showing that the fill was non-uniformly compacted. The natural water content value ranges from 3% to 32%, with a median of 13%, indicating the sample with relatively high clay content and/or presence of organics.

4.3 Glacial Tills

Native glacial till deposit, denoted as “Sandy Silt Till”, Silty Clay Till” and “Silty Sand Till”, predominated the native overburden above the bedrock in all boreholes. The tills were contacted below the earth fill layer at the depths ranging from 0.8 to 4.0 m below the prevailing ground surface.

Tactile examinations of the soil samples indicated that the till was slightly cemented to being cohesive in places where the till changes from a sand/silt till into a clay till. The glacial till consists of a random mixture of particle sizes ranging from clay to gravel; depending on the grain size distribution and visual examination of the soil samples. Occasional sand seams were also observed in the soil samples at various depths and locations. Grain size analyses were completed on 4 glacial till samples, and the results are plotted on Figures 12 and 13.

The recorded ‘N’ values, recorded in the number of blows per 30 cm of penetration, and natural water content of the glacial till samples are summarized in Table 1.

Table 1 - SPT 'N' Values and Natural Water Content of Glacial Tills

Soil Type	SPT ‘N’ Values	Natural Water Content (%)	Compactness/Consistency
Silty Sand Till	71 and 84	11 and 13	Very dense
Sandy Silt Till	17 to over 100 median 68	8 to 20 median 13	Compact to very dense, generally very dense
Silty Clay Till	12 to 24 median of 14	12 to 17 median 15	Stiff to very stiff, generally stiff

The engineering properties of glacial tills are listed below:

- Moderate to high frost susceptibility, depending on the percentage of fine particles, with high soil-adfreezing potential.
- The shear strength is derived from consistency and internal friction.
- In excavation, the tills will generally be stable in relatively steep cuts; however, the silt and sand seams or layers may slough under prolonged exposure.

4.4 Sand

A sand layer was contacted below the silty clay till in Borehole 1. It is fine to medium grained with a trace of silt. The recorded ‘N’ value is 41 per 30 cm of penetration, indicating the sand is dense in compactness. The natural water content value is 17%, indicating that trapped water is in the sand layer.

4.5 Shale

Shale bedrock was contacted below the depths ranging from 5.5 to 8.5 m, or below El. 126.9 to 131.4 m in all boreholes. The shale bedrock is dark grey in colour, indicating Georgian Bay formation. It is interbedded with occasional hard siltstone, limestone and dolomite bands. The presence of shale debris found in the overlying till renders it is difficult to delineate the actual surface of the bedrock.

The bedrock within the investigated depth can be penetrated by power-augering with some difficulty in grinding through the hard layers. The upper layer of the shale within the investigated depth is in a weathered condition, becoming sound with depth. The weathered condition extends to a depth of several metres below the interface of the bedrock.

The quality of the rock was assessed in Boreholes 1, 4, 6 and 7 through rock coring. The interface between weather and sound bedrock is at the depths ranging from 9.5 to 10.4 m below grade. The resulting RQD ranges from 0% to 86%, indicating the rock quality is very poor to excellent.

Five (5) rock specimens at the anticipated depths of foundation from Boreholes 1, 4, 6 and 7 were selected for Unconfined Compression Strength (UCS) test (in accordance with ASTM D7012) in the laboratory. The resulting compressive strength yielded from 15.6 to 21.7 MPa. The test results are presented in Figure 14.

The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil, except the laminated limy and sandy layers would remain as rock slabs. When excavating the sound shale, slight lateral displacement of the excavation walls is often experienced. This is due to the release of residual stress stored in the bedrock mantle and the swelling characteristic of the rock.

Infiltrated precipitation and groundwater from the overburden soils may permeate the fissures in the bedrock and, in places, will be under subterranean artesian pressure.

5.0 GROUNDWATER CONDITION

Groundwater levels were measured from the monitoring wells and the findings were summarized in Table 1.

Table 2 - Groundwater Levels from Monitoring Wells

BH/MW No.	**Ground El. (m)**	**Well Depth (m)**	**July 10, 2025**		**July 21, 2025**	
			Depth (m)	**El. (m)**	**Depth (m)**	**El. (m)**
1D	136.8	7.7	4.7	132.1	4.8	132.0
1S	136.8	5.8	4.5	132.3	4.4	132.4
4	135.3	12.3	4.1	131.2	4.1	131.2
6D	133.8	8.1	4.0	129.8	-	Dry
6S	133.8	5.5	4.7	129.1	-	Dry
7D	136.9	10.3	5.3	131.6	7.1	129.8
7S	136.8	6.9	4.9	131.9	5.1	131.7

Groundwater level from the monitoring wells was recorded at the depths ranging from 4.0 to 7.1 m below the prevailing ground surface, or between El. 129.1 m and El. 132.4 m, while BH/MW6D and 6S were dry on July 21, 2025. The groundwater level subject to seasonal fluctuations. Detailed groundwater condition within the investigated area will be discussed in the hydrogeological report, under separate cover.

6.0 **DISCUSSION AND RECOMMENDATIONS**

The borehole findings indicate that beneath a topsoil veneer, pavement structure and earth fill, the site is underlain by the strata of glacial tills with a layer of sand in one of the boreholes, overlying the shale bedrock contacted between El. 126.9 to 131.4 m.

Groundwater level from the monitoring wells was recorded at the depths ranging from 4.0 to 7.1 m below the prevailing ground surface, or between El. 129.1 m and El. 132.4 m, which is subject to seasonal fluctuations.

It is understood that Phase 1 of the development consists of a 25-storey high-rise condominium with a 2-level underground parking connected with the existing underground structures of the two existing buildings, while Phase 2 of the development consists of three 20-storey residential condominiums with 3-level underground parking proposed at the south portion of the site. The geotechnical findings warranting special consideration for the proposed development are presented below:

1. The bulk excavation of the 2 and 3-level underground parking, is estimated to be 6 to 7 m and 9 to 10 m below the existing grade, respectively, where very dense glacial till stratum, weather shale or sound bedrock is anticipated, which are suitable for the construction of conventional footings.
2. If higher bearing pressures are required, the foundation should be extended onto the sound shale bedrock.
3. Where safe sloped excavation is not feasible, a braced shoring system is necessary for the excavation and construction of the underground parking and building foundation.
4. Damp-proofing and perimeter drainage at the foundation level are necessary for the underground structure, connecting into the sump pit where water can be removed into the municipal sewer system.

The recommendations appropriate for the project are presented herein. One must be aware that the subsurface conditions may vary between boreholes. Should this become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

6.1 Foundations

The bulk excavation of the 2- and 3-level underground parking, is estimated to be 6 to 7 m and 9 to 10 m below the existing grade, respectively. The sound till stratum or weathered shale below the foundation level are suitable for the construction of conventional footings.

For Phase 1 of the development with 2 levels of underground parking, the recommended bearing pressures at Serviceability Limit State (SLS) and Ultimate Limit State (ULS) for the design of conventional spread and strip footings are presented below:

- Maximum Allowable Bearing Pressure (SLS) = 600 kPa
- Factored Ultimate Bearing Presssure (ULS) = 900 kPa

For Phase 2 of the development with 3 levels of underground parking, the recommended bearing pressures for the design of conventional spread and strip footings are presented below:

- Maximum Allowable Bearing Pressure (SLS) = 900 kPa
- Factored Ultimate Bearing Presssure (ULS) = 1300 kPa

The total and differential settlements of footings founded on sound native soil or weathered shale, designing for the bearing pressure at SLS, are estimated to be 25 mm and 20 mm, respectively.

Where higher bearing pressure is required, the footings can be found on sound bedrock. The total and differential settlements of the footings on sound shale bedrock will be negligible. Thus, the foundation can be designed using the bearing pressure at ULS. The recommended bearing pressure for foundation found onto the sound bedrock is given below:

- Factored Ultimate Bearing Pressure (ULS) = 1500 kPa

As a result, an abrupt differential settlement may occur in the foundation if it is partially founded on sound shale and the soil or weathered rock. It is recommended that the structure should be designed for the abrupt differential settlement of around 20 mm. Otherwise, the foundation of the entire structure should be extended into the shale bedrock.

The foundation subgrade should be inspected by a geotechnical engineer to ensure that the revealed conditions are compatible with the foundation requirements.

Since perched water from the overburden may be draining into the excavation, lean mix concrete of 6 to 8 cm in thickness should be poured in the area of foundation once it is exposed and inspected. Where the foundation subgrade consists of shale, the lean mix concrete bedding is essential to prevent any disturbance which may lead to disintegration of shale.

Foundations exposed to weathering, or in unheated areas, should have at least 1.2 m of earth cover for protection against frost action. For unheated underground parking structure, having the entrance door closed at most of the time, the earth cover can be reduced to 0.6 m for the perimeter walls and 0.9 m for the interior walls and columns, except in the area in close proximity to ventilation shafts and the door entrances.

The building foundations must meet the requirements specified in the latest Ontario Building Code. Based on the shear wave velocity sounding test results, using the Multi-Channel Analysis of Surface Waves (MASW) method, the structure should be designed to resist an earthquake force using Site Classification 'B' (soft rock) or 'C' (very dense soil) depending on the finished grade of the lowest level. If the foundation is to be founded on the soil stratum and the thickness of the soil stratum between the building foundation and bedrock is more than 3 m, Site Classification 'C' should be considered. Detail of the testing method and the results are presented in the report from Geophysics GPR International Inc., attached in the Appendix 'A' of this report.

6.2 **Underground Structure**

The perimeter walls of the underground structure should be designed to sustain a lateral earth pressure calculated using the soil parameters stated in Section 6.7. Any applicable surcharge loads adjacent to the structure; the hydrostatic pressure and uplift forces must also be considered in the design of the underground structure.

The underground structure must be damp-proofing and provided with perimeter drainage system, connecting into the sump pit where water can be removed into the municipal sewer system. Prefabricated drainage board, such as Miradrain 6000 or equivalent, must be provided between the shoring wall and the cast-in-place foundation wall (Drawing No. 3).

The subgrade for conventional slab-on-grade construction should consist of sound native soil or well compacted earth fill. The slab should be constructed on a granular bedding of 20 cm thick, consisting of 19-mm Crusher-Run Limestone, or equivalent, compacted to 100% Standard Proctor Dry Density (SPDD). The elevator pit, which is usually below the slab-on-grade and the drainage system, should be designed as a submerged 'tank' structure with waterproofed pit walls and pit floor.

The exterior grade should slope away from the building to prevent ponding of water in the areas adjacent to the underground structure.

6.3 **Underground Services**

The subgrade for the underground services should consist of sound native soil or well compacted inorganic earth fill. A Class 'B' bedding, consisting of 19-mm Crusher-Run Limestone, or equivalent, is recommended for construction of the underground services. The bedding material should be compacted to 98% SPDD.

The pipe joints into the manholes and catch basins should be leak-proof or wrapped with an appropriate waterproof membrane to prevent subgrade migration. Openings to subdrains and catch basins should be shielded with a fabric filter to prevent blockage by silting.

In order to prevent pipe floatation when the underground services trench is deluged with water, a soil cover of at least the diameter of the pipe should be in place at all times after completion of the pipe installation.

All buried metal pipes and fittings must be protected against corrosion. In determining the mode of protection, an estimated electrical resistivity of the encountered soil type as disclosed from the Table 4·can be used. Where necessary, this can be confirmed by testing the soil along the service alignment at the time of construction.

6.4 **Trench Backfilling and Excavated Areas**

The on-site inorganic soils are generally suitable for use as trench backfill. They should be free of deleterious materials or oversized (over 15 cm) boulders. The lift of each backfill layer should be limited to a thickness of 20 cm.

The spoil from the shale bedrock will contain rock slabs and boulders, rendering it virtually impossible to obtain uniform compaction. The shale is considered unsuitable for trench backfill.

The backfill in the trenches and excavated areas should be compacted to at least 95% SPDD. In the zone within 1.0 m below the pavement subgrade at the entrance driveway, the material should be compacted to at least 98% SPDD with the water content 2% to 3% drier than the optimum. The lift of each backfill layer should either be limited to a thickness of 20 cm, or the thickness should be determined by test strips.

In normal construction practice, the problem areas of ground settlement largely occur adjacent to manholes, catch basins, service crossings, foundation walls and columns. In areas which are inaccessible to a heavy compactor, sand backfill should be used and compacted using light equipment.

6.5 **Pavement Design**

The recommended pavement design is presented in Table 2.

Table 3 - Pavement Design for On-Grade Access Driveway

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL3
Asphalt Binder	65	HL8
Granular Base	200	Granular 'A' or equivalent
Granular Sub-base	250	Granular 'B' or equivalent

Where the pavement is to be built on a structural slab, such as the rooftop of the underground structure, a sufficient granular base and adequate drainage must be provided to prevent frost damage to the pavement. A waterproof membrane must be placed above the structural slab exposed to weathering to prevent water leakage, as well as to protect the reinforcing steel bars against brine corrosion. The recommended pavement structure to be placed on the roof of the underground garage is presented in Table 3.

Table 4 - Pavement Design above Underground Structure

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL3
Asphalt Binder	50	HL8
Granular Base	200	19-mm Crusher-Run Limestone, or equivalent
Granular Sub-base	100	Free-Draining Sand Fill

It is imperative that the subgrade within the 1.0 m zone below the underside of the granular base be compacted to at least 98% SPDD, with the moisture content 2% to 3% drier than the optimum. This is to provide adequate stability for the pavement construction. The final subgrade should be proof-rolled; any soft spot as identified should be subexcavated and replaced by organic-free on-site material and recompacted properly.

All the granular bases should be compacted to 100% SPDD.

The pavement subgrade will suffer a strength regression if water is allowed to saturate the mantle. Along the perimeter where runoff may drain onto the pavement, swale or an intercept subdrain system should be installed to prevent infiltrating precipitation from seeping into the granular bases (since this may inflict frost damage on the flexible pavement). Subdrains consisting of filter-wrapped weepers should also be installed at the lower spots in the paved areas. The subdrains should be at a depth of 0.3 m below the subgrade, backfilled with free-draining granular material, connected into the catch basins or storm manholes.

6.6 Soil Parameters

The recommended soil parameters for the project design are presented in Table 4.

Table 5 - Soil Parameters

Unit Weight and Bulk Factor	**Bulk Unit Weight (kN/m^3)**	**Estimated Bulk Factor**	
		Loose	**Compacted**
Earth Fill	20.0	1.25	1.00
Silty Clay Till	22.0	1.33	1.03
Sandy Silt Till/Silty Sand Till	22.5	1.25	1.00
Sand	20.0	1.20	0.98
Shale	24.0	1.40	1.15
Lateral Earth Pressure Coefficients	**Active K_a**	**At Rest K_o**	**Passive K_p**
Compacted Earth Fill	0.36	0.53	2.76
Silty Clay Till	0.33	0.50	3.00
Sandy Silt Till/Silty Sand Till	0.30	0.47	3.25
Shale	0.17	0.29	5.83

Table 4 - Soil Parameters (cont'd)

Estimated Coefficients of Permeability (K) and Percolation Time (T)	**K (cm/sec)**	**T (min/cm)**
Silty Clay Till	10^{-7}	Over 80
Sandy Silt Till	10^{-5}	20
Silty Sand Till	10^{-4}	12
Sand	10^{-3}	8
Estimated Electrical Resistivities		
Silty Clay Till/Shale	3000 ohm.cm	
Sandy Silt Till/Silty Sand Till	4500 ohm.cm	
Sand	5500 ohm.cm	
Coefficients of Friction		
Between Concrete and Granular Base	0.50	
Between Concrete and Sound Native Soil	0.35	

6.7 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. The types of soils are classified in Table 5.

Table 6 - Classification of Soils for Excavation

Material	Type
Sound Shale Bedrock	1
Glacial Till, Weathered Shale	2
Earth Fill and Drained Sand	3
Saturated Sand	4

Where safe sloped excavation is not feasible, a braced shoring system will be required for the deep excavation. Any unsupported soil above the shoring must be sloped properly.

The design parameters for shoring design and our recommendations are provided in the Appendix 'B' in this report. Assessment of any adjacent building structure should be conducted. The overburden load and surcharge from any adjacent structures should be considered in the shoring design.

Excavation into the shale bedrock will require extra effort, a rock-ripper will be required to facilitate the excavation. The excavation will become progressively more difficult with depth into the sound shale. Efficient removal of the sound shale may require pneumatic hammering.

In sound rock excavation, a vertical cut is acceptable provided that the bedding plane is horizontal. Any loose rock protruding from the excavation must be removed for safety.

After removal of any protruding loose rock, an 80 to 100 mm thick spray foam is recommended on the rock face to prevent weathering and disintegration of rock during construction. In addition, the compressible foam can prevent the excessive pressure on the concrete wall due to the release of intact stress from sound bedrock.

Groundwater yield from the glacial tills is anticipated to be limited in quantity and slow in rate. The fissures from weathered rock may contain groundwater under artesian pressure and the yield can be initially moderate and persistent. Upon initial release, the water is expected to drain readily and stop after a short period of time. The subsurface water can be drained and removed by pumping from sumps in the excavation.

In order to optimize the effect of dewatering in excavation, we recommend that the installation of soldier piles for the shoring system should be carefully monitored to record the contact elevation of the water-bearing soil layers and the static water level. This information should be reviewed by the dewatering contractor and Soil Engineers Ltd. in order to determine the necessary extent of the dewatering system on site.

6.8 **Monitoring of Performance**

Close monitoring of vertical and lateral movement of the shoring wall should be carried out and frequent site inspections be conducted to ensure that the excavation does not adversely affect the structural stability of the adjacent buildings and the existing underground utilities. Extra bracing or support may be required if any movement is found excessive. The contractor should maintain the shoring to ensure any movement is within the design limit.

Vibration control and pre-construction survey is strongly recommended for the adjacent properties and structures prior to any excavation activities at the site. Our office can provide further advice or undertaking the vibration control and pre-construction survey as necessary.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of Chelsea on the Green I Limited Partnership, for review by the designated consultants, financial institutions, and government agencies. Use of this report is subject to the conditions and limitations of the contractual agreement.

The material in the report reflects the judgement of Daric Yang, P.Eng., and Kin Fung Li, P.Eng., and in light of the information available to it at the time of preparation. Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, is the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

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LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms commonly employed on the borehole logs and figures, and in the text of the report, are as follows:

SAMPLE TYPES

AS	Auger sample
CS	Chunk sample
DO	Drive open (split spoon)
DS	Denison type sample
FS	Foil sample
RC	Rock core (with size and percentage recovery)
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance or 'N' Value:

The number of blows of a 63.5 kg hammer falling from a height of 76 cm required to advance a 51 mm outer diameter drive open sampler 30 cm into undisturbed soil, after an initial penetration of 15 cm.

Plotted as '○'

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows per each 30 cm of penetration of a 51 mm diameter, 90° point cone driven by a 63.5 kg hammer falling from a height of 76 cm.

Plotted as '—•—'

WH	Sampler advanced by static weight
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
NP	No penetration

SOIL DESCRIPTION

Cohesionless Soils:

'N' (blows/30 cm)	Compactness
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
> 50	very dense

Cohesive Soils:

Undrained Shear Strength (kPa)	'N' (blows/30 cm)	Consistency
<12	<2	very soft
12 to <25	2 to <4	soft
25 to <50	4 to <8	firm
50 to <100	8 to <15	stiff
100 to 200	15 to 30	very stiff
>200	>30	hard

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0 Field vane test in borehole; the number denotes the sensitivity to remoulding

△ Laboratory vane test

METRIC CONVERSION FACTORS

1 ft = 0.3048 m
1 inch = 25.4 mm
1 lb = 0.454 kg
1 ksf = 47.88 kPa

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JOB NO.: 2505-S117

LOG OF BOREHOLE: 1D

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Residential Development with 2 or 3-Level Underground Parking

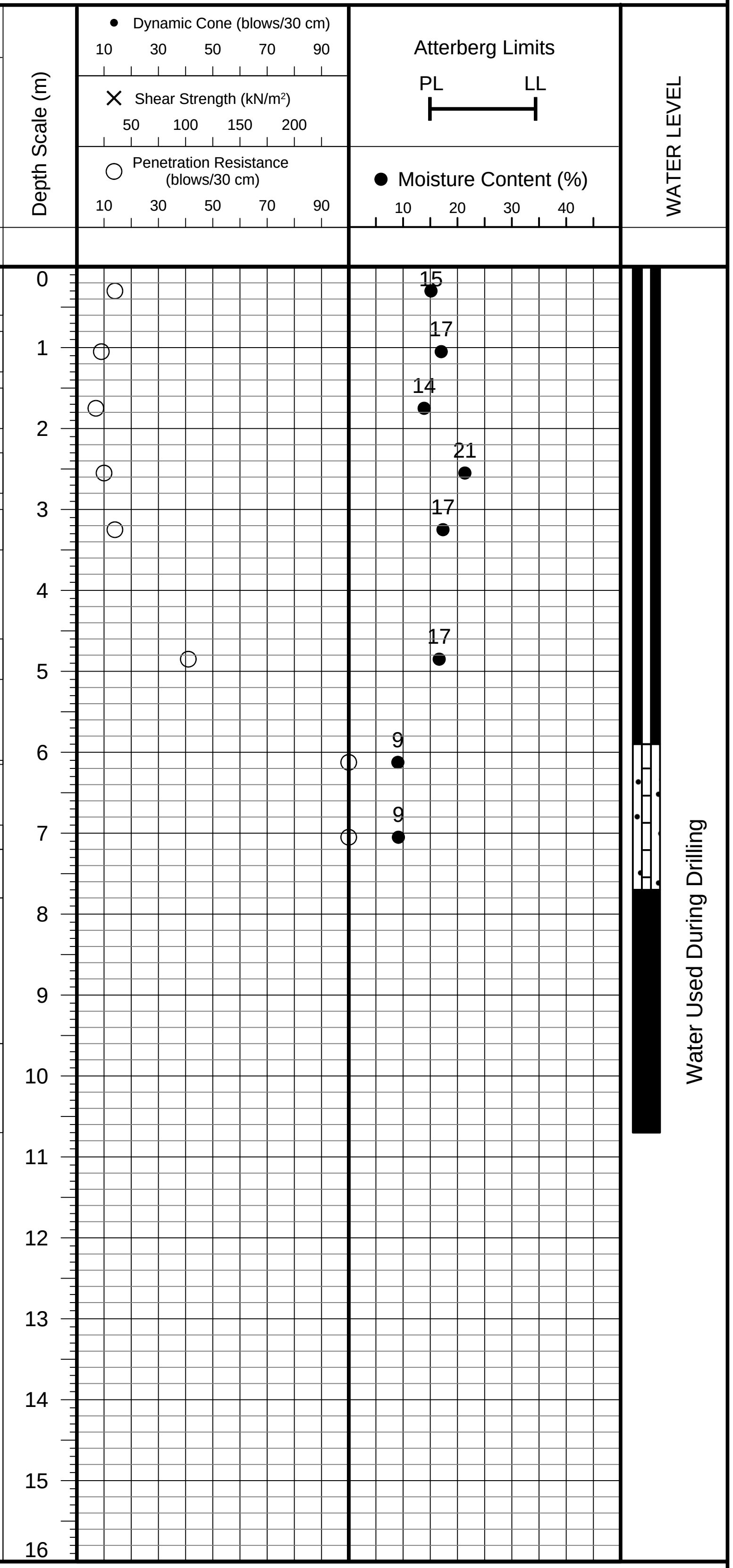
METHOD OF BORING: Hollow Stem Augers

PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

DRILLING DATE: June 13 and 16, 2025

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
136.8	Ground Surface			
0.0	Dark brown to grey **EARTH FILL** silty clay some sand occ. organics inclusion	1	DO	14
		2	DO	9
		3	DO	7
		4	DO	10
133.8 3.0	Brown, stiff **SILTY CLAY TILL** traces of sand and gravel	5	DO	14
132.8 4.0	Brown, dense, wet **SAND** fine to medium grained a trace of silt	6	DO	41
131.3 5.5	Grey **SHALE** occ. limestone seams _ _ _ weathered _ _	7	DO	50/5
		8	DO	50/13
		9	RC 100%	RQD 0%
		10	RC 100%	RQD 60%
		11	RC 100%	RQD 86%
126.1 10.7	**END OF BOREHOLE**			

Installed 50 mm Ø PVC monitoring well
to 7.7 m with 1.5 m screen
Bentonite seal backfill from 7.7 m to 10.7 m
Sand backfill from 6.2 to 7.7 m
Bentonite seal from 0.0 m to 6.2 m
Provided with a steel monument casing



Soil Engineers Ltd.

JOB NO.: 2505-S117

LOG OF BOREHOLE: 1S

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Residential Development
with 2 or 3-Level Underground Parking

METHOD OF BORING: Hollow Stem Augers

PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

DRILLING DATE: June 11 and 12, 2025

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	Depth Scale (m)
136.8	Ground Surface				
0.0	**Straight Auger to Intall The Nested Monitoring Well**				0
131.3 5.5	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 5.8 m completed with 1.5 m screen Sand backfill from 4.0 m to 5.8 m Bentonite seal from 0.0 m to 4.0 m Provided with a momument steel casing				

Depth Scale (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16

● Dynamic Cone (blows/30 cm) 10 30 50 70 90

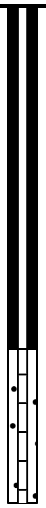
× Shear Strength (kN/m²) 50 100 150 200

○ Penetration Resistance (blows/30 cm) 10 30 50 70 90

Atterberg Limits PL LL

● Moisture Content (%) 10 20 30 40

WATER LEVEL



Soil Engineers Ltd.

JOB NO.: 2505-S117

LOG OF BOREHOLE: 2

FIGURE NO.: 3

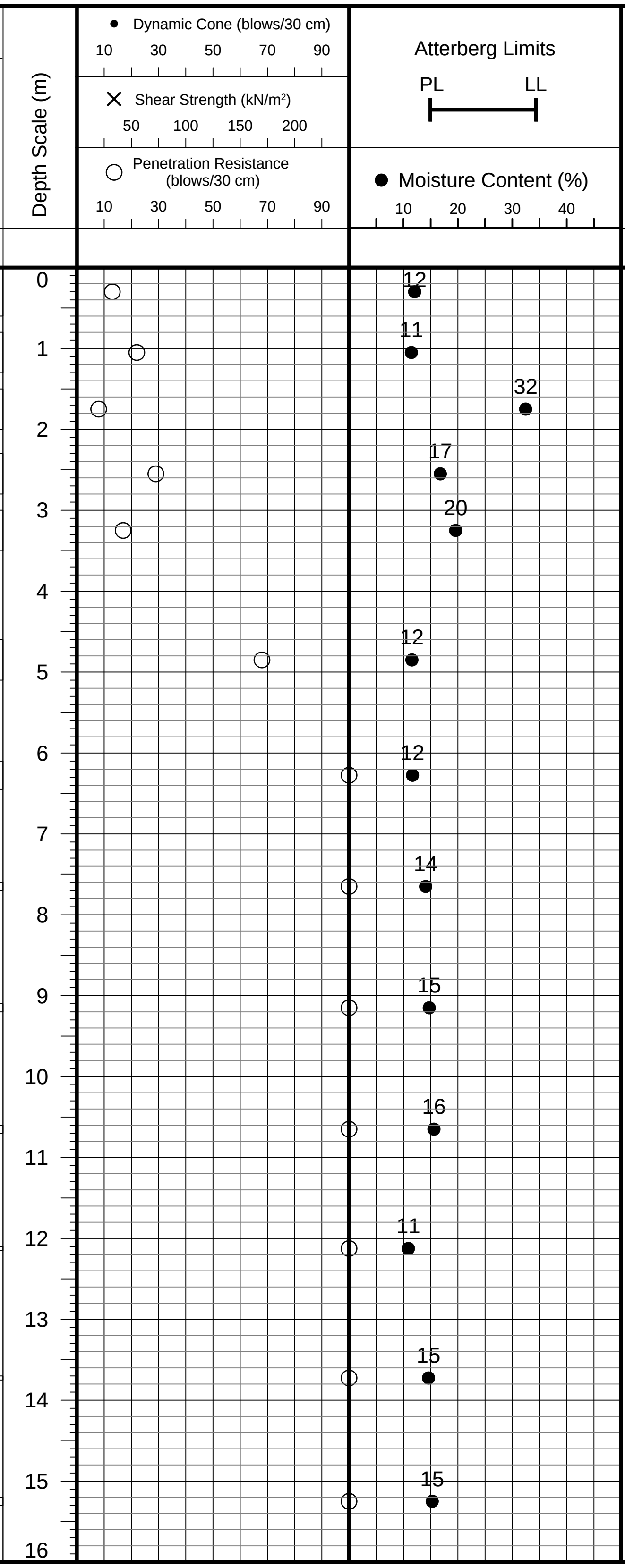
PROJECT DESCRIPTION: Proposed Residential Development with 2 or 3-Level Underground Parking

METHOD OF BORING: Hollow Stem Augers

PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

DRILLING DATE: June 12 and 13, 2025

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
136.8	Ground Surface			
0.0	**3 cm TOPSOIL**	1	DO	13
	Dark brown to grey **EARTH FILL** silty clay, some sand occ. organics inclusion	2	DO	22
		3	DO	8
		4	DO	29
133.8 3.0	Compact to very dense **SANDY SILT TILL** — — — — brown/grey traces of clay and gravel occ. sand seams and layers, cobbles and boulders	5	DO	17
		6	DO	68
		7	DO	82/20
129.8 7.0	Grey **SHALE** occ. limestone seams	8	DO	50/8
		9	DO	50/8
		10	DO	50/8
		11	DO	50/5
		12	DO	50/5
		13	DO	50/8
121.5 15.3	**END OF BOREHOLE**			



WATER LEVEL

Water Used During Drilling

Soil Engineers Ltd.

JOB NO.: 2505-S117

LOG OF BOREHOLE: 3

FIGURE NO.: 4

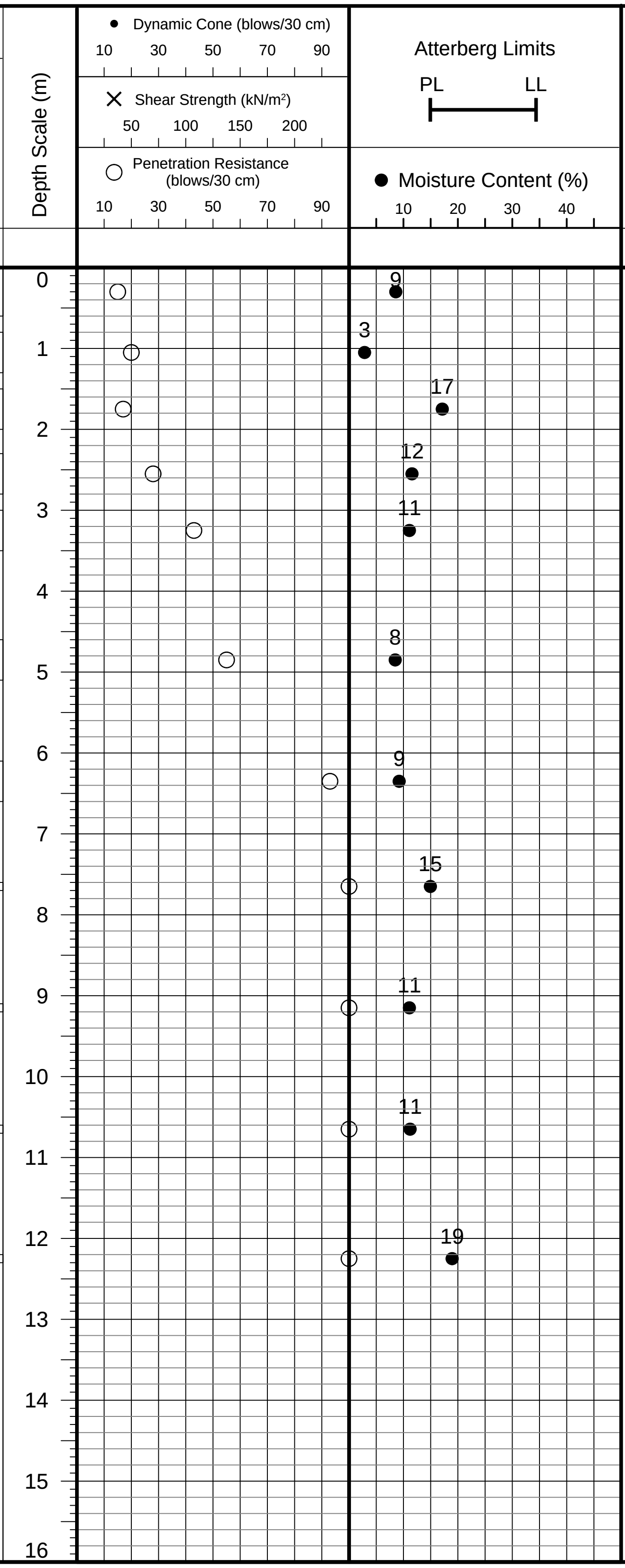
PROJECT DESCRIPTION: Proposed Residential Development with 2 or 3-Level Underground Parking

METHOD OF BORING: Hollow Stem Augers

PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

DRILLING DATE: June 16 and 17, 2025

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	WATER LEVEL
135.4	Ground Surface				
0.0	Dark brown to brown **EARTH FILL** sand occ. organics inclusion and cobbles and boulders clay	1	DO	15	Water Used During Drilling
		2	DO	20	
		3	DO	17	
		4	DO	28	
		5	DO	43	
131.4 4.0	Grey, very dense **SANDY SILT TILL** some sand traces of clay and gravel occ. sand seams and layers, cobbles and boulders	6	DO	55	
		7	DO	93	
		8	DO	50/10	
126.9 8.5	Grey **SHALE** occ. limestone seams	9	DO	50/8	
		10	DO	50/8	
		11	DO	50/5	
123.1 12.3	**END OF BOREHOLE**				



Soil Engineers Ltd.

JOB NO.: 2505-S117

LOG OF BOREHOLE: 4

FIGURE NO.: 5

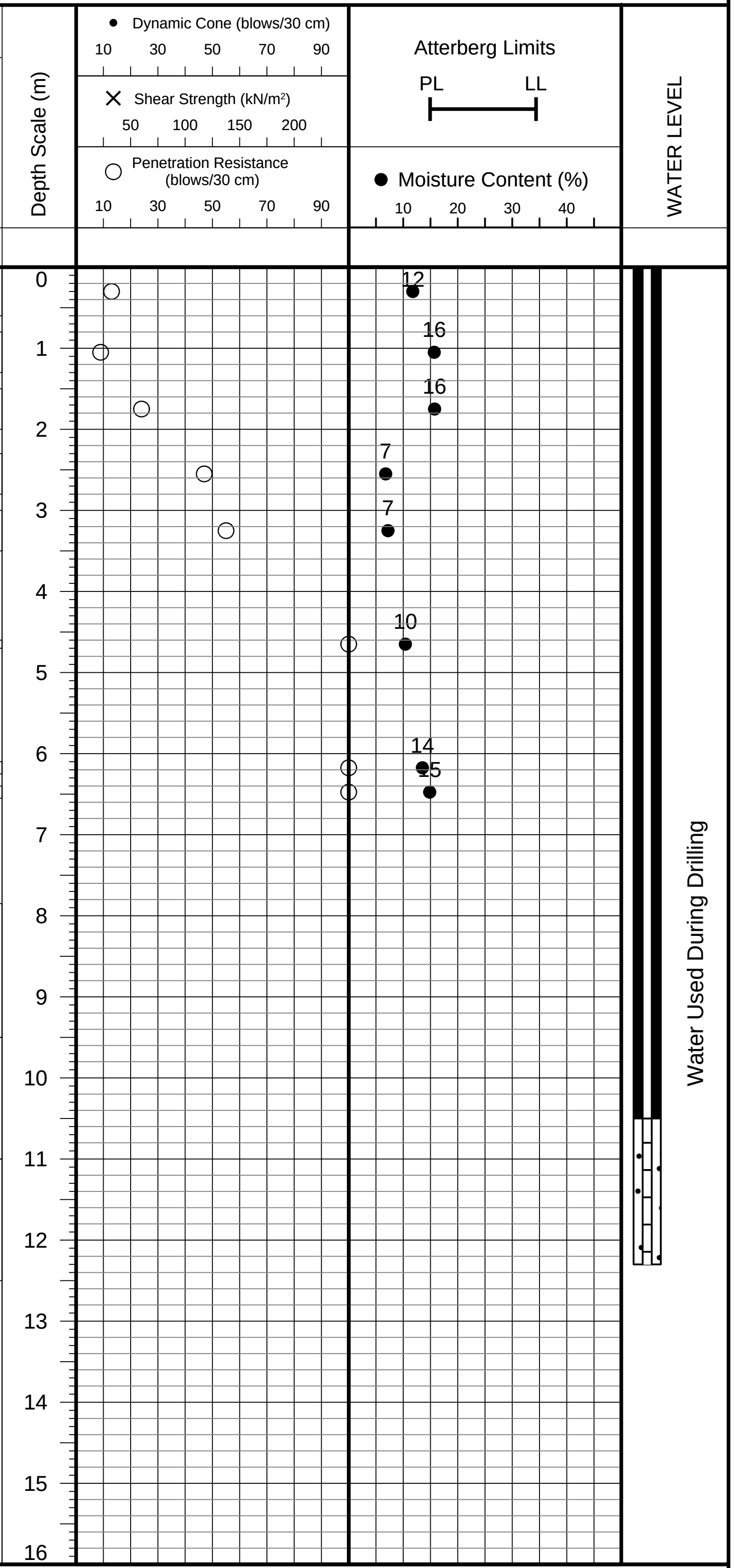
PROJECT DESCRIPTION: Proposed Residential Development
with 2 or 3-Level Underground Parking

PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

METHOD OF BORING: Hollow Stem Augers

DRILLING DATE: June 16 and 17, 2025

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
135.4	Ground Surface			
0.0	Dark brown/Brown **EARTH FILL** silty clay occ. organics inclusion occ. cobbles and boulders	1	DO	13
		2	DO	9
		3	DO	24
133.3 2.1	Brown **EARTH FILL** sand a trace of silt	4	DO	47
		5	DO	55
130.8 4.6	Grey, very dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams and layers, cobbles and boulders	6	DO	55/10
		7	DO	60/13
		8	DO	55/15
128.8 6.5	Grey _ _ _weathered **SHALE** occ. limestone seams	9	RC 100%	RQD 14%
		10	RC 100%	RQD 26%
		11	RC 100%	RQD 75%
122.9 12.5	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 12.3 m completed with 1.5 m screen Sand backfill from 10.5 m to 12.3 m Bentonite seal from 0.0 m to 10.5 m Provided with a momument steel casing			



Soil Engineers Ltd.

JOB NO.: 2505-S117

LOG OF BOREHOLE: 5

FIGURE NO.: 6

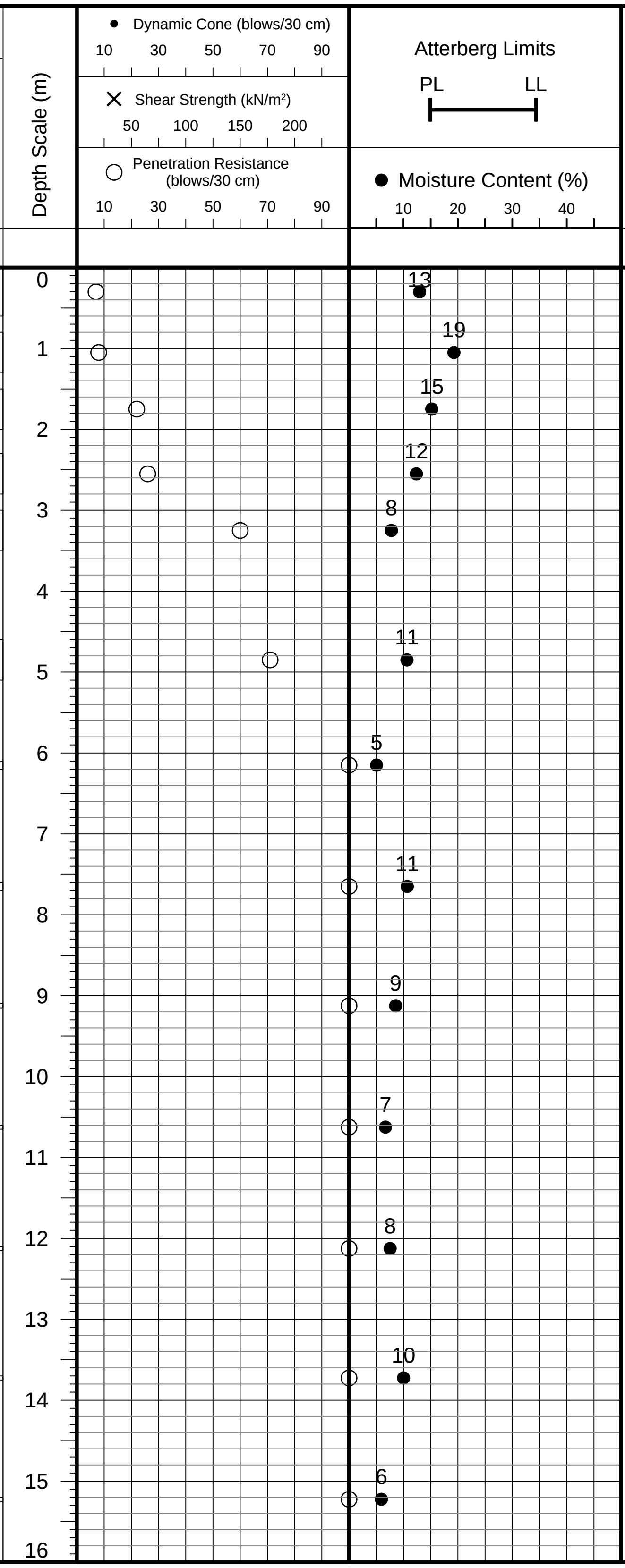
PROJECT DESCRIPTION: Proposed Residential Development with 2 or 3-Level Underground Parking

METHOD OF BORING: Hollow Stem Augers

PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

DRILLING DATE: June 10, 2025

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
134.2	Ground Surface			
0.0	Dark brown **EARTH FILL** sandy silt, some clay occ. organics inclusion	1	DO	7
		2	DO	8
		3	DO	22
		4	DO	26
131.2 3.0	Brown, very dense **SANDY SILT TILL**	5	DO	60
130.2 4.0	Grey, very dense, very moist **SILTY SAND TILL** traces of clay and gravel	6	DO	71
128.7 5.5	Grey **SHALE** occ. clay layers	7	DO	50/10
		8	DO	50/8
		9	DO	50/5
		10	DO	50/5
		11	DO	50/5
		12	DO	50/5
118.9 15.3	**END OF BOREHOLE**	13	DO	50/5



WATER LEVEL

Water Used During Drilling

Soil Engineers Ltd.

JOB NO.: 2505-S117

LOG OF BOREHOLE: 6D

FIGURE NO.: 7

PROJECT DESCRIPTION: Proposed Residential Development with 2 or 3-Level Underground Parking

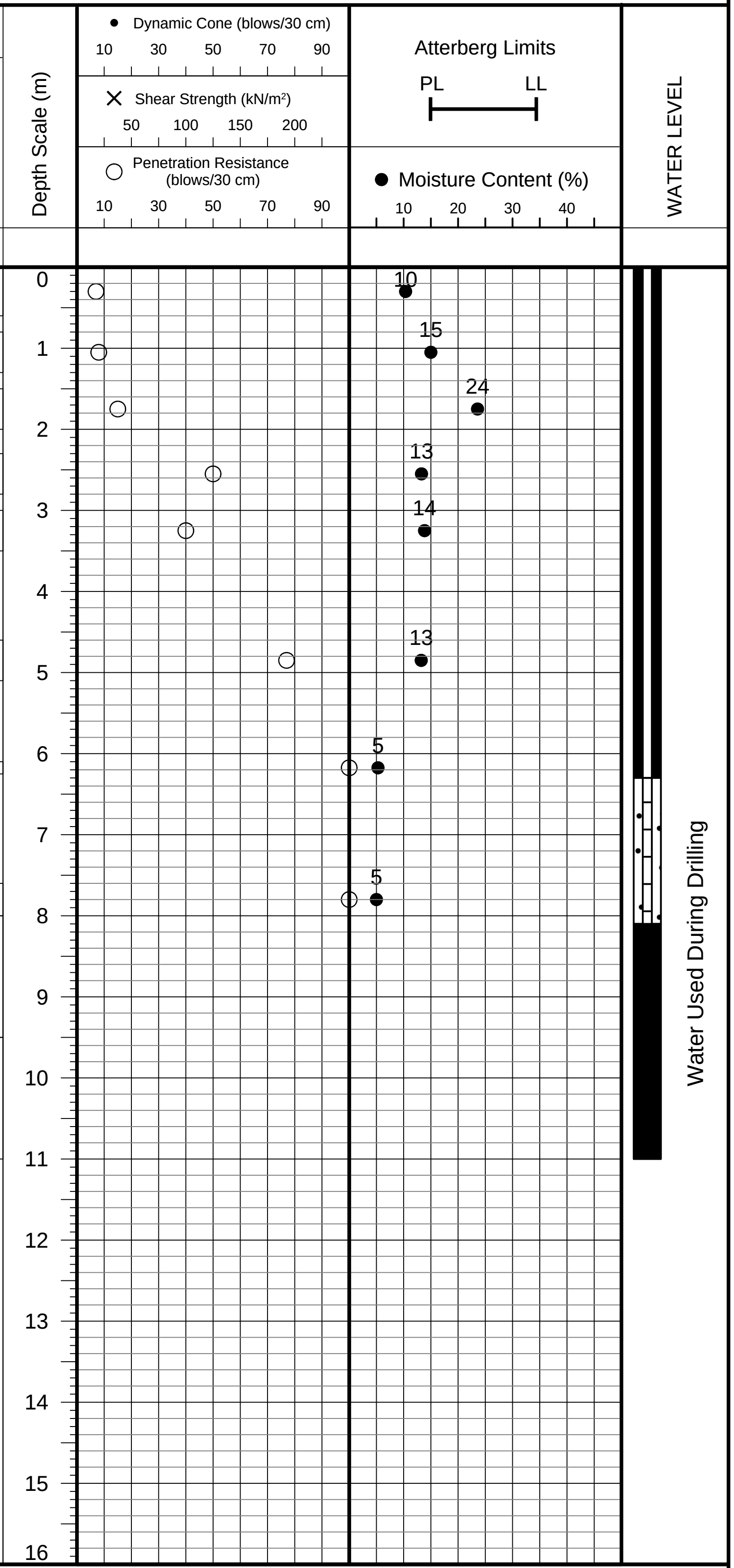
METHOD OF BORING: Hollow Stem Augers

PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

DRILLING DATE: June 19, 2025

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
133.9	Ground Surface			
0.0	**150 mm ASPHALTIC CONCRETE** **200 mm GRANULAR FILL**	1	DO	7
	Dark brown/Grey **EARTH FILL** silty clay, some sand occ. organics inclusion	2	DO	8
		3	DO	15
131.6 2.3	Dense to very dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams and layers, cobbles and boulders	4	DO	50
		5	DO	40
	brown grey	6	DO	77
128.4 5.5	Grey **SHALE** occ. limestone	7	DO	50/13
	weathered	8	DO	50/8
		9	RC 97%	RQD 63%
		10	RC 100%	RQD 65%
122.9 11.0	**END OF BOREHOLE**			

Installed 50 mm Ø PVC monitoring well
to 8.1 m with 1.5 m screen
Bentonite seal backfill from 8.1 m to 11.0 m
Sand backfill from 6.6 to 8.1 m
Bentonite seal from 0.0 m to 6.6 m
Provided with a steel monument casing



JOB NO.: 2505-S117

LOG OF BOREHOLE: 6S

FIGURE NO.: 8

PROJECT DESCRIPTION: Proposed Residential Development
with 2 or 3-Level Underground Parking

PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

METHOD OF BORING: Hollow Stem Augers

DRILLING DATE: June 19, 2025

● Dynamic Cone (blows/30 cm) 10 30 50 70 90

× Shear Strength (kN/m²) 50 100 150 200

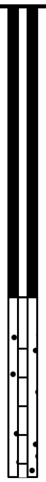
○ Penetration Resistance (blows/30 cm) 10 30 50 70 90

Atterberg Limits PL LL

● Moisture Content (%) 10 20 30 40

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	Depth Scale (m)	WATER LEVEL
133.9	Ground Surface					
0.0	**Straight Auger to Intall The Nested Monitoring Well**				0	
128.4 5.5	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 5.5 m completed with 1.5 m screen Sand backfill from 3.4 m to 5.5 m Bentonite seal from 0.0 m to 3.4 m Provided with a momument steel casing					

Depth Scale (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16



JOB NO.: 2505-S117

LOG OF BOREHOLE: 7D

FIGURE NO.: 9

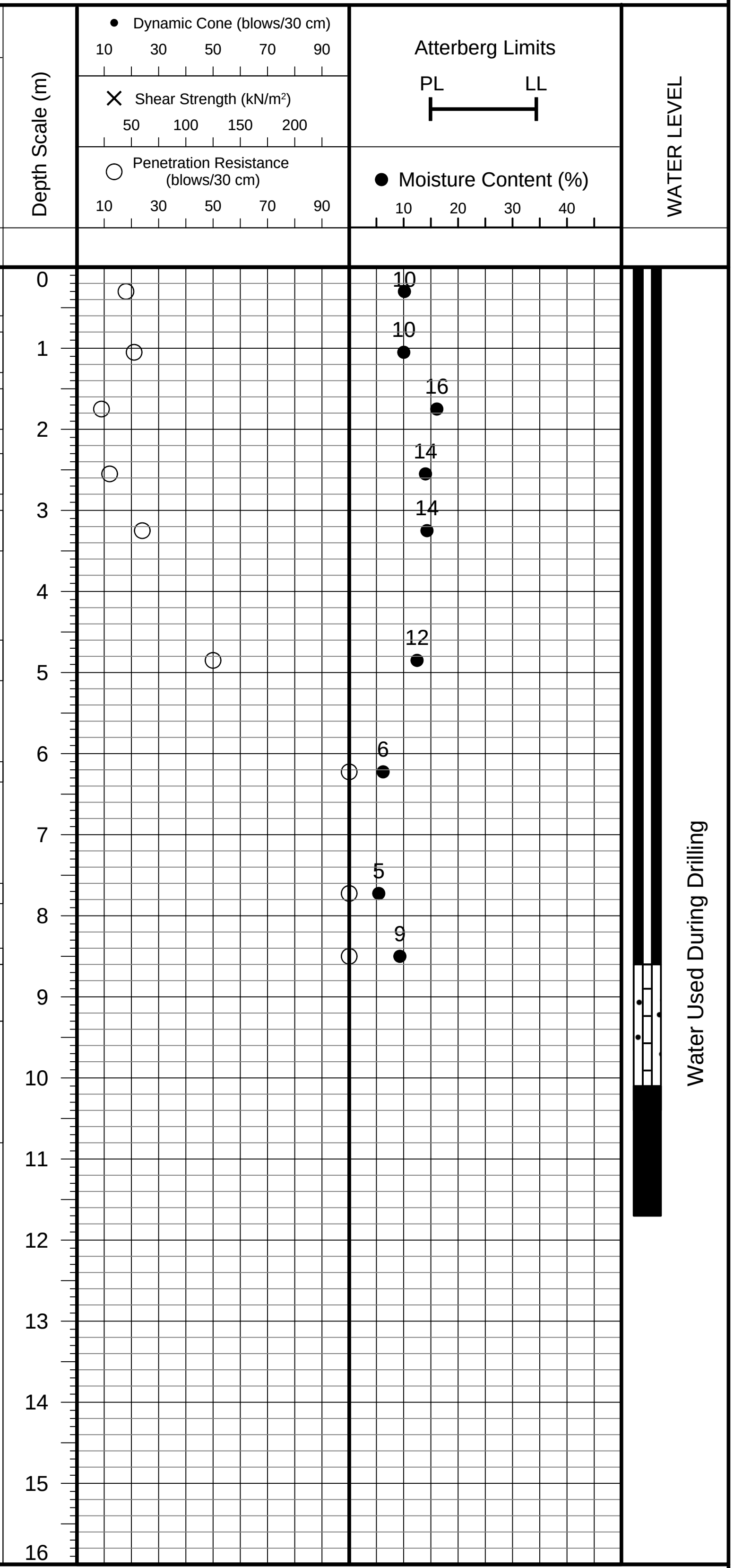
PROJECT DESCRIPTION: Proposed Residential Development with 2 or 3-Level Underground Parking

METHOD OF BORING: Hollow Stem Augers

PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

DRILLING DATE: June 12, 2025

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
136.9	Ground Surface			
0.0	Dark brown **EARTH FILL** silty clay occ. sand and gravel	1	DO	18
		2	DO	21
		3	DO	9
		4	DO	12
133.9 3.0	Brown, very stiff **SILTY CLAY TILL** traces of sand and gravel	5	DO	24
132.9 4.0	Grey, very dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams and layers, cobbles and boulders	6	DO	50
131.4 5.5	clay/shale conversion	7	DO	50/8
	Grey **SHALE** occ. limestone	8	DO	50/10
		9	DO	50/5
	weathered	10	RC 68%	RQD 18%
		11	RC 100%	RQD 65%
126.1 10.8	**END OF BOREHOLE**			



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JOB NO.: 2505-S117

LOG OF BOREHOLE: 7S

FIGURE NO.: 10

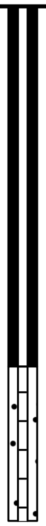
PROJECT DESCRIPTION: Proposed Residential Development
with 2 or 3-Level Underground Parking

METHOD OF BORING: Hollow Stem Augers

PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

DRILLING DATE: June 12, 2025

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	Depth Scale (m)	● Dynamic Cone (blows/30 cm) 10 30 50 70 90 × Shear Strength (kN/m²) 50 100 150 200 ○ Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL ● Moisture Content (%) 10 20 30 40	WATER LEVEL
136.9	Ground Surface							
0.0	**Straight Auger to Intall The Nested Monitoring Well**				0			
					1			
					2			
					3			
					4			
					5			
131.4 5.5	**END OF BOREHOLE**							
					6			
					7			
					8			
					9			
					10			
					11			
					12			
					13			
					14			
					15			
					16			



Soil Engineers Ltd.

JOB NO.: 2505-S117

LOG OF BOREHOLE: 8

FIGURE NO.: 11

PROJECT DESCRIPTION: Proposed Residential Development with 2 or 3-Level Underground Parking

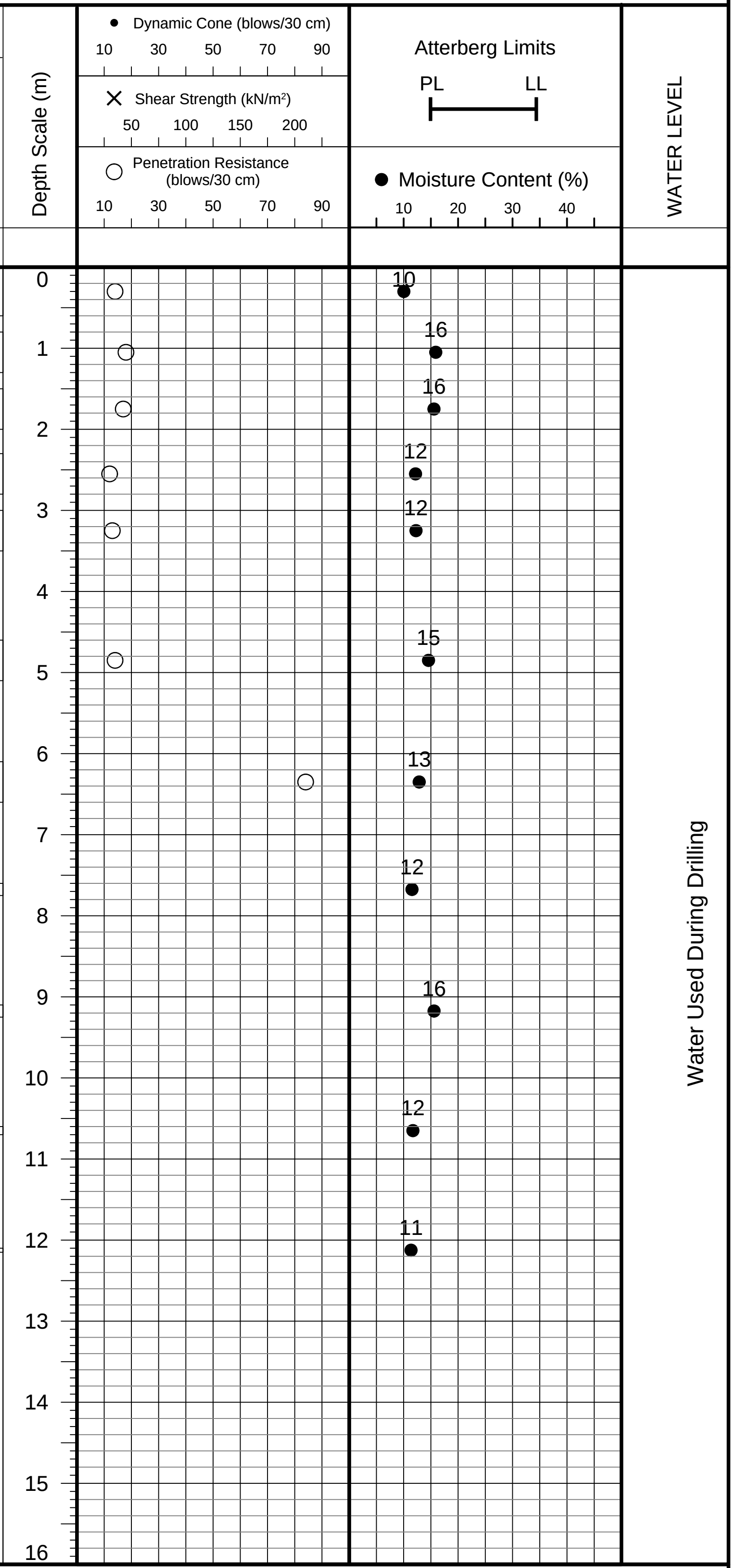
PROJECT LOCATION: 4100 Ponytail Drive, City of Mississauga

METHOD OF BORING: Hollow Stem Augers

DRILLING DATE: June 11 2025

El. (m) Depth (m)	SOIL DESCRIPTION
136.7	Ground Surface
0.0	Brown **EARTH FILL** sandy silt occ. organics inclusion
135.9 0.8	Stiff to very stiff **SILTY CLAY TILL** traces of sand and gravel brown grey
131.2 5.5	Grey, very dense, wet **SILTY SAND TILL** traces of clay and gravel occ. cobbles and boulders
129.7 7.0	Grey, very dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams and layers, cobbles and boulders
128.2 8.5	weathered Grey **SHALE** occ. limestone
124.5 12.2	**END OF BOREHOLE**

SAMPLES Number	Type	N-Value
1	DO	14
2	DO	18
3	DO	17
4	DO	12
5	DO	13
6	DO	14
7	DO	84
8	DO	50/13
9	DO	50/13
10	DO	50/8
11	DO	50/5

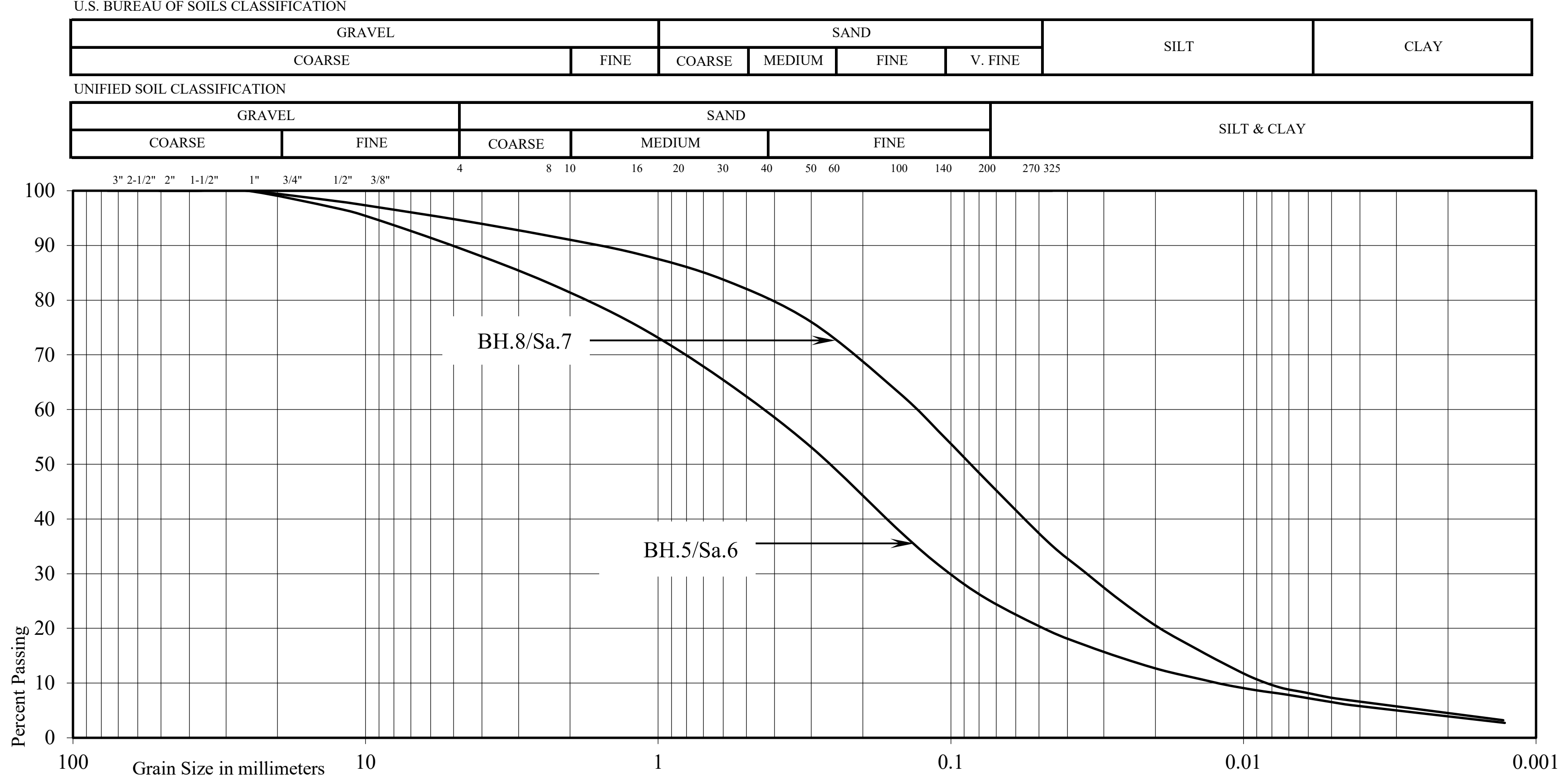


Soil Engineers Ltd.

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 2505-S063



Project: Proposed Residential Development with 3-Level Underground Parking

Location: South of Rathburn Road East and Ponytrail Drive, City of Mississauga

Borehole No:	5	8
Sample No:	6	7
Depth (m):	4.6	6.1
Elevation (m):	129.6	130.6

BH./Sa.	1/6	2/4
Liquid Limit (%) =	-	-
Plastic Limit (%) =	-	-
Plasticity Index (%) =	-	-
Moisture Content (%) =	11	13
Estimated Permeability (cm./sec.) =	10^{-4}	10^{-4}

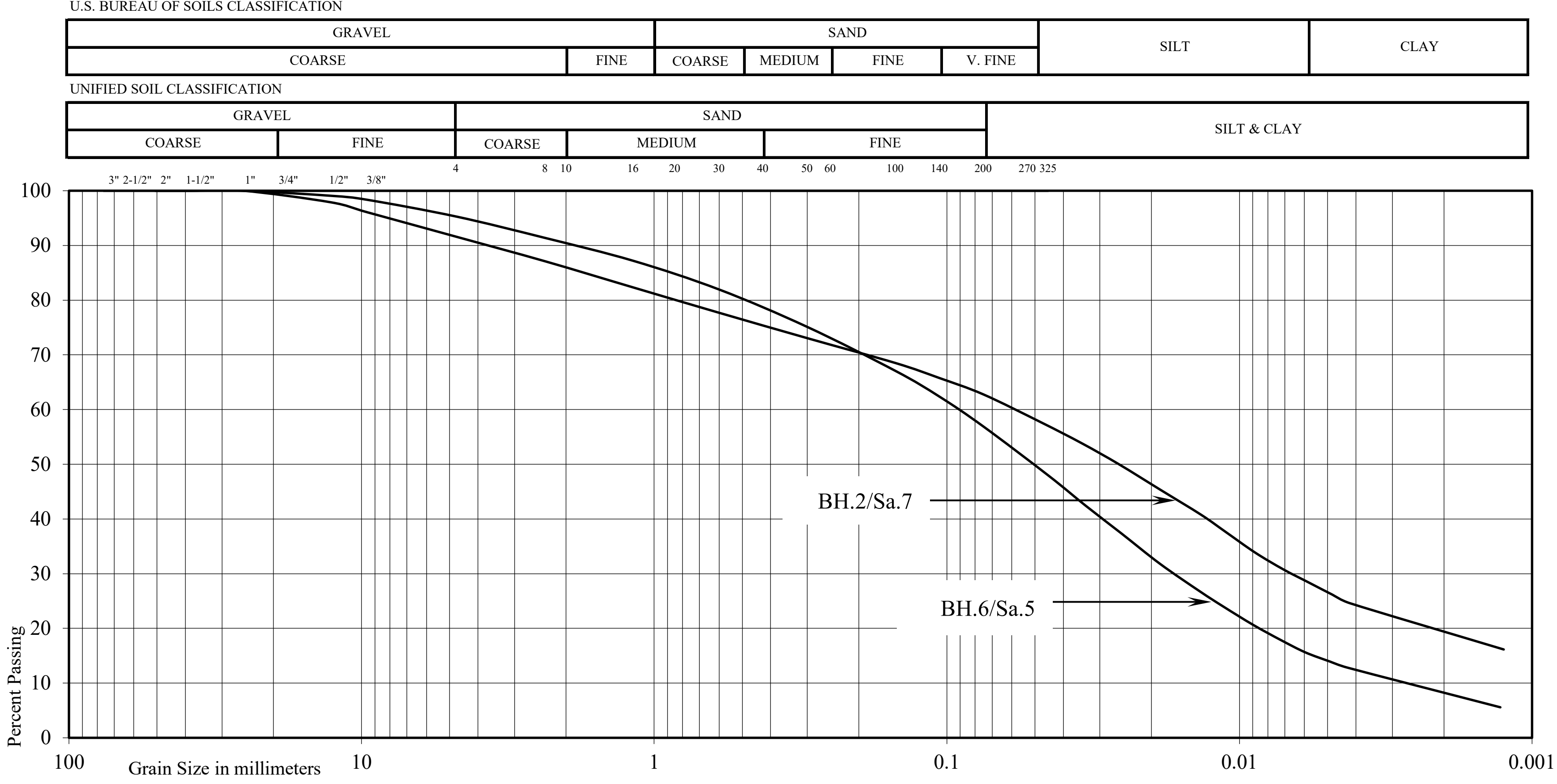
Classification of Sample [& Group Symbol]: SILTY SAND TILL, a trace to some gravel, a trace of clay

Figure: 12

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 2505-S117



Project: Proposed Residential Development with 3-Level Underground Parking

Location: South of Rathburn Road East and Ponytrail Drive, City of Mississauga

Borehole No:	2	6
Sample No:	7	5
Depth (m):	6.1	3.0
Elevation (m):	130.7	130.9

BH./Sa.	2/7	6/5
Liquid Limit (%) =	-	-
Plastic Limit (%) =	-	-
Plasticity Index (%) =	-	-
Moisture Content (%) =	12	14
Estimated Permeability (cm./sec.) =	10^{-7}	10^{-5}

Classification of Sample [& Group Symbol]: SANDY SILT TILL, sandy, traces of clay and gravel

Figure: 13

Soil Engineers Ltd.

ROCK CORE TEST REPORT

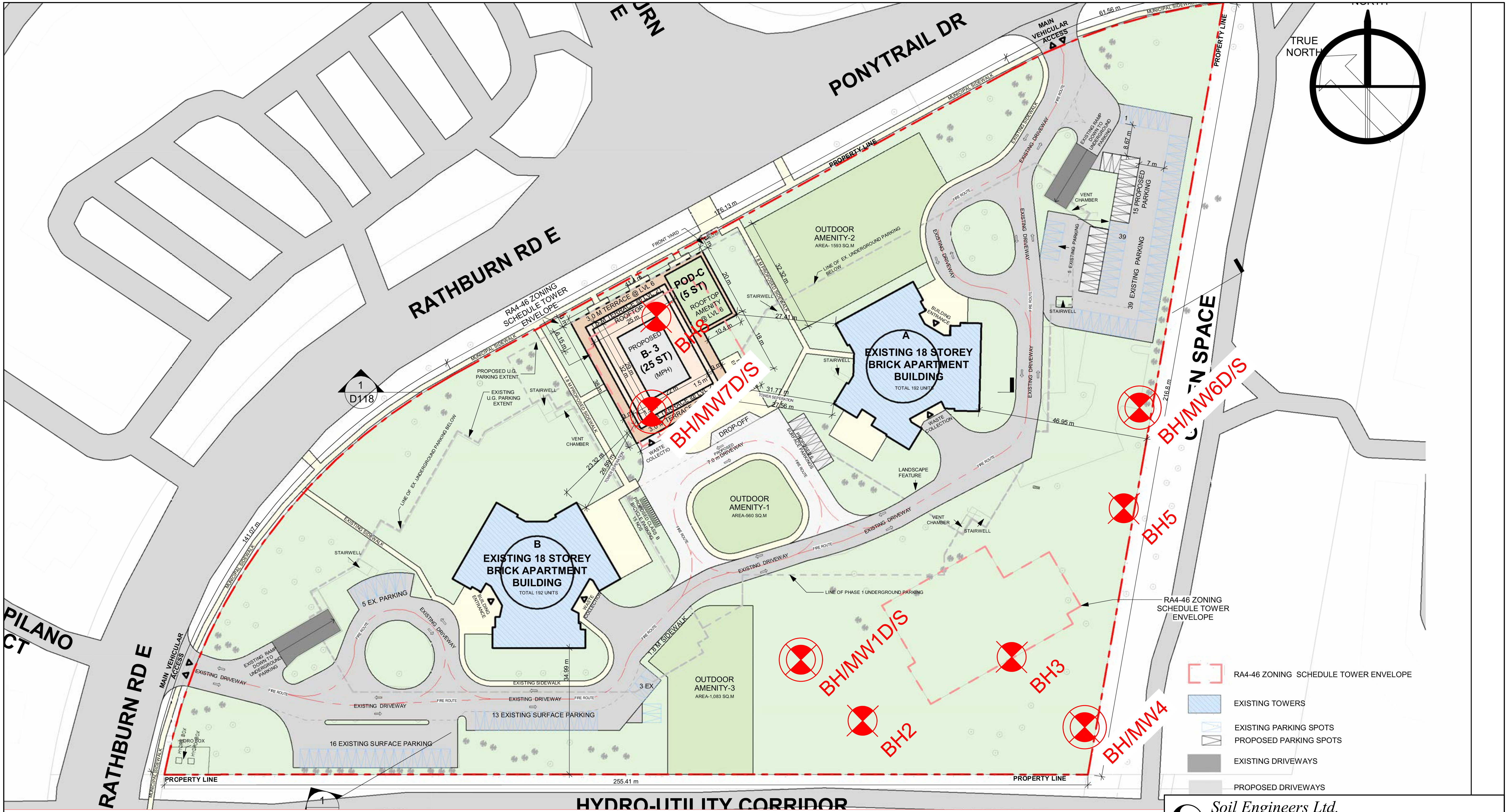
Job Location: 4100 Ponytail Drive, City of Mississauga

Reference No: 2505-S117 **Figure:** 14

Date: August, 2025

Figure 15

Core Number	BH 1	BH 4	BH 6	BH 7	BH 7
Depth, m	9.75	9.75	10.36	9.45	10.36
Elevation, m	127.05	125.65	123.54	127.45	126.54
Date Tested	Jul. 30, 2025	Jul. 30, 2025	Jul. 30, 2025	Jul. 4, 2025	Jul. 4, 2025
Core Diameter (mm)	60	60	60	60	60
Core Height (mm)	90	60	60	60	105
Density (kg / m^3)	2778	3478	3327	2664	2813
Corrected Compressive Stre	21.7	15.6	19.9	15.8	16.0



⊕ Borehole

⊕ Borehole w/ Monitoring Well

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GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

Borehole and Monitoring Well Location Plan

SITE: 4589 to 4601 Kingston Road, City of Toronto (Scarborough)

DESIGNED BY: D.Y.	**CHECKED BY:** K.L.	**DWG NO.:** 1
SCALE: 1:1000	**REF. NO.:** 2505-S117	**DATE:** August 2025

REV

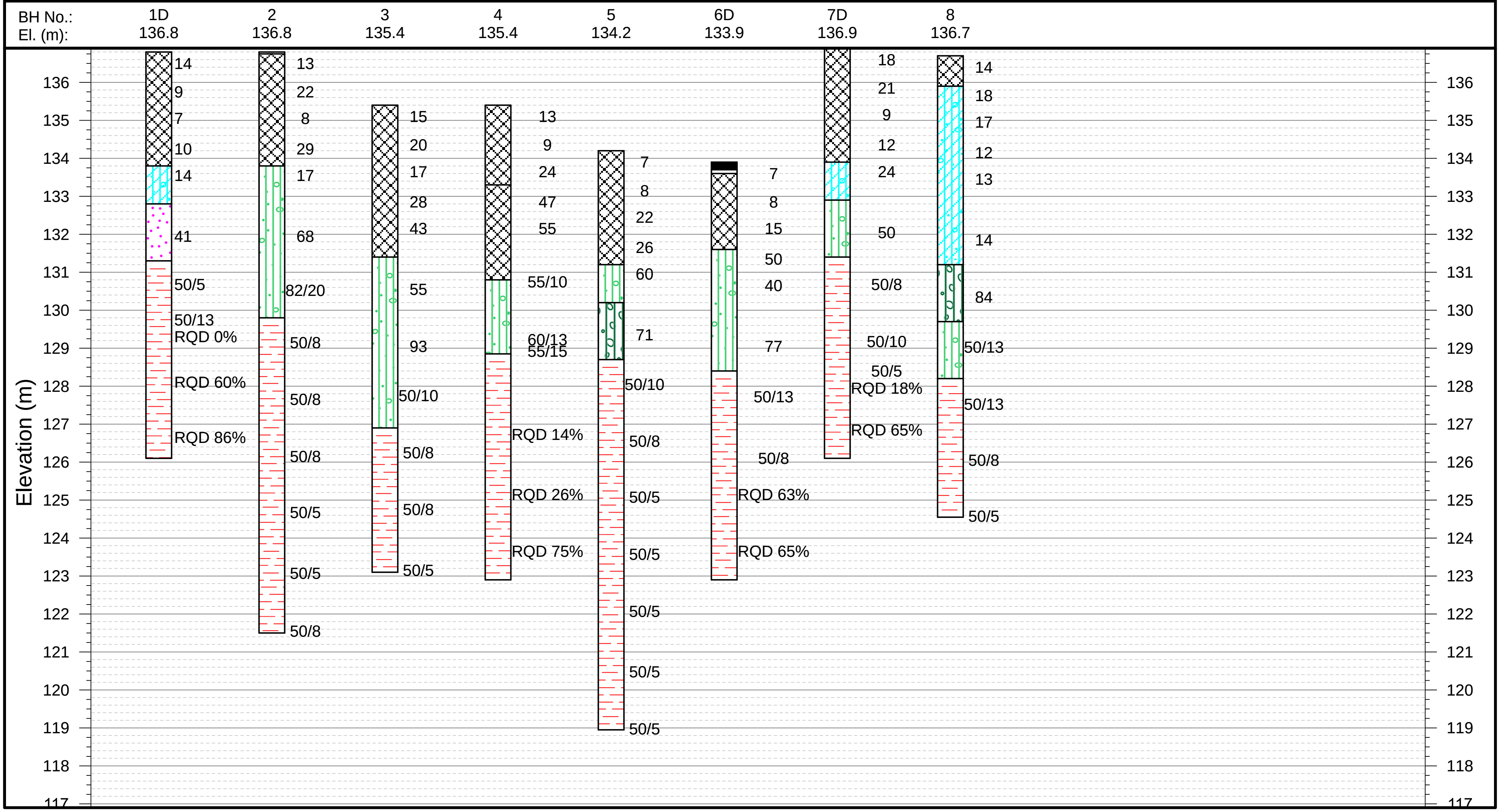
Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE

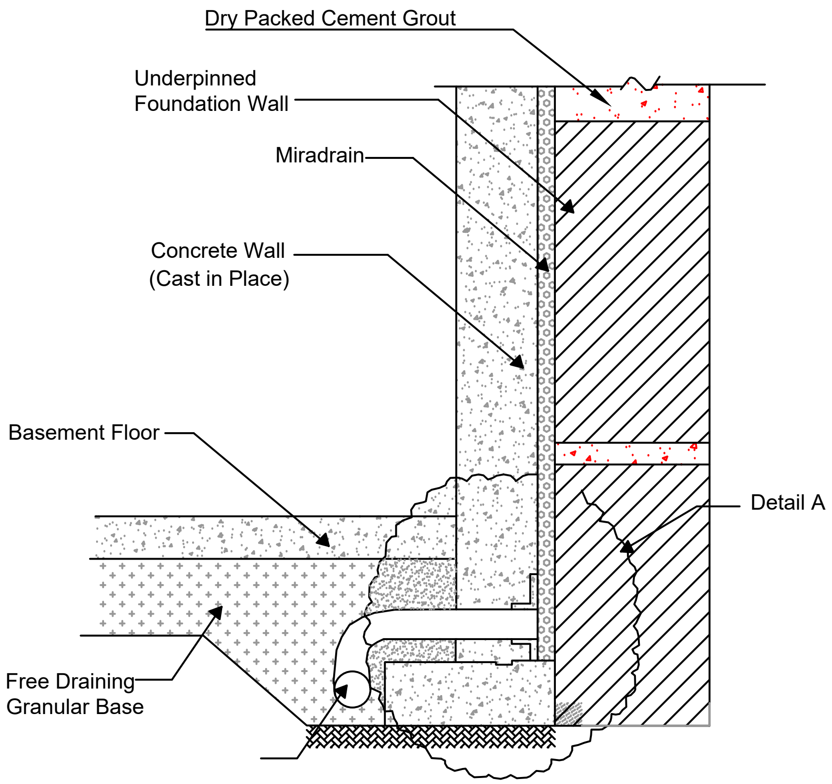
SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

JOB NO.:	2505-S117
REPORT DATE:	August 2025
PROJECT DESCRIPTION:	Proposed Residential Development with 2 or 3-Level Underground Parking
PROJECT LOCATION:	4100 Ponytail Drive, City of Mississauga

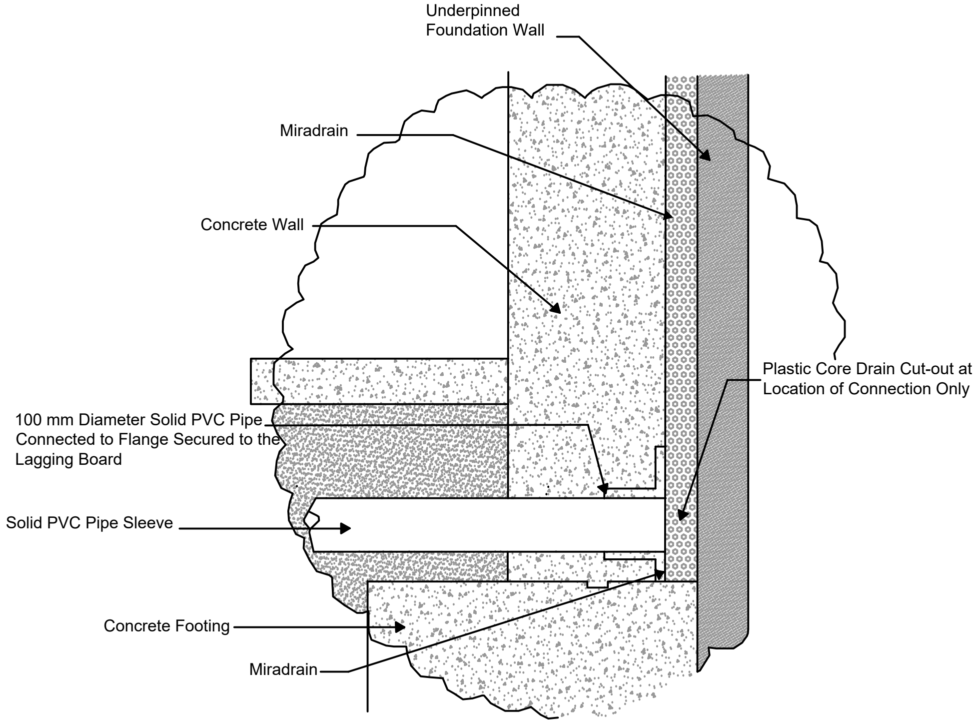
LEGEND

- ■ ASPHALT
- ▨ FILL
- ∴ GRANULAR
- ∴ SAND
- ▥ SANDY SILT TILL
- ☰ SHALE
- ▨ SILTY CLAY TILL
- ▥ SILTY SAND TILL
- ☰ TOPSOIL





TYPICAL SECTION



DETAIL A

NOTES:

1. A continuous blanket of prefabricated drainage system, Miradrain 6000 or equivalent, should extend continuously from the top of footings to the new footings.

2. All joints of the Miradrain should be taped. All openings above the concrete footing must be covered with filter fabric to prevent intrusion of fresh concrete into the core of the drain.

3. The perimeter drainage and any subfloor drainage systems must be kept separate.

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

PERMANENT DRAINAGE SYSTEM AGAINST SHORING

SITE: Blocks 149 and 150 - Mapleview Drive East and Madelaine Drive, City of Barrie

DESIGNED BY: D.Y.
CHECKED BY: K.H.
DWG NO.: 4
SCALE: N.T.S.
REF. NO.: 2505-S117
DATE: August 2025
REV -

APPENDIX ‘A’

MASW REPORT

REFERENCE NO. 2505-S117

GEOPHYSICS GPR
INTERNATIONAL INC.

July 22nd, 2025

Transmitted by email: daric.yang@soilengingeersltd.com
GPR Ref.: GPR-25-6439

Daric Yang, P.Eng.
Project Manager
Soil Engineers Ltd.
90 West Beaver Creek Rd, Suite 100,
Richmond Hill, Ontario
L4B 1E7

Subject: Shear Wave Velocity Surveys for Seismic Site Class Determination at 4100 Ponytrail Dr, Mississauga, Ontario

Dear Daric:

Geophysics GPR International Inc. has been requested by Soil Engineers Ltd. to carry out seismic shear-wave velocity (V_S) measurements at 4100 Ponytrail Dr, Mississauga, Ontario (Figure 1/ Figure 2).

The data collection was carried out June 30, 2025, by James Head and Gurmann Ubhi. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spread. Figures are presented in the Appendix.

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

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

MASW Principle

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

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

Survey Design

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

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

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

Interpretation Method and Accuracy of the Results

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

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

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

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

Results

The results of this investigation are presented as 1D shear-wave velocity profiles in Figure 5 and Figure 6 and in Tables 1 and 2.

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

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

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

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

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

Based on the above formula, the average Vs30 value at for sounding #1 is calculated as 700m/s over the depth interval of 0 to 30 m below grade (as determined through the MASW/SPAC and/or MAM methods). The minimum and maximum envelopes of the calculation over the same depth interval are 583 m/s and 783 m/s respectively.

For sounding #2, the Vs30 is calculated as 612 m/s over the depth interval of 0 to 30 m below grade (as determined through the MASW/SPAC and/or MAM methods). The minimum and maximum envelopes of the calculation over the same depth interval are 519 m/s and 716 m/s respectively.

Conclusions

Non-invasive geophysical surveys were carried out to measure shear-wave velocities for seismic site classification at 4100 Ponytrail Dr, Mississauga, Ontario (Figure 2). The seismic surveys used the MASW and the SPAC analysis methods to model the shear-wave velocities used in the calculation of the V_{s30} value. The calculations are presented in Tables 1 and 2 and summarized below.

Based on site specific borehole data provided by the client, weathered shale bedrock is noted at 5.5 m to 8.5 m below grade, with more competent bedrock noted several metres below this. The reader is referred to the geotechnical report for full borehole details.

Based on sentence 136 of Commentary ‘J’ of the Structural Commentaries NBC 2020, the V_{s30} value should represent the *top* 30m of soil or rock measured from the “ground surface” unless justification can be made otherwise. At the request of the client, the V_{s30} value has also been calculated taking into consideration the various FFE elevations below the grade level of the geophysical survey. The recalculated V_{s30}* values are presented in the table below. The use of the recalculated V_{s30}* values and the validity of assumptions related to ground surface level and depth interval used for the V_{s30}* calculation is at the justification and discretion of the design engineer.

The resolution and accuracy of the modelled velocities will decrease with depth and may involve some extrapolation within the bedrock. When extrapolating bedrock velocities, an assumption is made that the degree of weathering and fracturing in no worse at depth (as per Commentary ‘J’ Sentence 145 of the 2020 NBC Structural Commentaries).

Sounding	Elevation Interval	Depth Interval	V_{s30}	Site Class	Site Designation
#1 -Grade	135.5 to 105.5	0 to 30 m	700 m/s	C	X_{700}
#1 -FFE Opt A	130 to 100	5.5 to 35.5 m	1103 m/s	$B^{(1,2)}$	$X_{1103}^{(1,2)}$
#1 – FFE Opt B	125 to 95	10.5 to 40.5 m	1237 m/s	$B^{(1,2)}$	$X_{1237}^{(1,2)}$
#2 - Grade	138 to 8	0 to 30 m	612 m/s	C	X_{612}
#2 - FFE Opt A	130 to 100	8 to 38 m	1082 m/s	$B^{(1,2)}$	$X_{1082}^{(1,2)}$
#2 – FFE Opt B	125 to 95	13 to 43 m	1234 m/s	$B^{(1,2)}$	$X_{1234}^{(1,2)}$

Notes –

(1) Refer to Sentence 136 of Commentary ‘J’ of the Structural Commentaries NBC 2020, regarding site designation with regards to excavation and foundation levels.

(2) In order to consider site classes A or B, the depth to bedrock has to be taken into account as per Table 4.1.8.4.-B Note 2 of the OBC and sentence 92-Commentary ‘J’ of the Structural Commentaries NBC 2020, specifically, “*Site Classes A and B, are not to be used if there is more than 3 m of soil between the rock surface and the bottom of the spread footing or mat foundation, even if the computed average shear wave velocity is greater than 760m/s”*. **If the thickness of the softer material is greater than 3m, then site class “C” or site designation X_{760} should be applied as per Table 4.1.8.4.-A of the 2024 OBC Compendium.**

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

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

Analysis of the data was carried out by Andrés M. Rincón R. and reviewed by Ben McClement, P.Eng.

Ben McClement, P.Eng.

LICENSED PROFESSIONAL ENGINEER
2025-07-22
R.B. McCLEMENT
PROVINCE OF ONTARIO

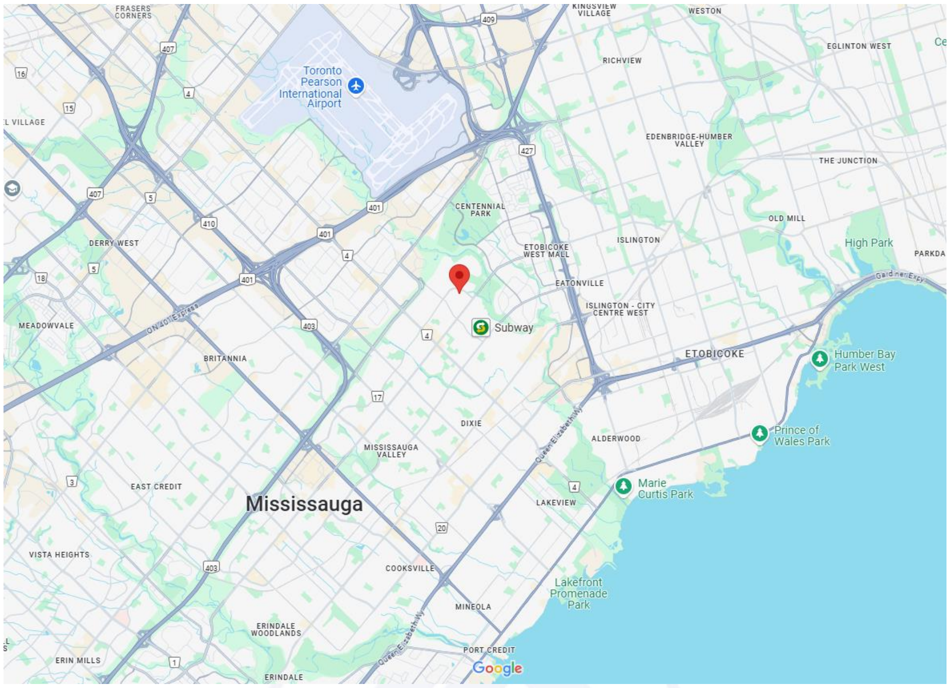


Figure 1: Regional location of the Site

(Source: *Open StreetMap™*)

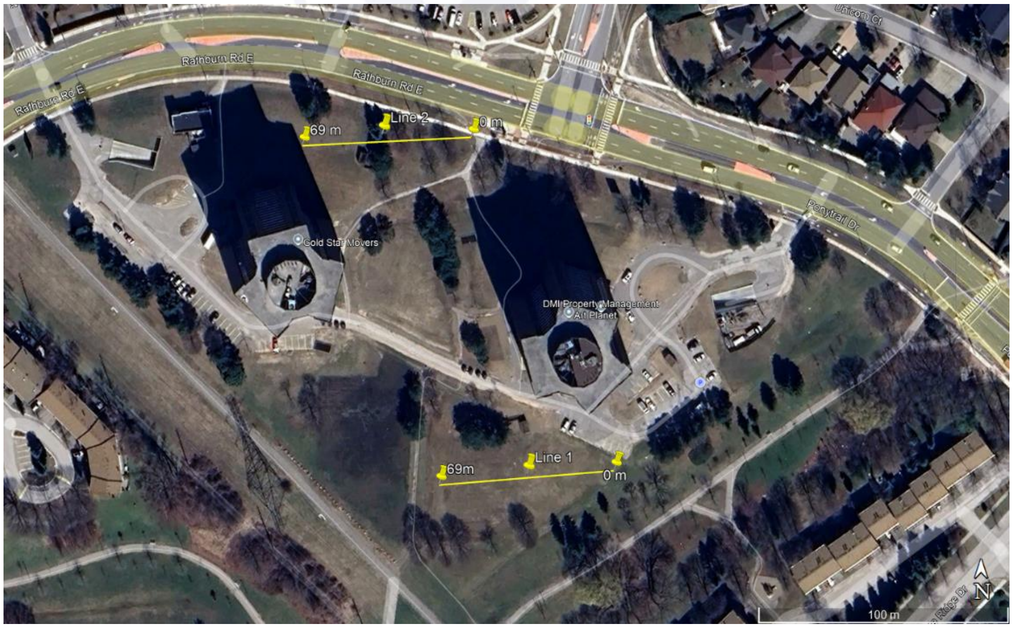


Figure 2: Location of the seismic spread

(Source: *Google Earth™*)

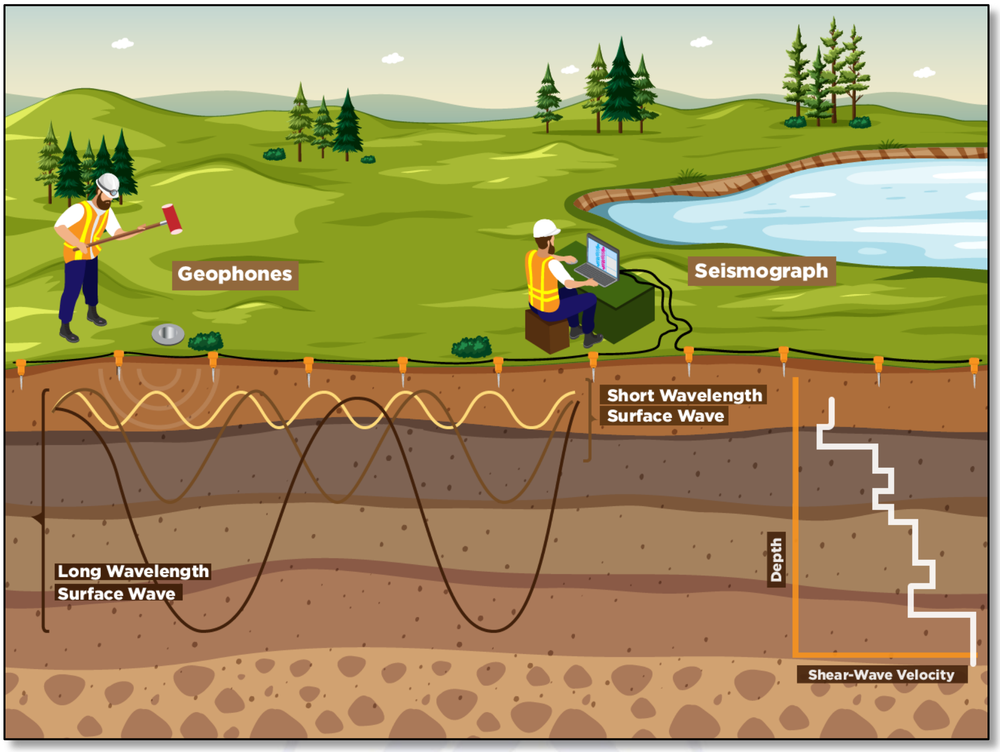


Figure 3: MASW Operating Principle

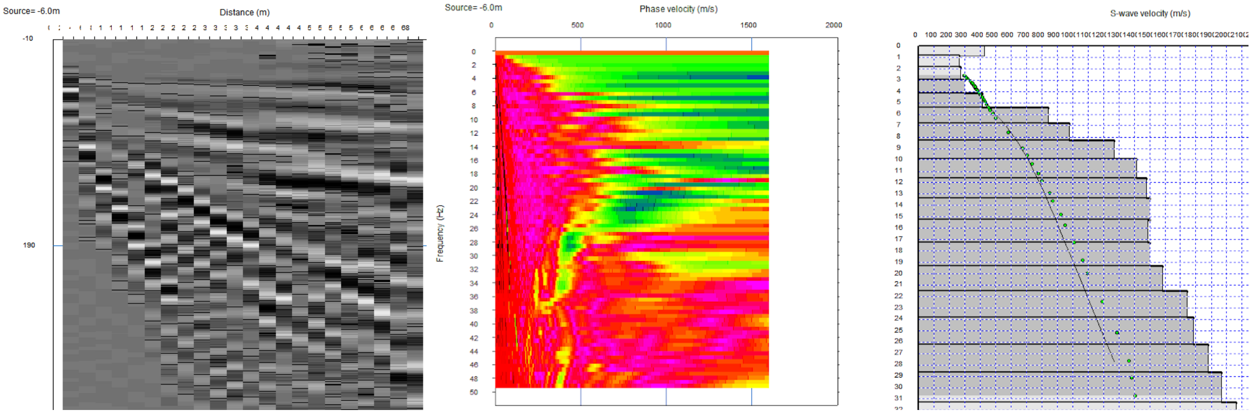


Figure 4: Example of a MASW/SPAC record, Phase Velocity - Frequency curve of the Rayleigh wave and resulting 1D Shear Wave Velocity Model

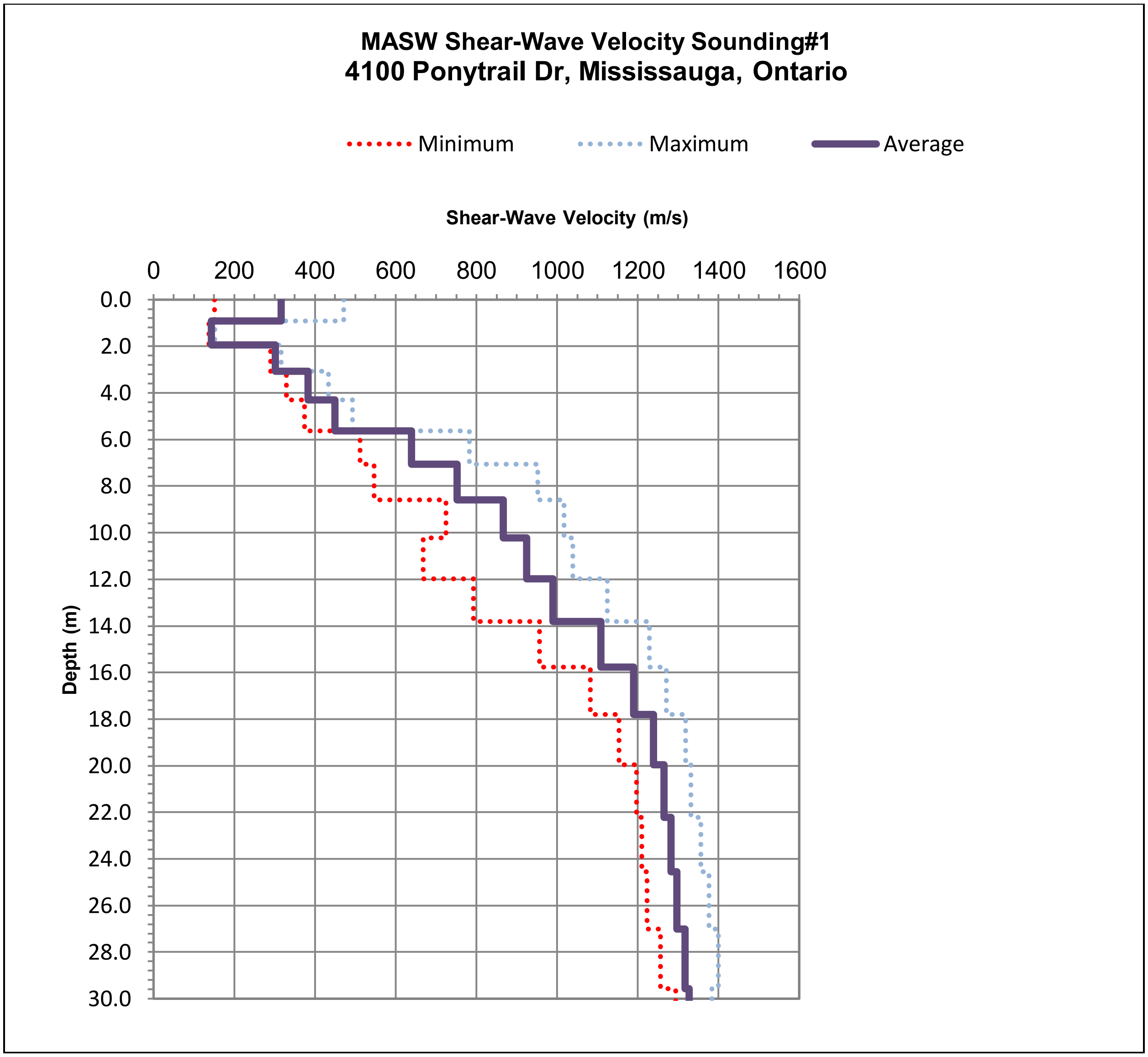


Figure 5: Shear-Wave Velocity Inversion Model from MASW/SPAC for Sounding #1

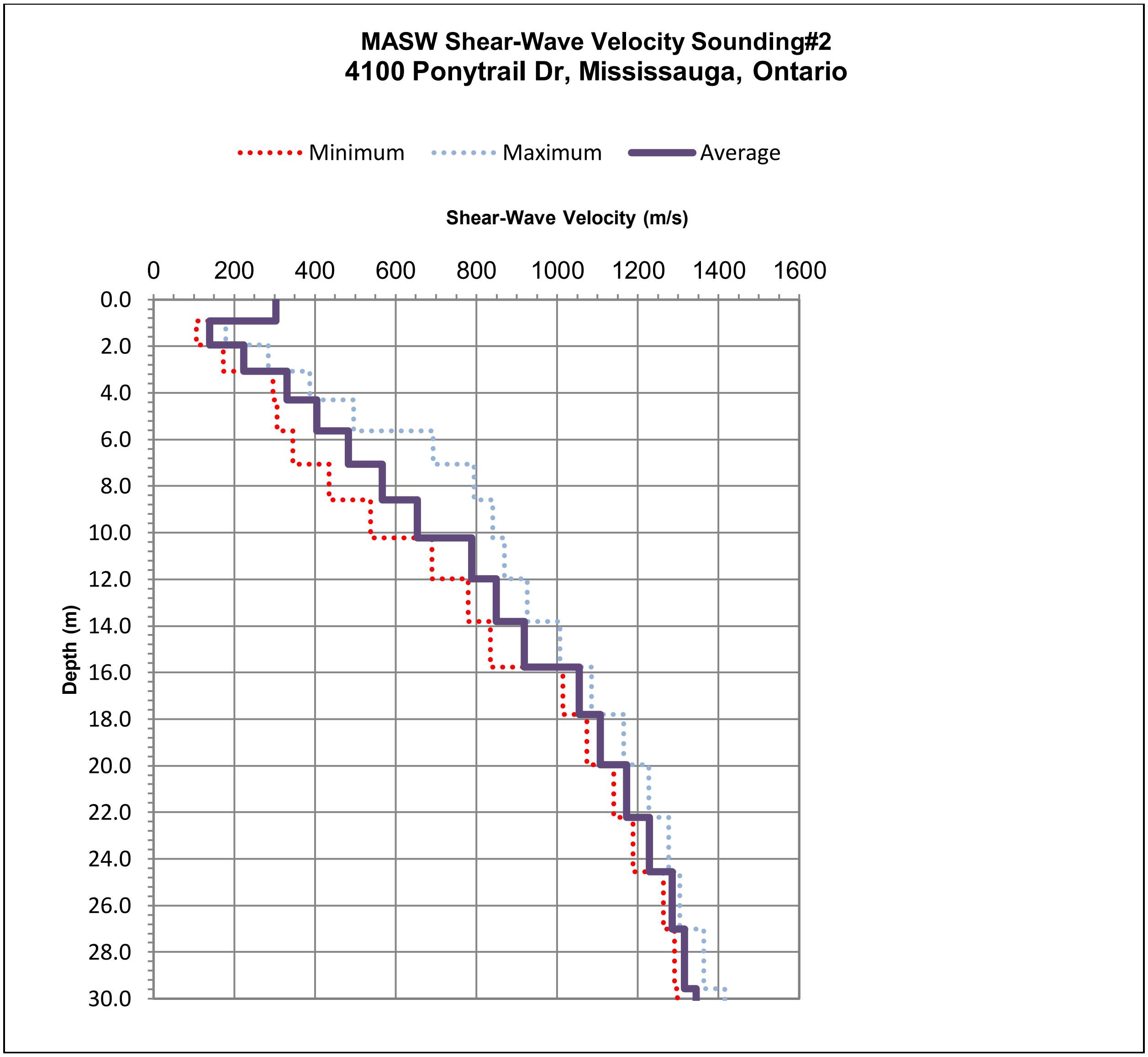


Figure 6: Shear-Wave Velocity Inversion Model from MASW/SPAC for Sounding #2

TABLE 1: V_{S30} Calculation for the Site Class for Sounding #1

Depth	Vs			Thickness	Cumulative Thickness	Delay for Vs Mean	Cumulative Delay	Vs Mean to given Depth
	Min.	Mean	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0.0	151	316	472	Grade Level				
0.9	137	143	152	0.9	0.9	0.002919	0.002919	316
1.9	291	302	316	1.0	1.9	0.007146	0.010066	193
3.1	330	383	434	1.1	3.1	0.003726	0.013792	223
4.3	374	450	492	1.2	4.3	0.003204	0.016996	253
5.6	512	639	782	1.3	5.6	0.002958	0.019954	282
7.1	546	752	952	1.4	7.1	0.002244	0.022198	318
8.6	724	867	1017	1.5	8.6	0.002041	0.024239	355
10.2	669	924	1039	1.6	10.2	0.001889	0.026127	392
12.0	793	989	1124	1.7	12.0	0.001882	0.028009	427
13.8	957	1109	1228	1.8	13.8	0.001862	0.029871	463
15.8	1082	1189	1270	1.9	15.8	0.001753	0.031625	498
17.8	1153	1239	1319	2.0	17.8	0.001721	0.033346	534
20.0	1196	1265	1331	2.1	20.0	0.001735	0.035081	569
22.2	1210	1283	1357	2.3	22.2	0.001780	0.036860	602
24.6	1224	1297	1377	2.4	24.6	0.001835	0.038695	635
27.0	1256	1317	1400	2.5	27.0	0.001894	0.040589	666
29.6	1294	1328	1383	2.6	29.6	0.001943	0.042532	695
30.0				0.4	30.0	0.000319	0.042851	700
32.2	1296	1349	1386	2.2	32.2			
40.5	1299	1402	1506	8.3	40.5			

TABLE 2: V_{S30} Calculation for the Site Class for Sounding #2

Depth	Vs			Thickness	Cumulative Thickness	Delay for Vs Mean	Cumulative Delay	Vs Mean to given Depth
	Min.	Mean	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0.0	303	303	303	Grade Level				
0.9	106	139	179	0.9	0.9	0.003040	0.003040	303
1.9	173	224	284	1.0	1.9	0.007364	0.010404	187
3.1	297	331	387	1.1	3.1	0.005031	0.015435	199
4.3	307	405	496	1.2	4.3	0.003714	0.019149	224
5.6	345	483	692	1.3	5.6	0.003289	0.022438	251
7.1	435	567	794	1.4	7.1	0.002967	0.025405	278
8.6	538	654	840	1.5	8.6	0.002706	0.028111	306
10.2	690	788	869	1.6	10.2	0.002504	0.030615	334
12.0	780	850	927	1.7	12.0	0.002207	0.032822	365
13.8	834	918	1008	1.8	13.8	0.002168	0.034990	395
15.8	1014	1054	1085	1.9	15.8	0.002118	0.037108	425
17.8	1074	1106	1165	2.0	17.8	0.001941	0.039049	456
20.0	1141	1172	1227	2.1	20.0	0.001942	0.040992	487
22.2	1188	1229	1277	2.3	22.2	0.001920	0.042912	518
24.6	1264	1285	1304	2.4	24.6	0.001915	0.044827	548
27.0	1291	1315	1363	2.5	27.0	0.001912	0.046739	578
29.6	1298	1344	1415	2.6	29.6	0.001945	0.048684	608
30.0				0.4	30.0	0.000315	0.049000	612
32.2	1300	1400	1552	2.2	32.2			
40.5	1322	1446	1650	8.3	40.5			

APPENDIX ‘B’

SHORING DETAILS

REFERENCE NO. 2505-S117

SHORING SYSTEM

Shoring will be required in an excavation to limit the horizontal and vertical movements of adjacent properties.

A shoring system consisting of soldier piles and lagging boards can be used in an excavation where slight movement in the adjacent properties is tolerable. In areas in close proximity to adjacent structures and where the excavation will be extending below the foundation level so that any movement in the adjacent properties is a concern, or in an excavation embedding into saturated sand or silt deposit, an interlocking caisson wall is more appropriate.

The design and construction of the shoring system should be carried out by a specialist designer and contractor experienced in this type of construction. All specifications for the design of the shoring system should be in accordance with the latest edition of the Canadian Foundation Engineering Manual (CFEM).

LATERAL EARTH PRESSURE

For single and multiple level supporting systems, the lateral earth pressure distributions on the shoring walls are shown on Drawing A1. The design soil parameters are provided in the geotechnical report.

The lateral earth pressure expressions do not include hydrostatic pressure build up behind the shoring. If the wall is designed to be water tight or undrained, such as a caisson wall, the anticipated hydrostatic pressure must be included behind the structure.

PILE PENETRATION

The depth of pile support should be calculated from the following expressions:

In Cohesionless Soils: $R = 1.5\ D\ K_p\ L^2\ \gamma$

where R = Ultimate Load to be restrained (kN)
D = Diameter of concrete filled hole (m)
K_p = Passive resistance in the silt till and sand deposits
L = Embedment depth of the pile (m)
γ = unit weight of the soil (kN/m^3)

In Cohesive Soils: $R = 9\, c_u\, D\, (L - 1.5\, D)$

where R = Ultimate Load to be restrained (kN)
D = Diameter of concrete filled hole (m)
L = Embedment depth of the pile (m)
c_u = Undrained shear strength of subsoil (kPa)

The shoring system should be designed for a factor of safety of F = 2.

For anchor supported shoring system, the global factor of safety against sliding and overturning of the anchored block of soil must also be considered.

The steel soldier piles in the shoring system must be installed in pre-augured holes. The lower portion will have to be filled with 20 MPa (3000 psi) concrete to the excavation level. The upper portion of the pile within the excavation depth should be filled with lean mix concrete or non-shrinkable cementitious filler (U-fill).

LAGGING

The following thicknesses of lagging boards have been recommended in CFEM:

Thickness of Lagging	Maximum Spacing of Soldier Piles
50 mm (2 in)	1.5 m (5 ft)
75 mm (3 in)	2.5 m (8 ft)
100 mm (4 in)	3.0 m (10 ft)

Local experience has indicated that the lagging board thickness of 75 mm has been adequate for soldier pile spacing of 3 m for soil conditions similar to those encountered at the subject site. However, it is important to consider all local conditions, such as the duration of excavation, the weather likely to be encountered through the construction period, seasonal variations in the ground water and ice lensing causing frost heave and softening of soils in determining the lagging thickness. During winter months, the shoring should be covered with thermal blankets to prevent frost penetration behind the shoring system which may result in unacceptable movements.

During construction of shoring, all the spaces behind the lagging board must be filled with free draining granular fill. If wet conditions are encountered, the space between the boards should be packed with a geotextile filter fabric or straw to prevent the loss of fine particles.

TIEBACK ANCHORS

The minimum spacing and the depths of the soil anchors should be as recommended in the CFEM.

All drilled holes for tieback anchors should be temporarily cased or lined to minimize the risk of caving. Systems involving high grout pressures should be avoided if working near other basements or buried services.

The tieback anchor lengths extending into the native soils can be estimated using an adhesion value of 40 kPa. Full scale load tests should be carried out on the tieback anchors in each type of soil and at each level of anchor support at the site to confirm the design parameters and the adhesion values. The test anchors should be loaded in a pattern as described in CFEM, to 200% of the design load or until there is a significant increase in the pullout rate. In the latter case, the design load must be limited to 50% of the maximum load at which the pullout increases. Based on the results of the pullout test, it may be necessary to modify the anchor design of the production anchors.

Each tieback anchor must be proof-loaded to 133% of the design load, and the anchor must be capable of sustaining this load for a minimum of 10 minutes without creep. The load may then be relaxed to 100% of the design and locked in. The higher the lock-in loads, the less will be the outward movement on the shoring wall after excavation.

RAKERS

An alternative to tieback anchor support of the shoring is to use raker footings. Rakers inclining at an angle of 45º, founded below the bottom of excavation should be designed for the allowable bearing pressure of 300 or 450 kPa, for sound native till or weathered shale bedrock, respectively.

The raker footings should be located outside the zone of influence of the buried portion of the soldier piles at a distance of not less than 1.5 of the length of embedment of the soldier pile.

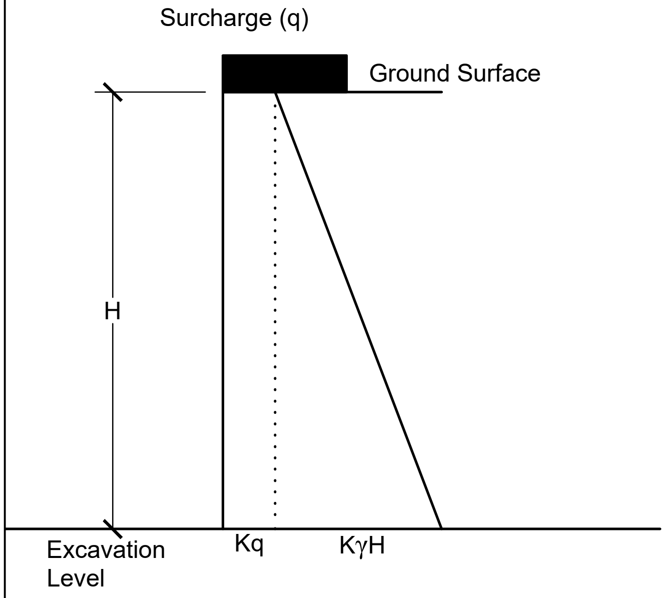
To prevent undermining of the raker footing, no excavation should be made within two times the width of raker footing on the opposite side of the raker.

When sloping berm excavation procedures are used, the rakers should be installed in trenches in the berm to minimize movement of the shoring wall being supported. In addition, the rakers can be pre-loaded and secured in place before removal of the earth berm.

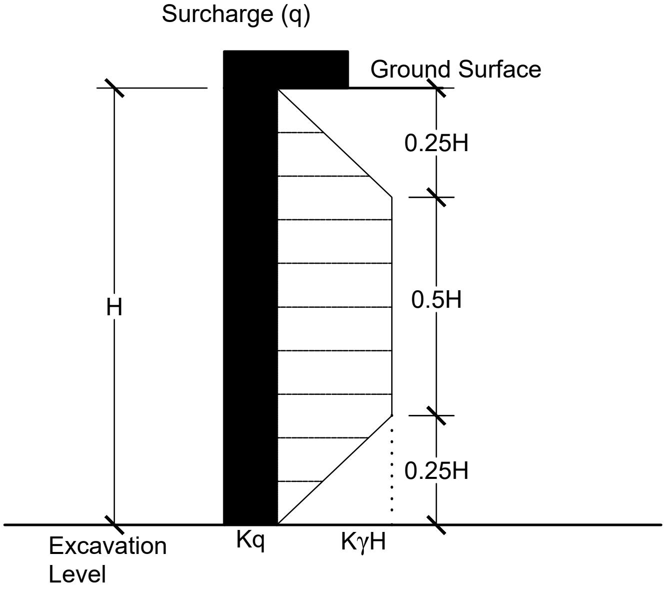
MONITORING OF PERFORMANCE

Close monitoring of the vertical and lateral movement of the shoring system, by inclinometers or by survey on targets, should be carried out at the site. Extra bracing or support may be required if any movement is found excessive. The contractor should maintain the shoring to ensure any movement is within the design limit.

TEMPORARY SHORING
Lateral Earth Pressures



Single Support System



Multiple Support System

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

Where
H = Height of Shoring (m)
γ = Unit Weight of Retained Soil (kN/m^3)
q = Surcharge (kPa)
K = Earth Pressure Coefficient

- If moderate ground and shoring movements are permissible then:
  $K = K_a$ = Active Earth Pressure Coefficient

- if there are building foundations within a distance of 0.5 H behind the shoring then:
  $K = K_o$ = Earth Pressure at rest

- If there are building foundations within a distance of between 0.5 H and H behind the shoring then:
  $K = 0.5 (K_a + K_o)$

Note:
1. The lateral pressure expression assumes effective drainage from behind the temporary shoring.
2. The earth pressure coefficients are specified in the geotechnical report.

Soil Engineers Ltd.
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Temporary Shoring - Lateral Earth Pressures

SITE: Blocks 149 and 150 - Mapleview Drive East and Madelaine Drive, City of Barrie

DESIGNED BY: D.Y.
CHECKED BY: K.H.
DWG NO.: A1
SCALE: N.T.S.
REF. NO.: 2505-S117
DATE: August 2025
REV -