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A REPORT TO
STAR SEEKER INC.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED MIXED-USE DEVELOPMENT

579 TO 619 LAKESHORE ROAD EAST, AND 1022 AND 1028 CAVEN STREET

CITY OF MISSISSAUGA

REFERENCE NO. 2102-S032

AUGUST 2022
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DISTRIBUTION

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1.0 INTRODUCTION

In accordance with written authorization from Mr. Paul Breda of Star Seeker Inc., dated February 3, 2021, a geotechnical investigation was carried out 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street, in the City of Mississauga.

The purpose of the investigation was to reveal the subsurface conditions and determine the engineering properties of the disclosed soils for the design and construction of the proposed mixed-use development. The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

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

The site is irregular in shape and encompasses an approximate area of 24,239m^2. At the time of the investigation, the site consisted of a commercial plaza with various stores, and houses located at 1022 and 1028 Caven Street. The majority of the area surrounding the buildings consist of asphalt paved drive aisles/driveways and parking, with grass-covered areas around the houses. The existing site is relatively flat, only varying in elevation by 1 to 2 m throughout.

Based on the drawing package provided, it is understood that the proposed development will consist of four buildings (Buildings A to D). Building A is a 16-storey building with a 6-storey podium; Building B consists of two 16-storey towers joined with a 6-storey podium; and Buildings C and D are 6-storey buildings joined with a 5-storey podium with ground floor retail spaces. The development will also include 2 levels of underground parking throughout the site with a 3rd level of underground parking for Buildings A and B.

3.0 FIELD WORK

The field work, consisting of 13 boreholes to depths of 4.9 to 10.1 m, with rock coring beyond the sampling depth at 3 boreholes to depths of 8.8 m and 9.3 m, was performed between September 27 and October 1, 2021, at the locations shown on the Borehole and Monitoring Well Location Plan, Drawing No. 1.

The boreholes were advanced at intervals to the sampling depths by truck-mounted, continuous-flight power-auger machines equipped for soil sampling. Standard Penetration

Tests, using the procedures described on the enclosed "List of Abbreviations and Terms", were performed at the sampling depths. The test results are recorded as the Standard Penetration Resistance (or 'N' values) of the subsoil. The relative density of the non-cohesive 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. The field work was supervised and the findings were recorded by Geotechnical Technicians.

In addition, 'HQ' size (63.5 mm core diameter) rock coring was carried out at 3 boreholes to assess the quality and soundness of the encountered shale bedrock. The quality of the rock has been assessed by applying the 'Rock Quality Designation' (RQD) classification, considering the total length of the recovered pieces 10 cm or longer against the length of the core run. The results are expressed as a percentage and are recorded on the Borehole Logs.

Upon completion of borehole drilling, sampling and rock coring, monitoring wells were installed at 6 borehole locations to facilitate a hydrogeological assessment, of which the study will be presented under a separate cover.

The ground elevation at each borehole location was determined using a handheld Global Navigation Satellite System (Trimble Geoexplorer 6000 series) equipment.

4.0 SUBSURFACE CONDITIONS

All boreholes were carried out on the asphalt-paved ground surface. The investigation has disclosed that beneath a pavement structure, and a layer of earth fill in places, the site is underlain by strata of silty clay till and/or sandy silt till. Beneath the overburden, shale bedrock was contacted at shallow to moderate depths throughout the site.

Detailed descriptions of the encountered subsurface conditions at the boreholes are presented on the Borehole Logs, comprising Figures 1 to 13, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing Nos. 2 and 3, and the engineering properties of the disclosed soils are discussed herein.

4.1 **Pavement Structure** (All Boreholes)

The pavement structure consists of 25 to 200 mm of asphalt concrete, overlying 100 to 250 mm of granular fill at all borehole locations.

4.2 **Earth Fill** (All Boreholes, except Boreholes 1, 4, 7 and 10)

The earth fill was encountered beneath the pavement structure, extending to depths of 0.8± to 1.7± m below the prevailing ground surface; it consists of sand, silt and clay, with a trace of gravel, and containing organic/topsoil inclusions in places.

The obtained 'N' values range from 3 to 20, with a median of 14 blows per 30 cm of penetration, indicating the earth fill was placed with varying degree of compaction.

The natural water content of the samples was determined and the results are plotted on the Borehole Logs; the values range from 3% to 23%, with a median of 15%, indicating that the earth fill is in a generally damp to very moist condition.

4.3 **Silty Clay Till** (All Boreholes)

The silty clay till dominated the upper soil stratigraphy; it was generally encountered beneath the pavement structure and/or earth fill, and extends to depths of 2.3± to 4.9± m from grade. The clay till consists of a random mixture of soils; the particle sizes range from clay to gravel, with the clay fraction exerting the dominant influence on its soil properties. It is embedded with sand seams and layers, cobbles, boulders and/or shale or rock fragments in places. A grain size analysis was performed on 1 representative silty clay till sample; the result is plotted on Figure 14.

The obtained 'N' values range from 5 to more than 100, with a median of 22 blows per 30 cm of penetration, indicating that the consistency of the clay till is firm to hard, being generally very stiff.

The Atterberg Limits of 1 representative silty clay till sample and the water content of all of the clay till samples were determined. The results are plotted on the Borehole Logs and summarized below:

Liquid Limit	28%
Plastic Limit	17%
Natural Water Content	9% to 25% (median 14%)

The above results and sample examinations show that the clay till has low plasticity. The natural water content generally lies below the plastic limit, confirming its generally very stiff consistency as disclosed by the 'N' values.

The engineering properties of the silty clay till are given below:

- High frost susceptibility, low water erodibility, and moderate soil-adfreezing potential.
- Low permeability, with an estimated coefficient of permeability of 10^{-7} cm/sec, and an estimated percolation time of more than 80 min/cm.
- The shear strength is derived from consistency and augmented by internal friction of the sand and silt.
- The clay till will generally be stable in a relatively steep cut. However, prolonged exposure will allow infiltrating precipitation to saturate the sand and silt seams and layers, which leads to localized sloughing.
- A poor pavement-supportive material, with an estimated California Bearing Ratio (CBR) value of 3% or less.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 2500 to 3500 ohm·cm.

4.4 **Sandy Silt Till** (All Boreholes except Boreholes 4 and 9)

The sandy silt till was encountered beneath the earth fill and silty clay till in majority of the boreholes. The silt till consists of a random mixture of soils; the particle sizes range from clay to gravel, with the silt fraction exerting the dominant influence on its soil properties. The till contains occasional sand seams and layers, cobbles and boulders. Grain size analyses were performed on 2 representative samples of the sandy silt till; the results are plotted on Figure 15.

The obtained ‘N’ values range from 22 to more than 100, with a median of 42 blows per 30 cm of penetration, indicating the till stratum is compact to very dense, being generally dense in relative density.

The natural water content of the soil samples were determined and the results are plotted on the Borehole Logs; the values range from 7% to 16%, with a median of 10%, indicating the till is in moist to very moist conditions.

The engineering properties of the sandy silt till are given below:

- High frost susceptibility and moderately low water erodibility.
- Relatively low permeability, with an estimated coefficient of permeability of 10^{-5} to 10^{-6} cm/sec, depending on the clay content, and an estimated percolation time of 30 to 50 min/cm.
- Its shear strength is primarily derived from the internal friction and is augmented by cementation.

- In steep cuts, the till will be relatively stable; however, under prolonged exposure, localized sheet collapse may occur in the zone where sand and silt layers are prevalent.
- A fair pavement-supportive material, with an estimated CBR value of 8%.
- Moderate corrosivity to buried metal, with an estimated electrical resistivity of 4000 to 4500 ohm·cm.

4.5 **Shale Bedrock** (All Boreholes except Boreholes 3, 7, 8 and 9)

Shale bedrock was encountered at depths below 4.6± to 6.1± m from the prevailing ground surface throughout the site. The lower zone of the soils above the shale bedrock, in places, may be derived from a clay-shale reversion rendering the boundary between the soil and shale bedrock interface difficult to delineate. At the remaining boreholes where shale bedrock was not noted on the Borehole logs, refusal to auger likely indicates the presence of bedrock below the refusal depths.

The shale is grey in colour, indicating that it is of Georgian Bay Formation; it is a laminated, sedimentary, moderately soft rock composed predominantly of clay material, and it is interbedded with about 20% sandstone and limy shale bands. The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil, but the laminated limy and sandy layers would remain as rock slabs.

The upper layers of the shale are often fissured as a result of the weathering process and/or overstressing by glaciation. Infiltrated precipitation and groundwater from the overburden soils will often permeate the fissures in the rock and, in places, will be under subterranean artesian pressure. However, because the shale is a clay rock, it is considered to be a material of low permeability and a poor aquifer; upon release through excavation, the water is likely to drain readily with a limited yield.

The upper 1.0± to 2.0± m of the shale is likely weathered and, in places, was penetrable by power augering, with some difficulty grinding through the hard layers. In other places, the shale was penetrable by power augering to 5 m into the bedrock. The obtained 'N' values in the shale bedrock, where split spoon sampling was able to commence, were more than 100 blows per 30 cm of penetration, and the water content values of the shale samples obtained from the sampler or auger range from 4% to 8%, with a median of 5%.

Rock coring was carried out in the shale bedrock starting at depths ranging from 5.8 to 6.2 m and terminating at depths of 8.8 m and 9.3 m, at Boreholes 1, 4 and 5. The recovery of 'HQ' size rock cores ranges from 92% to 100%. The RQD values range from 0% to 88%, indicating the quality of the shale varies from very poor to very good, where the quality becomes fair to good 1 to 2 m into the bedrock.

Uniaxial Compressive Strength (UCS) tests were carried out on 3 core samples of which the results are presented in Table 1.

Table 1 - Uniaxial Compressive Strength (UCS) Results

Borehole No.	Sample No.	Depth (m)	UCS (MPa)
1	7	5.8	24.5
4	8	7.5	18.6
5	9	7.0	17.5

The weathered rock can be excavated with considerable effort by a heavy-duty backhoe equipped with a rock-ripper; however, excavation will become progressively more difficult with depth into the sound shale. Efficient removal of the sound shale may require the aid of pneumatic hammering.

The excavated spoil may contain large amounts of hard limy and sandy rock slabs, rendering it virtually impossible to obtain uniform compaction. Therefore, unless the spoil is sorted, it is considered unsuitable for engineering applications.

In sound shale excavation, 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 characteristics of the rock.

5.0 GROUNDWATER CONDITION

The boreholes were checked for the presence of groundwater and the occurrence of cave-in during the drilling operation. Groundwater was encountered at depths of 4.3 m and 5.5 m from grade at Boreholes 7 and 11, respectively, and Borehole 2 caved at a depth of 7.5 m upon borehole completion. The groundwater likely represents perched groundwater at the borehole locations. The groundwater is subject to seasonal fluctuation.

Upon completion of borehole drilling and sampling, and rock coring, monitoring wells were installed in 6 selected Boreholes to facilitate a hydrogeological assessment; the results of the groundwater monitoring and hydrogeological assessment will be presented under a separate cover.

6.0 DISCUSSION AND RECOMMENDATIONS

All boreholes were carried out on the asphalt-paved ground surface. The investigation has disclosed that beneath a pavement structure, and a layer of earth fill in places, the site is underlain by strata of firm to hard, generally very stiff silty clay till and/or compact to very dense, generally dense sandy silt till. Beneath the overburden, shale bedrock was contacted below depths of 4.6± to 6.1± m from the prevailing ground surface throughout the site.

Upon completion of the borehole drilling, groundwater and/or cave-in was recorded at depths of 7.5 m, 4.3 m and 5.5 m below the prevailing ground surface at Boreholes 2, 7 and 11, respectively. The groundwater likely represents perched groundwater at the borehole locations. The groundwater at the site will be further discussed in the hydrogeological assessment. The groundwater is subject to seasonal fluctuation.

As previously mentioned, the property development will consist of four buildings (Buildings A to D). Building A is a 16-storey building with a 6-storey podium; Building B consists of two 16-storey towers joined with a 6-storey podium; and Buildings C and D are 6-storey buildings with ground floor retail spaces. In addition, the entire development will include 2 levels of underground parking throughout with a 3rd level of underground parking within Buildings A and B. An additional roadway entrance to the development will be constructed at 1022 and 1028 Caven Street.

The geotechnical findings which warrant special consideration are presented below:

1. After demolition of the existing structures, the cavities should be backfilled and the subgrade should be stabilized for construction.
2. The foundation details of the adjacent structures must be investigated and incorporated into the excavation, design and construction of the underground structure. A pre-construction survey and a monitoring program should be carried out for all adjacent structures in order to verify any potential future claims.
3. The excavation for the underground structure is anticipated to extend to at least 7 to 11 m below the ground surface onto the shale bedrock suitable to support the proposed development.
4. In conventional design and construction, the underground structure should be provided with a drainage system connecting into the municipal sewer. If the municipality does not allow the removal of groundwater into the sewer, the subsurface water has to be discharged into a cistern in the building or the underground structure has to be designed and constructed as a watertight structure.
5. Excavation should be carried out in accordance with Ontario Regulation 213/91. Braced shoring walls will be required for the excavation where a safe backing slope is not possible.

The recommendations appropriate for the project described in Section 2.0 are presented herein. One must be aware that the subsurface conditions may vary between boreholes.

Should subsurface variances become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

6.1 Site Preparation

The existing structures will be demolished for site development. After demolition of the existing structures, the cavities should be backfilled with selected on-site material, and compacted to the required density. The subgrade should be stabilized for construction.

The foundation details of the adjacent structures must be investigated and incorporated into the excavation, design and construction of the underground structure. A pre-construction survey and a monitoring program should be carried out for the adjacent structures in order to verify any potential future claims.

6.2 Foundations

The site will be redeveloped with 4 buildings with 2 levels of underground parking spanning across the entire site, and a 3rd level within Buildings A and B. It is anticipated that the lowest parking level will lie at least 10 to 11 m below the existing grade within Buildings A and B and 7 to 8 m below the existing grade throughout most of the site. Therefore, the bottom of excavation will likely lie within the shale bedrock.

In conventional design and construction, with an effective drainage system in the underground structure, the proposed development can be constructed on conventional spread and strip footings. The design bearing pressures for the design of footings are presented below:

- Maximum Bearing Pressure at Serviceability Limit State (SLS) = 1000 kPa
- Factored Ultimate Bearing Pressure at Ultimate Limit State (ULS) = 1600 kPa

The 3-level underground parking garage is expected to be placed within the sound bedrock. Accordingly, the design bearing pressure for the design of footings are presented below:

- Factored Ultimate Bearing Pressure at ULS = 1500 kPa

The elevator pit, which normally extends a few metres below the floor level, should be designed as a submerged ‘tank’ structure with waterproofed pit walls and pit floor.

If the drainage system is not practical or the municipality does not allow the removal of groundwater into the sewer, the underground structure has to be waterproofed and constructed on a raft foundation to resist the hydrostatic pressure. The design bearing pressures for raft foundations for the two-level underground garage are provided below:

- Maximum Bearing Pressure at SLS = 800 kPa
- Factored Ultimate Bearing Pressure at ULS = 1200 kPa

For the three-level underground garage, the design bearing pressures for raft foundations are provided below:

- Factored Ultimate Bearing Pressure at ULS = 1500 kPa

A Modulus of Subgrade Reaction (k_s) of 50 MPa/m can be used for the design of a raft foundation.

The total and differential settlements of foundations designed for the recommended bearing pressures at SLS on weathered shale are estimated to be 25 mm and 20 mm, respectively. The total and differential settlements of foundations in sound shale bedrock will be negligible.

The stepped footings between the 2-levels and 3-levels of underground parking should be stepped at a gradient of 1 vertical:1 horizontal gradient to maintain stability.

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

The shale bedrock will slake if left exposed for any length of time. It is, therefore, important that the footings are poured with concrete immediately on excavation and inspection. Alternatively, the footings should be skim coated with lean mix concrete, 8 to 10 cm in thickness, to minimize deterioration of rock at the bearing surface.

Footings 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 most of the time, the earth cover can be reduced to 0.6 m for perimeter walls and 0.9 m for interior walls and columns, except in the area in close proximity of the ventilation shafts and door entrances.

The building foundations should meet the requirements specified in the latest Ontario Building Code. The structure should be designed to resist an earthquake force using Site Classification 'C' (very dense soil and soft rock). Where a higher Site Classification is required, a downhole shearwave velocity test can be performed to assess the ground condition.

6.3 Underground Parking

The underground structure should be designed to sustain a lateral earth pressure calculated using the soil parameters stated in Section 6.8. Any applicable surcharge loads adjacent to the proposed building must also be considered in the design of the underground structure.

In areas where the perimeter walls extend into the shale bedrock, a compressible material, such as sprayed foam, 100 mm in thickness, should be placed between the concrete wall and the bedrock. This is to allow lateral expansion or movement of the rock face without causing damage to the foundation walls.

In conventional design, the perimeter walls of the underground garage should be dampproofed and provided with a perimeter subdrain encased in a fabric filter at the wall base.

Prefabricated drainage board, such as Miradrain 6000 or equivalent, must be provided between the shoring wall or rock face and the cast-in-place foundation wall as shown on Drawing No. 4.

The lower parkade slab should be constructed on a 200 mm thick granular bedding, consisting of 19-mm Crusher-Run Limestone, or equivalent, compacted to at least 98% Standard Proctor Dry Density (SPDD).

If the Municipality does not allow any discharge of subsurface water into the sewer system, a separate storage cistern should be provided or, otherwise, the entire underground structure will have to be waterproofed. In this case, the building will have to be founded on a raft foundation, and designed for the full depth of hydrostatic pressure on the foundation walls and below the foundation. The lower parkade slab will be poured on a granular fill above the raft where the utilities and service pipes will be laid.

The ground around the buildings must be graded to direct water away from the structures to minimize the frost heave phenomenon generally associated with the disclosed soils.

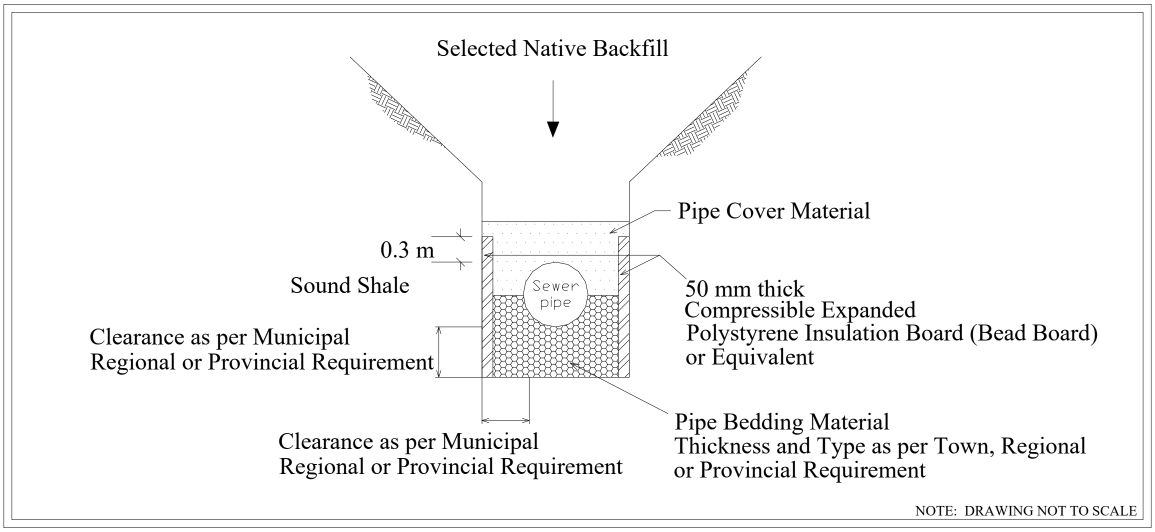
6.4 Underground Services

The subgrade for the underground services should consist of bedrock, sound natural soils or properly compacted organic-free earth fill. Where earth fill or badly weathered soil is encountered, it should be subexcavated and replaced with bedding material compacted to at least 98% SPDD.

Sewer construction will require rock excavation. In general, it can be carried out by using a backhoe equipped with a rock-ripper, but where trench excavation into the thick limy or sound shale is required, a pneumatic hammer should be used to break up the rock mass for excavation.

Where the pipe is to be placed in sound shale bedrock, the rock face must be lined with a cushioning layer such as Styrofoam, then backfilled with fine sand to 0.3 m above the crown of the pipe and flooded. The recommended scheme is illustrated in Diagram 1.

Diagram 1 - Sewer Installation in Sound Shale



A Class ‘B’ bedding consisting of compacted 19-mm Crusher-Run Limestone, or equivalent is recommended for the underground services construction. The pipe joints into manholes and catch basins should be leak-proof, or wrapped with an appropriate waterproof membrane, to prevent subgrade soil migration.

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

Openings to subdrains and catch basins should be shielded with a fabric filter to prevent blockage by silting.

The subgrade of underground services may have moderate to moderately high corrosivity to metal pipes and fittings; therefore, the underground services should be protected against soil corrosion. For estimation for the anode weight requirements, the estimated electrical resistivity given for the disclosed soils can be used. The proposed anode weight must meet the minimum requirements as specified by City of Mississauga and/or Region of Peel.

6.5 Backfilling in Trenches and Excavated Areas

The on site native soils are generally suitable for use as trench backfill. However, if the existing fill is to be used, it should be sorted free of any organic inclusions and other deleterious materials prior to reuse for backfilling. In addition, the backfill material should be free of large pieces (over 15 cm in size) of limy shale, sandstone and shale fragments. If encountered, the large pieces must be broken into sizes suitable for structural compaction.

The backfill should be compacted to at least 95% SPDD and increased to at least 98% SPDD below the floor slab. In the zone within 1.0 m below the pavement subgrade, the material should be compacted to at least 98% SPDD with the water content at 2% to 3% drier than the optimum. This is to provide the required stiffness for pavement construction.

Narrow trenches should be cut at 2 horizontal (H):1 vertical (V), or flatter, so that the backfill can be effectively compacted. Otherwise, soil arching will prevent the achievement of proper compaction. 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 settlement largely occur adjacent to manholes, catch basins, services crossings, foundation walls and columns. The lumpy clays and broken shale are generally difficult to compact in these close quarters. It is recommended that a granular backfill should be used for compaction in confined spaces with a smaller vibratory compactor.

6.6 Sidewalks and Landscaping

The on-site soils are frost susceptible. On-grade sidewalk and structures must be designed to tolerate ground movement due to frost heaving. In areas which are sensitive to frost-induced ground movement, such as the sidewalk and barrier-free ramp in front of building entrances, they can be constructed on a free-draining, non-frost-susceptible granular material such as Granular ‘B’, extending to a depth of 1.2 m, and provided with weeper subdrains at the base draining towards manholes or catch basins.

Alternatively, the sidewalk and barrier-free ramp subgrade can be insulated with 50-mm Styrofoam, with free-draining granular material for the ramp construction above grade only.

The finished ground surface must be graded to direct water away from the structure to minimize the frost heave phenomenon generally associated with the disclosed soils.

6.7 Pavement Design

Where the pavement is to be built over-top of the underground garage, 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 steel reinforcing bars against brine corrosion. For the proposed pavement structure on top of the underground parking garage, the recommended pavement structure is given in Table 2.

Table 2 - Pavement Design on Structural Slab

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	65	HL-8
Granular Base	200	OPSS Granular ‘A’ or equivalent
Granular Sub-base	250	OPSS Granular ‘B’ or equivalent

The recommended pavement design for on-grade access driveway is presented in Table 3.

Table 3 - Pavement Design for On-Grade Access Road

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	65	HL-8
Granular Base	200	OPSS Granular ‘A’ or equivalent
Granular Sub-base	400	OPSS Granular ‘B’ or equivalent

In preparation of the pavement, the final subgrade surface must be proof-rolled. Any soft spot identified should be subexcavated and replaced with selected on-site material, free of organics, and uniformly compacted to at least 98% SPDD. The granular bases should be compacted to 100% SPDD.

Along the perimeter where surface runoff may drain onto the pavement, 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 pavement). The subdrains should consist of filter wrapped weepers, and connected to the catch basins and storm manholes. The subdrains should be backfilled with free-draining granular material.

6.8 Soil Parameters

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

Table 4 - Soil Parameters

Bulk Unit Weight and Bulk Factor

	Bulk Unit Weight (kN/m^3)	Estimated Bulk Factor	
		Loose	Compacted
Existing Earth Fill	20.5	1.25	0.98
Silty Clay Till/Sandy Silt Till	22.0	1.30	1.03
Shale Bedrock	24.0	1.40	1.10

Lateral Earth Pressure Coefficients

	Active K_a	At Rest K_0	Passive K_p
Compacted Earth Fill	0.40	0.55	2.50
Silty Clay Till/Sandy Silt Till	0.33	0.50	3.00
Shale Bedrock	0.20	0.30	5.00

Coefficients of Friction

Between Concrete and Granular Base	0.50
Between Concrete and Sound Natural Soils or Shale Bedrock	0.35

6.9 Excavation

Where excavation is to be carried out close to any existing underground structure or services, one must be aware that the previous backfill is amorphous in structure and is susceptible to sloughing and sudden side collapse. Extreme caution must be exercised and test pits should be used to evaluate the safety of such excavation. The existing services must be properly secured, where necessary.

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

Table 5 - Classification of Soils for Excavation

Material	Type
Sound Shale Bedrock	1
Sound native Tills and weathered Shale Bedrock	2
Earth Fill and weathered Soil	3

For excavation in shale bedrock, a cut slope steeper than 1H:1V can be allowed, provided that the bedding plane of the rock is relatively horizontal and any loose rocks protruding from the excavation are removed for safety.

Excavation into the weathered shale and the hard till containing boulders will require extra effort using mechanical means with a rock-ripper to facilitate the excavation. This method can generally be employed to excavate the weathered shale to a depth of 2.0± m below the bedrock surface. Efficient removal of the sound shale will require the aid of pneumatic hammering.

The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil. 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. A compressible material, such as sprayed foam, 100 mm in thickness, should be placed on the shale bedrock in order to slow down the disintegration if it will be exposed for more than a few weeks.

Continuous groundwater is not anticipated within the depth of excavation. Groundwater seepage, if any, will be slow in rate and limited in quantity, due to their relatively low to low permeability. Groundwater under subterranean artesian pressure may occur in places within the shale bedrock, which is generally considered to be a poor aquifer. Therefore, the yield of groundwater from the bedrock, if encountered, will be appreciable initially; however, if allowed to drain freely, it will often dissipate or be depleted with time. Any need for dewatering can be confirmed through the hydrogeological assessment.

In areas where a safe backing slope is not possible, the excavation has to be supported by shoring. The overburden load, the surcharge from adjacent structures and the hydrostatic pressure, if any, should be included in the design of the shoring. The design parameters and our recommendations are provided in the Appendix.

6.10 Monitoring of Performance

It is recommended that 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.

Due to the presence of nearby buildings, the foundation details of the adjacent structures must be investigated and incorporated into the design and construction of the proposed project. It is recommended that a pre-construction survey and a monitoring program be carried out for all adjacent structures in order to verify any potential future liability claims.

Vibration control and pre-construction survey is strongly recommended for the adjacent properties and structures prior to any excavation activities at the site. Further advice or undertaking of the vibration control and pre-construction survey can be provided as necessary.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the accounts of Star Seeker Inc., and for review by the designated consultants and government agencies. Use of the report is subject to the conditions and limitations of the contractual agreement.

The material in the report reflects the judgement of Kelvin Hung, P.Eng., and Kin Fung Li, P.Eng., 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 are 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

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows for each foot of penetration of a 2-inch diameter, 90° point cone driven by a 140-pound hammer falling 30 inches.

Plotted as '—●—'

Standard Penetration Resistance or 'N' Value:

The number of blows of a 140-pound hammer falling 30 inches required to advance a 2-inch O.D. drive open sampler one foot into undisturbed soil.

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/ft)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

Cohesive Soils:

Undrained Shear Strength (ksf)	'N' (blows/ft)	Consistency
less than 0.25	0 to 2	very soft
0.25 to 0.50	2 to 4	soft
0.50 to 1.0	4 to 8	firm
1.0 to 2.0	8 to 16	stiff
2.0 to 4.0	16 to 32	very stiff
over 4.0	over 32	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
☐	Compression test in laboratory

For a saturated cohesive soil, the undrained shear strength is taken as one half of the undrained compressive strength

METRIC CONVERSION FACTORS

1 ft = 0.3048 metres

1lb = 0.454 kg

1 inch = 25.4 mm

1ksf = 47.88 kPa

JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

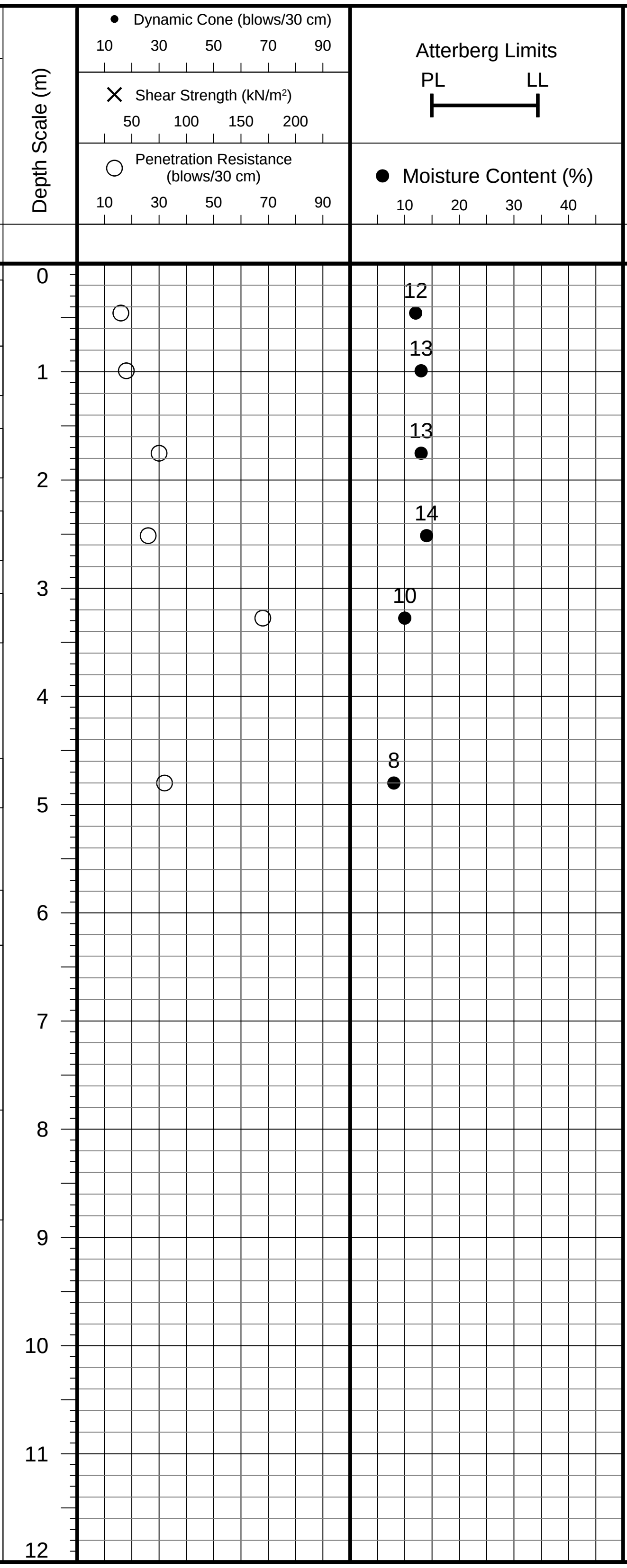
PROJECT DESCRIPTION: Proposed Mixed-Use Development

METHOD OF BORING: Solid Stem Augers and Rock Coring

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 30 and October 1, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	WATER LEVEL
82.6	Pavement Surface				
0.0	**150 mm ASPHALT CONCRETE** **150 mm GRANULAR FILL**				Dry and open on completion
	Stiff to very stiff weathered **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. sand seams and layers, cobbles and boulders brown grey	1 2 3 4	DO DO DO DO	16 18 30 26	
79.6 3.0	Grey, dense to very dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams and layers, cobbles and boulders shale fragments	5 6	DO DO	68 32	
77.0 5.6	Grey **SHALE BEDROCK** with limestone layers	7 8 9	RC RQD RC RQD RC RQD	100% 48% 100% 60% 100% 75%	
73.8 8.8	**END OF BOREHOLE**				



Soil Engineers Ltd.

Page: 1 of 1

JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

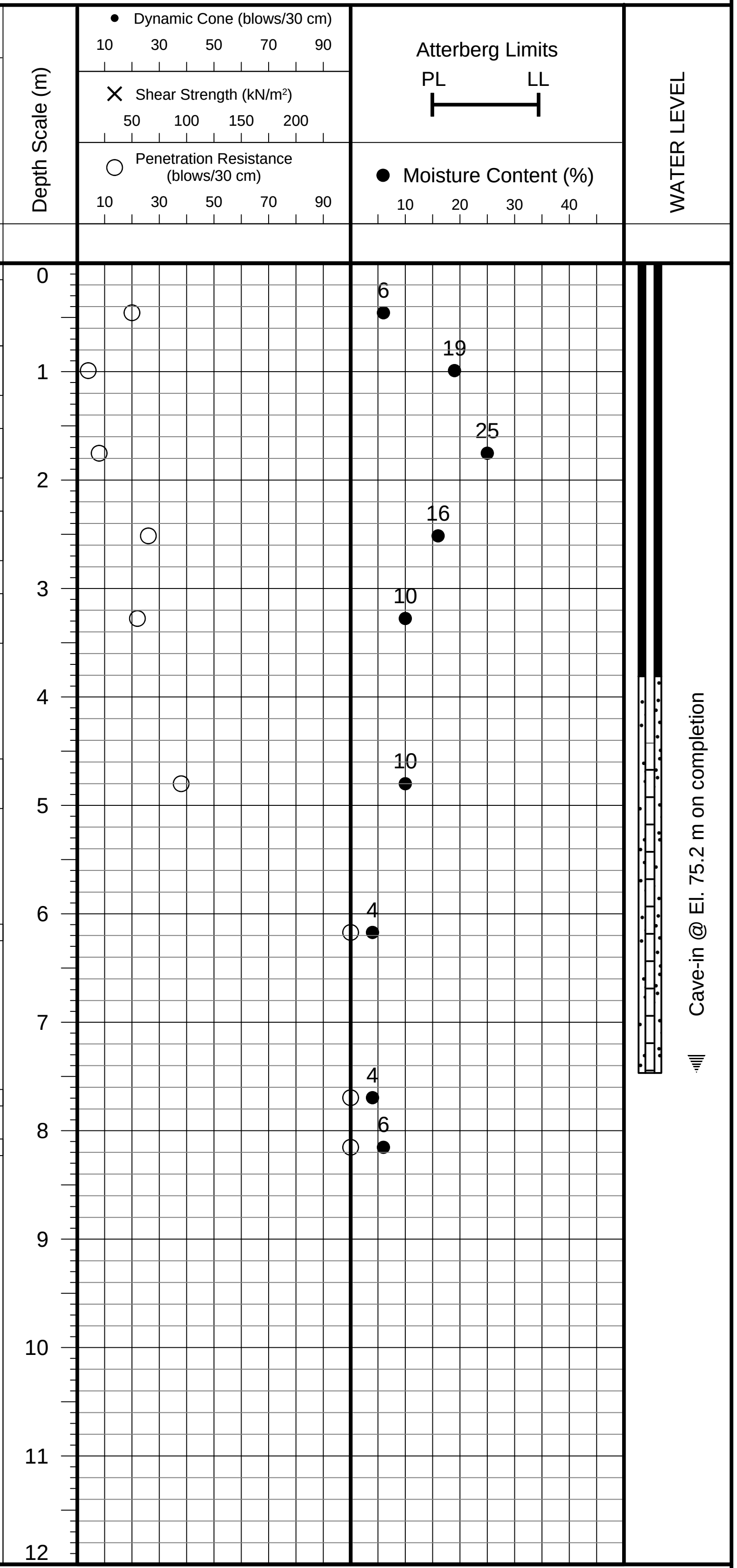
PROJECT DESCRIPTION: Proposed Mixed-Use Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: October 1, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
82.7	Pavement Surface			
0.0	**130 mm ASPHALT CONCRETE** **180 mm GRANULAR FILL** Dark brown **EARTH FILL** **(Silty Clay)** traces of sand and gravel with topsoil/organic inclusions/layers	1 2	DO DO	20 4
81.2 1.5	Brown, firm **SILTY CLAY TILL** traces of sand and gravel weathered	3	DO	8
80.3 2.4	Compact to dense **SANDY SILT TILL** brown grey a trace to some clay a trace of gravel occ. sand seams and layers, cobbles and boulders	4 5 6	DO DO DO	26 22 38
76.6 6.1	Grey **SHALE BEDROCK** with limestone layers	7 8 9	AS AS AS	50/15 50/15 50/15
74.5 8.2	**END OF BOREHOLE** **(Refusal to Auger)** Installed 50 mm Ø PVC monitoring well to 7.5 m (3.0 m screen) Sand backfill from 3.8 m to 7.5 m Bentonite holeplug from 0 m to 3.8 m Provided with a flushmount casing, with top and bottom caps and lock			



Soil Engineers Ltd.

JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

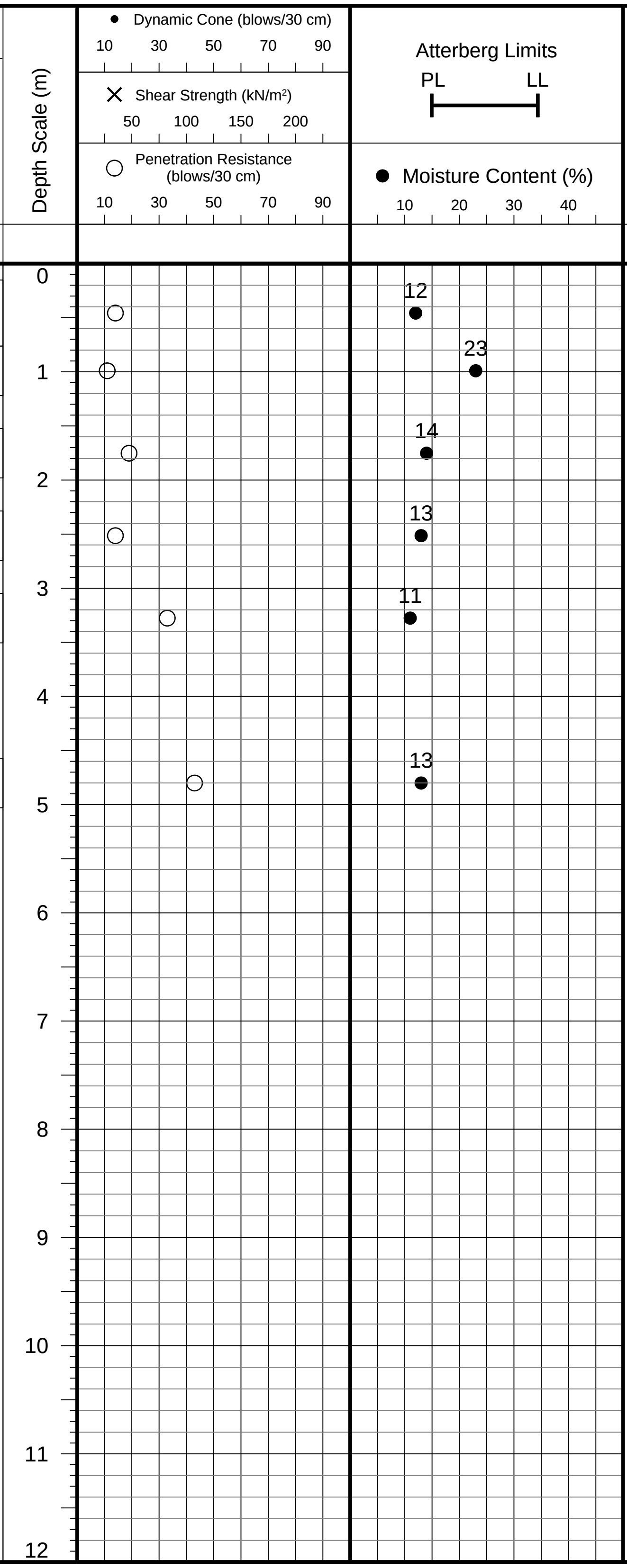
PROJECT DESCRIPTION: Proposed Mixed-Use Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 28, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	WATER LEVEL
82.3	Pavement Surface				
0.0	**100 mm ASPHALT CONCRETE** **130 mm GRANULAR FILL**	1	DO	14	Dry and open on completion
	Dark grey/dark brown **EARTH FILL** **(Silty Clay)** traces of sand and gravel with topsoil/organic inclusions/layer	2	DO	11	
80.8 1.5	Stiff to hard **SILTY CLAY TILL** brown grey traces of sand and gravel occ. sand seams and layers, cobbles and boulders	3	DO	19	
		4	DO	14	
		5	DO	33	
77.7 4.6	Grey, dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams and layers, cobbles and boulders	6	DO	43	
76.8 5.5	**END OF BOREHOLE** **(Refusal to Auger)**				



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Page: 1 of 1

JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 4

FIGURE NO.: 4

PROJECT DESCRIPTION: Proposed Mixed-Use Development

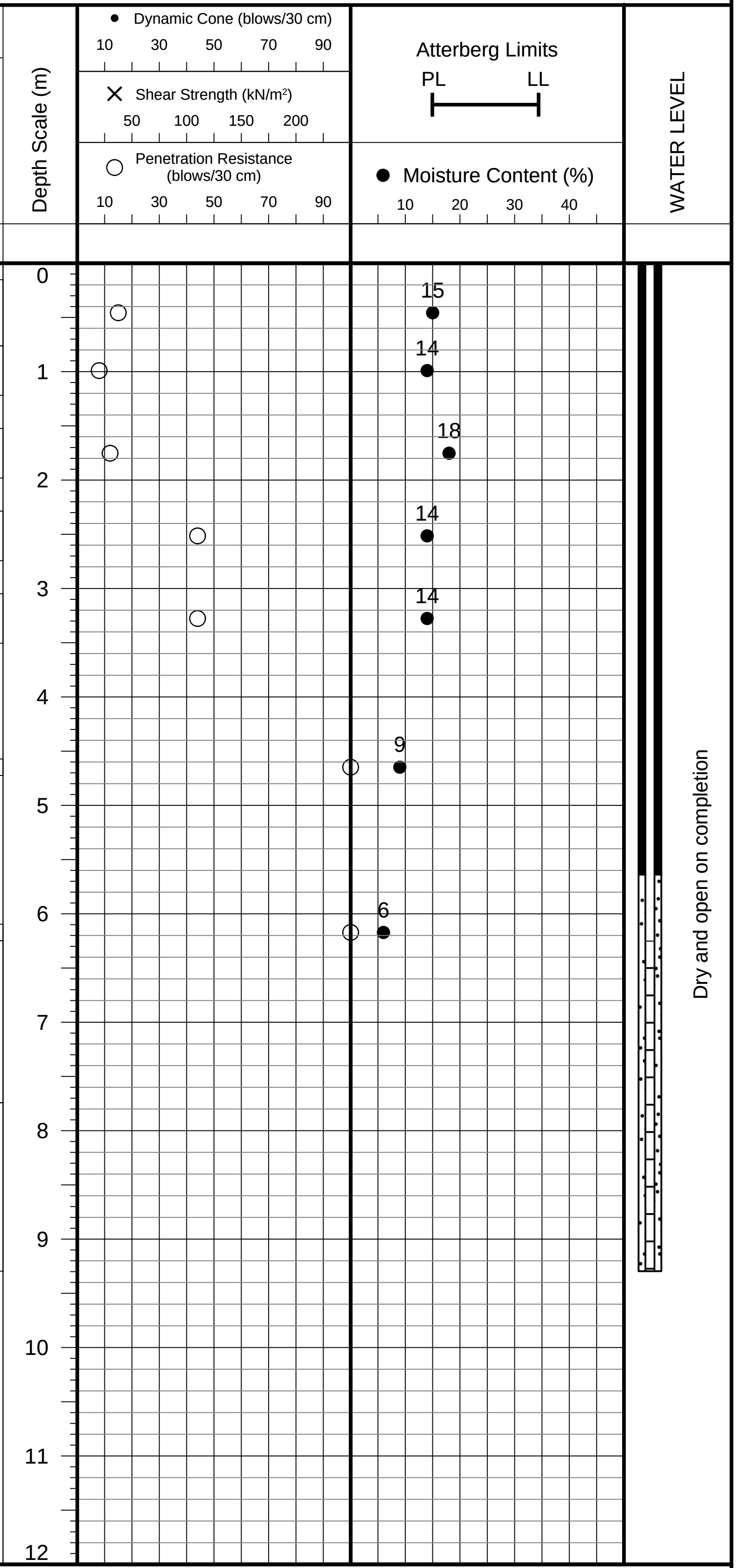
METHOD OF BORING: Solid Stem Augers and Rock Coring

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 30, 2021

El. (m) Depth (m)	SOIL DESCRIPTION
82.1	Pavement Surface
0.0	**200 mm ASPHALT CONCRETE** **250 mm GRANULAR FILL** Firm to hard **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. sand seams and layers, cobbles and boulders clay layer brown grey shale fragments
77.4 4.7	Grey **SHALE BEDROCK** with limestone layers
72.8 9.3	**END OF BOREHOLE** Installed 50 mm Ø PVC monitoring well to 9.3 m (3.0 m screen) Sand backfill from 5.6 m to 9.3 m Bentonite holeplug from 0 m to 5.6 m Provided with a flushmount casing, with top and bottom caps and lock

SAMPLES		
Number	Type	N-Value
1	DO	15
2	DO	8
3	DO	12
4	DO	44
5	DO	44
6	DO	50/15
7	AS	50/15
8	RC RQD	100% 54%
9	RC RQD	99% 88%



JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 5

FIGURE NO.: 5

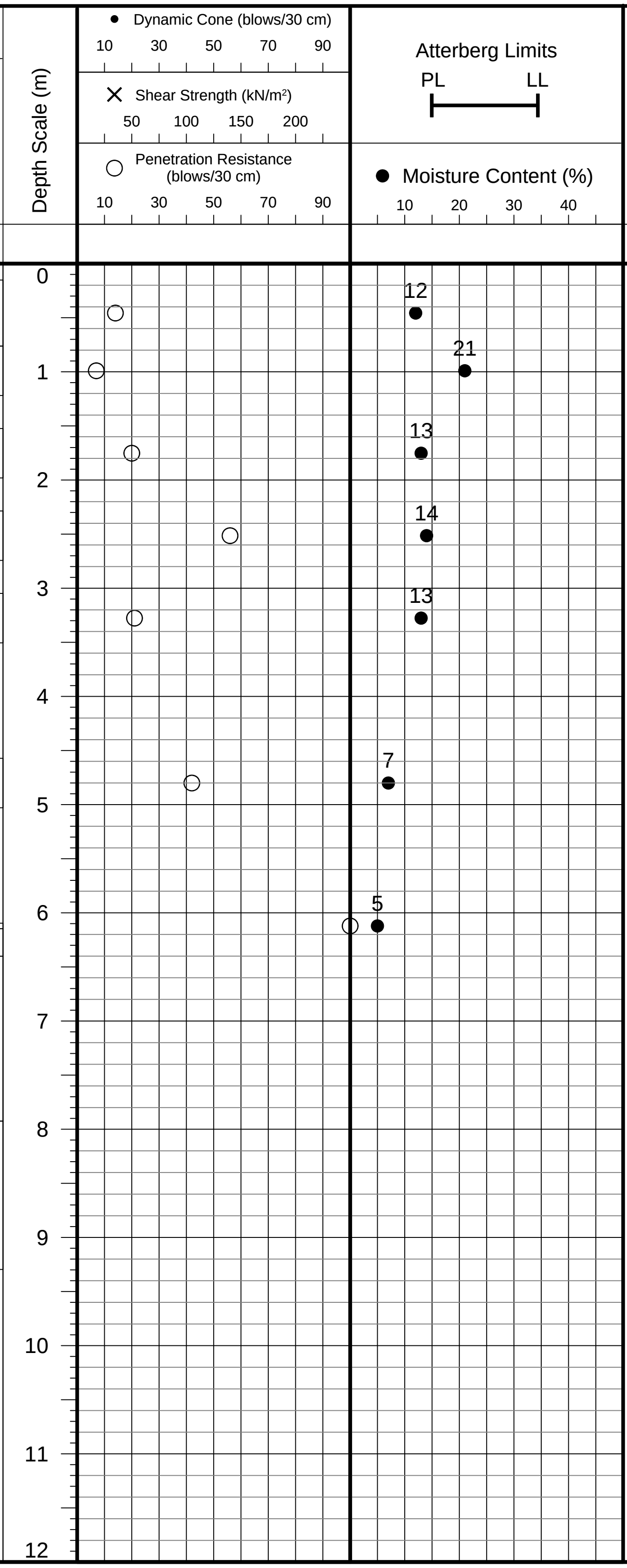
PROJECT DESCRIPTION: Proposed Mixed-Use Development

METHOD OF BORING: Solid Stem Augers and Rock Coring

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 29, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
82.4	Pavement Surface			
0.0	**100 mm ASPHALT CONCRETE** **150 mm GRANULAR FILL**			
	Brown/grey **EARTH FILL** **(Sandy Silt and Silty Clay)** a trace of gravel with topsoil/organic inclusions/layer	1	DO	14
		2	DO	7
80.7 1.7	Very stiff to hard **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. sand seams and layers, cobbles and boulders	3	DO	20
		4	DO	56
	— — brown grey	5	DO	21
77.8 4.6	Grey, dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams and layers, cobbles and boulders	6	DO	42
		7	AS	50/5
76.3 6.1	Grey **SHALE BEDROCK** with limestone layers	8	RC RQD	92% 0%
		9	RC RQD	100% 65%
		10	RC RQD	100% 53%
73.1 9.3	**END OF BOREHOLE**			



WATER LEVEL: Dry and open on completion

Soil Engineers Ltd.

LOG OF BOREHOLE NO.: 6

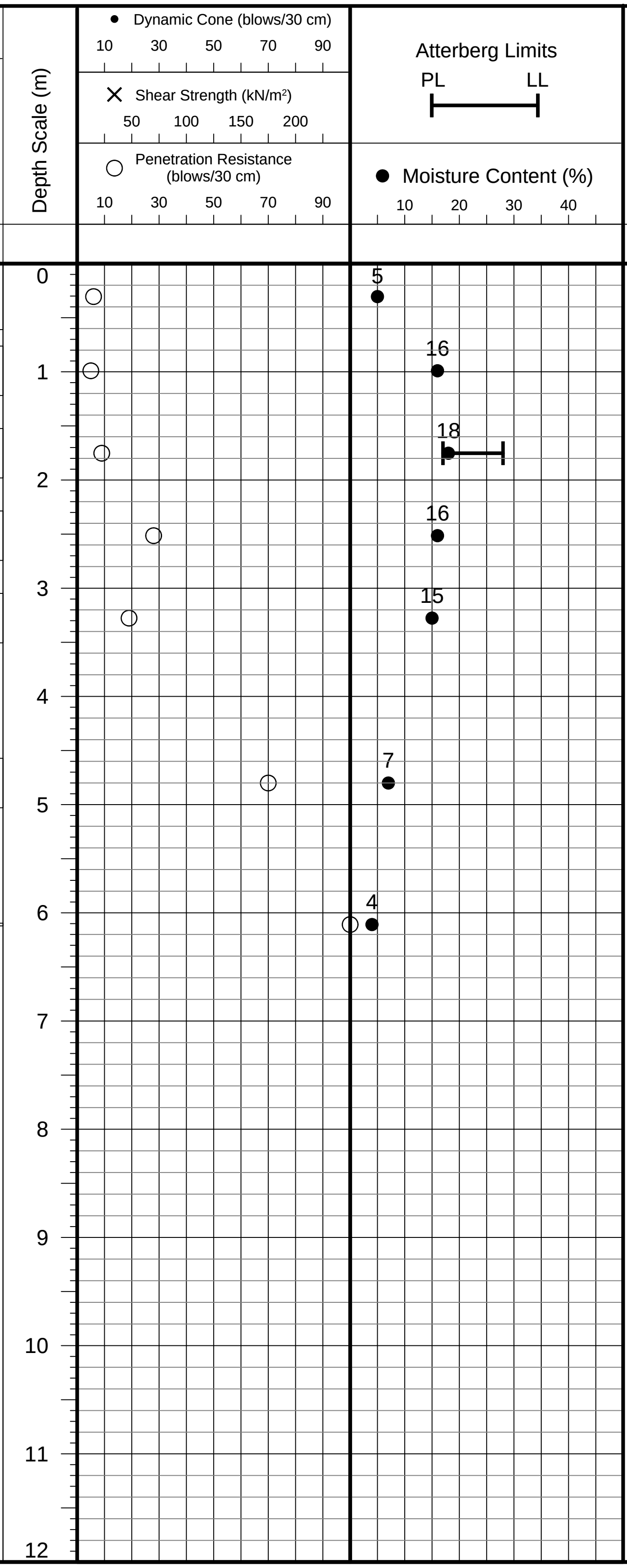
PROJECT DESCRIPTION: Proposed Mixed-Use Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 27, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
81.6	Pavement Surface			
0.0	**25 mm ASPHALT CONCRETE** **200 mm GRANULAR FILL** Brown **EARTH FILL** **(Sand and Silty Clay)** a trace of gravel with organic inclusions	1	DO	6
		2	DO	5
80.6 1.0	Brown, firm to very stiff clay layer **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. sand seams and layers, cobbles and boulders clay layer	3	DO	9
		4	DO	28
		5	DO	19
77.0 4.6	Grey, very dense **SANDY SILT TILL** some clay, a trace of gravel occ. sand seams and layers, cobbles and boulders	6	DO	70
75.5 6.1	Grey **SHALE BEDROCK** with limestone layers **END OF BOREHOLE**	7	AS	50/3



WATER LEVEL: Dry and open on completion

Soil Engineers Ltd.

JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 7

FIGURE NO.: 7

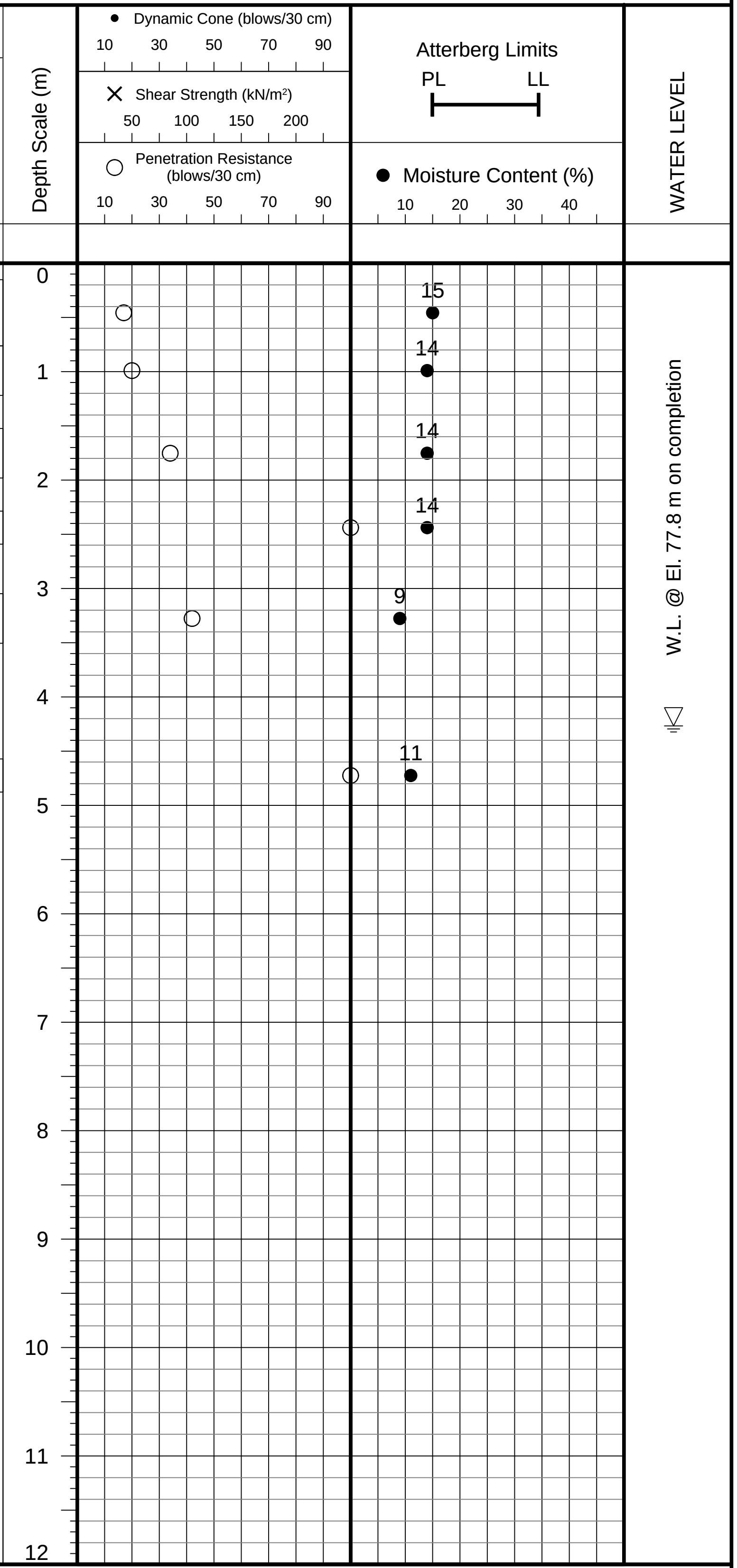
PROJECT DESCRIPTION: Proposed Mixed-Use Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 28, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
82.1	Pavement Surface			
0.0	**100 mm ASPHALT CONCRETE** **100 mm GRANULAR FILL**			
	Brown, very stiff to hard **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. sand seams and layers, cobbles and boulders	1	DO	17
		2	DO	20
		3	DO	34
		4	DO	50/15
79.1 3.0	Grey, dense to very dense **SANDY SILT TILL** a trace to some clay a trace of gravel occ. sand seams and layers, cobbles, boulders and shale fragments	5	DO	42
		6	DO	50/15
76.9 5.2	**END OF BOREHOLE** **(Refusal to Auger)**			



Soil Engineers Ltd.

JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 8

FIGURE NO.: 8

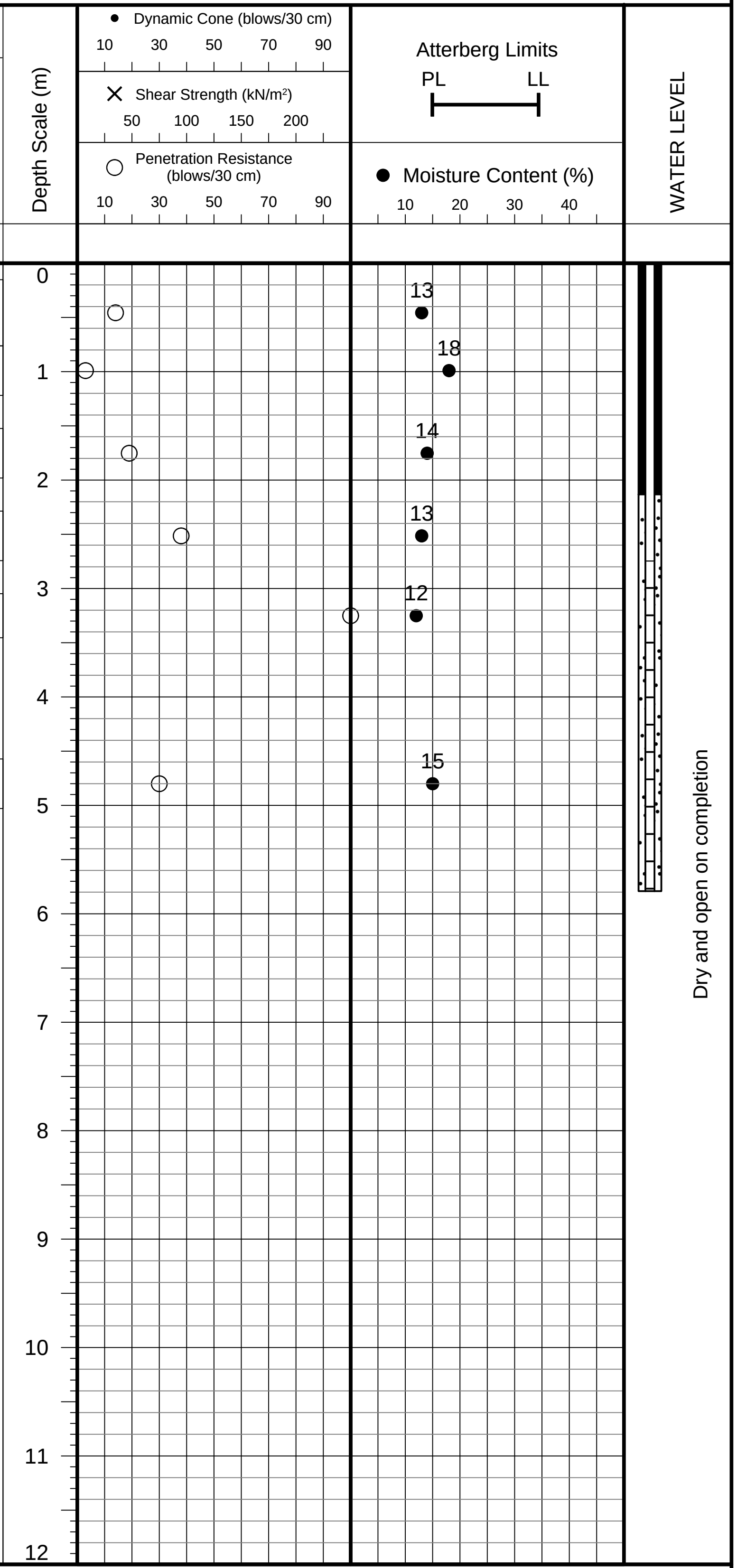
PROJECT DESCRIPTION: Proposed Mixed-Use Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 28, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
82.3	Pavement Surface			
0.0	**150 mm ASPHALT CONCRETE** **150 mm GRANULAR FILL**			
	Brown **EARTH FILL** **(Silty Clay and Silty Sand)** a trace of gravel organic inclusions	1 2	DO DO	14 3
80.8 1.5	Very stiff to hard **SILTY CLAY TILL** traces of sand and gravel occ. sand seams and layers, cobbles and boulders brown grey	3 4	DO DO	19 38
79.3 3.0	Grey, compact to very dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams and layers, cobbles, boulders and shale fragments	5 6	DO DO	66/25 30
76.5 5.8	**END OF BOREHOLE** **(Refusal to Auger)** Installed 50 mm Ø PVC monitoring well to 5.8 m (3.0 m screen) Sand backfill from 2.1 m to 5.8 m Bentonite holeplug from 0 m to 2.1 m Provided with a flushmount casing, with top and bottom caps and lock			



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JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 9

FIGURE NO.: 9

PROJECT DESCRIPTION: Proposed Mixed-Use Development

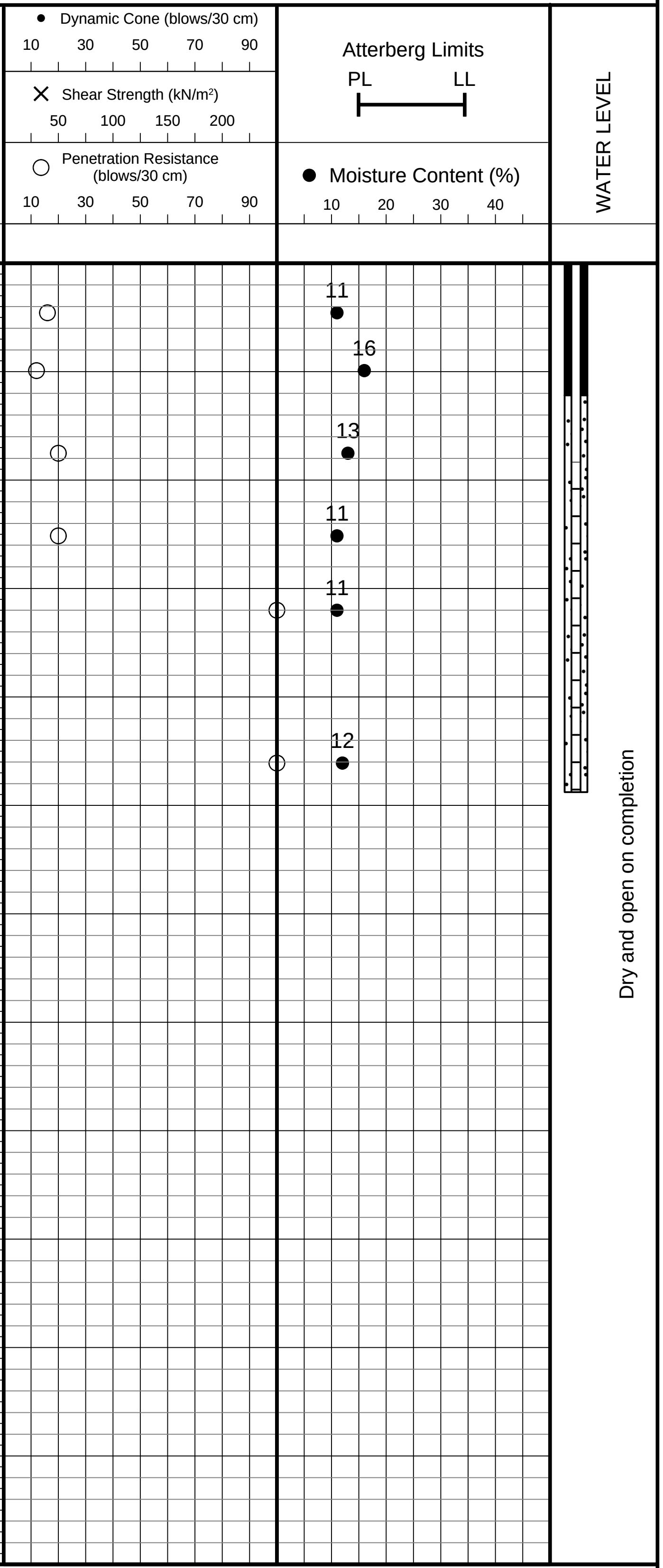
METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 28, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)
81.0	Pavement Surface				
0.0	**75 mm ASPHALT CONCRETE** **180 mm GRANULAR FILL**				0
	Brown **EARTH FILL** **(Silty Sand)** a trace of gravel	1	DO	16	
80.2 0.8		2	DO	12	1
	Stiff to hard brown / grey **SILTY CLAY TILL** traces of sand and gravel occ. sand seams and layers, cobbles and boulders	3	DO	20	
					2
		4	DO	20	
					3
		5	DO	50/15	
					4
		6	DO	50/8	
76.1 4.9	**END OF BOREHOLE** **(Refusal to Auger)**				5

Installed 50 mm Ø PVC monitoring well to 4.9 m (3.0 m screen)
Sand backfill from 1.2 m to 4.9 m
Bentonite holeplug from 0 m to 1.2 m
Provided with a flushmount casing, with top and bottom caps and lock



Dry and open on completion

JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 10

FIGURE NO.: 10

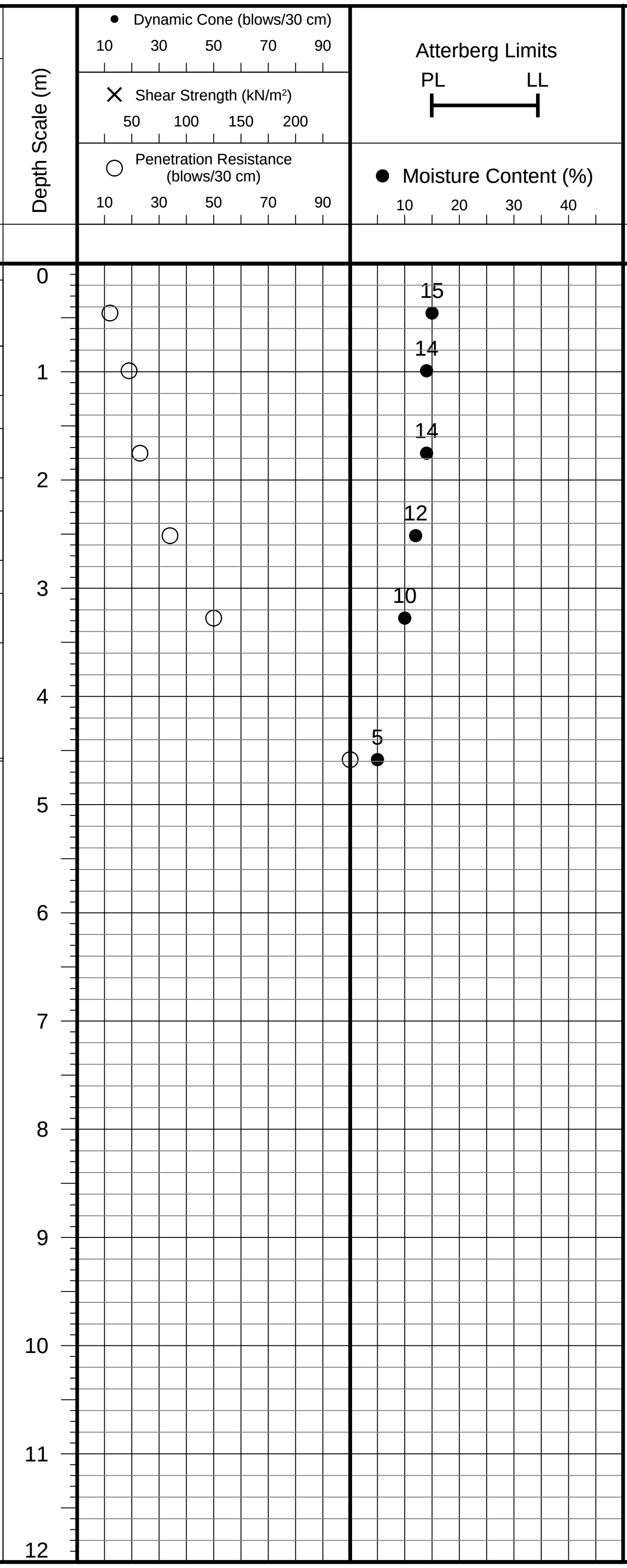
PROJECT DESCRIPTION: Proposed Mixed-Use Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 27, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
81.5	Pavement Surface			
0.0	**75 mm ASPHALT CONCRETE** **200 mm GRANULAR FILL**			
	Stiff to hard **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. sand seams and layers, cobbles and boulders — — weathered — — brown grey	1 2 3 4	DO DO DO DO	12 19 23 34
78.5 3.0	Grey, dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams and layers, cobbles, boulders and shale fragments	5	DO	50
76.9 4.6	Grey **SHALE BEDROCK** with limestone layers	6	AS	50/3
76.3 5.2	**END OF BOREHOLE** **(Refusal to Auger)**			



WATER LEVEL

Dry and open on completion

Soil Engineers Ltd.

JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 11

FIGURE NO.: 11

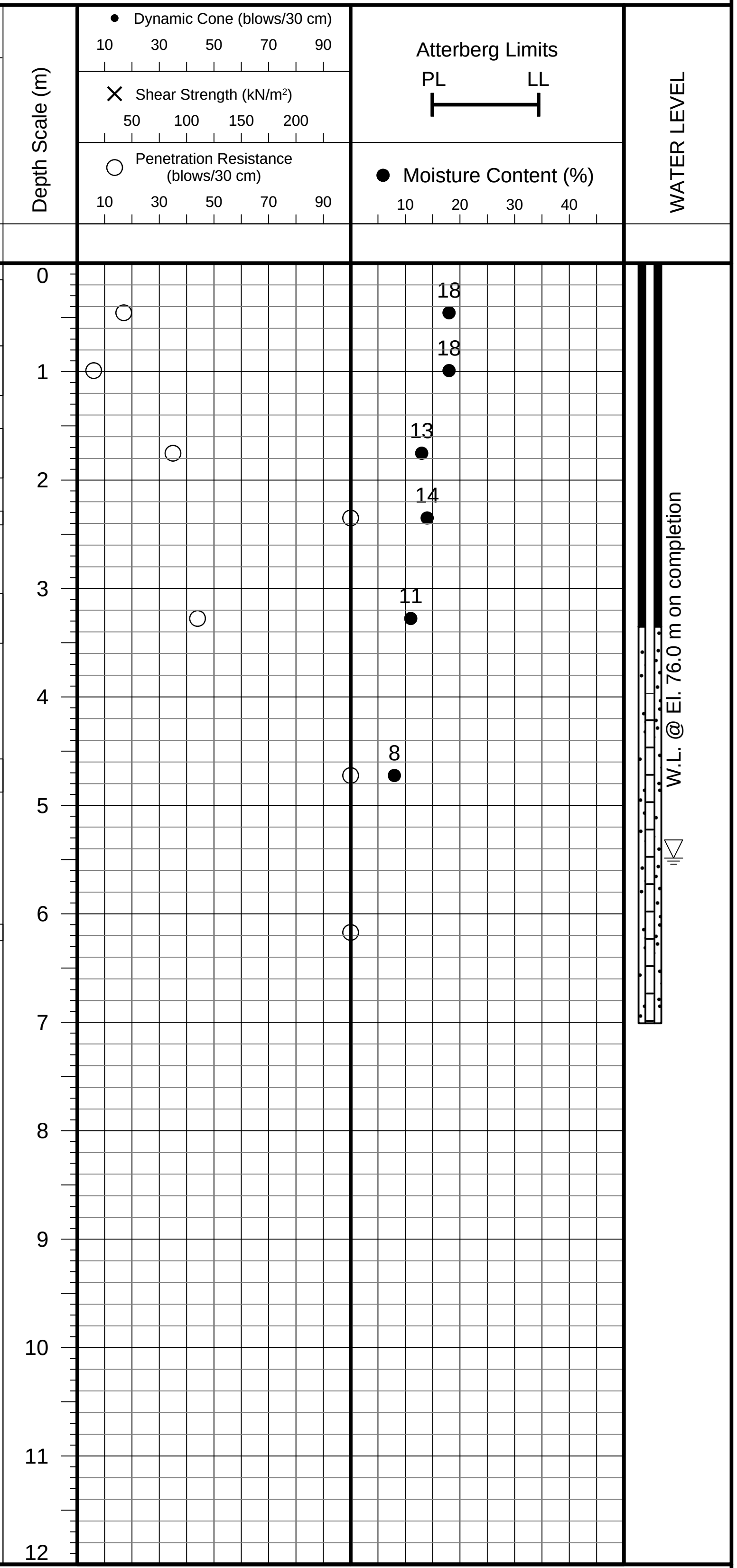
PROJECT DESCRIPTION: Proposed Mixed-Use Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 27, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
81.5	Pavement Surface			
0.0	**75 mm ASPHALT CONCRETE** **200 mm GRANULAR FILL**			
	Brown **EARTH FILL** **(Sand and Silty Clay)** a trace of gravel	1	DO	17
		2	DO	6
80.4 1.1	Brown, hard **SILTY CLAY TILL** traces of sand and gravel occ. sand seams and layers, cobbles and boulders	3	DO	35
		4	DO	50/13
78.5 3.0	Grey, dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams and layers, cobbles and boulders	5	DO	44
76.9 4.6	Grey **SHALE BEDROCK** with limestone layers	6	DO	50/15
		7	NR	50/15
74.5 7.0	**END OF BOREHOLE** **(Refusal to Auger)** Installed 50 mm Ø PVC monitoring well to 7.0 m (3.0 m screen) Sand backfill from 3.4 m to 7.0 m Bentonite holeplug from 0 m to 3.4 m Provided with a flushmount casing, with top and bottom caps and lock			



JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 12

FIGURE NO.: 12

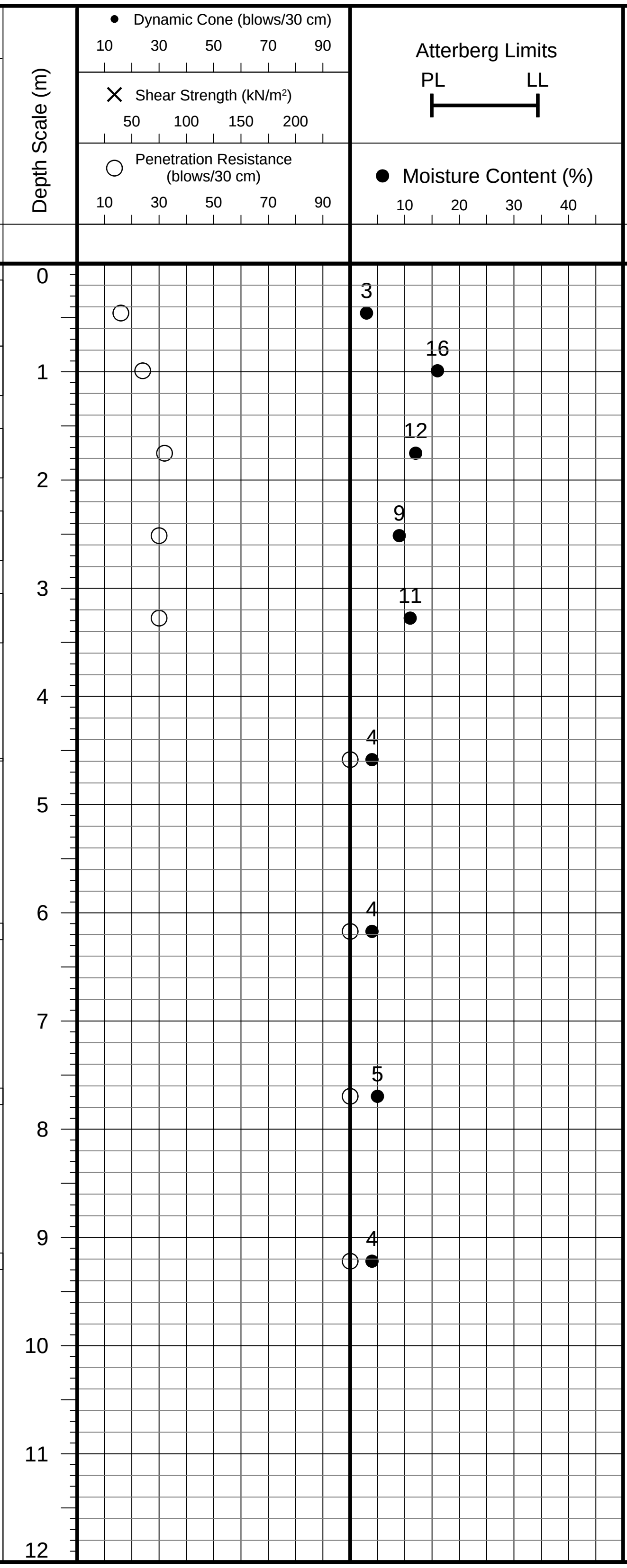
PROJECT DESCRIPTION: Proposed Mixed-Use Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

DRILLING DATE: September 28, 2021

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
80.8	Pavement Surface			
0.0	**200 mm ASPHALT CONCRETE** **150 mm GRANULAR FILL** Brown **EARTH FILL** **(Sand)** a trace of silt sandy silt layer/ brown grey	1	DO	16
80.0 0.8	Very stiff **SILTY CLAY TILL** traces of sand and gravel occ. sand seams and layers, cobbles and boulders	2 3	DO DO	24 32
78.5 2.3	Grey, compact **SANDY SILT TILL** a trace to some clay a trace of gravel occ. sand seams and layers, cobbles and boulders	4 5	DO DO	30 30
76.2 4.6	Grey **SHALE BEDROCK** with limestone layers	6 7 8 9	AS AS AS AS	50/3 50/15 50/15 50/15
71.0 9.8	**END OF BOREHOLE** **(Refusal to Auger)**			



WATER LEVEL

Dry and open on completion

Soil Engineers Ltd.

Page: 1 of 1

JOB NO.: 2102-S032

LOG OF BOREHOLE NO.: 13

FIGURE NO.: 13

PROJECT DESCRIPTION: Proposed Mixed-Use Development

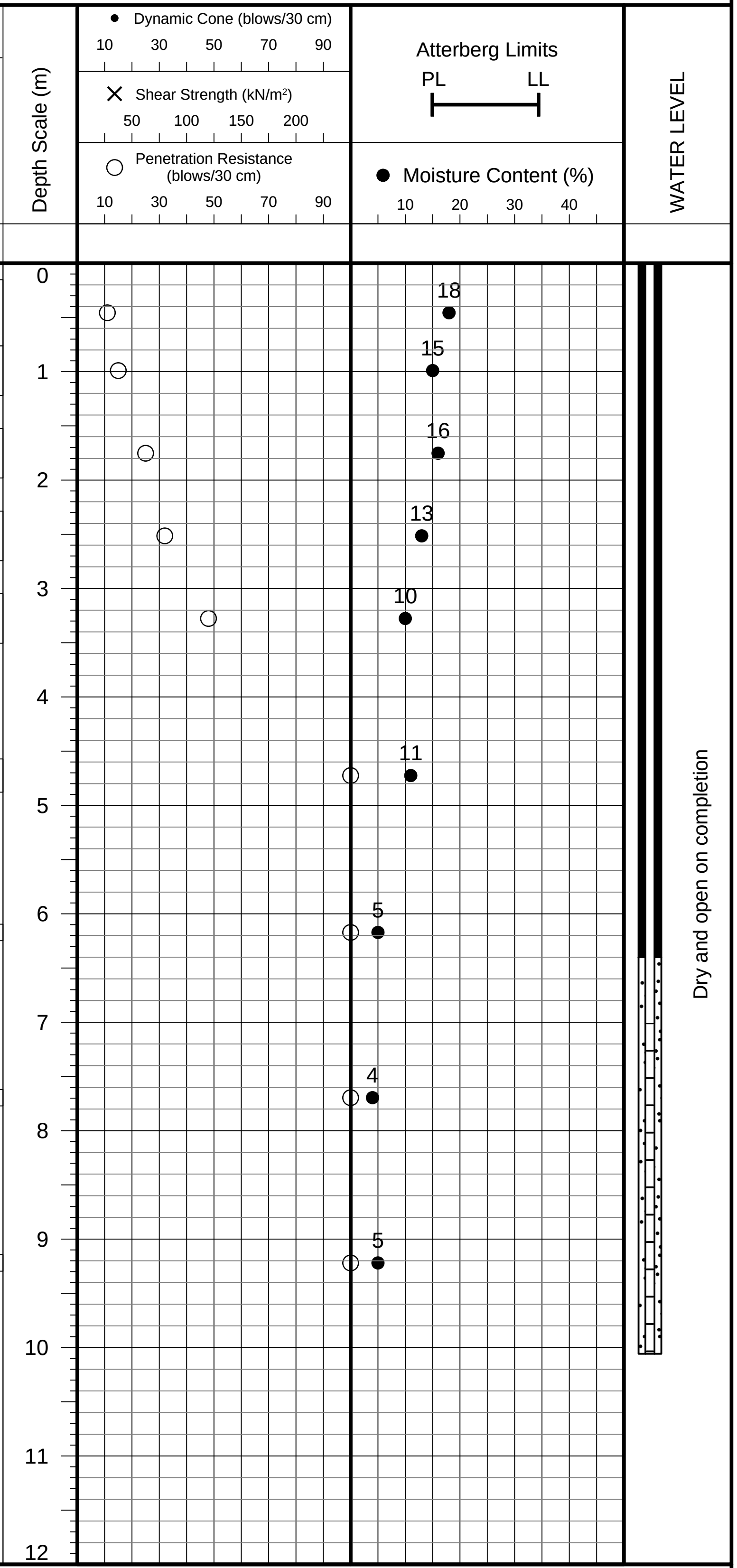
METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION:
579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
Northwest Quadrant of Lakeshore Road East and Caven Street
City of Mississauga

DRILLING DATE: September 27, 2021

El. (m) Depth (m)	SOIL DESCRIPTION
81.6	Pavement Surface
0.0	**75 mm ASPHALT CONCRETE** **150 mm GRANULAR FILL**
	Brown **EARTH FILL** **(Sand and Silty Clay)** a trace of gravel organic inclusions
80.1 1.5	Brown, very stiff **SILTY CLAY TILL** traces of sand and gravel occ. sand seams and layers, cobbles and boulders
78.6 3.0	Grey, dense to very dense **SANDY SILT TILL** a trace to some clay a trace of gravel occ. sand seams and layers, cobbles and boulders
75.5 6.1	Grey **SHALE BEDROCK** with limestone layers
71.5 10.1	**END OF BOREHOLE** **(Refusal to Auger)** Installed 50 mm Ø PVC monitoring well to 10.1 m (3.0 m screen) Sand backfill from 6.4 m to 7.0 m Bentonite holeplug from 0 m to 3.4 m Provided with a flushmount casing, with top and bottom caps and lock

SAMPLES Number	Type	N-Value
1	DO	11
2	DO	15
3	DO	25
4	DO	32
5	DO	48
6	DO	50/15
7	AS	50/15
8	AS	50/15
9	AS	50/15



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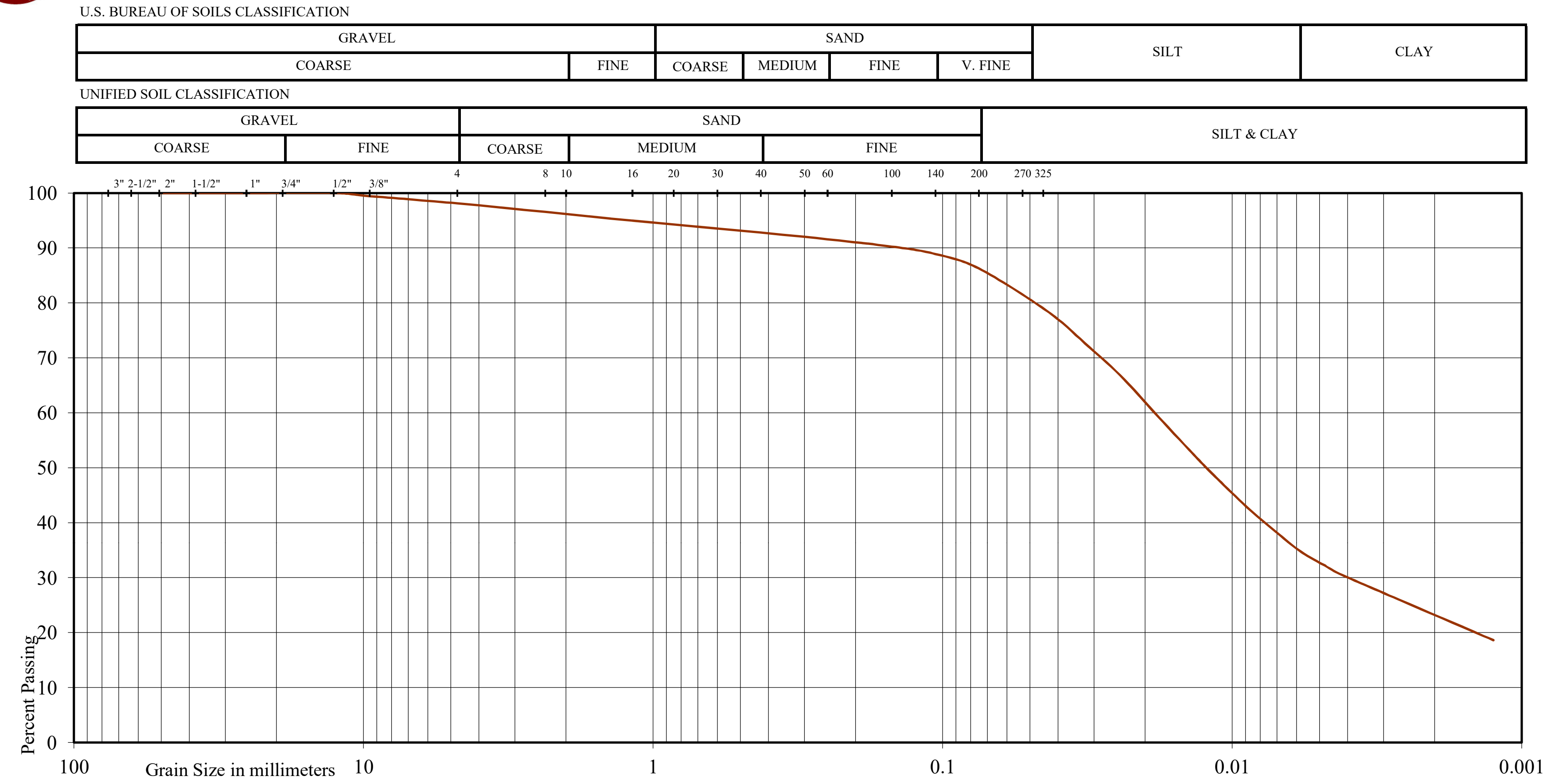
GRAIN SIZE DISTRIBUTION

Reference No: 2102-S032



Project: Proposed Mixed-Use Development

Location: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street

City of Mississauga

Borehole No: 6

Sample No: 3

Depth (m): 1.8

Elevation (m): 79.8

Liquid Limit (%) = 28

Plastic Limit (%) = 17

Plasticity Index (%) = 11

Moisture Content (%) = 18

Estimated Permeability

(cm./sec.) = 10^{-7}

Classification of Sample [& Group Symbol]: SILTY CLAY TILL

some sand, a trace of gravel

Figure: 14

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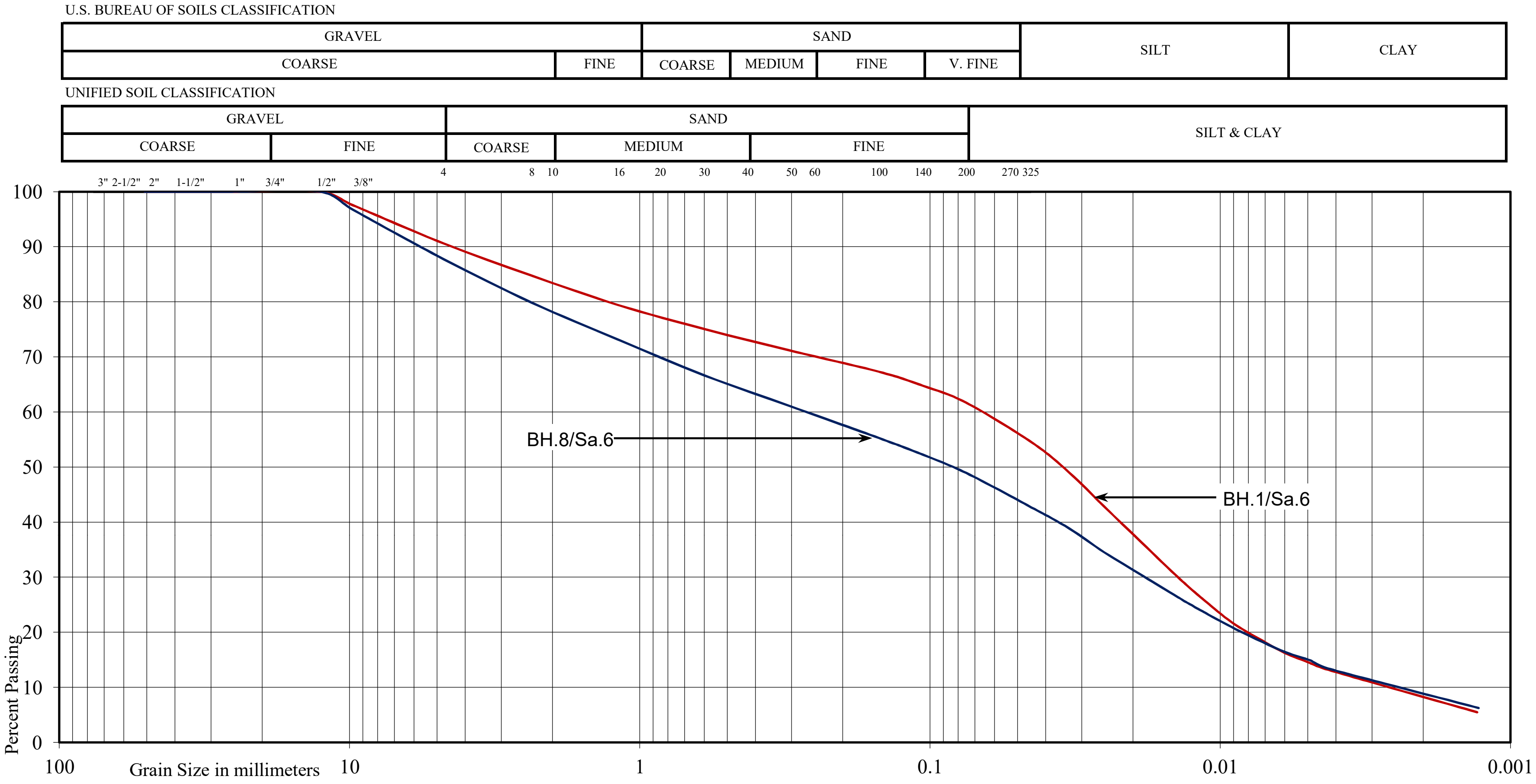
GRAIN SIZE DISTRIBUTION

Reference No: 2102-S032



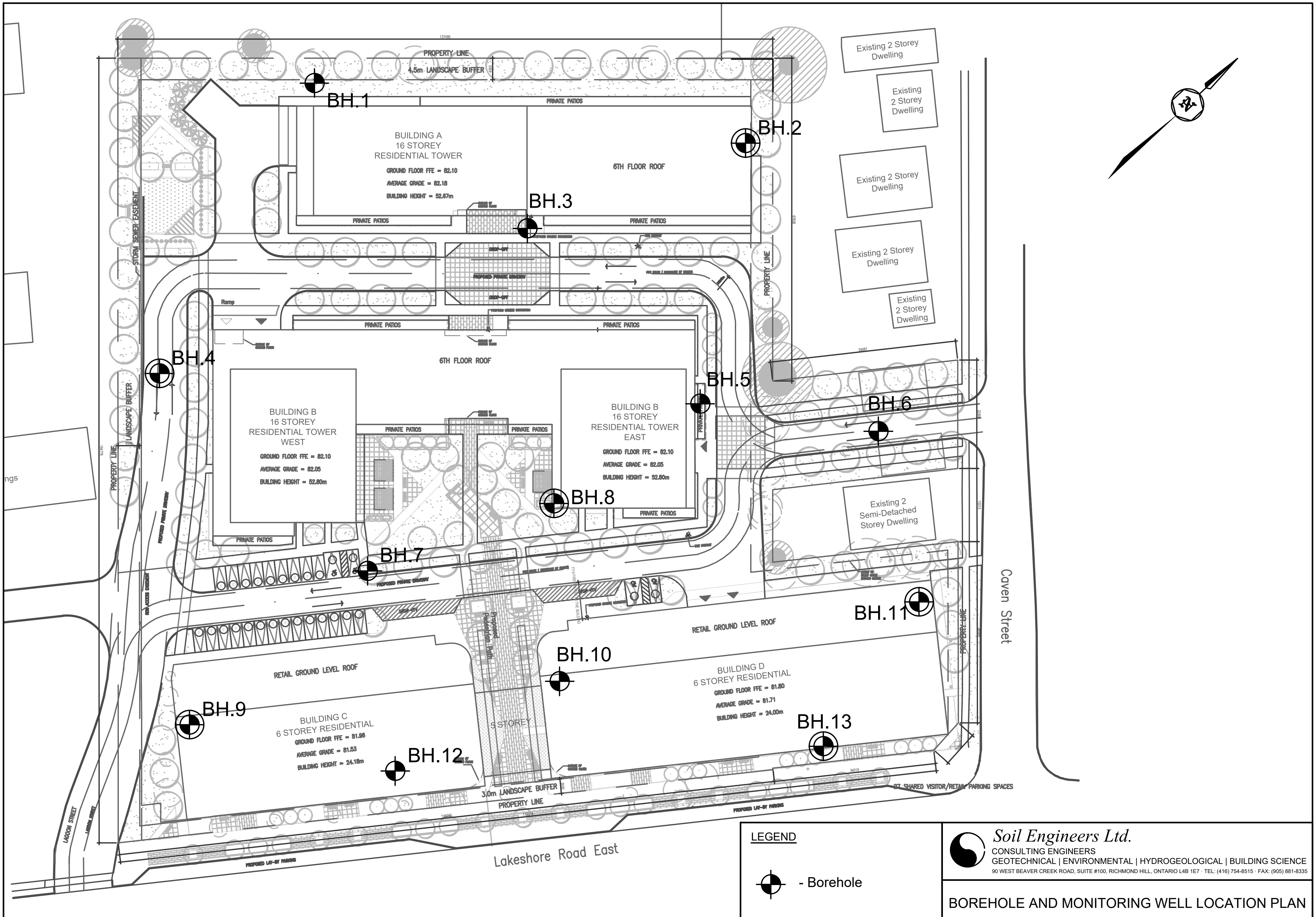
Project:	Proposed Mixed-Use Development	
Location:	579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street	
	City of Mississauga	
Borehole No:	1	8
Sample No:	6	6
Depth (m):	4.8	4.8
Elevation (m):	77.8	77.5

BH./Sa.	1/6	8/6
Liquid Limit (%) =	-	-
Plastic Limit (%) =	-	-
Plasticity Index (%) =	-	-
Moisture Content (%) =	8	15
Estimated Permeability (cm./sec.) =	10^{-5}	10^{-5}

Classification of Sample [& Group Symbol]: SANDY SILT TILL

traces of clay and gravel, occasional shale fragments

Figure: 15



LEGEND

⊕ - Borehole

⊕ - Borehole with monitoring well

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BOREHOLE AND MONITORING WELL LOCATION PLAN

SITE: 579 to 619 Lakeshore Road East and 1022 and 1028 Caven Street, City of Mississuaga

DESIGNED BY: -	**CHECKED BY:** -	**DWG NO.:** 1

SCALE: 1:1000	**REF. NO.:** 2102-S032	**DATE:** August 2022	**REV** 1

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SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

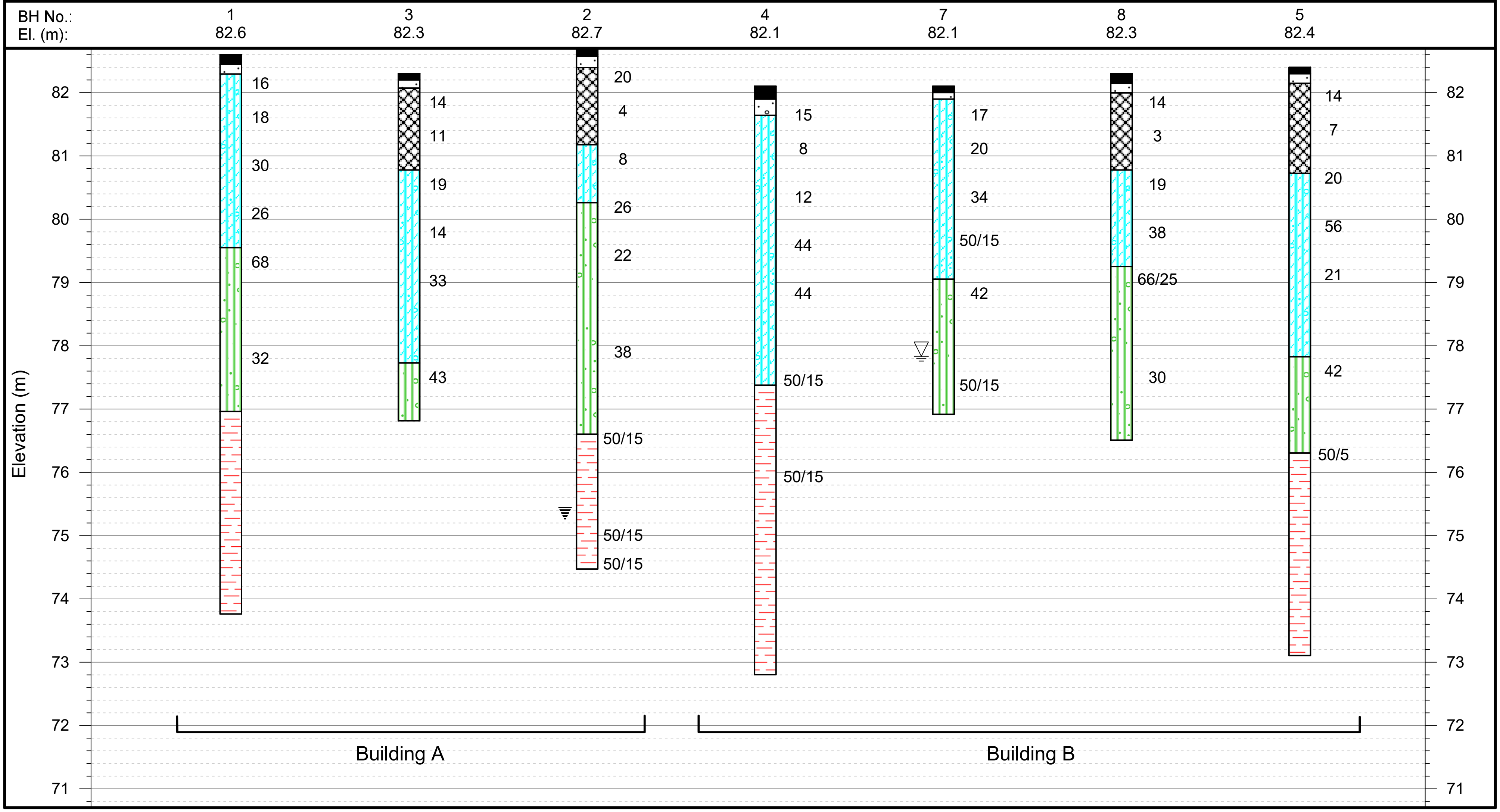
JOB NO.: 2102-S032
REPORT DATE: August 2022
PROJECT DESCRIPTION: Proposed Mixed-Use Development

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

LEGEND

■ ASPHALT
⁘ GRANULAR
▩ FILL
▥ SANDY SILT TILL
▨ SILTY CLAY TILL
▤ SHALE

▽ WATER LEVEL (END OF DRILLING)
≡ CAVE-IN



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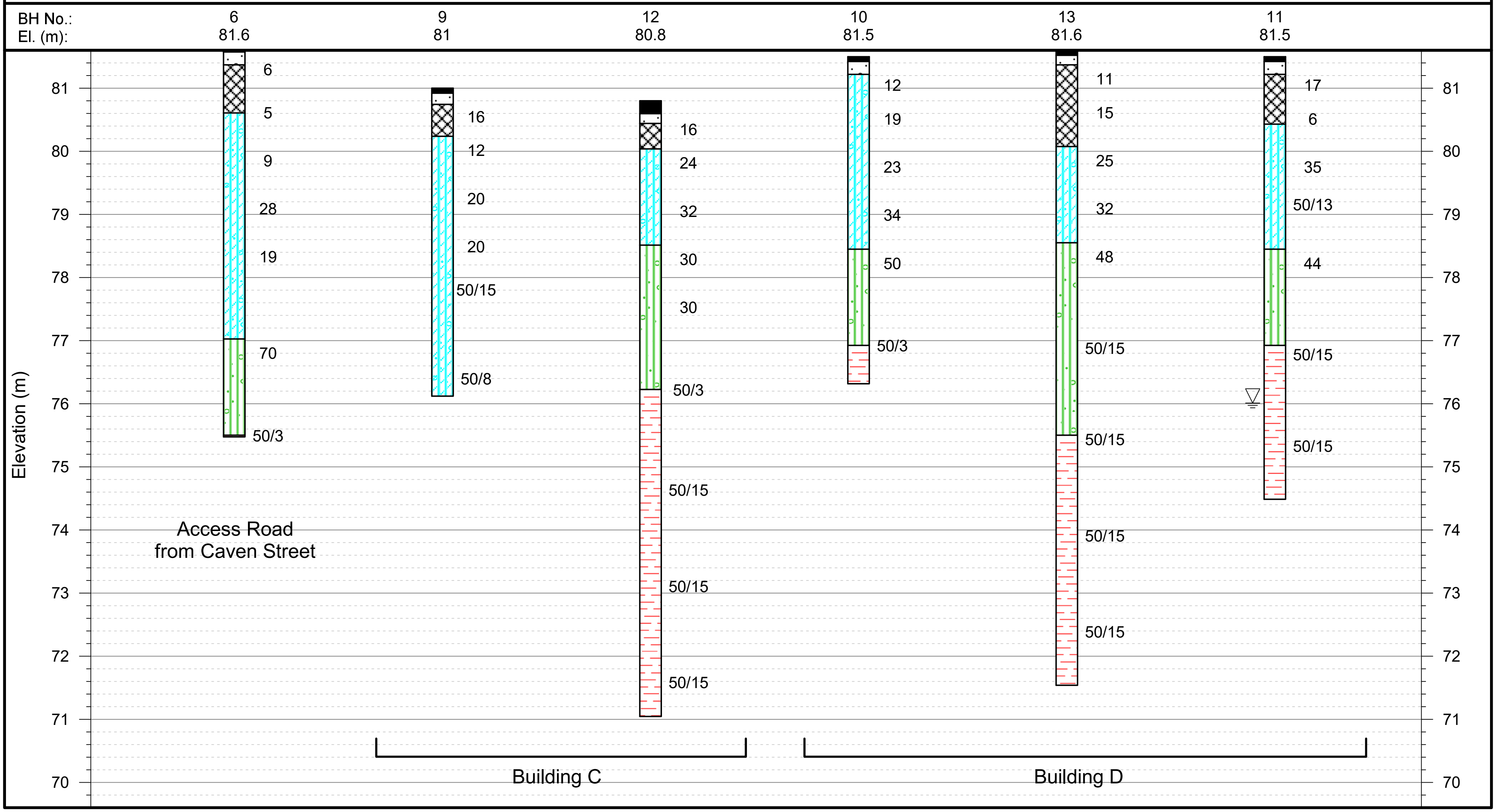
SUBSURFACE PROFILE
DRAWING NO. 3
SCALE: AS SHOWN

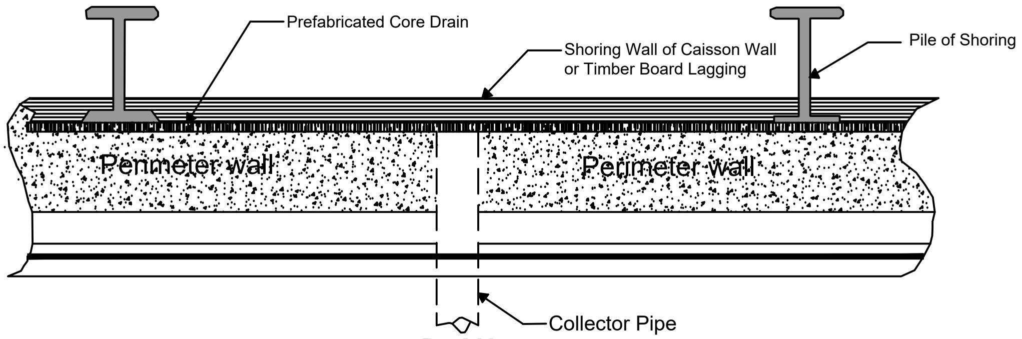
JOB NO.: 2102-S032
REPORT DATE: August 2022
PROJECT DESCRIPTION: Proposed Mixed-Use Development

PROJECT LOCATION: 579 to 619 Lakeshore Road East, and 1022 and 1028 Caven Street
City of Mississauga

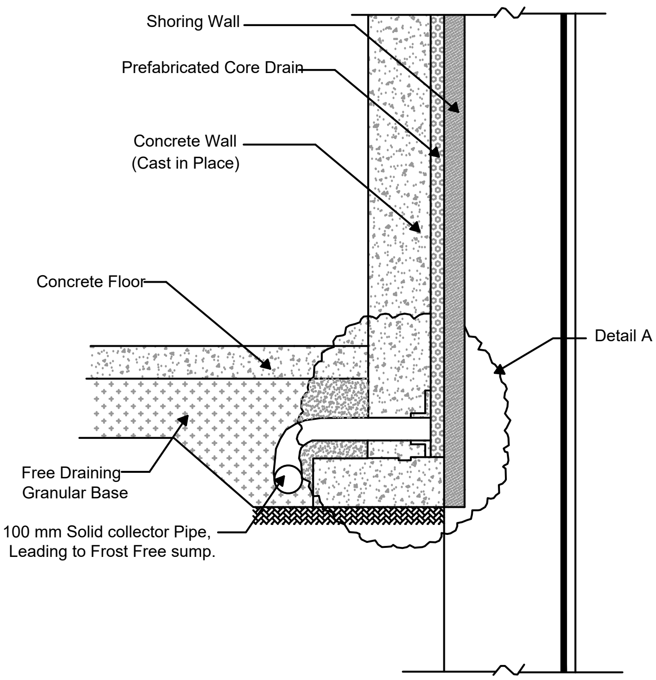
LEGEND

- ■ ASPHALT
- ▨ FILL
- ▥ SILTY CLAY TILL
- ⁘ GRANULAR
- ▥ SANDY SILT TILL
- ☰ SHALE
- ▽ WATER LEVEL (END OF DRILLING)

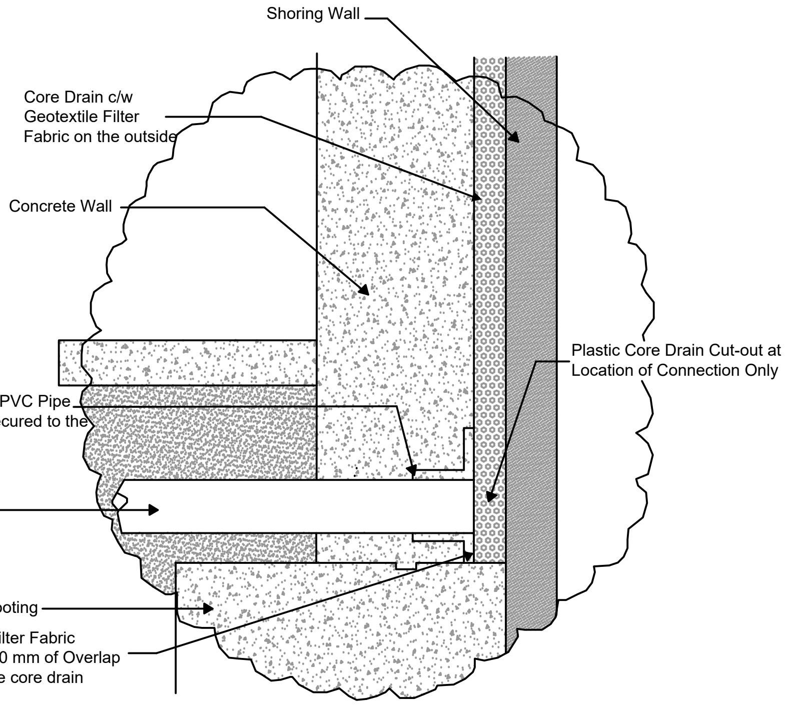




PLAN



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 ground surface.

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. Backfill behind the lagging board must be free draining. Filter fabric or straw should be used to prevent loss of fines behind the lagging.

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

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PERMANENT PERIMETER DRAINAGE SYSTEM
(WITH SHORING)

SITE: 579 TO 619 LAKESHORE ROAD EAST, AND 1022 AND 1028 CAVEN STREET
CITY OF MISSISSAUGA

DESIGNED BY: K.L.
CHECKED BY: B.S.
DWG NO.: 4
SCALE: N.T.S.
REF. NO.: 2102-S032
DATE: AUGUST 2022
REV -

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FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 542-2769

APPENDIX

SHORING DESIGN

REFERENCE NO. 2102-S032

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 deposits, 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 into shale bedrock should be at least 1.0 m below the bottom of excavation.

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 can be estimated using adhesion value of 60 kPa within the overburden and 600 kPa where the tieback anchor is grouted in shale. Full scale load tests should be carried out on the tieback anchors in each type of soils 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 in the weathered shale bedrock below the bottom of excavation should be designed for the allowable bearing pressure of 500 kPa. Raker footings extending into the sound shale can be designed for the allowable bearing pressure of 800 kPa.

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.

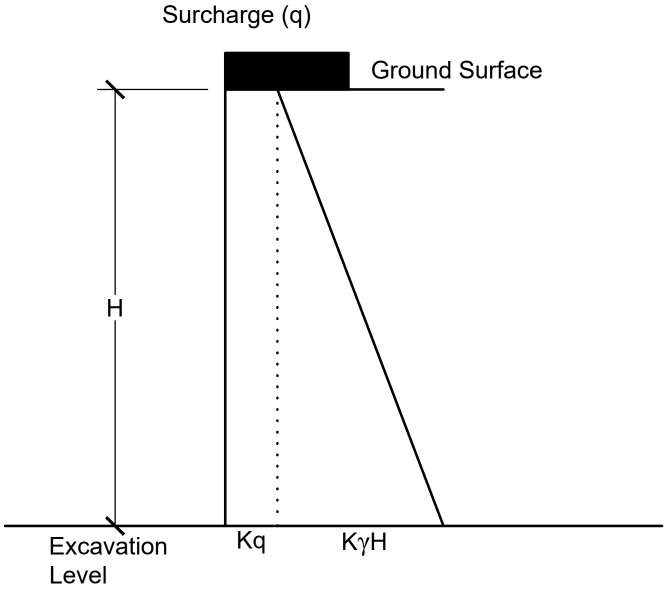
To prevent undermining of the raker footing, no excavation should be made within two times the width of raker footing on the opposite side of the raker.

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.

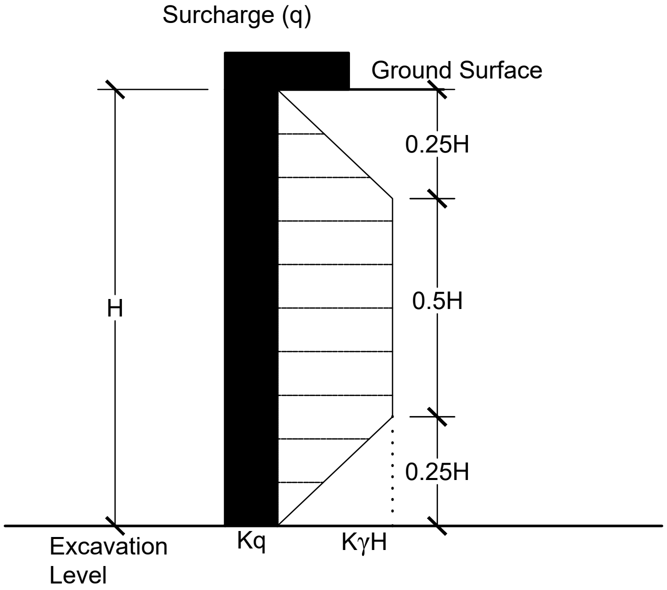
MONITORING OF PERFORMANCE

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.

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TEMPORARY SHORING
LATERAL EARTH PRESSURES

SITE: 579 TO 619 LAKESHORE ROAD EAST, AND 1022 AND 1028 CAVEN STREET
CITY OF MISSISSAUGA

DESIGNED BY: K.L. | **CHECKED BY:** B.S. | **DWG NO.:** A1

SCALE: N.T.S. | **REF. NO.:** 2102-S032 | **DATE:** AUGUST 2022 | **REV** -