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A REPORT TO
2216469 ONTARIO INC.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED OFFICE BUILDING

3650 EGLINTON AVENUE WEST

CITY OF MISSISSAUGA

REFERENCE NO. 2211-S176

OCTOBER 2023
(REVISION OF REPORT DATED FEBRUARY 2023)

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1.0 INTRODUCTION

In accordance with written authorization date November 27, 2022, from Mr. Gurpreet Paul, Director of 2216469 Ontario Inc., a geotechnical investigation was carried out in the property of 3650 Eglinton Avenue West 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 a proposed office building. The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The City of Mississauga is located on Halton-Peel till plain, where the subsoil consists of glacial drift overlying shale bedrock of Queenston or Georgian Bay Formation. In places, the drift has been modified by lacustrine sand, silt, clay and reworked till deposited by the water action of the glacial lake known as Peel Ponding.

The subject site is rectangular in shape, having a site area of 3907 m^2. It is located on the south side of Eglinton Avenue West, approximately 60 m west of Ridgeway Drive. At the time of investigation, the site consisted of a residential building and a detached shed/garage with paved parking and driveway at street level. The site gradient gently slopes to the north with minor undulation.

According to the architectural drawing (Drawing A 1.30), prepared by Caricari Lee Architects dated in August 2023, the proposed development will consist of the construction of a two-storey commercial/office building with on-grade drive lane and parking. The Finished Floor Level will be at El. 183.15 m.

3.0 FIELD WORK

The field work, consisting of four (4) sampled boreholes extending to the depths of 12.4 to 12.5 m from the prevailing ground surface, was performed between December 28 and 30, 2022, at the locations shown on the Location Plan, Drawing No. 1.

The boreholes were advanced at intervals to the sampling depths by track-mounted machine using solid stem auger and equipped with split spoon sampler for soil sampling. Split spoon samples were recovered for soil classification and laboratory testing. 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. The field work was supervised and the findings recorded by a Geotechnical Technician.

The ground elevation at each borehole location was obtained using a hand-held Global Navigation Satellite System (GNSS) equipment.

4.0 SUBSURFACE CONDITIONS

The investigation has disclosed that beneath the pavement structure or a topsoil veneer, and a layer of earth fill, the site is underlain by strata of glacial till extending to the maximum investigated depth of all boreholes.

Detailed descriptions of the subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 4, 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 Topsoil (Boreholes 2 to 4, inclusive)

The revealed topsoil at the ground surface in the boreholes is approximately 13 to 30 cm in thickness. Thicker topsoil layer may be contacted in areas beyond the borehole locations, particularly in treed areas.

4.2 Pavement Structure (Borehole 1)

Borehole 1 was carried out on the paved driveway, which consisted of 51 mm thick asphaltic concrete, overlying a layer of granular fill, approximately 150 mm in thickness.

4.3 **Earth Fill** (All boreholes)

A layer of earth fill was contacted below the topsoil and pavement structure in all boreholes, extending to depths of 1.0 to 2.1 m below the ground surface. The fill consists of a mixture of sand, silt and clay, topsoil and organic inclusions.

The obtained ‘N’ values range from 4 to 11, with a median of 7 blows per 30 cm of penetration, showing that the fill was placed with nominal compaction, but not in a uniform manner, and having partially self-consolidated.

The natural water content values of the fill samples range from 14% to 27%, with a median of 18%, indicating the fill is in moist to wet conditions.

4.4 **Glacial Till** (All Boreholes)

The glacial till, denoted as ‘sandy silt till’, ‘silty sand till’ and ‘silty clay till’, predominates the revealed stratigraphy in all boreholes beneath the earth fill. It extends to the termination depth of all boreholes. It consists of a random mixture of particle sizes ranging from clay to gravel. It contains occasional sand seams and layers, with cobbles and boulders. Grain size analyses were performed on a representative sample for each type of the glacial till; the results are plotted on Figures 5 to 7.

The obtained ‘N’ values (blows per 30 cm of penetration) of the glacial till along with its respective density/consistency are shown in Table 1.

Table 1 - Relative Density/Consistency of the Glacial Till

	N Values	**Relative Density/Consistency**
Sandy Silt Till	18 to 36 (median 33)	Compact to dense (generally dense)
Silty Sand Till	43 to over 50 (median over 50)	Dense to very dense (generally very dense)
Silty Clay Till	14 to 22 (median 18)	Stiff to very stiff (generally very stiff)

The Atterberg Limits of a representative sample of the silty clay till was determined. The resulting Liquid Limit and Plastic Limit are 24% and 16% respectively, showing a low plasticity.

The natural water content of the glacial till samples ranges from 7% to 13%, showing a moist condition.

The engineering properties of the glacial till are given below:

- High frost susceptibility and high soil-adfreezing potential.
- Low water erodibility.
- The till will be relatively stable in steep excavation; however, prolonged exposure may lead to localized sloughing.

5.0 GROUNDWATER CONDITION

The boreholes were checked for the presence of groundwater upon completion of drilling. Groundwater level in the monitoring wells was also recorded on January 18, 2023. The data are plotted on the Borehole Logs and summarized in Table 2 and Table 3.

Table 2 - Groundwater Levels (Upon Completion of Drilling)

Borehole No.	Ground Elevation (m)	Borehole Depth (m)	Measured Groundwater Level	
			Depth (m)	Elevation (m)
1	182.4	12.3	5.8	176.6
2	180.0	12.5	Dry	
3	181.7	12.6	9.5	172.2
4	182.3	12.6	Dry	

Table 3 - Groundwater Levels in Monitoring Wells

Borehole No.	Ground Elevation (m)	Well Depth (m)	Measured Groundwater Level	
			January 18, 2023	
			Depth (m)	Elevation (m)
1	182.4	12.3	7.26	175.14
2	180.0	12.5	Dry	
3	181.7	12.6	Dry	
4	182.3	12.2	Dry	

Upon completion of drilling, groundwater was recorded at the depth ranging from 5.8 to 9.5 m below grade or between El. 172.2 m and El. 176.6 m. In addition, groundwater was detected at 7.26 m below the prevailing ground surface, or at El. 175.14 m, in Borehole 1 on January 18, 2023. The remaining monitoring wells were dry.

Based on the groundwater findings, continuous groundwater is not anticipated within the depth of investigation. Localized perched water may be contacted in the sand/silt seams within the till. The water level may be subject to seasonal fluctuations.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath the pavement structure, a topsoil veneer, and a layer of earth fill, the site is underlain by strata of glacial till.

Upon completion of drilling, groundwater was recorded at the depth ranging from 5.8 to 9.5 m below grade or between El. 172.2 m and El. 176.6 m. In addition, groundwater was detected at 7.26 m below the prevailing ground surface, or at El. 175.14 m, in Borehole 1 on January 18, 2023. The remaining monitoring wells were dry.

It is understood that the proposed development will consist of the construction of a two-storey commercial/office building with on-grade drive lane and parking. The Finished Floor Level will be at El. 183.15 m. The geotechnical findings warranting special consideration for the proposed project are presented below:

1. After demolition of the existing structures, the demolition debris must be disposed offsite. The cavity must be backfilled with selected inorganic soils, which should be compacted properly for building and/or pavement support.
2. The existing earth fill is not suitable to support any structure that is sensitive to movement. It should be subexcavated, sorted free of any organics and/or deleterious material prior to be reused for structural backfill.
3. The proposed building can be supported on conventional strip and spread footings, founded on native sound soils. The foundation subgrade must be inspected to assess its suitability for bearing the designed foundations.
4. A Class ‘B’ bedding consisting of compacted 19-mm Crusher-Run Limestone (CRL), or equivalent, is recommended for the design of the underground services.
5. Excavation should be carried out in accordance with Ontario Regulation 213/91.

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

6.1 Site Preparation

After demolition of the existing structures, the demolition debris must be disposed offsite. The cavity must be backfilled with selected inorganic soils, compacted to at least 98% Standard Proctor Dry Density (SPDD) in lifts no more than 20 cm in thickness in order to provide building and/or pavement support.

6.2 Foundation

The proposed building can be supported on conventional strip and spread footings, founded on sound native soils below the frost penetration depth of at least 1.2 m from the exterior grade.

Based on the borehole findings, the recommended Maximum Allowable Soil Bearing Pressure at Serviceability Limit State (SLS) and the Factored Ultimate Soil Bearing Pressure at Ultimate Limit State (ULS) for the design of footings founded on the sound native soils are presented below.

- Maximum Allowable Soil Bearing Pressure (SLS) = 150 kPa
- Ultimate Soil Bearing Pressure (ULS) = 250 kPa

The total and differential settlements of the footings designed using bearing capacities at SLS are estimated to be 25 mm and 15 mm respectively.

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

Where existing earth fill was contacted at the foundation subgrade, extended footings can be considered by subexcavating the footing trench into the native soil stratum to a size of 30% wider than the designed footing width and filling with lean-mix concrete up to the underside of footing (USF) after suitable founding soil is exposed and inspected.

If groundwater seepage is encountered in footing excavations, or where the subgrade is found to be wet, the footing subgrade should also be protected by a concrete mud-slab immediately after exposure. This will prevent construction disturbance and costly rectification.

Foundations exposed to weathering or in unheated areas, should have at least 1.2 m of earth cover for protection against frost action.

The building foundations should meet the requirements specified in the latest Ontario Building Code and the structures should be designed to resist an earthquake force using Site Classification 'D' (stiff soil).

6.3 **Slab-On-Grade Construction**

The subgrade for slab-on-grade construction must consist of sound native soils or properly compacted inorganic earth fill. In preparation of the subgrade, the subgrade should be inspected and assessed by proof-rolling. Any weathered and/or loose/unsuitable soil should be subexcavated, sorted free of any deleterious material, aerated and uniformly compacted to at least 98% SPDD.

The concrete slab must be constructed on a minimum 15 mm thick granular bedding, consisting of 19-mm Crusher-Run Limestone (CRL), compacted to 100% SPDD.

The elevator pit, which normally extends a few metres below the lowest floor level, should be designed as a submerged 'tank' structure with waterproofed pit walls and pit floor. Otherwise, a drainage system connecting to a positive outlet, should be considered.

6.4 **Underground Services**

The underground services should be founded on sound native soil or properly compacted inorganic earth fill. Where incompetent or weathered soil is encountered, it should be subexcavated and replaced with the bedding material, compacted to at least 98% SPDD.

A Class 'B' bedding, consisting of 19mm CRL, or equivalent, is recommended for the underground service construction.

The pipe joints connecting into the manholes and catch basins must be leak-proof to prevent the migration of fines through the joints. Openings to subdrains and catch basins should be shielded with a fabric filter to prevent blockage by silting.

A soil cover, having a thickness equal to two times the diameter of the pipe should be in place at all times after pipe installation to prevent pipe floatation when the trench is deluged with water derived from precipitation.

For estimation for the anode weight requirements, the electrical resistivities of the disclosed soils can be used. The proposed anode weight must meet the minimum requirements as specified by the municipality standard.

6.5 Backfilling in Trenches and Excavated Areas

Most of the on-site inorganic soils, where it is free of concentrated topsoil and deleterious materials, are suitable for trench backfill. However, the backfill material should be sorted free of boulders or oversized rock pieces (over 15 cm in size). Where spaces are limited for stockpiling the on-site excavated material, the soil should be disposed off-site and replaced with verified exported fill when backfilling is required.

The backfill in service trenches should be compacted to at least 98% SPDD, particularly in the zone within 1.0 m below the pavement or slab-on-grade. The material should be compacted with the water content at 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 and services crossings, foundation walls and columns. In areas which are inaccessible to a heavy compactor, sand backfill should be used and compacted with lighter equipment.

6.6 Pavement Design

The recommended pavement design for on-grade parking and driveway is presented in Table 4.

Table 4 - Pavement Design (On-Grade Parking and Driveway)

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

In preparation of the pavement subgrade, it should be proof-rolled; any soft subgrade identified should be subexcavated and replaced by the properly compacted organic-free on-site material. In the zone within 1.0 m below the pavement subgrade, the backfill should be compacted to at least 98% SPDD, with the water content 2% to 3% drier than the optimum. All the granular bases should be compacted to 100% SPDD.

Along the perimeter where surface runoff may drain onto the pavement, or water may seep into the granular base, a swale or an intercept subdrain system should be installed. Subdrains, consisting of filter-wrapped weepers, should also be installed below the granular sub-base in the low area such as around catch basins. They should be connected to the catch basins and storm manholes in the paved areas and backfilled with free-draining granular material.

6.7 Soil Parameters

The recommended soil parameters for the project design are given in Table 5.

Table 5 - Soil Parameters

Unit Weight and Bulk Factor	**Bulk Unit Weight (kN/m^3)**	**Estimated Bulk Factor**	
		Loose	**Compacted**
Earth Fill	21.5	1.20	1.00
Glacial Till	22.5	1.33	1.05

Lateral Earth Pressure Coefficients	**Active**	**At Rest**	**Passive**
	K_a	**K_o**	**K_p**
Compacted Earth Fill	0.40	0.56	2.56
Glacial Till	0.33	0.50	3.00

Estimated Coefficients of Permeability	K (cm/sec)
Sandy Silt Till/Silty Sand Till	10^{-5} to 10^{-6}
Silty Clay Till	10^{-7}

Estimated Electrical Resistivity	Ω (ohm·cm)
Sandy Silt Till/Silty Sand Till	4500
Silty Clay Till	3500

Coefficients of Friction	
Between Concrete and Granular Base	0.50
Between Concrete and Sound Native Soils	0.35

6.8 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. The types of soils are classified in Table 6.

Table 6 - Classification of Material for Excavation

Material	Type
Sound Till	2
Earth Fill and Weathered soil	3

If water seepage is encountered during excavation, the water yield will be slow in rate and limited in quantity, and it can be controlled by conventional pumping from sumps.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account 2216469 Ontario Inc., and for review by the designated consultants and government agencies. The material in the report reflects the judgment of Daric Yang, B.A.Sc. and Kin Fung Li, P.Eng., in light of the information available to it at the time of preparation.

Use of the report is subject to the conditions and limitations of the contractual agreement. Any uses which a Third Party makes of this report, and/or any reliance on decisions to be made based on it 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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LICENSED PROFESSIONAL ENGINEER
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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 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 '—•—'

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 '○'

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)	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 (kPa)	'N' (blows/30 cm)	Consistency
less than 12	less than 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
over 200	over 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

JOB NO.: 2211-S176

LOG OF BOREHOLE: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Office Building

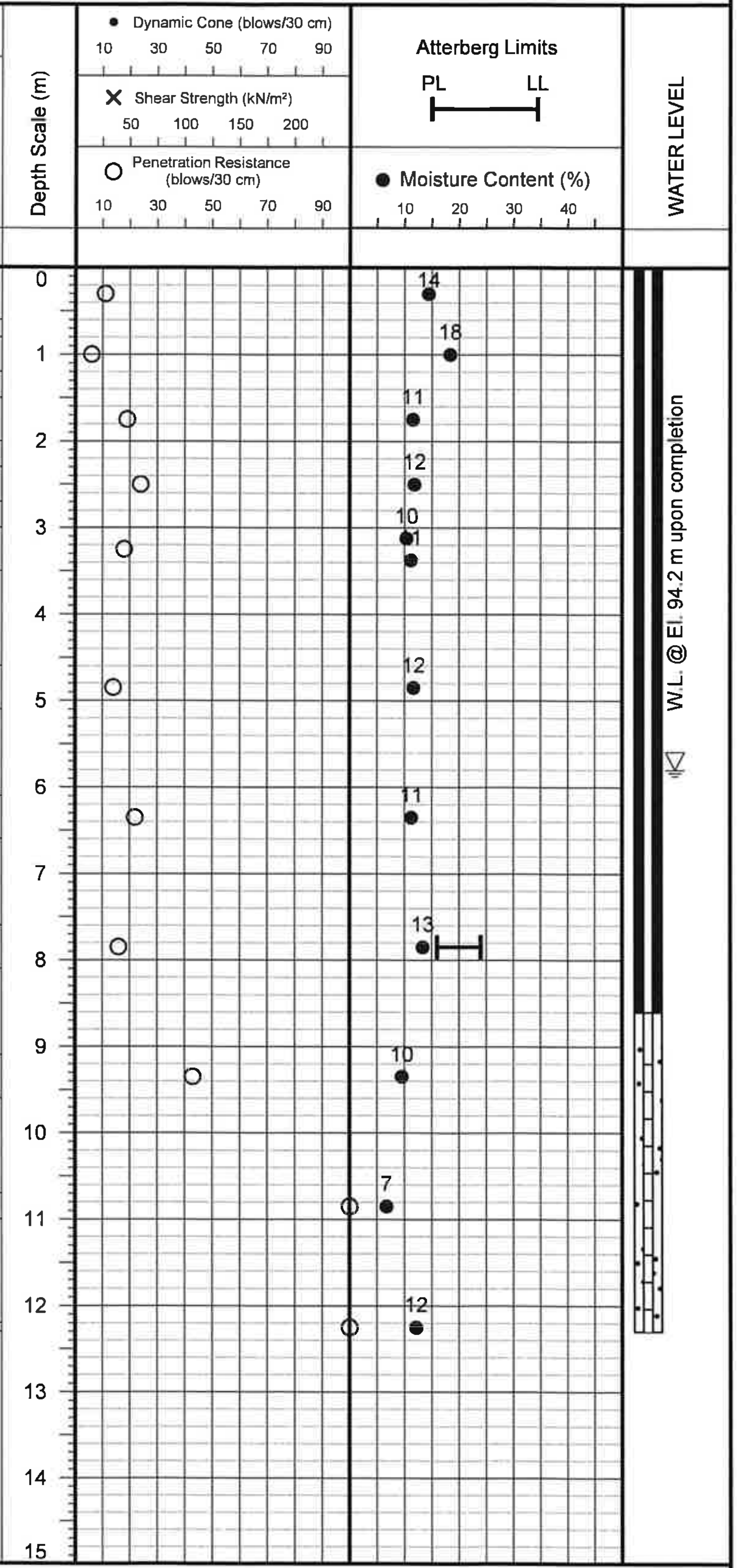
METHOD OF BORING: Solid Stem

PROJECT LOCATION: 3650 Eglinton Avenue West, City of Mississauga

DRILLING DATE: December 28, 2022

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES		
		Number	Type	N-Value
182.4	Ground Surface			
0.0	**50 mm Asphalt** **150 mm Granular Fill**	1	DO	11
	EARTH FILL Dark brown sandy silt to brown silty clay a trace of sand occ. topsoil inclusion	2	DO	6
180.9 1.5	Brown, compact **SANDY SILT TILL** a trace of gravel occ. sand seams and layers, and cobbles	3	DO	19
		4	DO	24
179.2 3.3	Grey, stiff to very stiff **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers, and cobbles	5A 5B	DO	18
		6	DO	14
		7	DO	22
		8	DO	16
173.9 8.5	Grey, dense to very dense **SILTY SAND TILL** a trace of gravel occ. cobbles and boulders	9	DO	43
		10	DO	50/15
		11	DO	50/15
170.1 12.3	**END OF BOREHOLE**			

Installed 50 mm Ø monitoring well to 12.3 m
completed with 3.1 m PVC slotted screen
Sand backfill from 8.6 to 12.3 m
Bentonite seal from 0.0 m to 8.6 m
Provided with a flush mount



JOB NO.: 2211-S176

LOG OF BOREHOLE: 2

FIGURE NO.: 2

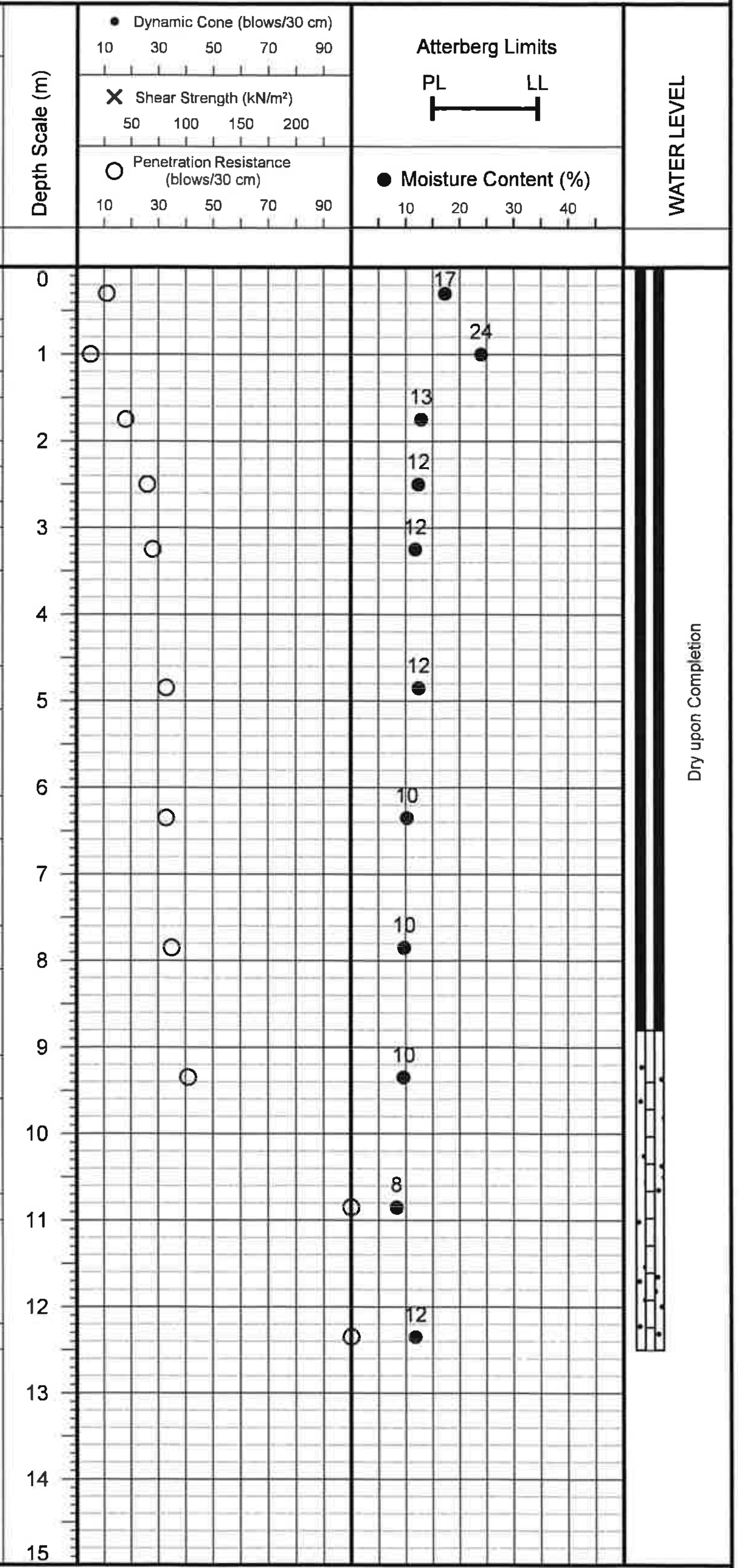
PROJECT DESCRIPTION: Proposed Office Building

METHOD OF BORING: Solid Stem

PROJECT LOCATION: 3650 Eglinton Avenue West, City of Mississauga

DRILLING DATE: December 28, 2022

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
182.0	Ground Surface			
0.0	**15 cm Topsoil** **EARTH FILL** Brown silty clay a trace of sand occ. organics and rootlets	1	DO	11
		2	DO	5
180.7 1.3	Compact to dense **SANDY SILT TILL** a trace of gravel occ. sand seams and layers, and cobbles	3	DO	18
		4	DO	26
		5	DO	28
	brown grey			
		6	DO	33
		7	DO	33
		8	DO	35
		9	DO	41
171.7 10.3	Grey, very dense **SILTY SAND TILL** a trace of gravel occ. cobbles and boulders	10	DO	50/15
		11	DO	50/15
169.7 12.5	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 12.5 m completed with 3.1 m PVC slotted screen Sand backfill from 8.8 to 12.5 m Bentonite seal from 0.0 m to 8.8 m Provided with a monument casing			



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JOB NO.: 2211-S176

LOG OF BOREHOLE: 3

FIGURE NO.: 3

PROJECT DESCRIPTION: Proposed Office Building

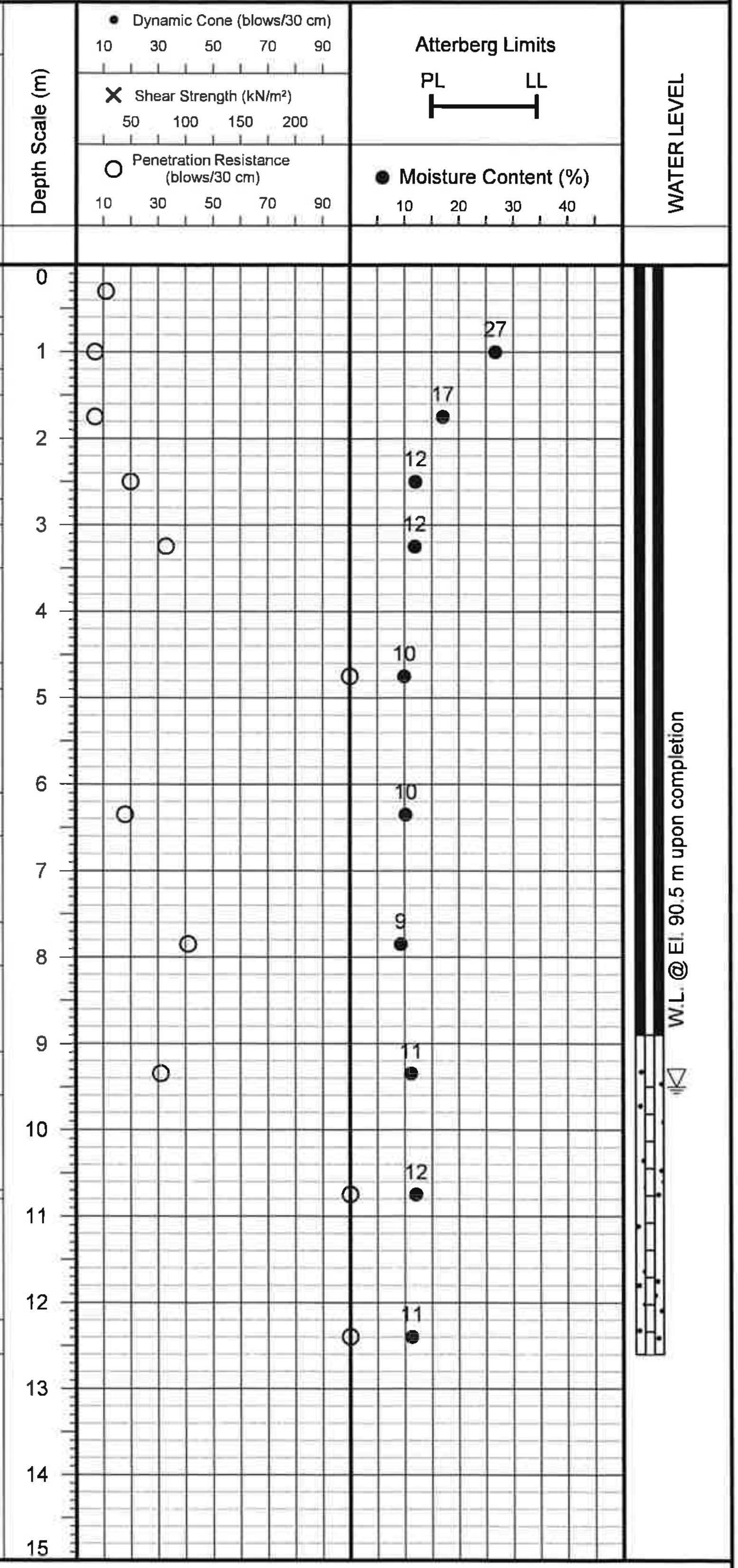
METHOD OF BORING: Solid Stem

PROJECT LOCATION: 3650 Eglinton Avenue West, City of Mississauga

DRILLING DATE: December 29, 2022

El. (m) Depth (m)	SOIL DESCRIPTION
181.7	Ground Surface
0.0	**13 cm Topsoil** **EARTH FILL** Dark brown to brown silty clay occ. organics and rootlets
179.6 2.1	Brown, compact to very dense **SANDY SILT TILL** a trace of gravel occ. sand seams and layers occ. cobbles and boulders
176.2 5.5	Grey, very stiff **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers
174.6 7.1	Grey, dense **SANDY SILT TILL** a trace of gravel occ. sand seams and layers, and cobbles
171.4 10.3	Grey, very dense **SILTY SAND TILL** a trace of gravel occ. cobbles and boulders
169.4 12.6	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 12.6 m completed with 3.1 m PVC slotted screen Sand backfill from 8.9 to 12.2 m Bentonite seal from 0.0 m to 8.9 m Provided with a monument casing

SAMPLES		
Number	Type	N-Value
1	DO	11
2	DO	7
3	DO	7
4	DO	20
5	DO	33
6	DO	50/10
7	DO	18
8	DO	41
9	DO	31
~~10~~	~~DO~~	~~50/13~~
11	DO	50/10



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JOB NO.: 2211-S176

LOG OF BOREHOLE: 4

FIGURE NO.: 4

PROJECT DESCRIPTION: Proposed Office Building

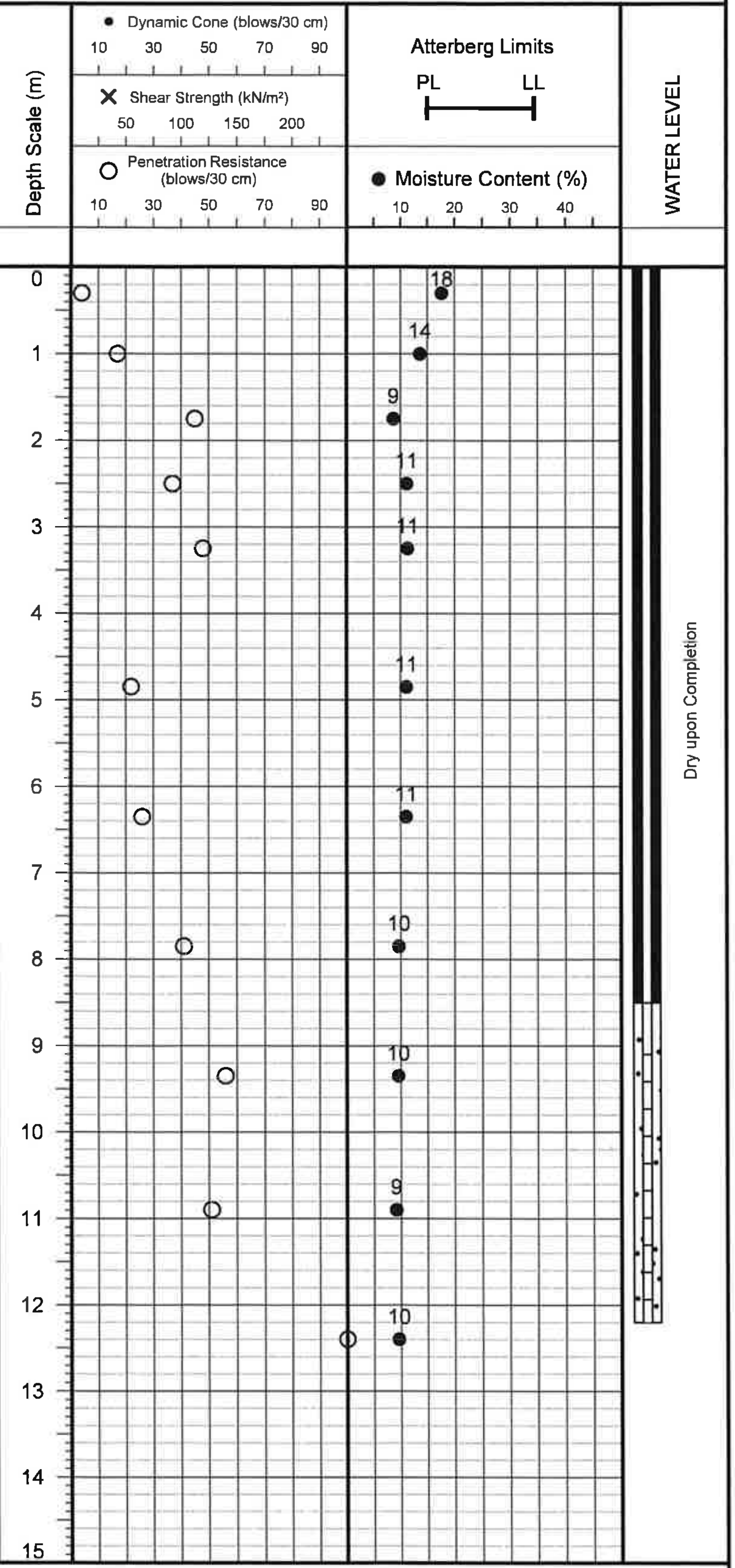
METHOD OF BORING: Solid Stem

PROJECT LOCATION: 3650 Eglinton Avenue West, City of Mississauga

DRILLING DATE: December 30, 2022

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)
182.3	Ground Surface				
0.0	**30 cm Topsoil**				0
	EARTH FILL Dark brown sandy silt occ. topsoil inclusion	1	DO	4	
181.3 1.0		2	DO	17	1
	Compact to very dense **SANDY SILT TILL** a trace of gravel occ. sand seams and layers, and cobbles	3	DO	45	2
		4	DO	37	
		5	DO	48	3
	brown grey				4
		6	DO	22	5
		7	DO	26	6
		8	DO	41	8
		9	DO	56	9
172.0 10.3	Grey, very dense **SILTY SAND TILL** a trace of gravel occ. cobbles and boulders	10	DO	51	11
169.7 12.6		11	DO	50/15	12
	END OF BOREHOLE				13

Installed 50 mm Ø monitoring well to 12.2 m
completed with 3.1 m PVC slotted screen
Sand backfill from 8.5 to 12.2 m
Bentonite seal from 0.0 m to 8.5 m
Provided with a monument casing



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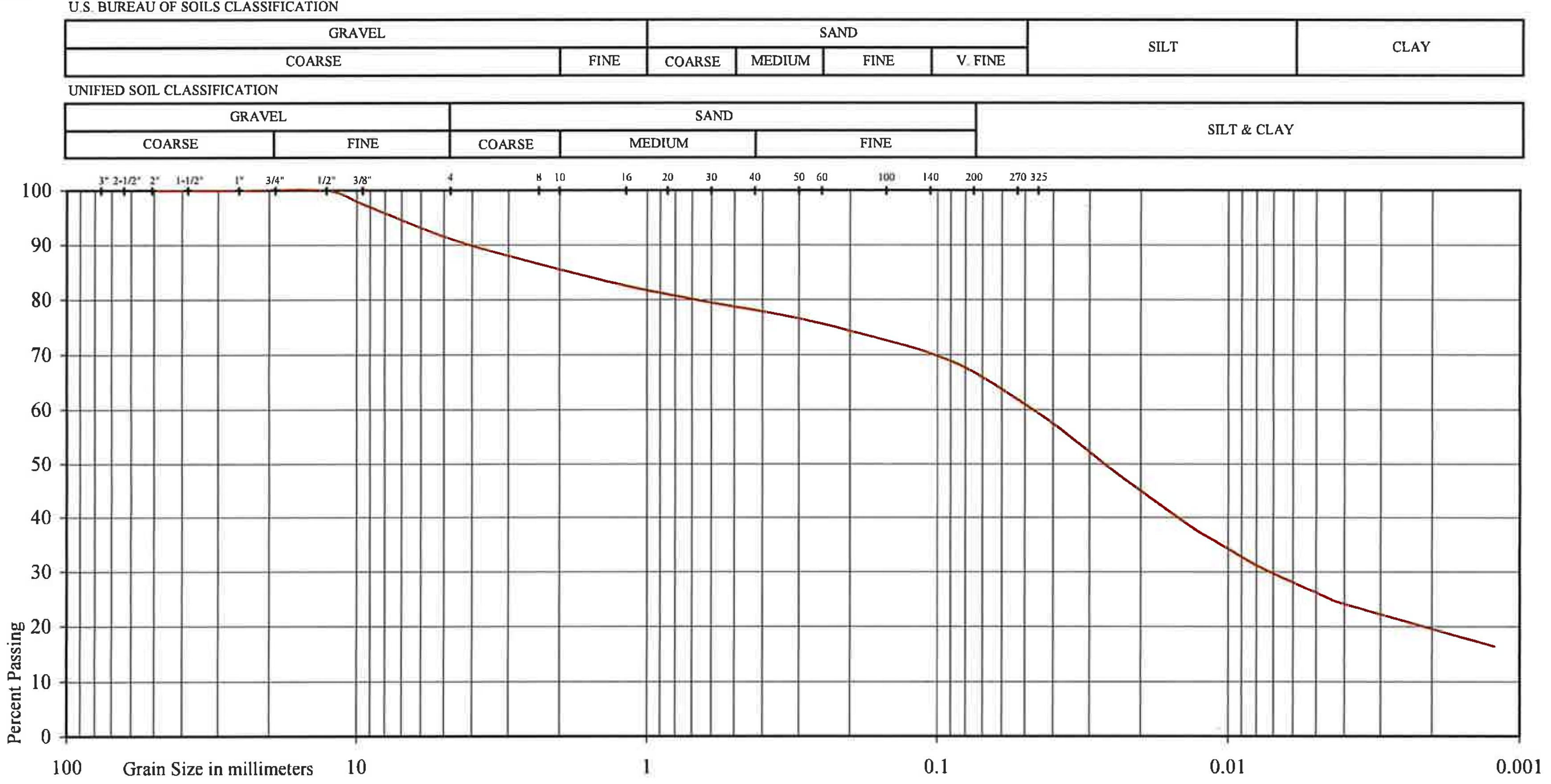
GRAIN SIZE DISTRIBUTION

Reference No: 2211-S176



Project: Proposed Office Building

Location: 3650 Eglinton Avenue West, City of Mississaga

Borehole No: 1

Sample No: 8

Depth (m): 7.9

Elevation (m): 174.5

Liquid Limit (%) = 24

Plastic Limit (%) = 16

Plasticity Index (%) = 8

Moisture Content (%) = 13

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

Classification of Sample [& Group Symbol]: SILTY CLAY, TILL
sandy, a trace of gravel

Figure: 5

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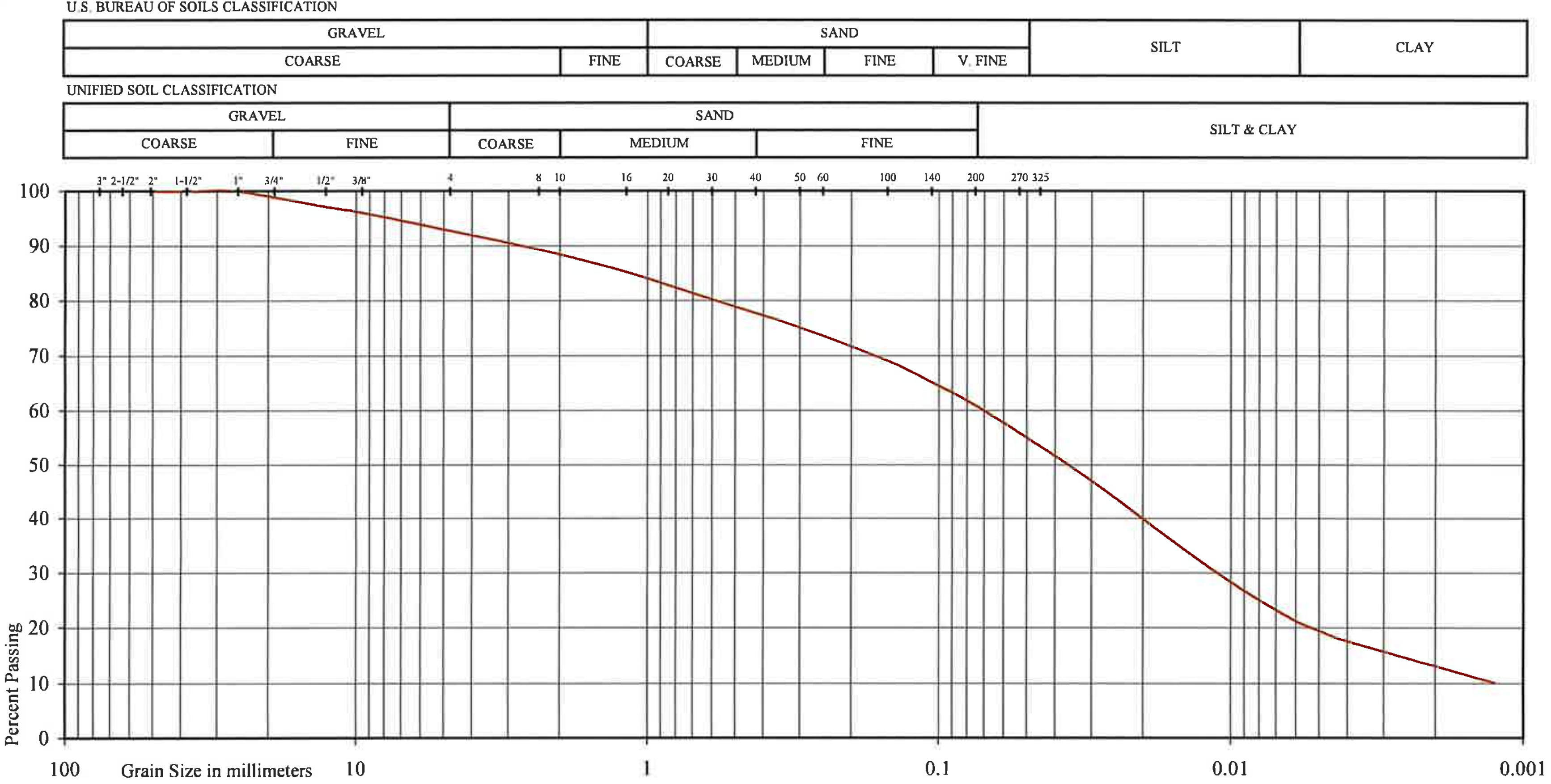
GRAIN SIZE DISTRIBUTION

Reference No: 2211-S176



Project: Proposed Office Building

Location: 3650 Eglinton Avenue West, City of Mississaga

Borehole No: 2

Sample No: 9

Depth (m): 9.4

Elevation (m): 172.611

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 10

Estimated Permeability

(cm./sec.) = 10^{-6}

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

some clay, a trace of gravel

Figure: 6

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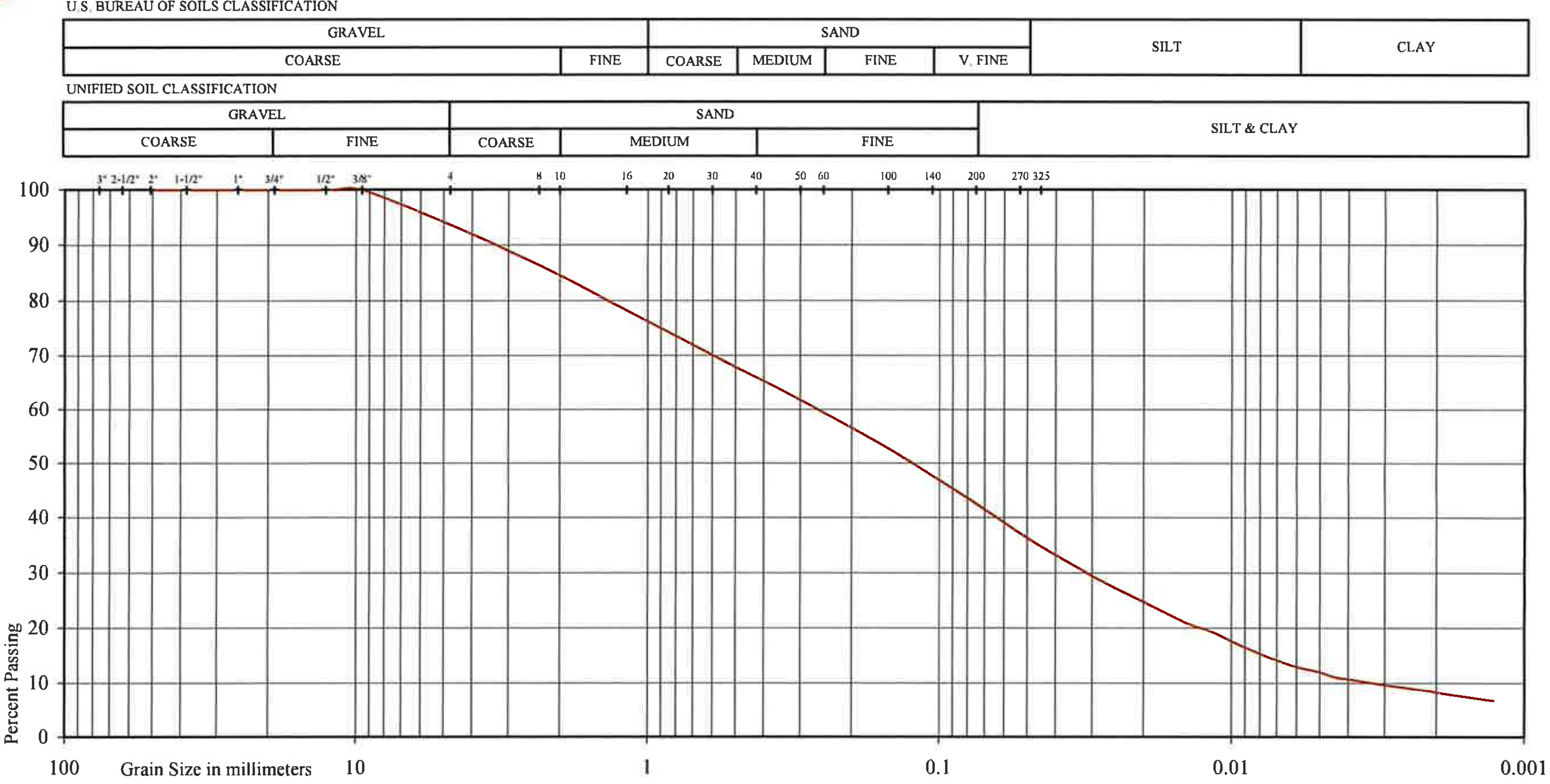
GRAIN SIZE DISTRIBUTION

Reference No: 2211-S176



Project: Proposed Office Building

Location: 3650 Eglinton Avenue West, City of Mississaga

Borehole No: 3

Sample No: 11

Depth (m): 12.3

Elevation (m): 169.439

Liquid Limit (%) = -

Plastic Limit (%) = -

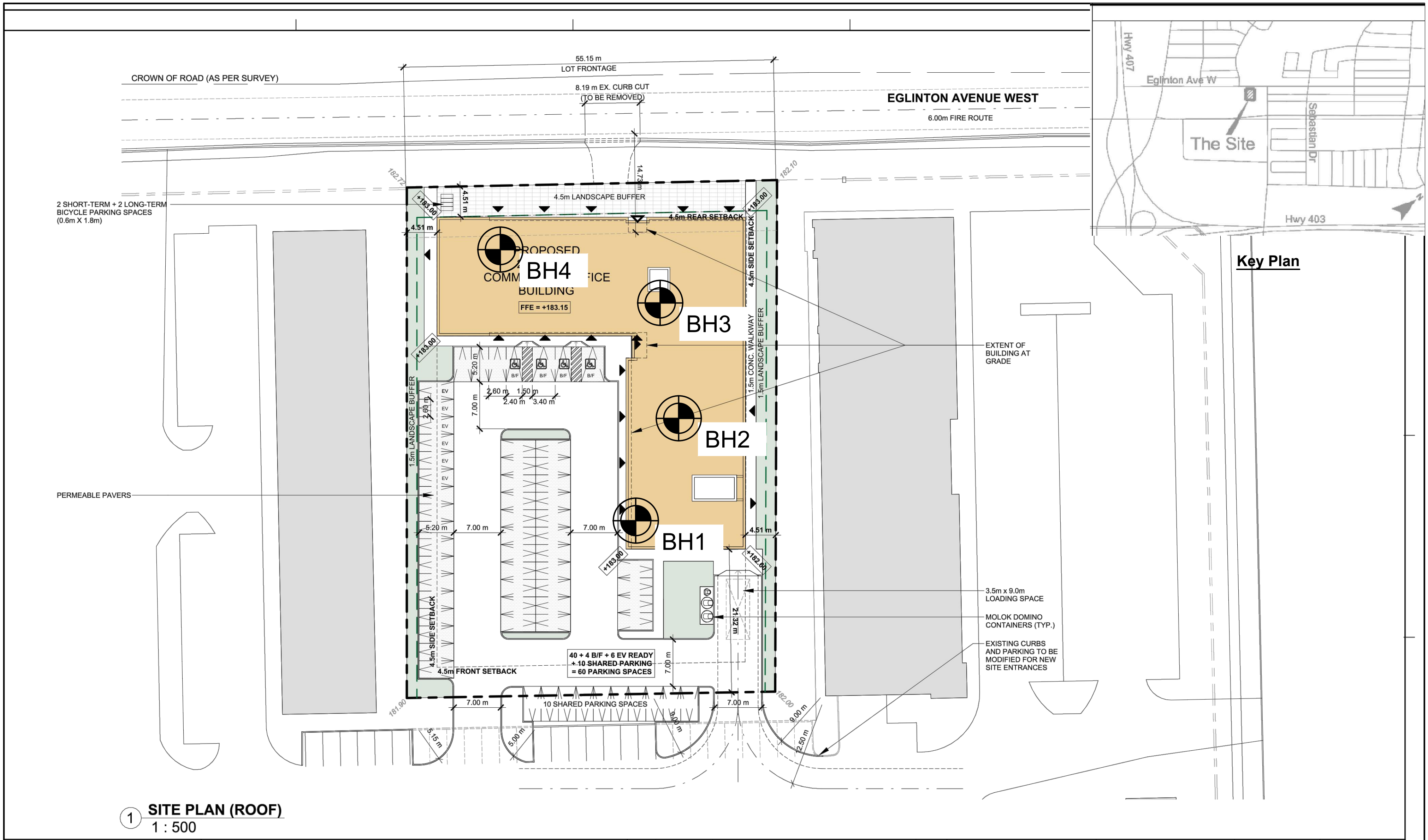
Plasticity Index (%) = -

Moisture Content (%) = 11

Estimated Permeability (cm./sec.) = 10^{-5}

Classification of Sample [& Group Symbol]: SILTY SAND, TILL
traces of clay and gravel

Figure: 7



Key Plan

1 SITE PLAN (ROOF)
1 : 500

CARICARI LEE ARCHITECTS

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CLA

PROJECT NAME
EGLINTON OFFICE BUILDING
3650 EGLINTON AVENUE W, MISSISSAUGA, ON

DRAWING TITLE
SITE PLAN

No.	Date:	Issued / Revision:	By:

CONTRACTOR TO VERIFY ALL DIMENSIONS ON THE SITE AND REPORT ANY DICREPANCIES TO THE ARCHITECT BEFORE PROCEEDING WITH THE WORK.

ALL DRAWINGS ARE THE PROPERTY OF THE ARCHITECT AND MUST BE RETURNED

SCALE	1 : 500
PROJECT NO.	22032
DATE	AUGUST 2023

NORTH:
PN TN

DRAWING NO.
A 1.30

4:52:14 PM

LEGEND

⊕ Borehole With Monitoring Well

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Borehole and Monitoring Well Location Plan

SITE: 3650 Eglinton Avenue West, City of Mississauga

DESIGNED BY: D.Y.
CHECKED BY: K.L.
DWG NO.: 1
SCALE: 1:800
REF. NO.: 2211-S176
DATE: September 2023
REV

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SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

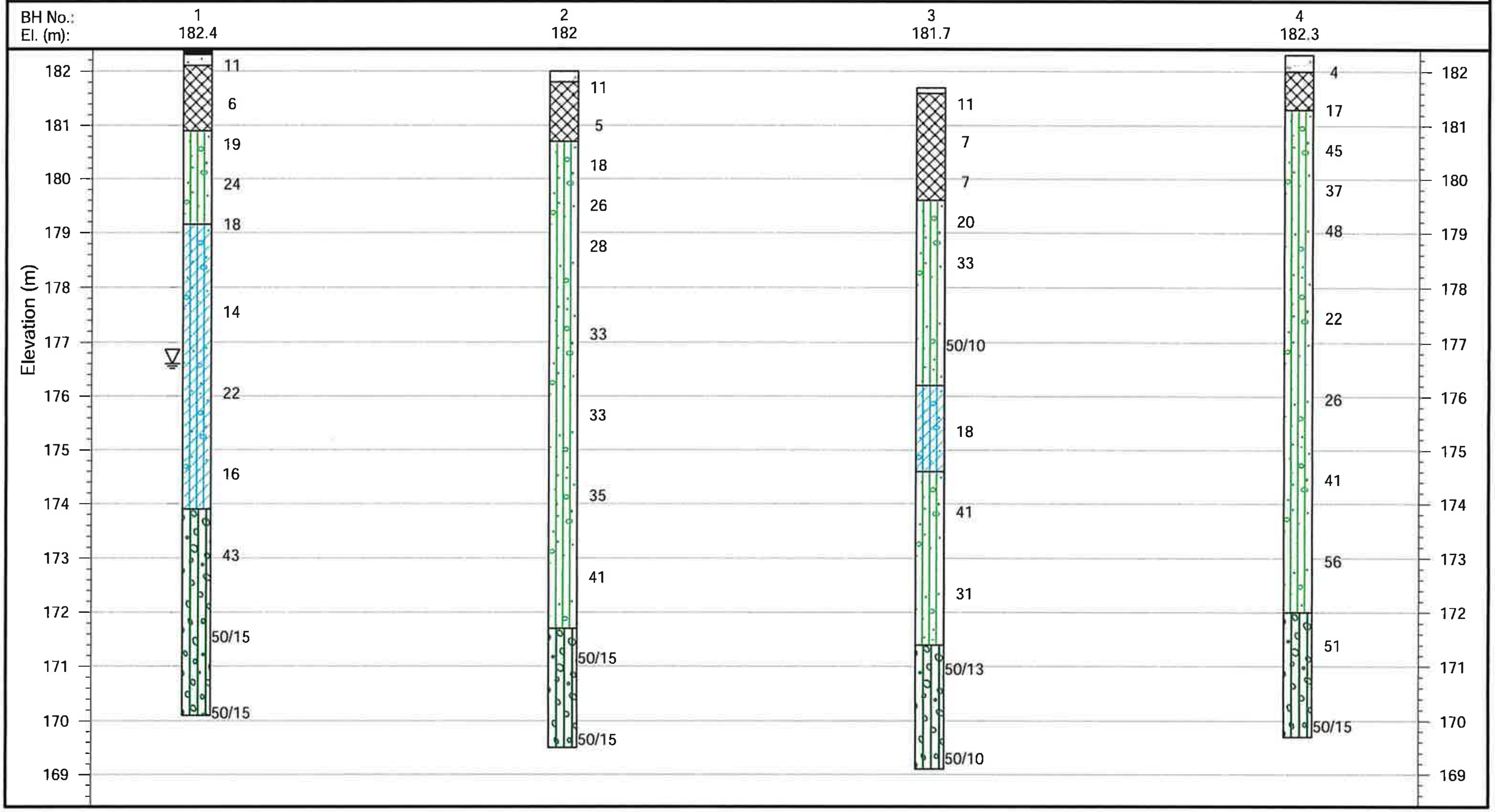
JOB NO.: 2211-S176
REPORT DATE: Feburary 2023
PROJECT DESCRIPTION: Proposed Office Building

PROJECT LOCATION: 3650 Eglinton Avenue West, City of Mississauga

LEGEND

- ■ ASPHALT
- ▨ FILL
- GRANULAR
- SANDY SILT TILL
- SILTY CLAY TILL
- SILTY SAND TILL
- TOPSOIL

∇ WATER LEVEL (END OF DRILLING)



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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 A

SHORING DESIGN

REFERENCE NO. 2211-S176

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 an area with close proximity of adjacent structure and the excavation will be extending below the foundation level where 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 buildup behind the shoring. If the wall is designed to be watertight 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 Cohesive Soils: $R = 9\ C_u\ D\ (L- 1.5\ D)$

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

L = Embedment depth of the pile m

C_u = Undrained shear strength of subsoil kPa

K_p = Passive resistance in cohesionless soils -
γ = unit weight of the soil kN/m^3

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).

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 an adhesion value of 50 kPa. 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 45°, founded in the native soil deposit below the bottom of excavation should be designed for the allowable bearing pressure of 100 kPa.

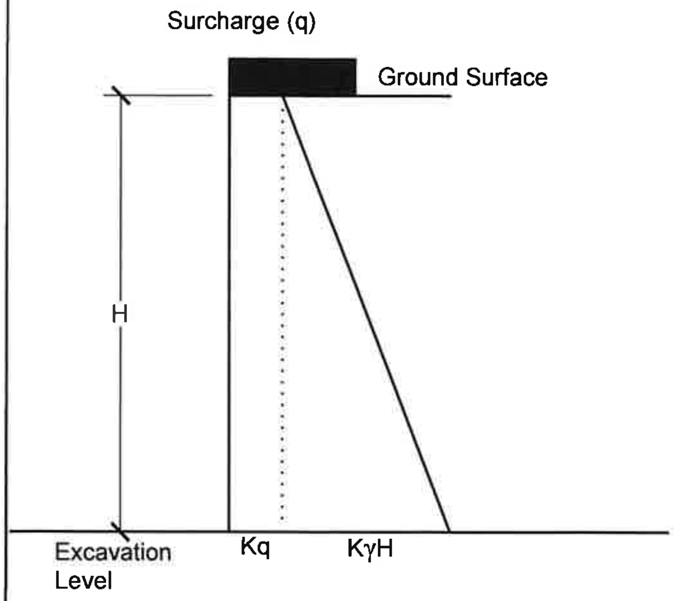
The raker footings should be located outside the zone of influence of the buried portion of the soldier piles at a distance of not less than 1.5 of the length of embedment of the soldier pile.

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.

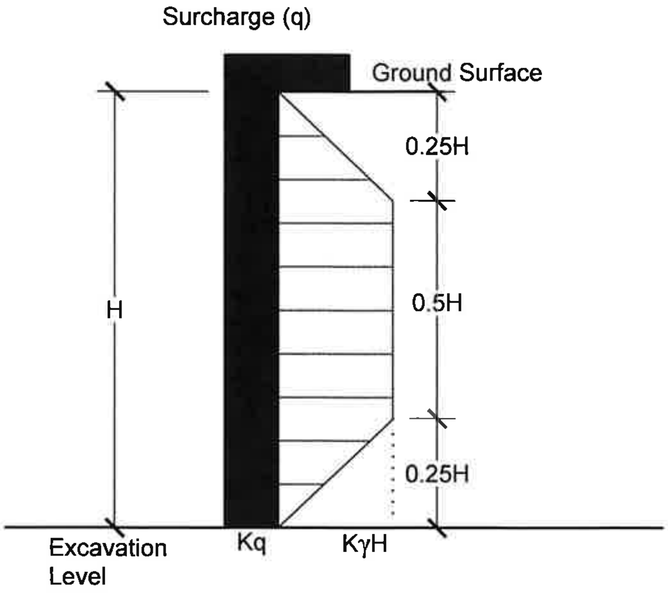
MONITORING OF PERFORMANCE

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 21 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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Lateral Earth Pressure and Temporary Shoring

SITE: 3650 Eglinton Avenue West, City of Mississauga

DESIGNED BY: D.Y.	**CHECKED BY:** K.L.	**DWG NO.:** A1	
SCALE: N.T.S.	**REF. NO.:** 2211-S176	**DATE:** February 2023	**REV**