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A REPORT TO
LOUISVILLE HOMES LIMITED

A GEOTECHNICAL INVESTIGATION
FOR
PROPOSED STACKED TOWNHOUSE DEVELOPMENT

1884 LIVERPOOL ROAD AND 1885 GLENDALE DRIVE

CITY OF PICKERING

REFERENCE NO. 2411-S164

MARCH 2026
(REVISION OF REPORT DATED MARCH 2025)

DISTRIBUTION

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1.0 INTRODUCTION

In accordance with a written authorization from Ms. Amanda Lazaridis, Project Manager of Louisville Homes Ltd., dated November 22, 2024, a geotechnical investigation was carried out at 1884 Liverpool Road and 1885 Glendale Drive in the City of Pickering.

The purpose of this investigation was to reveal the subsurface conditions and determine the engineering properties of the disclosed soils for the design and construction of a proposed 3-storey stacked townhouse block with a basement and an underground parking level. The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The site is located in the physiographic region of the Iroquois Plain where the Clay Plain, Drumlins and Sand Plain intersects. The soil stratigraphy consists of lacustrine sands, silts, clays and reworked till, where they bed onto shale bedrock of Georgian Bay Formation at moderate to considerable depths.

The subject site has a total area of 0.23 hectares and is located at the northwest corner Liverpool Road and Glenanna Road in the City of Pickering. The property consists of 1884 Liverpool Road and 1885 Glendale Drive. At the time investigation, the subject site is occupied by two dwellings and a detached garage, with associated driveways and landscaping. The grading of the site generally descends towards the west.

A review of the Site Plan prepared by Micacchi Architecture Inc., dated January 30, 2026, indicates that the proposed development will consist of a 3-storey stacked townhouse structure with basements and an underground parking level.

3.0 FIELD WORK

The field work, consisting of 5 sampled boreholes to depths ranging from 11.1 to 14.2 m, was performed between December 30, 2024 and January 2, 2025 at the locations shown on the Borehole and Monitoring Well Location Plan, Drawing No. 1, enclosed.

All boreholes were advanced at intervals to the sampling depths by a track-mounted machine with solid-stem augers equipped for soil sampling. Standard Penetration Tests (SPTs), 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.

Relatively weak soil deposit was contacted in the boreholes. Field vane shear tests were performed in the weak clay layer in Boreholes 2, 3, 4 and 5, to determine the undrained shear strength.

Upon completion of borehole drilling, monitoring wells were installed in all boreholes. Details of the monitoring wells are presented in the borehole logs.

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

4.0 SUBSURFACE CONDITIONS

The investigation revealed that beneath the pavement structure or a topsoil veneer, the site is underlain by a stratum of sand overlying silt, silty clay/silty clay till and silty sand silt. In places, the sand till beds onto shale bedrock where refusal to augering was encountered.

Detailed descriptions of the encountered subsurface conditions are presented on the Boreholes Logs, comprising Figures 1 to 5, 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 Pavement Structure

The revealed pavement structure consists of 50 mm asphalt over 130 mm and 250 mm of granular fill in Boreholes 3 and 4, respectively.

4.2 Topsoil

Topsoil was encountered at the surface of Boreholes 1, 4 and 5, which is 33 cm, 25 cm and 38 cm, respectively. In Borehole 4, a layer of topsoil, 15 cm in thickness, was encountered beneath the pavement structure. Thicker topsoil may be contacted in places beyond the borehole locations.

4.3 Sand

The sand deposit was contacted near the ground surface in all boreholes. In Boreholes 4 and 5, a lower sand deposit was also encountered beneath the silty clay/silty clay till deposit. The sand near the ground surface is fine grained with some silt while the sand deposit encountered below the clay/clay till are fine to medium grained with some silt. Grain size analysis was performed on 1 sand sample; the result is plotted on Figure 6.

The obtained 'N' values range from 1 to 14, with a median of 7 blows per 30 cm of penetration, indicating the sand deposit is very loose to compact, being generally loose in relative density. The sand near the ground surface is generally weathered.

The natural water content values of the sand ranges from 6% to 23%, with a median of 14%, indicating it is in a moist to wet, generally very moist condition.

The engineering properties of the sand deposit are given below:

- Low to high frost susceptibility depending on its silt content.
- High water erodibility.
- In excavation, the sand will slough and run with water seepage. It will boil with a piezometric head of 0.3 m.

4.4 Silty Clay/Silty Clay Till

The silty clay and silty clay till were encountered generally beneath the sand deposit, except Borehole1, where it was encountered beneath the sand and silt deposit.

The obtained 'N' values of the silty clay and silty clay till range from the weight of hammer (i.e. 0 blows) to 67 blows per 30 cm of penetration indicating very soft to hard consistency. The high 'N' values were encountered in Borehole 1 only. The consistency of the clay and clay till in Boreholes 2, 3 ,4 and 5 are generally very soft. Field vane shear tests were performed in the clay and clay till; the undrained shear strength ranges from 36 to over 120 kPa, having the sensitivity values of 2.4 to 9.6, indicating the clay is sensitive to remoulding.

The Atterberg Limits were performed on 2 silty clay samples; the resulting Liquid Limits are 20% and 23%, and Plastic Limits are 15% and 16%, indicating a low plasticity. The water

content of silty clay and silty clay till samples range between 15% and 38%, with a median of 22%, indicating generally very moist conditions.

The engineering properties of the silty clay deposit are presented below:

- High frost susceptibility and high soil-adfreezing potential.
- Low water erodibility.
- The overall shear strength of the clay and clay till is susceptible to impact disturbance; i.e., the disturbance will induce a build-up of pore pressure within the soil mantle, resulting in soil dilation and a reduction of shear strength.
- The soft clay and clay till will consolidate under surcharge loads.
- In excavations, the sides of the soft clay and clay till may fail and the base may heave due to overstressing.

4.5 **Silt**

The silt deposit was encountered in Borehole 1 and occurs as seams and layers within the silty clay and silty clay till deposit. Sample examinations show that the silt is fine grained with variable amounts of sand and clay. A grain size analysis was performed on a representative sample and the result is presented on Figure 7.

The recorded 'N' values of the silt range from 44 to over 100, with a median of over 100 blows per 30 cm of penetration, showing dense to very dense relative density.

The natural water content of the silt deposit ranges from 13% to 17%, with a median of 15%, indicating the silt is moist to very moist.

The engineering properties of the silt deposit are presented below:

- High in frost susceptibility and high soil ad-freezing potential.
- High water erodibility.
- High capillarity and water retention capacity, the wet silt will dilate when disturbed.
- Due to its dilatancy, the sear strength of the wet silt is susceptible to dynamic disturbance.
- In excavation, the silt will slough, run with seepage and the bottom will boil under a piezometric head of 0.3 m.

4.6 Silty Sand Till

The silty sand till was contacted beneath the clay/clay till and sand in Boreholes 3 and 5, below the depth of 12.7 m and 10.7 m from the ground surface, respectively. It consists of a random mixture of particle sizes ranging from clay to gravel, with the sand and silt being the dominant fraction. Grain size analysis was performed on a representative sample of the till and the result is plotted on Figure 8.

The relative density of the deposit is dense to very dense, as inferred from the 'N' values of 33, 36 and 90 blows per 30 cm of penetration.

The natural water content of the soil samples is 9% and 10%, showing generally moist conditions.

The engineering properties of the till deposit are listed below:

- High frost susceptibility and low water erodibility.
- The till will be stable in relatively steep cuts; however, under prolonged exposure, localized sheet sliding may occur.

4.7 Shale Bedrock

Weathered shale bedrock was encountered in Boreholes 1 and 3 at depths of 12.2 m and 13.7 m from the ground surface, respectively. The shale bedrock is grey in colour, of Georgian Bay Formation. The rock quality, however, was not determined by rock coring, which is beyond the scope of the current investigation.

The upper rock profiles can be fissured as a result of the weathering and/or being overstressed by glacial loading. The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil, but the limy shale and sandstone bands would remain as rock slabs.

5.0 GROUNDWATER CONDITION

Groundwater seepage encountered during augering of the boreholes was recorded. Each borehole was checked for the presence of groundwater and cave-in upon their completion. The data was plotted on the Borehole Logs and summarized in Table 1.

Table 1 - Groundwater Levels

Borehole No.	Ground Elevation (m)	Seepage Encountered During Augering			Measured Groundwater Level and Cave-in* on Completion	
		Depth (m)	El. (m)	Amount	Depth (m)	El. (m)
1	91.3	-	-	-	Dry	-
2	89.6	4.6	85.0	Slight	7.6*	82.0*
3	89.2	2.3	86.9	Slight	4.6	84.6
4	88.9	2.3	86.6	Slight	4.6	84.3
5	88.1	1.0	87.1	Some	2.7*	85.4*

Groundwater level in the monitoring wells was recorded on 3 occasions between January 10 and February 7, 2025. The data is presented in Table 2.

Table 2 – Groundwater Levels in Monitoring Wells

BH No.	Ground Elevation (m)	Monitoring Well Depth (m)	Groundwater Level					
			Jan. 10, 2025		Jan. 24, 2025		Feb. 7, 2025	
			Depth (m)	El. (m)	Depth (m)	El. (m)	Depth (m)	El. (m)
1	91.3	6.1	4.6	86.7	3.8	87.5	2.5	88.8
2	89.6	6.1	1.8	87.8	2.3	7.3	2.3	87.3
3	89.2	6.1	2.4	86.8	2.4	86.8	2.5	86.7
4	88.9	6.1	2.4	86.5	2.3	86.6	2.3	86.6
5	88.1	6.1	1.9	86.2	1.8	86.3	1.8	86.3

Groundwater levels in the monitoring wells were recorded between 1.8 m and 4.6 m from existing grade, or between El. 88.8 m and 86.3 m. It is subject to seasonal fluctuation.

Additional discussion on the groundwater condition within the property will be presented in the hydrogeological report, under separate cover.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation revealed that beneath the pavement structure or a topsoil veneer, the site is underlain by a stratum of sand overlying silty clay/silty clay till and/or silt. At lower depths, deposits of silty sand till and sand were encountered and beds onto shale bedrock.

Groundwater levels in the monitoring wells were recorded between 1.8 m and 4.6 m from existing grade, or between El. 88.8 m and 86.3 m. It is subject to seasonal fluctuation. Additional discussion on the groundwater condition within the property will be presented in the hydrogeological report under separate cover.

Based on the site plan provided, the development consists of a 3-storey staked townhouse structure with a basement and 1 underground parking level. The proposed finished floor elevation of the underground parking garage is El. 83.86 m.

The geotechnical findings which warrant special consideration are presented below:

1. The founding elevation of the proposed structure is below El. 83.86 m, where the soil consists of either very dense silt at Borehole 1 adjacent to Liverpool Road, and very soft clay beyond Borehole 1. The clay has limited capacity for foundation design.
2. The existing dwellings will be demolished for the construction of the proposed development. All debris must be removed off site and any cavity must be properly backfilled and compacted to create a working platform for construction.
3. Where safe sloped excavation is not feasible, temporary shoring should be provided prior to excavation. All excavation should be carried out in accordance with the Ontario Regulation 213/91.

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

6.1 Site Preparation

The existing dwellings and associated structures will be demolished for the construction of the new development. After demolition of the existing structures, underground utilities, foundations and pavement, the construction debris must be disposed off-site prior to construction. The cavities must be backfilled with approved inorganic soil and compacted in

layers, to at least 98% of the Standard Proctor dry density (SPDD), for the preparation of a construction platform. If the site is to be raised, a pre-loading scheme may be required. This should be further assessed once the site grading has been established.

6.2 Foundation

A review of the Architectural Plan indicates that the proposed development will consist of a 3-storey stacked townhouse structure with basements over an underground parking level. The finished floor slab of the underground parking level is at El. 83.51 m, where the foundation is expected to be underlain by very dense silt at Borehole 1, and very soft silty clay/silty clay till was encountered in all other borehole locations.

The groundwater level recorded in the monitoring wells ranges from El. 88.8 to 86.3 m, therefore, depending on the in-flow of groundwater around the proposed structure, the foundation, including the elevator pit, may have to be designed as a submerged 'tank' structure, which is waterproofed and designed to resist the hydrostatic pressure. The parking level slab will be poured on a granular fill above the raft where the utilities and service pipes will be laid. This can be further assessed once the hydrogeological assessment has been completed.

Accordingly, the recommended net bearing pressures along with the modulus of subgrade reaction for the design of a raft foundation founded approximately 5.8 to 9.0 m below the prevailing ground surface, at approximately El. 82.5 m, is presented below:

- Maximum Allowable Soil Bearing Pressure at SLS = 50 kPa
- Factored Ultimate Soil Bearing Pressure at ULS = 80 kPa
- Modulus of Subgrade Reaction = 15 MPa/m.

The total and differential settlements for the raft foundation designed at SLS, are estimated to be 25 mm and 20 mm, respectively.

A mud slab consisting of lean mix concrete, 6 to 8 cm in thickness, should be placed on the foundation subgrade immediately after exposure and inspection as a working platform for the construction of the raft foundation.

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

The foundation subgrade must be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to assess its suitability for bearing the designed foundations.

In order to minimize differential settlement between the very dense soils encountered in Borehole 1 and the very soft clay in all other boreholes, deep foundation to support the structure on the weaker soil can also be considered. Deep foundation such as micro piles can be considered. The appropriate founding elevation is expected to be below El. 75 m, or over 13 to 14 m below the prevailing ground surface in in the vicinity of Boreholes 2 to 5, inclusive.

The design load of micro piles can be assessed by the prospective foundation contractor in these specialties. Full scale load test in the field must be conducted to confirm the load carrying characteristics of piles.

The installation of the micro piles must be supervised and inspected by either a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that the construction of piles is compatible with the foundation design requirements.

The pile caps and grade beams exposed to weathering should have at least 1.2 m of earth cover for protection against frost action. The floor slab should be designed as a structural slab tied into the grade beams.

Alternatively, soil improvement consisting of ram-aggregate piers system by Geopiers, rigid inclusions method from Keller or controlled modulus columns from Menard can be considered. Upon completion of the soil improvement, conventional footings can be constructed for the proposed development. The appropriate bearing capacity would be provided by the soil improvement contractor.

The foundation should meet the requirement specified in the Ontario Building Code and the structure should be designed to resist an earthquake force using Site Classification ‘E’ (soft soil). If deep foundation is considered, the Site Classification ‘C’ (very dense soil) can be used if the founding elevation is below El. 75.0 m. A Multichannel Analysis of Surface Waves seismic test to determine subsurface shear-weave velocities (V_s) can be carried out to verify the Site Classification to confirm structural design requirements.

6.3 Underground Structure

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

As discussed previously, due to the relatively high groundwater level, the underground structure should be waterproofed. 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 parking level slab will be poured on a granular fill above the raft where the utilities and service pipes will be laid.

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

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

6.4 Underground Services

The subgrade for the underground services should consists of undisturbed native soil or compacted earth fill. Where the subgrade is weak, the subgrade should be subexcavated and replaced with the bedding material, compacted to at least 98% SPDD in lifts no more than 20 cm thick.

A Class 'B' bedding, consisting of compacted 19-mm CRL, is recommended for the construction of the underground services.

The sewer joints connecting into manholes and catch basins should be leak-proof or wrapped with an appropriate waterproof membrane to prevent subgrade migration.

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

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

6.5 Backfilling in Trenches and Excavated Areas

On-site inorganic soils are too wet to be reused as trench backfill. Prior to be reused for backfilling, they should be aerated by spreading them thinly on the ground during warm and dry weather. Where aeration is not feasible, the wet material should be removed off site and the imported inorganic fill should be used instead.

The backfill should be compacted to at least 95% SPDD. In the zone below the slab-on-grade or within 1.0 m below the pavement subgrade, the backfill should be compacted at least 98% SPDD with the water content at 2% to 3% drier than the optimum. This is to provide the required stiffness for slab or pavement construction. 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, service crossings, foundation walls and columns. In areas which are inaccessible to a heavy compactor, light duty compactor can be used on imported sand or granular backfill.

6.6 Pavement Design

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

Table 3 - Pavement Design (Roof of Underground Garage)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	50	HL3
Asphalt Binder	50	HL8
Granular Base	200	Granular ‘A’ or equivalent
Granular Sub-base	100	Free-Draining Sand Fill

For the on-grade driveway portion of the access road extending to Glendale Drive, the recommended pavement structure is given in Table 4:

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

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	50	HL3
Asphalt Binder	50	HL8
Granular Base	150	Granular 'A'
Granular Sub-base	450	Granular 'B'

In order to provide a stable subgrade for the driveway construction, the subgrade within the 1.0 m zone below the underside of the granular base be compacted to at least 98% SPDD with the moisture content 2% to 3% drier than the optimum. This is to provide adequate stability for the pavement construction. The granular base and sub-base should be compacted to 100% SPDD.

Along the perimeter, where 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 flexible pavement.

Subdrains consisting of filter-wrapped weepers should also be installed along the curbs and in lower spots to prevent ponding and they should be connected to the catch basins or stormwater manholes. The subdrains should be 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	**Unit Weight (kN/m^3)**		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Loose**	**Compacted**
Silty Sand Till	22.5	12.5	1.33	1.03
Silty Clay/Silty Clay Till	21.0	11.0	1.30	1.00
Silt and Sand	20.5	10.5	1.20	0.98

Lateral Earth Pressure Coefficients	**Active K_a**	**At Rest K_0**	**Passive K_p**
Silty Sand Till	0.33	0.30	3.00
Silty Clay/Silty Clay Till/Silt/Sand	0.40	0.55	2.50

Effective Shear Strength Parameters	**Cohesion c' (kPa)**	**Angle of Internal Friction, ϕ'**
Silty Sand Till	2	30°
Silty Clay and Silty Clay Till	5	24°
Silt and Sand	0	30°

Coefficient of Permeability (K) and Percolation Time (T)	**K (cm/sec)**	**T (min/cm)**
Silty Clay/Silty Clay Till	10^{-7}	More than 80
Silt	10^{-6}	50
Sand	10^{-3}	8

Coefficients of Friction	
Between Concrete and Granular Base	0.50
Between Concrete and Natural Soils	0.35

6.8 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. For excavation purposes, the types of soils are classified in Table 6.

Table 6 - Classification of Soils for Excavation

Material	Type
Hard Silty Clay and Silty Sand Till	2
Earth Fill, Stiff Clay/Clay Till, drained Sand and Silt	3
Very soft to soft Clay/Clay Till and Sand and Silt below groundwater	4

Where safe sloped excavation is not feasible, a braced shoring system will be required. The design parameters for the shoring and our recommendations are provided in the Appendix. The overburden load and the surcharge from any adjacent structures should also be considered in the design of the shoring.

In excavation, the groundwater yield will be slow to moderate. The groundwater can be lowered by dewatering to at least 1.0 m below the bottom of the excavation. The method, volume and rate of dewatering for both construction and long-term discharge should be verified by the hydrogeological specialist.

In order to optimize the effect of dewatering in excavation, it is recommended that the installation of soldier piles for the shoring system be carefully monitored to record the observed static groundwater levels. This information should be reviewed by the dewatering contractor and Soil Engineers Ltd. in order to verify the necessary extent of the dewatering system on site.

6.9 Monitoring of Performance

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

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

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of Louisville Homes Limited and for review by its designated consultants and government agencies. The material in this report reflects the judgement of Kelvin Hung, P.Eng. and Bernard Lee, P.Eng., in light of the information available to it at the time of preparation.

Use of this report is subject to the conditions and limitations of the contractual agreement. Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, is the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

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LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

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

SAMPLE TYPES

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

PENETRATION RESISTANCE

Standard Penetration Resistance or 'N' Value:

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

Plotted as '○'

Dynamic Cone Penetration Resistance:

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

Plotted as '—●—'

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

SOIL DESCRIPTION

Cohesionless Soils:

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

Cohesive Soils:

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

Method of Determination of Undrained Shear Strength of Cohesive Soils:

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

△ Laboratory vane test

METRIC CONVERSION FACTORS

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

JOB NO.: 2411-S164

LOG OF BOREHOLE: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Stacked Townhouse Development

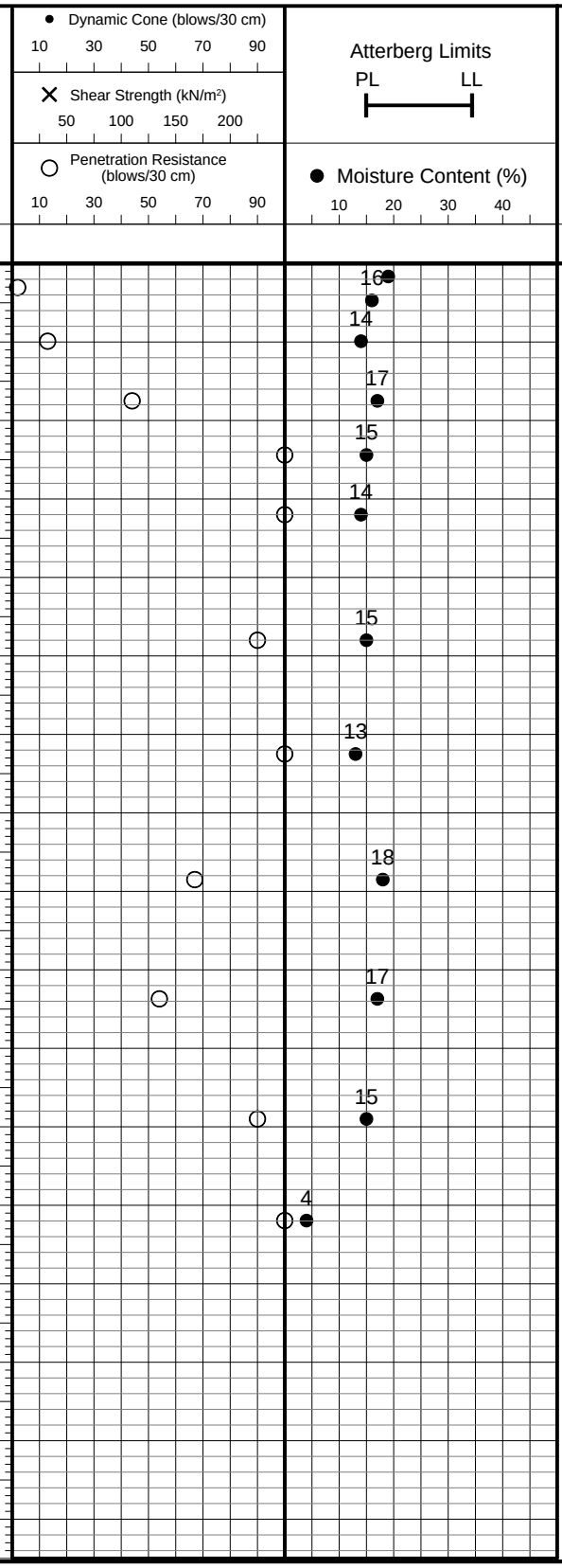
METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 1884 Liverpool Road and 1885 Glendale Drive
City of Pickering

DRILLING DATE: December 31, 2024
January 2, 2025

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
91.3	Ground Surface			
0.0	**33 cm TOPSOIL**	1A	DO	2
	Brown, loose to compact	1B		
	SAND			
	fine grained, some silt to silty — weathered	2	DO	13
89.8				
1.5	Dense to very dense	3	DO	44
		4	DO	50/15
	— brown grey	5	DO	50/15
	SILT	6	DO	90
	some clay	7	DO	50/15
83.7				
7.6	Grey, hard	8	DO	67
	SILTY CLAY	9	DO	54
	a trace of sand occ. silt seams and layers			
80.6				
10.7	Grey, very dense **SILT** a trace to some clay with shale fragments	10	DO	90
79.1				
12.2	Grey, weathered **SHALE**	11	DO	100/3

Installed 50 mm Ø PVC monitoring well
to 6.1 m with 3.0 m screen
Sand backfill from 2.4 to 6.1 m
Bentonite seal from 0.0 m to 2.4 m
Provided with a steel monument casing



WATER LEVEL

Dry on completion

Soil Engineers Ltd.

JOB NO.: 2411-S164

LOG OF BOREHOLE: 2

FIGURE NO.: 2

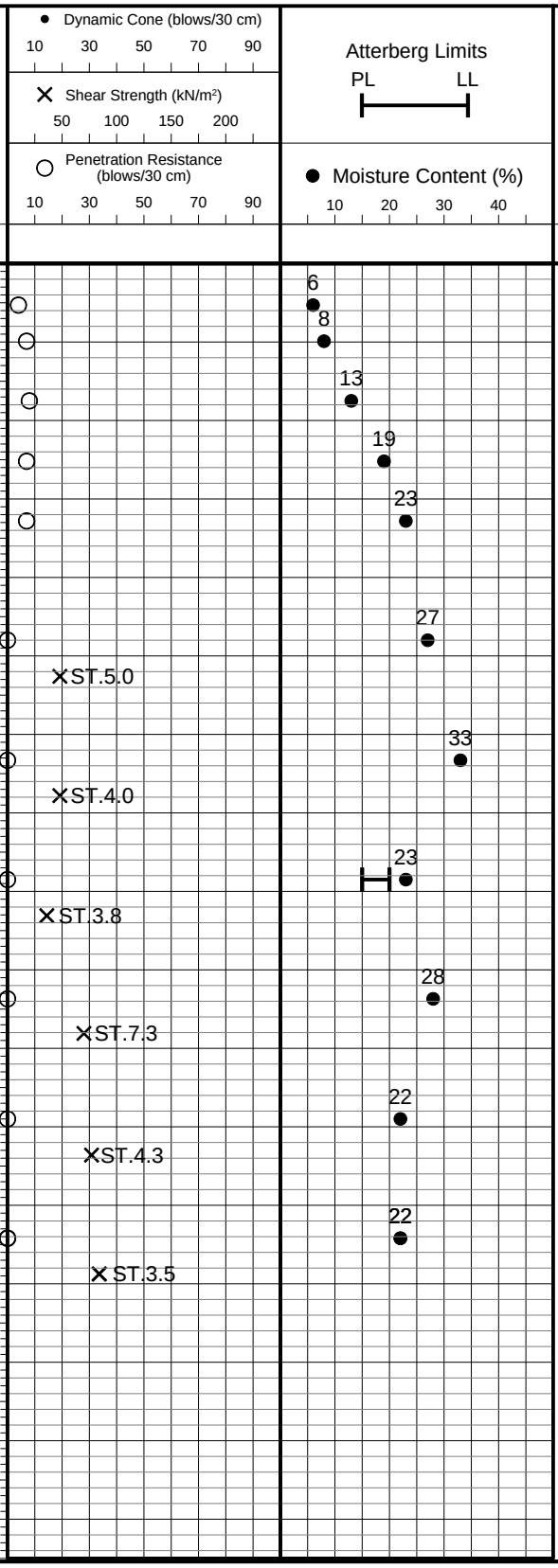
PROJECT DESCRIPTION: Proposed Stacked Townhouse Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 1884 Liverpool Road and 1885 Glendale Drive
City of Pickering

DRILLING DATE: December 30, 2024

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
89.6	Pavement Surface			
0.0	**50 mm ASPHALT** **130 mm GRANULAR FILL** Brown, loose to compact	1	DO	4
	weathered **SAND** fine grained, some silt	2	DO	7
		3	DO	8
87.3 2.3	Grey, stiff to very soft	4	DO	7
	brown grey	5	DO	7
	SILTY CLAY a trace of sand occ. silt layers and cobbles	6	DO	0
	wet below 4.6 m	7	DO	0
82.0 7.6	Grey, very soft **SILTY CLAY TILL** some sand, a trace of gravel occ. cobbles wet	8	DO	0
80.5 9.1	Grey, very soft **SILTY CLAY** a trace of sand occ. silt layers wet	9	DO	0
78.9 10.7	Grey, very soft **SILTY CLAY TILL** some sand, a trace of gravel occ. cobbles wet	10	DO	0
		11	DO	0
76.5 13.1	**END OF BOREHOLE** Installed 50 mm Ø PVC monitoring well to 6.1 m with 3.0 m screen Sand backfill from 2.7 to 6.1 m Bentonite seal from 0.0 m to 2.7 m Provided with a steel monument casing			



WATER LEVEL

Cave-in @ El. 82.0 m on completion

Soil Engineers Ltd.

JOB NO.: 2411-S164

LOG OF BOREHOLE: 3

FIGURE NO.: 3

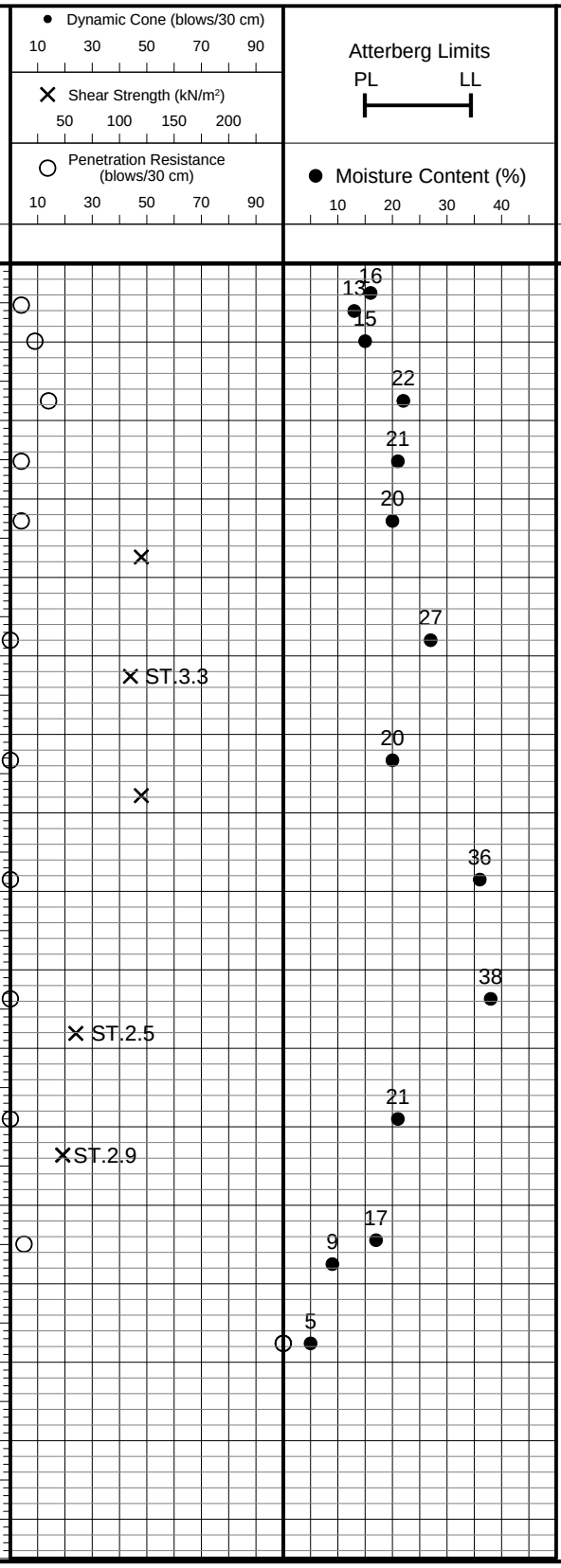
PROJECT DESCRIPTION: Proposed Stacked Townhouse Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 1884 Liverpool Road and 1885 Glendale Drive
City of Pickering

DRILLING DATE: December 30/31, 2024

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
89.2	Pavement Surface			
0.0	**50 mm ASPHALT** **250 mm GRANULAR FILL** **15 cm TOPSOIL**	1A 1B	DO	4
	Brown, loose to compact — weathered **SAND** fine grained, some silt wet below 1.5 m	2	DO	9
		3	DO	14
86.9 2.3	Grey, firm to very soft	4	DO	4
		5	DO	4
		6	DO	0
	SILTY CLAY			
	silt layer	7	DO	0
		8	DO	0
	a trace of sand occ. silt layers			
	wet	9	DO	0
78.5 10.7	Grey, very soft to firm	10	DO	0
	SILTY CLAY TILL some sand, a trace of gravel occ. cobbles			
76.5 12.7	Grey, very dense **SILTY SAND TILL** some gravel, a trace of clay occ. shale fragments	11A 11B	DO	5
75.5 13.8	Grey, weathered **SHALE**	12	DO	100/10
	END OF BOREHOLE			
	Installed 50 mm Ø PVC monitoring well to 6.1 m with 3.0 m screen Sand backfill from 2.7 to 6.1 m Bentonite seal from 0.0 m to 2.7 m Provided with a steel flushmount cover			



WATER LEVEL

W.L. @ El. 84.6 m on completion

Soil Engineers Ltd.

JOB NO.: 2411-S164

LOG OF BOREHOLE: 4

FIGURE NO.: 4

PROJECT DESCRIPTION: Proposed Stacked Townhouse Development

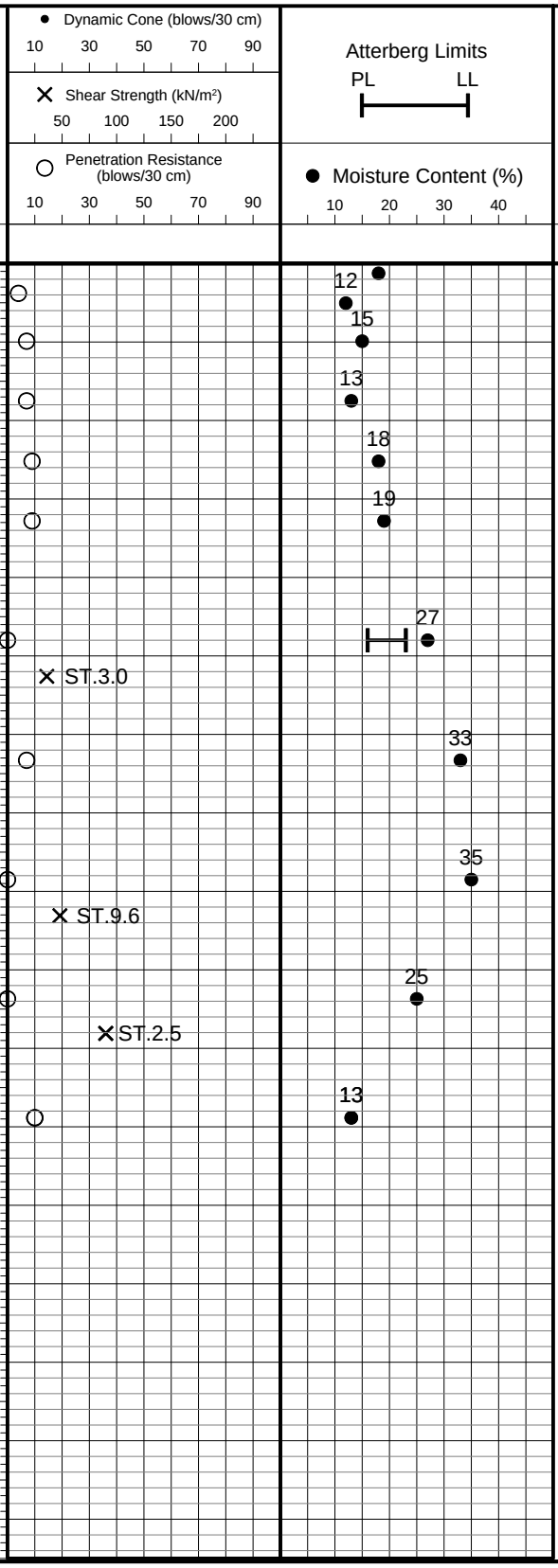
METHOD OF BORING: Solid Stem Augers

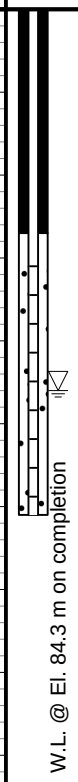
PROJECT LOCATION: 1884 Liverpool Road and 1885 Glendale Drive
City of Pickering

DRILLING DATE: January 2, 2025

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
88.9	Ground Surface			
0.0	**25 cm TOPSOIL**	1A		
	Brown, loose to compact	1B	DO	4
	weathered	2	DO	7
	SAND			
	fine grained, some silt	3	DO	7
86.6 / 2.3	Grey, firm to very soft	4	DO	9
		5	DO	9
		6	DO	0
	SILTY CLAY			
		7	DO	7
		8	DO	0
	a trace of sand occ. silt layers			
	wet	9	DO	0
78.2 / 10.7	Grey, loose	10	DO	10
77.8 / 11.1	**SAND** fine to coarse grained with shale fragments			
	END OF BOREHOLE			

Installed 50 mm Ø PVC monitoring well
to 6.1 m with 3.0 m screen
Sand backfill from 2.7 to 6.1 m
Bentonite seal from 0.0 m to 2.7 m
Provided with a steel monument casing





Soil Engineers Ltd.

JOB NO.: 2411-S164

LOG OF BOREHOLE: 5

FIGURE NO.: 5

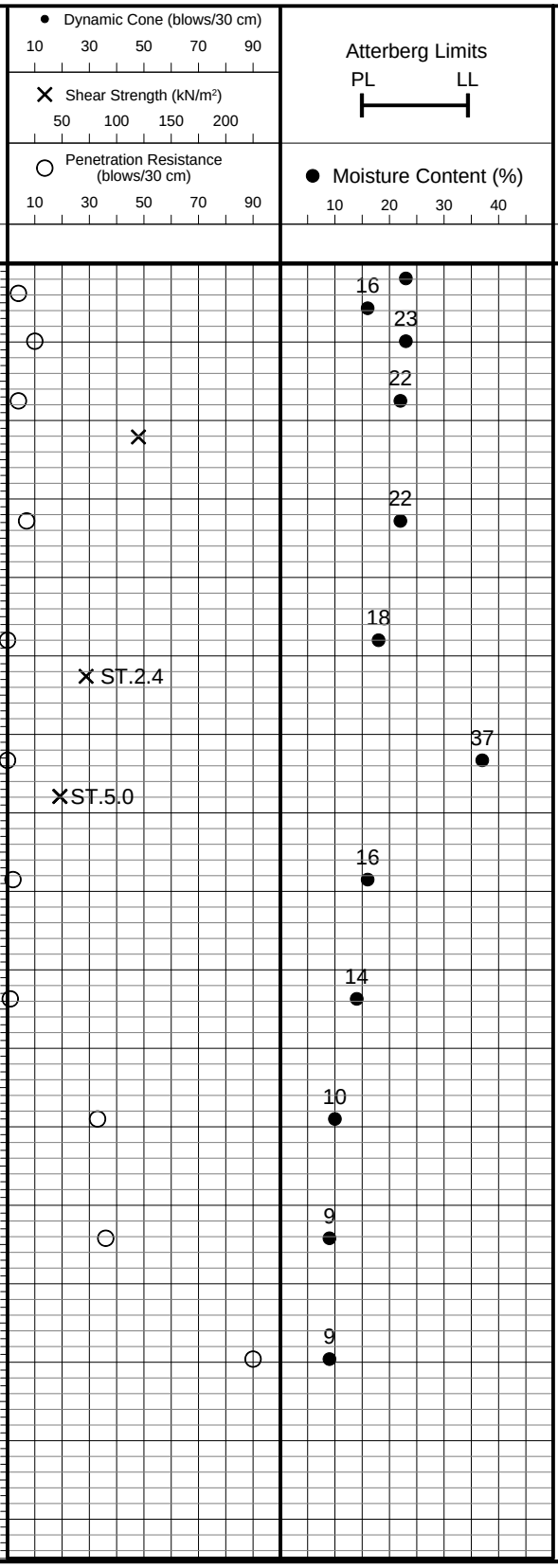
PROJECT DESCRIPTION: Proposed Stacked Townhouse Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 1884 Liverpool Road and 1885 Glendale Drive
City of Pickering

DRILLING DATE: January 2, 2025

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
88.1	Ground Surface			
0.0	**38 cm TOPSOIL**	1A	DO	4
	Brown, loose to compact	1B		
	SAND			
	fine grained, some silt — weathered	2	DO	10
86.6	wet below 1.0 m			
1.5	Firm to very soft	3	DO	4
		4	DO	7
	SILTY CLAY	5	DO	0
	a trace of sand occ. silt layers	6	DO	0
	wet			
80.5				
7.6	Grey, soft **SILTY CLAY TILL** some sand, a trace of gravel occ. cobbles	7	DO	2
79.0				
9.1	Grey, very loose	8	DO	1
	SAND fine to medium grained some silt			
77.4				
10.7	Grey, dense to very dense	9	DO	33
	SILTY SAND TILL	10	DO	36
	some gravel, a trace of clay occ. cobbles and boulders	11	DO	90
73.9				
14.2	**END OF BOREHOLE**			
	Installed 50 mm Ø PVC monitoring well to 6.1 m with 3.0 m screen Sand backfill from 2.7 to 6.1 m Bentonite seal from 0.0 m to 2.7 m Provided with a steel monument casing			



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GRAIN SIZE DISTRIBUTION

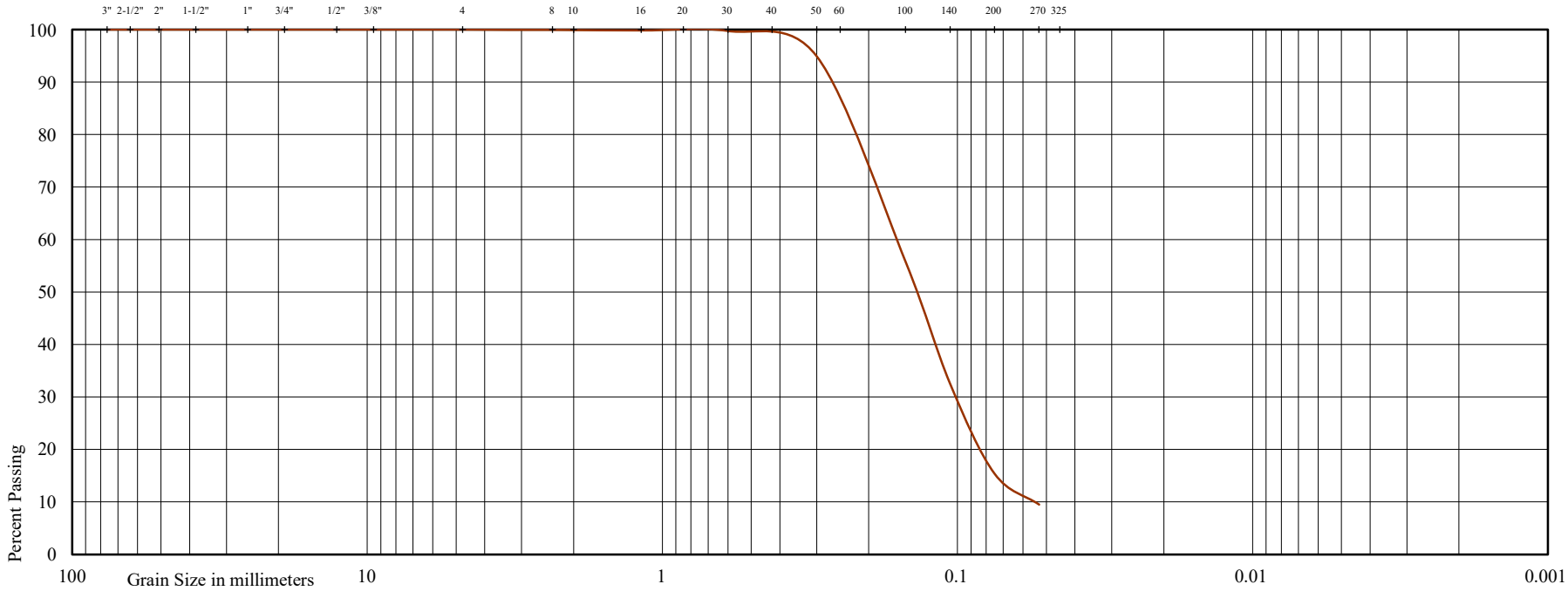
Reference No: 2411-S164

U.S. BUREAU OF SOILS CLASSIFICATION

GRAVEL		SAND				SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	V. FINE		

UNIFIED SOIL CLASSIFICATION

GRAVEL		SAND			SILT & CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Project: Proposed Stacked Townhouse Development

Location: 1884 Liverpool Road and 1885 Glendale Drive, City of Pickering

Borehole No: 2

Sample No: 2

Depth (m): 1.0

Elevation (m): 88.6

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 8

Estimated Permeability (cm./sec.) = 10^{-3}

Classification of Sample [& Group Symbol]: SAND

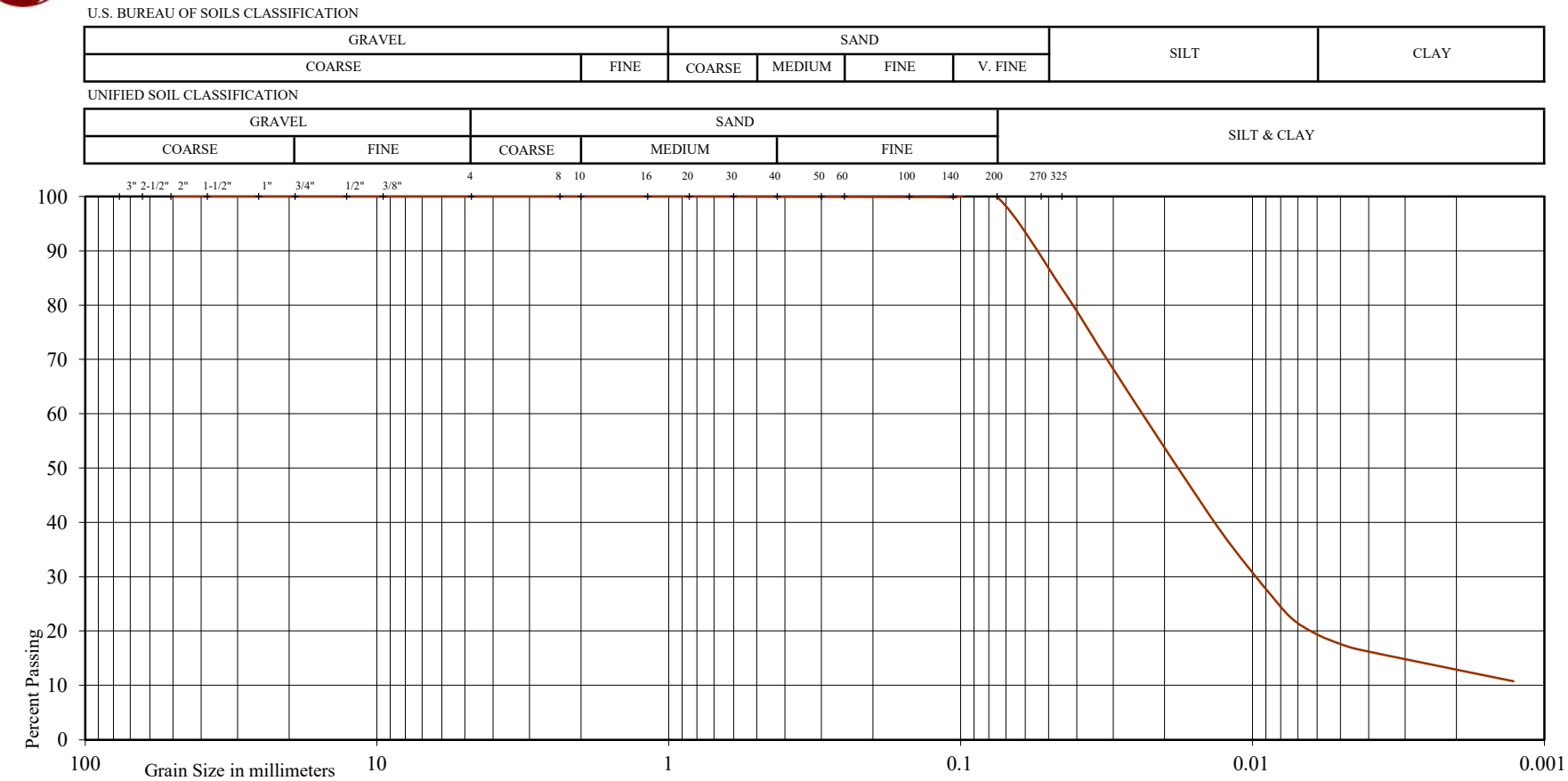
fine-grained, some silt

Figure: 6

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 2411-S164



Project: Proposed Stacked Townhouse Development

Location: 1884 Liverpool Road and 1885 Glendale Drive, City of Pickering

Borehole No: 1

Sample No: 7

Depth (m): 6.3

Elevation (m): 85.0

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 13

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

Classification of Sample [& Group Symbol]: SILT

some clay

Figure: 7

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

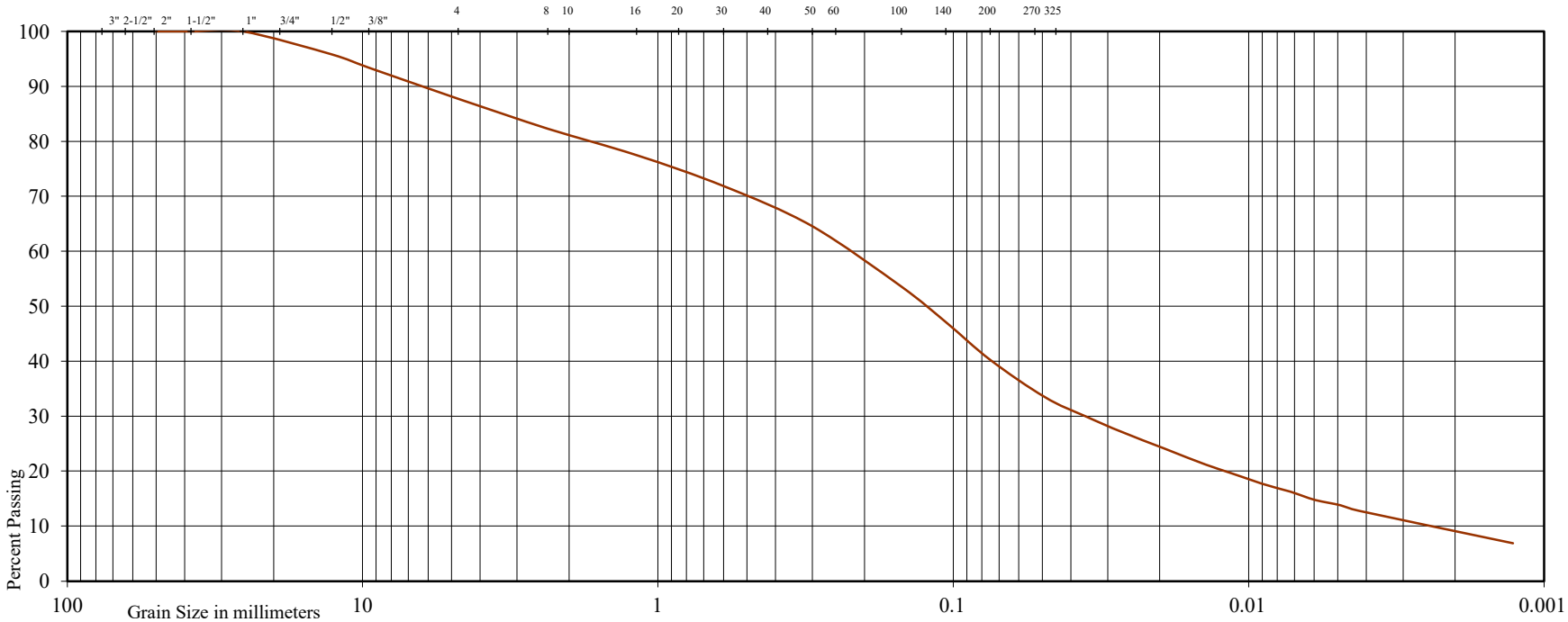
Reference No: 2411-S164

U.S. BUREAU OF SOILS CLASSIFICATION

GRAVEL		SAND				SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	V. FINE		

UNIFIED SOIL CLASSIFICATION

GRAVEL		SAND			SILT & CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Project: Proposed Stacked Townhouse Development

Location: 1884 Liverpool Road and 1885 Glendale Drive, City of Pickering

Borehole No: 5

Sample No: 10

Depth (m): 12.4

Elevation (m): 75.7

Liquid Limit (%) = -

Plastic Limit (%) = -

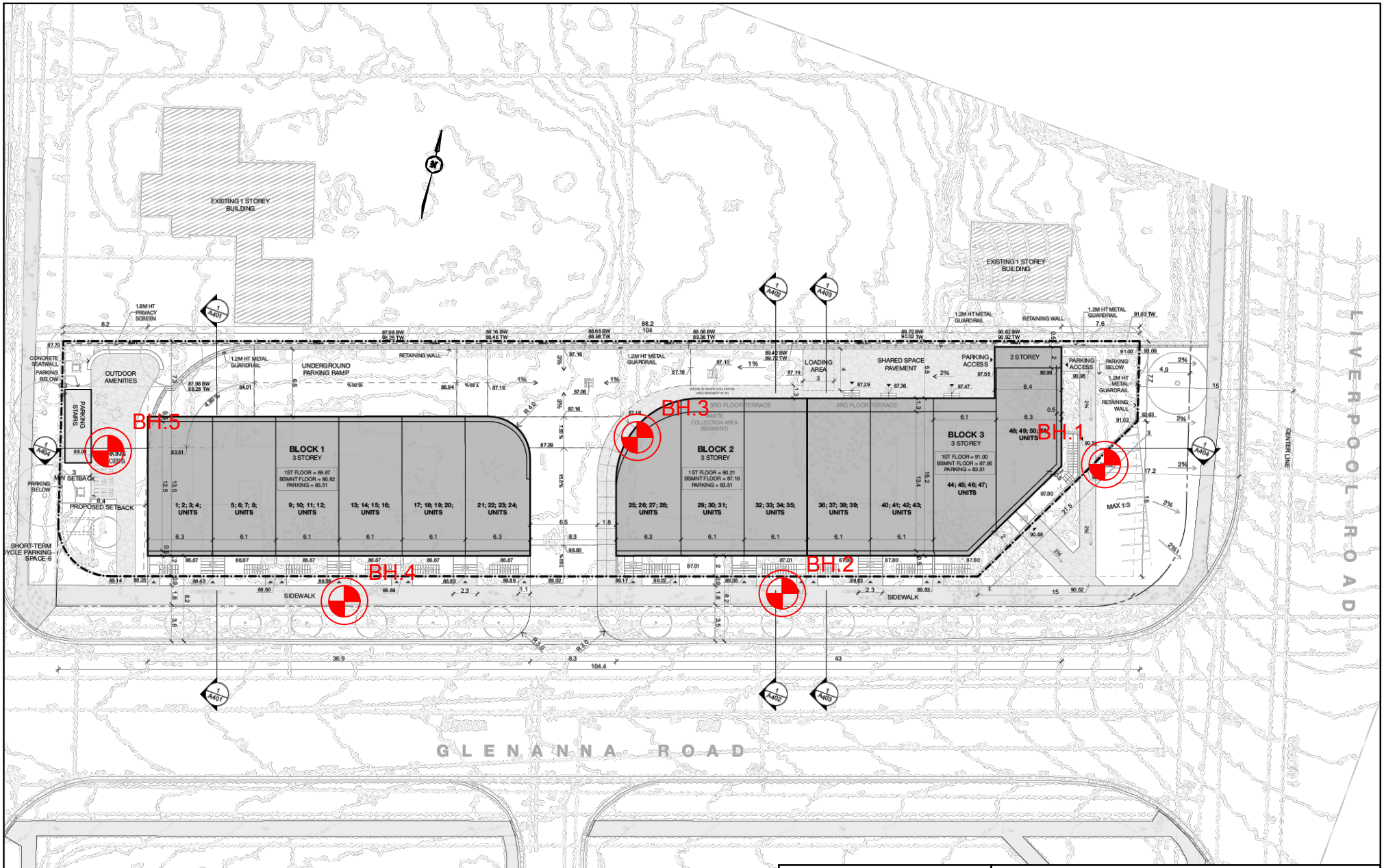
Plasticity Index (%) = -

Moisture Content (%) = 9

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

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

Figure: 8



LEGEND

- Borehole with monitoring well

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BOREHOLE AND MONITORING WELL LOCATION PLAN

SITE: 1884 Liverpool Road, City of Pickering

DESIGNED BY: -	**CHECKED BY:** -	**DWG NO.:** 1	
SCALE: 1:2000	**REF. NO.:** 2411-S164	**DATE:** March 2026	**REV** 1

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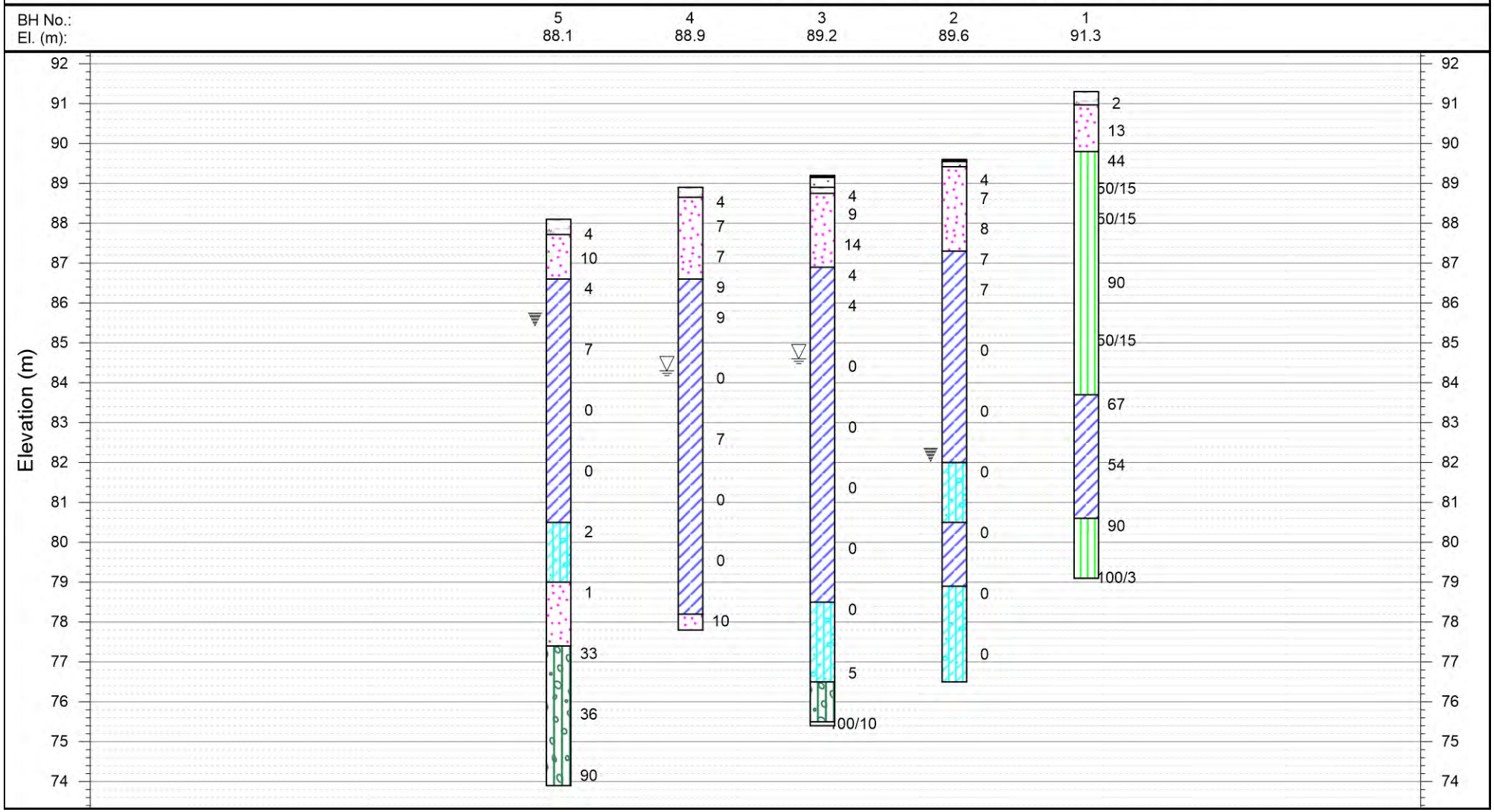
SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

JOB NO.: 2411-S164
REPORT DATE: March 2026
PROJECT DESCRIPTION: Proposed Stacked Townhouse Development
PROJECT LOCATION: 1884 Liverpool Road and 1885 Glendale Drive
City of Pickering

LEGEND

TOPSOIL, SAND, SILT, SILTY CLAY TILL, ASPHALT, SILTY SAND TILL, SILTY CLAY, SHALE, GRANULAR

WATER LEVEL (END OF DRILLING), CAVE-IN



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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. 2411-S164

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 should be calculated from the following expressions:

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

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

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

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

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

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

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

LAGGING

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

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

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

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

TIEBACK ANCHORS

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

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

The tieback anchor lengths can be estimated using an adhesion value of 20 kPa within the overburden. 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 clay/clay till below the bottom of excavation should be designed for the allowable bearing pressure of 25 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.

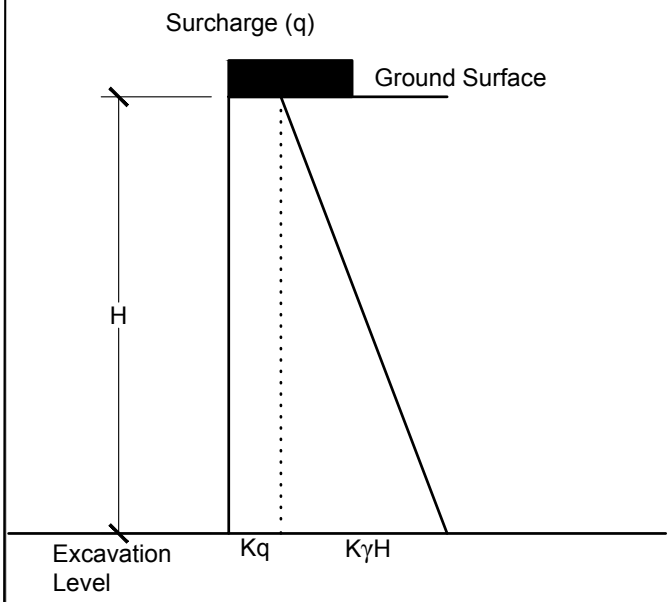
To prevent undermining of the raker footing, no excavation should be made within two times the width of raker footing on the opposite side of the raker.

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.

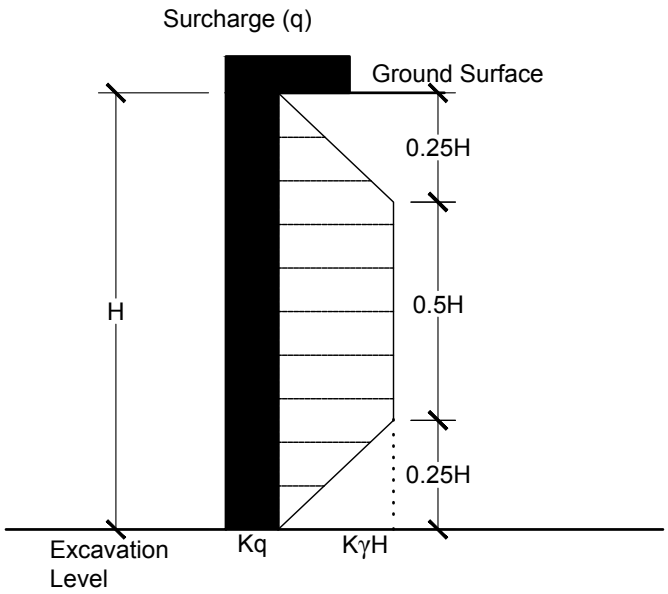
MONITORING OF PERFORMANCE

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	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 ON TEMPORARY SHORING

SITE: 1884 Liverpool Road and 1885 Glendale Drive, City of Pickering

DESIGNED BY: K.L.	**CHECKED BY:** B.S.	**DWG NO.:** A1	
SCALE: N.T.S.	**REF. NO.:** 2411-S164	**DATE:** March 2026	**REV** -