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A REPORT TO
PARADISE DEVELOPMENTS HERON'S HILL INC.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED MIXED USE BUILDING

1 HERON'S HILL WAY

CITY OF TORONTO

REFERENCE NO. 1908-S037

APRIL 2020
(REVISION OF REPORT DATED NOVEMBER 2019)

DISTRIBUTION

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1.0 INTRODUCTION

In accordance with the proposal dated August 9, 2019, Soil Engineers Ltd. was retained by Paradise Developments Heron’s Hill Inc. to perform a geotechnical investigation at the property of 1 Heron’s Hill Way in the City of Toronto.

The investigation is to reveal the subsurface conditions and to determine the engineering properties of the disclosed soils for the design and construction of a proposed mixed use building.

The geotechnical findings and resulting recommendations for the proposed development are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The City of Toronto is situated on Markham till plain where drift from three glacial periods dominates the soil stratigraphy. The drift is interstratified with lacustrine sand, silt, clay and interglacial sand.

The subject property, encompasses an area of 6,491 m^2, is located at the southeast quadrant of Yorkland Road and Heron’s Hill Way in the City of Toronto. At the time of investigation, the property consists of an office building at the west portion with a parking lot at the mid portion. The east portion is vacant. The existing site gradient is relatively flat, dropping slightly towards the east.

According to the concept plan prepared by Graziani + Corazza Architects Inc. dated March 17, 2020, a mixed-use building will be constructed to the east of the office building. The building will be a 39-storey structure with one underground parking level. The first 4 storeys will be used for above ground parking, office and amenity while the upper floors will be residential.

3.0 FIELD WORK

The field work, consisting of six (6) sampled boreholes and extending to depths of 21.4 to 30.6 m below the prevailing ground surface, was conducted between August 14 and 21, 2019. The borehole locations are shown on the Borehole Location Plan, Drawing No. 1.

The boreholes were advanced at intervals to the sampling depths by continuous-flight power-auger machines equipped with automatic hammer for Standard Penetration Tests and split-spoon sampler for soil sampling. Standard Penetration Tests, using the procedures described on the enclosed "List of Abbreviations and Terms", were performed in the overburden 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 granular 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.

Upon completion of drilling and sampling, a monitoring well was installed in each borehole for groundwater records and hydrogeological study. The depth and details of the monitoring wells are shown on the borehole logs, Figures 1 to 6.

The ground elevation at each borehole location was surveyed using the "Top of Manhole" located at the driveway entrance, as a temporary benchmark, having a geodetic elevation of 175.23 m, as shown on the Survey Plan prepared by R. Avis Surveying Inc. dated January 3, 2018.

4.0 SUBSURFACE CONDITIONS

The investigation revealed that beneath the topsoil veneer or pavers, with granular fill and a layer of earth fill in places, the site is underlain by glacial tills, with layers of sand and sandy silt.

Detailed descriptions of the subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 6, 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** (Boreholes 1 and 2)

The existing parking lot consists of interlocking stone pavers. At Boreholes 1 and 2, the paver is overlying a granular fill of 0.2 m and 1.0 m in thickness, respectively.

4.2 **Topsoil** (Boreholes 3 to 6, inclusive)

Topsoil, approximately 10 cm and 20 cm in thickness, was encountered at the ground surface in the vacant portion. The thickness of topsoil may vary randomly between boreholes and thicker topsoil layers may occur in places, especially in low-lying areas.

4.3 **Earth Fill** (All Boreholes)

A layer of earth fill was encountered at the borehole locations. It consists of sandy silt with pockets of sand and gravel. Asphalt and brick debris were encountered in some boreholes at depths of 1.5 to 1.8 m. The earth fill extends to a depth of 2.0 to 3.2 m from the prevailing ground surface.

A layer of granular fill was encountered below the topsoil at Borehole 4 location, extending to a depth of 1 m from the existing grade. It may represent the spoil material from the parking lot construction.

4.4 **Silt** (Boreholes 1 and 4)

The silt deposit was encountered beneath the earth fill, extending to a depth of 4.0 and 5.5 m from grade, respectively.

Sample examination revealed that the silt deposit consists of clay and sand seams. It is moist to wet, as confirmed by the natural water content of 14% to 24%, with a median of 21%.

The obtained ‘N’ values ranged between 7 and 31, with a median of 13 blows per 30 cm of penetration, indicating that the relative density of the silt deposit is loose to dense, being generally compact.

The following engineering properties of the silt deposit are deduced:

- High frost susceptibility and highly water erodible.
- The soil has a high capillarity and water retention capacity.

- The silt is a frictional soil where its shear strength is density dependent. Due to its dilatancy, the strength of the wet silt is susceptible to impact disturbance.
- In excavation, the wet silt will slough and run slowly with seepage bleeding from the cut face.
- Moderate corrosivity to buried metal, with an estimated electrical resistivity of 4500 ohm·cm.

4.5 **Silty Clay** (Boreholes 2, 3, 5 and 6)

The silty clay deposit was encountered below the earth fill, extending to a depth of 4.1 to 5.6 m from the prevailing ground surface. It is laminated with silt and sand layers, and the varved structure shows the clay is a lacustrine deposit.

The obtained ‘N’ values range from 6 to 20, with a median of 11 blows per 30 cm of penetration, indicating that the consistency of the silty clay is firm to very stiff, being generally stiff.

The natural water content of the silty clay samples range from 12% to 25%, with a median of 19%, showing a moist condition.

The following engineering properties of the silty clay are deduced:

- High frost susceptibility and high soil adfreezing potential.
- The laminated sand and silt layers are water erodible.
- The deposit is a cohesive-frictional soil where the shear strength is derived from consistency and augmented by the internal friction of the silt. The overall shear strength of the silty clay 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.
- In excavation, the clay will generally be stable in a relatively steep cut; however, long exposure will allow the silt seams to become saturated which may lead to localized sloughing.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 3000 ohm·cm.

4.6 **Silty Sand Till/ Sandy Silt Till** (All Boreholes)

The silty sand till and sandy silt till are predominant in the stratigraphy up to a depth of 10 to 21 m. A lower sandy silt till deposit was also encountered at a depth of 29 m in Boreholes 4.

The silt till and sand till consist of a random mixture of particle sizes ranging from clay to gravel, with sand and silt being the dominant fraction. They are amorphous in structure. Tactile examinations of the soil samples indicated that the tills are slightly cemented, displaying some cohesion. Grain size analyses were performed on four (4) representative samples. The results are presented on Figure 7.

Intermittent hard resistance to augering was encountered, indicating the presence of cobbles and boulders in the strata.

The obtained 'N' values range from 6 to over 100, with a median of 23 blows per 30 cm of penetration. This shows that the relative density of the till deposits is loose to very dense, being generally compact.

The natural water content values of the samples were determined; the results are plotted on the Borehole Logs. The values range from 8% to 22%, with a mean of 12%, indicating moist to very moist conditions, being generally moist.

The following engineering properties of the till deposits are deduced:

- High frost susceptibility and moderately water erodible to the sand seams and layers.
- The tills are frictional soils; the shear strength is primarily derived from the internal friction and is augmented by cementation.
- They will be stable in steep cuts; however, under prolonged exposure, localized sheet collapse will likely occur.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5000 ohm·cm.

4.7 **Silty Clay Till** (Boreholes 1 to 5, inclusive)

The silty clay till deposit was encountered in the boreholes at the lower stratigraphy, below 12.8 to 20.4 m from the prevailing ground surface. Similar to the silt till, 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. Hard resistance was encountered during augering, showing the till is embedded with occasional cobbles and boulders.

The obtained ‘N’ values of the silty clay till range from 38 to over 100 blows per 30 cm of penetration, showing the consistency of the clay till is hard.

The natural water content of the clay till samples ranges from 8% to 23%, with a median of 14%, indicating moist conditions.

The following engineering properties are deduced:

- High frost susceptibility and high soil adfreezing potential.
- The clay till is cohesive-frictional soils, the shear strength is derived from consistency and augmented by the internal friction of the sand and silt.
- In excavation, the clay till will generally be stable in a relatively steep cut; however, long exposure will allow the sand and silt to become saturated which may lead to localized sloughing.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 3500 ohm·cm.

4.8 **Sand** (Boreholes 2, 4, 5 and 6)

The sand deposit was contacted in some boreholes, below the silty sand till at a depth of 10.0 m to 14.0 m from grade. It is fine grained or well graded, with silt and gravel. Grain size analyses were performed on two (2) selected samples. The results are presented on Figures 8 and 9.

The obtained ‘N’ values range from 13 to over 100, with a median of 26 blows per 30 cm penetration, indicating its relative density is compact to very dense, being generally compact.

The natural water content value of the sand samples ranges from 9% to 18%, with a median of 11%, indicating moist to wet, being generally in very moist conditions. Due to its pervious nature, water could have been drained during the sampling and packing process. Hence, the in situ water content could be higher.

The following engineering properties of the sand deposit are deduced:

- Low to moderate frost susceptible.
- Highly water erodible.

- The shear strength is derived from internal friction and is density dependent.
- In excavation, the sand will slough and run with water seepage.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5000 ohm·cm.

5.0 GROUNDWATER CONDITION

Potable water was used for borehole drilling, thus the free groundwater was not measured in the boreholes upon completion of drilling.

Groundwater was recorded in the monitoring wells at multiple occasions between August 28 and November 4, 2019. The records are summarized in Table 1.

Table 1 - Groundwater Level in Monitoring Wells

Borehole/ Monitoring Well No.			**1**	**2**	**3**	**4**	**5**	**6**
Ground Elevation (m)			175.2	175.2	175.2	175.3	175.2	175.8
Recorded Groundwater Level	**August 28, 2019**	**Depth (m)**	10.0	11.7	9.6	17.5	15.4	16.5
		Elev. (m)	165.2	163.5	165.6	157.8	159.8	159.3
	September 12, 2019	**Depth (m)**	14.2	15.7	14.1	17.7	15.9	17.9
		Elev. (m)	161.0	159.5	161.1	157.6	159.3	157.9
	September 25, 2019	**Depth (m)**	13.5	15.7	14.1	17.7	15.9	17.9
		Elev. (m)	161.7	159.5	161.1	157.6	159.3	157.9
	October 9, 2019	**Depth (m)**	14.1	15.6	13.9	17.7	15.9	18.0
		Elev. (m)	161.1	159.6	161.3	157.6	159.3	157.8
	October 24, 2019	**Depth (m)**	14.1	15.7	13.7	17.7	15.9	18.0
		Elev. (m)	161.1	159.5	161.5	157.6	159.3	157.8
	November 4, 2019	**Depth (m)**	15.6	13.9	13.4	17.7	15.8	18.0
		Elev. (m)	159.6	161.3	161.8	157.6	159.4	157.8

Groundwater was recorded between depths of 9.6 m and 18.0 m below the prevailing ground surface. The stabilized groundwater level is anticipated between depths of 13.4 m and 18.0 m, or between El. 157.6 m and 161.8 m, which represents the groundwater regime in the vicinity. It is subject to seasonal fluctuation.

Any excavation extending into the groundwater level will require dewatering from closely spaced sump wells. Where continuous sand layer exists below the saturation level, the groundwater yield will become persistent and appreciable.

6.0 **DISCUSSION AND RECOMMENDATIONS**

The investigation revealed that beneath the topsoil veneer or pavers, with granular fill and a layer of earth fill in places, the site is predominantly underlain by loose to dense sandy silt till and silty sand till, overlying hard silty clay till, with compact sand and silt layers.

The stabilized groundwater level is anticipated between depths of 13.4 m and 18.0 m, or between El. 157.6 m and 161.8 m, which represents the groundwater regime in the vicinity. It is subject to seasonal fluctuation.

The proposed building will be a 39-storey structure with one underground parking level. The geotechnical findings warranting special consideration for the proposed building are presented below:

The foundation is below a depth of 4 m from the ground surface. The construction of a conventional underground parking structure will require subsurface drainage to collect the groundwater and dissipate into the sewage system or into a storage cistern. Alternatively, a submerged “tank” structure designing to resist the hydrostatic pressure can be constructed for the underground parking if the removal of groundwater is not practical.

1. Due to the subsoil conditions, the design bearing pressures for conventional footings are limited in the shallower stratum. Thus, the structure should be supported on caisson foundations extending into the sound tills below 18 to 20 m from grade.
2. Due to the proximity of the adjacent structures, temporary shoring will be required for deep excavation. The existing structures and any foundation loading within the angle of repose of 35º should be included in the design of the shoring.
3. A pre-construction survey is strongly recommended for the adjacent structures prior to any construction and excavation activities at the site.

4. The yield of groundwater below a depth of 14 m can be appreciable and persistent that vigorous pumping from closely spaced sump wells will be necessary to maintain stability in the excavation.

The recommendations appropriate for the project are based on the geotechnical findings of this investigation. 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 **Foundation**

It is understood that the proposed building will be a 39-storey structure with one underground parking level. The foundation level is below a depth of 4 m from the ground surface.

In the case where subsurface water cannot be removed for the design of a conventional underground parking structure, the underground structure should be water-proofed and designed as a "tank" with a raft to resist the hydrostatic pressure. The recommended design bearing pressures for a raft foundation at a depth of 4 m is presented below:

- Maximum Allowable End Bearing Pressure (SLS) = 120 kPa
- Factored Ultimate Soil Bearing Pressure (ULS) = 200 kPa

The total and differential settlements of the raft foundation, designing for the bearing pressure at SLS, are estimated to be 25 mm and 20 mm, respectively. A Modulus of Subgrade Reaction of 25 MPa/m can be used for the design of the raft foundation.

The raft foundation must be properly reinforced. A mud slab of lean mix concrete, 6 to 8 cm in thickness, will be required to provide a working platform for the workers to install the reinforcement, after the subgrade soil is inspected and approved by a soils engineer.

Due to the limited bearing capacity at the foundation level, the structure can be supported by caissons extending into the sound silty clay till at approximately 18 to 20 m from the prevailing ground surface. The design bearing pressures of caissons extending into the sound till are provided:

- Maximum Allowable End Bearing Pressure (SLS) = 1.0 MPa
- Factored Ultimate Soil Bearing Pressure (ULS) = 1.5 MPa

The total and differential settlements of caisson foundations, designing for the bearing pressure at SLS, are estimated to be within 25 mm and 20 mm, respectively.

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

Foundations exposed to weathering, or in unheated areas, should have at least 1.2 m of earth cover for protection against frost action. For unheated underground parking structure, having the entrance door closed at most of the time, the earth cover can be reduced to 0.6 m for the perimeter walls and 1.0 m for the interior walls and columns, except in the area in close proximity to ventilation shafts and the door entrances.

The foundations should meet the requirements specified in the Ontario Building Code (OBC) and the structure, supported on a raft foundation, should be designed to resist an earthquake force using Site Classification ‘D’ (stiff soil). Structures supporting on caisson foundations extending into the hard till stratum can be designed to resist an earthquake force using Site Classification ‘C’ (dense soil).

The foundation details of the adjacent structures must be investigated and incorporated into the design and construction of the proposed project. The existing structures and foundation loading within the angle of repose of 35º should be included in the design of the shoring.

It is recommended that a pre-construction survey and a monitoring program be carried out for the adjacent structures in order to verify any potential future liability claims.

6.2 Underground Structure

In conventional construction of one level underground structure, the perimeter walls should be provided with prefabricated drainage board over the entire wall below grade as shown on Drawing No. 3. The subdrains should be shielded by a fabric filter and covered with stone filter to prevent blockage by silting, installed on a positive gradient and discharge to a positive outlet.

The subsurface water should be discharged into the sewer system or a storage cistern. If the removal of groundwater is not possible, the underground structure will have to be waterproofed and designed for the full depth hydrostatic pressure.

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.

The soil parameters stated in Section 6.7 can be used to evaluate the earth pressure and hydrostatic pressure on the underground structure. Any applicable surcharge loads adjacent to the underground structure must also be considered in the design of the foundation.

6.3 Slab-on-Grade Construction

The subgrade for slab-on-grade construction should consist of sound natural soil or properly compacted inorganic earth fill. The subgrade should be inspected prior to slab-on-grade construction. Where soft subgrade is detected, it should be subexcavated and replaced with inorganic material, uniformly compacted to 98% or + of its maximum Standard Proctor dry density (SPDD) prior to placement of the granular base. The slab should be constructed on a granular base, 20 cm thick, consisting of 20-mm Crusher-Run Limestone (CRL), or equivalent, compacted to its maximum SPDD.

If the building is to be founded on a raft foundation, the slab-on-grade will be poured on a granular fill above the foundation where the underground utilities and pipes will be laid.

At the exterior, the concrete slab or sidewalk must be graded to direct water away from the structures to minimize the frost heave phenomenon generally associated with the disclosed soils. To prevent frost action induced by cold wintry drafts in areas where vertical ground movement cannot be tolerated, such as building entrances, interlocking stone pavement and concrete sidewalk must be constructed on free-draining, non-frost-susceptible granular material such as Granular ‘B’. It must extend to 1.2 m below the slab or pavement surface and be provided with positive drainage such as weeper subdrains connected to the storm system. Alternatively, the sidewalks and pavement should be insulated with 50-mm Styrofoam, or equivalent.

In order to prevent frost action induced by cold drafts near the garage entrance and in the areas of close proximity to air ventilation shafts, rigid insulation should be installed underneath the concrete slab and extending 1.5 m internally.

6.4 Underground Services

The subgrade for the underground services should consist of sound natural soils or properly compacted earth fill, free of organics. In areas where the subgrade consists of weathered soils, it should be subexcavated and replaced with bedding material compacted to at least 95% or + of its Standard Proctor Dry Density (SPDD).

A Class ‘B’ bedding, consisting of compacted 20-mm CRL or equivalent, is recommended for construction of the underground services. In water-bearing sand or silt where dewatering is required, a Class ‘A’ concrete bedding should be used.

The pipe joints should be leak-proof, or wrapped with a waterproof membrane to prevent subgrade migration through leakage at joints resulting from inadvertent faulty installation. Openings to subdrains and catch basins should be shielded with a fabric filter to prevent silting.

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

The in situ soils have moderately high corrosivity to buried metal. In determining the mode of protection, an electrical resistivity of 3000 ohm·cm should be used. This, however, should be confirmed by testing the soil along the water main alignment at the time of services construction.

6.5 Backfilling in Trenches and Excavated Areas

The backfill in service trenches and the excavated areas should be compacted to at least 95% of its SPDD and increase to 98% below the concrete floor slab. In the zone within 1.0 m below the subgrade, the backfill should be compacted with the water content 2% to 3% drier than the optimum to at least 98% of its SPDD.

In normal project construction practice, the problem areas of settlement largely occur adjacent to foundation walls, columns, manholes, catch basins and services crossings. In areas which are inaccessible to a heavy compactor, the interface of the native soils and sand backfill will have to be flooded for a period of at least 1 day.

The narrow trenches for services crossings should be cut at 1 vertical:2 horizontal so that the backfill in the trenches can be effectively compacted. Otherwise, soil arching in the trenches will prevent achievement of the proper compaction. In this case, imported sand fill must be used. Unless compaction of the backfill is carefully performed, the areas at the interface of the native soil and the sand backfill should preferably be flooded for at least 1 day.

6.6 Pavement Design

Where the pavement is to be built on structural slabs such as the underground garage rooftop, sufficient granular base and adequate drainage must be provided to prevent frost damage to the pavement. An impervious membrane must be placed above the structural slab exposed to weathering to prevent water leakage as well as to protect the reinforcing steel bars against brine corrosion. The recommended pavement structure to be placed on the underground garage rooftop is presented in Table 2.

Table 2 - Pavement Design (Roof of Underground Garage)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	35	HL-3
Asphalt Binder	60	HL-8
Granular Base	200	20-mm CRL or equivalent
Granular Sub-base	100	Free-Draining Sand Fill

For the on-grade access driveway between the road and the building, the recommended pavement design is presented in Table 3.

Table 3 - Pavement Design on Grade

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	35	HL-3
Asphalt Binder Light Duty Fire Route	 40 60	HL-8
Granular Base	150	20-mm CRL or equivalent
Granular Sub-base Light-Duty Fire Route	 250 350	50-mm CRL or equivalent

The granular base and sub-base should be compacted to 100% of the SPDD.

In order to provide a stable subgrade for pavement construction, it is imperative that the subgrade within the 1.0 m zone below the underside of the granular base be compacted to at least 98% of its SPDD with the moisture content 2% to 3% drier than the optimum. This is to provide adequate stability for the pavement construction.

Along the perimeter, where runoff may drain onto the pavement, a swale or an intercept subdrain system should be installed to prevent infiltrating precipitation from seeping into the granular bases (since this may inflict frost damage on the flexible pavement). Subdrains consisting of filter-wrapped weepers should also be installed in lower spots and they should be connected to the catch basins and storm 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 4.

Table 4 - Soil Parameters

Unit Weight and Bulk Factor

	Unit Weight (kN/m^3)		Estimated Bulk Factor	
	Bulk	**Submerged**	**Loose**	**Compacted**
Granular Fill/Earth Fill/Native Sand and Silt	21.5	11.5	1.25	1.00
Silty Clay	21.0	11.0	1.30	1.00
Silty Sand Till/Sandy Silt Till	22.5	12.5	1.25	1.03
Silty Clay Till	22.0	12.0	1.30	1.05

Lateral Earth Pressure Coefficients

	Active	At Rest	Passive
	K_a	K_o	K_p
Compacted Earth Fill/Silty Clay	0.40	0.55	2.50
Native Sand and Silt	0.35	0.50	3.00
Native Tills	0.30	0.45	3.30

Estimated Coefficient of Permeabilities and Percolation Rates

	Hydraulic Conductivity (K)	Percolation Rate (T)
Silty Clay/Silty Clay Till	10^{-7} cm/sec	over 80 min/cm
Sand	10^{-2} to 10^{-3} cm/sec	5 to 10 min/cm
Silt	10^{-5} cm/sec	25 min/cm
Silty Sand Till/Sandy Silt Till	10^{-5} to 10^{-6} cm/sec	30 to 60 min/cm

6.8 Excavation

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 Tills and Silty Clay	2
Earth Fill, drained Sand and Silts	3
Saturated Soils	4

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

In the excavation of one level underground structure, the groundwater yield from the percolation of surface water will be relatively slow in rate and limited in quantity, which can be removed by conventional pumping from sumps, where necessary. The yield will become moderate to appreciable and likely persistent in the sand deposit below the saturation level of 14 m from ground surface. Any excavation extending into the sand and silt deposits may require dewatering from closely spaced sump wells.

6.9 **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.

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

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of Paradise Developments Heron's Hill Inc., and for review by the designated consultants, financial institutions, and government agencies. Use of this report is subject to the conditions and limitations of the contractual agreement.

The material in the report reflects the judgment of Kin Fung Li, P.Eng., and Bennett Sun, 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 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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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.: 1908-S037

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Residential Development

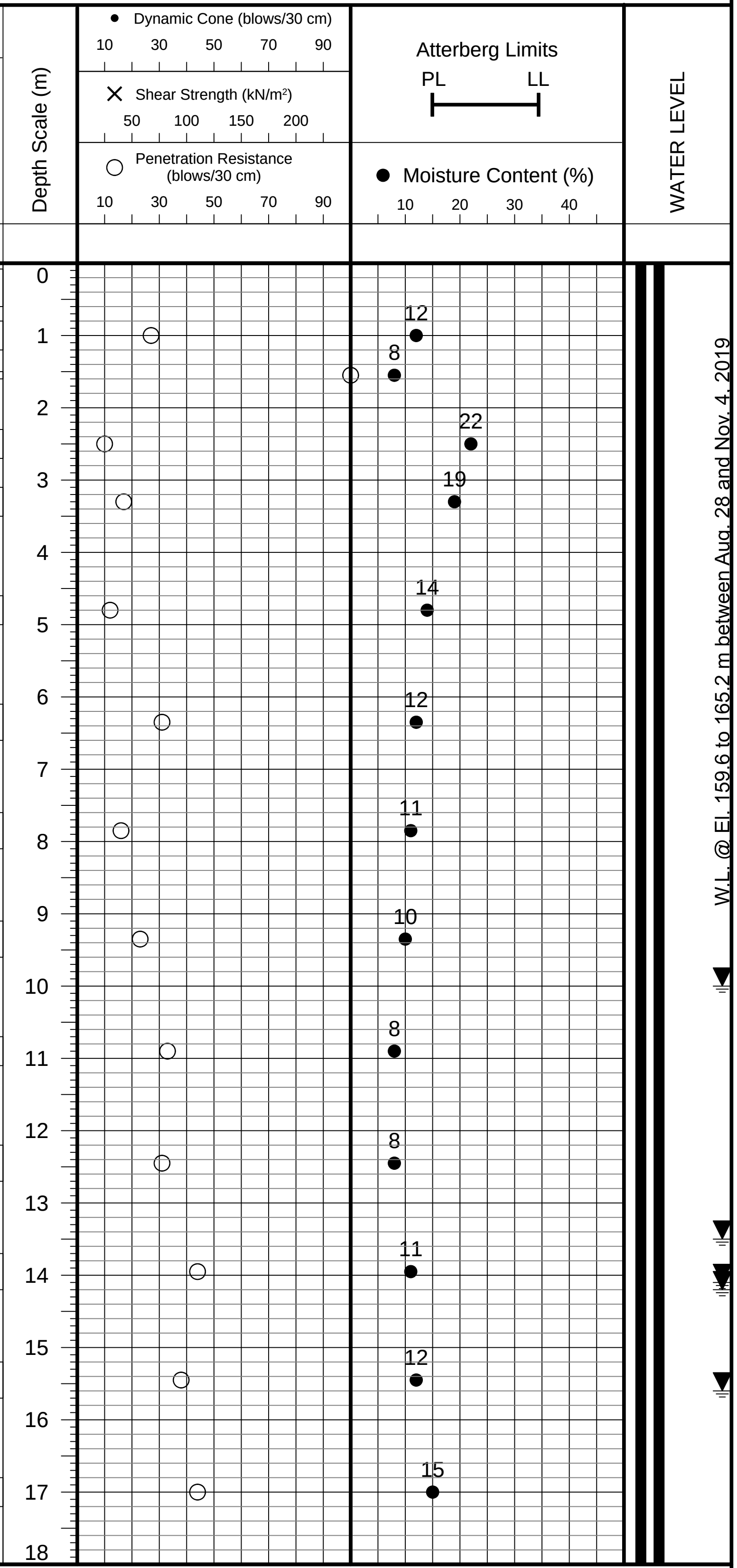
METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 14, 2019

El. (m) Depth (m)	SOIL DESCRIPTION
175.2	Ground Surface
0.0	**INTERLOCKING STONE OVER GRANULAR FILL**
	EARTH FILL grey sandy silt with occ. sand and gravel layers asphalt debris at 1.5 m
173.2 2.0	Brown, compact **SILT** a trace of clay occ. sand seams
171.2 4.0	Comapct to dense **SILTY SAND TILL** traces of clay and gravel occ. silt seams and layers brown grey
162.4 12.8	Grey, hard **SILTY CLAY TILL** low plasticity a trace of gravel occ. sand layers below 18 m
157.2	

SAMPLES		
Number	Type	N-Value
1	DO	-
2	DO	27
3	DO	50/8
4	DO	10
5	DO	17
6	DO	12
7	DO	31
8	DO	16
9	DO	23
10	DO	33
11	DO	31
12	DO	44
13	DO	38
14	DO	44



W.L. @ El. 159.6 to 165.2 m between Aug. 28 and Nov. 4, 2019

Soil Engineers Ltd.

JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

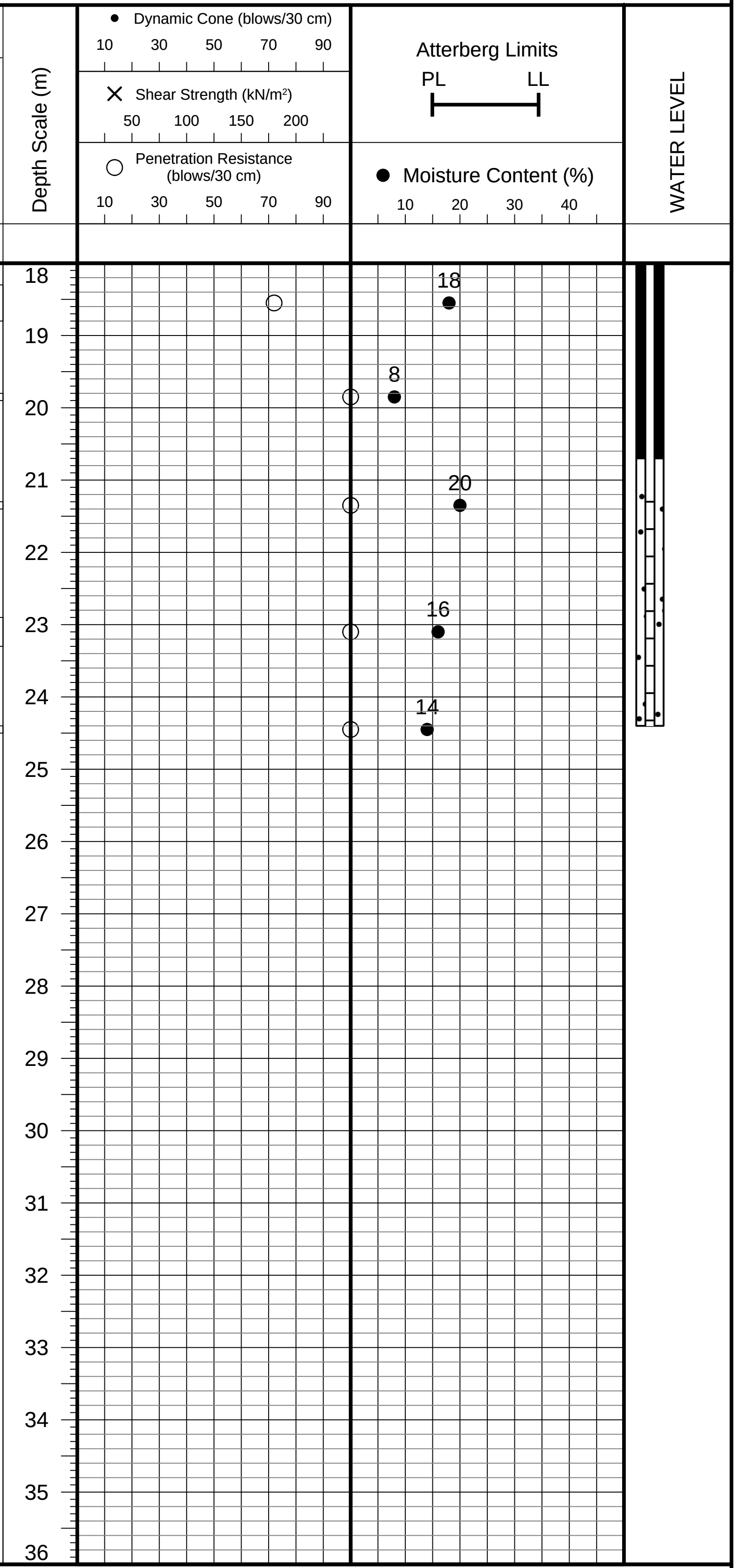
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 14, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
18.0	Grey, hard **SILTY CLAY TILL** low plasticity a trace of gravel occ. sand layers below 18 m	15	DO	72
		16	DO	50/8
		17	DO	50/3
		18	DO	86/23
150.7 24.5	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 24.4 m completed with 3.1 m screen Sand backfill from 20.7 to 24.4 m Bentonite seal from 0.0 to 20.7 m Provided with a monument steel casing	19	DO	50/10



Soil Engineers Ltd.

JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Hollow Stem Auger with Wash Boring

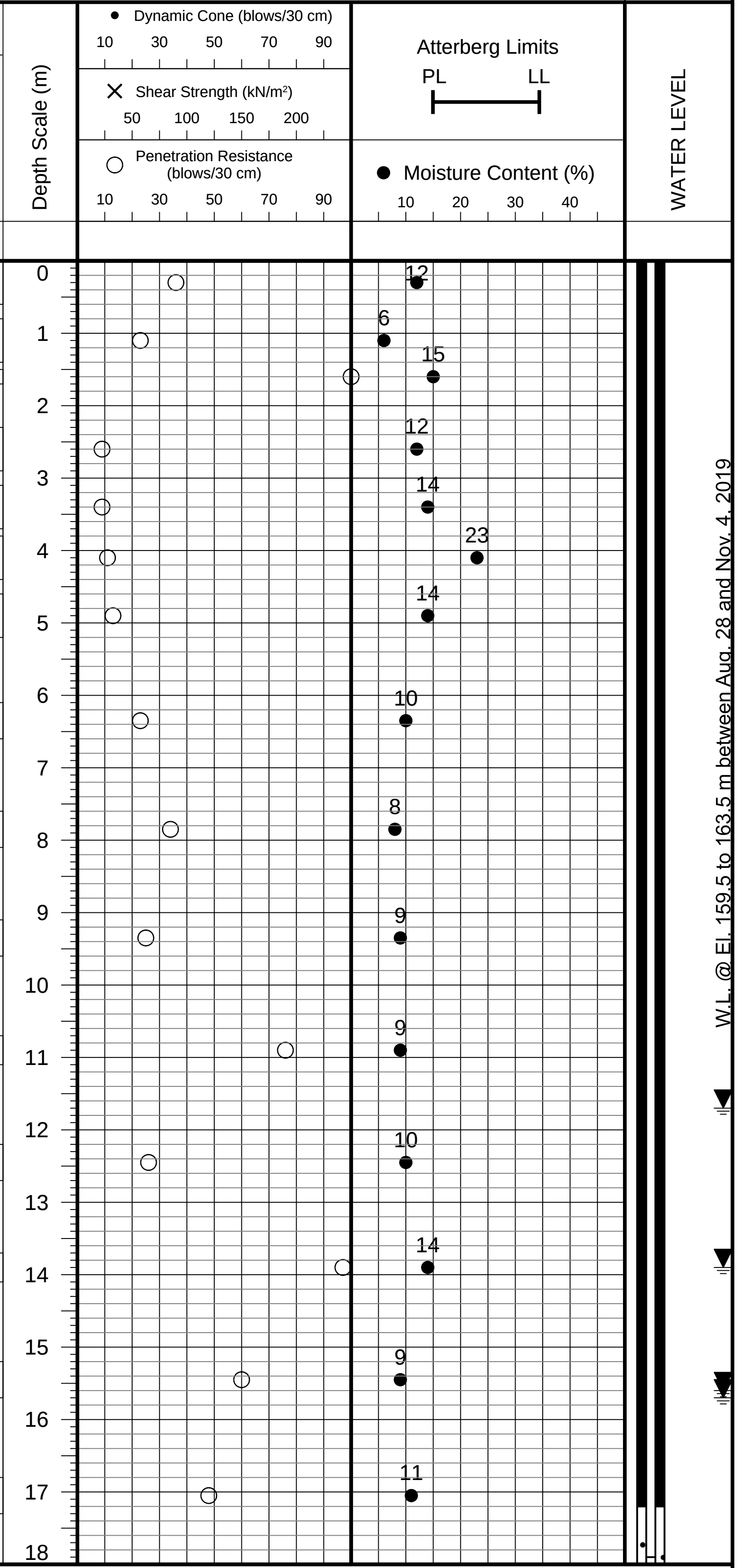
PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 20-21, 2019

El. (m) Depth (m)	SOIL DESCRIPTION
175.2	Ground Surface
0.0	**INTERLOCKING STONE OVER GRANULAR FILL**
	EARTH FILL brown sandy silt occ. sand and gravel pockets
173.1 2.1	Brown, stiff **SILTY CLAY** occ. silt and sand seams and layers
170.6 4.6	Compact to dense **SILTY SAND TILL** traces of clay and gravel — — brown / grey occ. silt seams, cobbles and boulders
164.2 11.0	Grey, compact to very dense **SAND** fine grained silty a trace of clay occ. silt seams and layers
160.6 14.6	Grey, hard **SILTY CLAY TILL** some sand and gravel occ. cobbles and boulders
157.2	

SAMPLES

Number	Type	N-Value
1	DO	36
2	DO	23
3	DO	50/5
4	DO	9
5	DO	9
6	DO	11
7	DO	13
8	DO	23
9	DO	34
10	DO	25
11	DO	63/25
12	DO	26
13	DO	81/25
14	DO	60
15	DO	48



Soil Engineers Ltd.

JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

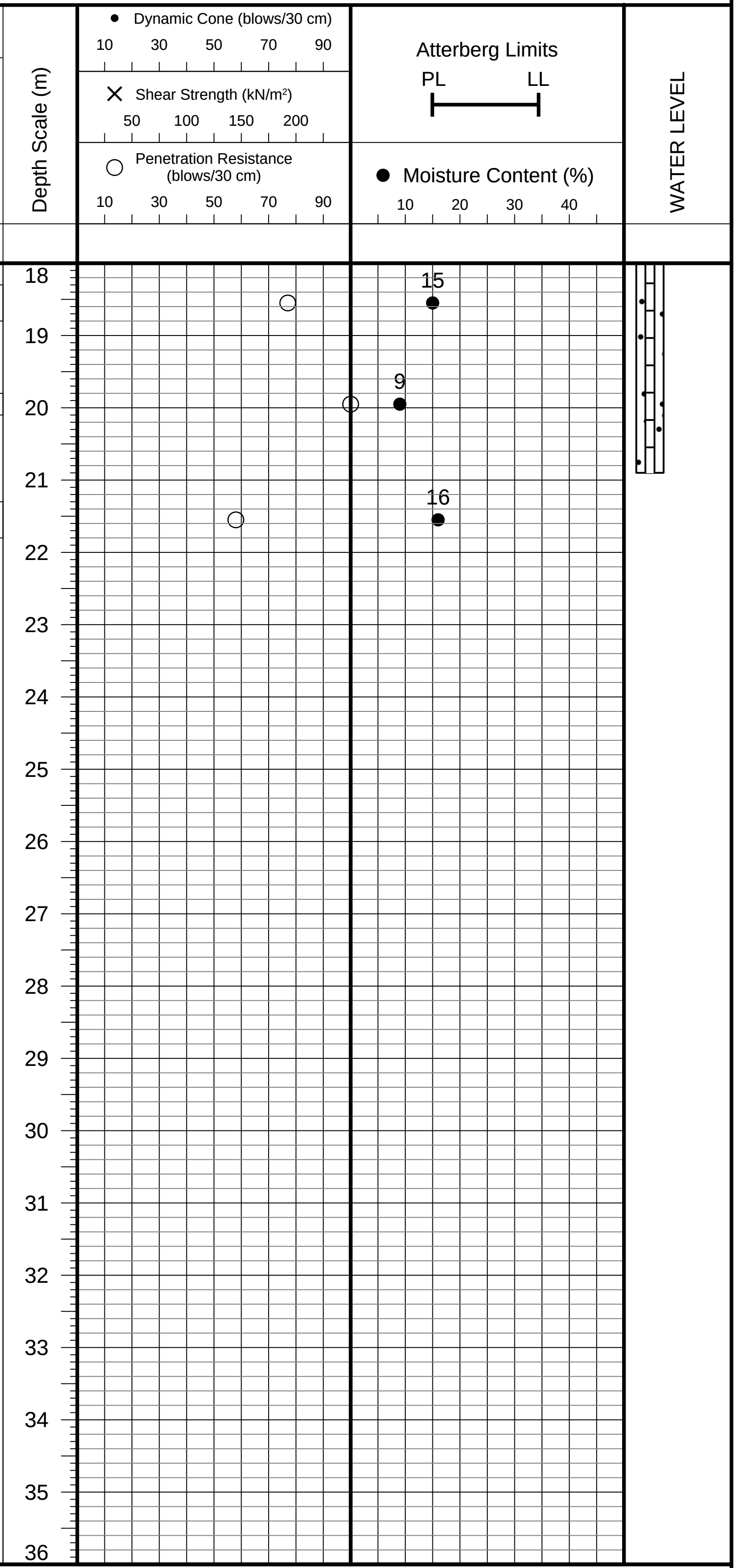
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 20-21, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
18.0	Grey, hard **SILTY CLAY TILL** some sand and gravel occ. cobbles and boulders	16	DO	77
		17	DO	50/13
153.4 21.8		18	DO	58
	END OF BOREHOLE Installed 50 mm Ø monitoring well to 20.9 m completed with 3.1 m screen Sand backfill from 17.2 to 20.9 m Bentonite seal from 0.0 to 17.2 m Provided with a monument steel casing			



Soil Engineers Ltd.

JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

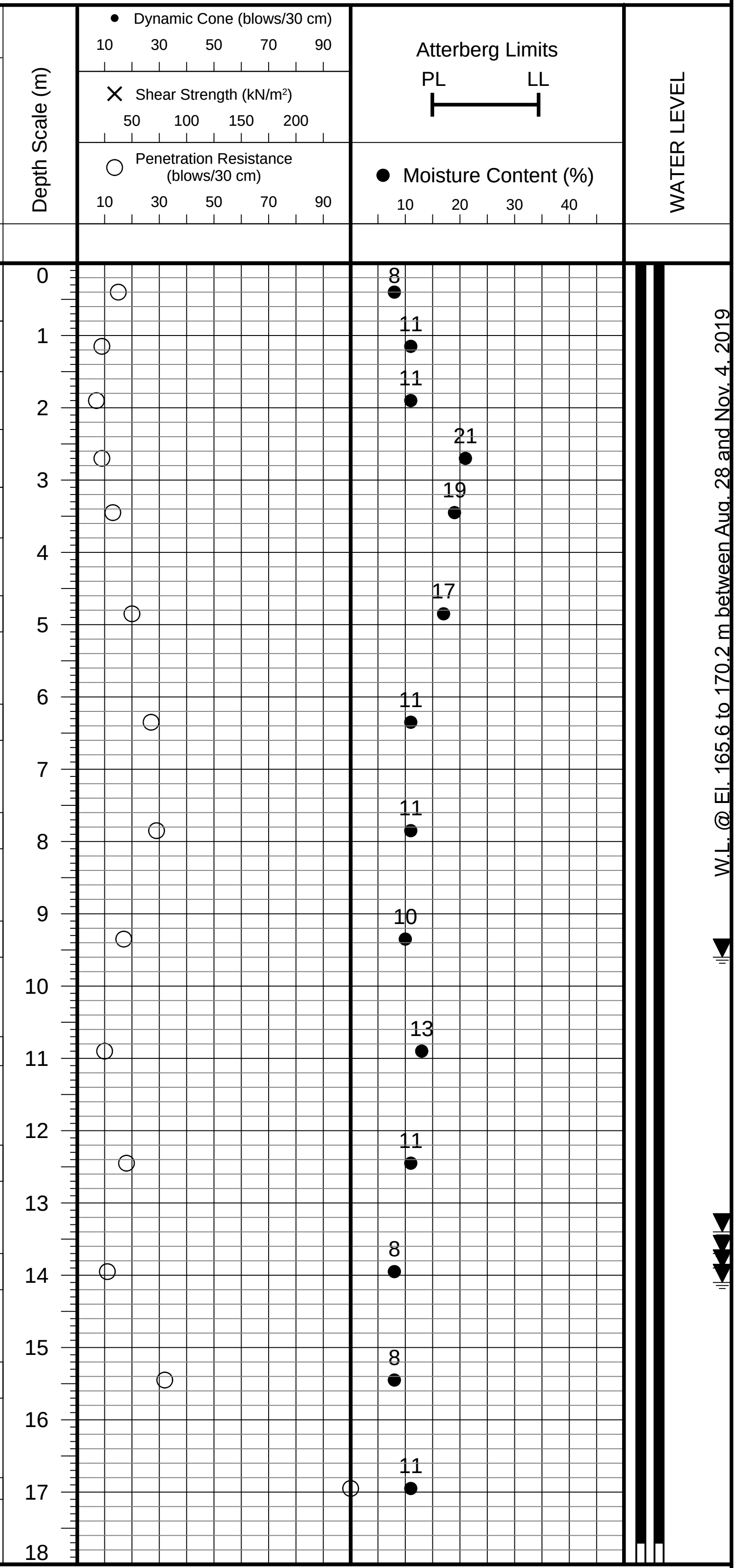
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 20-21, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
175.2	Ground Surface			
0.0	**10 cm TOPSOIL**			
	EARTH FILL dark brown to grey sandy silt with some clay occ. topsoil and rootlet inclusions	1	DO	15
		2	DO	9
		3	DO	7
172.5 2.7	Brown, stiff to very stiff **SILTY CLAY** occ. silt seams and layers	4	DO	9
		5	DO	13
		6	DO	20
170.0 5.2	Grey, compact to very dense **SILTY SAND TILL** traces of clay and gravel occ. silt seams, cobbles and boulders	7	DO	27
		8	DO	29
		9	DO	17
		10	DO	10
		11	DO	18
		12	DO	11
		13	DO	32
		14	DO	50/15
157.2				



W.L. @ El. 165.6 to 170.2 m between Aug. 28 and Nov. 4, 2019

Soil Engineers Ltd.

JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

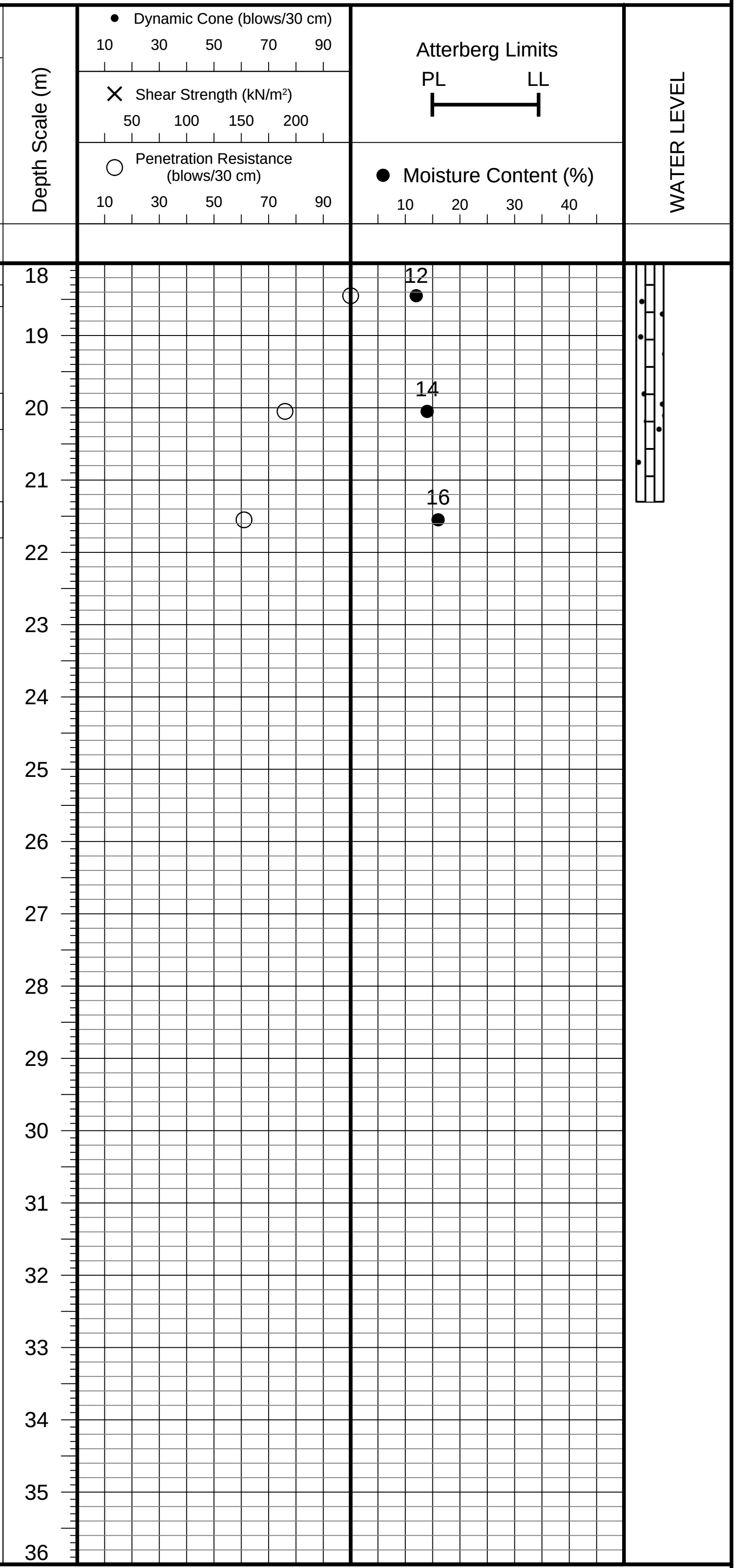
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 20-21, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
18.0	Grey, very dense **SILTY SAND TILL** traces of clay and gravel occ. silt seams, cobbles and boulders	15	DO	50/15
156.0 19.2	Grey, hard **SILTY CLAY TILL** a trace of gravel occ. sand seams, cobbles and boulders	16	DO	76
		17	DO	61
153.4 21.8	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 21.3 m completed with 3.1 m screen Sand backfill from 17.7 to 21.3 m Bentonite seal from 0.0 to 17.7 m Provided with a monument steel casing			



Soil Engineers Ltd.

JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 4

FIGURE NO.: 4

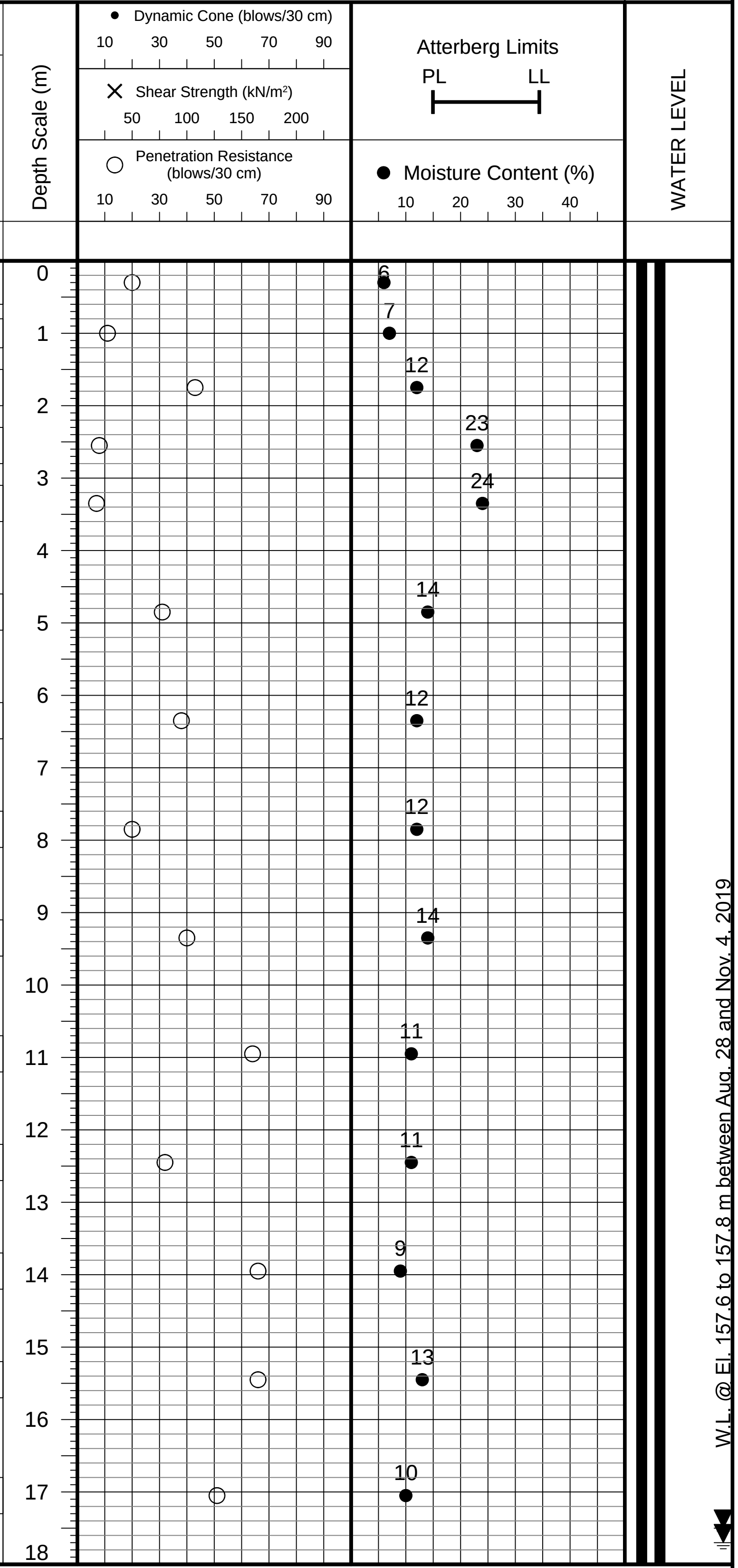
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 14-16, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
175.3	Ground Surface			
0.0	**20 cm TOPSOIL** **GRANULAR FILL**	1	DO	20
	EARTH FILL brown sandy silt — silty clay layers occ. topsoil inclusion and asphalt debris at 1.8 m	2	DO	11
		3	DO	43
		4	DO	8
172.4 2.9	Brown, loose to dense **SILT** a trace of clay occ. fine sand seams	5	DO	7
		6	DO	31
169.8 5.5	Grey, compact to dense **SILTY SAND TILL** a trace of gravel occ. sand seams, cobbles and boulders	7	DO	38
		8	DO	20
		9	DO	40
165.3 10.0	Grey, dense to very dense **SAND AND GRAVEL** some silt a trace of clay	10	DO	64
		11	DO	32
161.3 14.0	Grey, hard **SILTY CLAY TILL** low plasticity a trace to some gravel occ. sand seams and layers	12	DO	66
		13	DO	66
		14	DO	51
157.3				



W.L. @ El. 157.6 to 157.8 m between Aug. 28 and Nov. 4, 2019

Soil Engineers Ltd.

JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 4

FIGURE NO.: 4

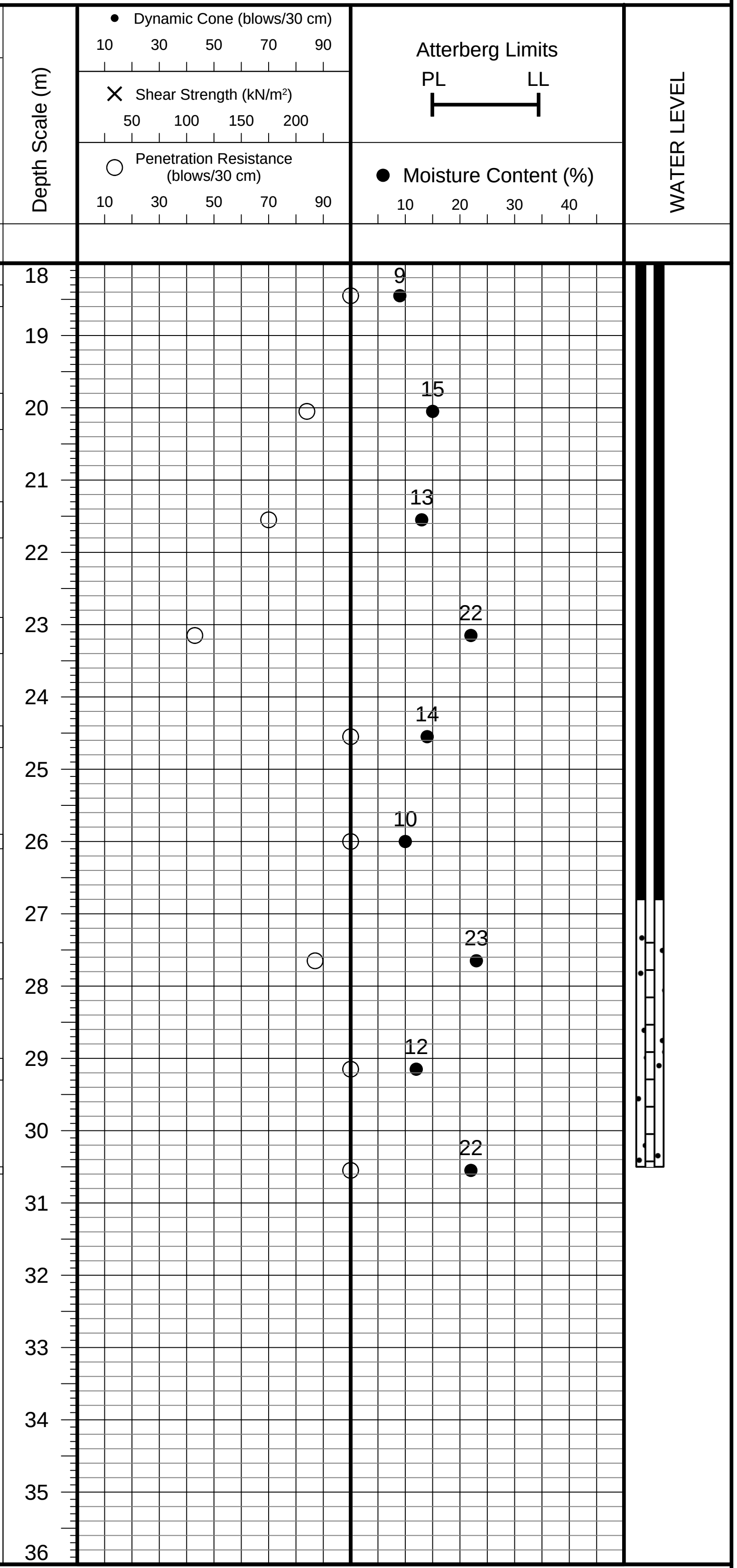
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 14-16, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
18.0	Grey, hard **SILTY CLAY TILL** low plasticity a trace to some gravel occ. sand seams and layers	15	DO	50/15
		16	DO	84
		17	DO	70
		18	DO	43
		19	DO	78/18
		20	DO	54/15
		21	DO	87
146.2 29.1	Grey, very dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams, cobbles and boulders	22	DO	50/13
144.7 30.6	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 30.5 m completed with 3.1 m screen Sand backfill from 26.8 to 30.5 m Bentonite seal from 0.0 to 26.8 m Provided with a monument steel casing	23	DO	50/10



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JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 5

FIGURE NO.: 5

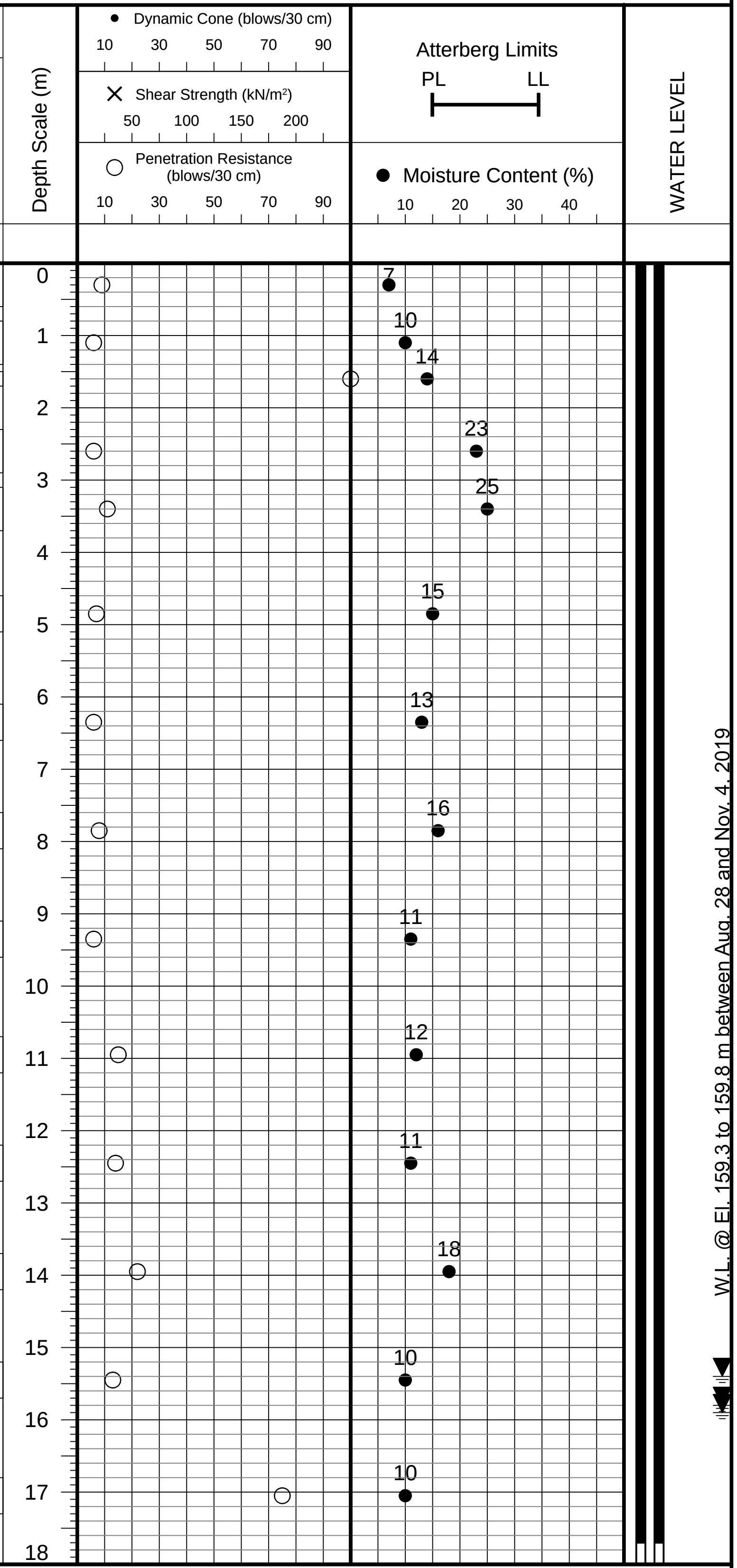
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 20, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES		
		Number	Type	N-Value
175.2	Ground Surface			
0.0	**10 cm TOPSOIL**	1	DO	9
	EARTH FILL brown sandy silt with sand and gravel occ. topsoil and rootlet inclusions	2	DO	6
		3	DO	50/8
172.6 2.6	Brown, firm to stiff **SILTY CLAY** varved occ. silt layers	4	DO	6
		5	DO	11
171.1 4.1	Grey, loose to compact **SILTY SAND TILL** traces of clay and gravel occ. silt layers and cobbles	6	DO	
		7	DO	7
		8	DO	
		9	DO	6
		10	DO	8
		11	DO	6
		12	DO	15
		13	DO	14
	clay layer	14	DO	22
		15	DO	13
159.0 16.2	Grey, very dense **SANDY SILT TILL** some gravel, a trace of clay occ. sand seams and layers, cobbles and boulders	16	DO	75
157.2				



W.L. @ El. 159.3 to 159.8 m between Aug. 28 and Nov. 4, 2019

Soil Engineers Ltd.

JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 5

FIGURE NO.: 5

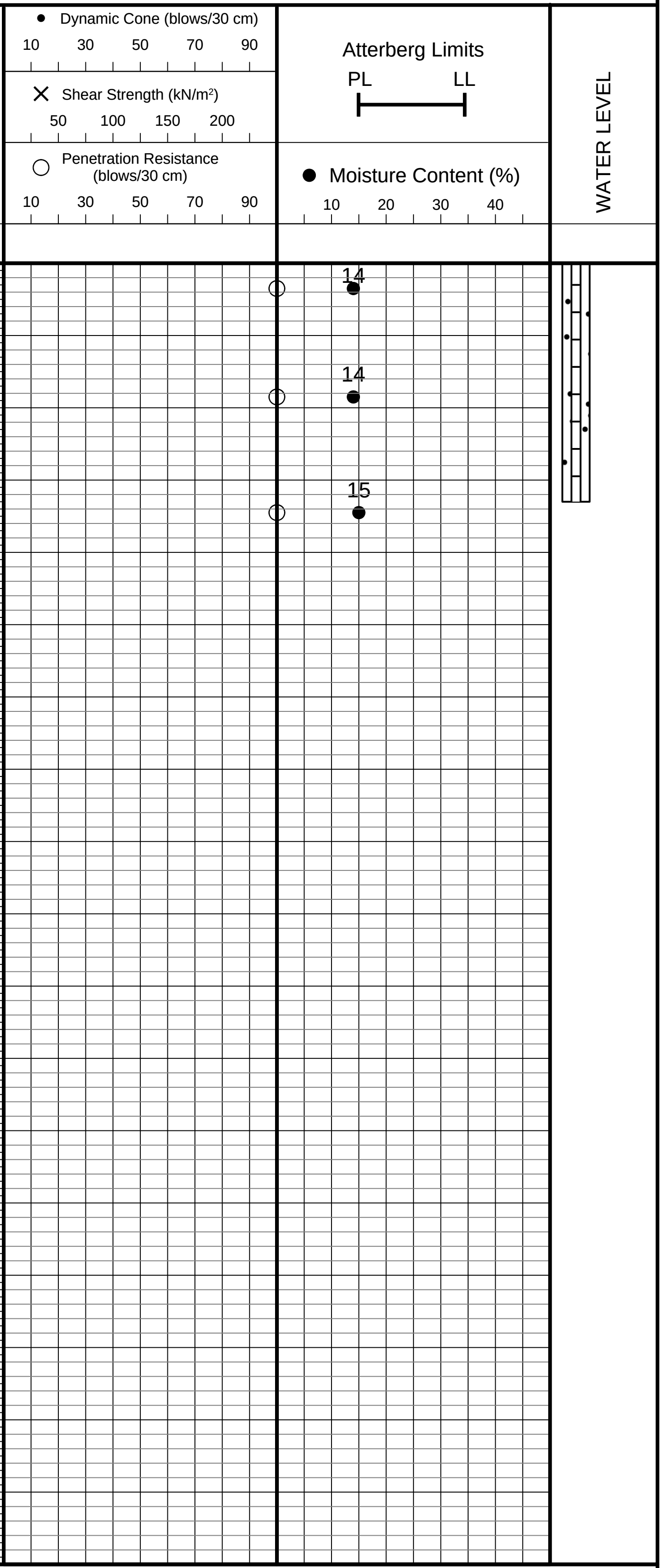
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 20, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)
18.0	Grey, very dense **SANDY SILT TILL** some gravel, a trace of clay occ. sand seams and layers, cobbles and boulders	17	DO	50/13	18
					19
		18	DO	50/13	20
154.8 20.4	Grey, hard **SILTY CLAY TILL** occ. sand seams, cobbles and boulders				21
		19	DO	50/13	
153.6 21.6	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 21.3 m completed with 3.1 m screen Sand backfill from 17.7 to 21.3 m Bentonite seal from 0.0 to 17.7 m Provided with a monument steel casing				22
					23
					24
					25
					26
					27
					28
					29
					30
					31
					32
					33
					34
					35
					36



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JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 6

FIGURE NO.: 6

PROJECT DESCRIPTION: Proposed Residential Development

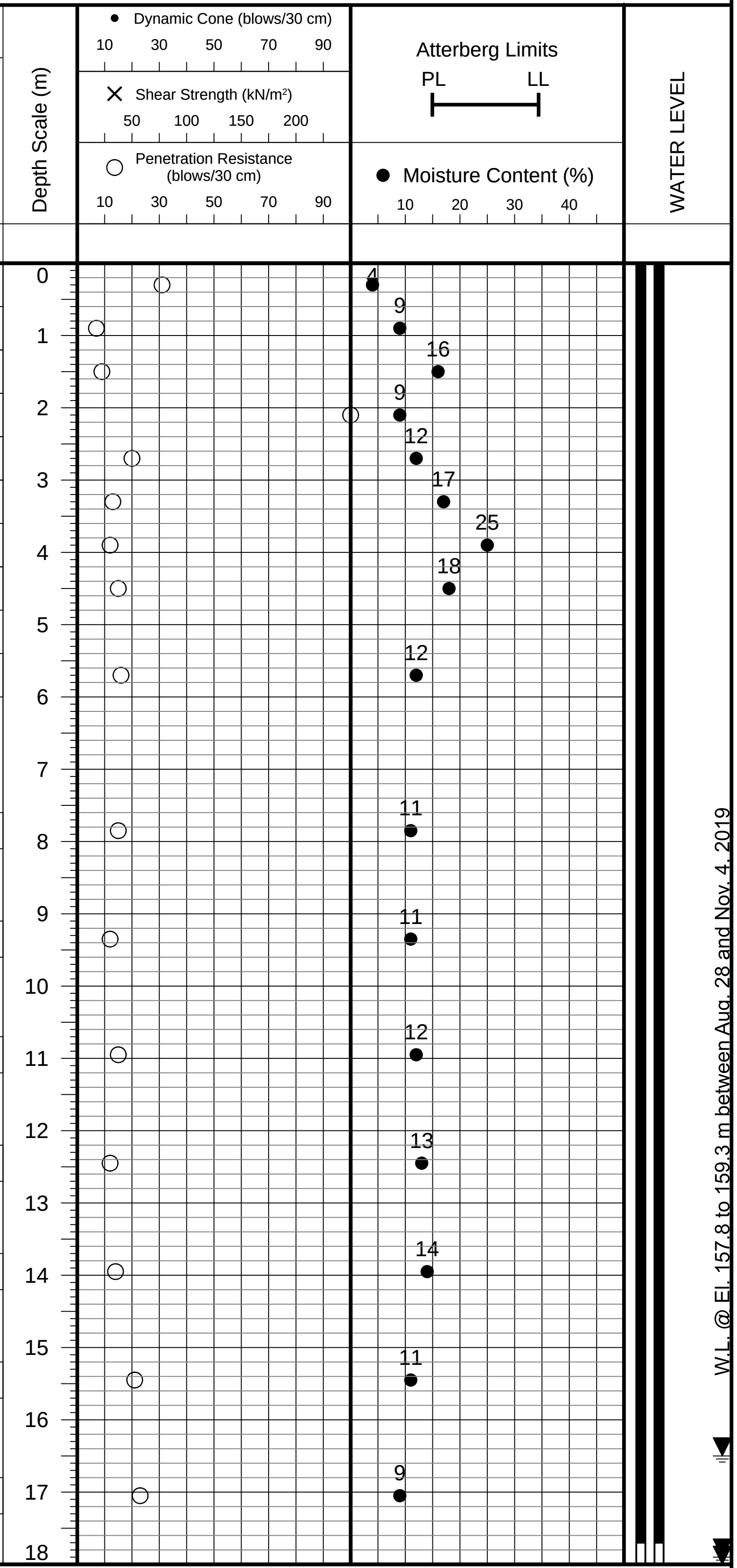
METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 16 & 19, 2019

El. (m) Depth (m)	SOIL DESCRIPTION
175.8	Ground Surface
0.0	**10 cm TOPSOIL**
	EARTH FILL brown sandy silt with sand and gravel asphalt and brick debris below 1.8 m
172.6 3.2	Brown, stiff **SILTY CLAY** occ. sand seams and layers
170.2 5.6	Grey, compact **SILTY SAND TILL** some clay a trace of gravel occ. silt layers, cobbles and boulders
161.8 14.0	Grey, compact **SAND** well graded some silt and gravel
158.7 17.1	Grey, compact to very dense **SANDY SILT TILL**
157.8	

SAMPLES		
Number	Type	N-Value
1	DO	31
2	DO	7
3	DO	9
4	DO	50/8
5	DO	20
6	DO	13
7	DO	12
8	DO	15
9	DO	16
10	DO	15
11	DO	12
12	DO	15
13	DO	12
14	DO	14
15	DO	21
16	DO	23



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JOB NO.: 1908-S037

LOG OF BOREHOLE NO.: 6

FIGURE NO.: 6

PROJECT DESCRIPTION: Proposed Residential Development

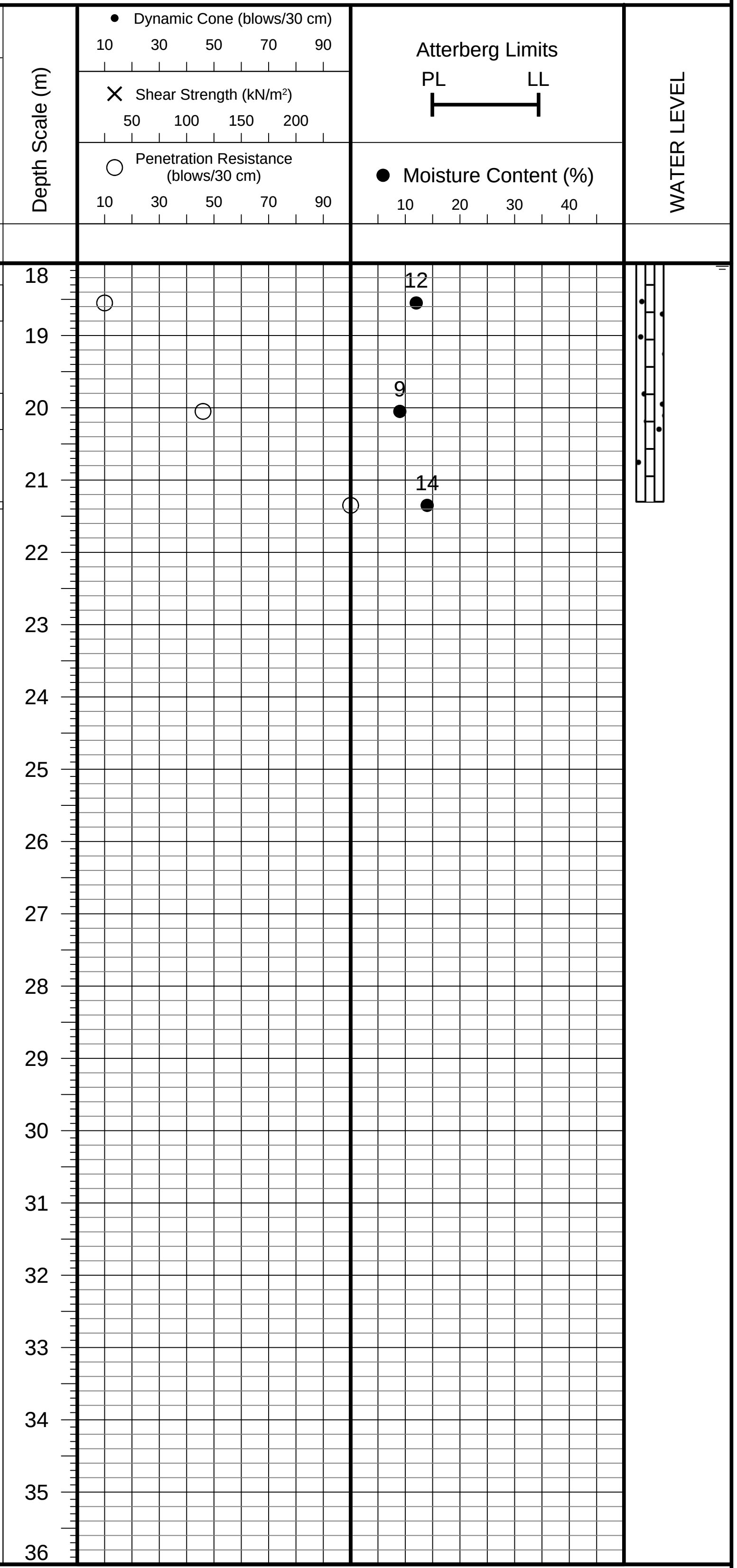
METHOD OF BORING: Hollow Stem Auger with Wash Boring

PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

DRILLING DATE: August 16 & 19, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
18.0	Grey, compact to very dense **SANDY SILT TILL** some clay a trace of gravel occ. sand seams and layers, cobbles and boulders	17	DO	10
		18	DO	46
154.4 21.4	**END OF BOREHOLE**	19	DO	50/10

Installed 50 mm Ø monitoring well to 21.3 m completed with 3.1 m screen
Sand backfill from 17.7 to 21.3 m
Bentonite seal from 0.0 to 17.7 m
Provided with a monument steel casing



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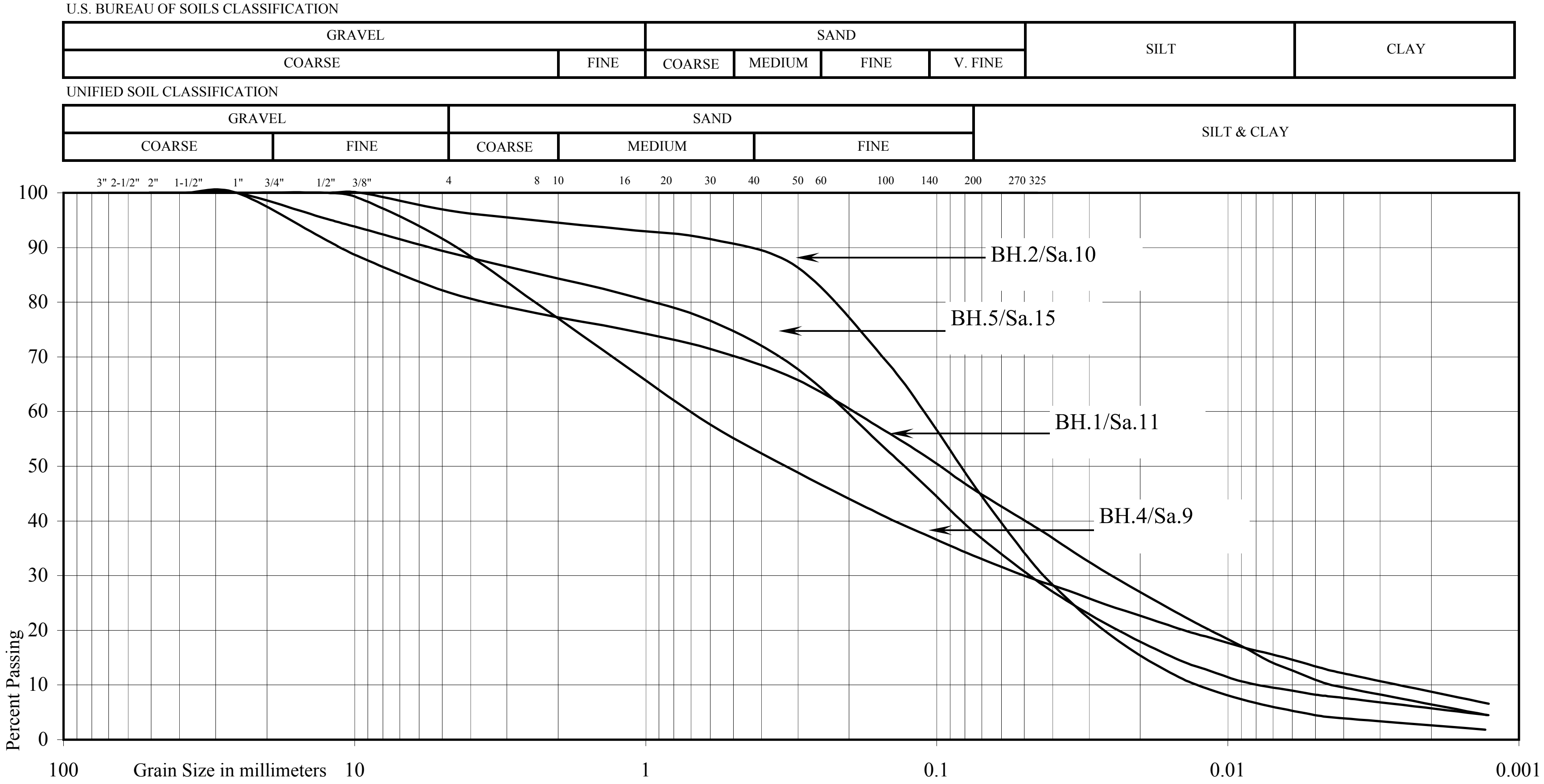
GRAIN SIZE DISTRIBUTION

Reference No: 1908-S037



Project: Proposed Mixed-Use Building

Location: 1 Heron's Hill Way, City of Toronto

Borehole No:	1	2	4	5
Sample No:	11	10	9	15
Depth (m):	12.5	9.3	9.3	15.5
Elevation (m):	87.5	90.7	90.8	84.5

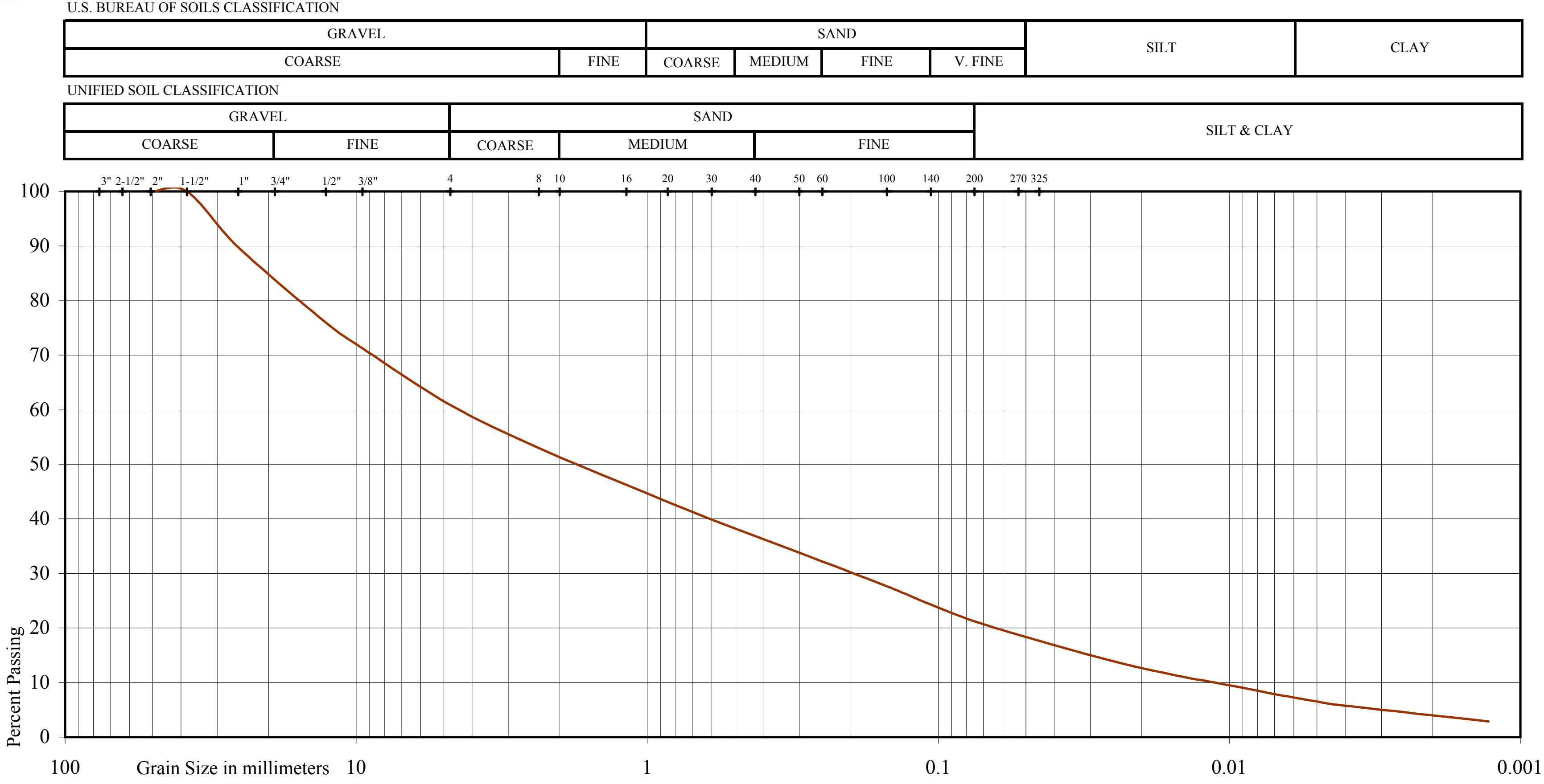
BH./Sa.	1/11	2/10	4/9	5/15
Liquid Limit (%) =	-	-	-	-
Plastic Limit (%) =	-	-	-	-
Plasticity Index (%) =	-	-	-	-
Moisture Content (%) =	8	9	14	10
Estimated Permeability (cm./sec.) =	10^{-6}	10^{-5}	10^{-6}	10^{-5}

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

Figure: 7

GRAIN SIZE DISTRIBUTION

Reference No: 1908-S037



Project: Proposed Mixed-Use Building

Location: 1 Heron's Hill Way, City of Toronto

Borehole No: 4

Sample No: 11 & 12

Depth (m): 11.0 - 12.5

Elevation (m): 89.1 - 87.6

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 11

Estimated Permeability
(cm./sec.) = 10^{-4}

Classification of Sample [& Group Symbol]: SAND AND GRAVEL, some silt, a trace of clay

Figure: 8

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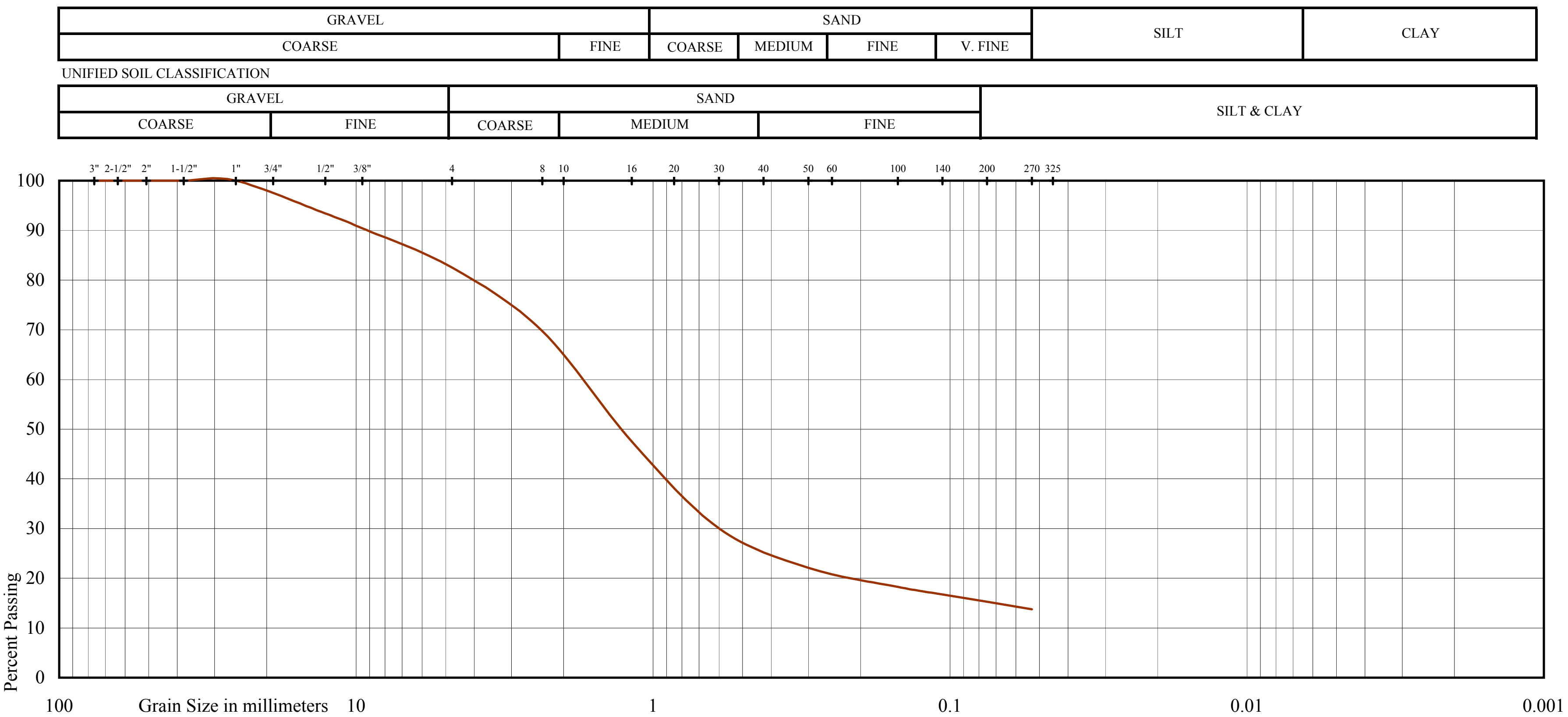
GRAIN SIZE DISTRIBUTION

Reference No: 1908-S037



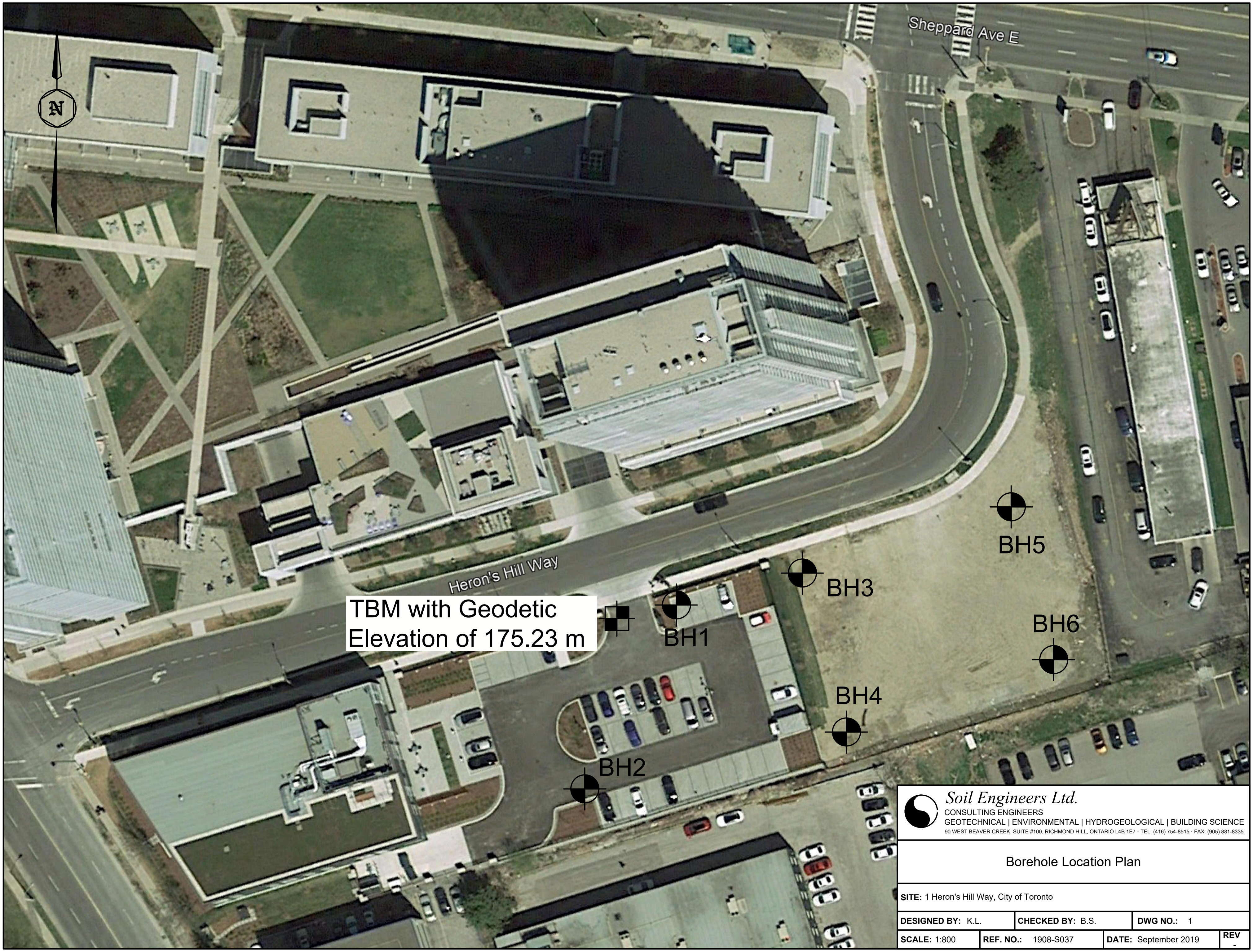
Project:	Proposed Mixed-Use Building
Location:	1 Heron's Hill Way, City of Toronto

Liquid Limit (%) =	-
Plastic Limit (%) =	-
Plasticity Index (%) =	-
Moisture Content (%) =	14
Estimated Permeability (cm./sec.) =	10^{-3}

Borehole No:	6
Sample No:	14
Depth (m):	14.1
Elevation (m):	86.5

Classification of Sample [& Group Symbol]: SAND, well graded, some gravel, a trace of ssilt

Figure: 9



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

Borehole Location Plan

SITE: 1 Heron's Hill Way, City of Toronto

DESIGNED BY: K.L.
CHECKED BY: B.S.
DWG NO.: 1
SCALE: 1:800
REF. NO.: 1908-S037
DATE: September 2019
REV -

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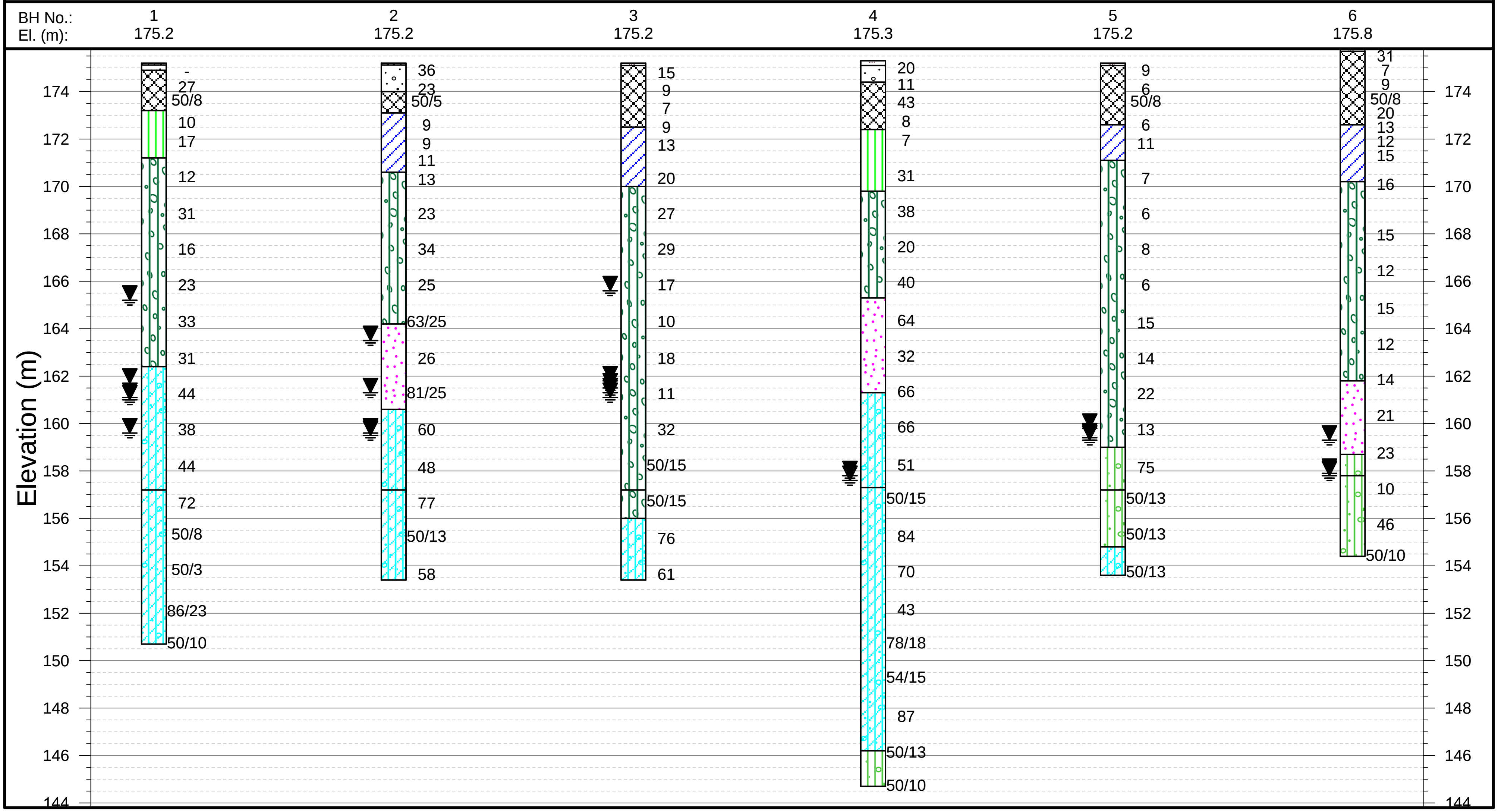
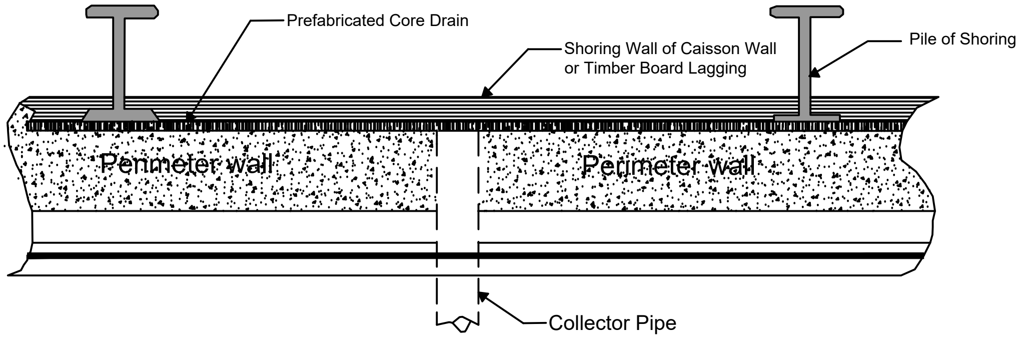
SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

JOB NO.: 1908-S037
REPORT DATE: November 2019
PROJECT DESCRIPTION: Proposed Residential Development
PROJECT LOCATION: 1 Heron's Hill Way, City of Toronto

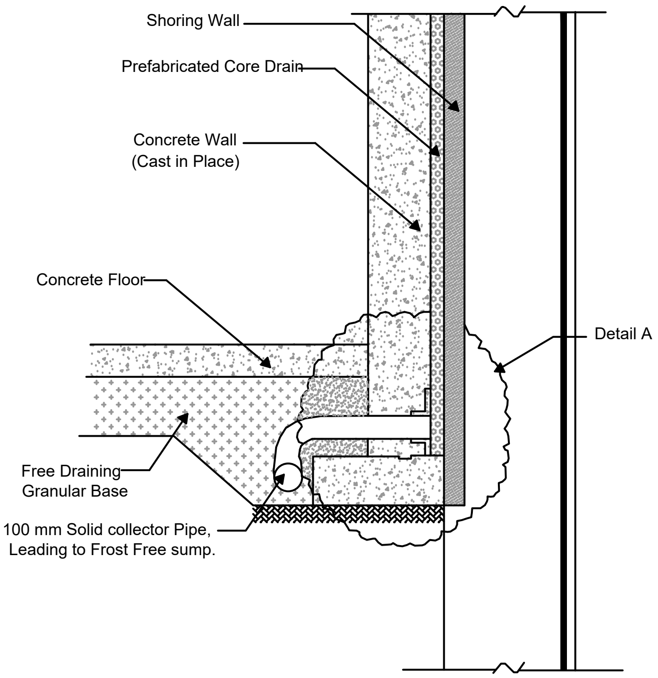
LEGEND

- TOPSOIL
- INTERLOCK STONE
- GRANULAR
- FILL
- SAND
- SILTY SAND TILL
- SANDY SILT TILL
- SILT
- SILTY CLAY
- SILTY CLAY TILL
- ▼ WATER LEVEL (STABILIZED)

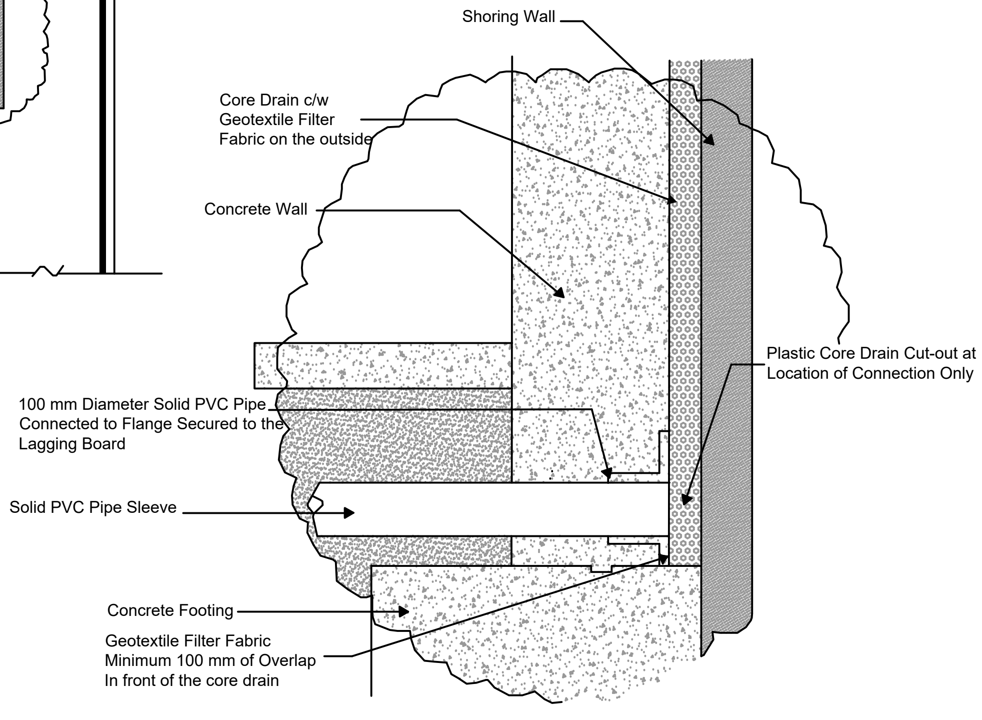




PLAN



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

Permanent Perimeter Drainage System

SITE: 1 Heron's Hill Way, City of Toronto

DESIGNED BY: K.L. | **CHECKED BY:** B.S. | **DWG NO.:** 3

SCALE: N.T.S. | **REF. NO.:** 1908-S037 | **DATE:** September 2019 | **REV** -

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BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	GRAVENHURST	HAMILTON
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APPENDIX A

SHORING DESIGN

REFERENCE NO. 1908-S037

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:

$$R = 1.5\ D\ K_p\ L^2\ \gamma$$

where	R	= Ultimate load to be restrained	kN
	D	= Diameter of concrete filled hole	m
	K_p	= Passive resistance of soils below the level of excavation	
	L	= Embedment depth of the pile	m
	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).

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 60 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 45º, founded in the native soil deposit below the bottom of excavation should be designed for the allowable bearing pressure of 90 kPa.

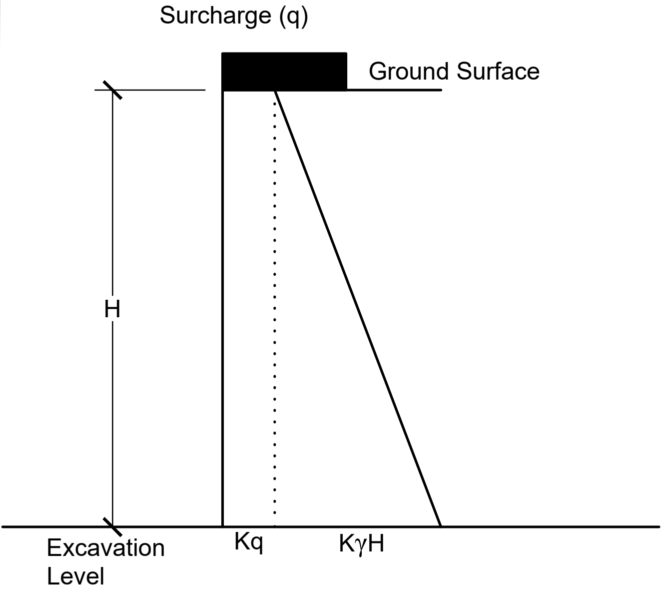
The raker footings should be located outside the zone of influence of the buried portion of the soldier piles at a distance of not less than 1.5 of the length of embedment of the soldier pile.

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.

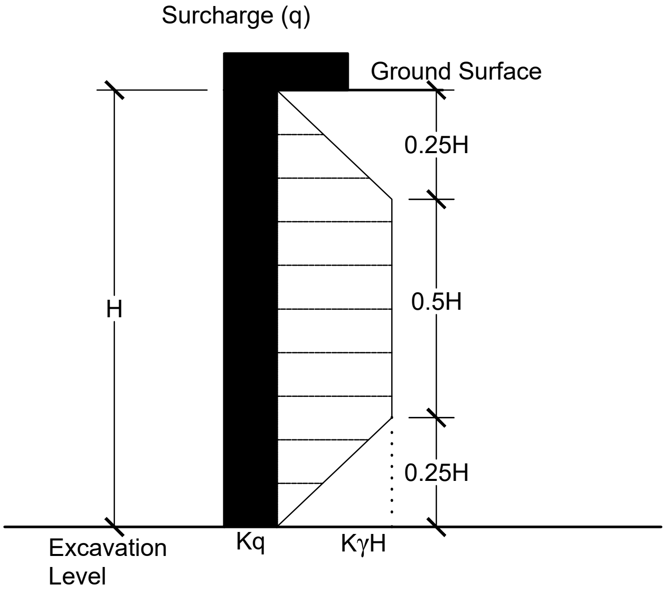
MONITORING OF PERFORMANCE

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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Temporary Shoring

SITE: 1 Heron's Hill Way, City of Toronto

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

SCALE: N.T.S. **REF. NO.:** 1908-S037 **DATE:** September 2019 **REV** -