Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION UPDATE
PROPOSED MIXED USE
COMMERCIAL AND RESIDENTIAL
31 HURON STREET
COLLINGWOOD, ONTARIO

Prepared for:	Streetcar 1230 Dundas Street East Toronto, Ontario M4M 1S3
Attention:	Mr. Jeff Magwood

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1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) was retained by Streetcar to conduct an updated geotechnical investigation for a proposed mixed use residential/commercial development located in the northwest corner of the intersection of Huron Street and Heritage Drive, in the Town of Collingwood, Ontario.

The previous investigation (File No: 1-19-0773-01, dated February 20, 2020) was based on a design concept that included a six (6) storey mixed-use residential and commercial building with one (1) level of underground parking, and involved the advancement of six (6) exploratory boreholes extending to a depth of about 8.4 m to 9.3 m below grade. Terraprobe is now advancing three deeper boreholes around boreholes 1, 5 and 6, from the previous investigation, and extending to the bedrock surface, due to the revised project design which now includes two (2) levels of underground parking, and to update the report. Three new boreholes will also be undertaken for a proposed park which is to be constructed to the immediate west of the proposed building. In the interim, this update to the geotechnical report is based on the findings of the previous field investigation and will require further updating after the pending supplemental field investigation is completed.

This report encompasses the results of the geotechnical investigation conducted for the proposed mixed use residential/commercial development to determine the prevailing subsurface soil and ground water conditions, and based on this information, provides geotechnical design recommendations for the foundations, basement floor slab, basement drainage, pavement, and earth pressure and seismic design parameters. Geotechnical comments are also included on pertinent construction aspects, excavation, backfill and ground water control.

Terraprobe is also conducting a Hydrogeological Study for this property. The findings of the investigation are reported under separate cover.

2.0 SITE AND PROJECT DESCRIPTIONS

The site is located in the northwest corner of the intersection of Huron Street and Heritage Drive, in the Town of Collingwood, Ontario, with a municipal address of 31 Huron Street. The general location of the site is presented on Figure 1. The site is located very close to Collingwood Harbour and the former Collingwood Shipyards.

The site consists of a rectangular shaped parcel of land with an area of approximately 4690 square metre (1.16 acres). The site is currently vacant.

The proposed development would include a six (6) storey mixed-used residential and commercial building with two (2) level of underground parking and lowest (P2) finished floor elevation FFE of 173.6 m. It is understood that the design is evolving and the structure will cover most of the site except for the northeast quadrant where at-grade parking is currently proposed.

3.0 INVESTIGATION PROCEDURE

The field investigation was conducted on January 13, 14, 15 and 20, 2020, and consisted of drilling and sampling a total of six (6) boreholes, extending to about 8.4 m to 9.3 m depth below grade. The approximate locations of the boreholes are shown on the enclosed Borehole Location Plan (Figure 2).

The boreholes were drilled by a specialist drilling contractor using track-mounted drill rig power auger. The borings were advanced using continuous flight solid stem augers, and were sampled at 0.75 m intervals (up to 3.0 m depth) and 1.5 m intervals (below 3.0 m depth) with a conventional 50 mm diameter split barrel samplers when the Standard Penetration Test (SPT) was carried out (ASTM D1586). The field work (drilling, sampling and testing) was observed and recorded by a member of our field engineering staff, who logged the borings and examined the samples as they were obtained.

All samples obtained during the investigation were sealed into clean plastic jars, and transported to our geotechnical testing laboratory for detailed inspection and testing. All borehole samples were examined (tactile) in detail by a geotechnical engineer, and classified according to visual and index properties. Laboratory tests consisted of water content determination on all samples; and Sieve and Hydrometer analysis on three (3) selected native soil samples (Borehole 1, Sample 5; Borehole 2, Sample 6 and Borehole 5, Sample 6). The measured natural water contents of individual samples and the results of the Sieve and Hydrometer analysis are plotted on the enclosed Borehole Logs at respective sampling depths. The results of Sieve and Hydrometer analysis are also summarized in Section 4.5 of this report, and appended.

Water levels were measured in open boreholes upon completion of drilling. Monitoring wells comprising 50 mm diameter PVC pipes were installed in selected boreholes (Boreholes 1, 2, 4 and 5) to facilitate ground water monitoring and the purpose of hydrogeological study. The PVC tubing was fitted with a bentonite clay seal as shown on the accompanying Borehole Logs. Water levels in the monitoring wells were measured on January 20, 2020. The results of ground water monitoring are presented in Section 4.6 of this report.

The borehole ground surface elevations were surveyed by Terraprobe using a Trimble R10 GNSS System. The Trimble R10 system uses the Global Navigation Satellite System and the Can-Net reference system to determine target location and elevation. The Trimble R10 system is reported to have an accuracy of up to 10 mm horizontally and up to 30 mm vertically.

It should be noted that the elevations provided on the Borehole Logs are approximate only, for the purpose of relating soil stratigraphy and should not be used or relied on for other purposes.

4.0 SUBSURFACE CONDITIONS

The specific soil conditions encountered at each borehole location are described in greater detail on the Borehole Logs, with a summary of the general subsurface soil conditions outlined below. This summary is intended to correlate this data to assist in the interpretation of the subsurface conditions encountered at the site.

It should be noted that the subsurface conditions are confirmed at the borehole locations only, and may vary between and beyond the borehole locations. The boundaries between the various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of geologic change.

4.1 Topsoil

A surficial layer of topsoil was encountered in all boreholes, varying in thickness from about 25 mm (Boreholes 1, 2 and 3) to 150 mm (Borehole 4). The topsoil was noted to be dark brown to black in colour and predominantly consisted of a sandy silt/sand and gravel matrix with organics.

The topsoil thickness noted on the Borehole Logs refers to the distinct topsoil layer present at the borehole location, however, organic inclusions extended deeper (typically about 300 m below grade or locally deeper) than the topsoil thickness layer noted on the Borehole Logs. The topsoil thickness to be removed/stripped for site development may differ from the topsoil thickness noted on the Borehole Logs. Therefore, this information is not sufficient for estimating topsoil quantities and/or associated costs. Consideration should be given to conduct a shallow test pit investigation to obtain a more precise topsoil thickness (if required).

4.2 Earth Fill

A zone of earth fill material was encountered in all boreholes beneath the surficial topsoil layer and extended to a depth of about 2.3 m below existing grade. The earth fill materials predominately consisted of sand and gravel with trace amounts of silt and clay or sandy silt with trace to some clay and trace amounts of gravel. Sporadic and a trace amount of organics/topsoil, as well as presence of rootlets were noted within the fill materials at varying depths.

The Standard Penetration Test results (‘N’ Values) obtained from the earth fill materials varied from 2 to 21 blows per 300 mm of penetration, indicating a typically very loose to compact relative density. It should be noted that some of the relative high ‘N’ Values obtained from the earth fill materials are likely due to frozen ground and may not necessarily represent the actual state of compactness of the material tested.

Measured moisture contents of the earth fill materials samples generally varied from about 6 to 55 percent by weight, indicating a moist to wet condition. It should be noted that some of the relative high water contents obtained from the earth fill materials are likely due to presence of topsoil within the earth fill materials.

4.3 Sandy Silt Till

Sandy silt till deposit, with trace amounts of clay, gravel and stone fragments was encountered beneath the fill layer in all boreholes and extended to the depths ranging from about 3.0 m to 6.1 m below grade.

N-values obtained from the undisturbed sandy silt till deposit ranged from about 20 to greater than 50 blows per 300 mm of penetration, indicating a compact to very dense (typically dense to very dense) relative density. The in-situ moisture contents of the glacial till samples ranged from 3 to 12%, indicating a moist to locally wet condition.

4.4 Sandy Silt

Sandy silt with trace amounts of clay and gravel was encountered beneath the sandy silt glacial till deposit in all boreholes and extended to the full depth of the investigation, except Boreholes 2, 3 and 4 where refusal was encountered on probable/inferred bedrock at depths of about 8.4 to 9.2 m below the existing grade.

N-values obtained from the sandy silt deposit ranged from about 27 to greater than 50 blows per 300 mm of penetration, indicating a compact to very dense (typically dense to very dense) relative density. The in-situ moisture contents of the sandy silt samples ranged from 7 and 22%, indicating a moist to wet condition.

The bedrock surface was inferred by auger/sample refusal in Borehole 2, 3 and 4 at depths ranging from about 8.4 to 9.2 m below grade, as noted above. The bedrock in the area generally consists of limestone of the Simcoe Group. The rock is horizontally bedded, and contains minor shale interbeds.

4.5 Geotechnical Laboratory Test Results

The geotechnical laboratory testing consisted of natural water content determination for all samples, while Sieve and Hydrometer analysis were conducted on selected native soil samples. The test results are plotted on the enclosed Borehole Logs at respective sampling depths.

The results (graphs) of the Sieve and Hydrometer (grain size) analysis are appended and a summary of these results is presented as follows:

Borehole No. Sample No.	Sampling Depth below Grade (m)	Percentage (by mass)				Descriptions (MIT System)
		Gravel	Sand	Silt	Clay	
Borehole 1, Sample 5	3.3	6	35	53	6	SILT AND SAND, trace clay, trace gravel
Borehole 2, Sample 6	4.8	16	16	60	8	SILT, some sand, some gravel, trace clay
Borehole 5, Sample 6	4.7	15	31	42	12	SANDY SILT, some gravel, some clay

4.6 Ground Water

Observations pertaining to the depth of water level and caving were made in the open boreholes immediately after completion of drilling, and are noted on the enclosed Borehole Logs. Monitoring wells were installed in Boreholes 1, 2, 4 and 5 to facilitate shallow ground water level monitoring and for the purpose of hydrogeological study. The ground water level measurements in the monitoring wells were taken on January 20, 2020 and are noted on the enclosed Borehole Logs. A summary of these observations is provided as follows:

Borehole No.	Depth of Borehole (m)	Upon Completion of Drilling		Water Level in Monitoring Well on January 20, 2020
		Depth to Cave (m)	Unstabilized Water Level (m)	Depth/Elev. (m)
BH 1	9.2	1.8	1.5	2.0/176.8
BH 2	9.3	2.1	1.5	2.3/176.9
BH 3	8.4	open	dry	Monitoring well not installed
BH 4	9.2	open	dry	2.4/176.1
BH 5	9.2	open	dry	2.7/175.7
BH 6	9.3	open	dry	Monitoring well not installed

The average measured ground water level was 176.32m Geodetic datum. The water levels noted above may fluctuate seasonally depending upon the amount of precipitation and surface runoff. More importantly, the nearby Collingwood Harbour water levels will greatly influence the site ground water levels. It is understood that the highest historical water level in Collingwood Harbour is about 177.50m. For design purposes, the stabilized ground water table is at about Elev. 178.0 ±m.

5.0 DISCUSSIONS AND RECOMMENDATIONS

The following discussion and recommendations are based on the factual data obtained from this investigation and are intended for the use of the owner and the design engineer. Contractors bidding or providing services on this project should review the factual data and determine their own conclusions regarding construction methods and scheduling.

This report is provided on the basis of these terms of reference and on the assumption that the design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards and guidelines of practice. If there are any changes to the site development features or there is any additional information relevant to the interpretations made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Terraprobe should be retained to review the implications of these changes with respect to the contents of this report.

5.1 Foundation

Boreholes 1 to 6 are located within or in the vicinity of the 6-storey building footprint. The boreholes encountered earth fill zone beneath the surficial topsoil layer extended to a depth of about 2.3 m below existing grade, underlain by undisturbed native soil deposits, extending to the full depth of the investigation in each borehole. Inferred bedrock surface was noted at about Elev. 166.3 to 169.9 m in Boreholes 2, 3 and 4 due to auger/sample refusal.

It is understood that the proposed mixed-use building would be a 6-storey structure with two (2) levels of underground parking garage (P2). Based on the updated design drawings provided by Streetcar (Project: 31 Huron Street, D-120 - Proposed Section A West-East and D-121 – Proposed Section B South-North, dated 31/07/2020), the lowest finished basement floor would be set at Elev. 173.60 m (average depth of about 5.0 m below existing grade). For design purposes, the stabilized ground water table at the site is at Elev. 178.0 ±m.

Below the P2 level (FFE of Elev. 173.6 m, conventional spread footings can be made to bear on the undisturbed (dewatered) very dense native soils (sandy silt glacial till and sandy silt). The following table summarizes the recommended geotechnical reaction and geotechnical resistance available at the borehole locations.

BH No.	Highest (Bottom) of Footing below Existing Ground Surface (m)	Highest (Bottom) of Footing Elevation (m)	Max. Geotechnical Reaction at SLS (kPa)	Max. Factored Geotechnical Resistance at ULS (kPa)	Bearing Stratum
1	5.1	173.6	500	750	Very Dense Sandy Silt (wet)
2	5.6	173.6	500	750	Very Dense Sandy Silt (wet)
3	4.6	173.6	500	750	Dense Sandy Silt (wet)
4	4.9	173.6	500	750	Very Dense Sandy (wet)
5	4.8	173.6	500	750	Very Dense Sandy Silt Till
6	5.1	173.6	500	750	Very Dense Sandy Silt (wet)

Notes: ULS=Ultimate Limit States; and SLS=Serviceability Limit States

The design bearing pressures as recommended allow for up to 25 mm of total settlement. This settlement will occur as load is applied and is linear elastic and non-recoverable. Differential settlement is a function of spacing, loading and foundation size.

Considering the proposed P2 finished floor level of Elev. 173.6 ±m, it is expected that foundations will be made up to 4 to 5 ±m below the prevailing ground water table at this site (Elev. 178 ±m). The ground water table must be lowered a minimum of 1.0 m below the lowest excavation elevation prior to any excavation and maintained at that level during construction. If the subgrade soils are not dewatered prior to excavation and maintained throughout construction, the subgrade soils will become disturbed and the recommendations provided above for bearing capacity will not be valid.

It must be noted that seasonal fluctuations in the ground water table may result in higher ground water levels than observed and reported.

5.1.1 Foundation Installation

All exterior foundations and foundations in unheated areas must be provided with a minimum soil cover of 1.4 m or equivalent insulation for frost protection.

It is recommended that all excavated footing base must be evaluated by a qualified geotechnical engineer to ensure that the founding soils exposed at the excavation base are consistent with the design bearing pressure intended by the geotechnical engineer.

Prior to pouring foundation concrete, the foundation subgrade should be cleaned of all deleterious materials such as topsoil, fill, softened, disturbed or caved materials, as well as any standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the foundation subgrade and concrete must be provided.

It is noted that the native soils tend to weather rapidly and deteriorate on exposure to the atmosphere or surface water. Hence, foundation bases which remain open for an extended period of time should be protected by a skim coat of lean concrete.

5.2 Basement Floor Slab

Based on the provided plans, it is understood that the finished floor elevation (FFE) of the proposed P2 level is to be set at Elev. 173.6 m. The P2 slab will be made on (dewatered) very dense sandy silt glacial till or sandy silt. The (dewatered) native soils constitute adequate subgrades for the support of a slab on grade. The modulus of subgrade reaction appropriate for the design of a fully drained slab-on-grade resting on the granular drainage layer overlying the very dense native soils is 40,000 kPa/m.

Prior to the construction of the slab, it is recommended that the glacial till subgrade be cut neat, or proof-rolled for the sandy silt subgrade, and inspected under the supervision of Terraprobe for obvious loose or disturbed areas as exposed, or for areas containing excessively deleterious materials or moisture. The native soil subgrade should be assessed and approved by Terraprobe prior to the placement of the slab-on-grade. Any disturbed areas shall be recompacted in place and retested, or else replaced with Granular B placed as engineered fill (in lifts 150 mm thick or less and compacted to a minimum of 98 % SPMDD). A static drum roller should be used to proof-roll the soils at this site, as vibration may cause unwanted disturbance, dilation and/or reduction in strength of the sandy silt.

It is necessary that building floor slabs be provided with a capillary moisture barrier and drainage layer. This is made by placing the slab on a minimum 200 mm layer of HL-8 Coarse Aggregate or 19 mm clear stone (OPSS.MUNI 1004) compacted by vibration to a dense state. The upper 50 mm of clear stone can be replaced with 50 mm of 19 mm crusher run limestone for a working surface. Provision of subfloor drainage is required in conjunction with perimeter drainage of the structure to collect and remove water that infiltrates under the floor, as discussed in Section 5.4.

Cohesionless soils will be encountered at the subgrade for the slab on grade. Therefore, a suitable non-woven geotextile filter (Terrafix 270R or equivalent approved by Terraprobe) must be installed (with a minimum 900 mm overlap) below the HL-8 Coarse Aggregate or 19 mm clear stone; otherwise, without proper filtering there may be entry of fines from the surrounding subgrade soils into the subfloor drainage layer. This loss of ground could result in a loss of support of the slab and clogging of the subfloor drainage system.

Regardless of the approach to slab construction, the floor slabs that are to have bonded floor finishes (such as tiles with adhesives) should be provided with a capillary moisture/vapour barrier. The floor manufacturers have specific requirements for moisture/vapour barrier; therefore, the floor designer/architect must ensure that a provision of appropriate moisture/vapour barrier conforming to specific floor finish product requirements is incorporated in the project specifications. Adequate testing must be carried out to ensure acceptable levels of moisture/relative humidity in the concrete slab prior to the installation of floor finish. Studies indicate that a provision of 200 mm thick 19 mm clear stone base (OPSS MUNI 1004) under the slab provides a good capillary moisture barrier provided the granular base is positively drained. However, this provision does not replace the floor manufacturers' specific requirement(s) for a moisture/vapour barrier.

5.3 Earth Pressure Design Parameters

Walls or bracings subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

$$P = K\,[\gamma\,(h - h_w) + \gamma' h_w + q] + \gamma_w h_w$$

Where:
- P = the horizontal pressure (kPa)
- K = the earth pressure coefficient
- h = the depth below the ground surface (m)
- h_w = the depth below the ground water level (m)
- γ = the bulk unit weight of soil (kN/m^3)
- γ_w = the bulk unit weight of water (9.8 kN/m^3)
- γ' = the submerged unit weight of the exterior soil, ($\gamma_{sat} - \gamma_w$)
- q = the complete surcharge loading (kPa)

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall, this equation can be simplified to:

$$P = K[\gamma h + q]$$

This equation assumes that free-draining granular backfill is used and positive drainage is provided to ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure. Resistance to sliding of retaining structures is developed by friction between the base of the footing and the soil. This friction **(R)** depends on the normal load on the soil contact (**N**) and the frictional resistance of the soil (**tan ϕ**) expressed as **$R = N \tan \phi$**. The factored geotechnical resistance at ULS is **0.8 R**.

Passive earth pressure resistance is generally not considered as a resisting force against sliding for conventional retaining structure design because a structure must deflect significantly to develop the full passive resistance.

The average values for use in the design of structures subject to unbalanced earth pressures at this site are tabulated as follow:

Parameter	Definition	Units
ϕ	angle of internal friction	degrees
γ	bulk unit weight of soil	kN/ m^3
K_a	active earth pressure coefficient (Rankine)	dimensionless
K_o	at-rest earth pressure coefficient (Rankine)	dimensionless
K_p	passive earth pressure coefficient (Rankine)	dimensionless

Stratum/Parameter	γ (kN/m^3)	Φ (degree)	K_a	K_o	K_p
Earth Fill	20.0	28	0.36	0.53	2.77
Undisturbed Sandy silt Till	21.0	32	0.31	0.47	3.25
Sandy Silt	21.0	32	0.31	0.47	3.25

The above values of the earth pressure coefficients are for the horizontal backfill grade behind the wall. The earth pressure coefficients for inclined grade will vary based on the inclination of the retained ground surface.

5.4 Basement Drainage

A separate hydrogeological report has been prepared by Terraprobe for this site (File. No. 1-19-0773-46), which provides the approximate amount of daily temporary (construction) and permanent ground water collection and discharge.

To assist in maintaining dry basements and preventing seepage, it is recommended that exterior grades around the building be sloped away at a 2 percent gradient or more, for a distance of at least 1.2 metres. Provision of nominal subfloor drainage is required in conjunction with the perimeter drainage of the structure, to collect and remove the water that infiltrates at the building perimeter and under the floor. Perimeter and subfloor drainage are required throughout below grade areas.

It is recommended that the subfloor drainage system consists of minimum 100 mm diameter perforated pipes wrapped in filter fabric spaced at a maximum spacing of 5 metres on centre. The pipes must be surrounded by a minimum of 100 mm of 19 mm clear stone/HL-8 Coarse Aggregate, and the pipe inverts should be a minimum 300 mm below the base of the slab. The elevator pits can be drained separately with an independent lower pumping sump or can be designed as water proof structures which are below the drainage level. A typical basement subdrain detail is provided in Appendix C. The subfloor drains can be constructed in trenches as shown in the typical detail, or alternatively they can be constructed on a flat subgrade sub excavated at least 300 mm below the base of the slab. The subdrain system should be outlet to a suitable discharge point under gravity flow, or connected to a sump located in the lowest level of the basement. The water from the sump must be pumped out to a suitable discharge point/positive outlet. The installation of the drains as well as the outlet must conform to the applicable plumbing code requirements.

In case the basement walls are constructed within an open excavation, perimeter foundation drains should be provided, consisting of perforated pipe with filter fabric (minimum 100 mm diameter) surrounded by a granular filter (minimum 150 mm thick), and freely outletting. The granular filter should consist of 19mm clear stone (OPSS.MUNI 1004) surrounded by a filter fabric (Terrafix 270R® or equivalent).

The basement wall must be provided with waterproofing provisions in conformance to the Section 9.13.2 of the Ontario Building Code. The basement wall backfill for a minimum lateral distance of 0.6 m out from the wall should consist of free-draining granular material (OPSS.MUNI 1010 Granular B), or provided with a prefabricated drain material (for instance, CCW MiraDRAIN 6000 series®, Terrafix Terradrain 600® or equivalent), see Appendix C for typical basement wall (open excavation) drainage details. The perimeter drain installation and outlet provisions must conform to the plumbing code requirements.

If the foundation walls are constructed against the shoring system (one-sided wall construction), drainage is provided by forming a drained cavity with prefabricated drain material, such as CCW MiraDRAIN 6000 series® (or Terrafix Terradrain 200®, or approved equivalent) which can be incorporated between the shoring and the cast-in-place concrete foundation wall. The water from the drainage composite material can be outlet through drainage ports (at about 3 m spacing at the base of the foundation wall and drained into the basement sumps using a solid pipe (separate from the subfloor drainage system) to remove collected water at the building sumps. Typical shored and open cut excavation drainage details are provided in Appendix C. Consideration should be given to waterproof the foundation walls in which case perimeter drainage is not required, however drainage board may still be used to provide added protection to the waterproofing membrane.

The drainage system is a critical structural element, since it keeps water pressure from acting on the basement floor slab or on the foundation walls. As such, the sump that ensures the performance of this system must have a duplexed pump arrangement for 100% pumping redundancy and these pumps must be on emergency power. The size of the pump should be adequate to accommodate the anticipated seepage and storm event flows. The subdrain system should be outlet to a suitable discharge point under gravity flow, or connected to a sump located in the lowest level of the basement. The water from the sump must be pumped out to a suitable discharge point/positive outlet. The installation of the drains as well as the outlet must conform to the applicable plumbing code requirements.

5.5 Earthquake Design Parameters

The current Ontario Building Code stipulates the methodology for earthquake design analysis, as set out in Subsection 4.1.8.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification.

The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A. of the Ontario Building Code. The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity (v_s) measurements have been taken. Alternatively, the classification is estimated on the basis of rational analysis of undrained shear strength (s_u) or penetration resistance (N-values).

$$\upsilon_{s-avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{\upsilon_{si}}} \qquad S_{u-avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{s_{ui}}} \qquad N_{avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

Shear Wave Velocity	Undrained Shear Strength	SPT N-values

Based on the borehole data (advanced to a maximum of 9.3 m depth below grade), it is understood that the proposed building will be founded on the sandy silt till/sandy silt deposit of compact to very dense relative density. It is expected that the deeper stratigraphy in this area is at least as competent as the lowest proven strata in the boreholes. On this basis, preliminary site seismic classification may be taken as Site Class C according to Table 4.1.8.4.A of the Ontario Building Code. Tables 4.1.8.4.B. and 4.1.8.4.C. of the current Ontario Building Code provide the applicable acceleration and velocity based site coefficients. The applicable acceleration and velocity based site coefficients for Site Class C are provided as follows:

Site Class	Values of F_a (acceleration based coefficients)				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) \geq 1.25$
C	1.0	1.0	1.0	1.0	1.0

Site Class	Values of F_v (velocity based coefficients)				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
C	1.0	1.0	1.0	1.0	1.0

It should be noted that the above site seismic designation is estimated on the basis of rational analysis of the limited energy-corrected Average Undrained Shear Strength information obtained from the boreholes advanced at the site only up to a maximum of 9.3 m depth below grade and with assumed undrained shear strength for the soil stratigraphy beneath the investigation depth. A site specific Multichannel Analysis of Surface Waves (MASW) may be conducted to confirm the site seismic classification.

5.6 Pavement

It is understood that main entrance driveways may be supported on soil subgrade, while other at-grade pavements will be constructed on concrete deck. Design recommendations for the entrance driveway pavement structure are provided in this section. For pavement structure supported on concrete deck, Terraprobe would provide pavement structure recommendations during the detailed design stage in consultation with structural engineer, architect and other design team.

5.6.1 Pavement Design

The asphalt pavement design for the entrance driveway supported on soil subgrade is provided in the following table:

Pavement Structural Layers	Entrance Driveway	Compaction Requirements
Hot Mix Asphalt Surface Course, OPSS 1150 HL 3	40 mm	as per OPSS 310
Hot Mix Asphalt Binder Course, OPSS 1150 HL 8	50 mm	
Base Course, OPSS.MUNI 1010, Granular A	150 mm	100 percent of Standard Proctor Maximum Dry Density (SPMDD) (ASTM D698)
Subbase Course, OPSS.MUNI 1010, Granular B	300 mm	

5.6.2 Drainage

Control of water is an important factor in achieving a good pavement life. Therefore, we recommend that provisions be made to drain the new pavement subgrade and its granular layers. Drainage can be achieved by installing catch basin(s) and a storm sewer system to collect surface runoff and, this system can also be used for subgrade drainage by installing subdrains that are designed to drain into the catch basins. The subgrade must be free of depressions and sloped at a grade of 3 percent to provide positive drainages.

Continuous pavement subdrains (designed to drain into catch basins) should be provided along both sides of driveway curb lines. All sub-drain arrangements should comply with Town of Collingwood Standard.

5.6.3 General Pavement Recommendations

HL 3 and HL 8 hot mix asphalt mixes should be designed, produced and placed in conformance with OPSS 1150 and OPSS 310 requirements and pertinent Town's standards.

Granular subbase material should meet the requirements of OPSS.MUNI 1010 and Town's standards. Granular materials should be compacted to 100 percent SPMDD at ±2 percent of the OMC.

PG 58-28, conforming to OPSS.MUNI 1101 is recommended in the HMA surface and binder courses. Tack coat SS-1 should be applied between hot mix asphalt binder course and surface course.

5.6.4 Subgrade Preparation

All topsoil, organics, soft/loose and otherwise disturbed soils should be stripped from the subgrade areas. The exposed subgrade is expected to consist of earth fill materials will be weakened by construction traffic when wet; especially if site work is carried out during the periods of wet weather. An adequate granular working surface would be likely required in order to minimize subgrade disturbance and protect its integrity in wet periods.

Immediately prior to placing the granular subbase, the exposed subgrade should be proof-rolled with a heavy rubber tired vehicle (such as a loaded gravel truck). The subgrade should be inspected for signs of rutting, distress and displacement. Areas displaying signs of rutting, distress and displacement should be recompacted and retested or, these materials should be locally excavated and replaced with well-compacted clean approved fill material.

The fill material may consist of either granular material or local inorganic soils provided that its moisture content is within ±2 percent of OMC. Fill material should be placed and compacted in accordance with OPSS.MUNI 501 and the subgrade should be compacted to 98 percent of SPMDD. The final subgrade surface should be sloped at least 3 percent to provide positive drainage.

5.7 Excavations

The boreholes data indicate that the earth fill materials (to average depth of 2.3m below ground surface) and undisturbed native soils would be encountered in the excavations. Excavations must be carried out in accordance with the *Occupational Health and Safety Act and Regulations for Construction Projects*. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety.

TYPE 1 SOIL

a. is hard, very dense and only able to be penetrated with difficulty by a small sharp object;
b. has a low natural moisture content and a high degree of internal strength;
c. has no signs of water seepage; and
d. can be excavated only by mechanical equipment.

TYPE 2 SOIL

a. is very stiff, dense and can be penetrated with moderate difficulty by a small sharp object;
b. has a low to medium natural moisture content and a medium degree of internal strength; and
c. has a damp appearance after it is excavated.

TYPE 3 SOIL

a. is stiff to firm and compact to loose in consistency or is previously-excavated soil;
b. exhibits signs of surface cracking;
c. exhibits signs of water seepage;
d. if it is dry, may run easily into a well-defined conical pile; and
e. has a low degree of internal strength

TYPE 4 SOIL

a. is soft to very soft and very loose in consistency, very sensitive and upon disturbance is significantly reduced in natural strength;
b. runs easily or flows, unless it is completely supported before excavating procedures;
c. has almost no internal strength;
d. is wet or muddy; and
e. exerts substantial fluid pressure on its supporting system.

The earth fill materials and native soils encountered in the boreholes are classified as Type 3 Soil above and Type 4 Soil below the prevailing ground water level, under these regulations.

Where workmen must enter excavations advanced deeper than 1.2 m, the trench walls should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The regulation stipulates the steepest slopes of excavation by soil type as follows:

Soil Type	Base of Slope	Steepest Slope Inclination
1	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
2	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
3	from bottom of trench	1 horizontal to 1 vertical
4	from bottom of trench	3 horizontal to 1 vertical

Minimum support system requirements for steeper excavations are stipulated in the Occupational Health and Safety Act and Regulations for Construction Projects, and include provisions for timbering, shoring and moveable trench boxes.

It should be noted that the glacial till deposit may contain larger particles (cobbles and boulders) that are not specifically identified in the Borehole Logs. The size and distribution of such obstructions cannot be predicted with borings, because the borehole sampler size is insufficient to secure representative samples of the particles of this size. Provision should be made in excavation contracts to allocate risks associated with time spent and equipment utilized to remove or penetrate such obstructions when encountered.

5.8 Ground Water Control

Ground water control and considerations pertaining to ground water and drainage are discussed in Terraprobe's Hydrogeological Study for the site under a separate cover (File No. 1-19-0773-46).

For design purposes, the stabilized ground water table at the site is Elev. 178 ±m. In general, the bulk excavation to the proposed P2 level (FFE of Elev. 173.6 ±m) will extend approximately 4 to 5 m below the stabilized ground water table (Elev. 178 ±m) at this site and excavations for foundations, elevator pits and sumps will extend even deeper below the stabilized ground water table.

Within the zone of excavation, the glacial till is considered a low to moderate permeability material, which will typically preclude significant free-flow of water. However, the earth fill and native cohesionless sandy silts are considered moderate to high permeability materials, which will permit the free-flow of water when wet. In addition, the glacial till deposit is expected to include relatively permeable silt/sand zones which may yield free-flowing water when penetrated.

Based on the provided site plans and building design, foundations will be made below the stabilized ground water table at this site. The ground water table must be lowered by positive dewatering to a minimum of 1.0 m below the lowest excavation elevation prior to any excavation and maintained at that level during construction. If the subgrade soils are not dewatered prior to excavation and maintained throughout construction, the subgrade soils will become disturbed and the recommendations provided in Section 5.1 for bearing capacity will not be valid.

It must be noted that seasonal fluctuations in the ground water table may result in higher ground water levels than observed and reported.

The subsurface information must be provided to a professional dewatering contractor who will be responsible for the design and installation of the dewatering systems. The dewatering system must be properly installed and screened to ensure that sediment and fine soils are not removed, which could result in settlement of the ground or structures near the site. Once the dewatering method and shoring system are designed, Terraprobe should be retained to evaluate the potential impacts (i.e. settlement) to nearby structures and land caused by lowering the water table.

The dewatering system must remain on until such time as the subfloor drainage system and sumps are fully operational.

5.8.1 Regulatory Requirements

The volume of water entering the excavation will be based on both ground water infiltration and precipitation events. Based on recent regulation changes within O.Reg. 63/16, the following dewatering limits and requirements are as follows:

- Construction Dewatering less than 50,000 L/day: The takings of both ground water and storm water **does not require** a Construction Dewatering Assessment Report (CDAR) and **does not require** a Permit to Take Water (PTTW) from the Ministry of the Environment and Climate Change (MOECC).
- Construction Dewatering greater than 50,000 L/day and less than 400,000 L/day: The taking of ground water and/or storm water **requires** a Construction Dewatering Assessment Report (CDAR) and **does not** require a Permit to Take Water (PTTW) from the Ministry of the Environment and Climate Change (MOECC).
- Construction Dewatering greater than 400,000 L/day: The taking of ground water and/or storm water **requires** a Construction Dewatering Assessment Report (CDAR) and **requires** a Permit to Take Water (PTTW) from the Ministry of the Environment and Climate Change (MOECC).

If it is expected that greater than 50,000 L/day of water will be pumped, a CDAR and/or a PTTW should be obtained as soon as possible in advance of construction to avoid possible delays. Depending on the

construction methodology for the site servicing (trench boxes or open cut, and length of trench) and the time of year (high versus low ground water levels), there is the possibility that water taking of greater than 50,000 L/day may occur at this site.

A CDAR takes up to 1 month to complete if monitoring wells are already installed on site. Once the CDAR is completed, it is uploaded to the Environmental Activity and Sector Registry (EASR), which registers the construction dewatering with the MOECC without the need for a permit. If the results of the CDAR indicate that greater than 400,000 L/day will be pumped, a PTTW application must be submitted to the MOECC. A PTTW application can take up to an additional 3 months for the MOECC to process upon completion of the CDAR. Note that Environmental Compliance Assessments, Impact Study Reports and applicable municipal, provincial and conservation authority approvals (completed by others) will be required as part of the CDAR.

5.9 Backfill

The topsoil and earth fill materials containing excessive amounts of organic inclusion should not be reused as backfill in settlement sensitive areas, such as beneath the floor slabs, trench backfill and pavement areas. However, these materials may be stockpiled and reused for landscaping purposes.

The existing earth fill materials are considered suitable (with selection and sorting as required) for backfill provided the moisture content of these soils is within ±2 percent of the OMC. Any soil material with ±2 percent or higher in-situ moisture content than its OMC, could be put aside to dry or be tilled to reduce the moisture content so that it can be effectively compacted. Alternatively, materials of higher moisture content could be wasted and be replaced with imported material which can be readily compacted.

The existing earth fill materials will likely require selection and sorting to be reused as backfill. The selection and sorting must be conducted under the supervision of a geotechnical engineer. The site soils will be best compacted with a heavy sheep foot type roller.

The backfill should consist of clean earth and be placed in lifts of 150mm thickness or less, and heavily compacted to a minimum of 95 percent SPMDD at a water content close to optimum (within 2 percent). The upper 600 mm of the pavement subgrade (at driveways outside of the basement roof deck) must be compacted to a minimum of 98 percent SPMDD.

It should be noted that the soils encountered on the site are generally not free draining, and will be difficult to handle and compact should they become wetter as a result of inclement weather or seepage. Hence, it can be expected that the earthworks will be difficult and may incur additional costs if carried out during the wet periods (i.e. spring and fall) of the year.

5.10 Shoring Design Consideration

The site is bounded by bounded by municipal roadways Side Launch Way to the north, Heritage Drive to the east, Huron Street to the south and a vacant lot followed by commercial development to the west. No excavation shall extend below the foundations of existing adjacent structures without adequate alternative support being provided.

Where excavations cannot be sloped, they can be supported using a shoring system such as soldier piles and lagging shoring or a continuous interlocking caisson wall shoring. Continuous interlocking caisson wall shoring is to be used where the excavation must be constructed as a rigid shoring system, to preserve the integrity and support of the soil beneath existing foundations of the adjacent buildings in a state approximating the at-rest condition or for groundwater cut-off, for the bulk excavation.

The shoring system would best be supported by pre-stressed soil anchors extending beneath the adjacent lands. Pre-stressed anchors are installed and stressed in advance of excavation and this limits movement of the shoring system as much as is practically possible. The use of anchors on adjacent properties requires the consent of the adjacent land owners, expressed in encroachment agreements. The City Transportation and Works Department negotiates "permits" for the encroachment in City lands, which are generally allowed.

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Temporary shoring should be carried out by a licensed Professional Engineer experienced in shoring design.

The detailed design of the proposed building was not available at the time of preparation of this report. The sections along the perimeter of the site will likely have to be shored to preserve the integrity of the boundary conditions (adjacent structures and roads). No excavation shall extend below a line cast as one vertical to one horizontal (1V:1H) from foundations of the existing structures without adequate alternate support being provided. Where the adjacent building foundations are removed from the excavation, a foundation which lies above a line drawn upward at 7 vertical to 10 horizontal (7V:10H) from the closest excavation edge is within the zone of potential influence of the excavation, and support for the existing foundations must be carefully assessed and possibly augmented.

The groundwater levels measured on January 20, 2020 in the monitoring wells installed in Boreholes 1, 2, 4 and 5 indicated that the groundwater levels ranged from about 2.0 to 2.7 m below existing grade (about Elev. 175.7 m to Elev. 176.9 m) in the boreholes. For design purposes, the stabilized groundwater table is at about EL. 178.0 ± m).

Consideration should be given to an impermeable shoring system (i.e. interlocking caisson wall), socketed into the bedrock, to support the excavation and around the entire perimeter for groundwater control purposes due to the presence of wet cohesionless materials encountered within the excavation depth. Convectional soldier piles and lagging shoring system may be used provided adequate positive dewatering is carried out to lower the groundwater table to at least 1.0 m below the lowest excavation level.

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Temporary shoring should be carried out by a licensed Professional Engineer experienced in shoring design.

The sections along the perimeter of the site will likely have to be shored to preserve the integrity of the boundary conditions (adjacent structures and roads). No excavation shall extend below a line cast as one vertical to one horizontal from foundations of the existing structures without adequate alternate support being provided. Where the adjacent building foundations are removed from the excavation, a foundation which lies above a line drawn upward at 10 horizontal to 7 vertical from the closest excavation edge is within the zone of potential influence of the excavation, and support for the existing foundations must be carefully assessed and possibly augmented.

The shoring requirements for the site will have to be examined in detail with respect to the site boundary constraints, once the development details and the building footprint are finalized. Depending upon the boundary conditions and structures located in the vicinity, ground water condition and dewatering details, the shoring system may consist of a rigid (interlocking drilled caissons) or a steel soldier piles and timber lagging shoring system, or a combination of both.

5.10.1 Earth Pressure Distribution

If the shoring is supported with a single level of earth anchor or bracing, a triangular earth pressure distribution similar to that used for the basement wall design is appropriate, and defined by:

$$\boldsymbol{P = K[\gamma H + q]}$$

where, $\boldsymbol{P}$ = the horizontal pressure at depth, **H** (kPa)
$\boldsymbol{K}$ = the earth pressure coefficient
$\boldsymbol{H}$ = the total depth of the excavation (m)
$\boldsymbol{\gamma}$ = the bulk unit weight of soil, (kN/m3)
$\boldsymbol{q}$ = the complete surcharge loading (kPa)

Where multiple supports are used to support the excavation, research has shown that a distributed pressure diagram more realistically approximates the earth pressure on a shoring system of this type, when restrained by pre-tensioned anchors. The multi-level supported shoring supporting predominately cohesive soil can be designed based on an earth pressure distribution consisting of a trapezoidal pressure distribution with a maximum pressure defined by:

$$\boldsymbol{P = 0.65\,K\,(\gamma H + q)}$$

where, $\boldsymbol{P}$ = the horizontal pressure at depth, **h** (kPa)
$\boldsymbol{K}$ = the earth pressure coefficient
$\boldsymbol{H}$ = the total depth of the excavation (m)
$\boldsymbol{\gamma}$ = the bulk unit weight of soil, (kN/m3)
$\boldsymbol{q}$ = the complete surcharge loading (kPa)

For ground water pressure distribution along the shoring wall in conjunction with the above soil pressures, the stabilized ground water table should be taken at Elev. 178.0 ±m. The ground water pressure distribution is only applicable where an impermeable boundary condition is created along the perimeter of the excavation, as is the case with a continuous interlocking caisson wall. Conventional soldier pile and lagging do not experience the water pressures, as water is allowed to drain freely through the wall.

5.10.2 Caisson and Soldier Pile Toe Design

Caisson and soldier pile toes are recommended to extend through the sandy silt and made to bear in the sound bedrock below Elev. 169 ±m. The factored ultimate vertical bearing capacity for the design of a pile, embedded in the sound bedrock, is 10 MPa. The factored ultimate lateral bearing capacity of the unweathered rock is at least 2 MPa. The horizontal resistance of the solider pile toes will be developed by embedment below the base of the excavation, where resistance is developed from passive earth pressure. It is noted that the resistance from these soils will be different depending on whether the soils are dewatered, or remain below the ground water level. Where soils exist beneath the ground water level, the unit weight of the soil is diminished by buoyancy. The design of the shoring will therefore have to consider the construction plan and sequence with respect to depth of ground water control.

The soils at this site are cohesionless, permeable and sufficiently wet such that augered borings made into these soils will be unstable. It is necessary to advance temporarily cased holes to prevent excess caving during the soldier pile and all augered hole installations. Drill holes for piles, caissons, and/or fillers, utilizing temporary liners, mud drilling techniques, and/or other methods as deemed necessary by the contractor may be required to prevent issues such as: groundwater inflow or loss of soil into the drill holes, and disturbance to placed concrete, basal instability and loss of bearing.

The ground water table must be lowered a minimum of 1.0 m below the lowest excavation elevation prior to any excavation and maintained at that level during construction. Once the dewatering method and shoring system are designed, Terraprobe should be retained to evaluate the potential impacts (i.e. settlement) to the shoring system (ex. pile toes) caused by lowering the water table.

5.10.3 Shoring Support

If anchor support is necessary and determined to be feasible, the shoring system should be supported by pre-stressed soil anchors extending beneath the adjacent lands. Pre-stressed anchors are installed and stressed in advance of excavation and this limits movement of the shoring system as much as is practically possible. It will be necessary to secure encroachment agreements from the Region/City and the adjacent land owners, in order to use soil anchors on the adjoining properties. Pre-construction condition surveys should be carried out for the adjacent structures to establish existing conditions prior to excavation and mitigate the possibility of spurious claims for excavation induced damages. Access to the properties for such surveys must be part of any encroachment agreements. A careful evaluation of the subsurface soil conditions is required by the shoring designer to establish appropriate levels/elevations and design the soil anchors. The anchor design will be governed by the weakest material in the profile. It is imperative that a detailed design is carried out at different anchor levels and locations, and the anchors must be tested at each level.

Conventional earth anchors could be made with continuous hollow stem augers or alternatively post-grouted anchors can be made. The design adhesion for earth anchors is controlled as much by the installation technique as the soil and therefore a proto-type anchor must be made in each anchor level executed to demonstrate the anchor capacity and validate the design assumptions. A proto-type anchor must be made to demonstrate the anchor capacity (performance tested to 200% of the design load). All production anchors must be proof-tested to 133% of the design load, to validate the design assumptions.

The subsurface soils are sufficiently cohesionless, permeable and/or wet that augered holes could experience caving. It will be necessary to advance temporarily cased holes to maintain sidewall support and to prevent the ingress of water during installation, use slurry, etc. or other means or methods deemed necessary by the contractor.

Conventional earth anchors made in the generally dense to very glacial till or sandy silt may be designed using a working adhesion of 50 kPa. It is expected that post-grouted anchors can be made such that an anchor will likely carry about 60 to 70 kN/m of adhered anchor length (at a nominal diameter of 150 mm) in the dense to very dense glacial tills and sandy silt depending upon the material type as confirmed by a performance/load test. It should be noted that these values are provided as preliminary guidance only and the actual anchor performance must be verified by a performance/load test.

Alternatively, rock anchors can be considered extending into the bedrock (Simcoe Group) and can be designed using a working adhesion of 620 kPa.

Regardless, the subsurface soil information should be reviewed by the shoring designer to decide on the suitable type of earth anchors and anchor capacity to be employed at this site.

If adjacent land owners are not agreeable to anchored support then internal bracing or rakers would be necessary. The dense to very dense glacial till and sandy silt below the proposed P2 level (FFE of Elev. 173.6 ±m) are suitable for the placement of raker foundations. Raker footings established on the undisturbed (dewatered) native soils at an inclination of 45 degrees can be designed for a maximum factored geotechnical resistance at ULS of 350 kPa.

It will be necessary to secure encroachment agreements from the Region/Town and the adjacent land owners, in order to use soil anchors on the adjoining properties. Pre-construction condition surveys should be carried out for the adjacent structures to establish existing conditions prior to excavation and mitigate the possibility of spurious claims for excavation induced damages. Access to the properties for such surveys must be part of any encroachment agreements.

A careful evaluation of the subsurface soil conditions is required by the shoring designer to establish appropriate levels/elevations and design the soil anchors. The anchor design will be governed by the weakest material in the profile. It is imperative that a detailed design is carried out at different anchor levels and locations, and the anchors must be tested at each level.

5.11 Quality Control

Excavations on this site must be shored to preserve the integrity of the surrounding properties and structures. The current Ontario Building Code stipulates that engineering review of the subsurface conditions is required on a continuous basis during the installation of earth retaining structures. Terraprobe should be retained to provide this review, which is an integral part of the geotechnical design function as it relates to the shoring design considerations. Terraprobe can provide detailed shoring design services for the project, if requested. All foundations must be monitored by the geotechnical engineer on a continuous basis as they are constructed. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.2 of the current Ontario Building Code. If Terraprobe is not retained to carry out foundation evaluations during construction, then Terraprobe accepts no responsibility for the performance or non-performance of the foundations, even if they are ostensibly constructed in accordance with the conceptual design advice provided in this report.

Concrete for this structure will be specified in accordance with the requirements of CAN3 - CSA A23.1. Terraprobe maintains a CSA certified concrete laboratory and can provide concrete sampling and testing services for the project as necessary.

The requirements for fill placement on this project should be stipulated relative to SPMDD, as determined by ASTM D698. In-situ determinations of density during fill placement by Procedure Method B of ASTM D2922 are recommended to demonstrate that the contractor is achieving the specified soil density. Terraprobe is a CNSC licensed operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary.

Terraprobe can provide thorough in-house resources, quality control services for Building Envelope, Roofing and Structural Steel in accordance with CSA W178, as necessary, for the Structural and Architectural quality control requirements of the project. Terraprobe is certified by the Canadian Welding Bureau under W178.1-1996.

6.0 LIMITATIONS AND RISK

6.1 Procedures

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained by Terraprobe.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to exist between sampling points can differ from those that actually exist.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities so that they may draw their own conclusions as to how the subsurface conditions may affect them.

6.2 Changes in Site and Scope

It must also be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. Groundwater levels are particularly susceptible to seasonal fluctuations.

The discussion and recommendations are based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructability issues and quality control may not be relevant or complete for the revised project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

This report was prepared for the express use of Streetcar and their retained design consultants and is not for use by others. This report is copyright of Terraprobe Inc. and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe Inc. and Streetcar who are the authorized users.

It is recognized that the regulatory agencies in their capacities as the planning and building authorities under Provincial statues, will make use of and rely upon this report, cognizant of the limitations thereof, both expressed and implied.

We trust the foregoing information is sufficient for your present requirements. If you have any questions, or if we can be of further assistance, please do not hesitate to contact us.

Yours truly,

Terraprobe Inc.

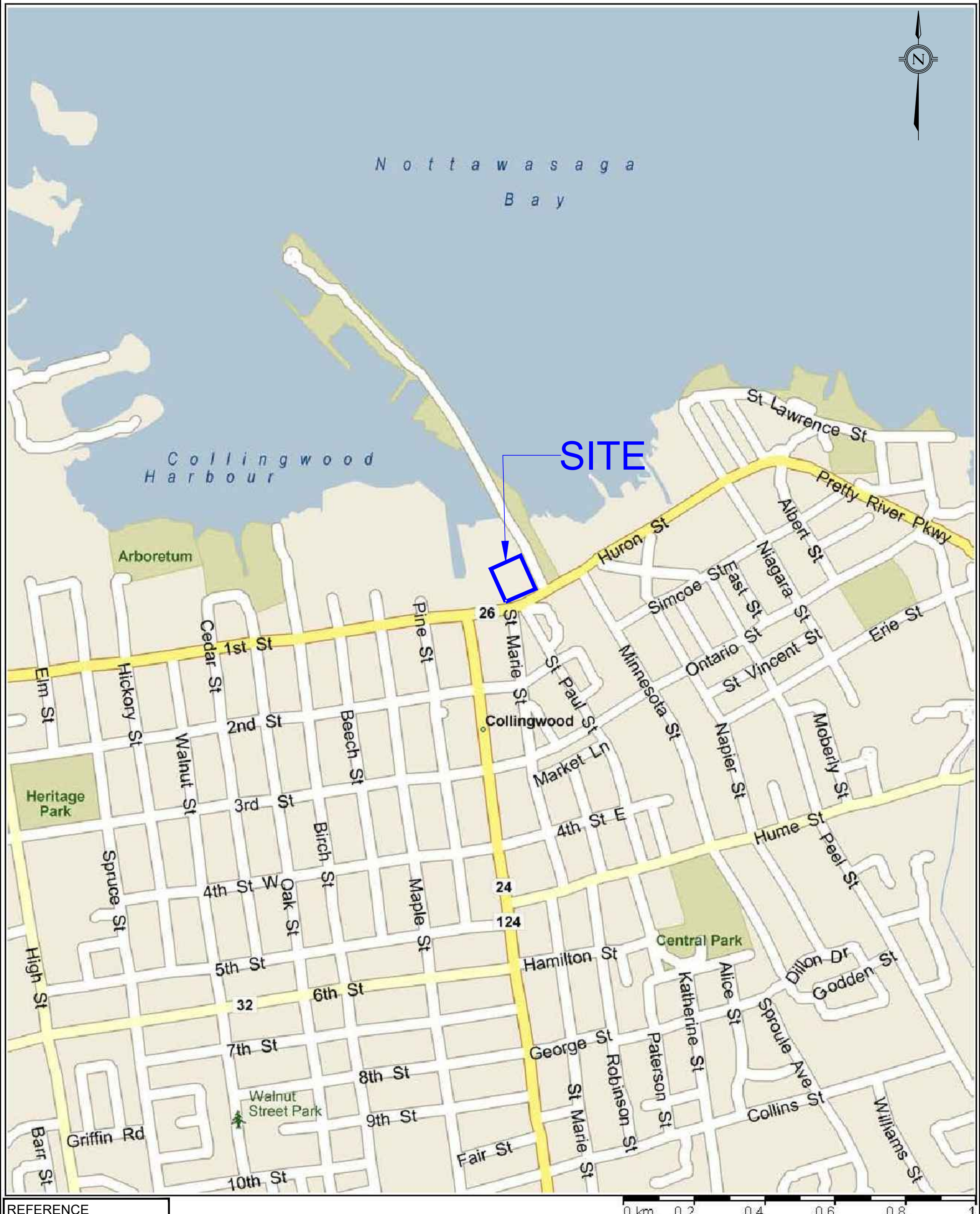
LICENSED PROFESSIONAL ENGINEER
O.B.
O. D. BENJAMIN
90337073
July 31, 2020
PROVINCE OF ONTARIO

Osbert (Ozzie) Benjamin, P.Eng.
Senior Project Manager, Geotechnical

Michael Tanos, P. Eng.
Principal

FIGURES

TERRAPROBE INC.



REFERENCE
Microsoft Streets and Trips

Terraprobe

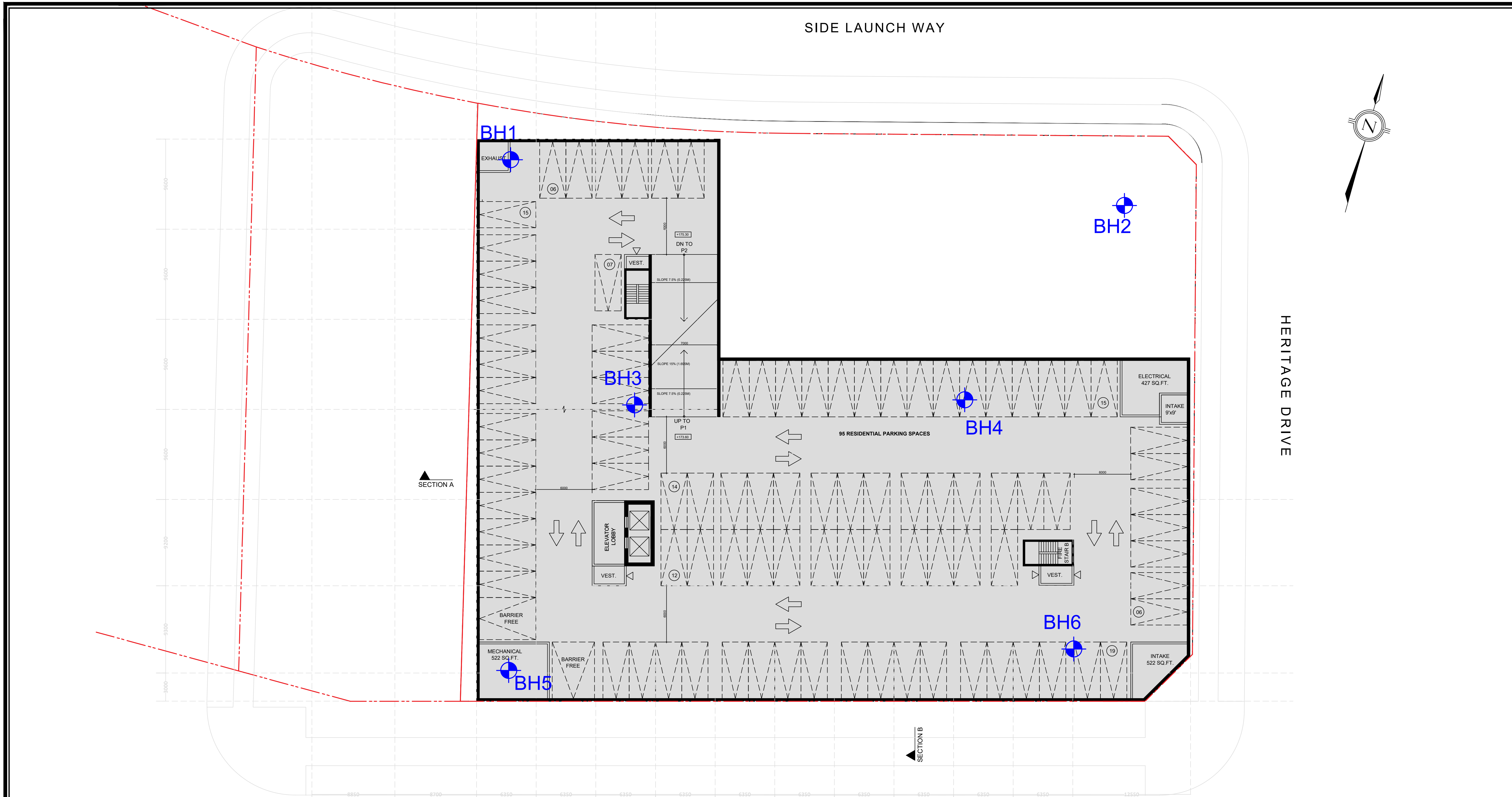
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title: **SITE LOCATION PLAN**

File. No.: 1-19-0773-01

FIGURE : 1

Z:\1-Project Files\2019\1-19-0773 - 31 Huron Street, Collingwood\01-GEO Investigation\A. Dwgs, Logs\AutoCAD\1-19-0773-01 FIGURE 1.dwg, Kamal



PARKING LEVEL P2

TOTAL GFA	36310 SQ.FT.
RESIDENTIAL SUITE GFA	0 SQ.FT.
RETAIL GFA	0 SQ.FT.
PARKING	95 SPACES

LEGEND

Approximate Borehole Location

REFERENCE

Project: 31 Huron Street, Collingwood
Drawing : Proposed Floor Plan P2
Issue Date: 30/730
By: Streetcar (Rethink Urban Living)

HURON STREET

4 2 0 10m

SCALE 1:400

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title: **BOREHOLE LOCATION PLAN**

File No. 1-19-0773-01

FIGURE : 2

APPENDICES

TERRAPROBE INC.

Terraprobe

ABBREVIATIONS AND TERMINOLOGY

SAMPLING METHODS

AS	auger sample
CORE	cored sample
DP	direct push
FV	field vane
GS	grab sample
SS	split spoon
ST	shelby tube
WS	wash sample

PENETRATION RESISTANCE

Standard Penetration Test (SPT) resistance ('N' values) is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a standard 50 mm (2 in.) diameter split spoon sampler for a distance of 0.3 m (12 in.).

Dynamic Cone Test (DCT) resistance is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a conical steel point of 50 mm (2 in.) diameter and with 60° sides on 'A' size drill rods for a distance of 0.3 m (12 in.)."

COHESIONLESS SOILS

Compactness	'N' value
very loose	< 4
loose	4 – 10
compact	10 – 30
dense	30 – 50
very dense	> 50

COHESIVE SOILS

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

COMPOSITION

Term (e.g)	% by weight
trace silt	< 10
some silt	10 – 20
silty	20 – 35
sand *and* silt	> 35

TESTS AND SYMBOLS

MH	mechanical sieve and hydrometer analysis
w, w_c	water content
w_L, LL	liquid limit
w_P, PL	plastic limit
I_P, PI	plasticity index
k	coefficient of permeability
γ	soil unit weight, bulk
G_s	specific gravity
φ'	internal friction angle
c'	effective cohesion
c_u	undrained shear strength

	Unstabilized water level
	1st water level measurement
	2nd water level measurement
	Most recent water level measurement
3.0 +	Undrained shear strength from field vane (with sensitivity)
C_c	compression index
c_v	coefficient of consolidation
m_v	coefficient of compressibility
e	void ratio

FIELD MOISTURE DESCRIPTIONS

Damp	refers to a soil sample that does not exhibit any observable pore water from field/hand inspection.
Moist	refers to a soil sample that exhibits evidence of existing pore water (e.g. sample feels cool, cohesive soil is at plastic limit) but does not have visible pore water
Wet	refers to a soil sample that has visible pore water

APPENDIX A

TERRAPROBE INC.

Terraprobe

LOG OF BOREHOLE 1

Project No.	: 1-19-0773-01	Client	: 31 Huron Street Inc.	Originated by	: HH
Date started	: January 13, 2020	Project	: 31 Huron Street	Compiled by	: HA
Sheet No.	: 1 of 1	Location	: Collingwood, Ontario	Checked by	: MMT

Position : E: 562341, N: 4928129 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers

SOIL PROFILE

Elev Depth (m)	Description
178.7	**GROUND SURFACE**
	25mm **TOPSOIL**
	FILL, sand and gravel, trace silt, trace clay, trace organics, loose, brown, wet
176.4 / 2.3	**SANDY SILT**, trace clay, trace gravel, dense, grey, moist (GLACIAL TILL)
175.7 / 3.0	**SANDY SILT**, trace clay, trace gravel, dense to very dense, grey, wet
169.5 / 9.2	**END OF BOREHOLE**

SAMPLES

Number	Type	SPT 'N' Value
1	SS	8
2	SS	6
3	SS	6
4	SS	45
5	SS	50
6	SS	64
7	SS	49
8	SS	63
9	SS	50 / 75mm

Depth Scale (m): 0, 1, 2, 3, 4, 5, 6, 7, 8, 9
Elevation Scale (m): 178, 177, 176, 175, 174, 173, 172, 171, 170

Penetration Test Values (Blows / 0.3m)
X Dynamic Cone
10 20 30 40
Undrained Shear Strength (kPa)
O Unconfined + Field Vane
● Pocket Penetrometer ■ Lab Vane
40 80 120 160

Moisture / Plasticity
Plastic Limit | Natural Water Content | Liquid Limit
PL MC LL
10 20 30

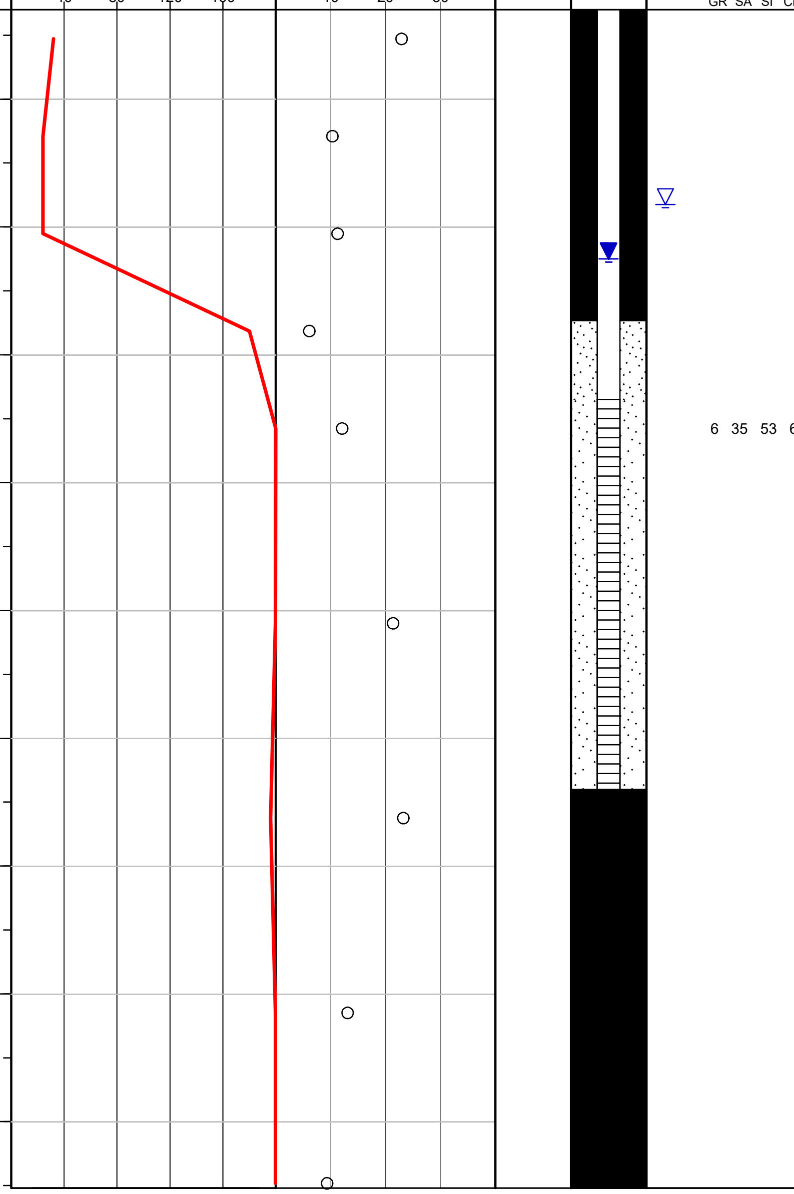
Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level

Lab Data and Comments
GRAIN SIZE DISTRIBUTION (%) (MIT)

GR	SA	SI	CL
6	35	53	6



WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Jan 20, 2020	2.0	176.8

Unstabilized water level measured at 1.5 m below ground surface; borehole caved to 1.8 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

LOG OF BOREHOLE 2

Terraprobe

Project No.	: 1-19-0773-01	Client	: 31 Huron Street Inc.	Originated by	: HH
Date started	: January 13, 2020	Project	: 31 Huron Street	Compiled by	: HA
Sheet No.	: 1 of 1	Location	: Collingwood, Ontario	Checked by	: MMT

Position : E: 562395, N: 4928156 (UTM 17T) Elevation Datum : Geodetic

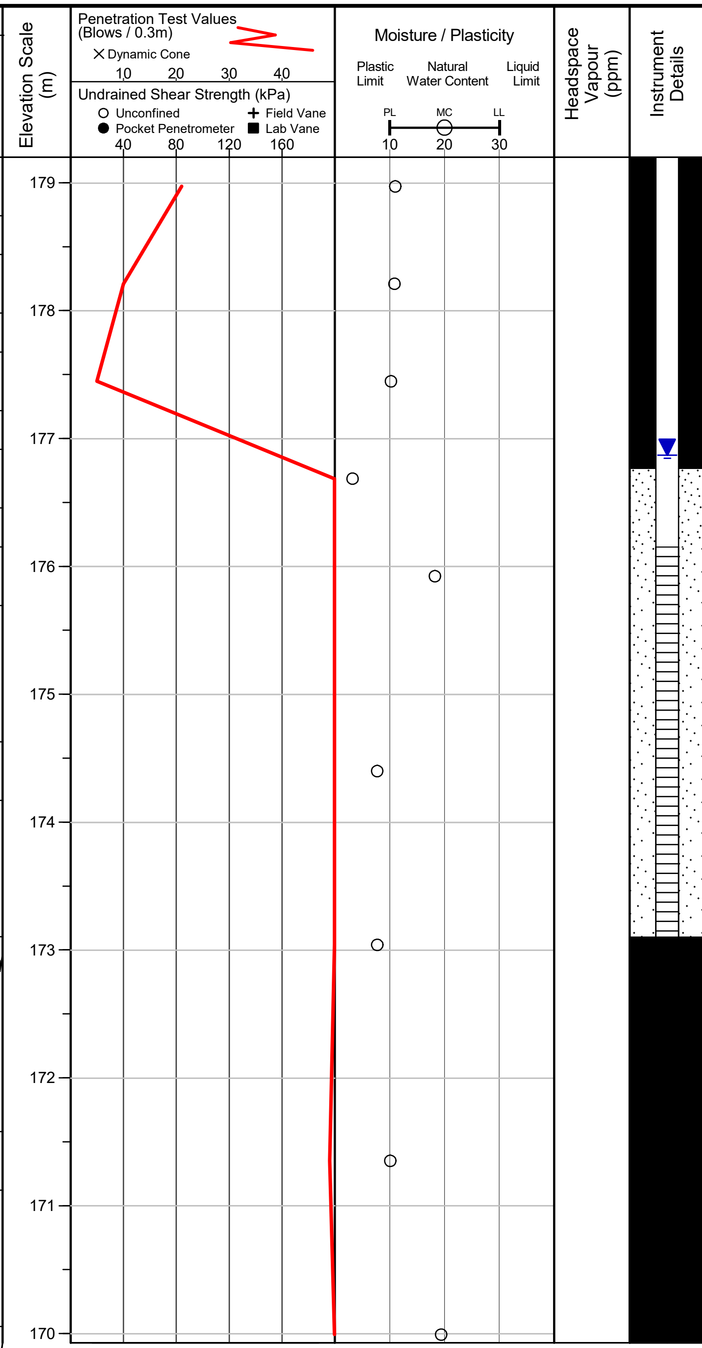
Rig type : Track-mounted Drilling Method : Solid stem augers

SOIL PROFILE

Elev Depth (m)	Description
179.2	GROUND SURFACE
	25mm **TOPSOIL**
	FILL, sand and gravel, trace silt, trace clay, trace organics, loose to compact, brown, moist to wet
176.9 / 2.3	**SANDY SILT**, trace clay, trace gravel, stone fragments, very dense, grey, moist (GLACIAL TILL)
176.2 / 3.0	**SANDY SILT**, trace clay, trace gravel, dense to very dense, grey, wet
	...silt, some sand, some gravel
	...stone fragments
	...some gravel, dense to very dense
169.9 / 9.3	...possible bedrock

SAMPLES

Number	Type	SPT 'N' Value	Lab Data and Comments: GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
1	SS	21	
2	SS	10	
3	SS	5	
4	SS	59	
5	SS	73	
6	SS	61	16 16 60 8
7	SS	50 / 125mm	
8	SS	49	
9	SS	50 / 125mm	



END OF BOREHOLE

Unstabilized water level measured at 1.5 m below ground surface; borehole caved to 2.1 m below ground surface upon completion of drilling.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Jan 20, 2020	2.3	176.9

file: 1-19-0773-46 bh logs.gpj

Terraprobe

LOG OF BOREHOLE 3

Project No. : 1-19-0773-01	Client : 31 Huron Street Inc.	Originated by : DH
Date started : January 20, 2020	Project : 31 Huron Street	Compiled by : HA
Sheet No. : 1 of 1	Location : Collingwood, Ontario	Checked by : MMT

Position : E: 562363, N: 4928099 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Solid stem augers

SOIL PROFILE

Depth Scale (m)	Elev / Depth (m)	Description
0	178.2	GROUND SURFACE
		25mm **TOPSOIL**
		FILL, sand and gravel, trace silt, trace clay, trace organics, loose, brown, wet
	175.9 / 2.3	**SANDY SILT**, trace clay, trace gravel, stone fragments, dense, grey, moist (GLACIAL TILL)
	173.6 / 4.6	**SANDY SILT**, trace clay, trace gravel, dense to very dense, grey, wet
		...possible bedrock
	169.8 / 8.4	**END OF BOREHOLE**

Borehole was dry and open upon completion of drilling.

SAMPLES

Number	Type	SPT 'N' Value
1	SS	4
2	SS	5
3	SS	4
4	SS	46
5	SS	46
6	SS	42
7	SS	65
8	SS	50 / 125mm
9	SS	50 / 50mm

Elevation Scale (m)

Penetration Test Values (Blows / 0.3m)
× Dynamic Cone
10 20 30 40

Undrained Shear Strength (kPa)
○ Unconfined + Field Vane
● Pocket Penetrometer ■ Lab Vane
40 80 120 160

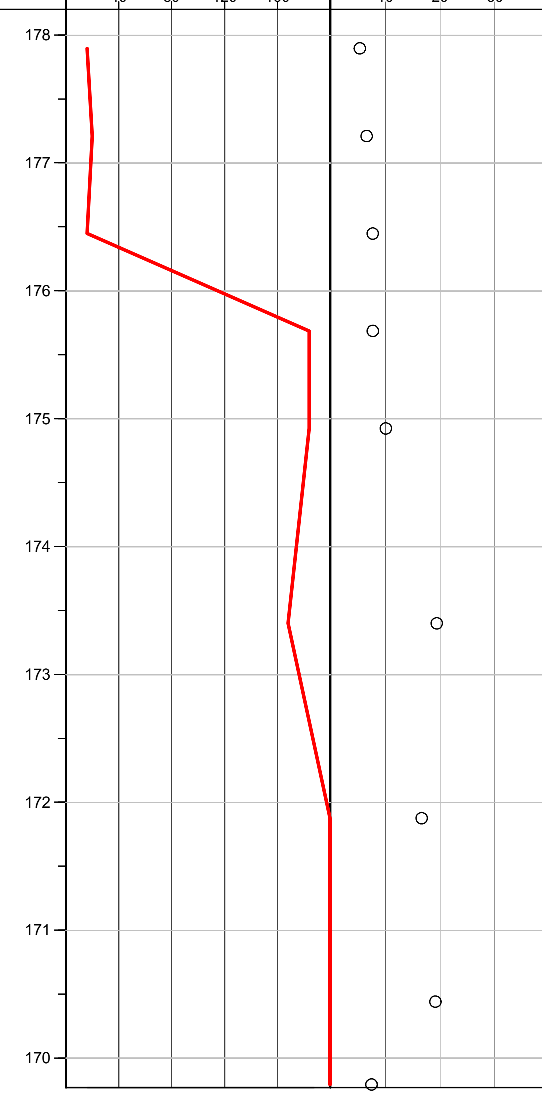
Moisture / Plasticity
Plastic Limit / Natural Water Content / Liquid Limit
PL MC LL
10 20 30

Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level

Lab Data and Comments
GRAIN SIZE DISTRIBUTION (%) (MIT)
GR SA SI CL



file: 1-19-0773-46 bh logs.gpj

LOG OF BOREHOLE 4

Terraprobe

Project No. : 1-19-0773-01	Client : 31 Huron Street Inc.	Originated by : DH
Date started : January 14, 2020	Project : 31 Huron Street	Compiled by : HA
Sheet No. : 1 of 1	Location : Collingwood, Ontario	Checked by : MMT

Position : E: 562394, N: 4928126 (UTM 17T) — Elevation Datum : Geodetic

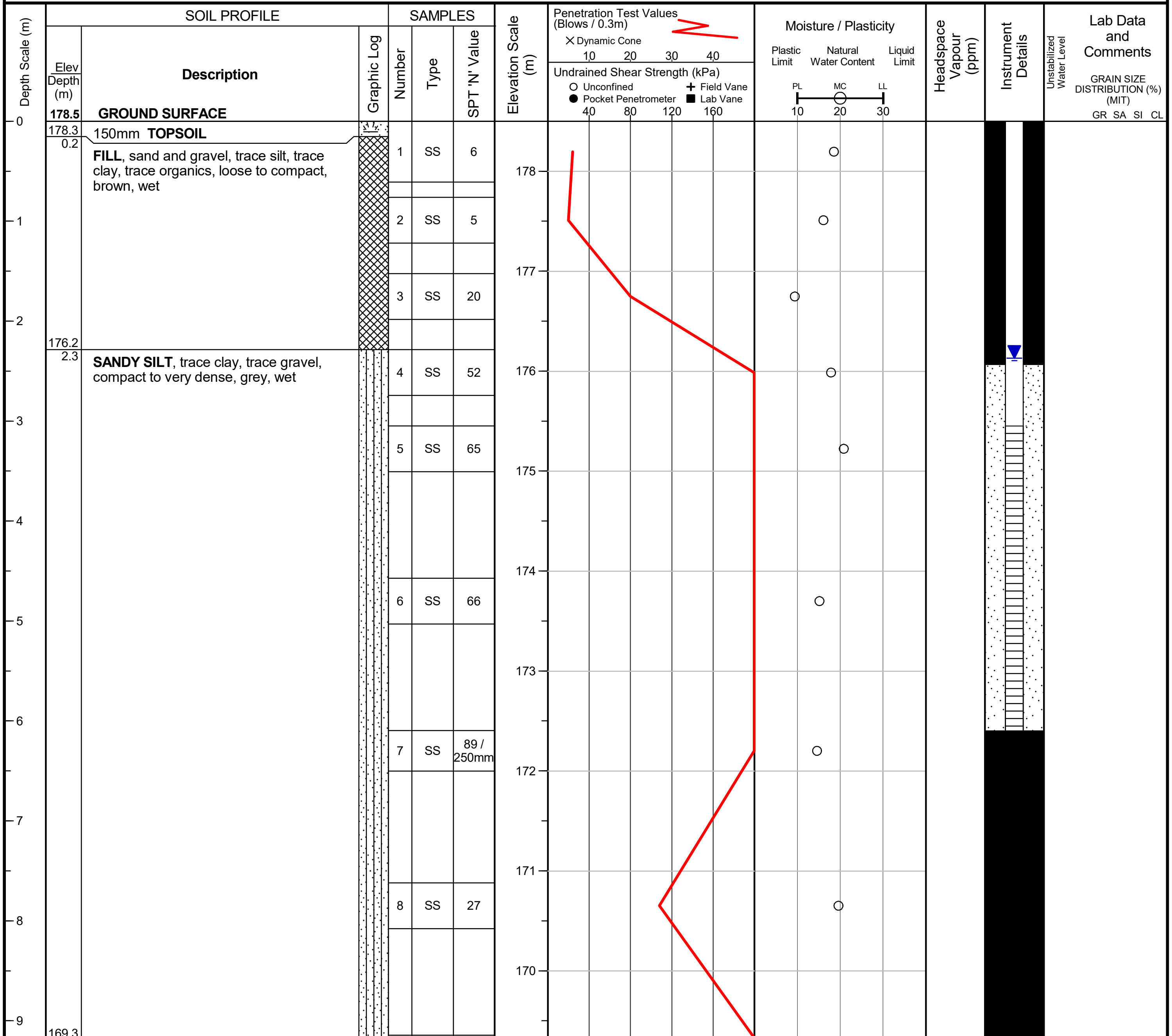
Rig type : Track-mounted — Drilling Method : Solid stem augers

SOIL PROFILE

Elev / Depth (m)	Description
178.5	**GROUND SURFACE**
178.3 / 0.2	150mm **TOPSOIL**
	FILL, sand and gravel, trace silt, trace clay, trace organics, loose to compact, brown, wet
176.2 / 2.3	**SANDY SILT**, trace clay, trace gravel, compact to very dense, grey, wet
169.3 / 9.2	**END OF BOREHOLE** Auger refusal on inferred bedrock

SAMPLES

Number	Type	SPT 'N' Value
1	SS	6
2	SS	5
3	SS	20
4	SS	52
5	SS	65
6	SS	66
7	SS	89 / 250mm
8	SS	27
9	SS	50 / 50mm



Borehole was dry and open upon completion of drilling.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Jan 20, 2020	2.4	176.1

file: 1-19-0773-46 bh logs.gpj

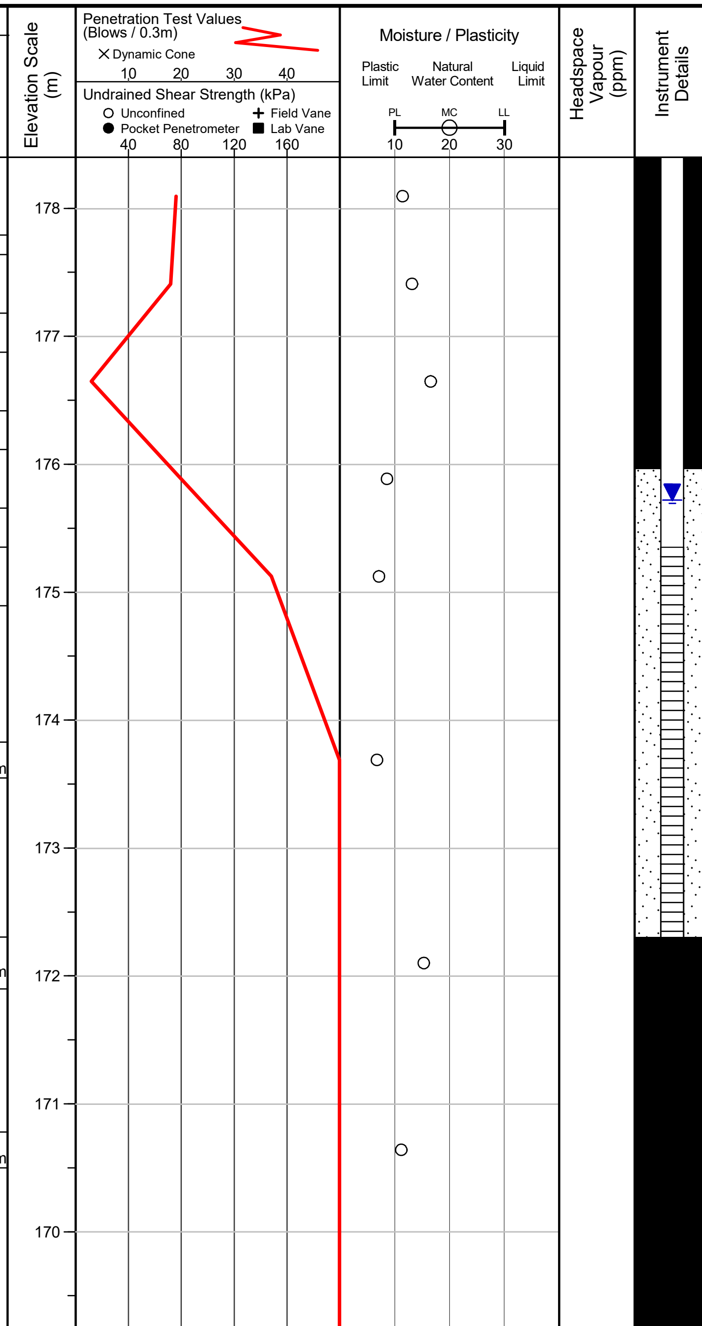
LOG OF BOREHOLE 5

Terraprobe

Project No. : 1-19-0773-01	Client : 31 Huron Street Inc.		Originated by : DH
Date started : January 15, 2020	Project : 31 Huron Street		Compiled by : HA
Sheet No. : 1 of 1	Location : Collingwood, Ontario		Checked by : MMT

Position : E: 562365, N: 4928084 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers

Elev / Depth (m)	Description	Sample Number	Type	SPT 'N' Value	Lab Data and Comments (GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL)
178.4	**GROUND SURFACE**				
	115mm **TOPSOIL**				
	FILL, sandy silt, trace to some clay, trace gravel, trace organics, very loose to compact, greyish brown, moist	1	SS	19	
		2	SS	18	
		3	SS	3	
176.1 / 2.3	**SANDY SILT**, trace clay, trace gravel, compact to very dense, grey, moist (GLACIAL TILL)	4	SS	20	
		5	SS	37	
	...some gravel, some clay	6	SS	50 / 125mm	15 31 42 12
172.3 / 6.1	**SANDY SILT**, trace clay, trace gravel, very dense, grey, wet	7	SS	85 / 250mm	
		8	SS	50 / 125mm	
169.2 / 9.2	...at 9.1 m, trace rock fragments	9	SS	50 / 50mm	



END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Jan 20, 2020	2.7	175.7

file: 1-19-0773-46 bh logs.gpj

LOG OF BOREHOLE 6

Terraprobe

Project No. : 1-19-0773-01	Client : 31 Huron Street Inc.	Originated by : DH
Date started : January 14, 2020	Project : 31 Huron Street	Compiled by : HA
Sheet No. : 1 of 1	Location : Collingwood, Ontario	Checked by : MMT

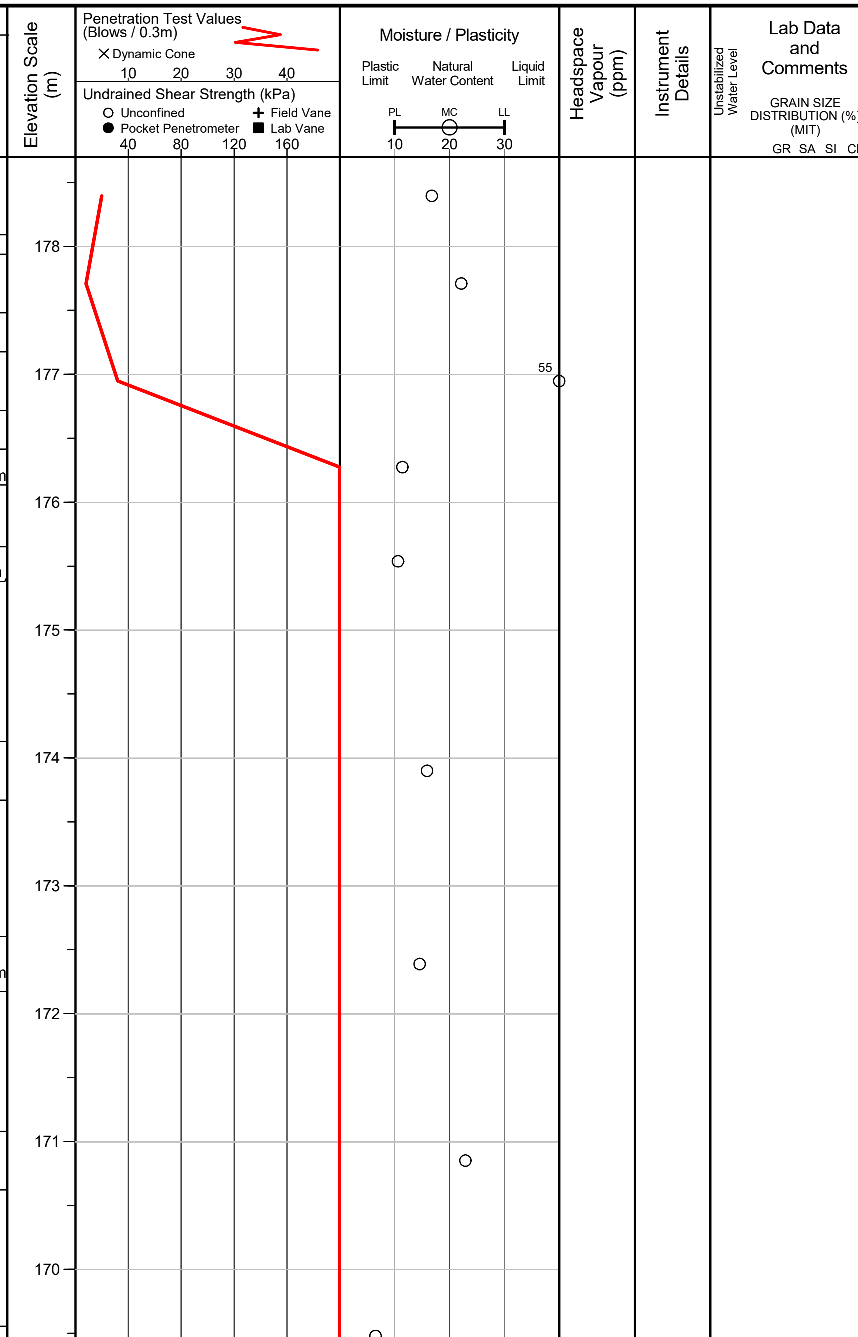
Position : E: 562415, N: 4928113 (UTM 17T) | Elevation Datum : Geodetic

Rig type : Track-mounted | Drilling Method : Solid stem augers

Elev / Depth (m)	Description	Graphic Log	Sample Number	Type	SPT 'N' Value	Moisture Content (%)
178.7	GROUND SURFACE					
	80mm **TOPSOIL**					
	FILL, sandy silt, trace to some clay, trace gravel, trace organics, very loose to loose, greyish brown, moist		1	SS	5	
			2	SS	2	
			3	SS	8	55
176.4 / 2.3	**SANDY SILT**, trace clay, trace gravel, very dense, grey, moist (GLACIAL TILL)		4	SS	50 / 125mm	
175.7 / 3.0	**SANDY SILT**, trace clay, trace gravel, very dense, grey, wet		5	SS	50 / 75mm	
			6	SS	63	
			7	SS	91 / 275mm	
			8	SS	74	
169.4 / 9.3	...trace rock fragments		9	SS	50 / 150mm	

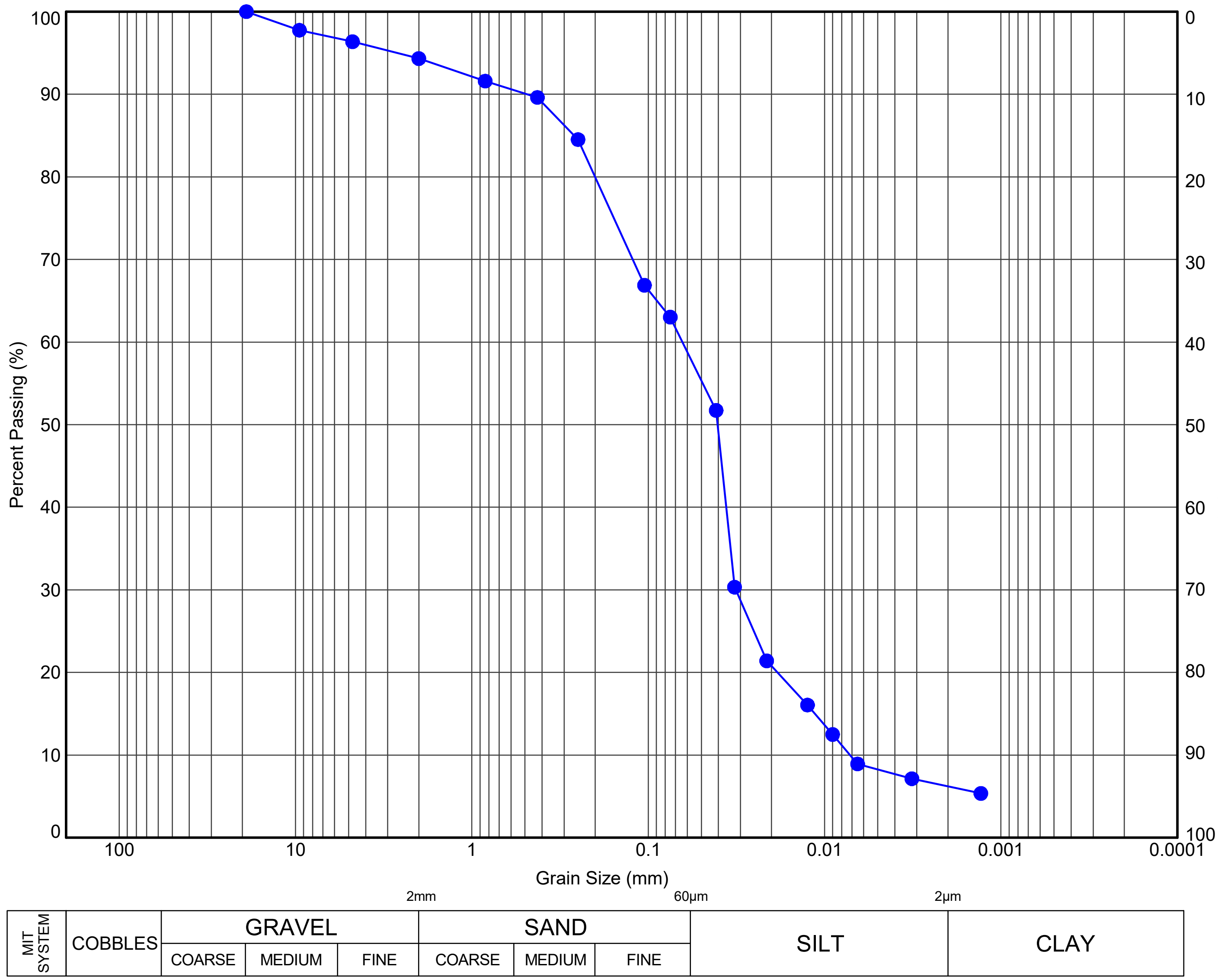
END OF BOREHOLE

Borehole was dry and open upon completion of drilling.



APPENDIX B

TERRAPROBE INC.



MIT SYSTEM

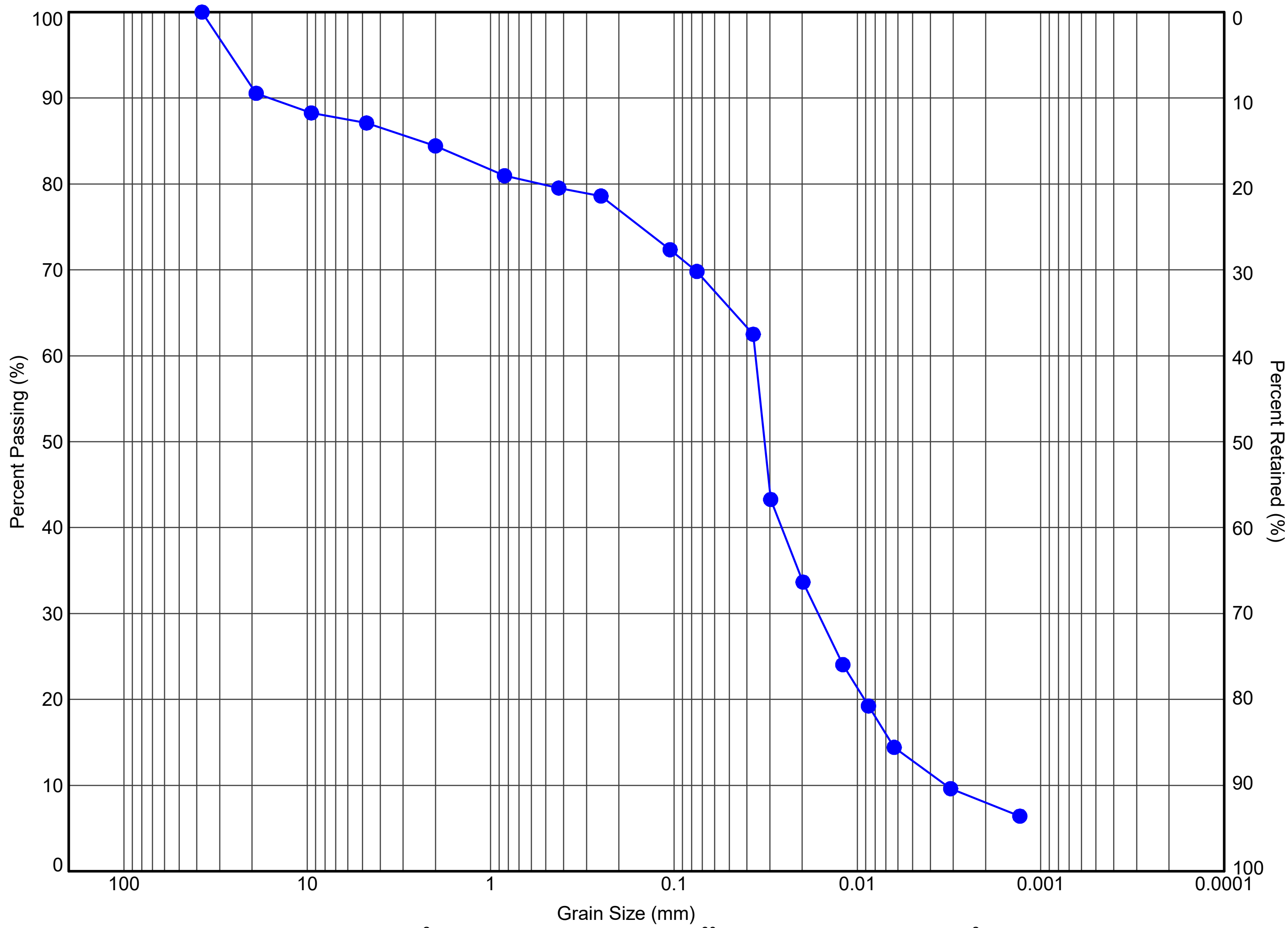
	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**1**	SS5	3.3	175.4	6	35	53	6	

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

GRAIN SIZE DISTRIBUTION
SILT AND SAND, TRACE CLAY, TRACE GRAVEL

File No.: **1-19-0773-01**



MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

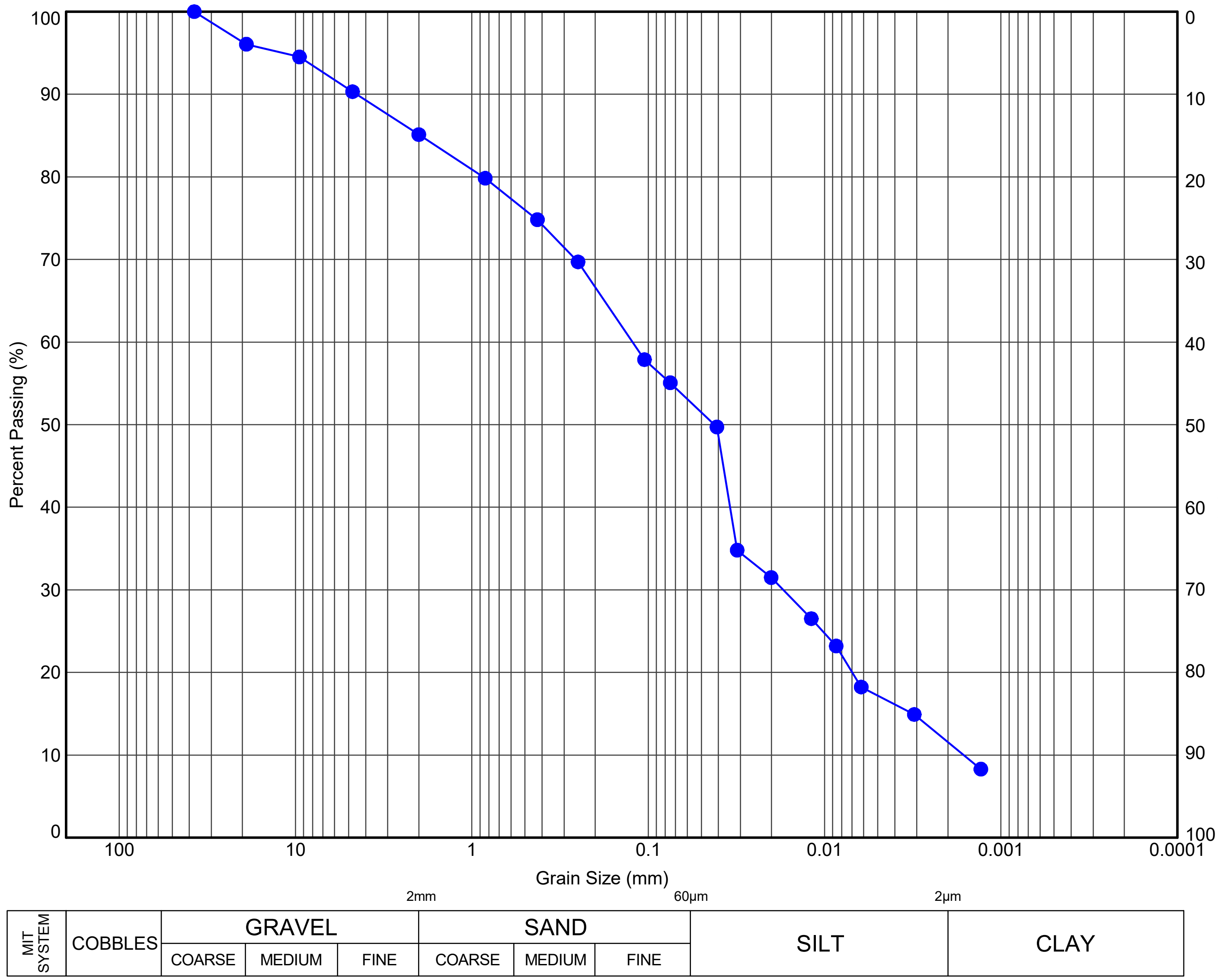
					MIT SYSTEM				
	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**2**	SS6	4.8	174.4	16	16	60	8	

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

GRAIN SIZE DISTRIBUTION
SILT, SOME SAND, SOME GRAVEL, TRACE CLAY

File No.: **1-19-0773-01**



MIT SYSTEM

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**5**	SS6	4.7	173.7	15	31	42	12	

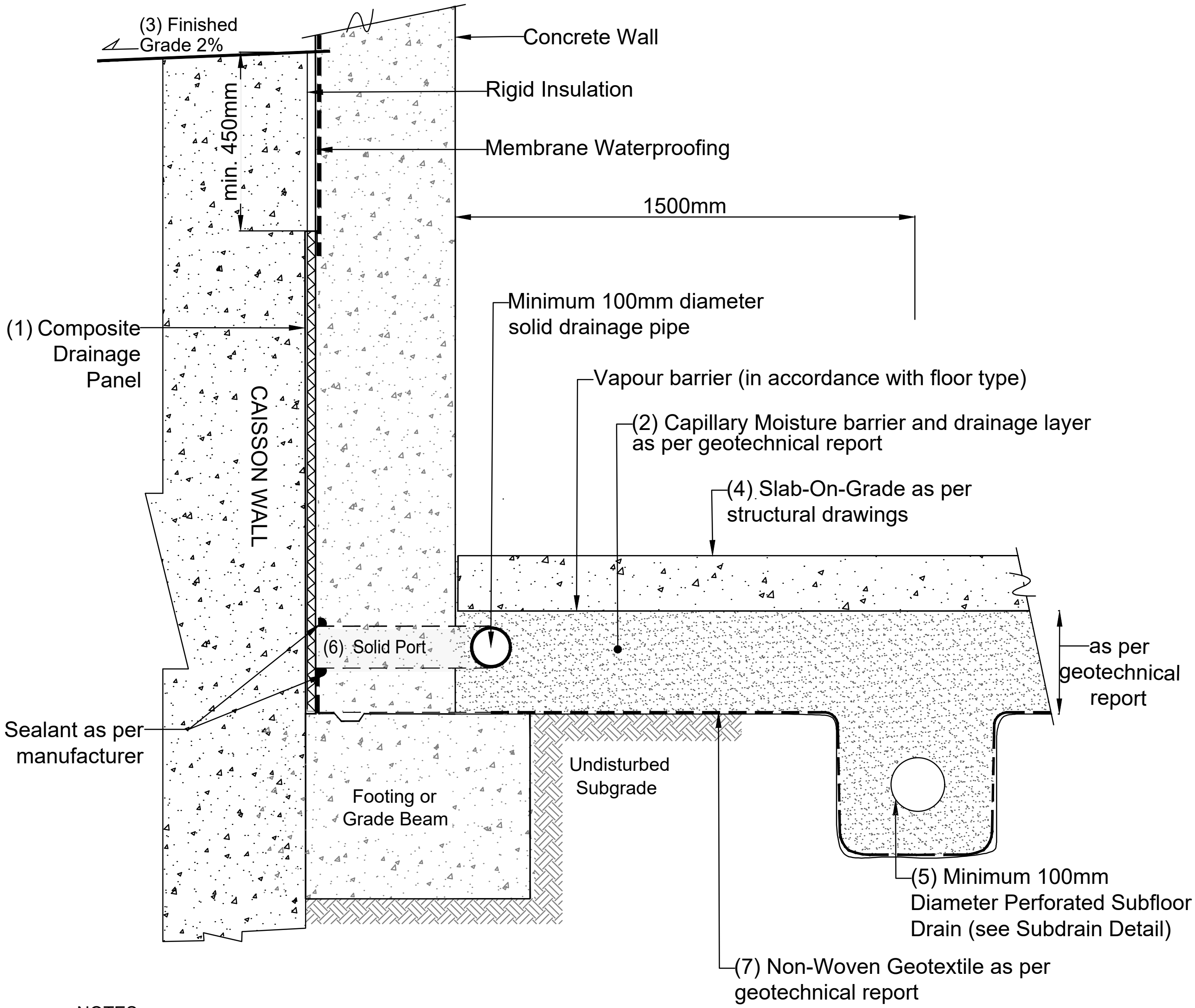
Terraprobe

11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:	**GRAIN SIZE DISTRIBUTION** **SANDY SILT, SOME GRAVEL, SOME CLAY**
File No.:	**1-19-0773-01**

APPENDIX C

TERRAPROBE INC.



NOTES

1) Prefabricated composite drainage panels to consist of Miradrain 6000, or approved equivalent. Panels should provide continuous cover as per manufacturer's requirements.

2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS. MUNI 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular “A” (OPSS. MUNI 1010) compacted to 98% SPMDD where vehicular traffic is required. A vapour barrier may be required depending on floor type.

3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.

4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.

5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report. Don't connect subfloor drains to perimeter drains.

6) Embedded ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in solidpipe.

7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

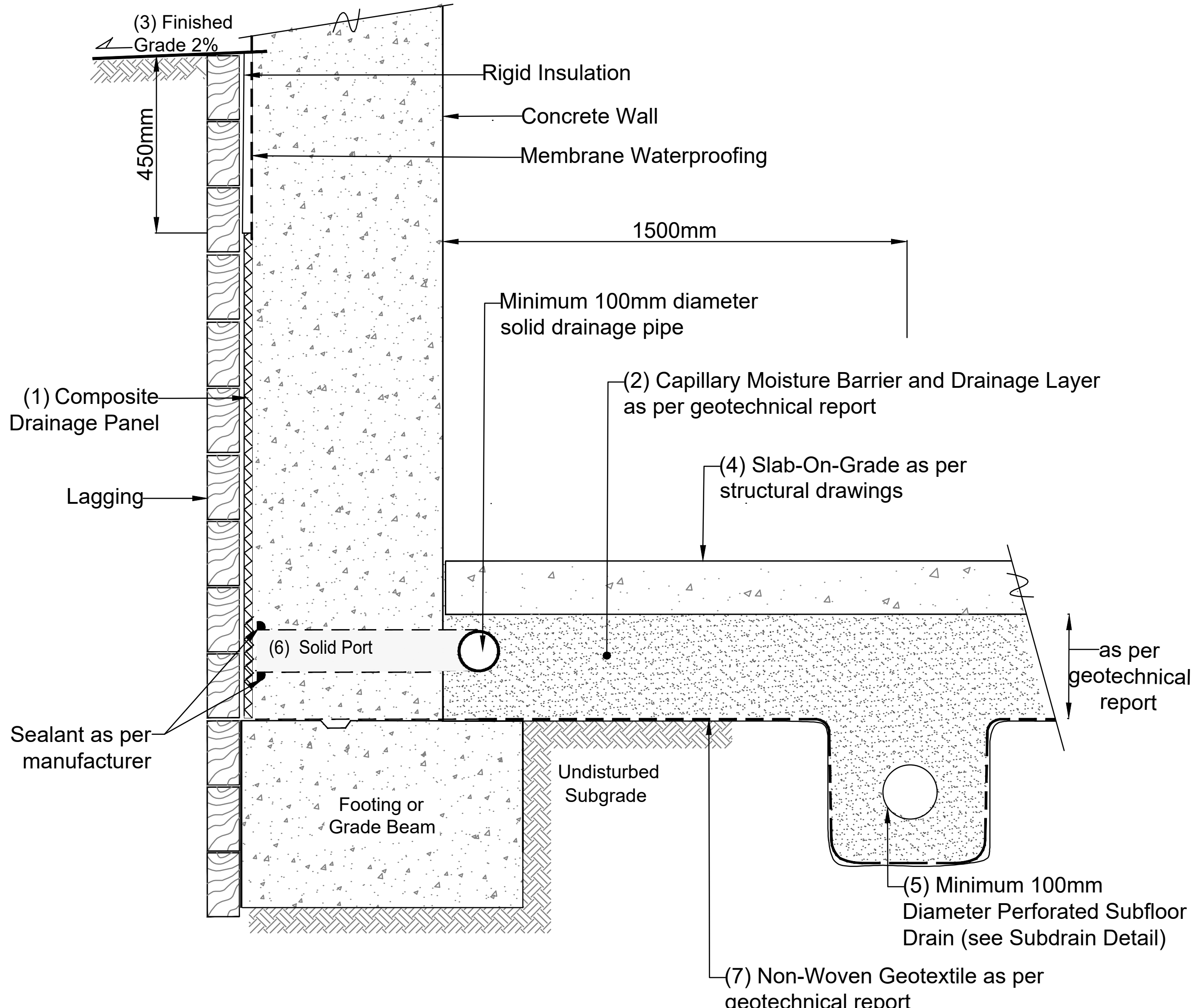
8) Geotechnical report contains specific details. Final detail must be reviewed before system is considered acceptable to use.

N.T.S

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

SCHEMATIC DRAINAGE DETAIL
CASSION WALL SHORING SYSTEM
(DRAINED BASEMENT CONDITION)



NOTES

1) Prefabricated composite drainage panels to consist of Miradrain 6000, or approved equivalent. Panels should provide continuous cover as per manufacturer's requirements.

2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS. MUNI 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular “A” (OPSS. MUNI 1010) compacted to 98% SPMDD where vehicular traffic is required. A vapour barrier may be required depending on floor type.

3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.

4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.

5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report. Don't connect subfloor drains to perimeter drains.

6) Embedded ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in solidpipe.

7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

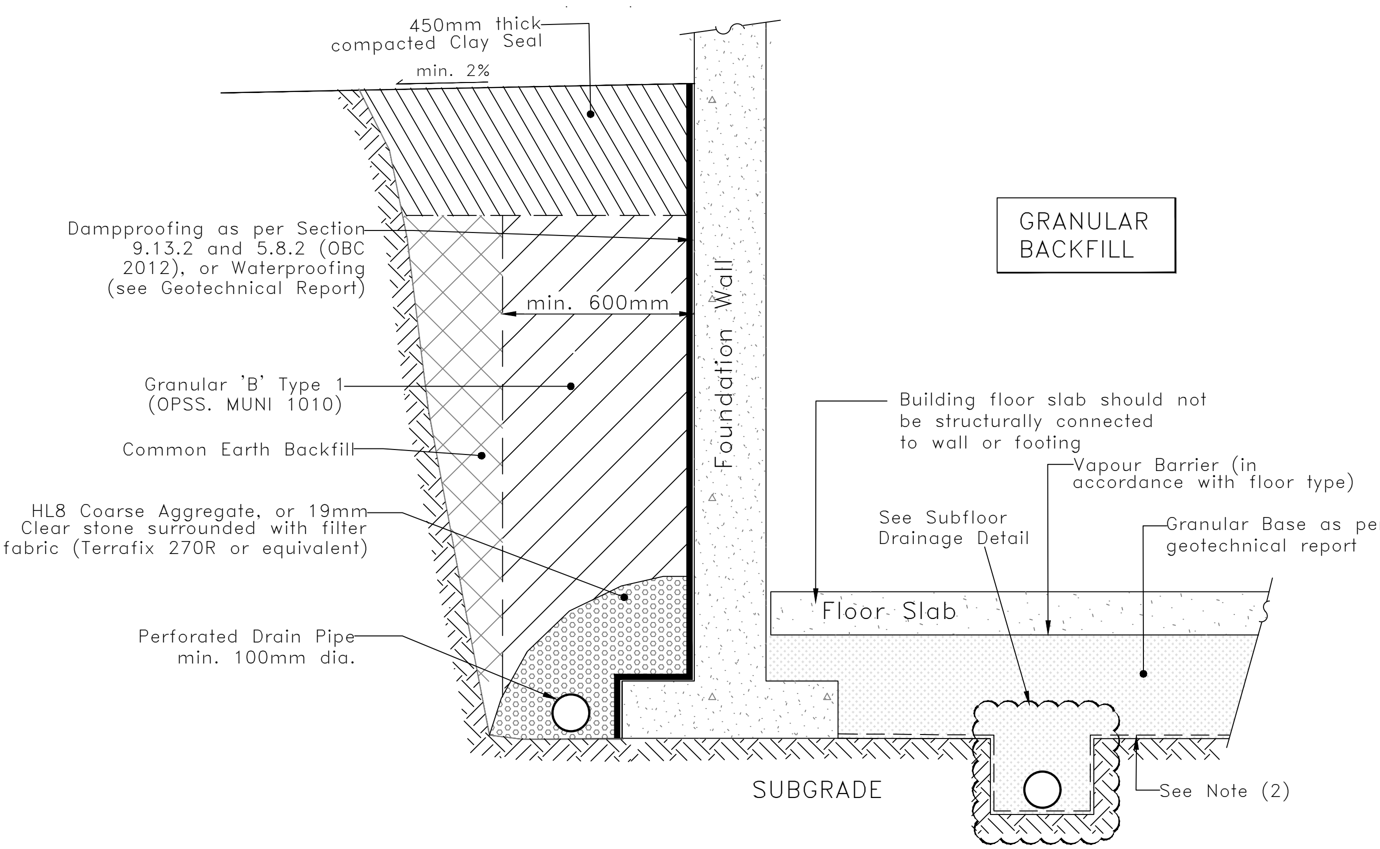
8) Geotechnical report contains specific details. Final detail must be reviewed before system is considered acceptable to use.

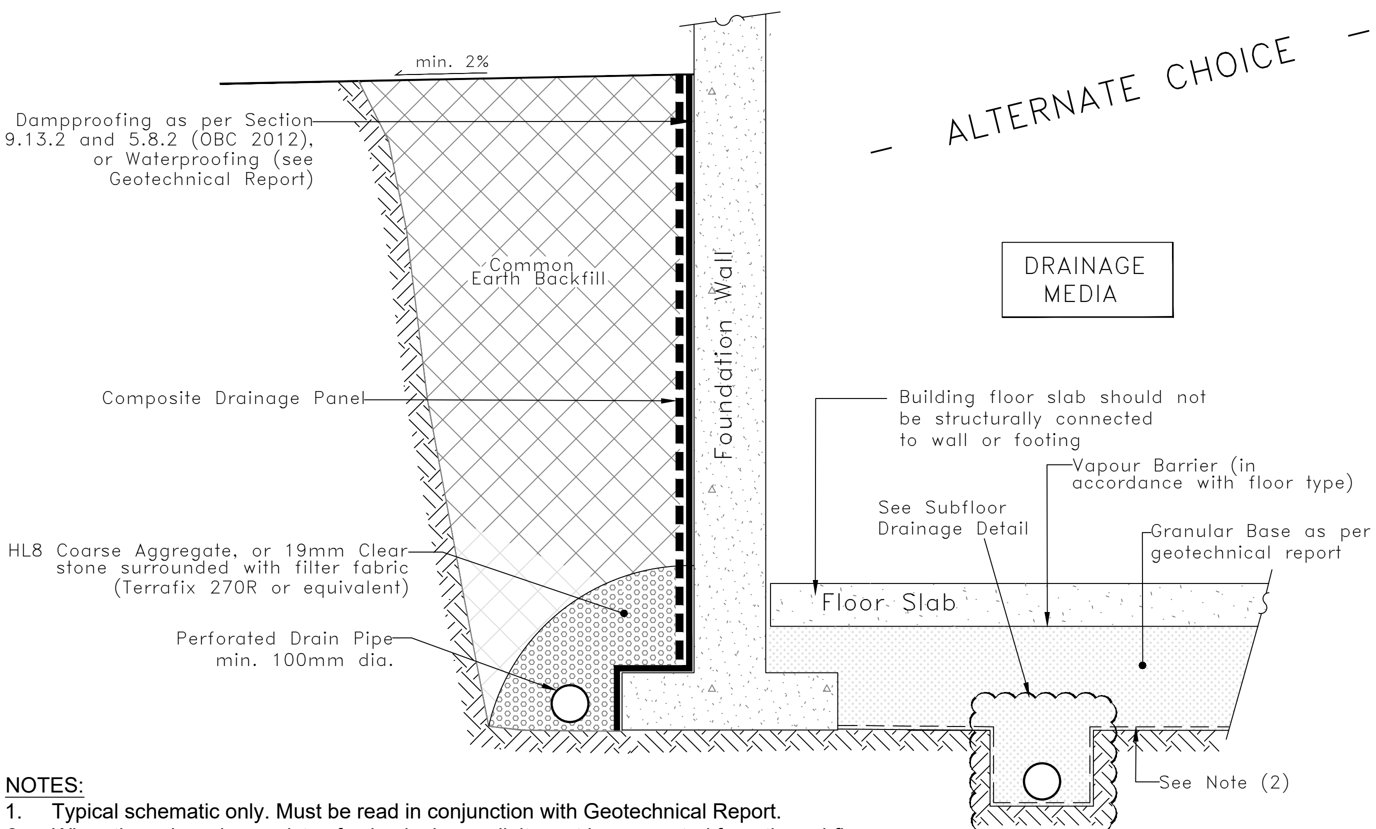
N.T.S.

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

SCHEMATIC DRAINAGE DETAIL
SOLDIER PILE & LAGGING SHORING SYSTEM
(DRAINED BASEMENT CONDITION)





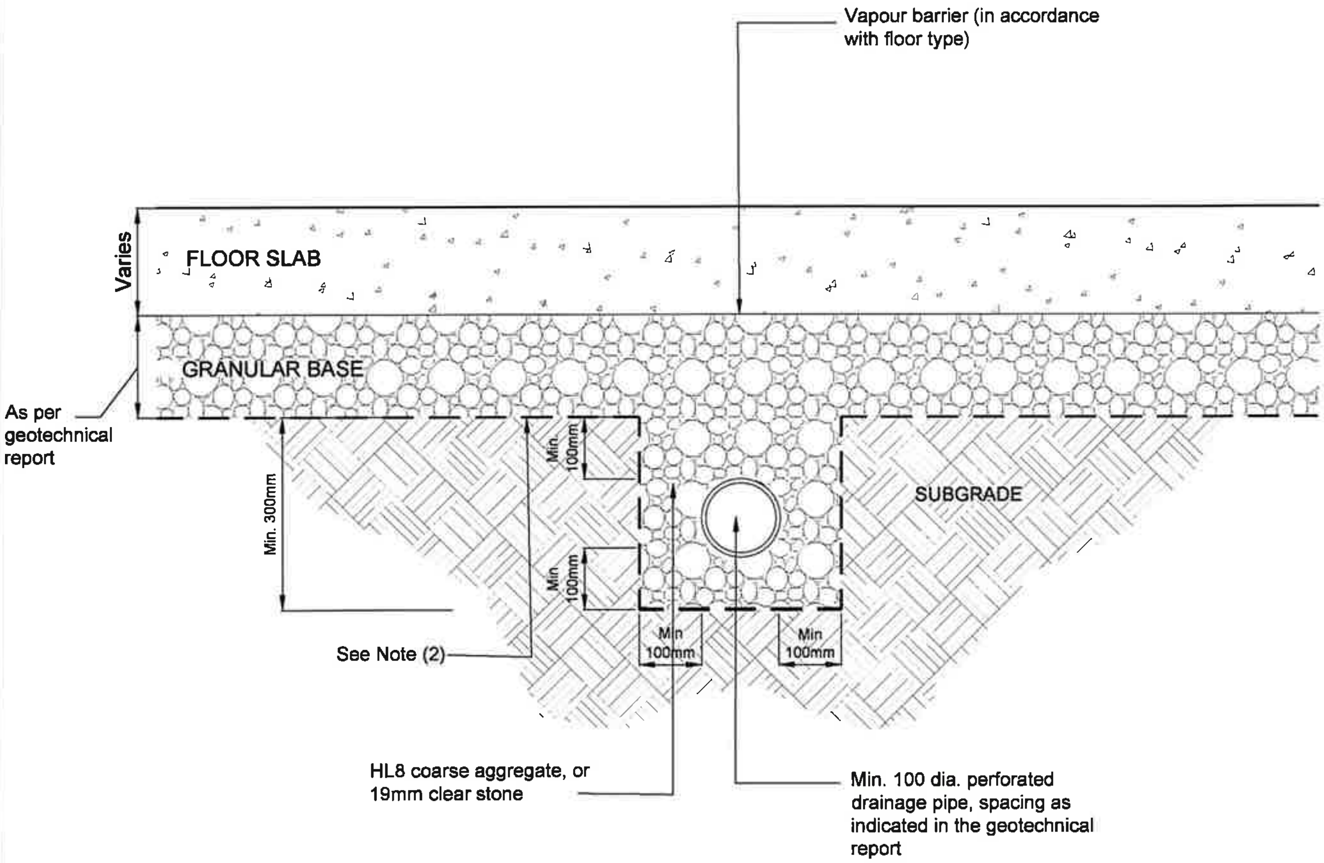
NOTES:

1. Typical schematic only. Must be read in conjunction with Geotechnical Report.
2. When the subgrade consists of cohesionless soil, it must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).
3. Not to Scale

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

TYPICAL BASEMENT DRAINAGE SCHEMATIC
(OPEN EXCAVATION)



NOTES:

1. Typical schematic only. Must be read in conjunction with Geotechnical Report.
2. When the subgrade consists of cohesionless soil, it must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).
3. Not to Scale

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

TYPICAL BASEMENT SUBDRAIN DETAIL