Terraprobe

Consulting Geotechnical & Environmental Engineering Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED USE DEVELOPMENT
121 HUME STREET COLLINGWOOD, ONTARIO

Prepared for: 2554281 Ontario Ltd.
C/o Greenland Consulting Engineers
120 Hume Street
Collingwood, Ontario
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1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) was retained by 2554281 Ontario Ltd. to conduct a geotechnical investigation for a proposed mixed use residential/commercial development located in the northwest corner of the intersection of Hume Street and Market Street, in the Town of Collingwood, Ontario.

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 groundwater 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 groundwater control.

Terraprobe has previously carried out review of Phase One and Phase Two Environmental Site Assessments (ESA) of the site, carried out by others. The findings of our review are documented under separate cover.

Terraprobe is also conducting groundwater monitoring for the site, the findings of which are reported under separate cover(s).

2.0 SITE AND PROJECT DESCRIPTIONS

The site is located in the northwest corner of the intersection of Hume Street and Market Street, in the Town of Collingwood, Ontario, with a municipal address of 121 Hume Street. The general location of the site is presented on Figure 1. The site is located about 500 m from Collingwood Harbour and the former Collingwood Shipyards.

The site consists of an irregular shaped parcel of land with an area of approximately 3,000 ±square metre. The site is currently vacant.

It is understood that the site was previously occupied by an automotive service centre which was subsequently demolished, and remediation activities including soil excavation/removal were carried out.

Based on the architectural drawing set titled “Hume Hub Mixed Use Development” prepared by ACK Architects, dated Nov. 2019 (Rev. 13, Dec. 17, 2020), it is understood that the proposed development would include a four (4) storey mixed-used residential and commercial building with one (1) level of underground parking (P1). Based on the Site Grading Plan (File No. D11420; Drawing No. 16006-GR1, by Urban Watershed Group Ltd., stamped Dec. 17, 2020) it is further understood that the proposed basement finished floor will be set at about Elevation: 180.30 m.

3.0 INVESTIGATION PROCEDURE

The field investigation was conducted on December 7, 2020, and consisted of drilling and sampling a total of four (4) boreholes, extending to about 2.6 to 3.8 m depth below grade. The approximate locations of the boreholes are shown on the enclosed Borehole Location Plan (Figure 2A – Existing Condition and Figure 2B – Proposed Condition).

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 two (2) selected native soil sample (Borehole 1, Sample 4 and Borehole 3, Sample 5). 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 4.4 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 each borehole to facilitate groundwater monitoring. 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 December 19, 2020. The results of groundwater monitoring are presented in Section 4.5 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 Earth Fill

A zone of earth fill material was encountered from the ground surface in all boreholes and extended to a depth of about 2.3 to 3.1 m below existing grade. The earth fill materials predominately consisted of sand with some gravel and trace amounts of silt. Sporadic and a trace amounts of organics was noted within the fill materials at varying depths.

The Standard Penetration Test results ('N' Values) obtained from the earth fill materials varied from 3 to 28 blows per 300 mm of penetration, indicating a typically very loose to compact relative density.

Measured moisture contents of the earth fill materials samples generally varied from about 6 to 23 percent by weight, indicating a moist to wet condition.

4.2 Silty Sand (Glacial) Till

A silty sand glacial till deposit, with trace to some clay and some gravel was encountered beneath the fill layer in Boreholes 1, 2 and 3, and extended to the depths ranging from about 2.6 to 3.8 m below grade, auger/sample refusal on inferred bedrock surface.

N-values obtained from the undisturbed silty sand till deposit were in excess of 50 blows per 300 mm of penetration, indicating a very dense relative density. The in-situ moisture contents of the glacial till samples ranged from 5 to 8 percent by weight, indicating a moist condition.

4.3 Inferred Bedrock

The bedrock surface was inferred by auger/sample refusal in all boreholes, at depths ranging from about 2.6 to 3.8 m (Elev. 181.3 m to 182.4 ±m) below grade. The bedrock in the area generally consists of limestone of the Simcoe Group. The rock is horizontally bedded, and contains minor shale interbeds.

4.4 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 two selected native soil sample. 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 4	2.5	16	43	33	8	SILTY SAND, some gravel, trace clay
Borehole 3, Sample 5	3.1	20	36	34	10	SILTY SAND, some gravel, some clay

4.5 Groundwater

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 all boreholes to facilitate shallow groundwater level monitoring. The groundwater level measurements in the monitoring wells were taken on December 19, 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 December 19, 2020
		Depth to Cave (m)	Unstabilized Water Level (m)	Depth/Elev. (m)
BH 1	2.6	open	2.3	1.7/183.0
BH 2	2.7	open	dry	2.0/183.0
BH 3	3.8	open	dry	2.0/183.1
BH 4	3.1	open	dry	2.0/183.1

The groundwater levels encountered within the monitoring wells (December 19, 2020 monitoring) were found to be at an elevation ranging from about 183.0 m to 183.1 m. For design purposes, the stabilized groundwater level on the site is estimated to be at approximately Elev. 183.1 ±m (highest groundwater level recorded in Boreholes 3 and 4 and extrapolated to Boreholes 1 and 2). Groundwater conditions and levels will vary seasonally and could be higher during wetter seasons/years. The seasonal groundwater levels for the site will be confirmed through on-going groundwater monitoring program.

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 4 are located within or in the vicinity of the 4-storey building footprint. The boreholes encountered an earth fill zone extended to a depth of about 2.3 to 3.0 m below existing grade, underlain by undisturbed native soil deposits and/or inferred bedrock, extending to the full depth of the investigation in each borehole.

It is understood that the proposed mixed-use building would be a 4-storey structure with one level of underground parking garage. Based on the Site Grading Plan (File No. D11420; Drawing No. 16006-GR1, by Urban Watershed Group Ltd., stamped Dec. 17, 2020) it is understood that the proposed basement finished floor will be set at about Elevation: 180.30 m. As such, the P1 finished floor level will be set below the inferred bedrock surface.

Subject to confirmation of the sound bedrock by adequate number of additional boreholes with rock coring at detail design stage, conventional spread footings can be made to bear on sound (unweathered) bedrock below Elev. 181 ±m, and designed using a maximum factored geotechnical resistance at Ultimate Limit States (ULS) of 5,000 kPa. Serviceability Limit States (SLS) does not apply for shallow foundations bearing directly on bedrock since the loads required for appreciable settlement to occur would be much greater than the factored resistance at the ULS. Foundations installed in accordance with the above recommendations would be expected to experience very little settlement likely limited to the elastic deformation of the concrete.

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.

Footings stepped from one level to another bearing on bedrock should be at a slope not exceeding 1 vertical to 1 horizontal for the above bearing pressures to be applicable. There must also be a minimum of 300 mm horizontal ledge between the edge of any footing and the top of the rock cut down to another footing.

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

Prior to pouring concrete for the footings, the footing subgrade must be cleaned of all deleterious materials such as softened, disturbed or caved materials, as well as any weathered rock or standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided. It should be noted that the bedrock surface can weather 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

It is understood that the basement slab (P1) is to be made at about Elev. 180.30 ±m on the inferred limestone bedrock of the Simcoe Formation, which is suitable for the support of a slab on grade. The modulus of subgrade reaction appropriate for design of the slab resting on an aggregate drainage layer overlying unweathered (sound) bedrock is 80,000 kPa/m.

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 19 mm stone (OPSS.MUNI 1004) compacted by vibration to a dense state. This material also serves as the drainage media for the subfloor drainage system. Provision of subfloor drainage is required in conjunction with the perimeter drainage of the structure.

The subfloor drainage system is an important building element, as such the storm sump which ensures the performance of this system must have a duplexed pump arrangement for 100% pumping redundancy and this pump must be provided with emergency power as needed. Basement and subfloor drainage provisions are further discussed in Section 5.4 of this report.

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 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 accepted levels of moisture/relative humidity in the concrete slab prior to the installation of floor finishes. Studies indicate that a provision of 200 mm thick 19 mm clear stone base (OPSS.MUNI 1004) under the slab helps provide a good capillary moisture break provided the granular base is positively drained. However, this provision does not replace the floor manufacturer's specific requirement(s) for a moisture and vapour barrier.

The under-slab vapour retarder specifications, selection and installation shall conform to ASTM E1745 and ASTM E1643. The moisture vapour measurement tests shall conform to RH: ASTM F2170, RH: ASTM F2420 and Calcium Chloride: ASTM F1869. The Surface Applied Moisture Vapour Barrier system shall meet the guidelines established in ASTM F3010-13.

5.3 Lateral Earth Pressure Design Parameters

Given the proposed project design and the site subsurface conditions, an interlocking concrete caisson wall will be required down into the bedrock and completely surrounding the site in order to permit safe and dry excavation for the proposed building construction. The permanent caisson wall will act to support the surrounding ground and to cut-off the building area from ground water seepage. Walls or bracings subject to unbalanced lateral 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 groundwater 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 ϕ**. 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 follows:

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 Silty Sand Till	21.0	34	0.28	0.44	3.54
Limestone Bedrock	24.0	28	n/a	n/a	n/a

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

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 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 3 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 pit located in the lowest level of the basement. The water from the sump pit 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).

$$\upsilon_{s-avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{\upsilon_{si}}}$$

Shear Wave Velocity

$$S_{u-avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{s_{ui}}}$$

Undrained Shear Strength

$$N_{avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

SPT N-values

Based on the borehole data (advanced to a maximum of 3.8 m depth below grade), it is understood that the proposed building will be founded on unweathered (sound) bedrock of the Simcoe Formation. Therefore, it is recommended that the site designation for seismic analysis is Class B for structures founded on bedrock, as per 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 same code provide the applicable acceleration and velocity based site coefficients.

Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) \geq 1.25$
B	0.8	0.8	0.9	1.0	1.0

Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
B	0.6	0.7	0.7	0.8	0.8

5.6 Pavement

It is understood that project will include at-grade parking and/or driveways supported on soil subgrade, and constructed on concrete deck. Design recommendations for the pavement structure supported on soil subgrade are provided in this section.

5.6.1 Pavement Design

The industry pavement design methods are based on a design life of 15 to 20 years for typical weather conditions depending on actual traffic volumes. Traffic data was not provided to Terraprobe for review in preparation of this report. The following pavement thickness design is provided on the above noted considerations and subgrade basis.

Performance Asphaltic Concrete Pavement Structure

Pavement Structural Layers	Entrance Driveway (Heavy-Duty/Truck Traffic)	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	80 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	350 mm	

The following minimal pavement design for the entrance driveway supported on soil subgrade is provided, which will provide an estimated service period of about 8 to 10 years depending on actual traffic volumes. The cost of this pavement design should be compared to the performance design which could be expected to last about twice as long before significant maintenance and rehabilitation.

Minimal Asphaltic Concrete Pavement Structure

Pavement Structural Layers	Entrance Driveway (Heavy-Duty Truck Traffic)	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	60 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 catchbasin(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 catchbasins. 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 catchbasins) should be provided along both sides of driveway curblines. 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 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 Pavement Above Underground Parking Structure

It is understood that the project will include at-grade driveway and parking lot supported over the P1 below grade parking deck. The following design is provided for the pavement above the parking structure.

5.7.1 Typical Pavement above Parking Garage

A typical Pavement Components make-up above a concrete deck slab are as follows:

-Reinforced Concrete Slab Substrate

-Waterproofing with Protection Board (to be specified by Architect)

-Drainage Board (to be specified by Architect)

-Filter Fabric (to be specified by Architect)

-Min 300 mm Base Granular ‘A’ or 19 mm Crusher run limestone

-65 mm HL8

-40 mm HL3

The parking garage concrete structure must be designed to support the applicable loads of vehicles, i.e. fire trucks, garbage trucks, etc. which will access the laneway. Compaction requirements to meet those provided in Section 5.6.3.

5.7.2 Drainage

Drainage measures shall be provided for the pavement above the garage deck. This is typically provided by provision of 2 to 3 % grade on the garage deck and installing a subdrain system. A perimeter drain at the edge(s) of the garage deck should be considered in addition to above noted subdrain.

5.8 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.

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 groundwater 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.
The overburden soils can be removed by conventional excavation equipment. 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.

Under the Act and Regulations, bedrock of the Simcoe Group Formation is not considered a soil. Where the excavation penetrates the bedrock, a vertical excavation made in sound bedrock is nominally self-supporting provided the rock bedding is horizontally oriented. The vertical rock face must be inspected by a geotechnical engineer to ensure no other support system is required to prevent the spalling of loose rock, and to ensure that all loose material at risk of falling upon a worker is removed (Section 233 of the above noted regulations). Should it be deemed necessary, rock bolts can anchor a layer of protective mesh that will protect workers from loose material spalling from the face of excavation.

The rock excavation may be conducted through line drilling in conjunction with hoe ramming. The line drilling is a process of drilling closely-spaced vertical holes into the rock (100 mm diameter, 300 mm centre to centre spacing) to provide a preferential break path for the excavation in the vertical plane and allow much easier breaking of the bedrock with a hydraulic ram. Excavating detailed shapes for foundations and the edges of the excavation are normally accomplished with hoe mounted hydraulic rams. The ability to remove the rock in a vertical face without over-excavation and dislodging of additional rock is largely dependent on the skill of the machine operator.

Very hard limestone layer may be encountered during excavation. Provision must be made in excavation contracts to allocate risks associated with the time spent and equipment utilized to remove or penetrate such obstructions. In the case of excess rock removal not intrinsic to the project requirements, the risk and responsibility for the excess rock removal under these circumstances, and the supply and placement of the extra concrete to restore the foundation grade, must be addressed in the contract documents for foundations, excavation, and shoring contractors.

Controlled blasting is not recommended due to the close proximity of the excavation area to the nearby existing structures.

5.9 Groundwater Control

For design purposes, the stabilized groundwater table at this site is taken as Elev. 183.1 ±m. In general, the excavation to the proposed P1 level at Elev. 180.30 ±m and lower protrusions for foundations, elevator pits and sumps will extend further below the design groundwater table at this site. There is also perched water in the fill materials.

The shoring wall may be designed using a continuous concrete caisson wall socketed into the low-permeability bedrock. This shoring approach provides a fully-continuous temporary groundwater cut-off barrier, which will enable the site to be excavated during construction without inducing additional flow into the excavation from the over-burden. Dewatering inside an excavation protected by a cut-off wall may be conducted using conventional sump arrangements, and therefore it may be possible to control discharge such that a Permit to Take Water (PTTW) from the Ministry of Environment (50,000 L/day) may not be required. If caisson walls are advanced as cut-off walls on all sides of the excavation, the site may be excavated without exceeding the discharge limit for groundwater, but may still need to be exceeded during precipitation events.

Alternatively, if permeable shoring (i.e. soldier piles with lagging) is used to support the excavation, a positive dewatering system will be required to lower the groundwater table in the overburden prior to excavation for lagging board installation. Without prior dewatering, the fill and silty sand till at the site could potentially slough into the excavation during lagging board installation which could result in loss of ground outside the excavation. It is expected that seepage from the glacial Till and from horizontal fractures in the bedrock will be of limited extent. This seepage may be allowed to drain into the excavation and then pumped out. Issues of delay in excavation due to localized seepage control must be addressed in the excavation contract.

Precipitation events will be the primary contributor of water entering the excavation. Large precipitation events will create volume in excess of 50,000 L/day, which will then need to be pumped out of the excavation.

This dewatering can be staged over the course of multiple days so as not to exceed the 50,000 L/day limit for water removal without an EASR posting; otherwise, an EASR posting can be obtained in advance of construction to avoid possible delays. If required, a hydrogeological investigation can be carried out to determine the estimated volumes of water to be discharged for construction purposes.

5.10 Backfill

The 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 sortingas 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.

5.11 Shoring Design Consideration

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 may 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, groundwater 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.

5.11.1 Earth Pressure Distribution

A single level of support would be likely required for shoring system, and a triangular earth pressure distribution similar to that used for the basement wall design, is appropriate for this case,

$$P = K(\gamma h + q)$$

Where:
- **P** = the horizontal pressure (kPa)
- **K** = the earth pressure coefficient
- **h** = the depth below the ground surface (m)
- **γ** = the bulk unit weight of soil (kN/m^3)
- **q** = the complete surcharge loading (kPa)

Applicable soil parameters are included in the Earth Pressure Design Parameters Section (Section 5.3).

5.11.2 Caisson and Soldier Pile Toe Design

It is envisaged that the caisson and soldier pile toes will be made in the sound bedrock (sound bedrock must be confirmed by advancement of suitable number of additional boreholes with rock coring) of the Simcoe Formation. 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 untethered 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 groundwater level. Where soils exist beneath the groundwater 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 groundwater 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.

5.11.3 Shoring Support

If anchor support is necessary and determined to be feasible, the shoring system should be supported by pre-stressed rock 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. Rock pins/bolts may be required at the exposed cut-face of the bedrock depending upon the excavation shoring design approach

Anchors made in bedrock of the Simcoe Formation may be designed using a working adhesion of 620 kPa. Proto-type anchors must be performance tested to 200% of the design load to demonstrate the anchor capacity. All production anchors must be proof-tested to 133% of the design load, to validate the design assumptions.

Where anchors cannot be used, internal bracing or raker support will be necessary. Raker footings established on the sound bedrock at an inclination of 45 degrees can be designed for a maximum geotechnical resistance at ULS of 2,500 kPa.

5.12 Quality Control

Excavations on this site may require shoring to preserve the integrity of the surrounding properties and structures. The 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 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 2554281 Ontario Ltd. 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 2554281 Ontario Ltd. 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
90337072
Jan 7, 2021
PROVINCE OF ONTARIO

O. Benjamin

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

Michael Tanos

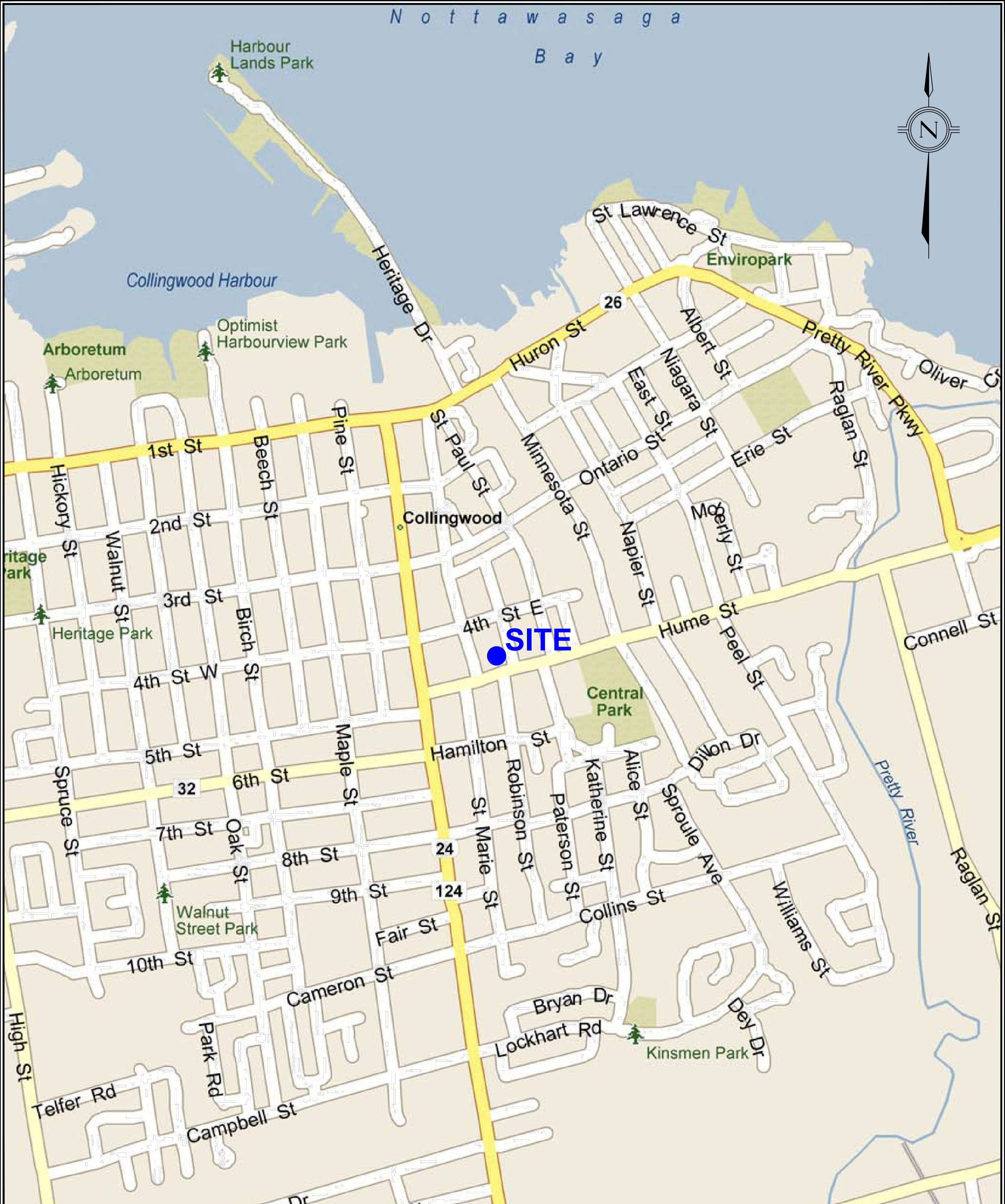
Michael Tanos, P. Eng.
Review Principal

ENCLOSURES

TERRAPROBE INC.

FIGURES

TERRAPROBE INC.



REFERENCE
Maps data @ 2020 Google

n 0.2 0.4 0.6 0.8 1 1.2

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

Title: SITE LOCATION PLAN	FIGURE : 1
File No. 1-20-0483-01	

REFERENCE
Maps data @ 2020 Google

LEGEND
Approximate Borehole Location

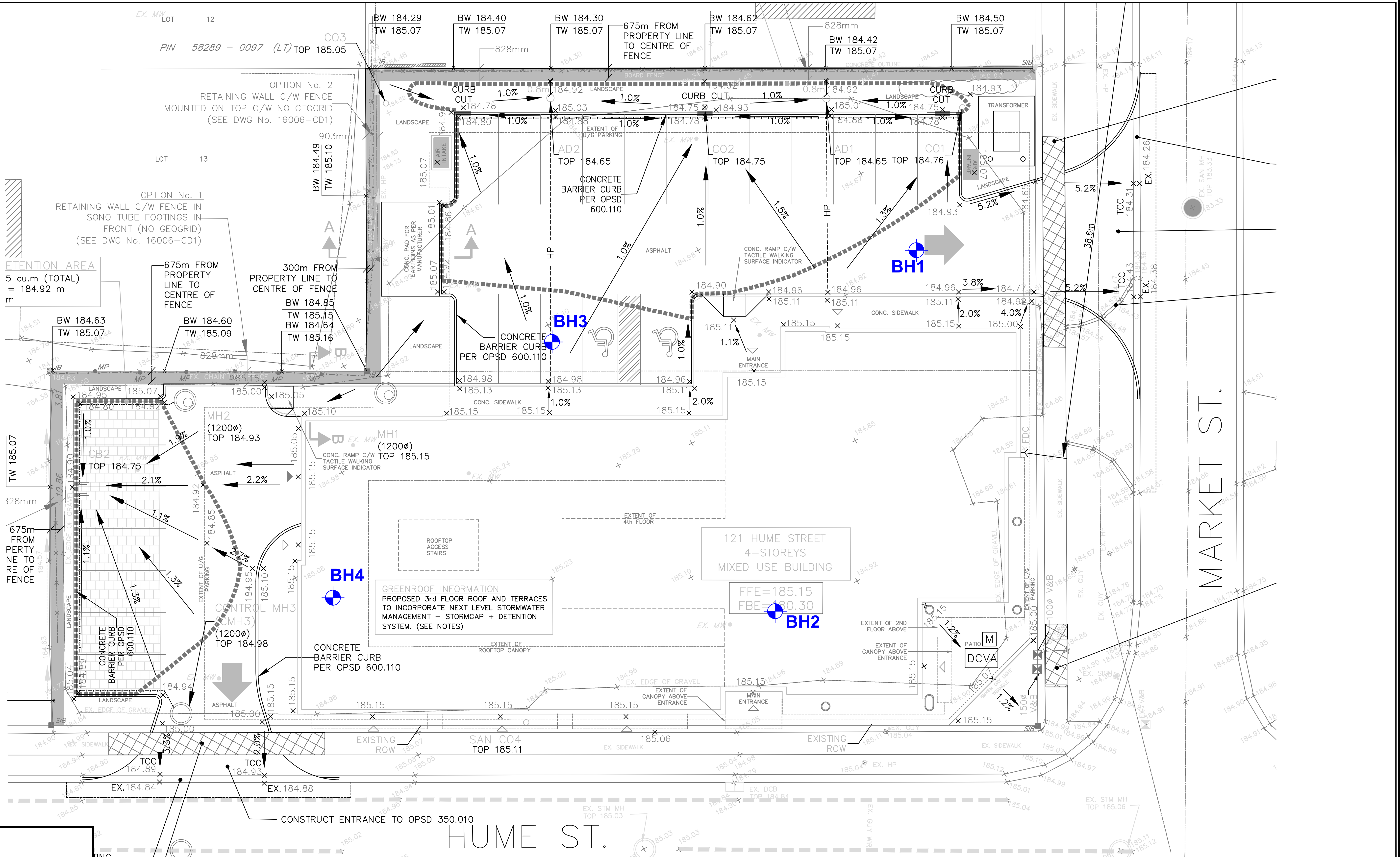
SCALE 1:250
0 2 4 6 8 10

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

Title: BOREHOLE LOCATION AND SITE FEATURES PLAN (EXISTING CONDITION)

File No. 1-20-0483-01

FIGURE : 2A



REFERENCE
Site Grading Plan, Proposed Mixed Use Development, File No.: D11420
Dwg No.: 16006-GR1, Project No.:
Dated: May 09, 2017
By: Urban Watershed Group Ltd.

LEGEND
Approximate Borehole Location

SCALE 1:250
0 2 4 6 8 10

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

Title: BOREHOLE LOCATION AND SITE FEATURES PLAN (PROPOSED CONDITION)

File No. 1-20-0483-01

FIGURE : 2B

APPENDIX A

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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 or close to plastic limit) but does not have visible pore water

Wet refers to a soil sample that has visible pore water

Terraprobe

LOG OF BOREHOLE 1

Project No. : 1-20-0483-01 | Client : 2554281 Ontario c/o Greenland Consulting Engineers | Originated by : RS

Date started : December 7, 2020 | Project : 121 Hume St | Compiled by : HR

Sheet No. : 1 of 1 | Location : Collingwood, Ontario | Checked by : OB

Position : E: 562581, N: 4927314 (UTM 17T) | Elevation Datum : Geodetic

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

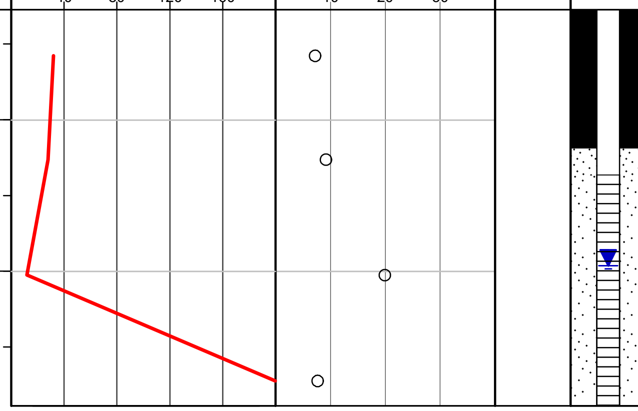
Depth Scale (m)	Elev Depth (m)	Description	Graphic Log	Sample Number	Type	SPT 'N' Value	Elevation Scale (m)	Moisture (Natural Water Content)	Instrument Details	Unstabilized Water Level	Lab Data and Comments (Grain Size Distribution (%) (MIT) GR SA SI CL)
0	184.7	**GROUND SURFACE**									
		FILL, sand, some silt, trace to some gravel, trace clay, very loose to loose, brown, moist		1	SS	8					
1				2	SS	7	184				
		...at 1.5 m, wet below		3	SS	3	183				
2	182.4 / 2.3	**SILTY SAND**, some gravel, trace to some clay, very dense, grey, moist (GLACIAL TILL)		4	SS	77 / 175mm				▽	16 43 33 8
	182.1 / 2.6										...at 2.6m, auger grinding

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 (PL), Natural Water Content (MC), Liquid Limit (LL): 10, 20, 30

Headspace Vapour (ppm)



END OF BOREHOLE
Auger refusal on inferred bedrock

Unstabilized water level measured at 2.3 m below ground surface; borehole was open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 19, 2020	1.7	183.0

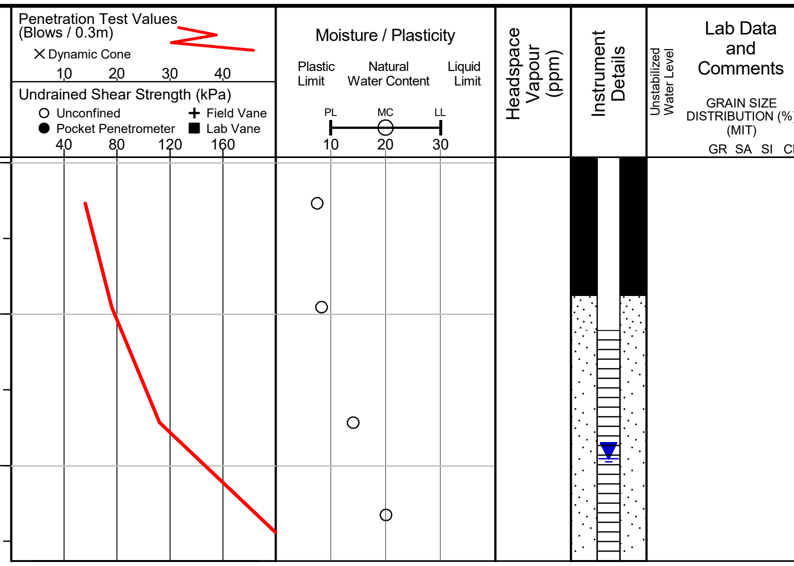
LOG OF BOREHOLE 2

Terraprobe

Project No. : 1-20-0483-01	Client : 2554281 Ontario c/o Greenland Consulting Engineers	Originated by : RS
Date started : December 7, 2020	Project : 121 Hume St	Compiled by : HR
Sheet No. : 1 of 1	Location : Collingwood, Ontario	Checked by : OB

Position : E: 562560, N: 4927306 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value	Elevation Scale (m)
	185.0	**GROUND SURFACE**					185
0		**FILL**, sand, some silt, trace to some gravel, trace clay, compact, brown, moist		1	SS	14	
1				2	SS	19	184
				3	SS	28	
2							183
	182.6 2.4	...at 2.3 m, wet below, organic staining below		4A	SS	65 / 225mm	
	182.3 2.7	**SILTY SAND**, some gravel, trace to some clay, very dense, brown, moist (GLACIAL TILL)		4B			



END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 19, 2020	2.0	183.0

Terraprobe

LOG OF BOREHOLE 3

Project No. : 1-20-0483-01	Client : 2554281 Ontario c/o Greenland Consulting Engineers	Originated by : RS
Date started : December 7, 2020	Project : 121 Hume St	Compiled by : HR
Sheet No. : 1 of 1	Location : Collingwood, Ontario	Checked by : OB

Position : E: 562552, N: 4927284 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Solid stem augers

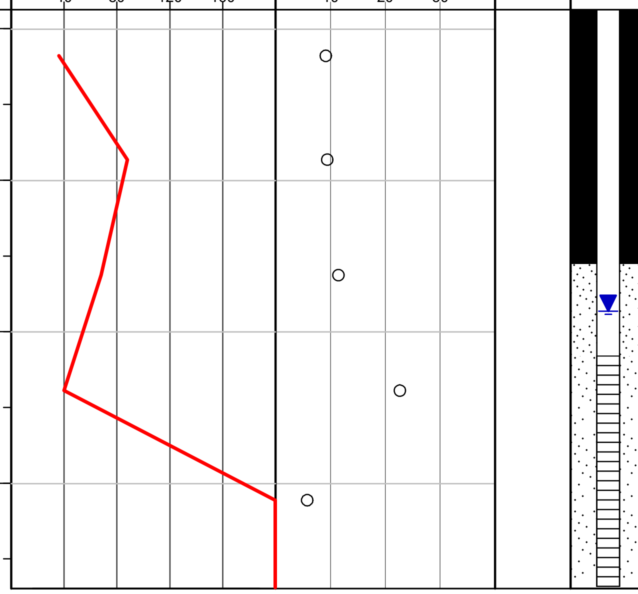
Depth Scale (m)	Elev Depth (m)	Description (SOIL PROFILE)	Graphic Log	Number	Type	SPT 'N' Value	Elevation Scale (m)	Moisture / Plasticity (MC %)	Lab Data and Comments: GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
0	185.1	GROUND SURFACE					185		
		FILL, sand, some silt, trace to some gravel, trace clay, loose to compact, brown, moist		1	SS	9			
1				2	SS	22	184		
2				3	SS	17	183		
		...at 2.3 m, wet below		4	SS	10			
3	182.1 3.0	**SILTY SAND**, some gravel, trace to some clay, very dense, grey, moist (GLACIAL TILL)		5	SS	88 / 225mm	182		20 36 34 10
	181.3 3.8	Inferred bedrock		6	SS	50 / 15mm			
	181.3 3.8	**END OF BOREHOLE**							

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



Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 19, 2020	2.0	183.1

file: 1-20-0483-01 bh logs.gpj

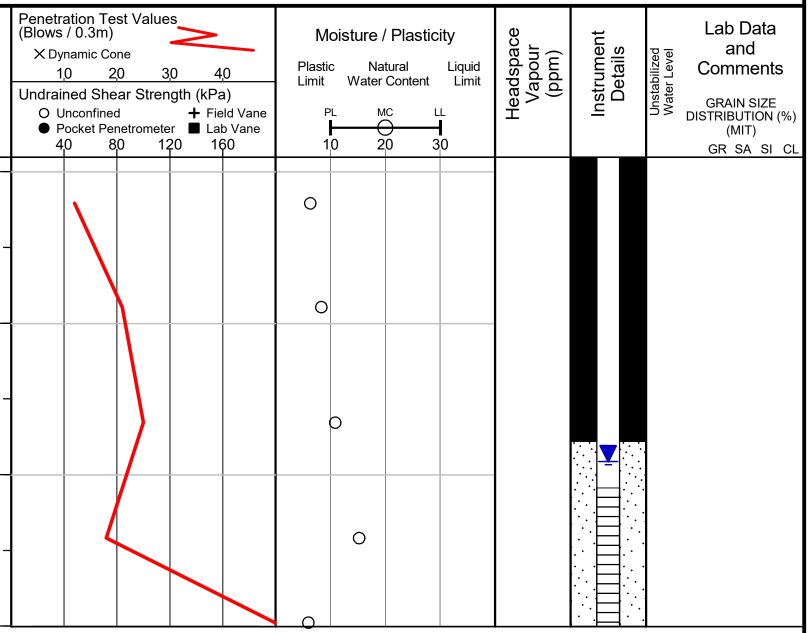
LOG OF BOREHOLE 4

Terraprobe

Project No. : 1-20-0483-01	Client : 2554281 Ontario c/o Greenland Consulting Engineers	Originated by : RS
Date started : December 7, 2020	Project : 121 Hume St	Compiled by : HR
Sheet No. : 1 of 1	Location : Collingwood, Ontario	Checked by : OB

Position : E: 562576, N: 4927292 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value	Elevation Scale (m)
	185.1	GROUND SURFACE					
0		**FILL**, sand, some silt, trace to some gravel, trace clay, compact, brown, moist		1	SS	12	185
1				2	SS	21	184
2				3	SS	25	183
				4	SS	18	
3	182.1						182
	182.0 / 3.1	Inferred bedrock		5	SS	50 / 50mm	



END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

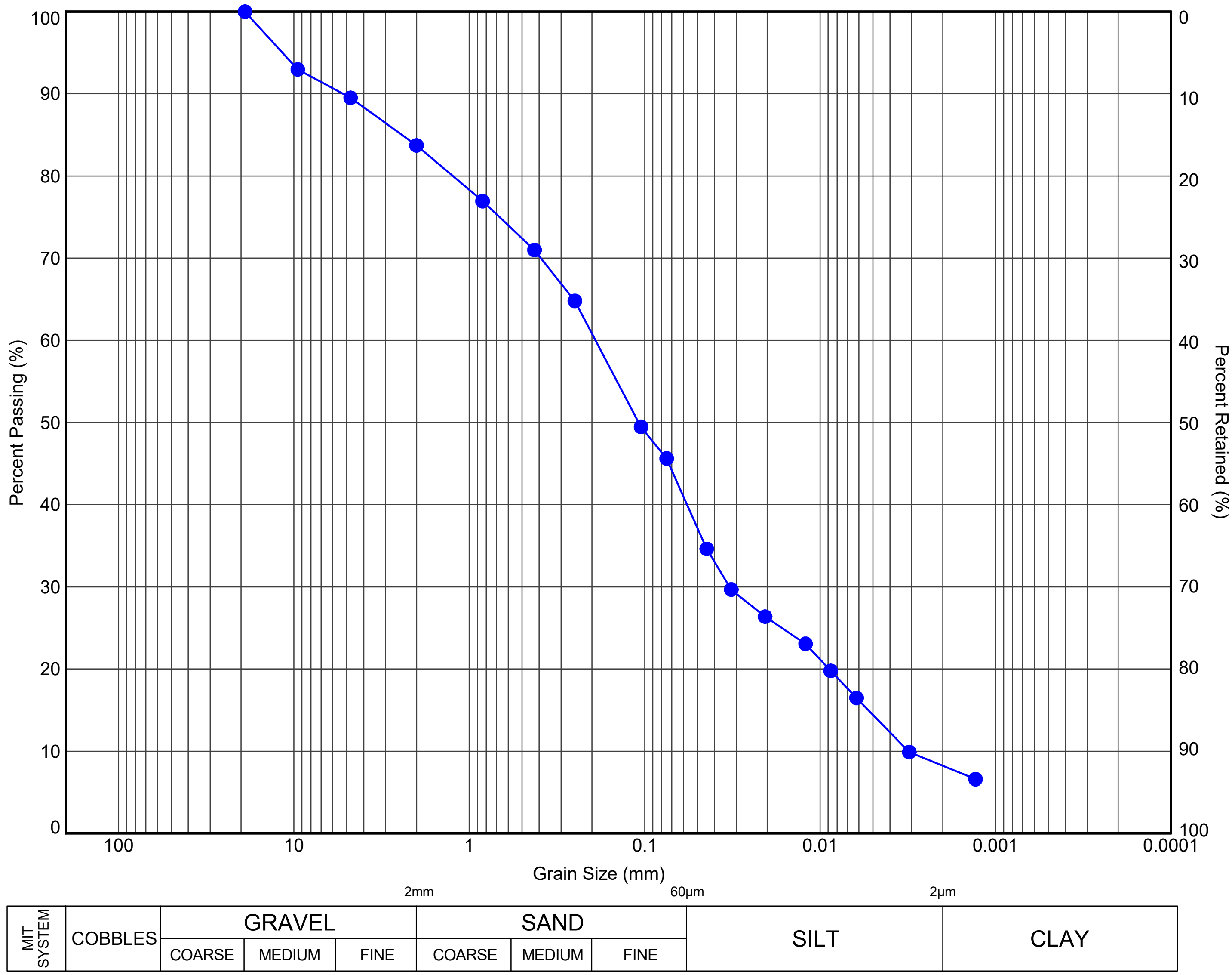
50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 19, 2020	2.0	183.1

APPENDIX B

TERRAPROBE INC.



MIT SYSTEM

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**1**	SS4	2.5	182.3	16	43	33	8	

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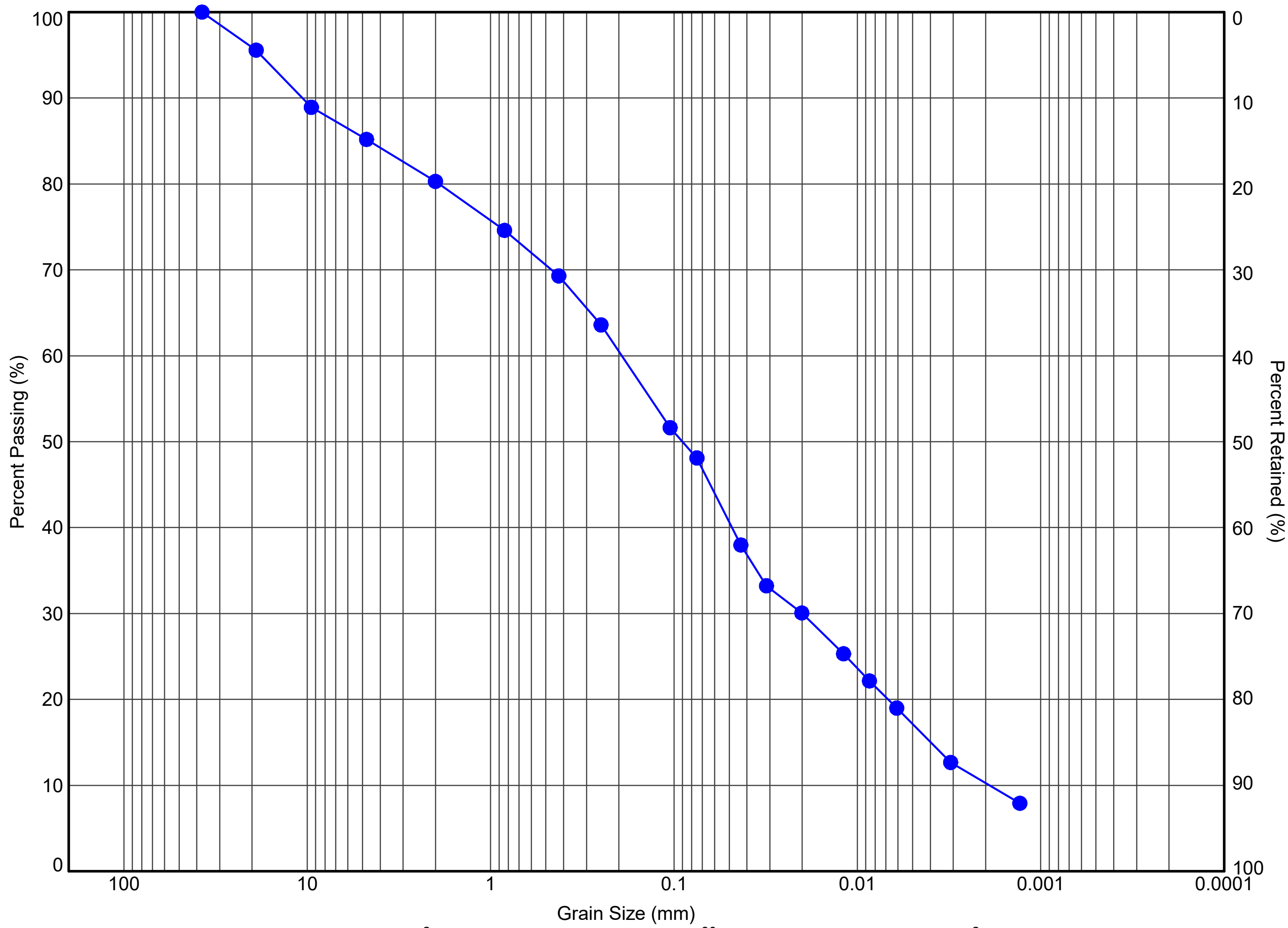
(905) 796-2650

Title:

GRAIN SIZE DISTRIBUTION
SILTY SAND, SOME GRAVEL, TRACE CLAY

File No.: **1-20-0483-01**



		2mm			60µm		2µm	
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, %)*
●	**3**	SS5	3.2	181.9	20	36	34	10	

Terraprobe

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(905) 796-2650

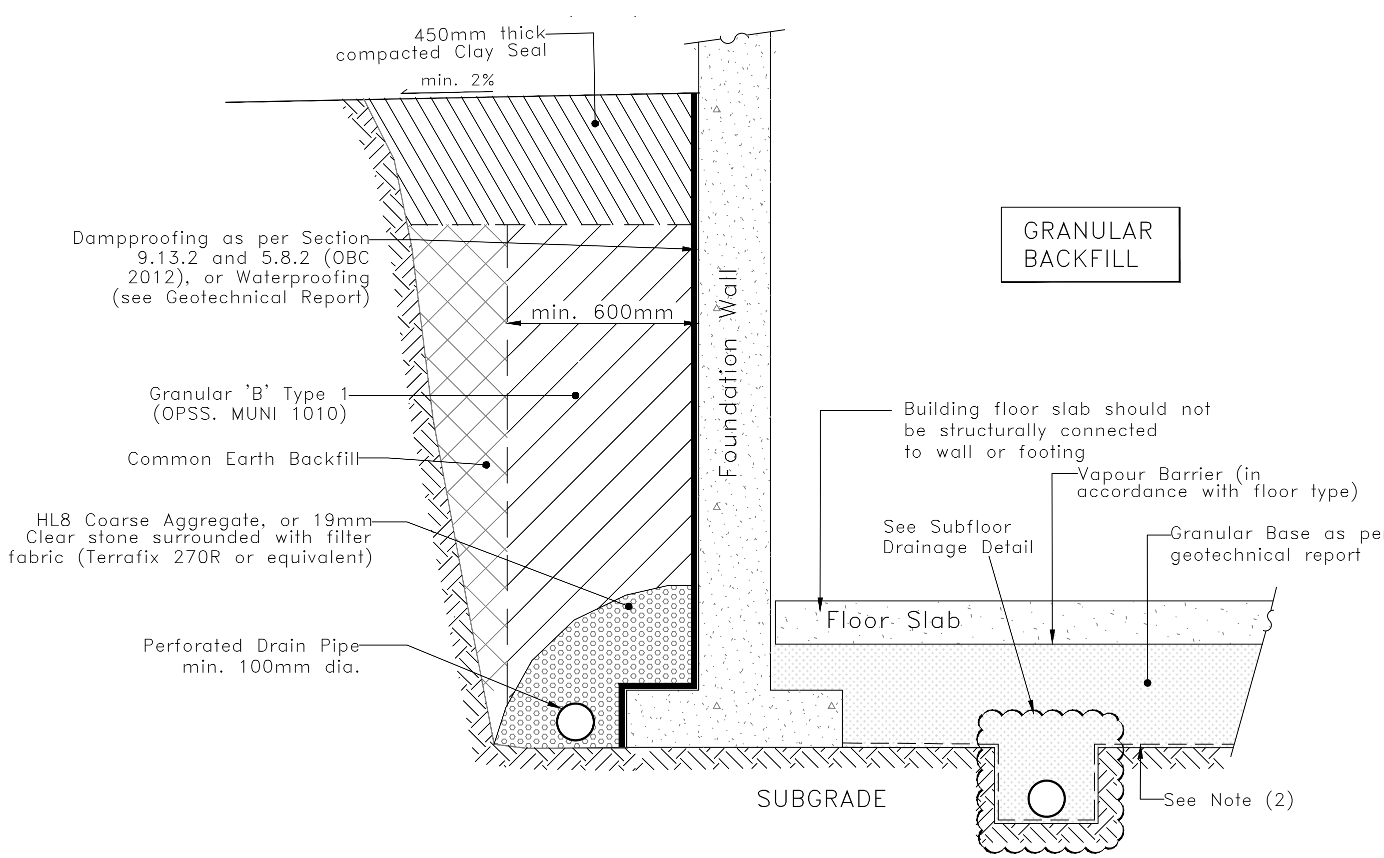
Title:

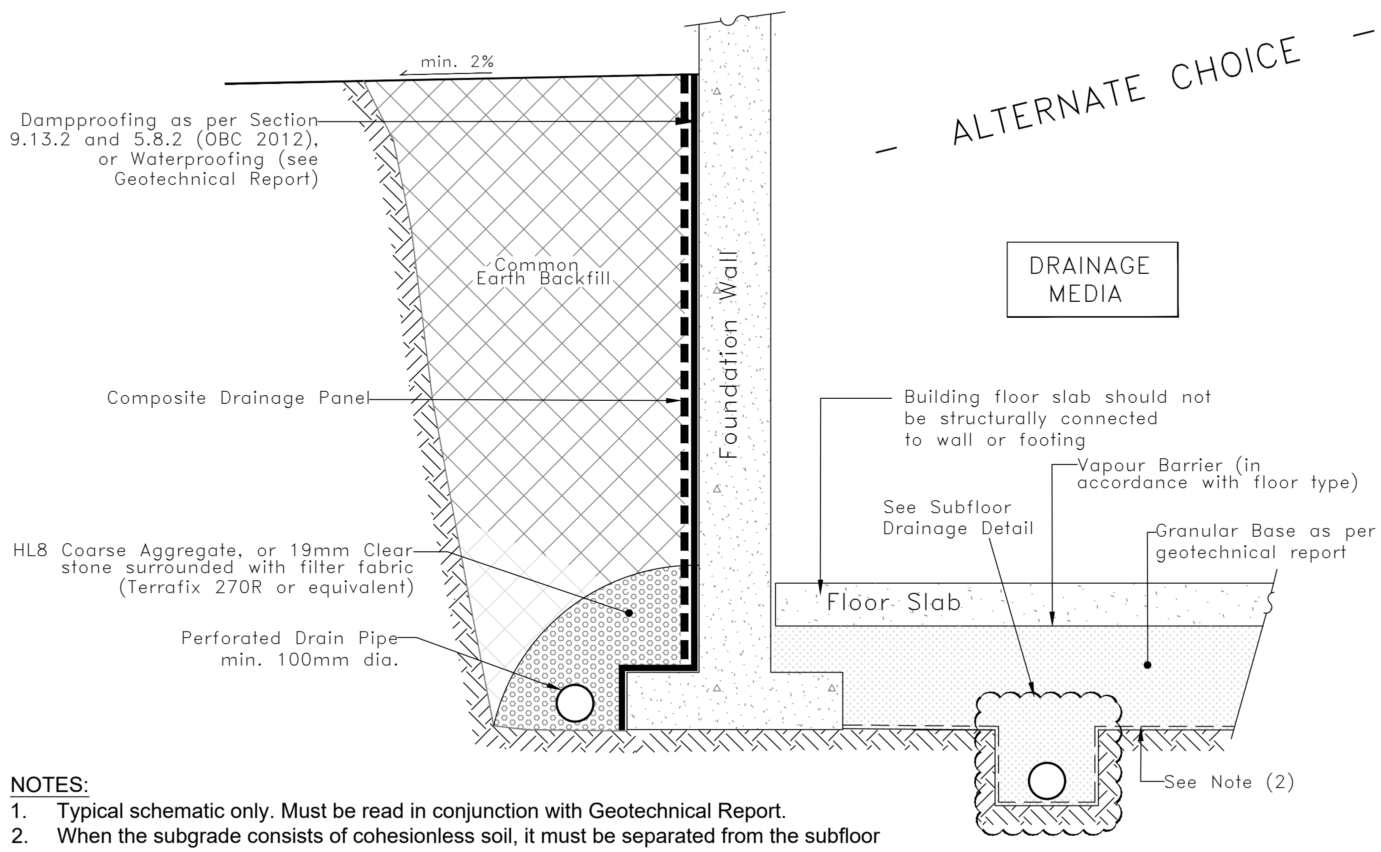
GRAIN SIZE DISTRIBUTION
SILTY SAND, SOME GRAVEL, SOME CLAY

File No.: **1-20-0483-01**

APPENDIX C

TERRAPROBE INC.



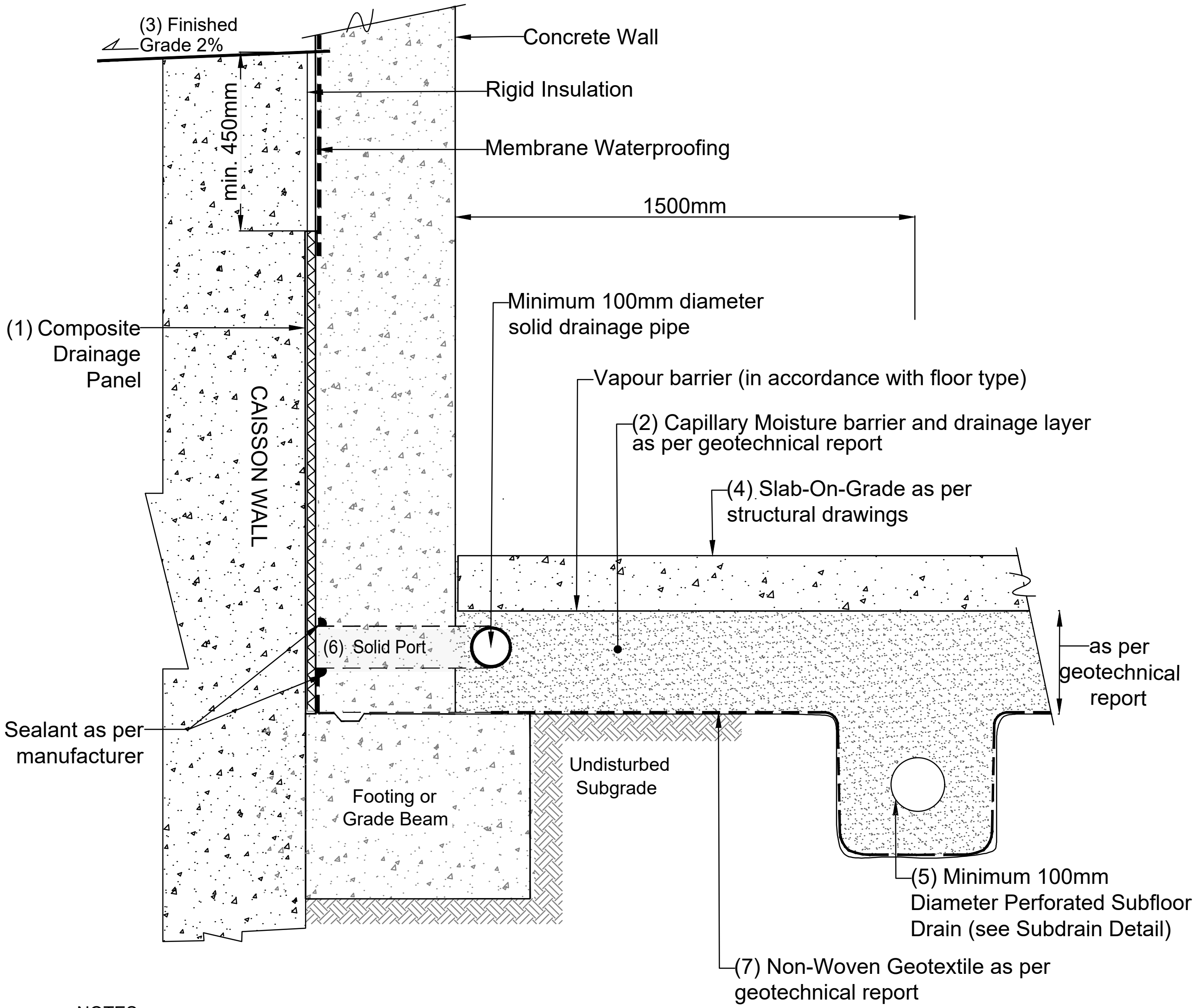


NOTES:
1. Typical schematic only. Must be read in conjunction with Geotechnical Report.
2. When the subgrade consists of cohesionless soil, it must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).
3. Not to Scale

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) 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

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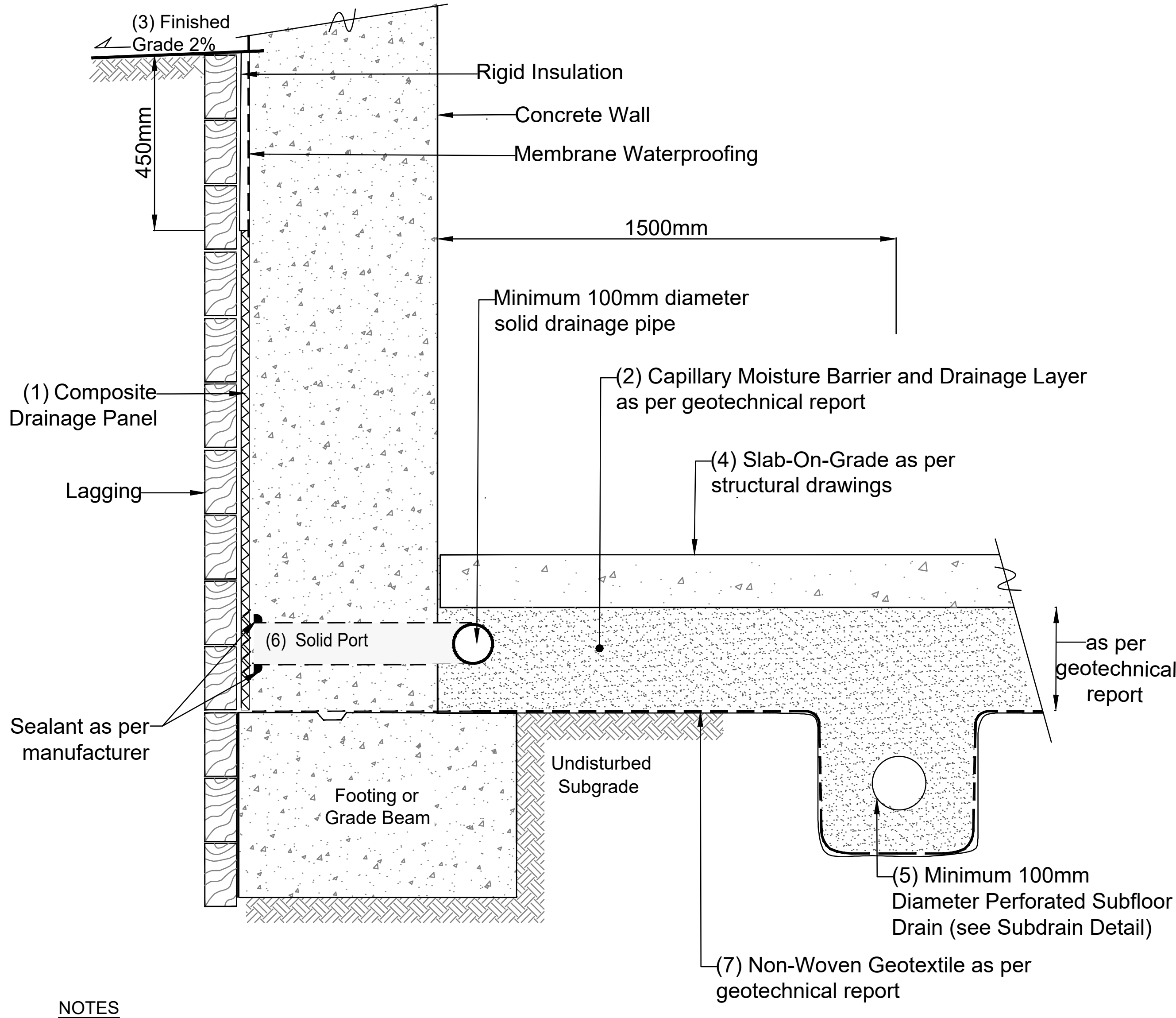
Title:

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.

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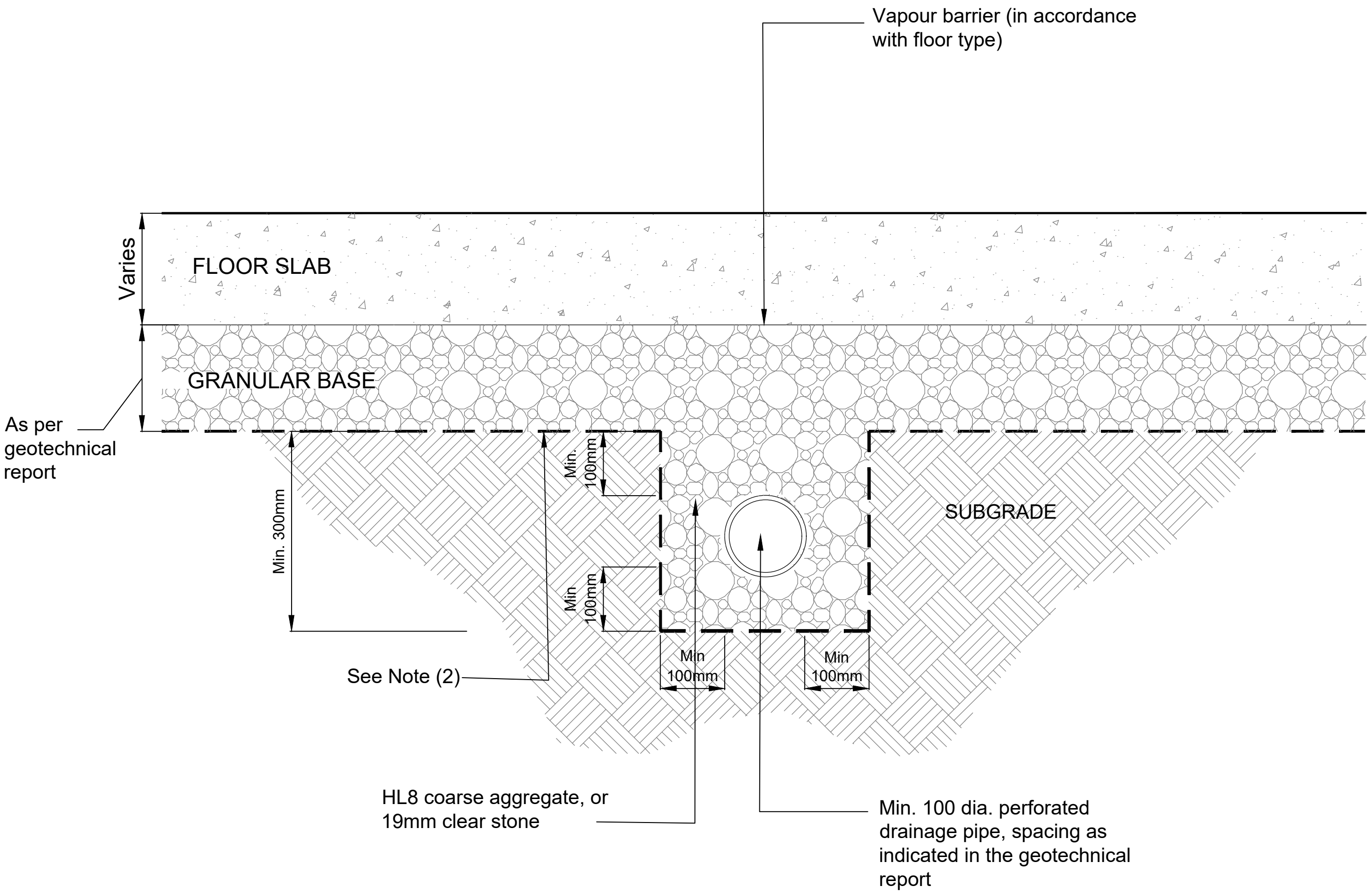
Title:

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

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Title:

TYPICAL BASEMENT SUBDRAIN DETAIL