Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION AND ENGINEERING DESIGN REPORT 441 MAPLE AVENUE BURLINGTON, ONTARIO

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1.0 THE PROJECT

Terraprobe Inc. was retained by Mr. Sam Badawi to conduct a subsurface investigation and provide geotechnical engineering design advice for the proposed redevelopment of municipal address 441 Maple Avenue. The property under consideration in this report is located north of Lakeshore Road and Maple Avenue, in Burlington, Ontario. A site location plan is provided as Figure 1.

The site is currently occupied by a low-rise commercial building. The proposed redevelopment includes demolishing the existing structure and constructing a tower rising from a podium resting on an underground parking structure. The massing options for the proposed redevelopment are as follows:

1. An 11-storey structure resting on a two-level underground parking structure (P2) with an assumed finished floor elevation (FFE) of Elev. 76 ±m (about 7 m below grade).

2. A 17-storey structure resting on a three-level underground parking structure (P3) with an assumed FFE of Elev. 73 (about 10 m below grade).

3. A 24-storey structure resting on a four-level underground parking structure (P4) with an assumed FFE of Elev. 70 ±m (about 13 m below grade).

Regardless of the proposed underground parking structure configuration, the foundations for the proposed structure will be made to bear on sound bedrock, which is at Elev. 77.7 m.

The following drawings were provided to Terraprobe and were reviewed in preparation of this report:

- "441 Maple Avenue, Burlington, Ontario – Massing Options (Podium and Tower) and Schematic Floor Plans", dated October 24, 2017, by Michael Spaziani Architect Inc.

A total of four (4) exploratory boreholes were advanced at the site. The locations of the boreholes are provided on the Borehole Location Plans as Figure 2 (Existing Condition) and Figure 3 (Proposed Condition). A subsurface profile of the boreholes is provided as Figure 4. The results of the individual boreholes are recorded on the Borehole and Rock Core Logs in Appendix A. A summary of the geotechnical laboratory tests is provided in Appendix B. The rock core photographs are provided in Appendix C.

Interpretation, analysis and advice with respect to the geotechnical engineering aspects of the proposed development are provided, based on the information secured from this investigation. The anticipated construction conditions pertaining to foundation design, seismic site classification, slab on grade design, earth pressure design, basement drainage, excavation, shoring design, short term dewatering and other constructability recommendations are discussed.

2.0 SUBSURFACE CONDITIONS

The boreholes were advanced on November 13 and 14, 2017. The borehole elevations and coordinates are provided relative to geodetic datum (NAD 83). The horizontal coordinates are reported relative to the Universal Transverse Mercator geographic coordinate system (UTM Zone 17T).

The subsurface soil, rock and ground water conditions encountered in the boreholes are presented on the attached Borehole and Rock Core Logs in Appendix A. The stratigraphic boundaries indicated on the geotechnical Borehole Logs are inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil or rock type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed in a series of widely spaced boreholes, and will vary between and beyond the borehole locations. The discussion has been simplified in terms of the major soil and rock strata for the purposes of geotechnical design.

Asphaltic concrete material and aggregate thicknesses provided in the report were obtained at individual borehole locations, as measured through the collar of the open borehole. Thicknesses may vary between and beyond borehole locations.

2.1 Stratigraphy

The following stratigraphy is based on the borehole findings, as well as the geotechnical laboratory testing conducted on selected representative soil samples. The summary elevations below are provided for general guidance only. Detailed depths are given in the following subsections. In general, three main stratigraphic units were encountered on site as follows:

1. Pavement structure underlain by a layer of earth fill (about 1.3 m thick on average)
2. Native soils which primarily consists of silt and residual soils (about 1.6 m thick on average)
3. Bedrock of the Queenston Formation at Elev. 79 ±m.

There is ground water within the native soils (low permeability soils which will preclude the free flow of water) at about 3.5 ±m below existing grade (about Elev. 79.5± m).

2.1.1 Pavement Structure and Earth Fill

The boreholes encountered a pavement structure at grade consisting of 50 to 75 mm of asphaltic concrete underlain by 50 to 325 mm of aggregate. Underlying the pavement structure, the boreholes encountered a layer of earth fill extending to depths of 0.8 to 2.3 m below grade (Elev. 80.6 to 81.8 m). The earth fill varies in composition from sand to clayey silt, and generally includes trace amounts of deleterious materials (asphalt and brick fragments, cinders and organics). Due to the variation and inconsistent

placement of the earth fill material, the relative density/consistency of the earth fill varies but is on average compact/stiff.

2.1.2 Native Soils

Underlying the earth fill, the boreholes encountered undisturbed native soils consisting of silts and residual soils at a depth of 0.8 to 2.3 m below grade (Elev. 80.6 to 81.8 m), extending to depths of 2.3 to 4.5 m below grade (Elev. 78.6 to 80.0 m). Borehole 1 encountered a zone of native sand at a depth of 2.3 m below grade (Elev. 80.8 m) overlying the native silt at a depth of 2.6 m below grade (Elev. 80.5 m). The silts encountered in Boreholes 1, 2 and 4, contain trace to some clay and sand, trace gravel, and vary in colour from reddish brown to reddish grey, and are moist. The residual soils (likely fully weathered bedrock derived from the underlying Queenston Formation) encountered in Boreholes 2 and 3 consists primarily of silt, contain trace amounts of clay, sand, gravel and shale fragments, and are consistently red and moist. The Standard Penetration Test (SPT) results (N-Values) measured in the native soils ranged from 16 to 60 blows per 300 mm of penetration, indicating a compact to very dense relative density (on average dense).

2.1.3 Bedrock

Underlying the native soils, bedrock of the Queenston Formation was confirmed in Borehole 1 at a depth of 4.5 m below existing grade (Elev. 78.6 m) and inferred in the remaining boreholes at depths of 2.3 to 4.3 m below existing grade (Elev. 78.8 to 80.0 m). The boreholes were terminated within the bedrock at depths of 4.3 to 17.3 m below existing grade (Elev. 65.8 to 78.8 m). A total of about 12.3 m of rock core was recovered from Borehole 1. A detailed rock core log is included in Appendix A, and the photographs of the core are included in Appendix C.

The bedrock of the Queenston Formation is a deposit predominantly consisting of thickly bedded to massive red shale of Ordovician age. The formation contains interbeds of light grey / green calcareous shale, siltstone, sandstone, and limestone/dolostone which are discontinuous and nominally 50 to 300 mm thick. The shale is a weak strength rock, whereas the harder calcareous and limestone/dolostone beds are considered medium strength rock and are considered to be hard layers. The percentages of limestone (hard layers) are shown on the rock core log and ranged from.0 to 12 % limestone per core run.

A summary of properties with respect to the shale within the Queenston Formation was presented in the Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects* (March 1983), as follows:

	Uniaxial Compressive Strength (MPa)	**Young's Modulus (GPa)**	**Poisson's Ratio**
Average	8.7	1.3	0.32

	Uniaxial Compressive Strength (MPa)	Young's Modulus (GPa)	Poisson's Ratio
Range	7.2 to 9.6	0.5 to 2.3	0.28 to 0.35

There is typically a zone of weathering at the contact between the rock of the Queenston Formation and the soil overburden. In the Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects*, there is reproduced from Skempton, Davis and Chandler, a *typical weathering profile of a low durability shale*, that characterizes the shale surface into three grades of weathering and four zones described as follows:

	Zone	Description	Notes
Fully Weathered	IVb	soil like matrix only	indistinguishable from glacial drift deposits, slightly clayey, may be fissured
Partially Weathered	IVa	soil like matrix with occasional pellets of shale less than 3 mm dia.	little or no trace of rock structure, although matrix may contain relic fissures
	III	soil like matrix with frequent angular shale particles up to 25 mm dia.	moisture content of matrix greater than the shale particles
	II	angular blocks of unweathered shale with virtually no matrix separated by weaker chemically weathered but intact shale	spheroidal chemical weathering of shale pieces emanating from relic joints and fissures, and bedding planes
Unweathered (Sound)	I	shale	regular fissuring

In Borehole 1, fully weathered to partially weathered bedrock was observed to be approximately 0.9 m thick, and the transition to sound/unweathered (Zone I) bedrock was encountered at a depth of 5.4 m below existing grade (Elev. 77.7 m). A summary of the bedrock elevations is tabulated as follows:

Borehole	Ground Surface Elev. (m)	Top of Weathered Bedrock Depth / Elev. (m)	Top of Sound (Zone I) Bedrock Depth / Elev. (m)
1	83.1	4.5 / 78.6	5.4 / 77.7
2	83.1	4.3 / 78.8*	**
3	82.3	2.3 / 80.0*	**
4	82.6	3.0 / 79.6*	**

*Rock coring not conducted; bedrock elevation inferred by split-spoon samples and auger refusal.

**Rock coring not conducted; depth/elevation of sound bedrock unknown.

Rock Quality Designation (RQD) refers to the total length of those pieces of sound core which are 100 mm or greater in length in a core run, expressed as a percentage of the total length of that core run. Sound pieces of rock are those pieces separated by natural fractures or bedding, and not machine induced or subsequent artificial breaks. The RQD of the rock core recovered from the sound (Zone I) bedrock ranges from 62 to 97 %. While the RQD for the runs in the sound bedrock can be on the lower side, it is not indicative of the actual good strength and bearing capacity of the Queenston Formation. Since the

bedding is horizontal in this formation, and there are many bedding layers within this rock mass, the RQD is typically low and is a function of the regular repeating bedding planes, not of stress induced fractures leading to a weaker rock mass. The results of this investigation in this regard are typical of the deposit.

2.2 Ground Water

Borehole 1 was filled with water (due to rock coring) on completion and measuring the unstabilized ground water level after drilling was not practical. Unstabilized ground water level and depth to cave observations were made in the remaining boreholes after drilling, as noted on the borehole logs. A total of five (5) 50 mm diameter monitoring wells were installed in the boreholes to facilitate long-term ground water monitoring (a nested shallow well was installed adjacent to Borehole 1). Stabilized ground water level measurements were made in the monitoring wells at least one week after the completion of drilling. The ground water measurements are shown on the Borehole Logs and are summarized as follows.

Borehole No.	Depth of borehole (m)	On completion (m)		Strata Screened	Water Depth / Elevation (m)
		Unstabilized Water Level	Depth to Cave		May 17, 2017
1	17.3	Not Measured	Not Measured	Bedrock (Elev. 65.8 to 68.9 m)	4.9 / 78.2
	4.0	Dry	Open	Earth Fill and Native Soils (Elev. 79.0 to 82.1 m)	2.2 / 80.9
2	4.3	4.0	4.1	Earth Fill and Native Soils (Elev. 79 to 82.1 m)	3.7 / 79.4
3	4.6	4.1	4.3	Earth Fill and Native Soils (Elev. 78.0 to 81.1 m)	3.4 / 78.9
4	4.6	Dry	4.3	Native Soils and Inferred Bedrock (Elev. 78.3 to 81.4 m)	3.7 / 78.9

For design purposes, the stabilized ground water table is at Elev. 79.5± m, in the native soils. The stabilized ground water level in the Queenston Formation is at Elev. 78 ±m. In general, the excavation for the proposed underground structure (P2, P3 or P4 configuration with a FFE at Elev. 76, 73 or 70 ±m, respectively) will extend below the stabilized ground water table at Elev. 79.5 ±m.

The cohesionless earth fill and native sand are cohesionless and considered high permeability materials, which will allow for the free flow of water. The native soils contain a high percentage of fines and are considered low permeability materials, which will preclude the free flow of water when wet. The sound bedrock is considered a low permeability material. Some flow is to be expected in the fractures in the bedrock, predominately in the weathered zones.

Construction dewatering at adjacent sites, existing building drains or dewatering systems, and seasonal fluctuations may cause significant changes to the depth of the ground water table over time.

3.0 GEOTECHNICAL ENGINEERING DESIGN

The following discussion and recommendations are based on the factual data obtained from this investigation, and are intended for 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 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.

3.1 Foundation Design Parameters

The proposed redevelopment includes demolishing the existing structure and constructing a tower rising from a podium resting on an underground parking structure. The massing options for the proposed redevelopment are as follows:

1. An 11-storey structure resting on a two-level underground parking structure (P2) with an assumed finished floor elevation (FFE) of Elev. 76 ±m (about 7 m below grade).

2. A 17-storey structure resting on a three-level underground parking structure (P3) with an assumed FFE of Elev. 73 (about 10 m below grade).

3. A 24-storey structure resting on a four-level underground parking structure (P4) with an assumed FFE of Elev. 70 ±m (about 13 m below grade).

Regardless of the proposed underground parking structure configuration, the foundations for the proposed structure will be made to bear on sound bedrock below Elev. 77.7 m.

Conventional spread footing foundations made to bear on sound Queenston Formation bedrock below Elev. 77.7 m and down to Elev. 67 ±m may be designed using a maximum factored geotechnical resistance at ultimate limit state (ULS) of 5,000 kPa. The maximum net geotechnical reaction at serviceability limit state (SLS) should be limited to 4000 kPa. The SLS bearing is a function of acceptable total and differential settlement. The settlement of foundations made on the sound bedrock is elastic, linear and non-recoverable. The settlement occurs as load is applied. There have been a number of load tests carried out in the sound Queenston Formation that have indicated that the rock formation has predictable and similar response to loading over its area of occurrence. These tests have yielded parameters to estimate the elastic compression of the rock under applied loading. This compression is a

function of the pressure applied and the size of the area loaded. To estimate the settlement of foundations of different sizes and assess differential settlement between foundation units the following relationship can be used.

$$\delta = 1000 q_{SLS} \left[\frac{2}{1+0.4/B_f}\right]^2 \frac{1}{k}$$

where: $\boldsymbol{\delta}$ = estimated vertical displacement in the rock beneath the centre of the loaded foundation (mm)

$\boldsymbol{q_{SLS}}$ = applied SLS bearing pressure on the rock at the base of the foundation (kPa)

$\boldsymbol{B_f}$ = the nominal foundation width (m)

$\boldsymbol{k}$ = modulus of displacement (kPa/m): 600,000 kPa/m for sound bedrock.

Footings stepped from one level to another must be at a slope not exceeding 7 vertical to 10 horizontal. The design earth cover for frost protection of foundations exposed to ambient environmental temperatures is 1.2 metres in the Greater Toronto area. Experience suggests that the temperature in "unheated" underground parking levels two or more levels below grade with normal ventilation provisions is not as severe as the ambient open air condition. Certainly, the earth cover required to prevent frost effects on foundations in the lower parking levels need not be any greater than 1.2 metres, and experience in a number of structures has shown that perimeter foundations provided with 600 mm of cover perform adequately as do interior isolated foundations with 900 mm of cover. At locations adjacent to ventilation shafts, it is normal practice to provide insulation to ensure that foundations are not affected by the cold air flow.

Prior to pouring concrete for the footings, the footing subgrade must be cleaned of all deleterious materials such as softened, disturbed, or caved material, as well as any weathered bedrock or standing water. Zones of rubblized bedrock encountered at the footing elevation must be sub-excavated, and foundations must be made to bear on sound bedrock. Terraprobe must inspect and approve the proposed footing subgrades to confirm sound bedrock is encountered at the founding elevations. If construction proceeds during freezing weather conditions, temporary frost protection for the footing bases and concrete must be provided. The bedrock surface can weather and deteriorate on exposure to the atmosphere or surface water. Therefore, foundation bases that will remain open and exposed for an extended period of time should be protected by applying a skim coat of lean concrete.

3.2 Earthquake Design Parameters

The Ontario Building Code (2012) 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 4.1.8.4A of the Ontario Building Code (2012). 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**

For spread footings resting uniformly on sound (unweathered) bedrock, it is recommended that the site designation for seismic analysis is Class B, as per Table 4.1.8.4.A of the Ontario Building Code (2012). 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				
	$S_a(0.1) \leq 0.1$	$S_a(0.1) = 0.2$	$S_a(0.1) = 0.3$	$S_a(0.1) = 0.4$	$S_a(0.1) \geq 0.5$
B	0.6	0.7	0.7	0.8	0.8

3.3 Earth Pressure Design Parameters

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

Stratum/Parameter	γ	φ	K_a	K_o	K_p
Compact Granular Fill Granular 'B' (OPSS 1010)	21	32	0.31	0.47	3.26
Existing Earth Fill	19	29	0.35	0.52	2.88
Native Sands and Silts	20	32	0.31	0.47	3.26

Residual Soils	21	29	0.35	0.52	2.88
Queenston Formation (Bedrock)	28	26	n/a	n/a	n/a

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

The above earth pressure parameters pertain to a horizontal grade condition behind a retaining structure. Values of earth pressure parameters for an inclined retained grade condition will vary.

Walls 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 at depth, **h** (m)
K = the earth pressure coefficient
$\mathbf{h_w}$ = the depth below the groundwater level (m)
γ = the bulk unit weight of soil, (kN/m^3)
γ**'** = the submerged unit weight of the exterior soil, (γ - 9.8 kN/m^3)
q = the complete surcharge loading (kPa)

The above equation pertains to a horizontal grade condition behind a retaining structure. Values of earth pressure against retaining structures for an inclined retaining grade condition will vary.

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall that would otherwise act in conjunction with the earth pressure, this equation can be simplified to:

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

To ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure, where the structure is made directly against a shored excavation, drainage is provided by forming a drained cavity with prefabricated drain core material covering the excavation face and designed to discharge collected water into a perimeter/underfloor drainage system. Where the structure is built by open cut excavation methods, this equation assumes that free-draining granular backfill such as Granular 'B' (OPSS 1010) is used and effective drainage is provided.

Consideration must also be given to the possible effects of frost on structures retaining earth. Pressures induced by freezing in frost-susceptible soils exert pressures and are effectively irresistible.

3.3.1 Rock Pressure

The empirical approach for the design of foundation walls below bedrock level has been to use a uniform pressure distribution for the design of the basement walls below the top of bedrock elevation, which is consistent with the maximum earth pressure calculated for the lowest level of soil in the profile. This approach is likely conservative but it recognizes the practical requirement to have a foundation wall of a consistent width through the lower reach of the building.

Deeper than 2 m into the bedrock (i.e. below Elev. 76.6 ±m), this approach does not recognize the potential for pressures on the basement wall due to time dependant rock swell that results when locked in horizontal stresses are released. It presupposes that there is sufficient time between cutting of the rock face and construction of the building structure to allow the rock to de-stress and swell. Experience suggests that if there is a 120-day period after the rock cut, before the rock is restrained by the structure, that there has been sufficient swell and no significant stresses are imposed on the structural wall. Depending on the building construction sequence some provision for compressible material at the foundation perimeter in rock may be necessary. In a conventional excavation and construction progression it should be expected that there will be a crushable layer provided behind the lowest one or two parking level walls. Generally by the time the lowest level wall and first suspended slab is cast there has been sufficient time for the other exposed rock to de-stress and swell.

Where pits are made for sumps and elevators or other such features which are incorporated within the major excavation, there must be careful consideration of the potential for rock squeeze effects if the pits are to be cast directly against the rock face. For such structures, a crushable layer can be placed between the rock and the concrete. Ethafoam is typically used in this application and the walls are designed for 25% to 50% compressive strength of the foam. At 50% compression 220 Ethafoam plank material will provide a resistance of 18 psi (124 kPa). At 25% compression 220 Ethafoam plank material will provide a resistance of 9 psi (65 kPa). The 10% crush compressive strength of this material is 7 psi (50 kPa), which will allow for concrete placement. Alternatively, if the rock is over excavated by at least 600 mm and the pits and sumps are backfilled with 19 mm clear stone (OPSS 1004), then there is sufficient give in the backfill to accommodate the rock swell.

Rock squeeze effects are not relevant to spread footing foundation excavations as the foundation concrete strength exceeds any rock squeeze pressures.

3.3.2 Sliding Resistance

The factored geotechnical resistance to sliding of earth retaining structures is developed by friction between the base of the footing and the rock. This friction **(R)** depends on the normal load on the rock contact **(N)** and the frictional resistance of the rock **(tan φ)** expressed as: $\mathbf{R = N \tan \varphi}$. This is an unfactored resistance. The factored resistance at ULS is $\mathbf{R_f = 0.8\ N \tan \varphi}$.

3.4 Slab on Grade Design Parameters

The lowest parking level slab (P2, P3 or P4 configuration with a FFE at Elev. 76, 73 or 70 ±m, respectively) is to be made on sound bedrock of the Queenston Formation, which is suitable for the support of a slab. The modulus of subgrade reaction appropriate for design of the slab resting on sound bedrock is 80,000 kPa/m.

Use of excavated bedrock spoil to restore subgrade elevations is to be specifically prohibited. The excavated bedrock spoil cannot be adequately compacted to provide support for the slab on grade. All excavated bedrock is not reusable below any settlement sensitive areas.

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 300 mm layer of 19 mm clear stone (OPSS 1004) compacted by vibration to a dense state. Basement drainage is required as discussed in Section 3.5.

3.5 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 % gradient or more, for a distance of at least 1.2 m. 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 of 9 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 D. It is recommended to cut the rock subgrade neat to 300 mm beneath the floor slab and place subdrains directly on the subgrade. The subfloor drainage layer is then comprised of 300 mm of 19 mm clear stone (OPSS 1004).

Prefabricated drainage composites, such as Miradrain 2000 (Mirafi) or Terradrain 200 (Terrafix), should be incorporated between shoring walls or rock face and the cast-in-place concrete foundation wall to make a drained cavity. Drainage from the cavity must be collected at the base of the wall in non-perforated pipes and conveyed directly to the sumps. The flow to the building sump from the subsurface drainage will be governed largely by the building perimeter drainage collection during rainfall and runoff events. Typical shored and open cut excavation drainage details are provided in Appendix D.

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 ground seepage and storm event flows. It is expected that the seepage can be controlled with typical widely available commercial sump pumps.

4.0 DESIGN CONSIDERATIONS FOR CONSTRUCTABILITY

4.1 Excavations

Excavations must be carried out in accordance with the Occupational Health and Safety Act, Ontario Regulation 213/91 (as amended), Construction Projects, Part III – Excavations, Sections 222 through 242. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. For practical purposes, the earth fill and native soils are considered Type 3 soils.

Where workers 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 maximum 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.

Bedrock of the Queenston Formation is not considered a soil under the Act. 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 Terraprobe 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). If deemed necessary, rock bolts can be used to anchor a layer of protective mesh that will protect workers from loose rock spalling from the face of excavation.

The overburden soils can be removed by conventional excavation equipment. Excavations at this site may encounter construction debris, deleterious materials and other obstructions in the earth fill and

cobble/boulder obstructions in the native soils. Fragments of shale and limestone are also likely to be encountered in the transition zone between the native soil and the bedrock. The size and distribution of cobbles/boulders/obstructions or bedrock fragments cannot be predicted with boreholes, as the sampler size is insufficient to secure representative particles of this size. The bedrock below the site, while predominantly shale, contains harder beds. It is likely that some thick layers as much as 600 mm of hard limestone/dolostone may be encountered. The risk and responsibility for the removal and disposal of cobbles/boulders/obstructions, and the removal or penetration of these harder layers must be addressed in the contract documents for foundations, excavations and shoring contractors.

The Queenston Formation is a rippable rock that can be removed with conventional excavation equipment once it has been displaced by a ripper tooth or a hoe ram. The hard layers of limestone/dolostone within the shale formation are normally broken with hoe mounted hydraulic rams before excavation. The percentage of hard layers per core run is indicated on the rock core log. Excavating detailed shapes for foundations and the edges of the excavation are normally accomplished with hoe mounted hydraulic rams. Excavated bedrock cannot be used as compacted backfill in any settlement sensitive areas.

Where a harder layer coincides with the foundation level, it may be necessary to remove the entire thickness of the hard layer to expose the founding level. It is virtually impossible to remove a portion of one of these layers. This can result in 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 foundation and excavation contractors.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the exposed rock in the foundation excavations is required. The rock beneath this site is susceptible to frost damage. Consideration must be given to frost effects, such as heave or softening, on exposed surfaces in the context of this particular project development. If foundation construction proceeds during the hot summer months, extreme heat has also been found to cause relatively rapid degradation of exposed rock surfaces. Depending on the weather at the time of construction it could be necessary to make final rock cuts for foundations and immediately seal these cuts with a concrete skim coat to preserve the integrity of the bearing surface.

4.2 Ground Water Control

For design purposes, the stabilized ground water table is at Elev. 79.5± m, in the native soils. The stabilized ground water level in the Queenston Formation is at Elev. 78 ±m. In general, the excavation for the proposed underground structure (P2, P3 or P4 configuration with a FFE at Elev. 76, 73 or 70 ±m, respectively) will extend below the stabilized ground water table at Elev. 79.5 ±m.

The cohesionless earth fill and native sand are cohesionless and considered high permeability materials, which will allow for the free flow of water. The native soils contain a high percentage of fines and are considered low permeability materials, which will preclude the free flow of water when wet. The sound bedrock is considered a low permeability material. Some flow is to be expected in the fractures in the bedrock, predominately in the weathered zones. In general, the volume of water anticipated to flow into the open excavation is such that temporary pumping from the excavation using a conventional sump pump arrangement is expected to suffice for the control of seepage.

4.3 Earth-Retention Shoring Systems

The site is bounded by existing parking lots and commercial buildings to the north, east and west, and Maple Avenue to the south. 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.

If the shoring system is to be used to provide ground water control, the entire excavation would be constructed using a continuous interlocking caisson wall shoring, cut off into the sound bedrock. The caisson fillers need only extend to the level where the bedrock is intact, self-supporting and relatively free of fractures that make significant volumes of water.

Exposed rock faces will weather and deteriorate. It may be necessary to provide draped steel mesh over the excavation faces to protect workmen beneath the rock faces from spalls. The mesh directs small rock spalls down the face and precludes toppling of any significant size pieces of material.

The shoring system would best be supported by pre-stressed soil anchors extending beneath the adjacent lands and municipal roads. 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.

4.3.1 Lateral 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. For the cohesionless soil, the multi-level supported shoring can be designed based on an earth pressure distribution consisting of a rectangular pressure distribution with a maximum pressure defined by:

$$\boldsymbol{P = 0.65\, K[\gamma H + q]}$$

where, P = the horizontal pressure at depth, **H** (kPa)
K = the earth pressure coefficient (see Section 3.3)
H = the total depth of the excavation (m)
γ = the bulk unit weight of soil, (kN/m3)
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.

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. 79.5 ±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.

The bedrock induces no pressure on shoring systems. The requirement for lagging support of partially weathered rock depends on the cleanliness of the excavation break.

4.3.2 Caisson and Soldier Pile Toe Embedment

Caisson and soldier pile toes should made in bedrock of the Queenston Formation. The factored ULS vertical bearing capacity for the design of a pile embedded in the sound bedrock is 5 MPa. The factored lateral capacity at ULS of the sound rock is 1 MPa.

The soils at this site are sufficiently cohesionless such that augered borings made into these soils will be unstable. It is necessary to advance temporarily cased holes to the bedrock surface 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.

The exposed Queenston Formation deteriorates with time. Exposed excavation faces have been found to flake and recede as much as 300 mm with 12 months exposure. This recession generally takes the form of coin-size shale particles dropping from the face on a constant basis. The deteriorated rock loses internal integrity and bearing capability. Typically, the piles advanced as part of the shoring wall are advanced at least 1 m below the base of the excavation to accommodate this weathering, to ensure the lateral and vertical capacities provided can be utilized.

4.3.3 Lateral Bracing Elements

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. The use of anchors on adjacent properties requires the consent of the adjacent land owners, expressed in encroachment agreements.

Where the excavation penetrates the bedrock, the rock excavation is nominally self-supporting in a vertical face, provided the rock bedding is horizontally oriented. Conventional anchors made in bedrock of the Queenston Formation can be designed using a working adhesion of 550 kPa. One or more prototype anchors must be performance-tested to 200% of the design load to demonstrate the anchor capacity and validate design assumptions. Given the potential variability in rock conditions and/or installation quality, all production anchors must also be proof-tested to 133% of the design load.

The sound bedrock below Elev. 77.7 ±m is suitable for the placement of raker foundations. Raker footings established on the sound bedrock at an inclination of 45 degrees can be designed for a maximum geotechnical resistance at ULS of 1 MPa.

4.4 Site Work

The earth fill, native soils and bedrock at this site will become disturbed and may lose their integrity to support when subjected to traffic, particularly when wet. It can be expected that a subgrade made in the native soils will be disturbed unless an adequate granular working surface is provided to protect the integrity of the subgrade soils from construction traffic, especially during periods of wet weather. Subgrade preparation works cannot be adequately accomplished during wet weather and the project must be scheduled accordingly. The disturbance caused by the traffic can result in the removal of disturbed soil and use of granular fill material for site restoration or underfloor fill that is not intrinsic to the project requirements.

The most severe loading conditions on the subgrade may occur during construction. Consequently, special provisions such as end dumping and forward spreading of earth and aggregate fills, restricted construction lanes, and half-loads during placement of the granular base and other work may be required, especially if construction is carried out during unfavourable weather.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the founding subgrade and concrete must be provided. The soil and bedrock at this site are susceptible to frost damage. Consideration must be given to frost effects, such as heave or softening, on exposed soil and bedrock surfaces in the context of this particular project development. If foundation construction proceeds during the hot summer months, extreme heat has also been found to cause relatively rapid degradation of exposed rock surfaces. Depending on the weather at the time of construction it could be necessary to

make final rock cuts for foundations and immediately seal these cuts with a concrete skim coat to preserve the integrity of the bearing surface.

4.5 Quality Control

Prior to the start of excavation, it is advisable to complete visual pre-construction condition surveys of adjacent lands and buildings. These surveys document the baseline condition at the start of construction for the adjudication of any damage claims related to the works. Even a well-executed shoring system designed with appropriate parameters cannot preclude all movements, and depending on the existing condition of the adjacent structure, small changes in stress or soil volume can cause displacements or cracking. The City of Burlington may require, as a condition of the excavation permit, that the shoring performance be monitored as excavation proceeds. Terraprobe provides shoring monitoring services.

The proposed structure will be founded on conventional spread footings. The foundation installations must be reviewed in the field by Terraprobe, the geotechnical engineer, as they are constructed. The on-site review of the condition of the foundation subgrade 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 (2012). 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 contained in this report.

The long term performance of the slab on grade is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved as much as practically possible. The design advice in this report is based on an assessment of the subgrade support capabilities as indicated by the boreholes. These conditions may vary across the site depending on the final design grades and therefore, the preparation of the subgrade and the compaction of all fill should be monitored by Terraprobe at the time of construction to confirm material quality, thickness, and to ensure adequate compaction.

The requirements for fill placement on this project have been stipulated relative to Standard Proctor Maximum Dry Density (SPMDD). In situ determinations of density during fill placement on site are required to demonstrate that the specified placement density is achieved. Terraprobe is a CNSC certified operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary, with our qualified technical staff.

Concrete 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.

Terraprobe staff can also provide quality control services for Building Envelope, Roofing and Structural Steel, 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.

5.0 LIMITATIONS AND USE OF REPORT

5.1 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 from this investigation.

The drilling work was carried out by a drilling contractor and was observed and recorded by Terraprobe on a full time basis. The boreholes were made by a continuous flight power auger machine using solid stem augers and HQ rock coring. The Terraprobe technician logged the boreholes and examined the samples as they were obtained. The samples obtained were sealed in clean, air-tight containers or placed in rock core boxes and transferred to the Terraprobe laboratory, where they were reviewed for consistency of description by a geotechnical engineer. Ground water monitoring wells were installed in the boreholes to measure long-term ground water levels.

The samples of the strata penetrated were obtained using the Split-Barrel Method technique (ASTM D1586). The samples were taken at intervals. The conventional interval sampling procedure used for this investigation does not recover continuous samples of soil at any borehole location. There is consequently some interpolation of the borehole layering between samples and indications of changes in stratigraphy as shown on the borehole logs are approximate.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. 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.

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 and geotechnical advice to completely identify all aspects of the site and works that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project must 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, and their approach to the construction works, cognizant of the risks implicit in the subsurface investigation activities.

5.2 Changes in Site and Scope

It must 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. In particular, caution should be exercised in the consideration of contractual responsibilities as they relate to control of seepage, disturbance of soils or rock, and frost protection.

The design parameters provided and the engineering advice offered in this report 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 design consultants 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, advice and comments relating to constructability issues and quality control may not be relevant or complete for the project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

5.3 Use of Report

This report is prepared for the express use of Mr. Sam Badawi and their retained design consultants. It 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.

Mr. Sam Badawi and their retained design consultants are authorized users.

It is recognized that the City of Burlington, in its capacity as the planning and building authority under Provincial statues, will make use of and rely upon this report, cognizant of the limitations thereof, both as are expressed and implied.

We trust that this report meets your present requirements. Should you have any questions regarding the information presented, please do not hesitate to contact our office.

Terraprobe Inc.

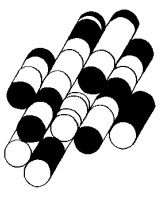
LICENSED PROFESSIONAL ENGINEER
J. J. CROWDER
100077148
18/12/2017
PROVINCE OF ONTARIO

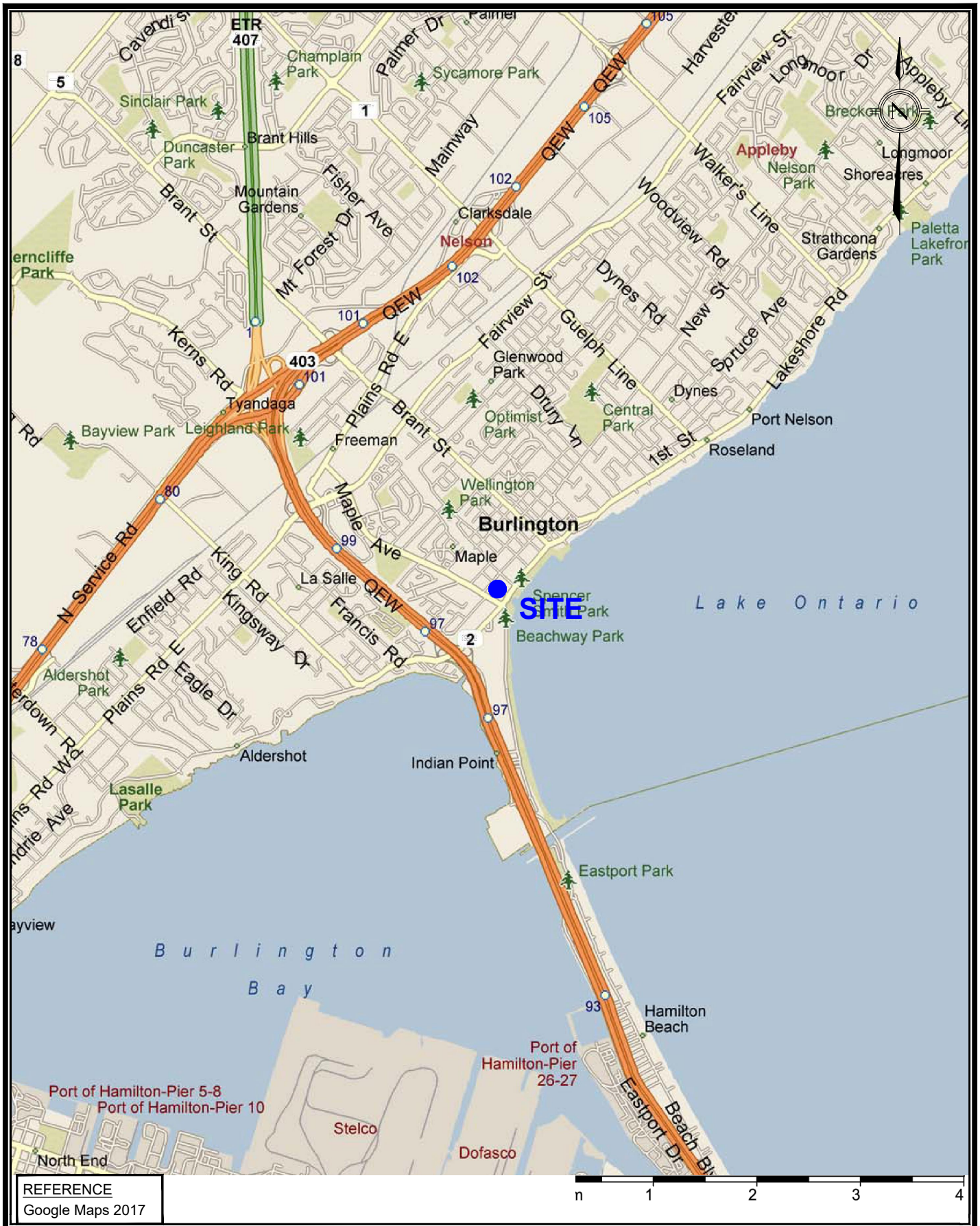
Nicholas Ng, B.A.Sc., EIT
Geotechnical Engineering Division

Jason Crowder, Ph.D., P.Eng.
Principal

FIGURES

TERRAPROBE INC.



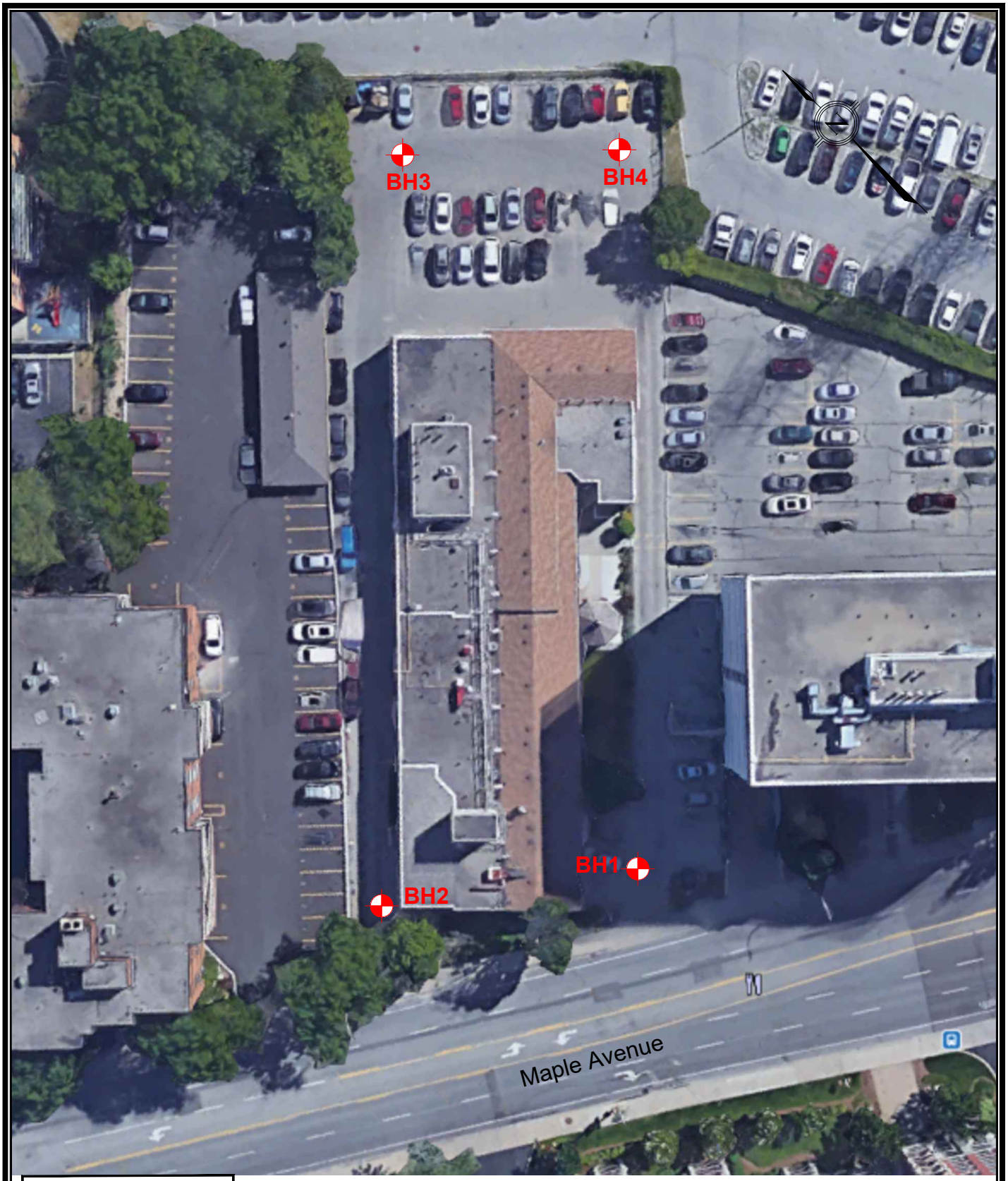


Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title: SITE LOCATION PLAN

File No. 1-17-0801-01

FIGURE : 1



LEGEND
Approximate
Borehole Location

SCALE 1:600
5 4 3 2 1 0 5m

REFERENCE
Google Earth 2017

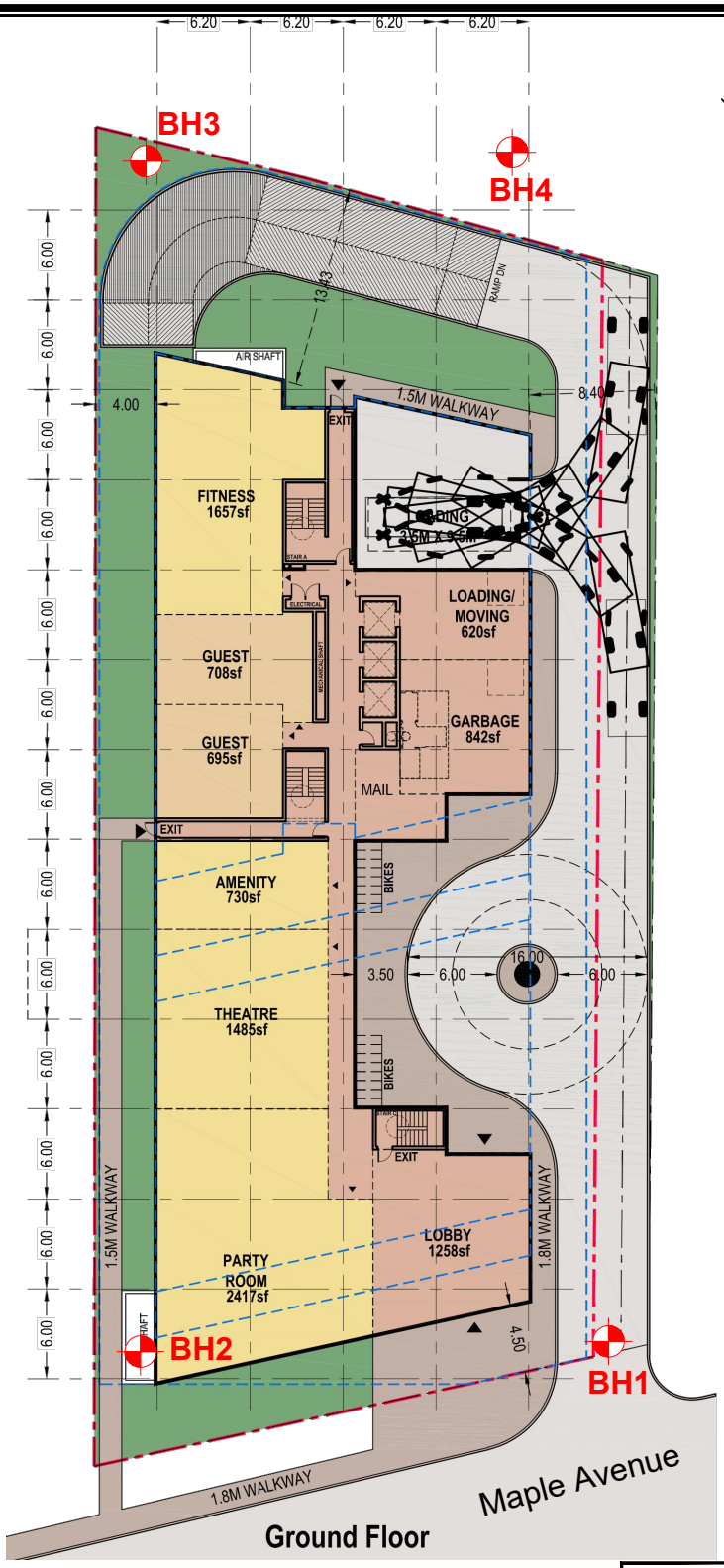
Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
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Title: BOREHOLE LOCATION PLAN
- EXISTING CONDITION

File No. 1-17-0801-01

FIGURE : 2

T:\1-Project Files\2017\1-17-0801 - 441 Maple Avenue, Burlington\01-GEO Investigation\A. Dwgs. Logs\AutoCAD\1-17-0801-01 FIG.dwg



LEGEND
Approximate
Borehole Location

SCALE 1:500
5 4 3 2 1 0 5m

REFERENCE
Schematic Floor Plans
441 Maple Avenue
Burlington, Ontario
Dated: October 24, 2017
By: Michael Spaziani Architect Inc.

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

Title: BOREHOLE LOCATION PLAN - PROPOSED CONDITION

File No. 1-17-0801-01

FIGURE : 3

T:\1-Project Files\2017\1-17-0801 - 441 Maple Avenue, Burlington\01-GEO Investigation\A. Dwgs. Logs\AutoCAD\1-17-0801-01 FIG.dwg

Alignment: A-A'
From E:597070, N:4797033
to E:597149, N:4797109

Elevation (m)

BH 2

50 mm dia. monitoring well

'N'

9
17
20
60
50 / 75mm
50 / 25mm

BH 1

50 mm dia. monitoring well

'N'

13
17
38
36
50 / 125mm
62%
67%
62%
97%
89%
97%
93%
87%

BH 3

50 mm dia. monitoring well

'N'

50 / 125mm
7
44
50 / 125mm
50 / 125mm
50 / 50mm

BH 4

50 mm dia. monitoring well

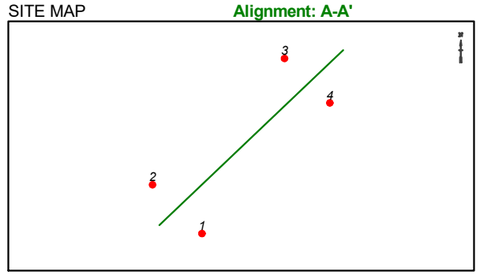
'N'

17
16
32
50 / 100mm
50 / 75mm
50 / 50mm

84
82
80
78
76
74
72
70
68
66
64

SITE MAP

Alignment: A-A'



LITHOLOGY GRAPHIC LEGEND

- Asphalt
- Aggregate
- Fill
- Sand
- Silt
- Bedrock
- Bedrock (cored)

Distance Along A-A' Baseline (m)

INTERPRETIVE LEGEND

- Unstabilized water level on completion of drilling
- Stabilized water level, most recent
- 30 SPT N-Value

- *FILL*
- *GRAVELS (gravel to gravelly sand)*
- *SILT TO SAND (not till)*
- *COHESIONLESS TILLS*
- *COHESIVE SOILS (clayey silt to clay, incl. tills)*

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Title: **SUBSURFACE PROFILE A-A'**

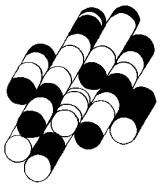
File No.: **1-17-0801-01**

FIGURE No.: **4**

Report: ISECTION - TABLOID - ELEV

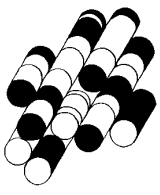
APPENDICES

TERRAPROBE INC.



APPENDIX A

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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

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ROCK CORE TERMINOLOGY

RECOVERY

TCR **Total Core Recovery** is the total length of core pieces, irrespective of their individual lengths obtained in a core run, and expressed as a percentage of the length of that core run.

SCR **Solid Core Recovery** is the total length of sound full-diameter core pieces obtained in a core run, expressed as a percentage of the length of that core run .

RQD **Rock Quality Designation** pertains to the sum of those pieces of sound core which are 10 cm or greater in length obtained in a core run, expressed as a percentage of the length of that core run.

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
QUALITY	**very poor**	**poor**	**fair**	**good**	**excellent**

JOINT CHARACTERISTICS

Joint Spacing (adapted from *Bieniawski 1989*, *ISRM 1981*)

Classification	Spacing
very close	< 60 mm
close	60 – 200 mm
moderately close	0.2 to 0.6 m
wide	0.6 to 2 m
very wide	> 2 m

Natural Fracture Frequency (per 0.3 m)
Refers to the number of natural fractures (joints, faults, etc.) which are present per 0.3m. Ignores mechanical or drill-induced breaks, and closed discontinuities (e.g. bedding planes).

Orientation

Orientation	Angle from horiz.
horizontal/flat	0 - 20°
dipping	20 - 50°
vertical	50 - 90°

Joint Filling

Description	Approx. φ`
tight, hard, non-softening	25 - 35
oxidation, surface staining only	25 - 30
slightly altered, clay-free	25 - 30
sandy particles, clay-free	2 - 25
sandy and silty, minor clay	1 - 24
non-softening clays	6 - 12
swelling clay fillings	n/a

Joint Aperture

Classification	Aperture
closed / tight	< 0.5 mm
gapped	0.5 to 10 mm
open	> 10 mm

Planarity
- Planar
- Undulating
- Stepped
- Irregular
- Discontinuous

Roughness
- Very rough
- Rough
- Smooth
- Slickensided
- Polished

Coating	Description
clean	no filling
veneer	< 1 mm filling
coating / infill	> 1 mm filling

GENERAL

Degree of Weathering (*after MTO, RR229 Evaluation of Shales for Construction Projects*)

Zone	Degree	Description
Z1	unweathered	shale, regular jointing
Z2	partially weathered	angular blocks of unweathered shale, no matrix, with chemically weathered but intact shale
Z3		soil-like matrix with frequent angular shale fragments < 25mm diameter
Z4a		soil-like matrix with occasional shale fragments < 3mm diameter
Z4b	fully weathered	soil-like matrix only

Strength classification (*after Marinos and Hoek, 2001*)

Grade	Term	UCS (MPa)	Field Estimate (Description)
R6	extremely strong	> 250	can only be chipped by geological hammer
R5	very strong	100 - 250	requires many blows from geological hammer
R4	strong	50 - 100	requires more than one blow from geological hammer
R3	medium strong	25 - 50	can't be scraped, breaks under one blow from geological hammer
R2	weak	5 - 25	can be peeled / scraped with knife with difficulty
R1	very weak	1 - 5	easily scraped / peeled, crumbles under firm blow of geo. hammer
R0	extremely weak	< 1	indented by thumbnail

Bedding Thickness (*Quarterly Journal of Engineering Geology, Vol 3, 1970*)

Very thickly bedded	> 2 m	Medium bedded	200 – 600mm	Very thinly bedded	20 – 60mm	Thinly Laminated < 6mm
Thickly bedded	0.6 – 2m	Thinly bedded	60 – 200mm	Laminated	6 – 20mm	

LOG OF BOREHOLE 1

Terraprobe

Project No. : 1-17-0801-01	Client : Sam Badawi	Originated by : AR
Date started : November 14, 2017	Project : 441 Maple Avenue	Compiled by : NN
Sheet No. : 1 of 2	Location : Burlington, Ontario	Checked by : JC

Position : E: 597088, N: 4797029 (UTM 17T) | Elevation Datum : Geodetic | Core Diameter : **HQ**, OD=96mm, ID=64mm

Rig type : Truck-mounted | Drilling Method : Solid stem augers, HQ rock coring

Depth Scale (m)	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value
0	83.1	**GROUND SURFACE**				
		75mm **ASPHALTIC CONCRETE**				
	82.7	300mm **AGGREGATE**		1	SS	13
	0.4	**FILL**, sand, some silt, trace gravel, trace clay, trace asphalt fragments, trace rock fragments, trace cinders, contains silt and clayey silt pockets, compact to dense, brown, moist ...at 0.8 m, wet		2	SS	17
1				3	SS	38
2	80.8					
	2.3	**SAND**, trace silt, trace clay, dense, reddish brown, moist		4A	SS	36
	80.5			4B		
	2.6	**SILT**, trace to some clay, trace to some sand, trace gravel, dense to very dense, reddish brown to reddish grey, moist		5	SS	50 / 125mm
3						
	78.6					
	4.5	**INFERRED BEDROCK**, shale, red		6	SS	
	78.1					
4	5.0	**QUEENSTON FORMATION** (See rock core log for details) ...at 5.4 m, transition to sound bedrock		1	RUN	
5						
6				2	RUN	
7						
8				3	RUN	
9						
10				4	RUN	

Elevation Scale (m): 83, 82, 81, 80, 79, 78, 77, 76, 75, 74, 73

Penetration Test Values (Blows / 0.3m) — Dynamic Cone: 10, 20, 30, 40

Undrained Shear Strength (kPa) — Unconfined, Pocket Penetrometer, Field Vane, Lab Vane: 40, 80, 120, 160

Moisture / Plasticity — Plastic Limit (PL), Natural Water Content (MC), Liquid Limit (LL): 10, 20, 30

Headspace Vapour (ppm)

Instrument Details — W1, W2

Unstabilized Water Level

Lab Data and Comments

GRAIN SIZE DISTRIBUTION (%) (MIT) — GR SA SI CL

...at 4.5m, auger refusal

...at 4.9m, casing installed, stared rock coring

file: 1-17-0801-01_borehole_logs.gpj

(continued next page)

LOG OF BOREHOLE 1

Terraprobe

Project No. : 1-17-0801-01	Client : Sam Badawi	Originated by : AR
Date started : November 14, 2017	Project : 441 Maple Avenue	Compiled by : NN
Sheet No. : 2 of 2	Location : Burlington, Ontario	Checked by : JC

Position : E: 597088, N: 4797029 (UTM 17T) | Elevation Datum : Geodetic | Core Diameter : **HQ**, OD=96mm, ID=64mm

Rig type : Truck-mounted | Drilling Method : Solid stem augers, HQ rock coring

Depth Scale (m)	Elev / Depth (m)	Description	Graphic Log	Sample Number	Sample Type	SPT 'N' Value	Elevation Scale (m)	Penetration Test Values (Blows / 0.3m) / Undrained Shear Strength (kPa)	Moisture / Plasticity	Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments
		(continued)										
11		**QUEENSTON FORMATION** (See rock core log for details) *(continued)*		4	RUN		72					
12				5	RUN		71					
13				6	RUN		70					
14							69					
15				7	RUN		68					
16				8	RUN		67					
17	65.8 / 17.3						66					

Penetration Test Values (Blows / 0.3m): Dynamic Cone — 10, 20, 30, 40

Undrained Shear Strength (kPa): Unconfined, Pocket Penetrometer, Field Vane, Lab Vane — 40, 80, 120, 160

Moisture / Plasticity: Plastic Limit (PL), Natural Water Content (MC), Liquid Limit (LL) — 10, 20, 30

Instrument Details: W1, W2

Unstabilized Water Level

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

END OF BOREHOLE

Water level and cave not measured upon completion of drilling.

W1: 50 mm dia. monitoring well installed.
W2: 50 mm dia. monitoring well installed.

W1 WATER LEVELS

Date	Water Depth (m)	Elevation (m)
Nov 30, 2017	2.2	80.9

W2 WATER LEVELS

Date	Water Depth (m)	Elevation (m)
Nov 30, 2017	4.9	78.2

file: 1-17-0801-01_borehole_logs.gpj

ROCK CORE LOG 1

Terraprobe

Project No. : 1-17-0801-01 | Client : Sam Badawi | Originated by : AR

Date started : November 14, 2017 | Project : 441 Maple Avenue | Compiled by : NN

Sheet No. : 1 of 2 | Location : Burlington, Ontario | Checked by : JC

Position : E: 597088, N: 4797029 (UTM 17T) | Elevation Datum : Geodetic | Core Diameter : **HQ**, OD=96mm, ID=64mm

Rig type : Truck-mounted | Drilling Method : Solid stem augers, HQ rock coring

Rock coring started at 5.0m below grade

QUEENSTON FORMATION

Shale, red, laminated to thinly bedded, weak; joints are horizontal, gapped to closed, planar, clean;

interbedded with **limestone**, grey, medium strong

...at 5.4m, transition to sound bedrock

Run 1 : 2% limestone 98% shale

Run 2 : 7% limestone 93% shale

Run 3 : 5% limestone 95% shale

Run 4 : 0% limestone 100% shale

Run 5 : 8% limestone 92% shale

Run 6 : 12% limestone 88% shale

Run 7 : 10% limestone 90% shale

Run	Elev Depth (m)	Recovery	Elevation (m)
	78.1		
	5.0		78
R1		TCR = **92%** SCR = **75%** RQD = **62%**	
	76.6		77
	6.5		
R2		TCR = **100%** SCR = **100%** RQD = **67%**	76
	75.1		
	8.0		75
R3		TCR = **100%** SCR = **97%** RQD = **62%**	
	73.6		74
	9.5		
R4		TCR = **97%** SCR = **97%** RQD = **97%**	73
	72.0		
	11.1		72
R5		TCR = **95%** SCR = **95%** RQD = **89%**	
	70.4		71
	12.7		
R6		TCR = **97%** SCR = **97%** RQD = **97%**	70
	68.9		
	14.2		69
R7		TCR = **100%** SCR = **97%** RQD = **93%**	
			68

Shale Weathering Zones: Z1 Z2 Z3 Z4

UCS (MPa) 5 25 50 100 250 — Estimated Strength: R1 R2 R3 R4 R5 R6

Natural Fractures — Frequency / Spacing

Frequency
>10
5
1
2
0
1
0
0
1
2
1
4
4
0
0
0
0
0
1
1
2
1
0
0
0
0
0
0
0
0
1

Laboratory Testing

Comments

5.1m: *250mm thick rubblized zone*

7.7m: *400mm long vertical joint, planar, closed clean, smooth*

8.0m: *50mm thick rubblized zone*

(continued next page)

Terraprobe

ROCK CORE LOG 1

Project No. : 1-17-0801-01 | Client : Sam Badawi | Originated by : AR

Date started : November 14, 2017 | Project : 441 Maple Avenue | Compiled by : NN

Sheet No. : 2 of 2 | Location : Burlington, Ontario | Checked by : JC

Position : E: 597088, N: 4797029 (UTM 17T) | Elevation Datum : Geodetic | Core Diameter : **HQ**, OD=96mm, ID=64mm

Rig type : Truck-mounted | Drilling Method : Solid stem augers, HQ rock coring

Depth (m)	Graphic Log	GENERAL DESCRIPTION	Run / Elev Depth (m)	Recovery	Elevation (m)	Shale Weathering Zones	UCS (MPa) / Estimated Strength	Natural Fractures: Frequency	Natural Fractures: Spacing	Laboratory Testing	Comments	Elevation (m)
			78.1			Z1 Z2 Z3 Z4	5 25 50 100 250 / R1 R2 R3 R4 R5 R6					
		QUEENSTON FORMATION **Shale**, red, laminated to thinly bedded, weak; joints are horizontal, gapped to closed, planar, clean;	R7 67.4	TCR = **100**% SCR = **97**% RQD = **93**%				0 0				
		Run 7 : 10% limestone *90% shale* interbedded with **limestone**, grey, medium strong	15.7					0				
16					67			3				67
			R8	TCR = **100**% SCR = **100**% RQD = **87**%				0			**16.5m:** *rubblized zone (mechanical fracture)*	
								2				
								8				
17		*Run 8 : 7% limestone* *93% shale*			66			1				66
			65.8					1				
			17.3m									
		END OF COREHOLE										

LOG OF BOREHOLE 2

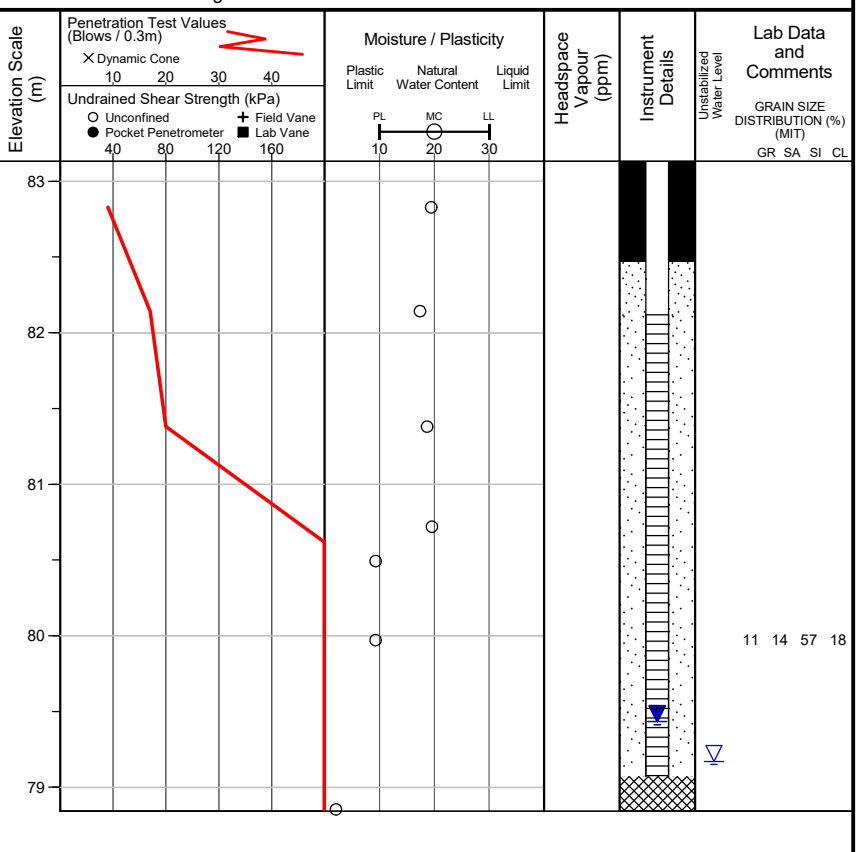
Terraprobe

Project No. : 1-17-0801-01	Client : Sam Badawi	Originated by : AR
Date started : November 13, 2017	Project : 441 Maple Avenue	Compiled by : NN
Sheet No. : 1 of 1	Location : Burlington, Ontario	Checked by : JC

Position : E: 597067, N: 4797050 (UTM 17T) Elevation Datum : Geodetic

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

Depth Scale (m)	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value
	83.1	**GROUND SURFACE**				
0	82.9	75mm **ASPHALTIC CONCRETE**				
	0.2	150mm **AGGREGATE**		1	SS	9
		FILL, clayey silt, sandy, trace gravel, trace asphalt fragments, trace brick fragments, trace organics, loose to compact, brown, moist		2	SS	17
1	81.6					
	1.5	**SILT**, trace to some clay, trace to some sand, trace gravel, compact, reddish brown to reddish grey, moist		3	SS	20
		...at 2.3 m, very dense		4A	SS	60
2	80.4			4B		
	2.7	**SILT**, some clay, some sand, trace gravel, trace shale fragments, very dense, red, moist (RESIDUAL SOIL)		5	SS	50 / 75mm
3	78.8					
	78.8	**INFERRED BEDROCK**, shale, red		6	SS	50 / 25mm
4	4.3	**END OF BOREHOLE**				



WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Nov 30, 2017	3.7	79.4

Unstabilized water level measured at 4.0 m below ground surface; borehole caved to 4.1 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

file: 1-17-0801-01_borehole_logs.gpj

LOG OF BOREHOLE 3

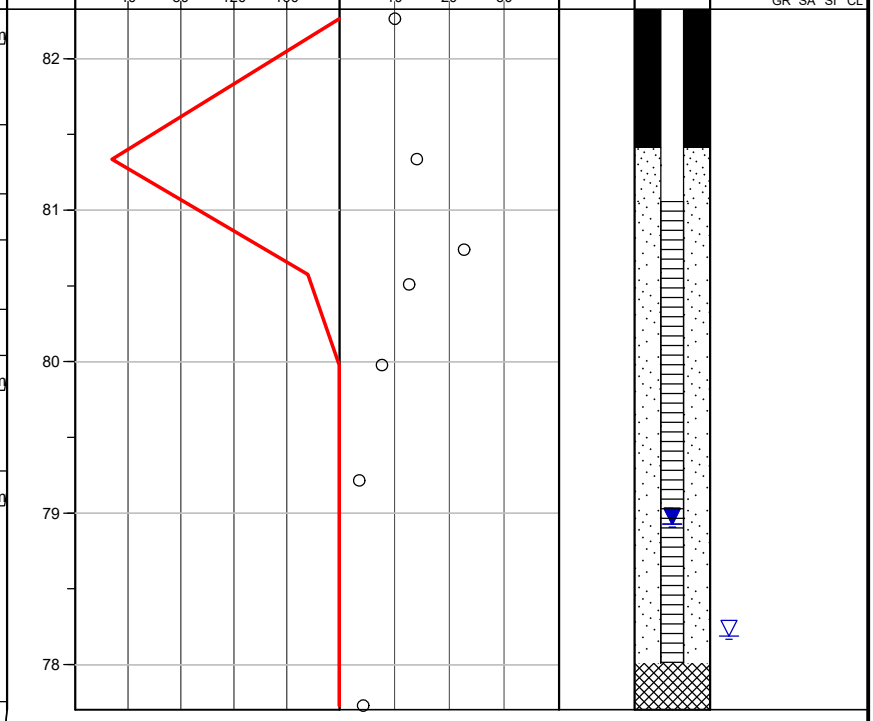
Terraprobe

Project No. : 1-17-0801-01	Client : Sam Badawi	Originated by : AR
Date started : November 13, 2017	Project : 441 Maple Avenue	Compiled by : NN
Sheet No. : 1 of 1	Location : Burlington, Ontario	Checked by : JC

Position : E: 597124, N: 4797105 (UTM 17T) Elevation Datum : Geodetic

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

Depth Scale (m)	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value
0	82.3	**GROUND SURFACE**				
		50mm **ASPHALTIC CONCRETE**		1	SS	50 / 125mm
	81.9	325mm **AGGREGATE**				
	0.4	**FILL**, sand, trace gravel, trace silt, trace asphalt fragments, trace brick fragments, trace rock fragments, trace cinders, trace organics, very dense, dark brown, moist		2	SS	7
	81.5					
	0.8	**FILL**, clayey silt, sandy, trace gravel, trace ashes, trace cinders, trace organics, trace clay, firm, mottled greyish brown to dark brown, moist		3A		
	80.6			3B	SS	44
	1.7	**SILT**, trace sand, trace gravel, trace shale fragments, dense, red, moist (RESIDUAL SOIL)				
	80.0					
	2.3	**INFERRED BEDROCK**, shale, red		4	SS	50 / 125mm
				5	SS	50 / 125mm
	77.7			6	SS	50 / 50mm
	4.6	**END OF BOREHOLE**				



Unstabilized water level measured at 4.1 m below ground surface; borehole caved to 4.3 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Nov 30, 2017	3.4	78.9

file: 1-17-0801-01_borehole_logs.gpj

LOG OF BOREHOLE 4

Terraprobe

Project No. : 1-17-0801-01	Client : Sam Badawi	Originated by : AR
Date started : November 13, 2017	Project : 441 Maple Avenue	Compiled by : NN
Sheet No. : 1 of 1	Location : Burlington, Ontario	Checked by : JC

Position : E: 597144, N: 4797086 (UTM 17T) Elevation Datum : Geodetic

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

Depth Scale (m)	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value
	82.6	**GROUND SURFACE**				
		75mm **ASPHALTIC CONCRETE**				
		50mm **AGGREGATE**		1	SS	17
		FILL, sand, some silt, trace gravel, trace asphalt fragments, trace cinders, trace organics, compact, brown, moist				
	81.8					
	0.8	**SILT**, trace to some clay, trace to some sand, trace gravel, compact to very dense, brownish grey, moist		2	SS	16
				3	SS	32
		...at 2.3 m, clay and silt, trace sand, trace gravel, very dense		4	SS	50 / 100mm
	79.6					
	3.0	**INFERRED BEDROCK**, shale, red		5	SS	50 / 75mm
	78.0					
	4.6	**END OF BOREHOLE**		6	SS	50 / 50mm

Elevation Scale (m): 82, 81, 80, 79

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

Undrained Shear Strength (kPa): Unconfined, Pocket Penetrometer, Field Vane, Lab Vane — 40 80 120 160

Moisture / Plasticity: Plastic Limit (PL) 10, Natural Water Content (MC) 20, Liquid Limit (LL) 30

Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level

Lab Data and Comments

GRAIN SIZE DISTRIBUTION (%) (MIT)

GR SA SI CL

2 6 42 50

Borehole was dry and caved to 4.3 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

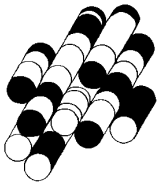
WATER LEVEL READINGS

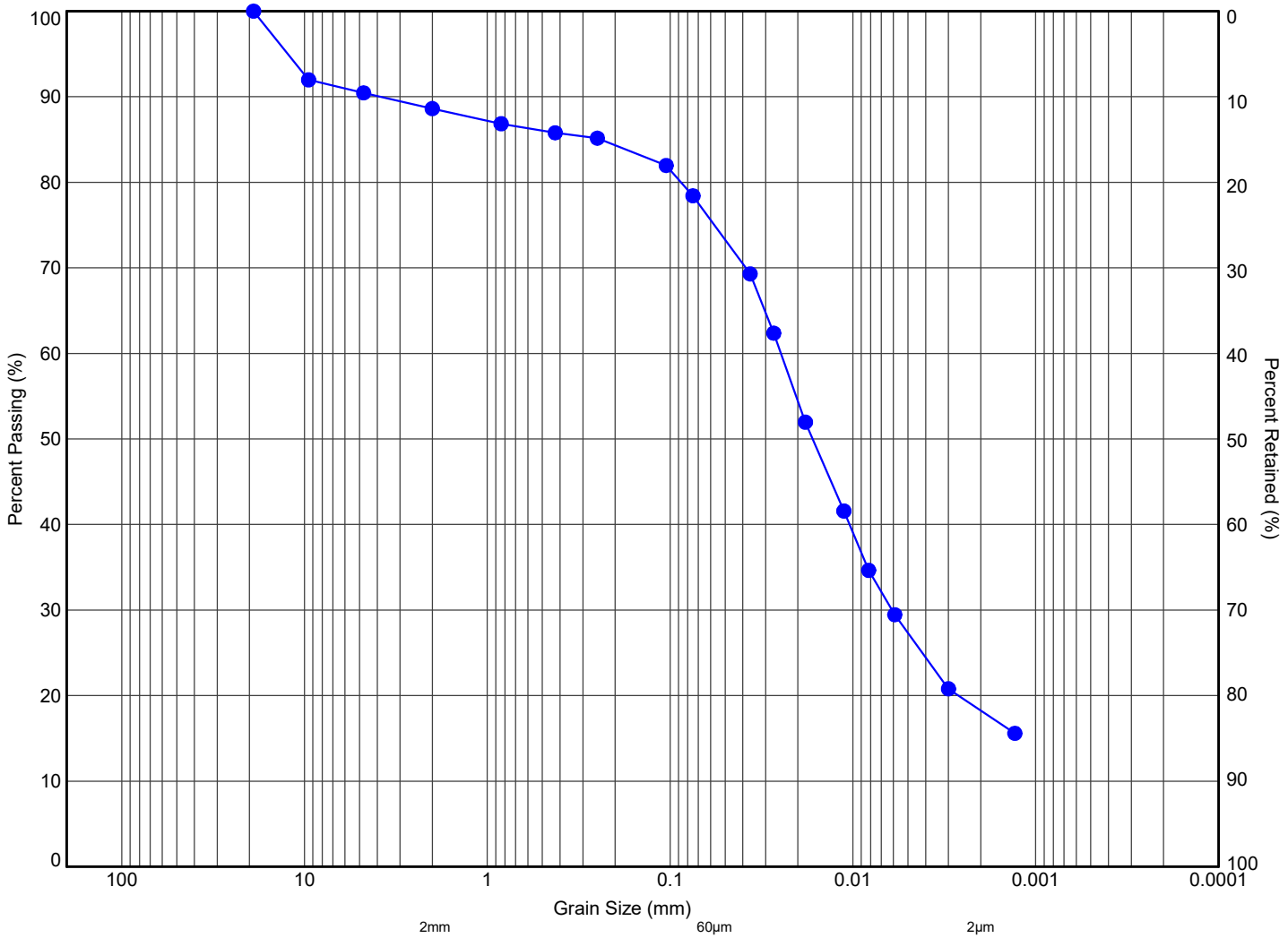
Date	Water Depth (m)	Elevation (m)
Nov 30, 2017	3.7	78.9

file: 1-17-0801-01_borehole_logs.gpj

APPENDIX B

TERRAPROBE INC.





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**	SS5	3.2	80.0	11	14	57	18	

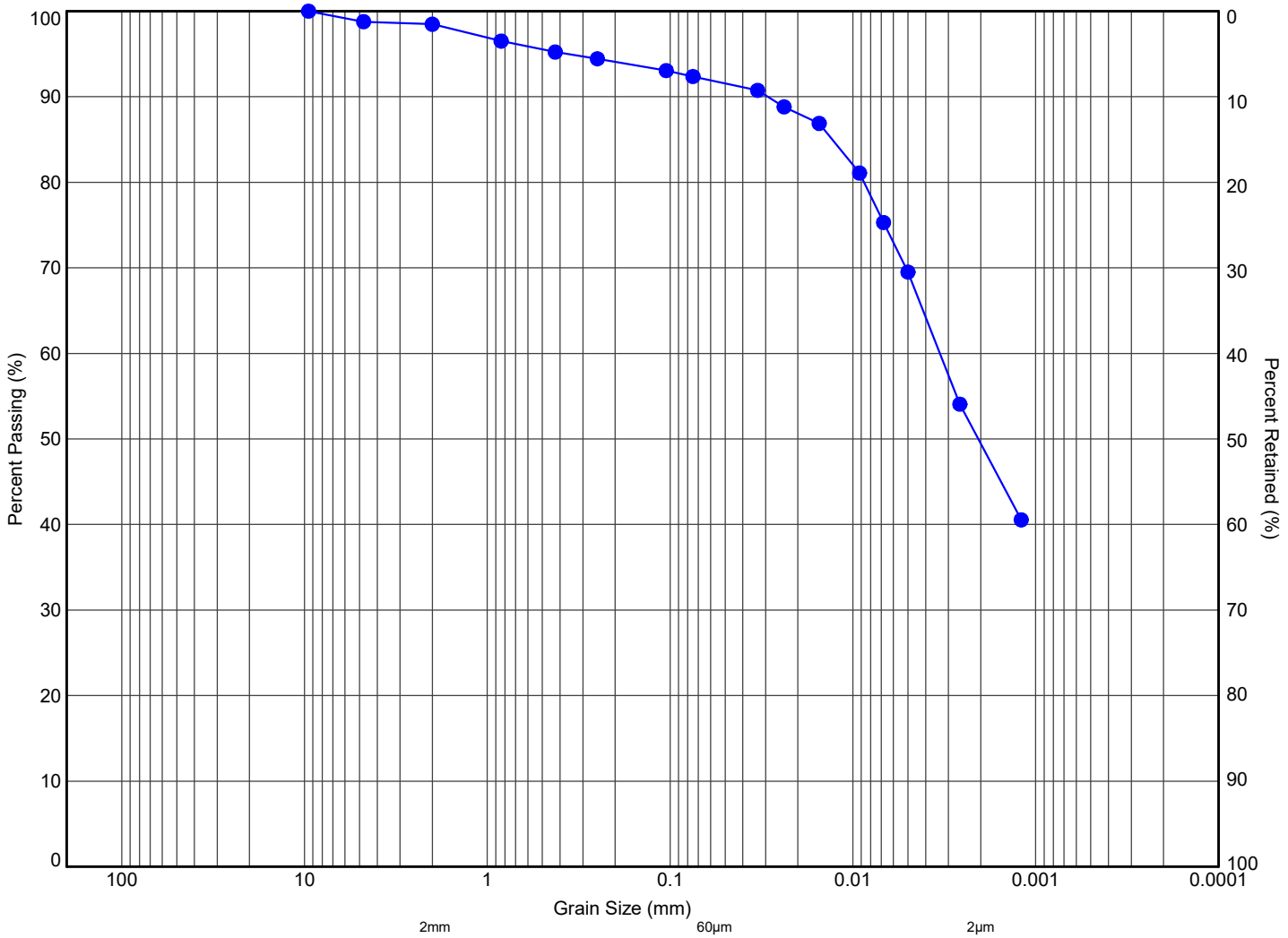
Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

GRAIN SIZE DISTRIBUTION

SILT, SOME CLAY, SOME SAND, SOME GRAVEL, (RESIDUAL SOIL)

File No.: **1-17-0801-01**



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, %)*
●	**4**	SS4	2.4	80.2	2	6	42	50	

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

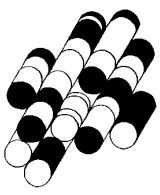
Title:

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

File No.: **1-17-0801-01**

APPENDIX C

TERRAPROBE INC.





Photograph 1

Run: 1

Depth: 5 to 6.5 m (Elev. 76.6 to 78.1 m)



Photograph 2

Run: 2

Depth: 6.5 to 8.0 m (Elev. 75.1 to 76.6 m)



Photograph 3

Run: 3

Depth: 8.0 to 9.5 m (Elev. 73.6 to 75.1 m)



Photograph 4

Run: 4

Depth: 9.5 to 11.1 m (Elev. 72.0 to 73.6 m)



Photograph 5

Run: 5

Depth: 11.1 to 12.7 m (Elev. 70.4 to 72.0 m)



Photograph 6

Run: 6

Depth: 12.7 to 14.2 m (Elev. 68.9 to 70.4 m)



Photograph 7

Run: 7

Depth: 14.2 to 15.7 m (Elev. 67.4 to 68.9 m)



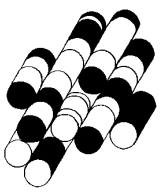
Photograph 8

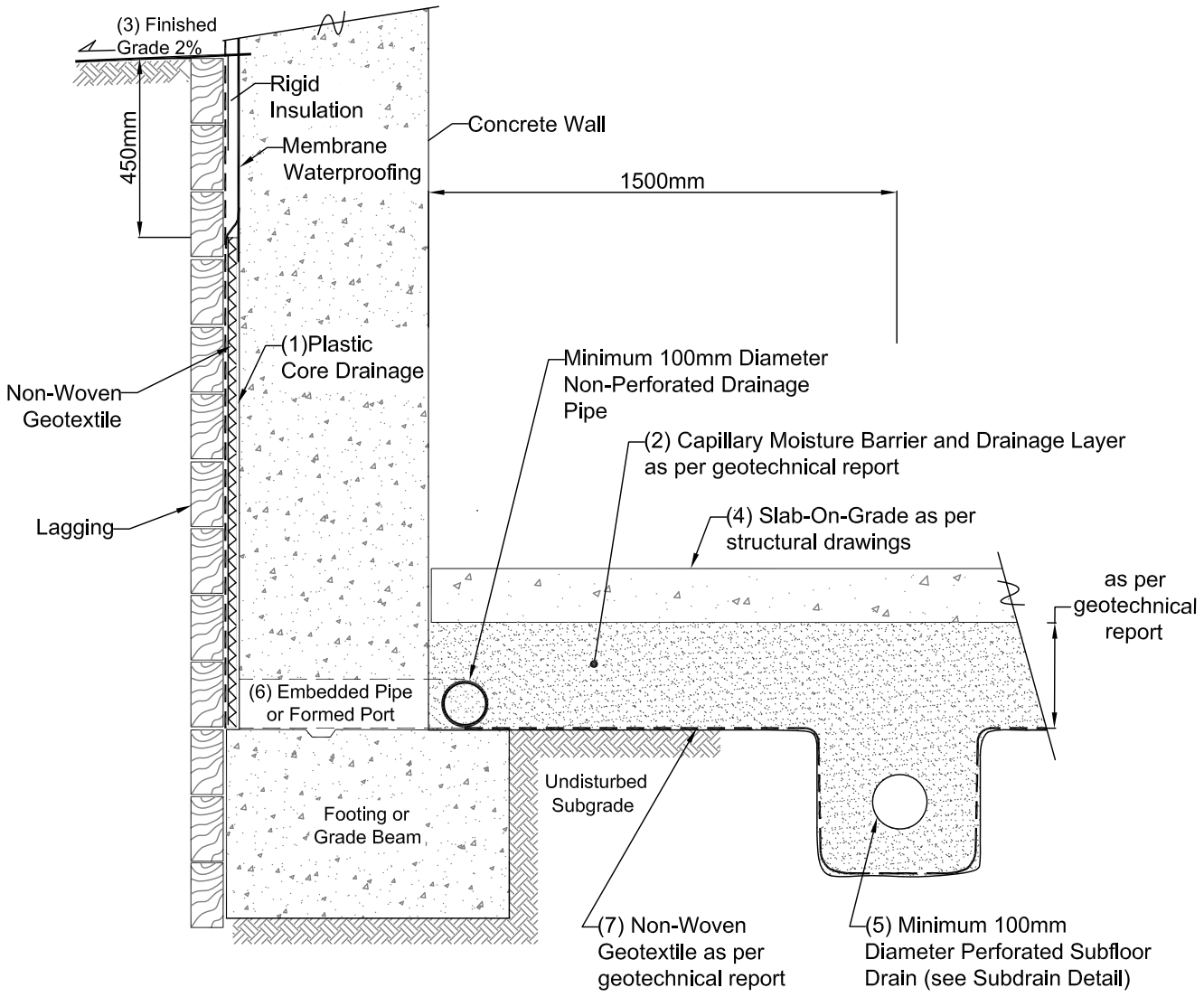
Run: 8

Depth: 15.7 to 17.3 m (Elev. 65.8 to 67.4 m)

APPENDIX D

TERRAPROBE INC.





NOTES

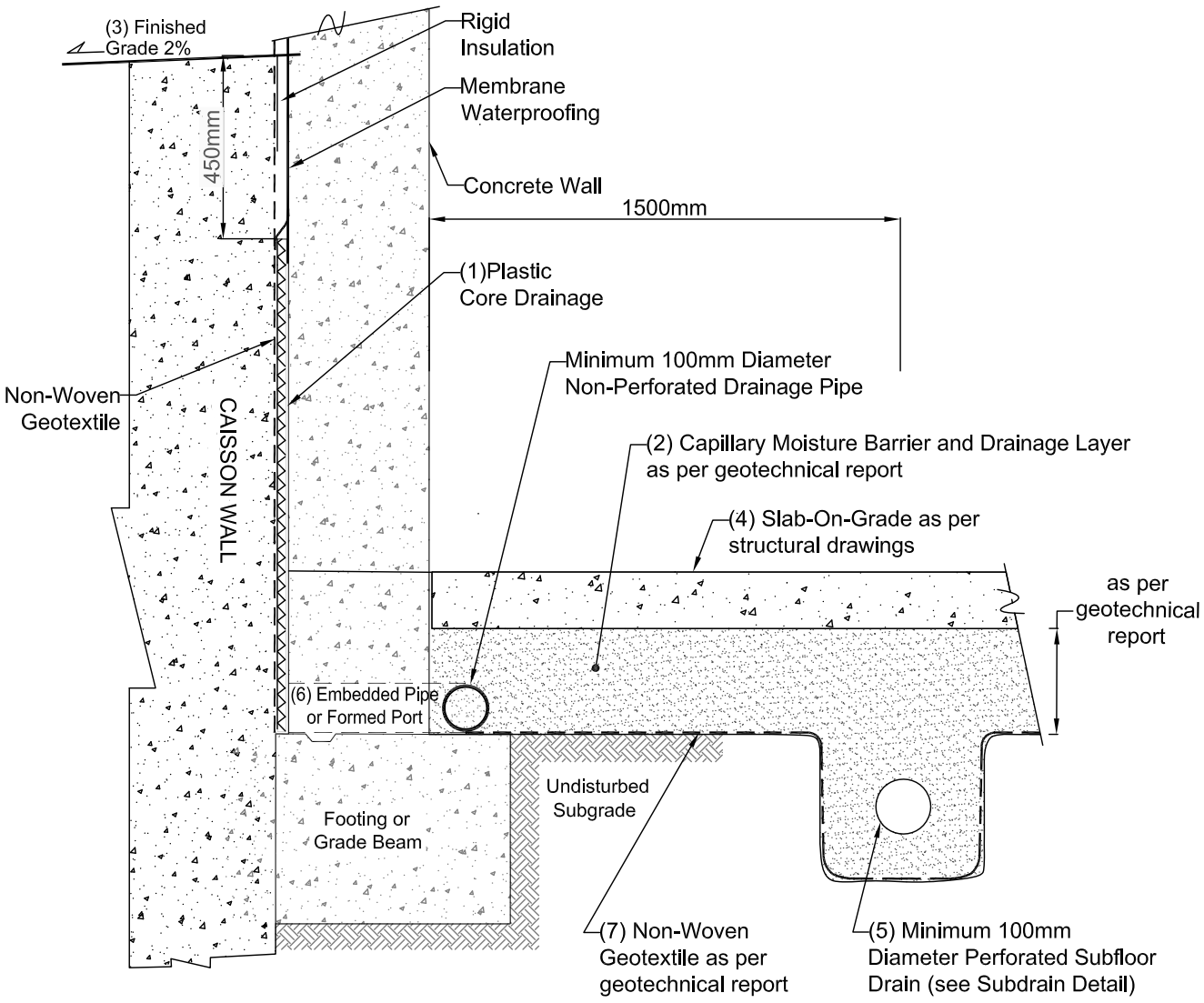
1) Prefabricated drainage panels to consist of Terrafix - TERRADRAIN 200, Mirafi - Miradrain 6000, or approved equivalent. Panels should provide continuous cover with a minimum overlap of 300mm.
2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular "A" (OPSS 1010) compacted to 98% SPMDD where vehicular traffic is required.
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.
6) Embedded pipes/formed 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 non-perforated pipe.
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).

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



NOTES

1) Prefabricated drainage panels to consist of Terrafix - TERRADRAIN 200, Mirafi - Miradrain 6000, or approved equivalent. Panels should provide continuous cover with a minimum overlap of 300mm.

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

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.

6) Embedded pipes/formed 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 non-perforated pipe.

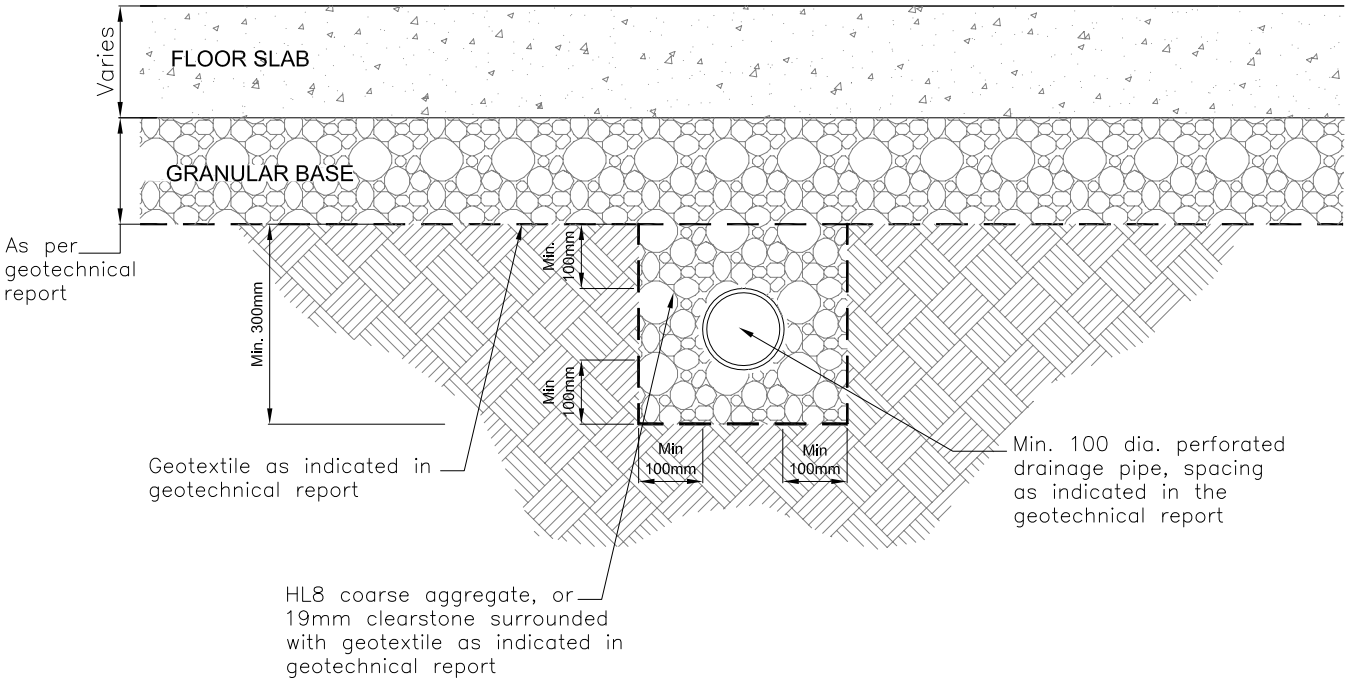
7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

N.T.S.

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

Title:

SCHEMATIC DRAINAGE DETAIL
CAISSON WALL SHORING SYSTEM



Schematic Only
Not to Scale

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

Title:

BASEMENT SUBDRAIN DETAIL