Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION AND ENGINEERING DESIGN
2107 & 2119 OLD LAKESHORE ROAD
BURLINGTON, ONTARIO

Prepared For: Carriage Gate Homes
421 Brant Street, Suite 200
Burlington, Ontario
L7R 2G3

Attention: Mr. Mark Bales

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1.0 THE PROJECT

Terraprobe was retained by Carriage Gate Homes to conduct a subsurface investigation and provide geotechnical engineering design advice for the proposed redevelopment of municipal addresses 2107 and 2119 Old Lakeshore Road, in Burlington, Ontario. The properties under consideration in this report are located southwest of the intersection of Lakeshore Road and Old Lakeshore Road. The address 2107 Old Lakeshore Road is a consolidated property formerly made of 2107, 2111, and 2114 Old Lakeshore Road. A site location plan is provided as Figure 1. Terraprobe is also providing a Phase One ESA and a hydrogeological investigation under separate covers.

The site is currently developed with two low-rise buildings and paved parking areas. As part of the new development, the existing buildings will be demolished and high rise towers over a mixed use podium resting on a four (4) level underground parking structure will be constructed at the site. It is expected that the proposed lowest parking level four (P4) finished floor elevation (FFE) will be set at Elev. 67 ± m within bedrock, and foundations will extend nominally 1 m deeper.

The drilling field work was carried out on November 23rd and 25th, 2016. The borehole locations are shown on Figure 2. The borehole and rock coring results are recorded on the borehole and rock core logs provided in Appendix A, and a summary of the geotechnical laboratory tests results are provided in Appendix B. The borehole locations were surveyed by Terraprobe for horizontal coordinates and geodetic elevations with a Trimble R10 Receiver connected to the Global Navigation Satellite System and the Can-Net Virtual Reference Station Network. These elevations are provided for the purpose of relating borehole stratigraphy and should not be used or relied on for other purposes.

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 excavation, foundation and slab construction, and shoring are discussed. This report provides detailed geotechnical engineering advice for detailed design and permitting purposes.

2.0 SUBSURFACE CONDITIONS

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

more specific subsurface details, refer to the enclosed Log of Borehole and Rock Core Log sheets in Appendix A, and the geotechnical laboratory results in Appendix B.

Asphaltic concrete material 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. In general, earth fill was encountered at the site, underlain by a cohesive glacial till, underlain by bedrock of the Queenston Formation.

2.1.1 Pavement Structure and Earth Fill

The boreholes encountered a pavement structure at the ground surface comprising 50 mm of asphaltic concrete underlain by 100 to 150 mm of aggregate.

Underlying the pavement structure, Borehole 2 encountered a zone of earth fill that extends to a depth of 1.5 m below grade (Elev. 78.1 ± m). The fill consists of reddish brown and moist sand, with some silt to being silty, and trace amounts of clay and gravel. The Standard Penetration Test (SPT) results (N-Values) measured in the fill were 15 and 16 blows per 300 mm of penetration, indicating a compact relative density.

2.1.2 Clayey Silt Glacial Till

Underlying the pavement structure in Borehole 1 and the earth fill in Borehole 2, a glacial till deposit with a cohesive matrix comprising clayey silt, with some sand and trace gravel was encountered. A hydrocarbon odour was detected in the recovered glacial till sample in Borehole 2. The till was found at a depth of 0.2 to 1.5 m below grade (Elev. 81.3 to 78.1 ± m) and extend to the inferred bedrock surface at a depth of 2.3 m below grade (Elev. 79.2 to 77.3 ± m). The till is greyish brown to reddish brown and moist. The SPT N-Values ranged from 6 to 88 blows per 300 mm of penetration, indicating a stiff to hard consistency.

2.1.3 Bedrock

Bedrock of the Queenston Formation was inferred and identified in Borehole 1 and inferred in Borehole 2, underlying the cohesive glacial till. The bedrock surface is inferred to be at a depth of 2.3 m below grade (Elev. 79.2 to 77.3 ± m). A total of 14.3 m of rock core was recovered from Borehole 1. Detailed rock core logs are included in Appendix A, and 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 hard layers are provided on the rock core log (indicated as "limestone") and ranged from 0 to 30% (16% on average) per core run (about 1.5 m).

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
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 glacial soil overburden. In the Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects,* there is a *typical weathering profile of a low durability shale*, reproduced from Kempton, Davis and Chandler, 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

Based on auger grinding, auger resistance, and samples recovered from the split spoon sampler, the bedrock surface was inferred in the boreholes at a depth of 2.3 m below grade (Elev. 79.2 to 77.3 ± m). The bedrock surface is at a higher elevation on the southwest side of the site and at a lower elevation on the northeast side of the site. In Borehole 1 where rock core was recovered, the zone of partially weathered (Zone II and III) shale was found to be approximately 0.5 m thick, and the transition to sound/unweathered (Zone I) shale was encountered at 2.8 m below grade (Elev. 78.7 ± m).

A summary of the bedrock elevations is tabulated below:

Borehole	Ground Surface Elev. (m)	Top of Weathered (Zone II / III) Bedrock, Depth / Elev. (m)	Top of Sound (Zone I) Bedrock, Depth / Elev. (m)	Method of Bedrock Identification
1	81.5	2.3 / 79.2	2.8 / 78.7	Inferred by split spoon samples, sampler bouncing, auger refusal; Proven by recovered rock core
2	79.6	2.3 / 77.3	-	Inferred by split spoon samples, sampler bouncing, auger refusal

*Rock coring not conducted in Borehole 2.

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 bedrock ranged from 47 to 99%. While the RQD for some of the runs in sound bedrock are 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

Unstabilized ground water level observations were made in the open boreholes during and after drilling, as noted on the borehole logs. A total of two (2) 50 mm diameter monitoring wells were installed in the boreholes to facilitate long-term ground water monitoring. The water levels measured within the wells are shown on the borehole logs and are summarized below:

Borehole No.	Elevation of Well Screen (m)	Stratum Captured by Well Screen	Depth / Elevation of Water Level in Well (m)	
			Dec. 8, 2016	Dec. 19, 2016
1	72.4 to 39.4	Bedrock	4.3 / 77.2	n/a*
2	78.1 to 76.6	Glacial Till; Weathered Bedrock	2.2 / 77.4	2.1 / 77.5

*Well location could not be accessed due to snow cover on Dec. 19, 2016.

Based on the above water level readings, the prevailing ground water level at the site is near Elev. 77.5 ± m. The earth fill zone may also contain perched water above the cohesive glacial till.

The earth fill is cohesionless and will allow for the free flow of water. The cohesive glacial till contains a high percentage of fines and is of low permeability, precluding the free flow of water. The sound bedrock is considered to be of low permeability.

Terraprobe's Hydrogeological Investigation (File No. 1-16-0680-46) should be consulted for additional information on the hydrogeological considerations at this site. It should be noted that regrading of the site,

construction dewatering, 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 DESIGN

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

The site is currently developed with two low-rise buildings and paved parking areas. As part of the new development, the existing buildings will be demolished and high rise towers over a mixed use podium resting on a four (4) level underground parking structure will be constructed at the site. It is expected that the proposed lowest parking level four (P4) finished floor elevation (FFE) will be set at Elev. 67 ± m within bedrock, and foundations will extend nominally 1 m deeper.

3.1 Foundation Design Parameters

The top of weathered bedrock was inferred at a depth of 2.3 ± m below existing grade (Elev. 79.2 to 77.3 ± m) in the boreholes. The transition to sound (unweathered) bedrock was identified by the recovered rock core in Borehole 1 at a depth of 2.8 m below grade (Elev. 78.7 ± m).

The lowest FFE is at approximately Elev. 67 ± m for four (4) levels of underground parking. Based on the factual data from the borehole logs, all foundations will be made as spread footings on sound bedrock of the Queenston Formation. Spread footings on sound Queenston Formation bedrock may be designed using a maximum factored geotechnical resistance at ULS of 5,000 kPa. The maximum net geotechnical reaction at SLS is 4,000 kPa, for an estimated total settlement of 25 mm. The settlement of foundations made on the sound bedrock is elastic, linear and non-recoverable. The settlement occurs as load is applied.

Footings stepped from one elevation to another should be offset at a slope not exceeding 7 vertical to 10 horizontal. All foundations exposed to ambient outside temperatures must have a minimum 1.2 m of soil cover or equivalent to protect against frost heave, as is the case for the one level of the underground parking prepared for the medical tower. 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. In our experience, the earth cover required to prevent frost effects on foundations in the lower parking levels need not be any greater than 1.2 m, and unmonitored experience in a number of structures has suggested that perimeter foundations provided with 600 mm of cover perform adequately as do interior isolated foundations with 900 mm of cover. This is the case for the proposed structure with four underground levels. At locations adjacent ventilation shafts, it is normal practise 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. 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. It should be noted that 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				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) \geq 1.25$
B	0.6	0.7	0.7	0.8	0.8

3.3 Earth Pressure Design Parameters

The parameters used in the determination of earth pressures acting on retaining walls are defined below.

Parameter	Definition	Units
φ	internal angle of 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

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

Stratum/Parameter	φ (deg.)	γ (kN/m^3)	K_a	K_o	K_p
Compact Granular Fill Granular 'B' (OPSS 1010)	32	21.0	0.31	0.47	3.25
Existing Earth Fill	29	19.0	0.35	0.52	2.88
Glacial Till	32	21.0	0.31	0.47	3.25
Queenston Formation (Bedrock)	28	26.0	n/a	n/a	n/a

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

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

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:

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

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. 77.2 to 75.3 ± 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 nominal excavation and construction progression it should be expected that there will be a crushable layer provided behind the lowest parking level wall. 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 P4 level slab is to be made near Elev. 67 ± m 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.

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 percent 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 spaced at a maximum of 9 metres on centre. The pipes must be surrounded on all sides by a minimum of 100 mm of 19 mm clear stone, 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. Terraprobe recommends 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).

Foundation walls must be damp-proofed in conformance to Section 5.8.2 of the Ontario Building Code (2012). Prefabricated drainage composites, such as Miradrain 2000 (Mirafi) or Terradrain 200 (Terrafix), should be incorporated between the shoring wall 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 storm water sump from the subsurface drainage will be governed largely by the building perimeter drainage collection during rainfall and runoff events. Typical shored 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 water and storm event flows. It is expected that the seepage can be controlled with typical widely available, commercial sump pumps. A hydrogeological report has been prepared by Terraprobe under a separate cover (File No. 1-16-0680-46) to address anticipated ground water flow into the building.

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 is considered a Type 3 soil and the glacial till is a Type 2 soil.

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.

Queenston Formation bedrock is not considered a "soil" under the act. Excavations made in bedrock can be vertical, provided the rock faces are scaled and maintained to preclude the possibility of spalls. Where this is not possible, protective mesh can be draped over the rock face when work is required in the area immediately beside the cut rock face.

The overburden soils can be removed by conventional excavation equipment. Excavations at this site will likely encounter native cobble/boulder obstructions in the glacial till. 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 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 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 are indicated on the rock core logs. Excavating detailed shapes for foundations and the edges of the excavation are normally accomplished with hoe mounted hydraulic rams.

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.

4.2 Shoring Design

The site is generally bounded by Lakeshore Road to the north, Old Lakeshore Road to the east and south, and a commercial building with a paved parking lot to the west.

No excavation shall extend below the foundations of existing adjacent structures without adequate alternative support being provided. Underpinning guidelines are provided as Appendix E. Where excavations cannot be sloped, they can be supported using a shoring system such as soldier piles with lagging or a rigid caisson wall.

Depending on whether the existing building to the west needs to be supported, a rigid shoring system to preserve the integrity and support of the soil beneath the existing foundations of the adjacent building in a state approximating the at-rest condition may be required. The remaining sides of the excavation can likely be supported by soldier piles with lagging.

4.2.1 Earth Pressure Distribution

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

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

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)

For the cohesive glacial till at the site, multi-level supported shoring can be designed based on an earth pressure distribution consisting of a trapezoidal pressure distribution with a maximum pressure defined by:

$$P = 0.8\,K[\gamma H + q]$$

For ground water pressure distribution along the shoring wall in conjunction with the above soil pressures, the ground water level should be taken at Elev. 77.5 ± 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 an interlocking caisson wall. Conventional soldier pile and lagging do not experience the ground water pressures, as water is allowed to drain freely through the wall.

4.2.2 Caisson and Soldier Pile Toe Design

Caisson and soldier pile toes will be made in sound bedrock of the Queenston Formation. The factored vertical bearing capacity for the design of a pile, embedded in the sound bedrock, is 10 MPa. The factored lateral bearing capacity of the sound rock is 1 MPa.

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.

4.2.3 Shoring Support

Where the excavation penetrates the bedrock, the rock excavation is nominally self-supporting in a vertical face, provided the rock bedding is horizontally oriented. Anchors made in bedrock of the Queenston Formation may be designed using a working adhesion of 550 kPa. A proto-type anchor must be made to demonstrate the anchor capacity (performance tested). All production anchors must be proof-tested to 133% of the design load, to validate the design assumptions.

4.3 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.

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.

4.4 Quality Control

The proposed structure will be founded on conventional spread footings. The foundation installations must be reviewed in the field by the geotechnical engineer 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 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 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 both 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, 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.

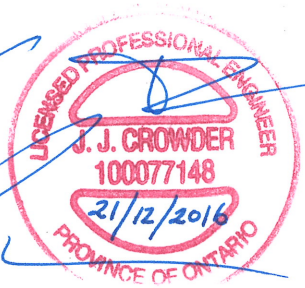
5.3 Use of Report

This report is prepared for the express use of Carriage Gate Homes 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. Carriage Gate Homes and their retained design consultants are authorized users.

It is recognized that The City of Burlington, in their 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.

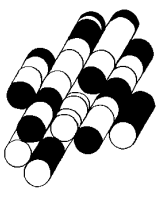


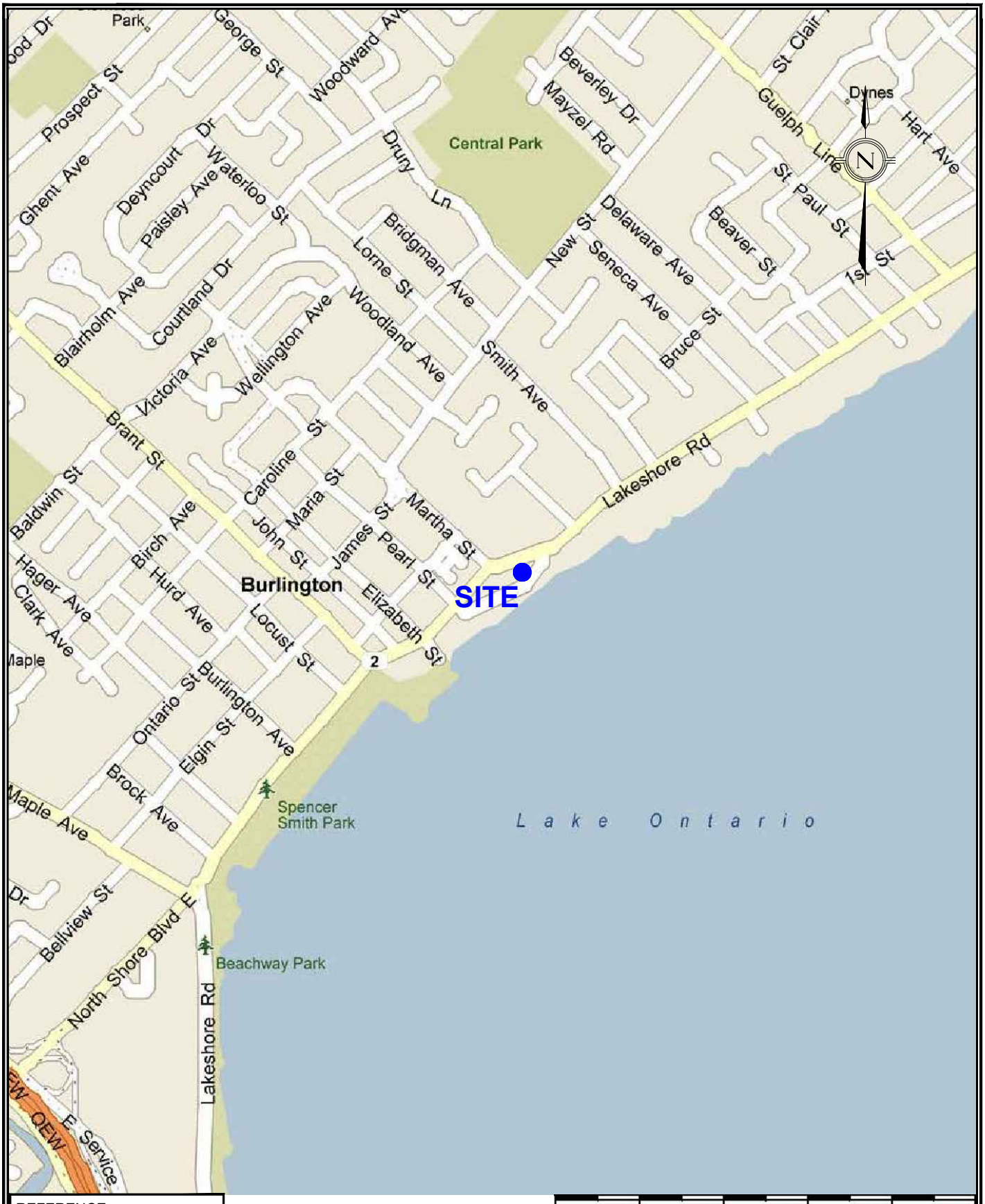
Russell Wiginton, B.Sc.(Eng.), EIT

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

FIGURES

TERRAPROBE INC.





REFERENCE
Microsoft Streets & Trips 2012

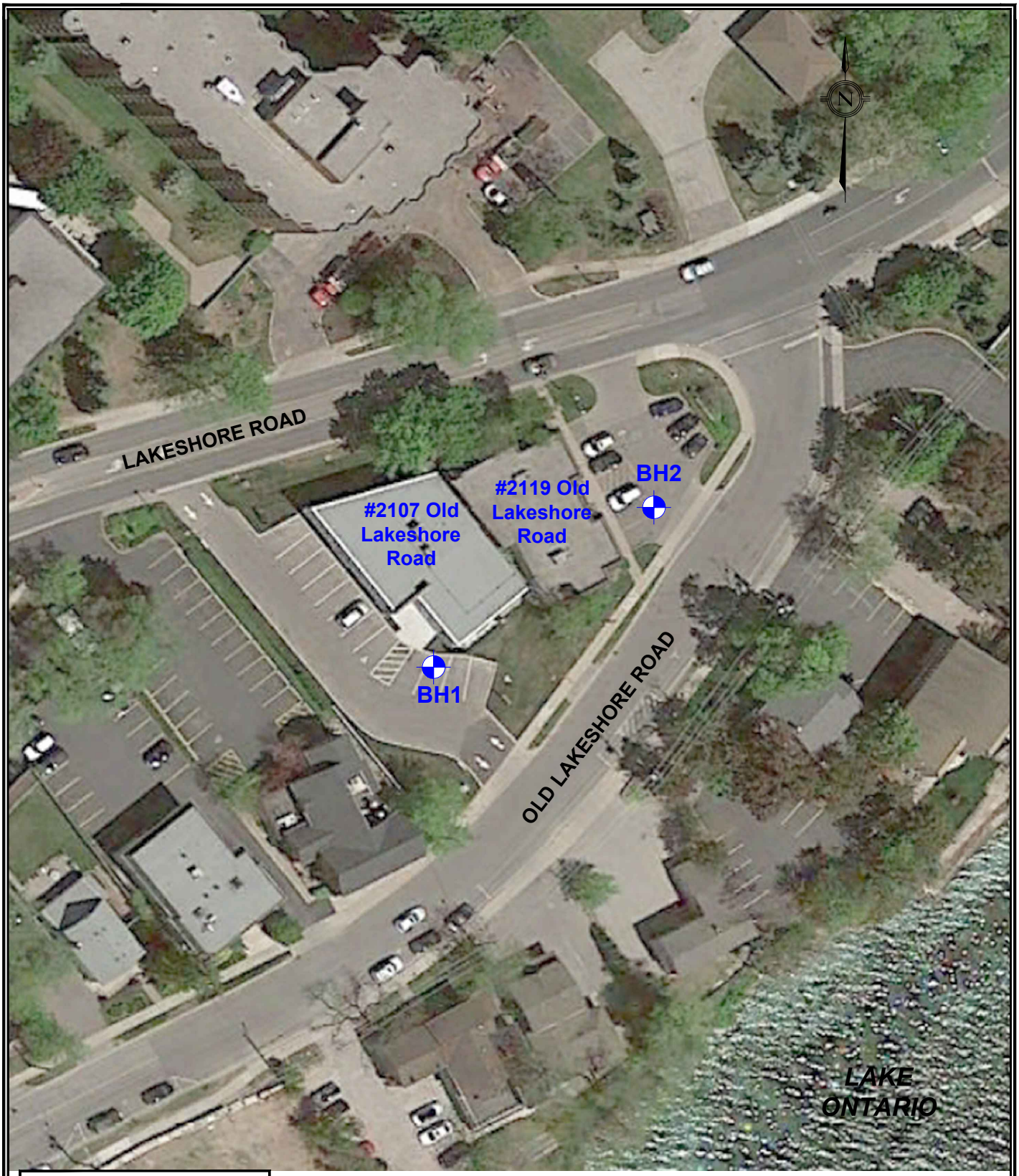
Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title: SITE LOCATION PLAN

File No. 1-16-0680-01

FIGURE : 1

Z:\1-Project Files\2016\1-16-0680 - SW of Old Lakeshore & Lakeshore Roads, Burlington\01-GEO Invest\A. Dwg, Logs\AUTOCAD\1-16-0680-01 FIG.dwg, Sandy



LEGEND

Approximate Borehole Location

SCALE 1:750

10 8 6 4 2 0 10m

REFERENCE
Google Earth 2015

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

Title: BOREHOLE LOCATION PLAN

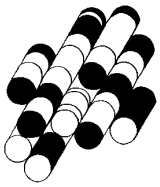
File No. 1-16-0680-01

FIGURE : 2

Z:\1-Project Files\2016\1-16-0680 - SW of Old Lakeshore & Lakeshore Roads, Burlington\01-GEO Invest\A. Dwg, Logs\AUTOCAD\1-16-0680-01 FIG.dwg, Sandy

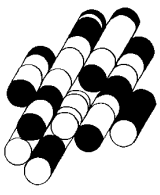
APPENDICES

TERRAPROBE INC.



APPENDIX A

TERRAPROBE INC.



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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	

Terraprobe

LOG OF BOREHOLE 1

Project No. : 1-16-0680-01 | Client : Carriage Gate Homes | Originated by : TG

Date started : November 25, 2016 | Project : SW of Old Lakeshore & Lakeshore Rd | Compiled by : NN

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

Position : E: 597988, N: 4797784 (UTM 17T) | Elevation Datum : Geodetic (NAD83) | 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
0	81.5	**GROUND SURFACE**				
	81.3	50mm **ASPHALTIC CONCRETE**				
	0.2	150mm **AGGREGATE**				
		CLAYEY SILT, trace to some sand, trace to some gravel, very stiff to hard, mottled greyish brown (GLACIAL TILL)		1	SS	20
1		...at 0.8 m, reddish brown		2	SS	88
				3	SS	63
2	79.2					
	2.3	**INFERRED BEDROCK**, shale, red, weathered		4	SS	50 / 125mm
	78.9					
	2.6	**QUEENSTON FORMATION** (See rock core log for details)		1	RUN	
3				2	RUN	
4						
5				3	RUN	
6						
7				4	RUN	
8						
9				5	RUN	
10				6	RUN	

Elevation Scale (m): 81, 80, 79, 78, 77, 76, 75, 74, 73, 72

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

Headspace Vapour (ppm)

Instrument Details — Unstabilized Water Level

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

...at 1.5m, augers grinding

(continued next page)

LOG OF BOREHOLE 1

Terraprobe

Project No. : 1-16-0680-01	Client : Carriage Gate Homes	Originated by : TG
Date started : November 25, 2016	Project : SW of Old Lakeshore & Lakeshore Rd	Compiled by : NN
Sheet No. : 2 of 2	Location : Burlington, Ontario	Checked by : JC

Position : E: 597988, N: 4797784 (UTM 17T) | Elevation Datum : Geodetic (NAD83) | 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	Elevation Scale (m)
		(continued)					
11		**QUEENSTON FORMATION** (See rock core log for details) *(continued)*		6	RUN		71
12				7	RUN		70
13				8	RUN		69
14				9	RUN		68
15							67
16				10	RUN		66
	64.6						65
	16.9	**END OF BOREHOLE**					

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

Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level

Lab Data and Comments

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

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 8, 2016	4.3	77.2

file: 1-16-0680-01 bh logs.gpj

Terraprobe

ROCK CORE LOG 1

Project No. : 1-16-0680-01 | Client : Carriage Gate Homes | Originated by : TG

Date started : November 25, 2016 | Project : SW of Old Lakeshore & Lakeshore Rd | Compiled by : NN

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

Position : E: 597988, N: 4797784 (UTM 17T) | Elevation Datum : Geodetic (NAD83) | Core Diameter : **HQ**, OD=96mm, ID=64mm

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

GENERAL DESCRIPTION

Rock coring started at 2.6m below grade

QUEENSTON FORMATION

Shale, red, laminated, weak; joints are horizontal, gapped to closed, unaltered;

interbedded with **limestone**, light grey, laminated to thinly bedded, medium strong

Run 1 : 0% limestone 100% shale

Run 2 : 30% limestone 70% shale

Run 3 : 13% limestone 87% shale

Run 4 : 28% limestone 72% shale

Run 5 : 23% limestone 77% shale

Run 6 : 8% limestone 92% shale

Run 7 : 24% limestone 76% shale

Run	Elev Depth (m)	Recovery	Elevation (m)
	78.9		
	2.6		
R1		TCR = **97%** SCR = **77%** RQD = **47%**	
	78.1		
	3.4		78
R2		TCR = **100%** SCR = **100%** RQD = **76%**	
	76.6		77
	4.9		
R3		TCR = **89%** SCR = **86%** RQD = **51%**	76
	75.0		
	6.5		75
R4		TCR = **100%** SCR = **100%** RQD = **99%**	
	73.5		74
	8.0		
R5		TCR = **100%** SCR = **100%** RQD = **99%**	73
	72.0		
	9.5		72
R6		TCR = **100%** SCR = **97%** RQD = **56%**	
	70.5		71
	11.0		
R7		TCR = **97%** SCR = **93%** RQD = **74%**	70
	68.9		
	12.6		69

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
10
2
3
5
1
1
0
5
>10
1
>10
5
0
0
0
2
0
1
0
1
2
0
2
0
0
0
10
5
3
10
1
1
2
0
0
1

Laboratory Testing

Comments

2.6-2.8m: *angular blocks*
2.8m: *transition to sound rock*

3.1m: *vertical fracture, clean, gapped, planar, smooth*
3.2-3.2m: *rubblized*

5.1m: *vertical fracture, clean, gapped, planar, rough*

5.4m: *rubblized*
5.5m: *dipping fracture, clean, gapped, planar, smooth*

5.9-6.1m: *core loss*

6.4m: *vertical fracture, clean, gapped, planar, rough*

9.8m: *rubblized*

11.2-11.3m: *rubblized*

(continued next page)

ROCK CORE LOG 1

Terraprobe

Project No. : 1-16-0680-01	Client : Carriage Gate Homes	Originated by : TG
Date started : November 25, 2016	Project : SW of Old Lakeshore & Lakeshore Rd	Compiled by : NN
Sheet No. : 2 of 2	Location : Burlington, Ontario	Checked by : JC

Position : E: 597988, N: 4797784 (UTM 17T) | Elevation Datum : Geodetic (NAD83) | Core Diameter : **HQ**, OD=96mm, ID=64mm

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

Depth (m)	GENERAL DESCRIPTION	Run / Elev Depth (m)	Recovery	Elevation (m)	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	Natural Fractures: Spacing	Laboratory Testing	Comments	Elevation (m)
	78.9										
13	**QUEENSTON FORMATION** **Shale**, red, laminated, weak; joints are horizontal, gapped to closed, unaltered;						1				
	interbedded with **limestone**, light grey, laminated to thinly bedded, medium strong	R8	TCR = **98**% SCR = **95**% RQD = **80**%	68			0 4 1				68
14	*Run 8 : 7% limestone* *93% shale*	67.4 14.1					5 4				
		R9	TCR = **100**% SCR = **91**% RQD = **53**%	67			>10 >10			**14.7-14.8m:** *rubblized* **15.0-15.0m:** *rubblized*	67
15	*Run 9 : 16% limestone* *84% shale*	65.9 15.6		66			>10 4 1			**15.6-15.7m:** *vertical fracture and horizontal fracture*	66
16		R10	TCR = **98**% SCR = **89**% RQD = **66**%	65			>10 5 3				65
	Run 10 : 9% limestone *91% shale*	64.6					0				
		16.9m									
	END OF COREHOLE										

file: 1-16-0680-01 bh logs.gpj

LOG OF BOREHOLE 2

Terraprobe

Project No. : 1-16-0680-01	Client : Carriage Gate Homes	Originated by : TG
Date started : November 23, 2016	Project : SW of Old Lakeshore & Lakeshore Rd	Compiled by : NN
Sheet No. : 1 of 1	Location : Burlington, Ontario	Checked by : JC

Position : E: 598019, N: 4797817 (UTM 17T) Elevation Datum : Geodetic (NAD83)

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

Depth Scale (m)	Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value
	79.6	**GROUND SURFACE**				
0	79.4	50mm **ASPHALTIC CONCRETE**				
	0.2	100mm **AGGREGATE**				
		FILL, sand, some silt to silty, trace gravel, trace clay, compact, reddish brown, moist		1	SS	16
1				2	SS	15
	78.1					
	1.5	**CLAYEY SILT**, some sand, trace gravel, hydrocarbon odour, firm, reddish brown, moist (GLACIAL TILL)		3	SS	6
2	77.3					
	2.3	**INFERRED BEDROCK**, shale, red, weathered (QUEENSTON FORMATION)		4	SS	97 / 275mm
3				5	SS	50 / 75mm
4				6	SS	50 / 25mm
5						
6	73.5			7	SS	50 / 25mm
	6.1	**END OF BOREHOLE**				

Elevation Scale (m): 79, 78, 77, 76, 75, 74

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): PID: <5; PID: <5; PID: 3250; PID: 2950; PID: 270; PID: 45

Instrument Details

Unstabilized Water Level

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

2 10 59 29

...at 4.0m, heavy grinding to 4.3m

Unstabilized water level measured at 4.9 m below ground surface; borehole was open upon completion of drilling.

50 mm dia. monitoring well installed.

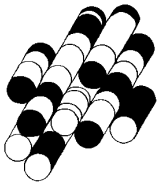
WATER LEVEL READINGS

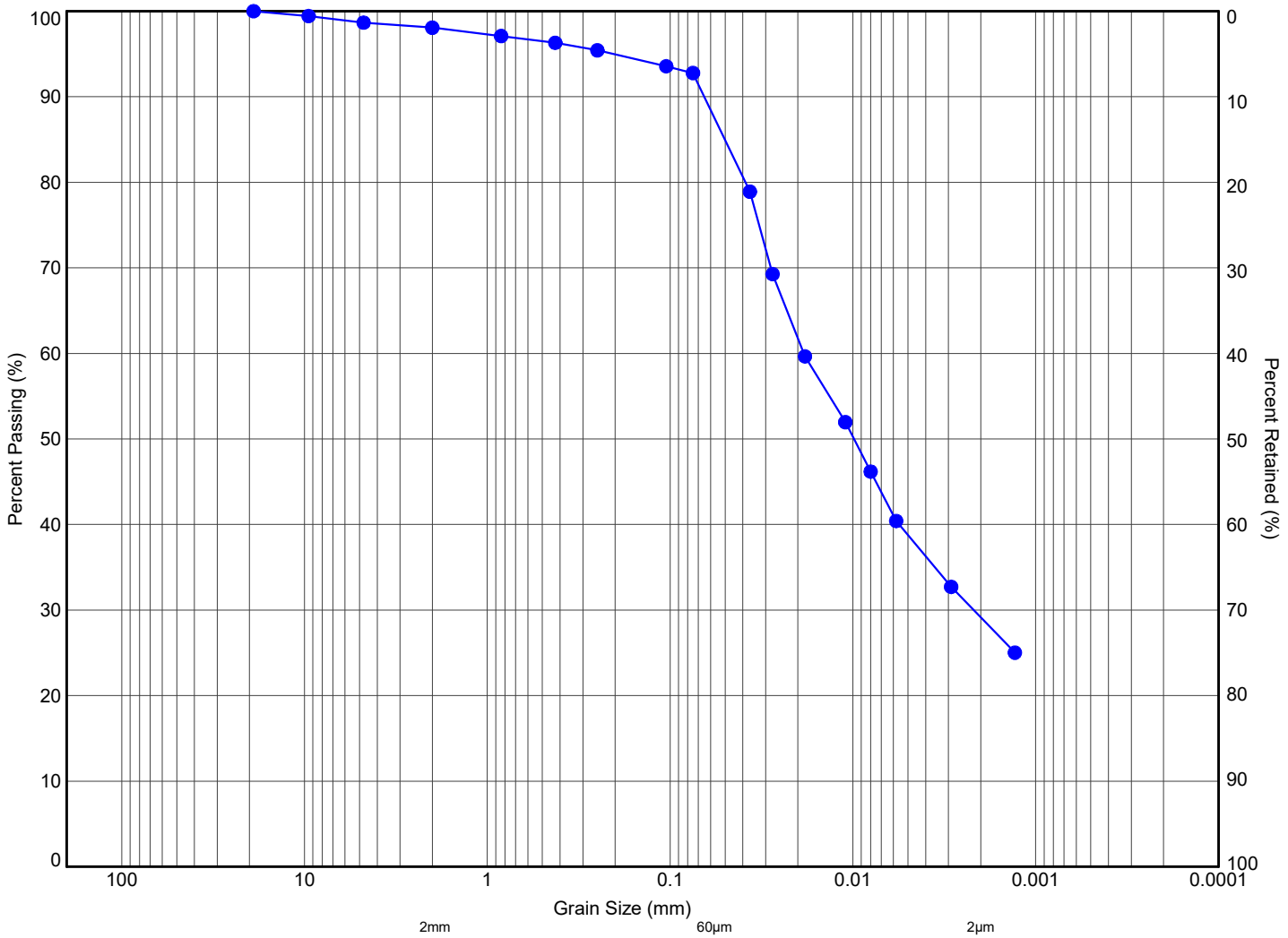
Date	Water Depth (m)	Elevation (m)
Dec 8, 2016	2.2	77.4
Dec 19, 2016	2.1	77.5

file: 1-16-0680-01 bh 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**	SS3	1.8		2	10	59	29	

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Title:

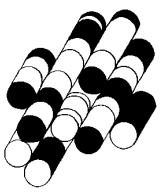
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, SOME SAND, TRACE GRAVEL (GLACIAL TILL)

File No.: **1-16-0680-01**

APPENDIX C

TERRAPROBE INC.



Borehole 1 - Rock Core Photographs



Runs: 1, 2, & 3
Depth: 2.6 to 5.6 m (Elev. 78.9 to 75.9 ± m)



Runs: 3, 4, & 5
Depth: 5.6 to 8.7 m (Elev. 75.9 to 72.8 ± m)



Runs: 5 & 6
Depth: 8.7 to 11.0 m (Elev. 72.8 to 70.5 ± m)



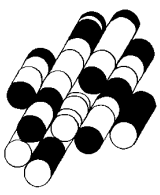
Runs: 7 & 8
Depth: 11.0 to 14.1 m (Elev. 70.5 to 67.4 ± m)

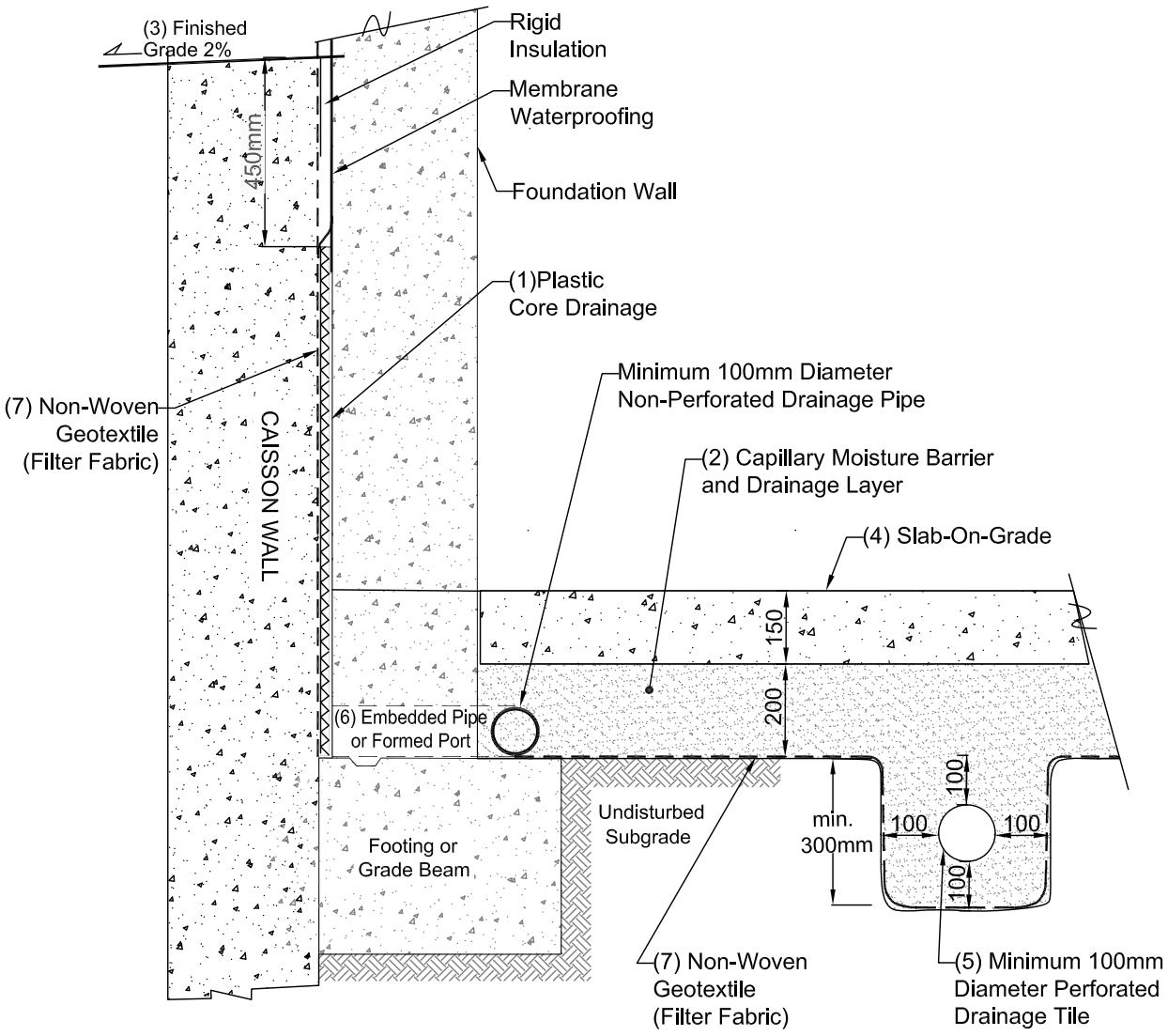


Run: 9 & 10
Depth: 14.1 to 16.9 m (Elev. 67.4 to 64.6 ± m)

APPENDIX D

TERRAPROBE INC.





NOTES

1) Prefabricated drainage panels to consist of Terrafix - TERRADRAIN 200, Mirafi - Miradrain 6000, or 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) vibrated 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 to slope away from wall at a minimum grade of 2%.

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

5) Drainage tile invert to be a minimum of 300mm below underside of the drainage layer. Subdrains to be set along perimeter and in parallel rows, one way, and at a distance recommended in the geotechnical report, as structural foundations permit. Drainage tile should have a minimum 100mm cover on all sides.

6) Embedded pipes/formed ports to be set a distance of 3m on-centre. Do not connect the non-perforated perimeter drainage pipes to the underfloor drainage tiles.

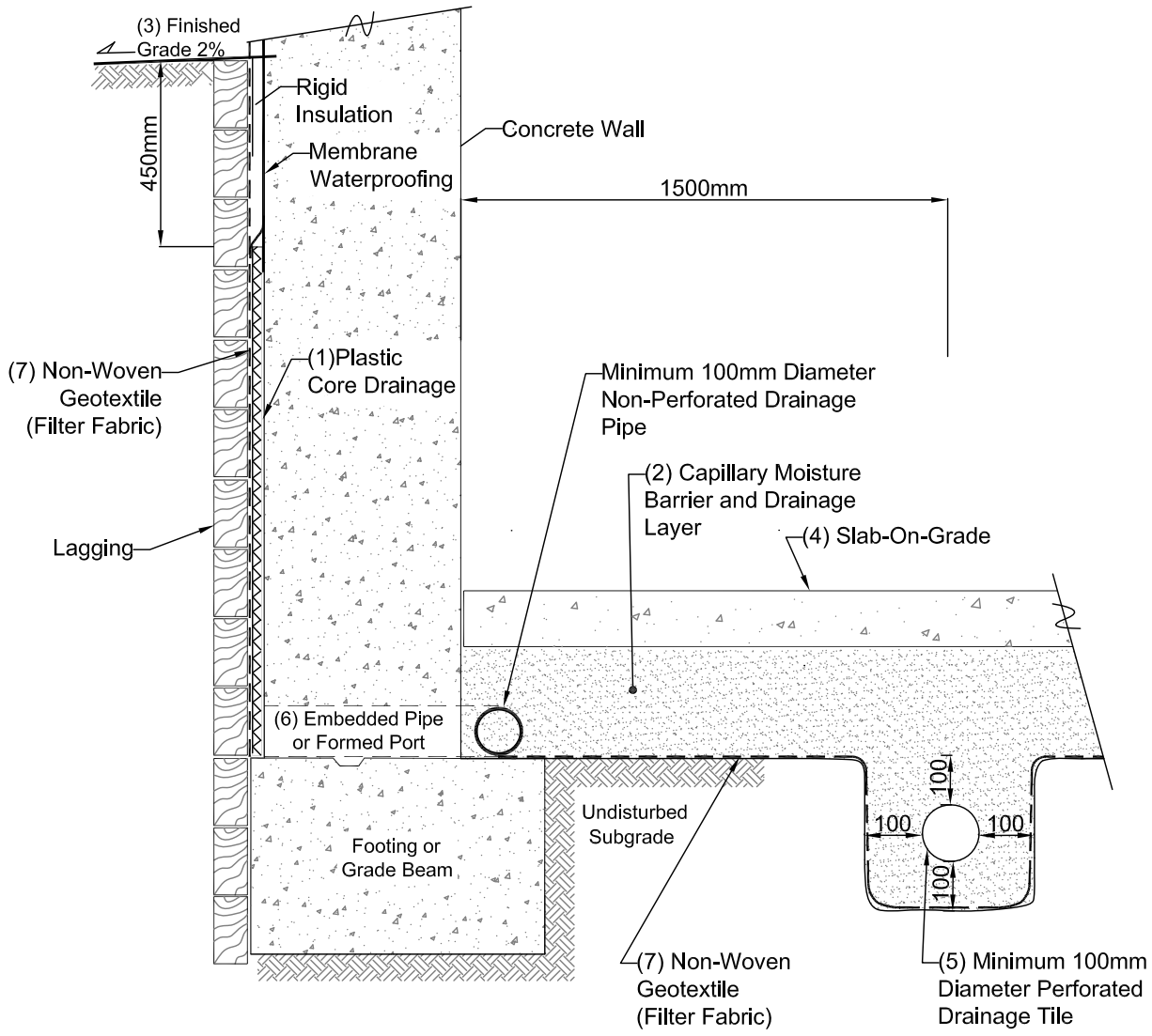
7) When the subgrade consists of cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or equivalent).

N.T.S.

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SCHEMATIC DRAINAGE DETAIL
CAISSON WALL SHORING SYSTEM



NOTES

1) Prefabricated drainage panels to consist of Terrafix - TERRADRAIN 200, Mirafi - Miradrain 2000, or 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) 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 finished grade away from wall at a minimum grade of 2%.

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

5) Drainage tile invert to be a minimum of 400mm below underside of floor slab, to be set in parallel rows, one way, and at a distance of 3m on-centre, as structural foundations permit. Drainage tile should have a minimum 100mm cover on all sides.

6) Embedded pipes/formed ports to be set a distance of 3m on-centre. Do not connect the non-perforated perimeter drainage pipes to the underfloor drainage tiles.

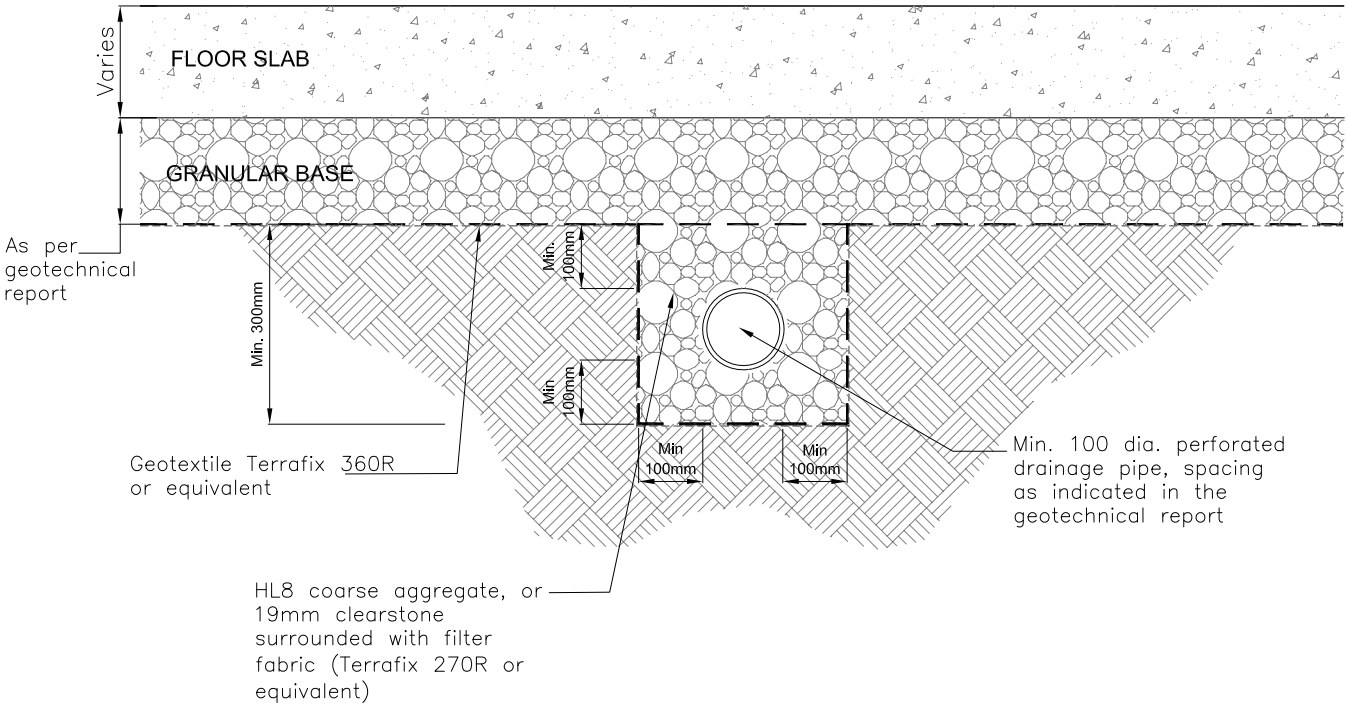
7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or equivalent).

N.T.S.

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Title:

SCHEMATIC DRAINAGE DETAIL
SOLDIER PILE & LAGGING SHORING SYSTEM



Schematic Only
Not to Scale

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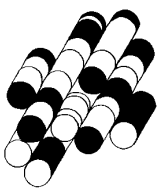
Title:

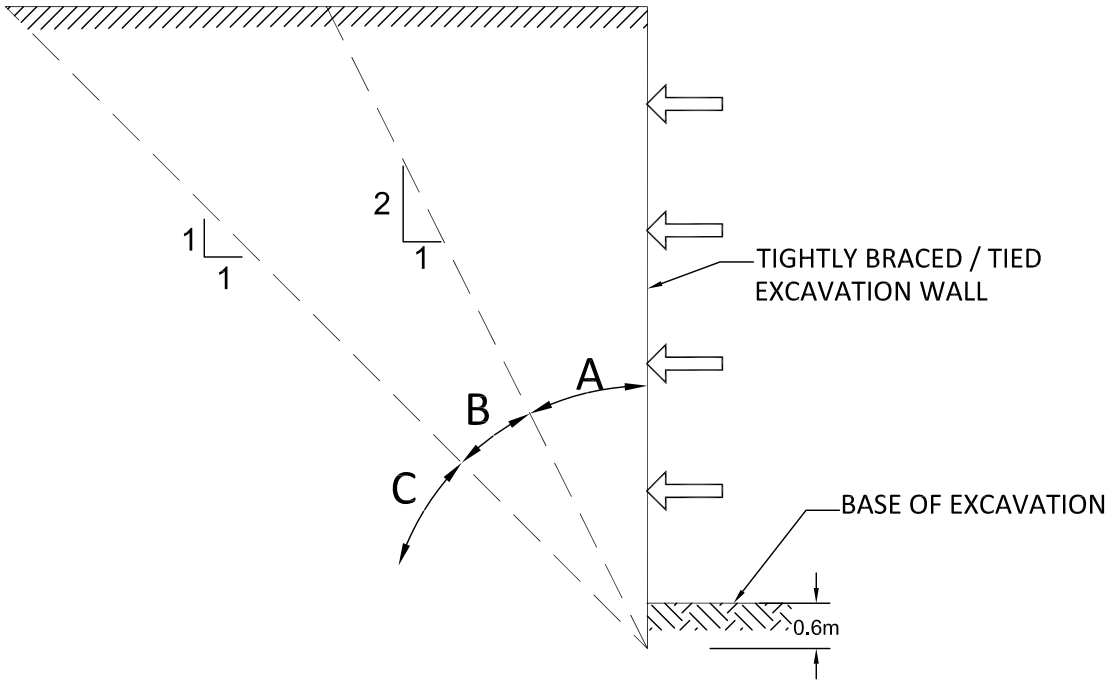
BASEMENT SUBDRAIN DETAIL

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APPENDIX E

TERRAPROBE INC.





Zone A: Foundations within this zone often require underpinning. Horizontal and vertical pressures on excavation wall of non-underpinned foundations must be considered.

Zone B: Foundation within this zone often do not require underpinning. Horizontal and vertical pressures on excavation wall of non-underpinned foundations must be considered.

Zone C: Foundations within this zone usually do not require underpinning.

REFERENCE:

User's Guide - NBC 2005 Structural Commentaries
(Part 4 of Division B) - Commentary K

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Title:

GUIDELINES FOR UNDERPINNING SOILS