Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

PRELIMINARY GEOTECHNICAL INVESTIGATION LAKESIDE VILLAGE PLAZA 5353 LAKESHORE ROAD BURLINGTON, ONTARIO

Prepared For: United Burlington Retail Portfolio Inc.
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File No. 71-15-5010
March 23, 2015

Distribution:

1 copy: United Burlington Retail Portfolio Inc. c/o Glanelm Property Management (pdf)
1 copy: Cynthia Zahoruk Architect Inc. (pdf)
2 copies: Terraprobe Inc., Stoney Creek

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1.0 THE PROJECT

Terraprobe Inc. was retained by Glanelm Property Management on behalf of United Burlington Retail Portfolio Inc. to carry out a geotechnical investigation at the Lakeside Village Plaza located at 5353 Lakeshore Road in Burlington, Ontario. A site location plan is provided as Figure 1. A proposal and cost estimate to carry out the investigation were provided in our letter of November 6, 2014 (revised February 13, 2015). Authorization to proceed with the work was provided by Cynthia Zahoruk Architect Inc. on behalf of United Burlington Retail Portfolio Inc. and Glanelm Property Management on February 10, 2015.

The purpose of the work was to investigate and report on the subsurface soil, rock and ground water conditions in a series of boreholes drilled at the site. Based on this information, advice is provided with respect to the geotechnical aspects of the proposed development, including the design of foundations, floor slabs-on-grade and pavements. The anticipated construction conditions pertaining to excavation, backfill and temporary ground water control are discussed also, but only with regard to how these might influence the design.

2.0 SITE AND PROJECT DESCRIPTION

2.1 Existing Site Conditions

The property is located on the north side of Lakeshore Road, between Kenwood Avenue and Hampton Heath Road. The Site Plan, Figure 2, presents the general arrangement of the property, as derived from a topographic survey of the site prepared by A.T. McLaren Limited, OLS, dated March 5, 2015.

At the time of this investigation, development on the property included a multi-unit commercial retail plaza. The existing parking lot is located on the south and east sides of the plaza. Concrete curbs formed the perimeter of the parking lot. Asphalt driveways from Lakeshore Road, Kenwood Avenue and Hampton Health Road provide primary access to the property. The site topography is relatively flat; however there is a fall of about 1.2 metre in elevation across the 160 m traverse of the site from the north to south. The high ground was at about elevation 82.2 m and the lowest at 81.0 m.

2.2 Site Geology

Based on published geological information for the general area of the site, the near surface overburden soil at and in the vicinity of the subject property consists of glacial till and bedrock of the Queenston Formation.[2] The Queenston Formation consists of a reddish brown shale, interbedded with limestone and calcareous sandstone. The geological mapping and regional well records indicates that the bedrock beneath the site is within 1 to 3 metres of existing grade.[3]

2.3 Proposed Development

Specific details regarding the proposed development and the like are not known at this time, however based on preliminary information provided by Cynthia Zahoruk Architect Inc., it is understood that the development could include 2 to 4 multi-storey buildings with adjoining low rise podiums. It is understood that the buildings are anticipated to be approximately 20 storeys high, with 2 to 3 levels of underground parking.

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[2] *Paleozoic Geology, Hamilton Area, Southern Ontario*; Ontario Division of Mines; Map No. 2336; 1976.

[3] *Bedrock Topography of the Hamilton Area, Southern Ontario*; Ontario Department of Mines; Map No. 2034; 1964.

3.0 PROCEDURE

The field work for this investigation was carried out on February 26 and 27, 2015, during which time twelve (12) boreholes were drilled to depths of about 3.1 to 4.6 metres below the existing ground surface. The locations of the boreholes are shown on the Borehole Location Plan, Figure 2. The results of the boreholes are shown on the Log of Borehole sheets presented in Appendix A.

The boreholes were drilled using a truck mounted power auger supplied and operated by a specialist drilling contractor. The boreholes were advanced using conventional hollow stem continuous flight augers. Standard penetration testing and sampling was carried out within the overburden in all of the boreholes at regular intervals of depth using conventional 50 mm outside diameter split spoon sampling equipment. After the drilling, sampling, and logging was completed, the boreholes were backfilled with auger cuttings and bentonite sealant. The pavement surface at the borehole locations were restored using a commercial grade cold mix asphaltic concrete.

The field work was observed throughout by members of our engineering staff who located the boreholes, arranged for the underground utility clearances at the borehole locations and cared for the samples obtained. The boreholes were located in the field with respect to the existing development features shown on a site survey drawing. The ground surface elevations at the boreholes were inferred from spot elevations shown on a topographic survey of the property prepared by A.T. McLaren Limited, which was understood to have been referred to the geodetic datum.

Ground water observations were made in each borehole during and upon completion of drilling and sampling. No provision was made for long-term ground water monitoring at the site.

All of the samples recovered in the course of the investigation were brought to our Stoney Creek laboratory for further examination and water content determinations. The results of moisture content tests are plotted on the Log of Borehole sheets in Appendix A. Soil quality analyses were not carried out as part of this assignment.

4.0 SUBSURFACE CONDITIONS

The subsurface soil, rock and ground water conditions encountered in the boreholes, and the results of the field and laboratory testing, are shown on the Log of Borehole sheets in Appendix A. A list of abbreviations and symbols are provided to assist in the interpretation of the borehole logs. It should be noted that the boundaries between the strata have been inferred from drilling observations and non-continuous samples. They generally represent a transition from one soil type to another and should not be inferred to represent exact planes of geological change. Further, conditions will vary between and beyond the locations investigated.

4.1 Soil Conditions

The following discussion has been simplified in terms of the major soil strata for the purposes of geotechnical design. In general, the boreholes encountered asphalt pavement and fill, overlying glacial till and weathered shale bedrock.

4.1.1 Asphalt Pavement

All of the boreholes were drilled within an existing parking lot and penetrated about 75 to 100 mm of asphaltic concrete. The asphalt was underlain by 300 to 500 mm of combined granular base and subbase.

4.1.2 Fill

Fill was encountered at most borehole locations and was found to vary in depth from 0.8 to 2.3 metres below existing ground surface. The fill was variable in nature but generally consisted of mixed shale and silt. Some silty sand and gravel fill was encountered at the location of Borehole 15-8. The N values, as determined in the Standard Penetration testing carried out within the fill, ranged from 8 to greater than 100 blows per 0.3m, inferring a loose to very dense state of packing. The range in penetration resistance observed suggests that the fill may not have been constructed as an engineered fill. The in-situ water content of the samples of fill recovered from the standard penetration testing ranged from about 6 to 18 percent.

4.1.3 Glacial Till

Most of the boreholes penetrated a stratum of clayey silt to silt till to depths of about 1.5 to 2.3 metres below existing ground surface. The till can generally be described, in geological terms, as an imbricate embedment of fragments of shale bedrock in a till matrix, thought to have been formed by glacial overriding of the parent (Queenston Formation) bedrock. In most instances the consistency and natural water content of the till was similar to the highly weathered zone of the Queenston Formation. The N values determined for the till ranged from 15 to greater than 100 blows per 0.3 m. The natural water content of the samples of till obtained from the penetration testing ranged from about 8 to 16 percent.

4.1.4 Weathered Shale Bedrock

All of the boreholes were terminated in weathered shale bedrock at depths ranging from about 3.1 to 4.6 metres below ground surface. The N values determined in the shale bedrock were typically greater than 100 blows per 0.3m. The natural water content of the samples of the shale recovered from the standard penetration testing ranged from about 4 to 8 percent. Detailed exploration of the bedrock was not carried out as part of this assignment; however the bedrock beneath the site is known to consist of the Queenston Formation which is comprised of predominantly thinly bedded reddish brown shale of Ordovician age. The shale contains interbeds of green calcareous shale, limestone, sandstone and siltstone.

There is typically a horizontal zone of weathering at the contact between the weak 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 reproduced from Skempton, Davis and Chandler, a typical weathering profile of 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

The augered borehole method used at this site is conventionally accepted investigative practise, however the interval sampling method does not define the bedrock surface with precision, particularly where the surface of the rock is weathered, weaker and easily penetrated by the auger. The change in resistance to augering in between Zones III and II in the shale profile is not profound. The top of rock as indicated on the Borehole Logs from this investigation is to be consistently interpreted as the surface of Zone II in the profile.

The physical properties of the Queenston Formation were presented in the Ontario Ministry of Transportation and Communication publication RR229 - *Evaluation of Shales for Construction Projects-An Ontario Shale Rating System*, 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

4.2 Ground Water

All of the boreholes were dry during and upon completion of drilling. The boreholes penetrated low permeability deposits of glacial till and/or weathered shale which generally precludes the free flow of ground water.

It should be noted that the conditions reported above may not necessarily represent stabilized conditions or the ground water conditions which will be encountered during construction. The ground water levels will vary due to seasonal effects and precipitation conditions.

5.0 DISCUSSION

The following discussion is based on our interpretation of the factual data obtained during this investigation and is intended for the use of the design engineer only. Comments made regarding the construction aspects are provided only in as much as they may impact on design considerations. Contractors bidding on or undertaking any work at the site should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, equipment capabilities, costs, sequencing and the like.

It is understood that 2 to 4 multi-storey buildings are proposed for the site. The buildings are expected to be underlain by up to 2 to 3 levels of underground parking. It should be noted that some of the details regarding the proposed development were of a conceptual nature when this report was prepared. For this reason, the geotechnical aspects of this project should be reviewed by this office at the final design stage.

5.1 Building Foundations

Buildings with at least one basement level structure will penetrate the shale bedrock of the Queenston Formation. Shallower foundations may be founded on the undisturbed glacial till. The existing fill is unsuitable for the support of building foundations.

Where the undisturbed glacial till deposit coincides with the foundation level appropriate for a structure, then they can be supported on conventional spread footing foundations. Foundations designed to bear on the undisturbed glacial till stratum can be designed using a factored bearing resistance at Ultimate Limit States (ULS) of 300 kPa and a bearing reaction of 200 kPa at Serviceability Limit States (SLS). The settlement of spread footings established in the glacial till at this design bearing resistance is expected to be less than 20 mm.

Spread footing foundations bearing on the weathered shale bedrock of the Queenston Formation can be designed for a bearing reaction of 3000 kPa (SLS). The estimated settlement is less than 10 mm under full design loading. The factored bearing resistance at ULS can be taken as at least 4500 kPa for limit design purposes. Higher design bearing resistances are feasible; however any change to the design bearing resistance given above should be discussed with our office.

It is recommended that the minimum footing width for strip footings be 500 mm, and a minimum footing width of 900 mm be used for the design of spread footings, regardless of the structural requirements. Footings stepped from one level to another should be at a slope not exceeding 7 vertical to 10 horizontal.

Footings exposed to freezing temperatures must be provided with at least 1.2 metres of earth cover for frost protection or equivalent insulation. Where there are two or more levels of underground space, it is generally accepted practice to provide a minimum of 450 mm earth cover for perimeter footings and a minimum of 900 mm of earth cover for interior column foundations in “unheated” parking garages. Only in the immediate area of ventilation shafts is 1.2 metres of earth cover or equivalent insulation used.

5.2 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 meters of the site stratigraphy, where shear wave velocity (v_s) measurements have been taken. Alternatively, the classification is estimated on the basis of rational analysis of undrained shear strength (s_u) or penetration resistance (N-values).

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

Shear wave velocity **Undrained shear strength** **SPT N-values**

Based on the above noted information, it is recommended that the site designation for seismic analysis be ‘Site Class C’, 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				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) \geq 1.25$
C	1.0	1.0	1.0	1.0	1.0
Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.20$	$S_a(1.0) = 0.30$	$S_a(1.0) = 0.40$	$S_a(1.0) \geq 0.50$
C	1.0	1.0	1.0	1.0	1.0

It should be noted that the seismic site designation above is estimated on the basis of rational analysis of penetration resistance (N-Values) observed in the boreholes (assuming the consistency for the deeper soil stratigraphy beyond the investigation depth was similar to that of the lowest soil strata penetrated within the investigation depth).

Consideration may be given to conducting a site specific Multichannel Analysis of Surface Waves (MASW) at this site to determine the average shear wave velocity in the top 30 metres of the site stratigraphy. An improved seismic site designation (Site Class B) may be possible.

5.3 Earth Pressure Design Considerations

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 (Rankin)	dimensionless
K_o	at-rest earth pressure coefficient (Rankin)	dimensionless
K_p	passive earth pressure coefficient (Rankin)	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	γ	Ka	Ko	Kp
Compact Granular Fill Granular 'B' (OPSS 1010)	32	21.0	0.31	0.47	3.25
Clayey Silt or Similar Fill	30	19.0	0.33	0.50	3.00
Glacial Till	34	19.0	0.28	0.44	3.54
Queenston Formation	28	25	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:

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

where,
P = the horizontal pressure at depth, **h** (m)
K = the earth pressure coefficient,
h_w = the depth below the ground water 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)

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

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

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

Special consideration must be given to structures with sufficient depth that the below grade space would penetrate the bedrock. Excavated bedrock from the Queenston Formation must not be used to backfill structures, firstly because it is unlikely that it can be compacted adequately, and secondly, because there are zones of the Queenston Formation that contain Illite, an expansive clay material. For better grade integrity, the use of imported granular materials is recommended to backfill the structures.

If a structure is made such that it is cast directly into an excavation in the bedrock, the empirical approach for the design of foundation walls below bedrock level has been to use a uniform pressure distribution which is consistent with the maximum earth pressure calculated for the lowest level of soil in the profile. This approach is likely conservative in that the rock is effectively self-supporting, but is recognizes the practical requirement to have a foundation wall of a consistent width through the lower reach of the building.

This approach does not recognize the potential for pressures on the basement walls due to the presence of locked-in horizontal stresses that are relieved when a rock cut is made. It is expected that there would be a sufficient time between the cutting of the rock face and construction of the building structure to allow the rock to de-stress. There is not much documented experience with stress relief and swell in the Queenston Formation. It is know that there are potentially locked in horizontal stresses. Excavations in Toronto in the Georgian Bay Formation, a similar shale deposit, experience stress relief and swell for three to four months when cut. If there is not sufficient time lag between the excavation and the subsequent placing of footings, walls and the suspended slab levels, then special provisions need to be made with respect to time dependent rock stress relief in the design of basement walls of structures.

Where pits are made for sumps and elevators, or other such features which are incorporated within the major excavation, there must also be consideration of the potential for rock squeeze effects if the pits are to be cast directly against the rock face. For such structures, a compressible layer can be placed between the rock and the concrete or alternatively the local structures can be over-excavated and backfilled.

5.4 Slab on Grade Design Parameters

Depending on the final design and site grading, the subgrade at the lowest floor level in the structures could consist of existing fill, glacial till or weathered shale bedrock of the Queenston Formation, all of which are capable of supporting a conventional lightly loaded slab on grade. The moduli of subgrade reaction appropriate for slab on grade design on the aforementioned soils or rock are as follows:

- Existing Fill: 15,000 kPa/m
- Glacial Till: 30,000 kPa/m
- Queenston Formation: 80,000 kPa/m

Any subgrades consisting of existing fill must be proof-rolled and inspected by the geotechnical engineer, prior to the placement of an aggregate base. Subgrade compaction or proof-rolling is not recommended for areas of undisturbed native soils as the compaction efforts will have a tendency to disturb these otherwise competent soils. If any soft or weak subgrade areas are identified, or if there are areas containing excessive amounts of deleterious/organic material or moisture, they must be locally sub-excavated and backfilled with approved clean earth fill or Granular B (OPSS 1010) compacted to a minimum of 96 percent of standard Proctor maximum dry density.

Any buildings with below grade space must have both perimeter and subfloor drainage. The subfloor drainage system is made by placing the slab on a minimum 200 mm layer of 19 mm stone (OPSS 1004) compacted by vibration to a dense state. The nominal spacing of subdrains, in the order of 8 to 10 metres is expected to provide an adequate avenue for the removal of water beneath the slab. It is recommended that the building storm sump systems anticipate a flow from the subfloor drainage of up to 75 litres/minute and that a duplex pumping arrangement on emergency power be provided. This flow is not anticipated to be a sustained flow but could be achievable under storm conditions. Sustained flow to the building sump from the perimeter and subfloor drainage could be as little as a few litres per minute. The pump capacity must be reviewed when the volume of flow from the site has been assessed during excavation.

All slabs on grade should be structurally separate from foundation walls and columns. Saw cut control joints should be incorporated into the slabs along column lines and at regular intervals. Interior load bearing walls should not be founded on the slab but on spread footings as outlined above.

The soil at this site is susceptible to frost effects which would have the potential to deform hard landscaping adjacent to the building. At locations where buildings are expected to have flush entrances, care must be taken in detailing the exterior slabs / sidewalks, providing insulation / drainage / non-frost susceptible backfill to maintain the flush threshold during freezing weather conditions.

5.5 Site Servicing

It is expected that site services will consist of storm and sanitary sewers and watermains, with relatively shallow inverts (less than 3 m). The invert elevation may be within the glacial till or the weathered shale strata. Excavations for underground services should be made as outlined in Section 6.1 of this report.

The locations and depths of any building foundations which would potentially be affected by the proposed utilities should be identified prior to commencing the excavation.

5.5.1 Pipe Bedding

The bedding materials should be adequately compacted to provide support and protection to the service pipes. Provided the base area for the sewer pipes and watermain are free of all soft and deleterious materials, the pipe bedding should comply with a Class B bedding configuration as per the requirements of OPSD 802.030 (rigid pipe) and/or OPSD 802.010 (flexible pipe). Where disturbance of the trench base has occurred, due to the presence of soft fine-grained soils, groundwater seepage and the like, the disturbed soils should be sub-excavated and replaced with suitably compacted granular fill. If standing water is present in the base of the service and watermain trenches then High Performance Bedding (HPB) and/or HL6 clear stone wrapped in geo-textile may be adopted as bedding material below the pipe to provide stabilization.

All granular bedding must be compacted to a minimum of 96 percent of standard Proctor maximum dry density or compacted by vibration to a dense state in the case of HPB or clear stone bedding.

5.5.2 Backfill

The excavated shale from the Queenston Formation is extremely difficult to place and compact successfully because even large compaction equipment does not necessarily break down the shale into a homogeneous mass. It has been our experience that compacted shale fills will intrinsically settle, regardless of the best efforts made in construction and control. In consideration of this, a better quality of construction and grade integrity would be achieved if or imported fill is used to backfill trenches and provide site grading in areas to be hard surfaced. Use of excavated shale in landscaping areas where the integrity of the finished grade level is not important can be considered.

If narrow trenches are constructed in areas where the subgrade integrity is important, then use of compacted granular fill is recommended for backfill. All backfill materials should be placed in 300 mm thick lifts with each lift uniformly compacted to at least 95 percent of standard Proctor maximum dry density. The upper 1 metre of backfill beneath areas to be developed as pavements should be compacted to 98 percent of standard Proctor maximum dry density.

5.6 Pavement Design

All deleterious soil should be stripped and the existing asphalt removed from areas to be developed for new pavements. It is recommended that the subgrade be cut as cleanly as possible to minimize disturbance and be proof rolled with a static roller to identify any loose or disturbed areas. The preparation of the subgrade and the compaction of all fills should be monitored at the time of construction.

If fill is required to raise the grade, there may by some select on-site fill which could be used, provided it is free of topsoil and other deleterious material, and is at a suitable placement water content. The fill should be placed in large areas where it can be uniformly compacted in 300 mm thick lifts with each lift uniformly compacted to at least 95 percent of standard Proctor maximum dry density. The upper 1 m of backfill beneath areas to be developed as pavements should be compacted to 98 percent of standard Proctor maximum dry density.

The final subgrade should be free of depressions and sloped (preferably at a minimum grade of two percent) to promote subgrade drainage. Effective drainage of the granular base and subbase should be achieved by properly filtered subgrade drains at the catch basin locations and along curb lines. The locations and extent of subgrade drainage required within the pavement areas should be reviewed by Terraprobe at the final design stage and verified during construction.

5.6.1 Asphalt Concrete Pavement Structure

Two alternative approaches to pavement design can be considered. A minimal pavement design is provided, which will provide adequate service for 8 to 10 years before significant maintenance and rehabilitation will be required.

Minimal Asphaltic Concrete Pavement Structure

Pavement Layer	Compaction Requirements	Car Parking Minimum Component Thickness	Truck Traffic Minimum Component Thickness
Surface Course Asphaltic Concrete HL3 (OPSS 1150)	93% Maximum Relative Density (OPSS 310)	65 mm	40 mm
Base Course Asphaltic Concrete HL8 (OPSS 1150)	93% Maximum Relative Density (OPSS 310)	N/A	60 mm
Base Course: 19mm Crusher Run Limestone *	98% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular B Type II (OPSS 1010) or 50mm Crusher Run Limestone	98% Standard Proctor Maximum Dry Density (ASTM-D698)	300 mm	300 mm

The cost of the minimal design should be compared to a more substantial performance design which could be expected to last about twice as long before significant maintenance and rehabilitation.

Performance Asphaltic Concrete Pavement Structure

Pavement Component	Compaction Requirements	Light Duty	Heavy Duty
Surface Course Asphaltic Concrete HL3F (OPSS 1150)	93% Maximum Relative Density (OPSS 310)	40 mm	40 mm
Base Course Asphaltic Concrete HL8 (OPSS 1150)	93% Maximum Relative Density (OPSS 310)	50 mm	80 mm

Pavement Component	Compaction Requirements	Light Duty	Heavy Duty
Granular Base Course: Granular 'A' (OPSS 1010) or 19 mm Crusher Run Limestone	98 percent of standard Proctor maximum dry density (ASTM-D698)	150 mm	150 mm
Granular Subbase Course: Granular B Type II (OPSS 1010) or 50 mm Crusher Run Limestone	98 percent of standard Proctor maximum dry density (ASTM-D698)	400 mm	400 mm

Some adjustment to the thickness of the granular subbase material may be required depending on the condition of the subgrade at the time of the pavement construction. The need for such adjustments can be best assessed by the geotechnical engineer during construction.

It is recommended that the placement of the wearing surface be delayed for at least one year after construction of the binder course to minimize the effects of post construction settlement of underground service backfill and the like. Prior to placing the wearing surface, the binder course should be evaluated and remedial work carried out as required in preparation for final construction.

6.0 DESIGN CONSIDERATIONS FOR CONSTRUCTABILITY

6.1 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 at this site, the existing fill is considered a Type 3 Soil, provided that effective ground water control is achieved where required and surface water is directed away from open excavations. The glacial till should be considered a Type 2 Soil.

Where workers must enter a trench or excavation the soil must be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The regulation stipulates safe 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 Sections 235 through 238 and 241 of the Act and Regulations and include provisions for timbering, shoring and moveable trench boxes.

Past experience in bedrock belonging to the Queenston Formation indicates that the till-shale transition zone and the underlying rock can be removed without blasting. The upper weathered portion of the shale bedrock can be removed with powerful excavators equipped with rock teeth. The removal of the underlying bedrock and the interbedded limestone and siltstone layers, however, will be more arduous and time consuming, and will require specialized rock buckets, with the assistance of hoe rams and other similar tools. Line drilling may be required, but likely only in sensitive areas such as where excavations are in close proximity to existing structures where it is necessary to limit over-break and vibrations. The relative ease/difficulty in excavation of bedrock will also depend on the size (width) and depth of the excavation. For example, excavation of narrow trenches into the shale will obviously be more difficult than forming large open cuts for structures in which excavators can operate from the base of the cut to 'pry' up hard layers. Some of the thicker "hard" layers may require additional hoe-ramming effort to remove, but this is not considered unusual for rock excavations in this area. The risk and responsibility for these issues must be addressed in the contract documents for foundations, excavation, and shoring contractors.

Excavations into the shale bedrock can be vertical to near-vertical, but not overhanging. The face of the excavation must be scaled of any loose rock to protect the workers working in the excavation. 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 shale (as well as the limestone, sandstone and siltstone interbeds) are transected by sub-vertical fractures, typically oriented at right angles to one another, and these fractures may preferentially cause the formation of slabs, wedges or blocks of loose unstable rock which will require removal and can often lead to ‘over-break’ beyond the intended excavation ‘cut line’.

In the bedrock, ground water seepage is expected to occur continuously through bedding planes, fractures, and along the more pervious sandstone and limestone interbeds. It should be possible to handle the infiltration by means of ditches and filtered sumps, fitted with appropriately sized pumps within the base of the excavation, as the excavation progresses. Be aware that fouling of pumps by powdery shale fines is sometimes a problem as is accumulation of suspended soil and rock fine particles in the ponded seepage water.

During excavation, gas monitoring should be carried out to ensure that any natural gas detected emanating from the bedrock has been adequately vented and dissipated.

It should be noted that surplus excavated soil resulting from the construction that is to be disposed of off-site, will require chemical analyses to assess the disposal site requirements. No chemical analyses of soil were carried out as part of this assignment. It should be noted however that sites accepting fill usually have aesthetic, or engineering property requirements, as well as chemical requirements for soil acceptance. Such requirements are site specific, so assessment of the appropriateness of the soil from this site for use at other locations was beyond the scope of the investigation.

6.2 Shoring Design

The depth of the overburden soil at this site varies from about 0.8 to 2.3 metres below existing grade. As such, there is very little soil to be retained at the edges of the proposed excavations. The extent to which the municipality will allow sloping of the soil at the edge of the site will dictate whether any shoring must occur. If a straight edge excavation is required above the top of the shale bedrock, then a system of steel soldier piles and lagging pinned into the top of the rock and anchored would be the appropriate solution. Terraprobe can provide shoring design services for this project if requested.

Rock anchorages are typically made in the Queenston Formation using a design bond stress of 620 kPa, without proof testing. Higher bond stresses are possible but proof testing of anchorages on a site by site basis is required. The use of soil anchors established in the overburden soil is not recommended because of the overburden character changes in short distances.

6.3 Site Work

The soil at this site is fine-grained and may become weakened or loosened when subjected to construction traffic. If site work is carried out during periods of wet weather, then it can be expected that the subgrade will be disturbed unless an adequate granular working surface is provided to protect the integrity of the subgrade soils from construction traffic. The disturbance caused by the traffic can result in the removal of disturbed soil and use of fill material for site restoration or underfloor fills that is not intrinsic to project requirements.

The timing of the major grading works on the site is critical to the performance of the work. It may not be feasible to carry out fill operations during wet or freezing conditions. The schedule must provide adequate time to complete the work, allowing for delays due to adverse weather.

The subgrade at this site is considered to be frost susceptible. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the exposed soil will be required. Consideration must be given to frost effects, such as heave or softening, on exposed soil surfaces in the context of this particular project development.

6.4 Quality Control

The foundation installations must be field reviewed by Terraprobe as they are constructed to ensure that the founding soil and/or rock exposed is consistent with the design bearing intended by the geotechnical engineer. 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.

The long term performance of the pavement structure and any slab-on-grade structures are 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. In situ determinations of density during fill and asphaltic 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 A23.1-09. 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 W178.1-14

7.0 LIMITATIONS AND USE OF REPORT

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.

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 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. Since the project is still in the design stage, all aspects of the project relative to the subsurface conditions cannot be anticipated. 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 to the revised project or complete. Terraprobe should be retained to review the implications of changes with respect to the contents of this report and should be retained to review design drawings and specifications prior to construction. It would likely be worthwhile to redraft this report once final building plans have been established so that the excavation requirements can be more specifically addressed.

This report was prepared for the express use of the United Burlington Retail Portfolio Inc., Glanelm Property Management 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. United Burlington Retail Portfolio Inc., Glanelm Property Management 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 the foregoing information is sufficient for your present requirements. If you have any questions, or if we can be of further assistance, please do not hesitate to contact us.

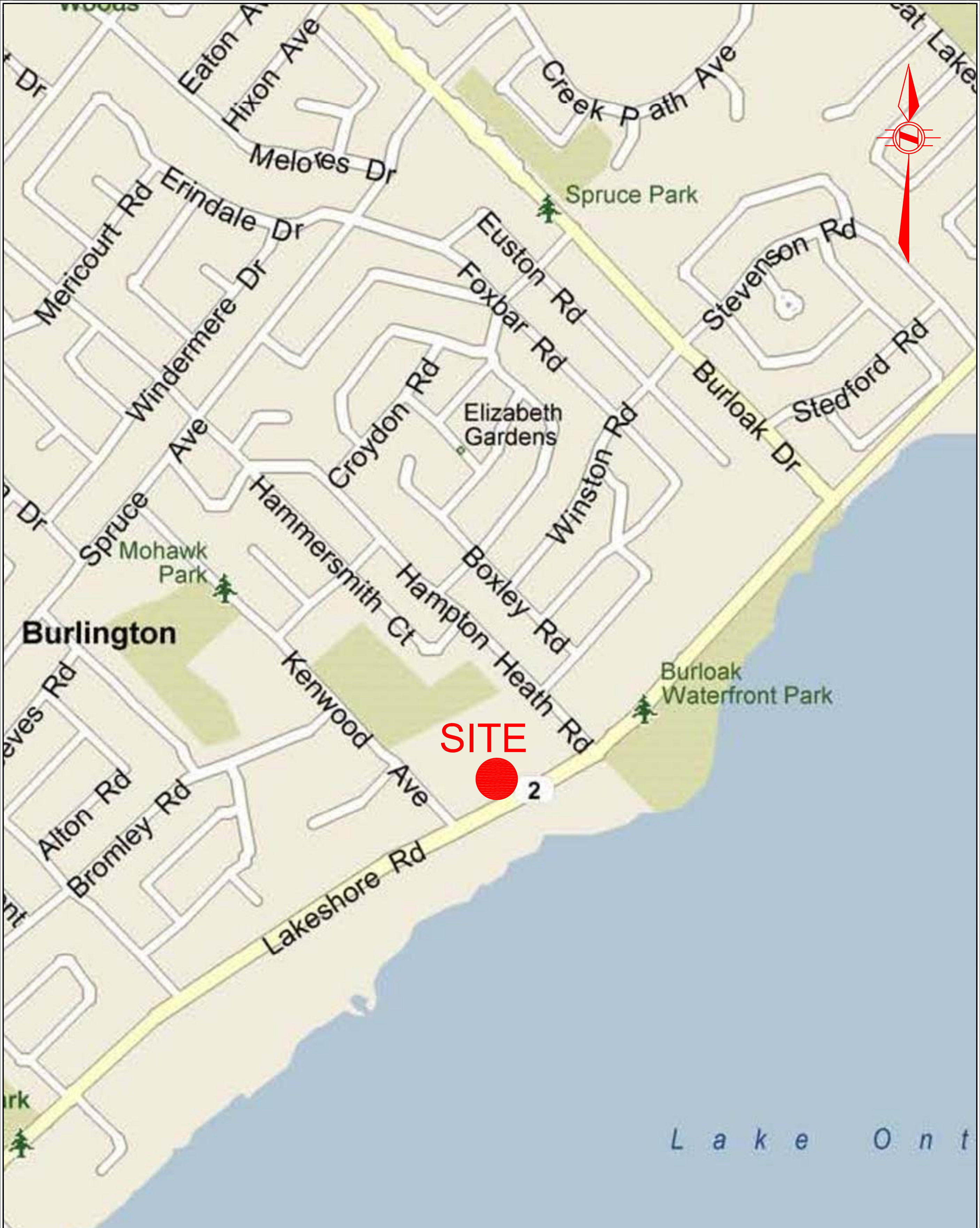
Terraprobe Inc.

LICENSED PROFESSIONAL ENGINEER
P. T. CANNON
Mar 23/15
PROVINCE OF ONTARIO

Patrick Cannon, P. Eng.
Associate

FIGURES

Terraprobe Inc.



Terraprobe

903 Barton Street - Unit 22, Stoney Creek, Ontario, L8E 5P5
Tel: (905) 643-7560, Fax: (905) 643-7559

Title: SITE LOCATION PLAN	FIGURE : 1
File No. 71-15-5010	

LEGEND
BH15-9 Borehole Location

Source:
A.T. McLaren Limited
LEGAL AND ENGINEERING SURVEYS
69 JOHN STREET SOUTH, SUITE 230
HAMILTON, ONTARIO, L8N 2B9
PHONE (905) 527-8559 FAX (905) 527-0032

Terraprobe
903 Barton Street - Unit 22, Stoney Creek, Ontario, L8E 5P5
Tel: (905) 643-7560, Fax: (905) 643-7559

Title:	BOREHOLE LOCATION PLAN	FIGURE : 2
File No.	71-15-5010	

LOGS OF BOREHOLES

APPENDIX A

Terraprobe Inc.

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
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

Terraprobe Inc.

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220 Bayview Drive, Unit 25
Barrie, Ontario L4N 4Y8
(705) 739-8355 Fax: 739-8369

Northern Ontario
1012 Kelly Lake Rd., Unit 1
Sudbury, Ontario P3E 5P4
(705) 670-0460 Fax: 670-0558

www.terraprobe.ca

LOG OF BOREHOLE 15-1

Terraprobe

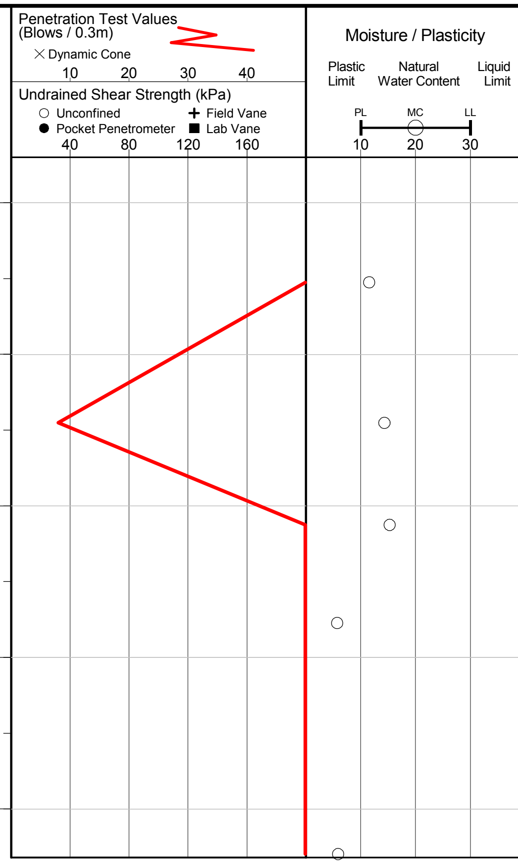
Client : United Burlington Retail Portfolio Inc.
Project : LAKESIDE
Location : 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO

Project No.: 71-15-5010
Date started : February 26, 2015
Sheet No. : 1 of 1

Position : E: 602818, N: 4802457 (UTM 17T)
Elevation Datum : "Geodetic"
Rig type : CME 45, truck-mounted
Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	Description	Graphic Log	Sample Number	Type	SPT 'N' Value	Elevation Scale (m)
0	84.3	GROUND SURFACE					
		100mm **ASPHALTIC CONCRETE**					84
	83.9 / 0.4	**FILL**, granular base					
		FILL, shale, mixed silt, very dense, reddish brown		1	SS	50 / 125mm	
1							83
	82.8 / 1.5	**FILL**, shale, mixed silt, loose, reddish brown		2	SS	8	
2	82.0 / 2.3	**WEATHERED SHALE**, reddish brown and grey		3	SS	50 / 125mm	82
3				4	SS	50 / 50mm	81
4							80
	79.7 / 4.6	**END OF BOREHOLE**		5	SS	50 / 50mm	

Borehole was dry and open upon completion of drilling.



Headspace Vapour | Instrument Details | Unstabilized Water Level | Lab Data and Comments — GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL

LOG OF BOREHOLE 15-2

Terraprobe

Client	: United Burlington Retail Portfolio Inc.	Project No.:	71-15-5010
Project	: LAKESIDE	Date started :	February 26, 2015
Location	: 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO	Sheet No. :	1 of 1

Position	: E: 602785, N: 4802429 (UTM 17T)	Elevation Datum	: "Geodetic"
Rig type	: CME 45, truck-mounted	Drilling Method	: Solid stem augers

Elev / Depth (m)	Description (SOIL PROFILE)	Sample Number	Type	SPT 'N' Value
81.4	**GROUND SURFACE**			
	100mm **ASPHALTIC CONCRETE**			
	FILL, granular base			
80.9 / 0.5	**FILL**, shale, mixed silt, very dense, reddish brown	1	SS	50 / 125mm
80.3 / 1.1	**CLAYEY SILT to SILT**, with grey shale interbeds, very dense, reddish brown (GLACIAL TILL)	2	SS	31
79.1 / 2.3	**WEATHERED SHALE**, reddish brown and grey	3	SS	50 / 75mm
		4	SS	50 / 50mm
76.8 / 4.6		5	SS	50 / 50mm

Depth Scale (m): 0, 1, 2, 3, 4

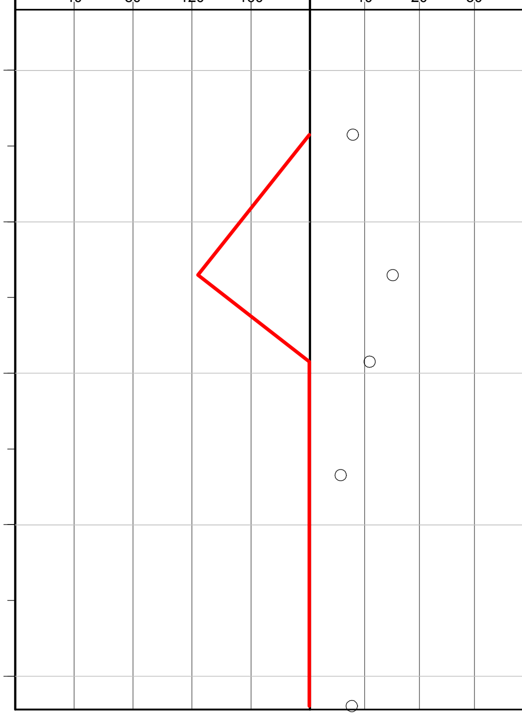
Elevation Scale (m): 81, 80, 79, 78, 77

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

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

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

Headspace Vapour | Instrument Details | Unstabilized Water Level | Lab Data and Comments — GRAIN SIZE DISTRIBUTION (%) (MIT): GR SA SI CL



END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Terraprobe

LOG OF BOREHOLE 15-3

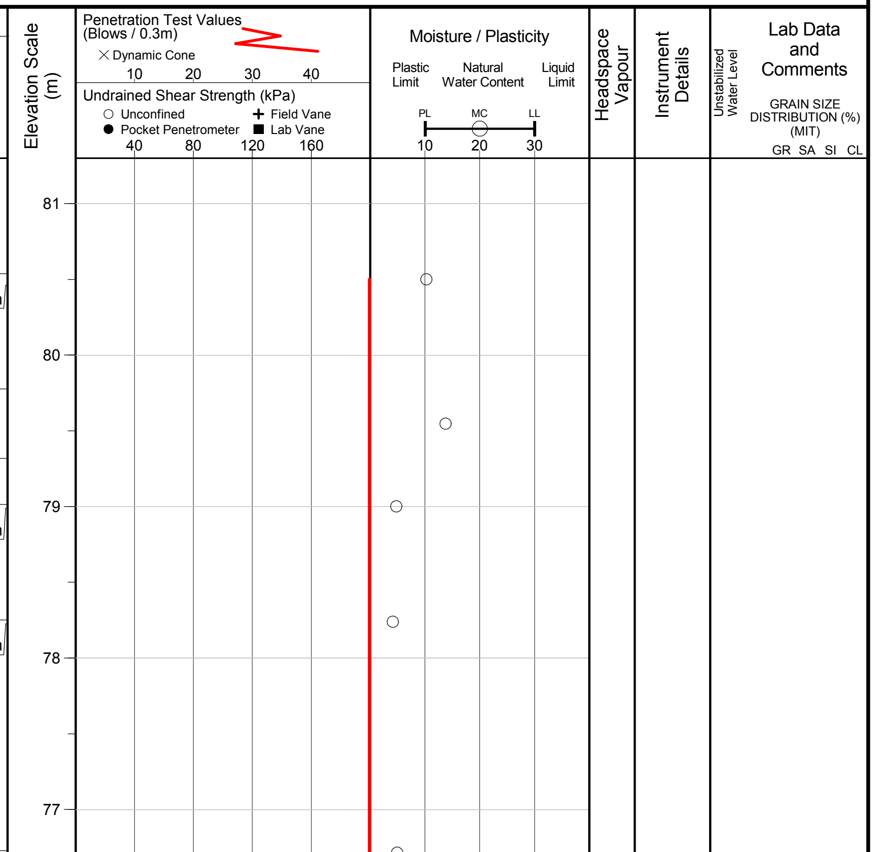
Client	: United Burlington Retail Portfolio Inc.	Project No.:	71-15-5010
Project	: LAKESIDE	Date started :	February 26, 2015
Location	: 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO	Sheet No. :	1 of 1

Position : E: 602751, N: 4802412 (UTM 17T) Elevation Datum : "Geodetic"

Rig type : CME 45, truck-mounted Drilling Method : Solid stem augers

Elev (m)	Depth (m)	Description (SOIL PROFILE)	Sample Number	Type	SPT 'N' Value
81.3	0	**GROUND SURFACE**			
		100mm **ASPHALTIC CONCRETE**			
		FILL, granular base			
80.8	0.5	**FILL**, shale, mixed silt, very dense, reddish brown	1	SS	50 / 75mm
80.2	1.1	**CLAYEY SILT to SILT**, with grey shale interbeds, very dense, reddish brown (GLACIAL TILL)	2	SS	84
79.0	2.3	**WEATHERED SHALE**, reddish brown and grey	3	SS	50 / 25mm
			4	SS	50 / 25mm
76.7	4.6	**END OF BOREHOLE**	5	SS	50 / 25mm

Borehole was dry and open upon completion of drilling.



LOG OF BOREHOLE 15-4

Terraprobe

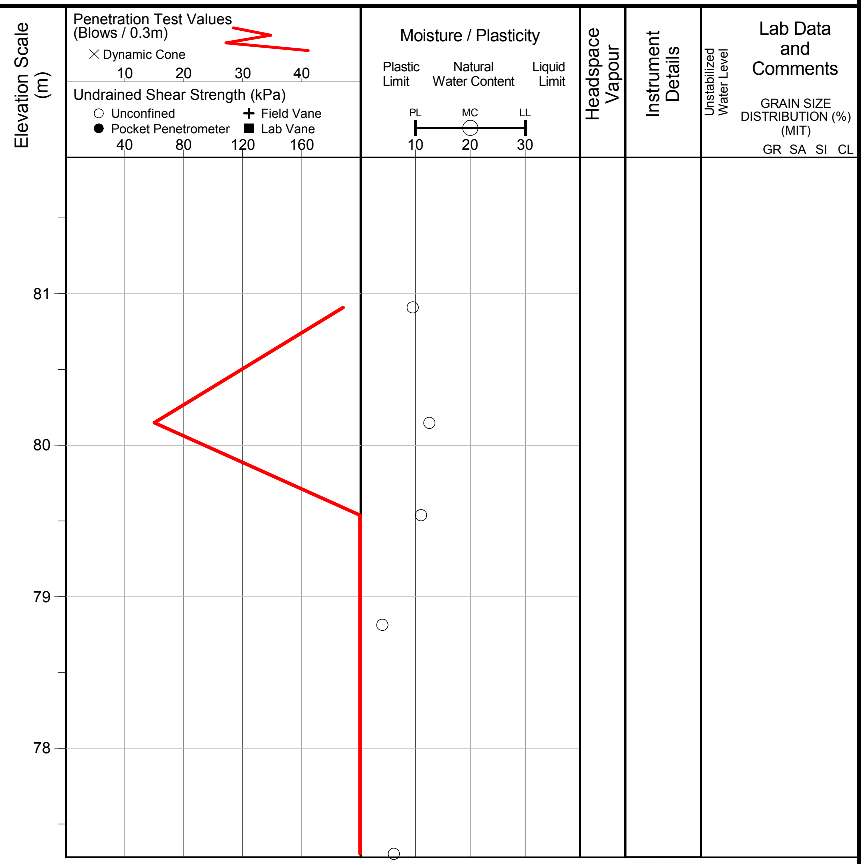
Client	: United Burlington Retail Portfolio Inc.	Project No.:	71-15-5010
Project	: LAKESIDE	Date started :	February 26, 2015
Location	: 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO	Sheet No. :	1 of 1

Position : E: 602764, N: 4802471 (UTM 17T)
Elevation Datum : "Geodetic"
Rig type : CME 45, truck-mounted
Drilling Method : Solid stem augers

Elev / Depth (m)	Soil Profile: Description	Sample Number	Sample Type	SPT 'N' Value
81.9	**GROUND SURFACE**			
	100mm **ASPHALTIC CONCRETE**			
	FILL, granular base			
81.3 / 0.6	**FILL**, shale, mixed silt, very dense, reddish brown	1	SS	47
80.8 / 1.1	**CLAYEY SILT to SILT**, with grey shale interbeds, very dense, reddish brown (GLACIAL TILL)	2	SS	15
79.6 / 2.3	**WEATHERED SHALE**, reddish brown and grey	3	SS	50 / 150mm
		4	SS	50 / 75mm
77.3 / 4.6		5	SS	50 / 50mm

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.



LOG OF BOREHOLE 15-5

Terraprobe

Client : United Burlington Retail Portfolio Inc.
Project : LAKESIDE
Location : 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO

Project No.: 71-15-5010
Date started : February 26, 2015
Sheet No. : 1 of 1

Position : E: 602803, N: 4802485 (UTM 17T)
Elevation Datum : "Geodetic"
Rig type : CME 45, truck-mounted
Drilling Method : Solid stem augers

Elev / Depth (m)	Description	Sample Number	Type	SPT 'N' Value
81.3	GROUND SURFACE			
0.0	100mm **ASPHALTIC CONCRETE**			
	FILL, granular base			
80.8 / 0.5	**FILL**, shale, mixed silt, compact, reddish brown	1	SS	17
79.8 / 1.5	**CLAYEY SILT to SILT**, with grey shale interbeds, very dense, reddish brown (GLACIAL TILL)	2	SS	36
79.0 / 2.3	**WEATHERED SHALE**, reddish brown and grey	3	SS	50 / 75mm
		4	SS	50 / 50mm
76.7 / 4.6		5	SS	50 / 25mm

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Column headings: SOIL PROFILE; SAMPLES; Depth Scale (m); Graphic Log; Elevation Scale (m); Penetration Test Values (Blows / 0.3m) — Dynamic Cone 10 20 30 40; Undrained Shear Strength (kPa) — Unconfined, Field Vane, Pocket Penetrometer, Lab Vane 40 80 120 160; Moisture / Plasticity — Plastic Limit, Natural Water Content, Liquid Limit; PL MC LL 10 20 30; Headspace Vapour; Instrument Details; Unstabilized Water Level; Lab Data and Comments — GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL

Depth scale: 0, 1, 2, 3, 4. Elevation scale: 81, 80, 79, 78, 77.

Terraprobe

LOG OF BOREHOLE 15-6

Client : United Burlington Retail Portfolio Inc.
Project : LAKESIDE
Location : 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO

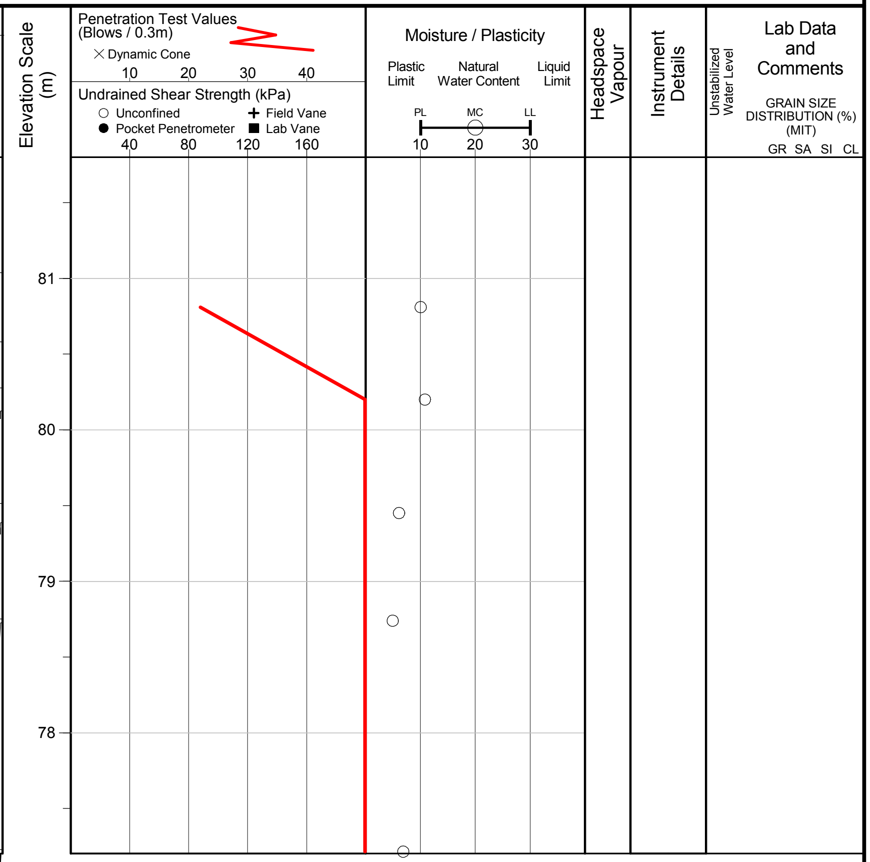
Project No.: 71-15-5010
Date started : February 26, 2015

Position : E: 602692, N: 4802426 (UTM 17T)
Rig type : CME 45, truck-mounted
Elevation Datum : "Geodetic"
Drilling Method : Solid stem augers

Depth Scale (m)	Elev / Depth (m)	Description (SOIL PROFILE)	Sample Number	Type	SPT 'N' Value
0	81.8	**GROUND SURFACE**			
		100mm **ASPHALTIC CONCRETE**			
		FILL, granular base			
	81.2 / 0.6	**FILL**, shale, mixed silt, compact, reddish brown			
1	80.9 / 0.9	**CLAYEY SILT to SILT**, with grey shale interbeds, very dense, reddish brown (GLACIAL TILL)	1	SS	22
	80.3 / 1.5	**WEATHERED SHALE**, reddish brown and grey	2	SS	50 / 150mm
2			3	SS	50 / 125mm
3			4	SS	50 / 25mm
4	77.2 / 4.6	**END OF BOREHOLE**	5	SS	50 / 25mm

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.



Terraprobe

LOG OF BOREHOLE 15-7

Client : United Burlington Retail Portfolio Inc.
Project : LAKESIDE
Location : 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO

Project No.: 71-15-5010
Date started : February 26, 2015
Sheet No. : 1 of 1

Position : E: 602713, N: 4802388 (UTM 17T)
Elevation Datum : "Geodetic"
Rig type : CME 45, truck-mounted
Drilling Method : Solid stem augers

Elev / Depth (m)	Description (SOIL PROFILE)	Sample Number	Type	SPT 'N' Value	Natural Water Content (%)
81.3	**GROUND SURFACE**				
	100mm **ASPHALTIC CONCRETE**				
	FILL, granular base				
80.7 / 0.6	**FILL**, shale, mixed silt, very dense, reddish brown				
80.4 / 0.9	**CLAYEY SILT to SILT**, with grey shale interbeds, very dense, reddish brown (GLACIAL TILL)	1	SS	50 / 150mm	~15
79.8 / 1.5	**WEATHERED SHALE**, reddish brown and grey	2	SS	50 / 150mm	~9
		3	SS	50 / 25mm	~6
		4	SS	50 / 50mm	~6
76.7 / 4.6		5	SS	50 / 25mm	~7

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Column headings: Depth Scale (m) (0–4); Graphic Log; Elevation Scale (m) (81–77); Penetration Test Values (Blows / 0.3m), × Dynamic Cone, 10 20 30 40; Undrained Shear Strength (kPa), ○ Unconfined, + Field Vane, ● Pocket Penetrometer, ■ Lab Vane, 40 80 120 160; Moisture / Plasticity: Plastic Limit (PL), Natural Water Content (MC), Liquid Limit (LL), 10 20 30; Headspace Vapour; Instrument Details; Unstabilized Water Level; Lab Data and Comments, GRAIN SIZE DISTRIBUTION (%) (MIT), GR SA SI CL

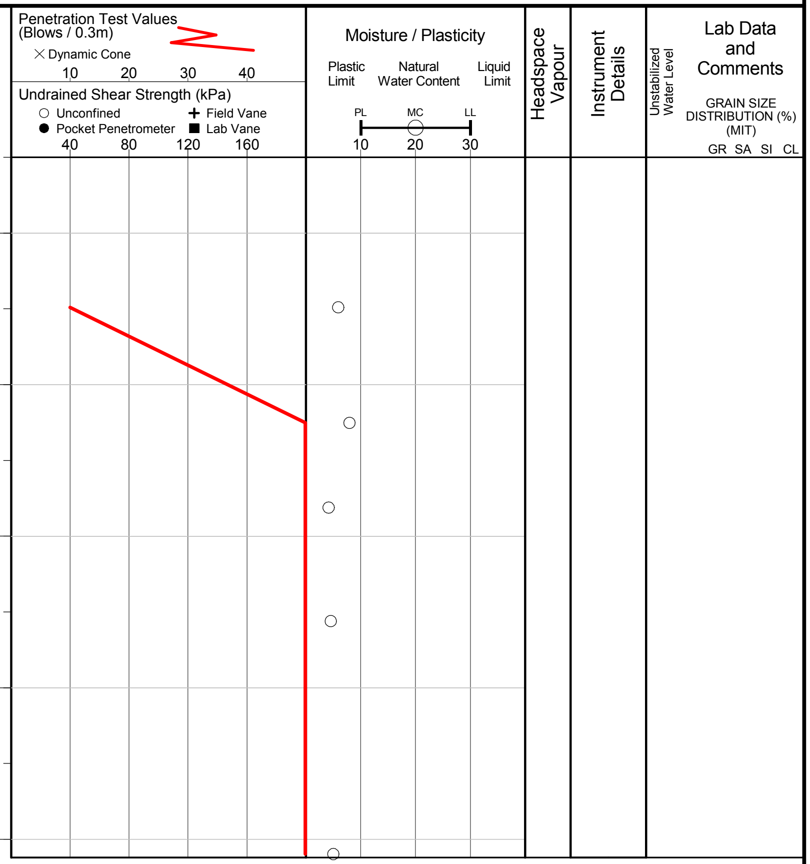
LOG OF BOREHOLE 15-8

Client : United Burlington Retail Portfolio Inc.
Project : LAKESIDE
Location : 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO

Project No.: 71-15-5010
Date started : February 27, 2015
Sheet No. : 1 of 1

Position : E: 602707, N: 4802478 (UTM 17T)
Elevation Datum : "Geodetic"
Rig type : CME 45, truck-mounted
Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	SOIL PROFILE Description	Graphic Log	SAMPLES Number	Type	SPT 'N' Value	Elevation Scale (m)
0	81.5	**GROUND SURFACE**					
		75mm **ASPHALTIC CONCRETE**					
		FILL, granular base					81
	80.9 / 0.6	**FILL**, silty sand and gravel, compact, brown		1	SS	10	
1							
	80.0 / 1.5	**CLAYEY SILT to SILT**, with grey shale interbeds, very dense, reddish brown (GLACIAL TILL)		2	SS	95	80
2	79.2 / 2.3	**WEATHERED SHALE**, reddish brown and grey		3	SS	50 / 50mm	79
3				4	SS	50 / 25mm	78
4							77
	76.9 / 4.6			5	SS	50 / 50mm	



END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

LOG OF BOREHOLE 15-9

Terraprobe

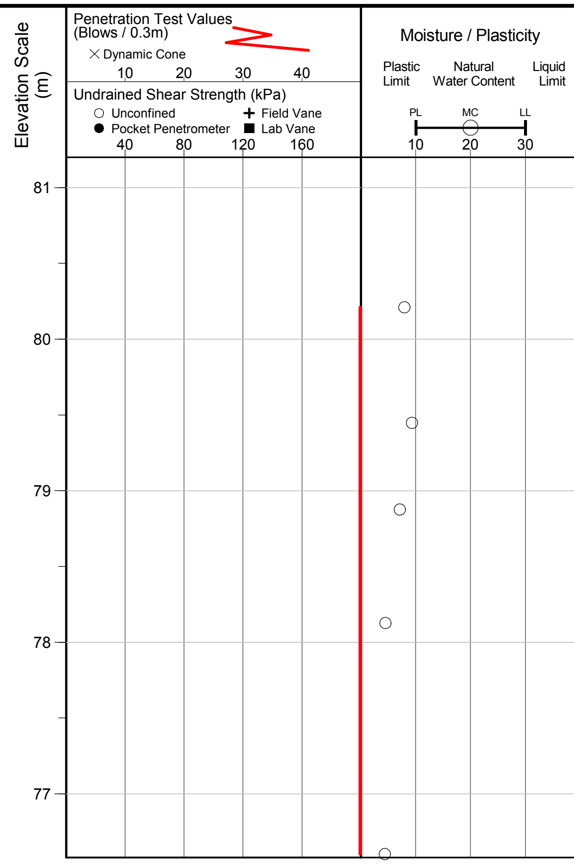
Client	: United Burlington Retail Portfolio Inc.	Project No.:	71-15-5010
Project	: LAKESIDE	Date started :	February 27, 2015
Location	: 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO	Sheet No. :	1 of 1

Position	: E: 602658, N: 4802363 (UTM 17T)	Elevation Datum	: "Geodetic"
Rig type	: CME 45, truck-mounted	Drilling Method	: Solid stem augers

SOIL PROFILE / SAMPLES

Elev Depth (m)	Description	Number	Type	SPT 'N' Value
81.2	**GROUND SURFACE**			
	75mm **ASPHALTIC CONCRETE**			
	FILL, granular base			
80.7 / 0.5	**FILL**, shale, mixed silt, very dense, reddish brown			
80.3 / 0.9	**CLAYEY SILT to SILT**, with grey shale interbeds, very dense, reddish brown (GLACIAL TILL)	1	SS	63
		2	SS	67
78.9 / 2.3	**WEATHERED SHALE**, reddish brown and grey	3	SS	50 / 75mm
		4	SS	50 / 50mm
76.6 / 4.6		5	SS	50 / 50mm

Depth Scale (m): 0, 1, 2, 3, 4



Headspace Vapour | Instrument Details | Unstabilized Water Level | Lab Data and Comments — GRAIN SIZE DISTRIBUTION (%) (MIT) — GR SA SI CL

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

LOG OF BOREHOLE 15-10

Terraprobe

Client : United Burlington Retail Portfolio Inc.
Project : LAKESIDE
Location : 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO

Project No.: 71-15-5010
Date started : February 27, 2015

Position : E: 602590, N: 4802454 (UTM 17T)
Rig type : CME 45, truck-mounted
Elevation Datum : "Geodetic"
Drilling Method : Solid stem augers

Elev (m)	Depth (m)	Description	Sample Number	Type	SPT 'N' Value
81.6	0	**GROUND SURFACE**			
		75mm **ASPHALTIC CONCRETE**			
		FILL, granular base			
81.1	0.5	**FILL**, shale, mixed silt, very dense, reddish brown			
80.8	0.8	**WEATHERED SHALE**, reddish brown and grey	1	SS	50 / 75mm
			2	SS	50 / 50mm
			3	SS	50 / 50mm
78.5	3.1		4	SS	50 / 50mm

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Column headings: Depth Scale (m); SOIL PROFILE; Graphic Log; SAMPLES; Elevation Scale (m); Penetration Test Values (Blows / 0.3m), × Dynamic Cone, 10 20 30 40; Undrained Shear Strength (kPa), ○ Unconfined, + Field Vane, ● Pocket Penetrometer, ■ Lab Vane, 40 80 120 160; Moisture / Plasticity, Plastic Limit, Natural Water Content, Liquid Limit, PL MC LL, 10 20 30; Headspace Vapour; Instrument Details; Unstabilized Water Level; Lab Data and Comments, GRAIN SIZE DISTRIBUTION (%) (MIT), GR SA SI CL

LOG OF BOREHOLE 15-11

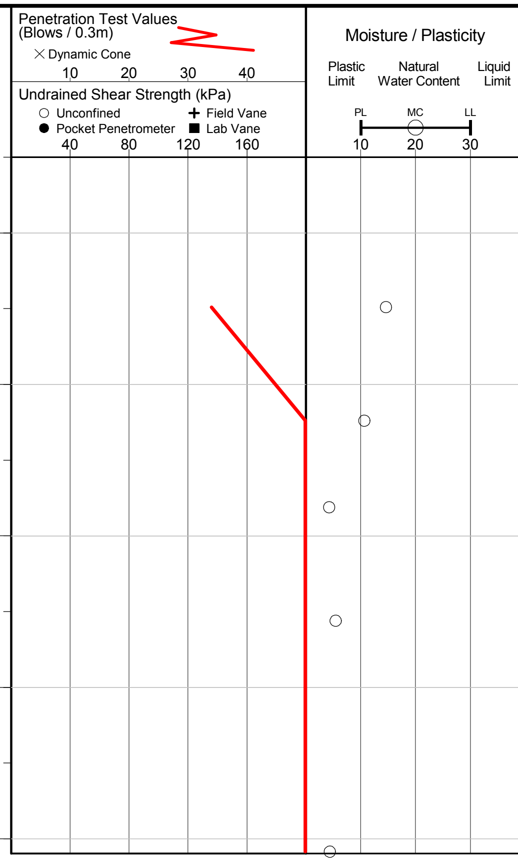
Terraprobe

Client	: United Burlington Retail Portfolio Inc.	Project No.:	71-15-5010
Project	: LAKESIDE	Date started :	February 27, 2015
Location	: 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO	Sheet No. :	1 of 1

Position	: E: 602786, N: 4802562 (UTM 17T)	Elevation Datum	: "Geodetic"
Rig type	: CME 45, truck-mounted	Drilling Method	: Solid stem augers

Elev / Depth (m)	SOIL PROFILE – Description	Sample Number	Type	SPT 'N' Value
81.5	**GROUND SURFACE**			
	75mm **ASPHALTIC CONCRETE**			
	FILL, granular base			
80.9 / 0.6	**FILL**, shale, mixed silt, dense, reddish brown			
80.6 / 0.9	**CLAYEY SILT to SILT**, with grey shale interbeds, very dense, reddish brown (GLACIAL TILL)	1	SS	34
		2	SS	67 / 275mm
79.2 / 2.3	**WEATHERED SHALE**, reddish brown and grey	3	SS	50 / 50mm
		4	SS	50 / 25mm
76.9 / 4.6	**END OF BOREHOLE**	5	SS	50 / 25mm

Borehole was dry and open upon completion of drilling.



Headspace Vapour | Instrument Details | Unstabilized Water Level | Lab Data and Comments: GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL

Depth Scale (m): 0, 1, 2, 3, 4 — Elevation Scale (m): 81, 80, 79, 78, 77

LOG OF BOREHOLE 15-12

Terraprobe

Client	: United Burlington Retail Portfolio Inc.	Project No.:	71-15-5010
Project	: LAKESIDE	Date started :	February 27, 2015
Location	: 5353 LAKESHORE ROAD, BURLINGTON, ONTARIO	Sheet No. :	1 of 1

Position	: E: 602816, N: 4802518 (UTM 17T)	Elevation Datum	: "Geodetic"
Rig type	: CME 45, truck-mounted	Drilling Method	: Solid stem augers

Elev (m)	Depth (m)	Description	Sample Number	Type	SPT 'N' Value
81.5	0.0	**GROUND SURFACE**			
		100mm **ASPHALTIC CONCRETE**			
		FILL, granular base			
80.9	0.6	**FILL**, shale, mixed silt, compact, reddish brown	1	SS	15
80.4	1.1	**CLAYEY SILT to SILT**, with grey shale interbeds, very dense, reddish brown (GLACIAL TILL)	2	SS	
79.7	1.8	**WEATHERED SHALE**, reddish brown and grey	3	SS	50 / 75mm
			4	SS	50 / 50mm
76.9	4.6		5	SS	50 / 50mm

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.