Geotechnical Investigation

Proposed Restaurant and Office Space
1850 Walkey Road
Ottawa, Ontario

Prepared for:

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LRL File No.: 170757

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1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by Marcello's Market and Deli Inc. to perform a geotechnical investigation for a proposed restaurant and office space located at 1850 Walkley Road, in Ottawa, Ontario.

The purpose of the investigation was to identify the subsurface conditions across the site by the completion of a borehole drilling program. Based on the visual and factual information obtained, this report will provide guidelines on the geotechnical engineering aspects of the design of the project, including construction considerations.

This report has been prepared in consideration of the terms and conditions noted above. Should there be any changes in the design features, which may relate to the geotechnical recommendations provided in the report, LRL should be advised in order to review the report recommendations.

2 SITE AND PROJECT DESCRIPTION

The site under investigation is currently a vacant lot, it is rectangular in shape, and has about 47.2 m frontage along Walkley Road, and a total surface area of 7,415 m^2. The location is presented in Figure 1 included in **Appendix A**. Some mature trees were present at the North portion of the site, and at the time of the investigation, the site was covered by snow. The terrain at the proposed site is considered to be relatively flat, with a slight mound at the Southwest corner of the property. Access to the site will come by way of Walkley Road, and is civically located at 1850 Walkley Road, Ottawa, Ontario.

It is our understanding that the proposed development will consist of a 700 m^2 restaurant to be constructed at the North portion of the site, a three-storey, 1,100 m^2 office building at the South portion of the site, and a paved area between the two buildings for subsequent parking.

3 PROCEDURE

The fieldwork for this investigation was carried out on January 22, 2018. Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of three (3) boreholes, labelled BH1 through BH3, were drilled inside the property within the proposed buildings' footprint, and asphalted area, where it was possible to do so. The approximate locations of the boreholes are shown in Figure 2 included in **Appendix A**.

The boreholes were advanced using a track mount CME 55 drill rig equipped with 200 mm diameter continuous flight hollow stem auger supplied and operated by George Downing Estate Drilling. A "two man" crew experienced with geotechnical drilling operated the drill rig and equipment.

Sampling of the overburden materials encountered in the boreholes was carried out at regular depth intervals using a 50.8 mm diameter drive open conventional spoon sampler in conjunction with standard penetration testing (SPT) "N" values. The SPT were conducted following the method **ASTM D1586** and the results of SPT, in terms of the number of blows per 0.3 m of split-spoon sampler penetration after first 0.15 m designated as "N" value.

The boreholes were advanced to depths ranging from 8.08 to 9.60 m below ground surface (bgs). Upon completion, the boreholes were backfilled and compacted using a combination of silica sand, bentonite and overburden cuttings.

The fieldwork was supervised throughout by a member of our engineering staff who oversaw the drilling activities, cared for the samples obtained and logged the subsurface conditions encountered within each of the boreholes. All soil samples collected from the boreholes were placed and sealed in plastic bags to prevent moisture loss. The recovered soil samples collected from the boreholes were classified based on visual examination of the materials recovered and the results of the in-situ testing. All soil samples were transported to our office for further examination by our geotechnical engineer.

Furthermore, all boreholes were surveyed and located using a Garmin Etrex Legend GPS (Global Positioning System) receiver using NAD 83 datum (North American Datum). LRL's field personnel determined the existing grade elevations at the borehole locations through a topographic survey carried out using the "Concrete Pad at the Gate of the Side Entrance of the Adjacent Property (Dymon Storage)" as a Temporary Bench Mark (TBM). The TBM was assumed to have an elevation of 100.00 m. Respective ground surface elevations of boring locations are shown on their respective boreholes logs

4 SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

4.1 General

A review of local surficial geology maps provided by the Department of Energy, Mines and Resources Canada suggest that the surficial geology for this area consist of blue-grey clay, silt, and silty clay; calcareous and fossiliferous at depth.

The subsurface conditions encountered in the boreholes were classified based on visual and tactile examination of the materials recovered from the boreholes and the results of in-situ laboratory testing. The soil descriptions presented in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil were conducted according to the procedure **ASTM D2487** and judgement, and LRL does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The subsurface soil conditions encountered at boreholes are given in their respective logs presented in **Appendix B**. A greater explanation of the information presented in the borehole logs can be found in **Appendix C** of this report. These logs indicate the subsurface conditions encountered at a specific test location only. Boundaries between zones on the logs are often not distinct, but are rather transitional and have been interpreted as such.

4.2 Topsoil

Topsoil of thickness ranging from 100 to 200 mm was found at all boring locations.

This material was classified as topsoil based on colour and the presence of organic material and is intended as identification for geotechnical purposes only. It does not constitute a statement as to the suitability of this layer for cultivation and sustaining plant growth.

4.3 Silty Clay

Underlying the topsoil, a deposit of reddish to brownish grey silty clay was encountered at all boring locations, it extended to depths ranging from 4.1 to 7.2 m bgs. Standard penetration tests were carried out in the silty clay material and the STP “N” value was found ranging from 7 to WH, indicating the deposit is firm to very soft in consistency. The natural moisture content was found varying between 39 and 73%, indicating wet or saturated condition.

An in-situ field vane shear test using a 145 x 65 mm tapered vane was carried-out in the clay deposit. The undrained shear strength value was calculated following the procedure **ASTM D2573**. The initial in-situ values were found ranging between 8 to 52 kPa, and the remold values ranged between 1 and 4 kPa. According to the Canadian Engineering Foundation Manual (CFEM, 2006), the silty clay material is considered to be highly sensitive (or quick).

A soil sample was collected from BH1 (SS3) between depths about 1.5 and 2.0 m bgs for laboratory gradation analyses. The gradation analyses comprised of sieve and hydrometer were conducted following the procedure **ASTM D422.** According to the Unified Soil Classification System, the soil sample in BH1 would be classified as silty clay. Details of laboratory analyses are reflected in **Table 1**.

Table 1: Gradation Analysis Summary

Sample Location	Depth (m)	Percent for Each Soil Gradation					Estimated Hydraulic Conductivity K (cm/s)
		Sand			Silt (%)	Clay (%)	
		Coarse (%)	Medium (%)	Fine (%)			
BH1	1.5 – 2.0	0.0	0.0	0.3	26.4	73.3	1 x 10^{-6}

Atterberg limits and moisture contents were conducted on the spoon soil sample collected between depths about 3.05 and 3.50 m in BH3 (SS5). Based on the test result, the sample yielded a plastic limit of 27% and corresponding liquid limit of 62%. These values indicate that the subsoil contains inorganic clays of high plasticity. A summary of these values are provided below in **Table 2**.

Table 2: Summary of Atterberg Limits and Water Contents

Sample Location	Parameter					
	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Water Content (%)	USCS Group Symbol
BH3	3.05 – 3.50	62	27	35	61	CH

One dimensional consolidation test of soil using incremental loading was performed on a silty clay sample taken from a Shelby tube collected from BH2 at depth between 3.0 and 3.6 m bgs, following the procedure **ASTM D2435**, the results are tabulated in **Table 3.**

Table 3: One-Dimensional Consolidation Test Results

Sample Location	Depth (m)	Effective Overburden Pressure (kPa)	Pre-consolidation Pressure (kPa)	Over-consolidation Pressure (kPa)	Initial Moisture Content (%)	Initial Void Ratio
BH2	3.3	28.9	95.0	66.1	79.1	2.38

The consolidation test was performed by an accredited laboratory, Stantec Consulting Ltd. The consolidation test results revealed that the clay is over consolidated, which is typical for clay deposits found in the Eastern Ontario. A pre-consolidation pressure from ST1 (BH2) taken at depth between 3.0 and 3.6 m was calculated based on the consolidation curve and was found to be approximately 105.0 kPa with an over-consolidation deviation of about 76.1 kPa. The effective overburden pressure is directly affected by groundwater level. Lowering of groundwater level will increase the overburden pressure and therefore available pre-consolidation pressure will reduce. If the groundwater is lowered by a significant amount, unacceptable settlement may be induced.

The laboratory reports can be found in **Appendix D** of this report.

4.4 Silty Sand Till

Underneath the silty clay in all boring locations, a deposit of silty sand till was encountered, and extended to depths of 8.1 and 9.6 m bgs (end of exploration depths). It generally consisted of some clay, some gravel sized stone, and can be described as dark grey in colour. The recorded SPT “N” values of this deposit varied from 20 to greater than 50 blows per 0.3 m of penetration, indicating the deposit is compact to very dense. Although the higher ‘N’ values reflects the presence of cobbles and boulders, rather than the state of packing of the material. The measured natural moisture content was found varying between 7 and 21%.

4.5 Groundwater Conditions

For long term water level monitoring, a 19 mm piezometer was installed in BH1. The piezometer was measured on February 21, 2018, and the water was found to be at 1.21 m bgs. The water level measurement is shown on the borehole logs presented in **Appendix B.**

It should be noted that groundwater levels could fluctuate with seasonal weather conditions, (i.e. rainfall, droughts, spring thawing) and due to construction activities at or in the vicinity of the site.

5 GEOTECHNICAL CONSIDERATIONS

This section of the report provides general geotechnical recommendations for the design aspect of the project based on our interpretation of the information gathered from the boreholes performed at this site and from the project requirements.

This section will detail the specific requirements and limitations with regard to allowable foundation bearing pressure and depth for shallow, and deep foundation options, grade raise, and size of the footings.

5.1 Foundations

The site under investigation is underlain by 4.1 to 7.2 m of sensitive silty clay overlaying silty sand glacial till. The proposed one-storey restaurant building, to be located on the North portion of the site, may be supported by shallow foundation founded over the undisturbed native silty clay, provided that the restrictions on grade raise and founding depth of footings as outlined in **Section 5.1.1** are followed. Alternatively, if the footing loads from the proposed one-storey restaurant building were to exceed the maximum allowable bearing pressure, consideration should be given to deep foundations to support the loads.

Based on the undrained shear strength measurements and consolidation test result, the encountered silty clay deposit does not have the sufficient capacity to support the expected medium to high loads imposed by the proposed three-storey office building on individual spread footings. The proposed three-storey building, to be located at the South portion of the site, should be supported by deep foundation founded below very dense silty clay till and over inferred bedrock, as discussed in **Section 5.1.2**.

5.1.1. Shallow Foundation

Conventional strip and column footings founded over the undisturbed native silty clay may be designed using a maximum allowable bearing pressure of serviceability limit state **(SLS)** and ultimate limit state **(ULS)** factored bearing resistance, as summarized in **Table 4**. The factored ULS value includes the geotechnical resistance factor of 0.5. This bearing capacity limits the allowable grade raise to 0.3 m and a maximum allowable founding depth of 1.2 m bgs. This bearing capacity also allows for a strip footing of width minimum 0.6 to maximum 0.9 m, and a pad footing of width minimum 0.9 to maximum 1.5 m on any side.

Where proposed footings have insufficient soil cover for frost protection, the use of insulation will be required, as outlined in **Section 5.6**.

Table 4: Geotechnical Bearing Resistance for Foundations on Native Silty Clay

Foundation Type	Foundation Width (m)	ULS (kPa)	SLS (kPa)
Strip Footing	0.6	75	55
Strip Footing	0.9	70	40
Strip Footing	1.2	70	35
Square Footing	0.9	85	65
Square Footing	1.2	85	60
Square Footing	1.5	80	50

In-situ field test may be required to check the strength and stability of the footings subgrade. Any incompetent subgrade areas as identified from in-situ testing must be sub-excavated and backfilled with approved structural fill. Similarly, any soft or wet areas should also be sub-excavated and backfilled with approved structural fill only. Prior to placing the approved structural fill, the subgrade comprised of the silty clay deposit should be inspected and approved by geotechnical engineer or a qualified geotechnical personnel. The bearing pressure is contingent on the water level being 0.3 m below the underside footing elevation in order to have stable and dry footings subgrade.

All the footings founded on clay, as well as foundation walls supported by such footings should be reinforced to bridge anomalies “soft areas” in the material, in consultation with the project structural engineer. Footings and foundation walls shall be reinforced, especially at segments where footings founding soils are comprised of partly structural fill and partly undisturbed native soil. If the strip footings need to be founded at different level, it is recommended to use the step footings specification as recommended in **Clause 9.15.3.9 of OBC 2012** or any updated version.

If the footings are wider or founded at a shallower depth, or if the grade raise is greater than that mentioned above, the above allowable bearing pressure at serviceability limit state may have to be reduced.

Prior to pouring footings concrete, the subgrade comprised of undisturbed silty clay should be inspected and approved by a geotechnical engineer or a representative of geotechnical engineer.

5.1.2. Deep Foundation (driven steel piles)

The design method for a particular deep foundation depends on the surrounding and founding soil type and end-bearing or toe resistance pressure. The most common and cost-effective deep foundations are driven steel piles which is considered suitable for this site.

The proposed office building could be supported on end bearing steel piles driven to refusal on inferred bedrock underlying glacial till layer. As most of the overburden soil found on this site is silty clay, it is unlikely that the piles will encounter any significant obstructions during pile installation except in glacial till deposit, where there is possibility to encounter cobbles or boulders.

If some of the piles do not fully penetrate the glacial till to reach the bedrock surface, pre-drilling through the glacial till could be considered. Alternatively, the axial resistance of these piles may need to be re-assessed based on their final depth and blow-count termination that are achieved. The capacities of these piles may have to be confirmed in the field by carrying out dynamic pile monitoring with PDA testing.

Typically, two types steel piles are used in for deep foundation of high bearing capacity. These are as follows:

i. Steel H piles; and

ii. Closed ended, concrete filled, steel pipe piles.

The depth to practical refusal over inferred bedrock underlying glacial till layer was established to range below about 9.0 to 10.0 m at this site. Steel H-pile or closed ended, concrete filled steel pipe (tube) piles may be used. To minimize the potential for damage to the pile tips during driving, the piles should be provided with a driving shoe as per OPSD standards 3000.100 and 3001.100, for H-pile and steel tube piles, respectively.

Pile driven to refusal generate high ultimate geotechnical capacity, typically equal to the structural capacity of the steel section of the pile. For design example, an HP 310 x 79 with area 9980 mm^2 and yield strength 350 MPa has an un-factored ultimate structural capacity of 3140 kN (assuming structural capacity reduced to 90 percent due to bulking, lateral loads and other complex situation). The maximum pile capacity for HP 310 x 79 driven to sound bedrock can therefore be considered for **Service Limit State (SLS) 1040 kN** and **Ultimate Limit State (ULS) 1250 kN**. A geotechnical resistance factor 0.4 should be used to the ultimate structural value to obtain the factored ultimate resistance of a pile driven to sound bedrock.

Closed ended, concrete filled steel pipe pile of 245 mm diameter can be considered to resist the geotechnical axial resistances as summarized in **Table 5.**

Table 5: Geotechnical Axial Resistance of Steel Pipe Pile

Pile Outside Diameter (mm)	Pipe Wall Thickness (mm)	Geotechnical Axial Resistance	
		Service Limit State (SLS), kN	Ultimate Limit State (ULS), kN
245	9	950	1140
	10	1050	1260

	11	1150	1380

This assumes that the steel has a minimum yield strength of 350 MPa and that the pipe pile is filled with 30 MPa concrete. Pipe piles should be equipped with a base plate having a thickness of at least 20 mm to limit damage to the pile tip during driving.

The piles should be driven no closer than three pile widths/diameters centre to centre.

All of the piles should be driven to refusal. The driving resistance criteria will be highly dependent on the required allowable load and the contractor's pile driving equipment. Typically, for drop hammer type piling rigs available in the Ottawa area, a refusal criteria of 20 blows for the last 25 millimetres of penetration would be sufficient to achieve the above allowable loads, assuming that about 35 kilojoules of energy is transferred to the pile per blow. The contractor should be required to submit to the geotechnical engineer a copy of the proposed pile size, piling equipment, methodology and driving resistance criteria prior to construction. The pile foundations should be designed according to Part 4 of the Ontario Building Code 2012 or any updated edition.

An allowance should be made in the specifications for this project for re-striking all of the piles at least once to confirm the design set and/or the permanence of the refusal and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first re-strike should receive additional re-striking until the design set criteria is met. All re-striking should be performed after 48 hours of the previous set. Furthermore, provisions should be made for dynamic load tests on test piles and for dynamic testing and analysis on selected production piles to verify the driving resistance criteria and pile capacities. For this project, the piles dynamic monitoring of three (3) to five (5) piles is recommended.

The post construction settlement of elements of the structure, other than the elastic shortening of the piles, should be negligible for end bearing piles driven to refusal over bedrock. For pile foundations, there is no restriction on grade raise in this site.

5.2 Structural Fill

For foundations set over undisturbed native soil and where excavation below the underside of the footings is performed in order to reach a suitable founding stratum, consideration should also be given to support the footings on structural fill. The structural fill should be placed over undisturbed native soils in layers not exceeding 200 mm and compacted to 98% of its Standard Proctor Maximum Dry Density (SPMDD) within ±2% of its optimum moisture content. In order to allow the spread of load beneath the footings and to prevent undermining during construction, the structural fill should extend minimum 1.0 m beyond the outside edges of the footings and then outward and downward at 1 horizontal to 1 vertical profile (or flatter) over a distance equal to the depth of the structural fill below the footing. Furthermore, the structural fill must be tested to ensure that the specified compaction level is achieved. If the thickness of structural fill exceeds 0.5 m, an approved bi-axial geogrid are required to be installed at mid-depth (preferably 0.2 - 0.3 m above the approved subgrade) of structural fill to minimize any differential settlement.

5.3 Settlement

The estimated total settlement of the shallow and deep foundations, designed using the recommended serviceability limit state capacity value, as well as other recommendations given above, will be less than 25 mm. The differential settlement between adjacent column footings is anticipated to be 15 mm or less.

5.4 Seismic

Based on the limited information of this geotechnical investigation and in accordance with the Ontario Building Code 2015 (Table 4.1.8.4.A.) and Canadian Foundation Engineering Manual (4th edition), the site can be classified for Seismic Site Response Site Class E.

The above classifications were recommended based on conventional method exercised for Site Classification for Seismic Site Response and in accordance with the generally accepted geotechnical engineering practice. It is recommended to carry out specific seismic testing, such as shear wave velocity test or approved equivalent test to confirm the above site classification.

5.5 Liquefaction Potential

Referring to Canadian Foundation Engineering Manual, 2006, the following criteria can be used to determine liquefaction susceptibility of fine grained soils.

- $w/w_L \geq 0.85$ and $I_p \leq 12$: Susceptible to liquefaction or cyclic mobility
- $w/w_L \geq 0.8$ and $12 \leq I_p \leq 20$: Moderately susceptible to liquefaction or cyclic mobility
- $w/w_L < 0.8$ and $I_p \geq 20$: No liquefaction or cyclic mobility, but may undergo significant deformations if cyclic shear stress > static undrained shear strength.

Laboratory plasticity test on a split spoon sample collected at an approximate depth of 3.1 – 3.5 m bgs exhibits the ratio of water content to liquid limit of approximately 0.96 (> 0.8), and I_p is 35 (>20). Based on the test results, the silty clay deposit is not susceptible to liquefaction or cyclic mobility, but may undergo significant deformations if cyclic shear stress > static undrained shear strength. However, there is still a possibility to encounter localized shallow groundwater, which is mostly perched water, and will be mitigated through appropriate sump pumping.

5.6 Frost Protection

All exterior footings located in any unheated portions of the proposed buildings should be protected against frost heaving by providing a minimum of 1.5 m of earth cover. Areas that are to be cleared of snow (i.e. sidewalks, paved areas, etc.) should be provided with at least 1.8 m of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided using a combination of earth cover and extruded polystyrene insulation. Detailed guidelines for footing insulation frost protection can be provided upon request.

In the event that foundations are to be constructed during winter months, the foundation soils are required to be protected from freezing temperatures using suitable construction techniques. The base of all excavations should be insulated from freezing temperatures immediately upon exposure, until heat can be supplied to the building interior and the footings have sufficient soil cover to prevent freezing of the subgrade soils.

5.7 Foundation Drainage

Permanent perimeter drainage is only required for buildings where basement or whenever any open spaces located below the finish ground are being considered. It is our understanding that basement construction is included as part of the proposed office building construction and hence perimeter drainage is required. Also, in order to minimize ponding of water adjacent to the foundation walls, roof water should be controlled by a roof drainage system that directs water away from the building to prevent ponding of water adjacent to the foundation wall.

5.8 Foundation Walls Backfill (Shallow Foundations)

To prevent possible foundation frost jacking and lateral loading, the backfill material against any foundation walls, grade beams, isolated walls, or piers should consist of free draining, non-frost susceptible material such as sand or sand and gravel meeting OPSS Granular B Type I or equivalent grading requirements.

The foundation wall backfill should be compacted to minimum 95% of its SPMDD using light compaction equipment, where no loads will be set over top. The compaction shall be increased to 98% of its SPMDD under walkways, slabs or paved areas close to the foundation or retaining walls. Backfilling against foundation walls should be carried out on both sides of the wall at the same time where applicable.

5.9 Slab-on-grade Construction

Conventional concrete slab-on-grade is considered feasible for both proposed restaurant and office buildings provided certain precautions are undertaken. For predictable performance for the proposed slab-on-grade, it should rest over undisturbed competent native soil (silty clay) or structural fill. Therefore, any loose and disturbed materials including organic or otherwise deleterious material shall be removed from the proposed building’s footprint. The exposed undisturbed native subgrade comprised of silty clay should then be inspected and approved by qualified geotechnical personnel.

Any underfloor fill needed to raise the general floor grade shall consist of OPSS Granular B Type I material or an approved equivalent, compacted to 95% of its SPMDD. The final lift shall be compacted to 98% of its SPMDD. A 200 mm Granular A meeting the **OPSS 1010** shall be placed underneath the slab and compacted to 100% of its SPMDD. Alternatively, if wet condition persists, 200 mm thickness of 19 mm clear stone meeting the **OPSS 1004** requirements shall be used instead of Granular A. Effective compacting effort shall be utilized to consolidate the clear stone.

It is also recommended that the area of extensive exterior slab-on-grade (sidewalks, ramp etc.) shall be constructed using Granular B subbase of thickness 300 mm and Granular A base of thickness 150 mm with incorporating subdrain facilities. The modulus of subgrade reaction (ks) for the design of the slabs set over competent native soil/structural fill is **18 MPa/m.**

It is considered that conventional slab-on-grade construction can be used for basement floor slab of proposed three-storey office building. Considering a full basement for the proposed office building, the slab floor elevation will be located approximately 0.95 m below groundwater table. To prevent hydrostatic pressure build up beneath the floor and potential groundwater infiltration, it is suggested that the granular base including 200 mm thickness of 19 mm clear stone is used underneath the floor slab. Drainage tile consisting of 100 mm diameter weeping tile wrapped with a filter cloth is also recommended to install underneath the floor slab with invert to be at least 300 mm below underside of the floor slab in parallel rows of 6.0 m spacing in one direction. In order to further minimize and control cracking, the floor slab shall be provided with wire or fibre mesh reinforcement and construction or control joints. The construction or control joints should be spaced equal distance in both directions and should not exceed 4.5 m. The wire or fibre mesh reinforcement shall be carried out through the joints.

If any areas of the proposed building area are to remain unheated during the winter period, thermal protection of the slab on grade may be required. The “Guide for Concrete Floor and Slab Construction”, **ACI 302.1R-04** is recommended to follow for the design and construction of vapour retarders below the floor slab. Further details on the insulation requirements could be provided, if necessary.

5.10 Retaining Walls and Shoring

The following **Table 6** below provides the suggested soil parameters for the design of retaining wall and/or shoring systems. For excavations near existing services and structures, the coefficient of earth pressure at rest (K_o) should be used. Material properties for shoring and permanent wall design (static) are shown in details in **Table 6**.

Table 6: Material Properties for Shoring and Permanent Wall Design (Static)

Type of Material	Bulk Density (kN/m^3)	Friction Angle (Φ)	Pressure Coefficient		
			At Rest (K_0)	Active (K_A)	Passive (K_P)
Granular A	23.0	34	0.44	0.28	3.54
Granular B Type I	20.0	31	0.48	0.32	3.12
Granular B Type II	23.0	32	0.47	0.31	3.25
Silty Clay	17.5	27	0.49	0.37	2.66

The above values are for a flat surface behind the wall, a straight wall and a wall friction angle of 0°. The designer should consider any difference between these coefficients, and make appropriate corrections for a sloped surface behind the wall, angled wall or wall friction as required. The bearing capacity for the design of a retaining wall are the same as provided for the building structure provided it is founded over the same soil stratum.

Retaining walls should also be designed to resist the earth pressures produces under seismic conditions. The total active thrust (P_{AE}) in seismic condition includes both a static component (P_A) and a dynamic component (ΔP_{AE}), and can be calculated as follows:

The active thrust, $P_{AE} = P_A + \Delta P_{AE}$

Where

$P_A = ½ K_A \gamma H^2$

(K_A = 0.31 for Granular B Type II. For other material, use relevant value for K_A from the above Table 4)

H = Total height of the wall (m)

γ = Unit weight of the backfill material (kN/m^3)

These dynamic thrust (ΔP_{AE}) can be calculated from

$\Delta P_{AE}, = 0.375\ (a_c \gamma H^2/g)$

Where

$a_c = (1.45 – a_{max}/g)a_{max}$

The peak ground acceleration (PGA) or a_{max}, for the Vars, Ottawa is 0.35g according to 2015 National Building Code Seismic Hazard Calculation and acceleration of gravity, g =

9.81 m/s^2. The seismic coefficient in the vertical direction is assumed to be negligible. The total active thrust P_{AE} may be considered to act at a height, h (m), from the base of the wall,

$$h = [P\ (H/3) + \Delta P_{AE}\ (0.6H)]/\ P_{AE}$$

Internal force acting on the reinforced zone, $P_{IR} = a_c \gamma_r HL/g$

Where

γ_r is the unit weight of reinforced zone.

Add P_{AE} and 0.5 P_{IR} to check the stability. Factor of safety (Seismic) ≥ 0.75 Factor of safety (Static)

5.11 Corrosion Potential and Cement Type

One (1) representative soil sample was submitted for soil sulphate (SO_4) analysis. The laboratory analyses revealed a maximum measured sulphate concentration of 0.006% (60 µg/g) in soil sample. Based on the CAN/CSA-A23.1 standards (Concrete Materials and Methods of Concrete Construction), a sulphate concentration of less than 0.1% (1000 µg/g) falls within the negligible category for sulphate attack on buried concrete. The test result from soil sample tested was below the noted threshold. As such, buried concrete for footings and foundations walls will not require any special additive to resist sulphate attack and the use of normal Portland cement is acceptable.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil resistivity was measured to be 3230 ohm-cm indicating that the steel structures with exposed surface in contact with the clay soil encountered at the site can be subjected to a moderate corrosive environment.

Any imported soils should be tested with regard to water soluble sulphate concentration and associated sulphate exposure level should be determined accordingly.

The laboratory Certificates of Analysis are presented in **Appendix D.**

5.12 Vibration Monitoring and Pre-construction Surveys

The required construction activities will generate some vibrations that will be perceptible to the nearby residences. The vibrations are expected to be greatest during pile driving, excavation and material placement. It is recommended that pre-construction surveys be carried out. **Table 7** provides vibration limits intended to prevent cracking and other structural problems.

Table 7: Suggested Peak Vibration Limits at Nearby Structures/Services

Frequency Range (Hz)	<10	10 - 40	>40
Peak Particle Velocity (mm/sec)	5	5 to 50	50

It is pointed out that these criteria, although conservative, were established to prevent damage to existing buildings and services. More stringent criteria may be required to prevent damage to vibration sensitive structure, equipment or utilities.

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation

It is anticipated that the depth of excavation for the proposed buildings will not be extended below 2.1 m bgs. Most of the excavation being carried out will be through silty clay. Excavation must be carried out in accordance with Occupational Health and Safety Act and Regulations for Construction Projects.

According to the Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 213/91 and its amendments, the surficial overburden expected to be excavated into at this site can be classified as Type 3. Therefore, shallow temporary excavations in overburden soil classified as Type 3 can be cut at 1 horizontal to 1 vertical (1H: 1V), for a fully drained excavation starting at the base of the excavation and as per requirements of the OHSA regulations.

In the event that the aforementioned slopes are not possible to achieve due to space restrictions, the excavation shall be shored according to OHSA O. Reg. 213/91 and its amendments. A geotechnical engineer shall design and approve the shoring and establish the shoring depth under the excavation profile. Refer to the parameters provided in **Table 6** in **Section 5.10** for use in the design of any shoring structures.

Any excavated material stockpiled near an excavation or trench should be stored at a distance equal to or greater than the depth of the excavation/trench and construction equipment, traffic should be limited near open excavation.

6.2 Groundwater Control

The below groundwater excavation depth for the construction of basement floor level (at proposed office building) could be up to 1.0 m. Based on the subsurface conditions encountered at this site, groundwater seepage or infiltration from the native silty clay into shallow temporary excavations during construction should be minor in nature and may increase with depth. However, it is anticipated that pumping from open sumps will be sufficient to control groundwater inflow through the vertical face of excavations. Any groundwater seepage or infiltration entering the excavation should be removed from the excavation by pumping from sumps within the excavations. Surface water runoff into the excavation should be minimized and diverted away from the excavation.

A permit to take water (PTTW) is required from Ministry of Environment and Climate Change (MOECC), Ontario Reg. 387/04, if more than 400,000 litres per day of groundwater will be pumped during a construction period less than 30 days. Registration in the Environmental Activity and Sector Registry (EASR) is required when the takings of ground water and storm water for the purpose of dewatering construction projects range between 50,000 and 400,000 litres per day.

The actual amount of groundwater inflow into open excavations will depend on several factors such as the contractor's schedule and rate of excavation, the size of excavation, and depth below the groundwater level and the time of year at which the excavation is executed. Considering that the groundwater levels at this site may fluctuate seasonally, it is possible that pumping rates in excess of 50,000 litres per day will be required. As such, EASR registration is anticipated to be required for the construction of proposed buildings at this site. This requirement can be confirmed by undertaking a hydrogeological study to determine maximum volume of groundwater inflow requiring dewatering.

6.3 Pipe Bedding Requirements

It is anticipated that the underground services required as part of this project will be founded over silty clay or clay. Alternately, underground services may be founded over properly prepared and approved structural fill, where excavation below the invert is required. Consequently all organic and fill material should be removed down to a suitable bearing layer. Any sub-excavation of disturbed soil should be removed and replaced with a Granular B Type I or approved equivalent, laid in loose lifts of thickness not exceeding 200 mm and compacted to 95% of its SPMDD. Bedding, thickness of cover material and compaction requirements for watermains and sewer pipes should conform to the manufacturers design requirements and to the detailed installations outlined in the Ontario Provincial Standard Specifications (OPSS) and any applicable standards or requirements from the City of Ottawa.

It is anticipated that the watermains and sewers will be founded above the ground water level at silty clay. If and when watermains and sewers are required to be founded below the groundwater table and silty clay will constitute the founding soil below the groundwater, it may be sensitive to disturbances and may also be susceptible to piping and scouring from water pressure at the base of the excavation. Therefore, special precautions should be taken in these areas to stabilize and confine the base of the excavation such as using recompression (thicker bedding) and/or dewatering methods (pre-pumping). In order to properly compact the bedding, the water table should be kept at least 300 mm below the base of the excavation at all time during the installation of the watermains.

As an alternative to Granular A bedding and only where wet conditions are encountered, the use of “clear stone” bedding, such as 19 mm clear stone, **OPSS 1004**, may be considered only in conjunction with a suitable geotextile filter (such as terrafix 270R or approved equivalent). Without proper filtering, there may be entry of fines from native soils and trench backfill into the bedding, which could result in loss of support to the pipes and possible surface settlements. The sub-bedding, bedding and cover materials should be compacted in maximum 200 mm thick lifts to at least 95% of its SPMDD within ±2% of its optimum moisture content using suitable vibratory compaction equipment.

6.4 Trench Backfill

All service trenches should be backfilled using compactable material, free of organics, debris and large cobbles or boulders. Acceptable native materials (if encountered and where possible) should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetrations (i.e. 1.8 m below finished grade) in order to reduce the potential for differential frost heaving between the new excavated trench and the adjacent section of roadway. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I or Granular C. Any boulders larger than 150 mm in size should not be used as trench backfill.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadway, the trench should be compacted in maximum 300 mm thick lifts to at least 95% of its SPMDD. The specified density may be reduced where the trench backfill is not located within or in close proximity to existing roadways or any other structures.

For trenches carried out in existing paved areas, transitions should be constructed to ensure that proper compaction is achieved between any new pavement structure and the existing pavement structure to minimize potential future differential settlement between the existing and new pavement structure. The transition should start at the subgrade level and extend to the underside of the asphaltic concrete level (if any) at a 1 horizontal to 1 vertical slope. This is especially important where trench boxes are used and where no side slopes is provided to the excavation. Where asphaltic concrete is present, it should be cut back to a minimum of 150 mm from the edge of the excavation to allow for proper compaction between the new and existing pavement structures.

7 REUSE OF ON-SITE SOILS

The existing surficial overburden soils consist mostly of silty clay. The overburden silty clay is considered to be frost susceptible and should not be used as backfill material directly against foundation walls or underneath unheated concrete slabs. However, these could be reused as general backfill material (service trenches, general landscaping/backfilling) if it can be compacted according to the specifications outlined herein at the time of construction and found free from any waste, organics and debris. Any imported material shall conform to OPSS Granular B – Type I or approved equivalent.

It should be noted that the adequacy of any material for reuse as backfill will depend on its water content at the time of its use and on the weather conditions prevailing prior to and during that time. Therefore, all excavated materials to be reused shall be stockpiled in a manner that will prevent any significant changes in their moisture content, especially during wet conditions. Any excavated materials proposed for reuse should be stockpiled in a manner to promote drying and should be inspected and approved for reuse by a geotechnical engineer.

8 RECOMMENDED PAVEMENT STRUCTURE

It is anticipated that the subgrade soil for the new parking and access lanes will consist mostly of silty clay. The construction of access lanes and parking areas will be acceptable over the undisturbed silty clay once all debris, organic material, or otherwise deleterious material are removed from the subgrade area. Furthermore, the silty clay must be compacted using a suitable heavy duty compacting equipment and approved by a geotechnical engineer prior to placing any granular base material.

Considering the wet soil condition at the subgrade elevation, the subgrade soil Resilient Modulus is **34.5 MPa**. The calculated minimum Granular Base Equivalency (GBE) is 630 for roadways and access lane. The following **Table 8** presents the recommended pavement structures to be constructed over a stable subgrade along the proposed parking areas and access lane or driveway as part of this project.

Table 8: Recommended Pavement Structure

Course	Material	Thickness (mm)	
		Light Duty Parking Area (mm)	Heavy Duty Parking Area (Access Roads, Fire Routes and Trucks) (mm)
GBE		450	630
Surface	HL3 A/C	50	40
Binder	HL8 A/C	-	50
Base course	Granular A	150	150

Sub base	Granular B Type II	300	450
Total:		500	690

Performance Graded Asphaltic Cement (PGAC) **58-34** is recommended for this project.

The base and subbase granular materials shall conform to **OPSS 1010** material specifications. Any proposed materials shall be tested and approved by a geotechnical engineer prior to delivery to the site and shall be compacted to 100% of its SPMDD. Asphaltic concrete shall conform to **OPSS 1150** and be placed and compacted to at least 93% of the Marshall Density. The mix and its constituents shall be reviewed, tested and approved by a geotechnical engineer prior to delivery to the site.

8.1 Paved Areas & Subgrade Preparation

The access lanes and parking areas shall be stripped of vegetation, debris and other obvious objectionable material. Following the backfilling and satisfactory compaction of any underground service trenches up to the subgrade level, the subgrade shall be shaped, crowned and proof-rolled. A loaded Tandem axle, dual wheel dump truck or approved equivalent heavy duty smooth drum roller shall be used for proof-rolling. Any resulting loose/soft areas should be sub-excavated down to an adequate bearing layer and replaced with approved backfill.

The preparation of subgrade shall be scheduled and carried out in manner so that a protective cover of overlying granular material (if required) is placed as quickly as possible in order to avoid unnecessary circulation by heavy equipment, except on unexcavated or protected surfaces. Frost protection of the surface shall be implemented if works are carried out during the winter season.

The performance of the pavement structure is highly dependent on the subsurface groundwater conditions and maintaining the subgrade and pavement structure in a dry condition. To intercept excess subsurface water within the pavement structure granular materials, sub-drains with suitable outlets should be installed below the pavement area’s subgrade if adequate overland flow drainage is not provided (i.e. ditches). The surface of the pavement should be properly graded to direct runoff water towards suitable drainage features. It is recommended that the lateral extent of the subbase and base layers not be terminated vertically immediately behind the curb/edge of pavement line but be extended beyond the curb.

9 INSPECTION SERVICES

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed site do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

All footing areas and any structural fill areas for the proposed structures should be inspected by LRL to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundations and slab-on-grade should be inspected to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for the pavement areas and underground services should be inspected and approved by geotechnical personnel. In-situ density testing should be carried out on the pavement granular materials, pipe bedding and backfill to ensure the materials meet the specifications for required compaction.

If footings are to be constructed during winter season, the footing subgrade should be protected from freezing temperatures using suitable construction techniques.

10 REPORT CONDITIONS AND LIMITATIONS

It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document or its use by a third party beyond the client specifically listed in the report is neither intended nor authorized by LRL Associates Ltd. Contractors bidding on or undertaking the works 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 construction techniques, schedule, safety and equipment capabilities.

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report.

The recommendations provided in this report are based on subsurface data obtained at the specific boring locations only. Boundaries between zones presented on the borehole are often not distinct but transitional and were interpreted. Experience indicates that the subsurface soil and groundwater conditions can vary significantly between and beyond the test locations. For this reason, the recommendations given in this report are subject to a field verification of the subsurface soil conditions at the time of construction.

The recommendations are applicable only to the project described in this report. Any changes to the project will require a review by LRL Associates Ltd., to insure compatibility with the recommendations contained in this project.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we may be of further services to you, please do not hesitate to contact the undersigned.

Yours truly,
LRL Associates Ltd.

A. GHIRIAN
100501824
06.03.2018

Brad Johnson, B.Sc. Eng
Geotechnical Services

Alireza Ghirian, P. Eng.
Senior Geotechnical Engineer

APPENDIX A

Site and Borehole Location Plan

LRL
ENGINEERING I INGÉNIERIE
5430 Canotek Road I Ottawa, ON, K1J 9G2
www.lrl.ca I (613) 842-3434

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED RESTAURANT AND OFFICE SPACE
1850 WALKLEY ROAD
OTTAWA, ONTARIO

DRAWING TITLE

SITE LOCATION
SOURCE: Map Data 2018 Google

CLIENT	DATE	PROJECT	
MARCELLO'S MARKET AND DELI INC.	JANUARY 23, 2018	170757	FIGURE 1

LRL
ENGINEERING I INGÉNIERIE
5430 Canotek Road I Ottawa, ON, K1J 9G2
www.lrl.ca I (613) 842-3434

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED RESTAURANT AND OFFICE SPACE
1850 WALKLEY ROAD
OTTAWA, ONTARIO

DRAWING TITLE

BOREHOLE LOCATION
SOURCE: Imagery 2018 Google, DigitalGlobe Map Data

CLIENT	DATE	PROJECT	
MARCELLO'S MARKET AND DELI INC.	JANUARY 23, 2018	170757	FIGURE 2

APPENDIX B

Borehole Logs

Borehole Log: BH1

Project No.: 170757

Project: Restaurant and Office Space

Client: Marcello's Market and Deli Inc.

Location: 1850 Walkley Road, Ottawa ON

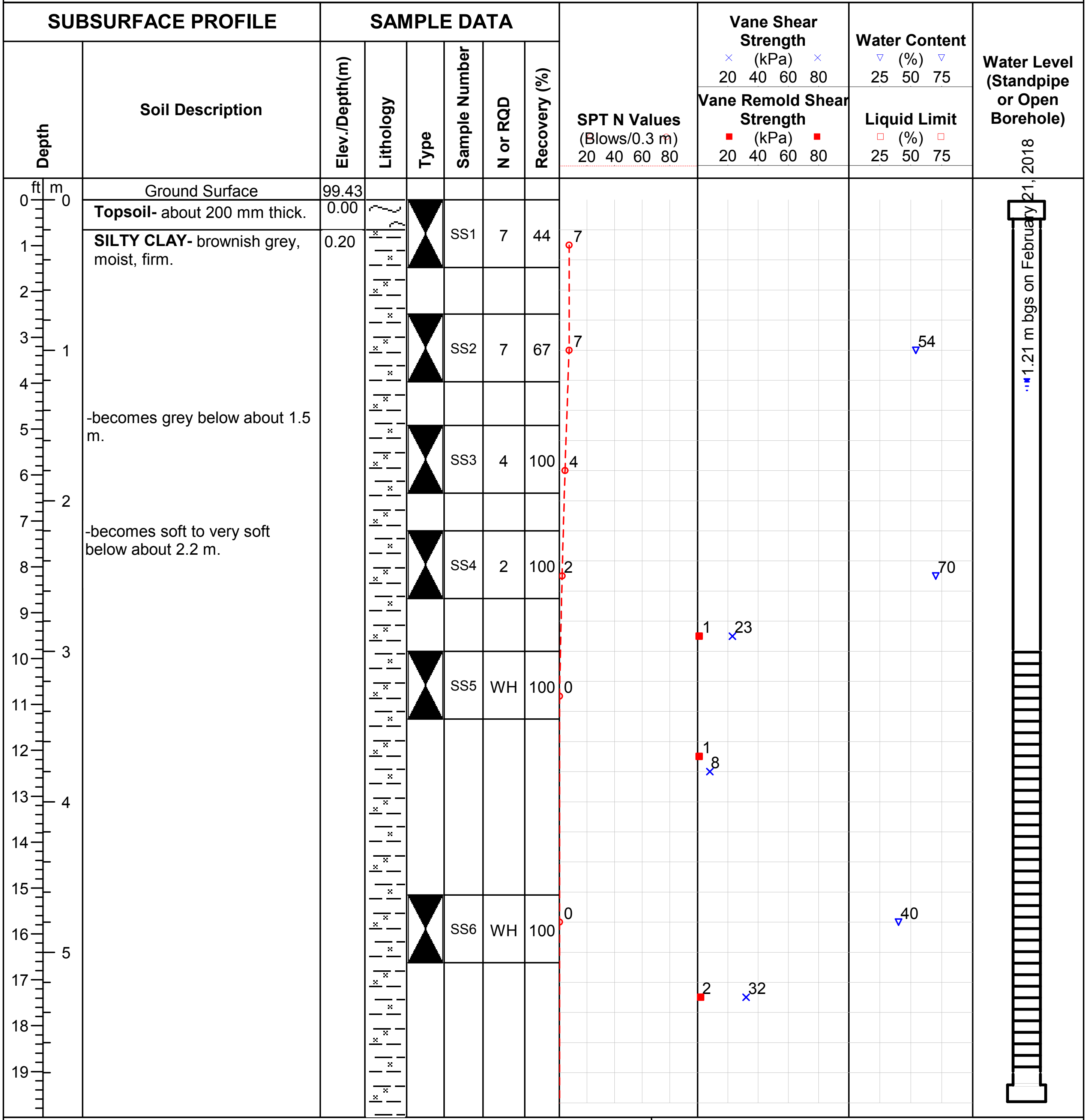
Date: January 23, 2018

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 55

Drilling Method: HSA



Soil Description	Elev./Depth(m)
Ground Surface	99.43
Topsoil- about 200 mm thick.	0.00
SILTY CLAY- brownish grey, moist, firm.	0.20
-becomes grey below about 1.5 m.	
-becomes soft to very soft below about 2.2 m.	

Type	Sample Number	N or RQD	Recovery (%)
	SS1	7	44
	SS2	7	67
	SS3	4	100
	SS4	2	100
	SS5	WH	100
	SS6	WH	100

Easting: 450299

Northing: 5025409

Site Datum: Top of Concrete Pad at Side Entrance Gate of Dymon Storage

Groundsurface Elevation: 99.427

Top of Riser Elev.: 100.527

Hole Diameter: 200 mm

Borehole Log: BH1

Project No.: 170757

Project: Restaurant and Office Space

Client: Marcello's Market and Deli Inc.

Location: 1850 Walkley Road, Ottawa ON

Date: January 23, 2018

Field Personnel: BJ

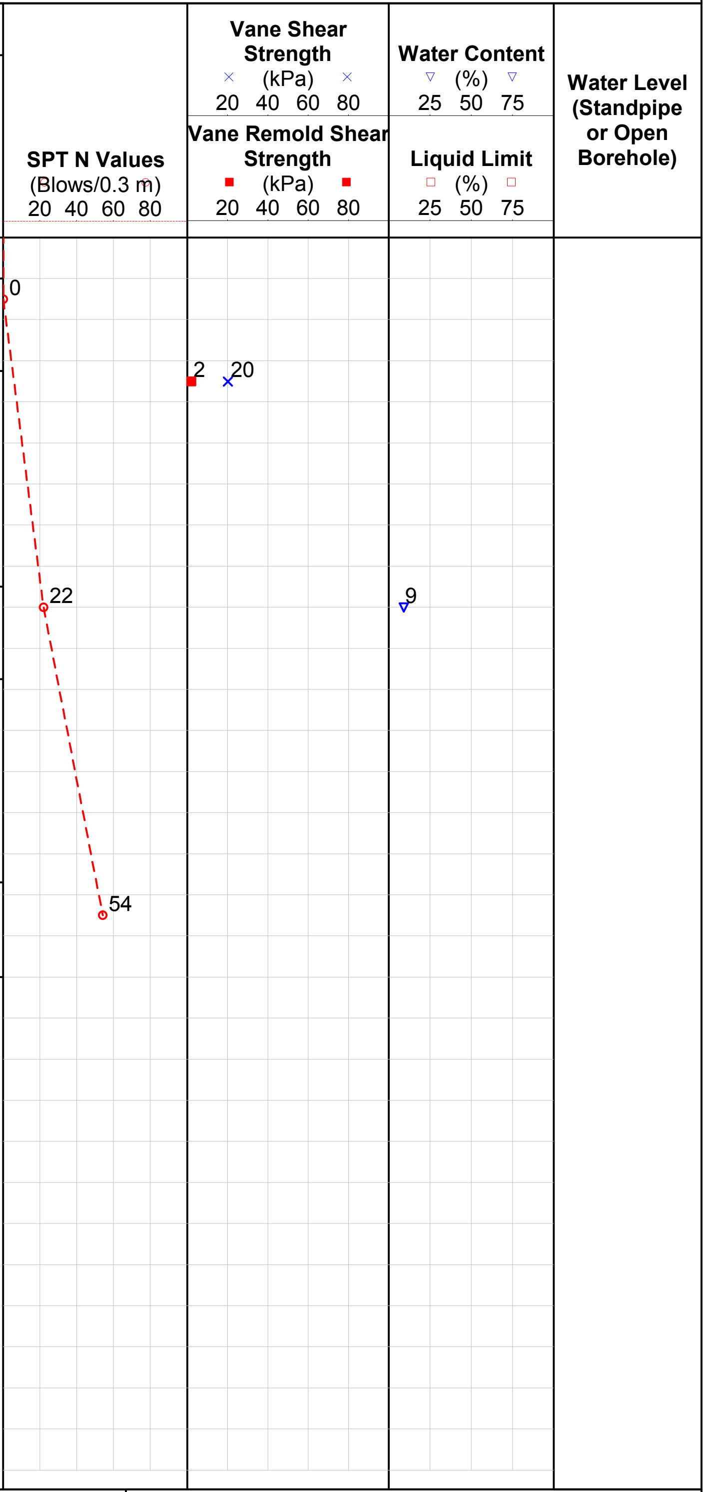
Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 55

Drilling Method: HSA

SUBSURFACE PROFILE / SAMPLE DATA

Depth (ft)	Depth (m)	Soil Description	Elev./Depth(m)	Type	Sample Number	N or RQD	Recovery (%)
20–23							
21				⧗	SS7	WH	100
23.5	7.16	**SILTY SAND TILL-** some clay, some gravel, dark grey, moist, compact to very dense.	92.27 7.16				
26	8			⧗	SS8	22	83
30.5				⧗	SS9	54	0
31.5	9.60	End of Borehole	89.83 9.60				



Easting: 450299

Northing: 5025409

Site Datum: Top of Concrete Pad at Side Entrance Gate of Dymon Storage

Groundsurface Elevation: 99.427

Top of Riser Elev.: 100.527

Hole Diameter: 200 mm

Borehole Log: BH2

Project No.: 170757

Project: Restaurant and Office Space

Client: Marcello's Market and Deli Inc.

Location: 1850 Walkley Road, Ottawa ON

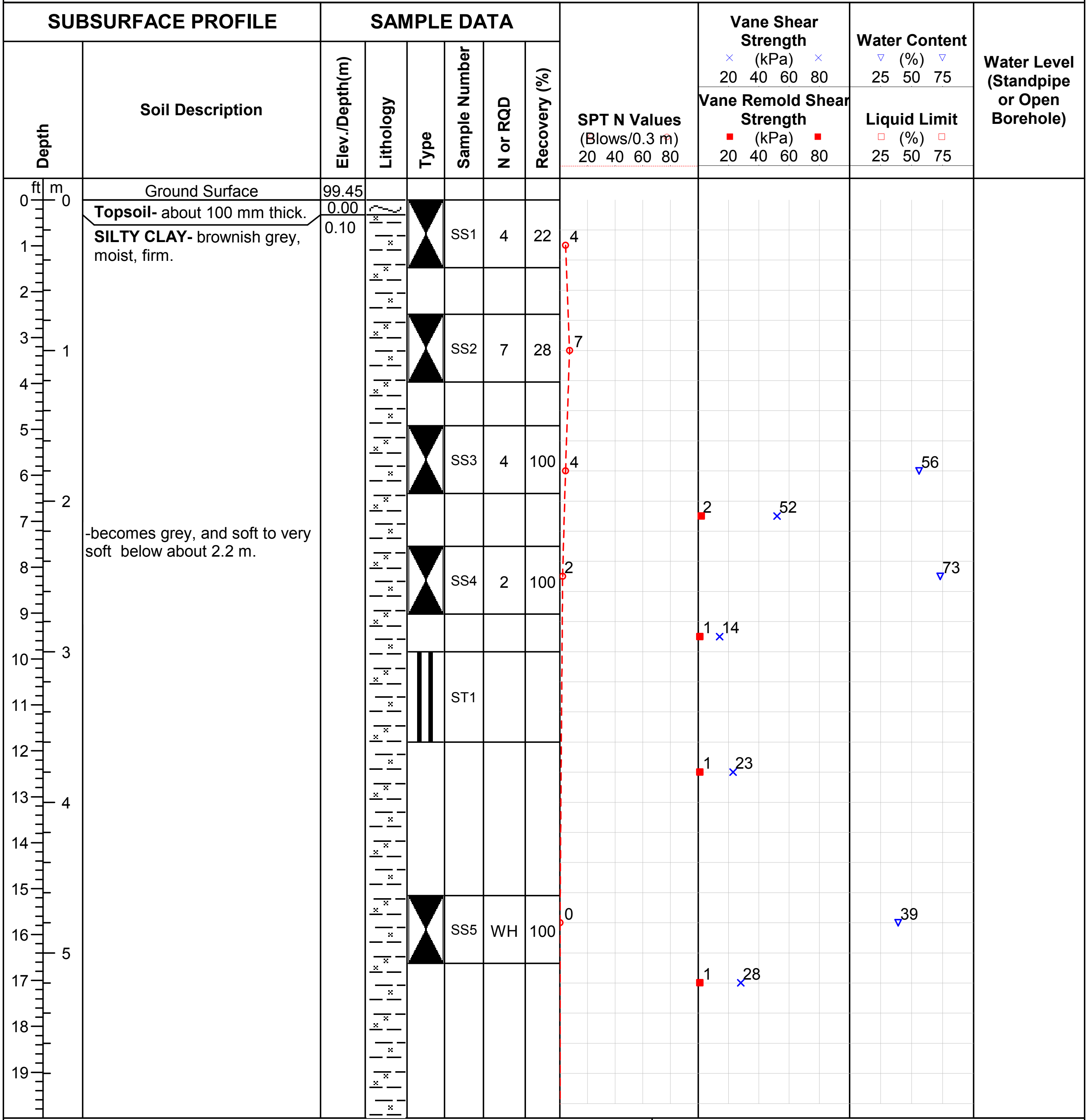
Date: January 23, 2018

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 75

Drilling Method: HSA



Soil Description	Elev./Depth (m)
Ground Surface	99.45
Topsoil- about 100 mm thick.	0.00
SILTY CLAY- brownish grey, moist, firm. -becomes grey, and soft to very soft below about 2.2 m.	0.10

Sample Number	N or RQD	Recovery (%)
SS1	4	22
SS2	7	28
SS3	4	100
SS4	2	100
ST1		
SS5	WH	100

Easting: 450278

Northing: 5025446

Site Datum: Top of Concrete Pad at Side Entrance Gate of Dymon Storage

Groundsurface Elevation: 99.450

Top of Riser Elev.: N/A

Hole Diameter: 200 mm

Borehole Log: BH2

Project No.: 170757

Project: Restaurant and Office Space

Client: Marcello's Market and Deli Inc.

Location: 1850 Walkley Road, Ottawa ON

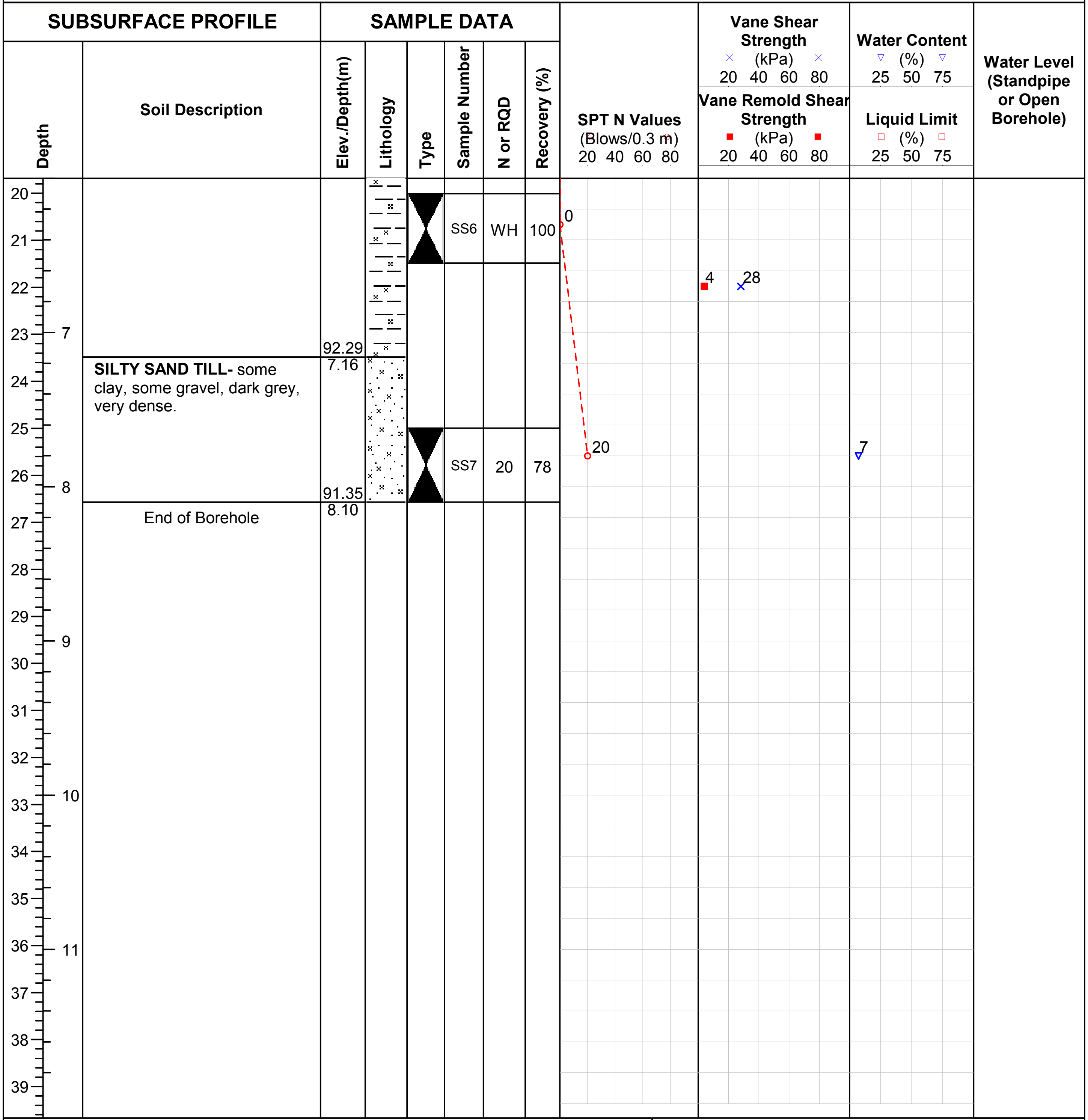
Date: January 23, 2018

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 75

Drilling Method: HSA



Soil Description	Elev./Depth(m)	Type	Sample Number	N or RQD	Recovery (%)
			SS6	WH	100
	92.29 7.16				
SILTY SAND TILL- some clay, some gravel, dark grey, very dense.			SS7	20	78
	91.35 8.10				
End of Borehole					

Easting: 450278

Northing: 5025446

Site Datum: Top of Concrete Pad at Side Entrance Gate of Dymon Storage

Groundsurface Elevation: 99.450

Top of Riser Elev.: N/A

Hole Diameter: 200 mm

Borehole Log: BH3

Project No.: 170757

Project: Restaurant and Office Space

Client: Marcello's Market and Deli Inc.

Location: 1850 Walkley Road, Ottawa ON

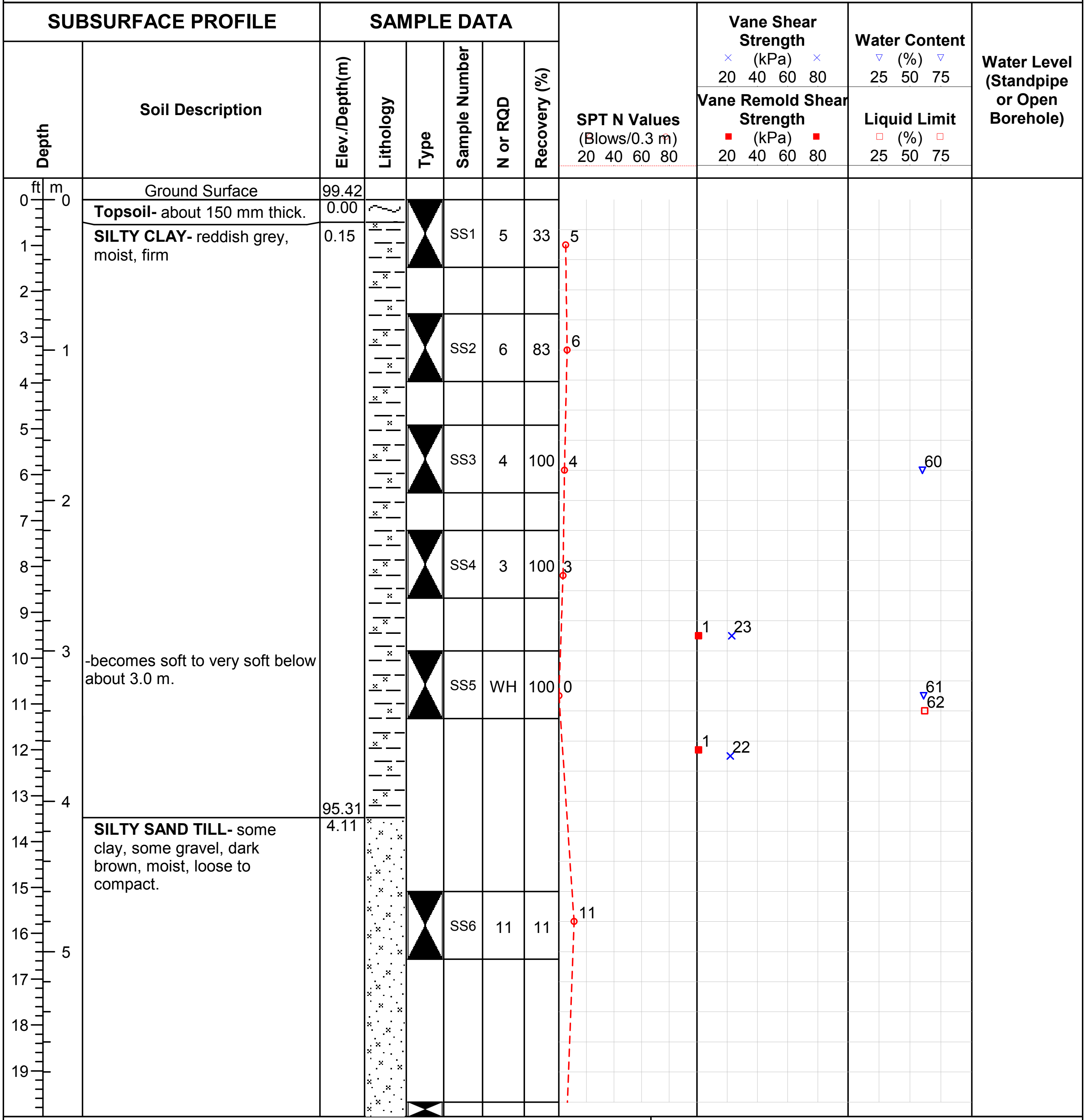
Date: January 22, 2018

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 75

Drilling Method: HSA



Soil Description	Elev./Depth(m)
Ground Surface	99.42
Topsoil- about 150 mm thick.	0.00
SILTY CLAY- reddish grey, moist, firm -becomes soft to very soft below about 3.0 m.	0.15
SILTY SAND TILL- some clay, some gravel, dark brown, moist, loose to compact.	95.31 4.11

Sample Number	N or RQD	Recovery (%)
SS1	5	33
SS2	6	83
SS3	4	100
SS4	3	100
SS5	WH	100
SS6	11	11

Easting: 450253

Northing: 5025503

Site Datum: Top of Concrete Pad at Side Entrance Gate of Dymon Storage

Groundsurface Elevation: 99.423

Top of Riser Elev.: N/A

Hole Diameter: 200 mm

Borehole Log: BH3

Project No.: 170757

Project: Restaurant and Office Space

Client: Marcello's Market and Deli Inc.

Location: 1850 Walkley Road, Ottawa ON

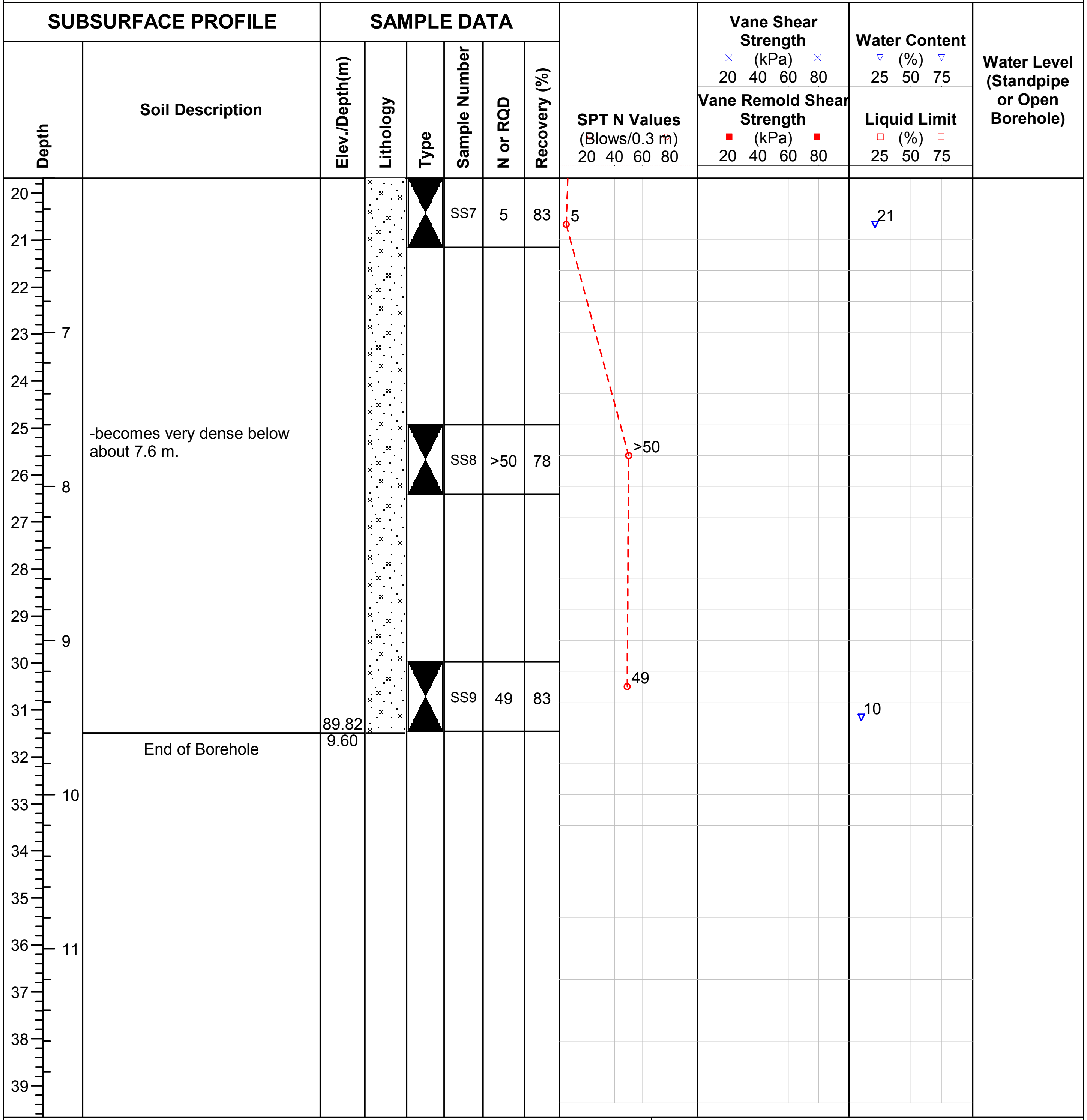
Date: January 22, 2018

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 75

Drilling Method: HSA



-becomes very dense below about 7.6 m.

89.82
9.60

End of Borehole

Sample Number	N or RQD	Recovery (%)
SS7	5	83
SS8	>50	78
SS9	49	83

Easting: 450253

Northing: 5025503

Site Datum: Top of Concrete Pad at Side Entrance Gate of Dymon Storage

Groundsurface Elevation: 99.423

Top of Riser Elev.: N/A

Hole Diameter: 200 mm

APPENDIX C

Symbols and Terms used in Borehole Logs

Symbols and Terms Used on Borehole and Test Pit Logs

1. Soil Description

The soil descriptions presented in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves some judgement and LRL Associates Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice. Boundaries between zones on the logs are often not distinct but transitional and were interpreted.

a. Proportion

The proportion of each constituent part, as defined by the grain size distribution, is denoted by the following terms:

Term	Proportions
“trace”	1% to 10%
“some”	10% to 20%
prefix (i.e. “sandy” silt)	20% to 35%
“and” (i.e. sand “and” gravel)	35% to 50%

b. Compactness and Consistency

The state of compactness of granular soils is defined on the basis of the Standard Penetration Number (N) as per ASTM D-1586. It corresponds to the number of blows required to drive 300 mm of the split spoon sampler using a metal drop hammer that has a weight of 62.5 kg and free fall distance of 760 mm. For a 600 mm long split spoon, the blow counts are recorded for every 150 mm. The “N” value is obtained by adding the number of blows from the 2nd and 3rd count. Technical refusal indicates a number of blows greater than 50.

The consistency of clayey or cohesive soils is based on the shear strength of the soil, as determined by field vane tests and by a visual and tactile assessment of the soil strength.

The state of compactness of granular soils is defined by the following terms:

State of Compactness Granular Soils	Standard Penetration Number “N”	Relative Density (%)
Very loose	0 – 4	<15
Loose	4 – 10	15 – 35
Compact	10 - 30	35 – 65
Dense	30 - 50	65 - 85
Very dense	> 50	> 85

The consistency of cohesive soils is defined by the following terms:

Consistency Cohesive Soils	Undrained Shear Strength (C_u) (kPa)	Standard Penetration Number “N”
Very soft	<12.5	<2
Soft	12.5 - 25	2 - 4
Firm	25 - 50	4 - 8
Stiff	50 - 100	8 - 15
Very stiff	100 - 200	15 - 30
Hard	>200	>30

c. Field Moisture Condition

Description (ASTM D2488)	Criteria
Dry	Absence of moisture, dusty, dry to touch.
Moist	Dump, but not visible water.
Wet	Visible, free water, usually soil is below water table.

2. Sample Data

a. Elevation depth

This is a reference to the geodesic elevation of the soil or to a benchmark of an arbitrary elevation at the location of the borehole or test pit. The depth of geological boundaries is measured from ground surface.

b. Type

Symbol	Type	Letter Code
	Auger	AU
	Split Spoon	SS
	Shelby Tube	ST
	Rock Core	RC

c. Sample Number

Each sample taken from the borehole is numbered in the field as shown in this column.

LETTER CODE (as above) – Sample Number.

d. Recovery (%)

For soil samples this is the percentage of the recovered sample obtained versus the length sampled. In the case of rock, the percentage is the length of rock core recovered compared to the length of the drill run.

3. Rock Description

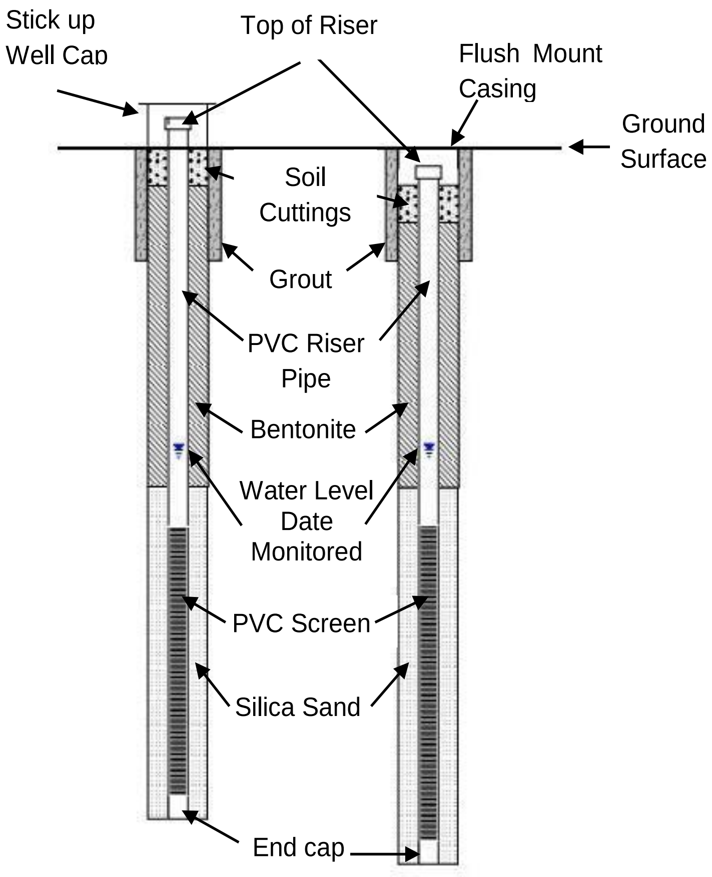
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given below.

Rock Quality Designation (RQD) (%)	Description of Rock Quality
0 –25	Very poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

Strength classification of rock is presented below.

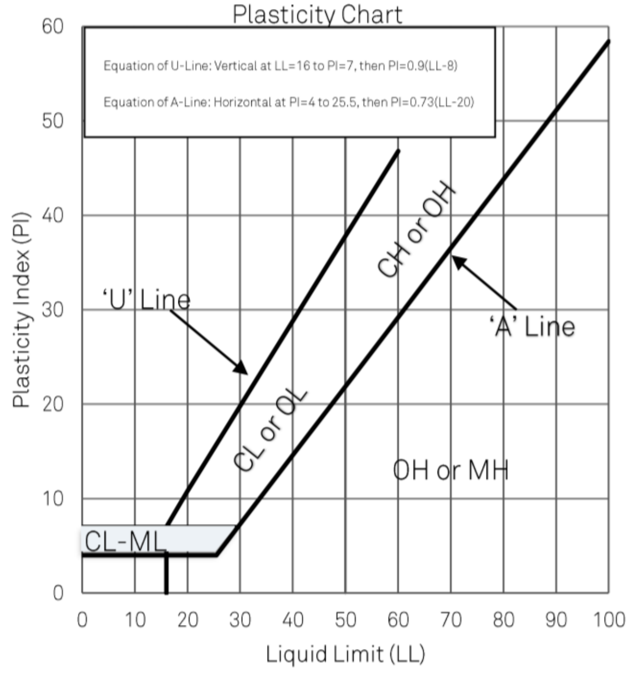
Strength Classification	Range of Unconfined Compressive Strength (MPa)
Extremely weak	< 1
Very weak	1 – 5
Weak	5 – 25
Medium strong	25 – 50
Strong	50 – 100
Very strong	100 – 250
Extremely strong	> 250

4. General Monitoring Well Data



5. Classification of Soils for Engineering Purposes (ASTM D2487)

(United Soil Classification System)

Major divisions			Group Symbol	Typical Names	Classification Criteria			
Coarse-grained soils More than 50% retained on No. 200 sieve* (>0.075 mm)	Gravels More than 50% of coarse fraction retained on No. 4 sieve(4.75 mm)	Clean gravels <5% fines	GW	Well-graded gravel	If 15% sand add "with sand" to group name.	Classification on basis of percentage of fines: Less than 5% pass No. 200 sieve - GW, GP, SW, SP More than 12% pass No. 200 sieve - GM, GC, SM, SC 5 to 12% pass No. 200 sieve - Borderline classifications, use of dual symbols	$C_u = \frac{D_{60}}{D_{10}} \geq 4$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravel			Not meeting either Cu or Cc criteria for GW	
		Gravels with >12% fines	GM	Silty gravel			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols If fines are organic add "with orgnic fines" to group name
			GC	Clayey gravel			Atterberg limits on or above "A" line and PI > 7	
	Sands 50% or more of coarse fraction passes No. 4 sieve(<4.75 mm)	Clean sands <5% fines	SW	Well-graded sand	If 15% gravel add "with gravel to group name		$C_u = \frac{D_{60}}{D_{10}} \geq 6$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sand			Not meeting either Cu or C ccriteria for SW	
		Sands with >12% fines	SM	Silty sand			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols If fines are organic add "with orgnic fines" to group name
			SC	Clayey sand			Atterberg limits on or above "A" line and PI > 7	
Fine-grained soils50% or more passes No. 200 sieve* (<0.075 mm)	Silts and Clays Liquid Limit <50%	Inorganic	ML	Silt	If 15 to 29% coarse-grained, add "with sand" or "with gravel" as appropriate. If > 30% coarse-grained, add "sandy" or "gravelly" as approprieate. Class as organic when oven dried liquid limit is < 75% of undried liquid limit.	 		
			CL	Lean Clay -low plasticity				
		Organic	OL	Organic clay or silt (Clay plots above 'A' Line)				
	Silts and Clays Liquid Limit >50%	Inorganic	MH	Elastic silt				
			CH	Fat Clay -high plasticity				
		Organic	OH	Organic clay or silt (Clay plots above 'A' Line)				
	Highly Organic Soils		PT	Peat, muck and other highly organic soils				

APPENDIX D

Laboratory Results

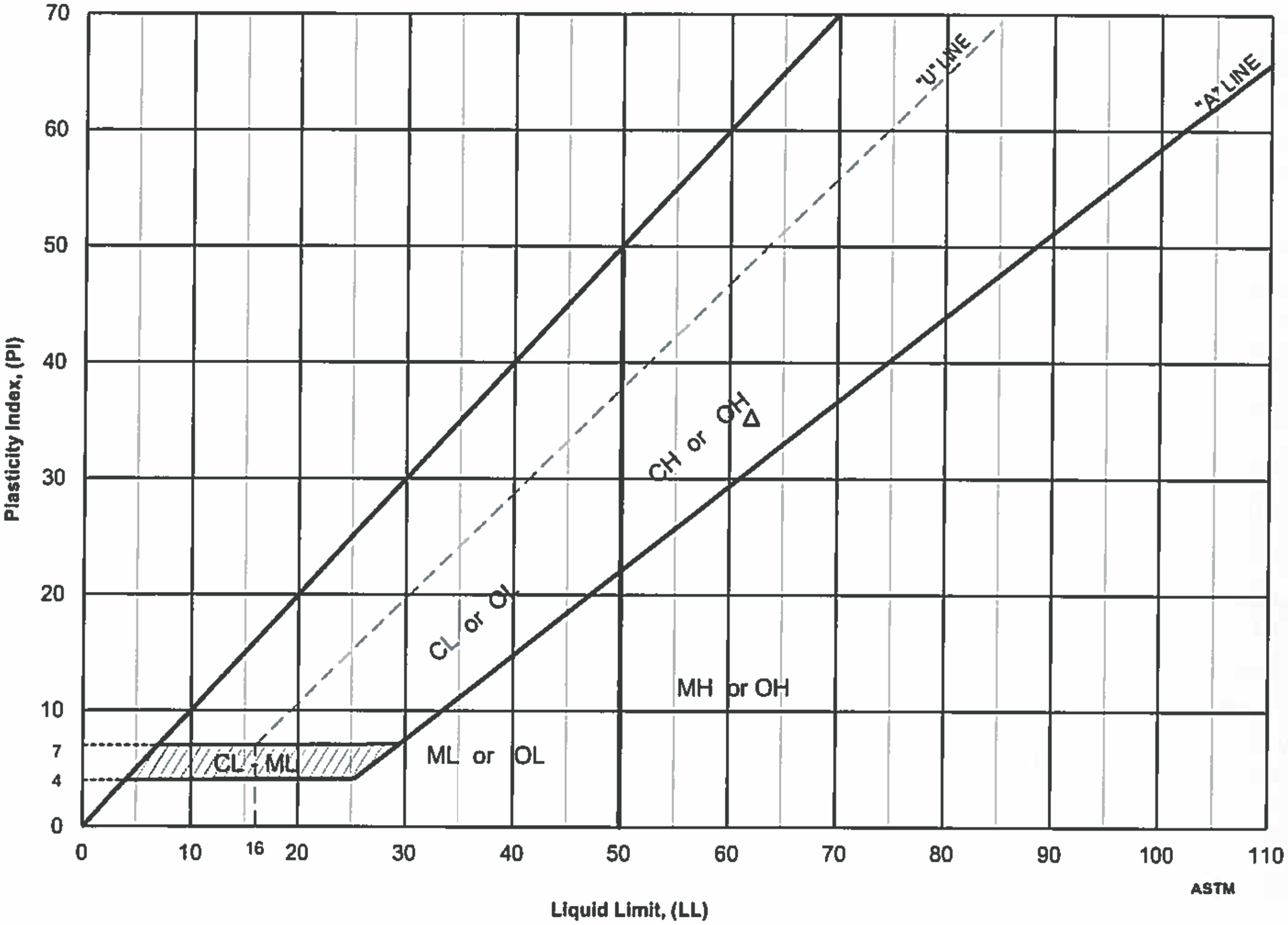
LRL Associates Ltd.

PLASTICITY INDEX

ASTM D 4318 / LS-703/704

Client:	Marcello's Market & Deli Inc.	**File No.:**	170757
Project:	Geotechnical Investigation	**Report No.:**	1
Location:	1850 Walkley Road, Ottawa, ON.	**Date:**	January 22, 2018

Plasticity Chart



	Location	Sample	Depth, m	Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Activity Number	USCS
△	BH 3	SS-5	3.05 - 3.51	61	62	27	35	0.96	n/d	CH

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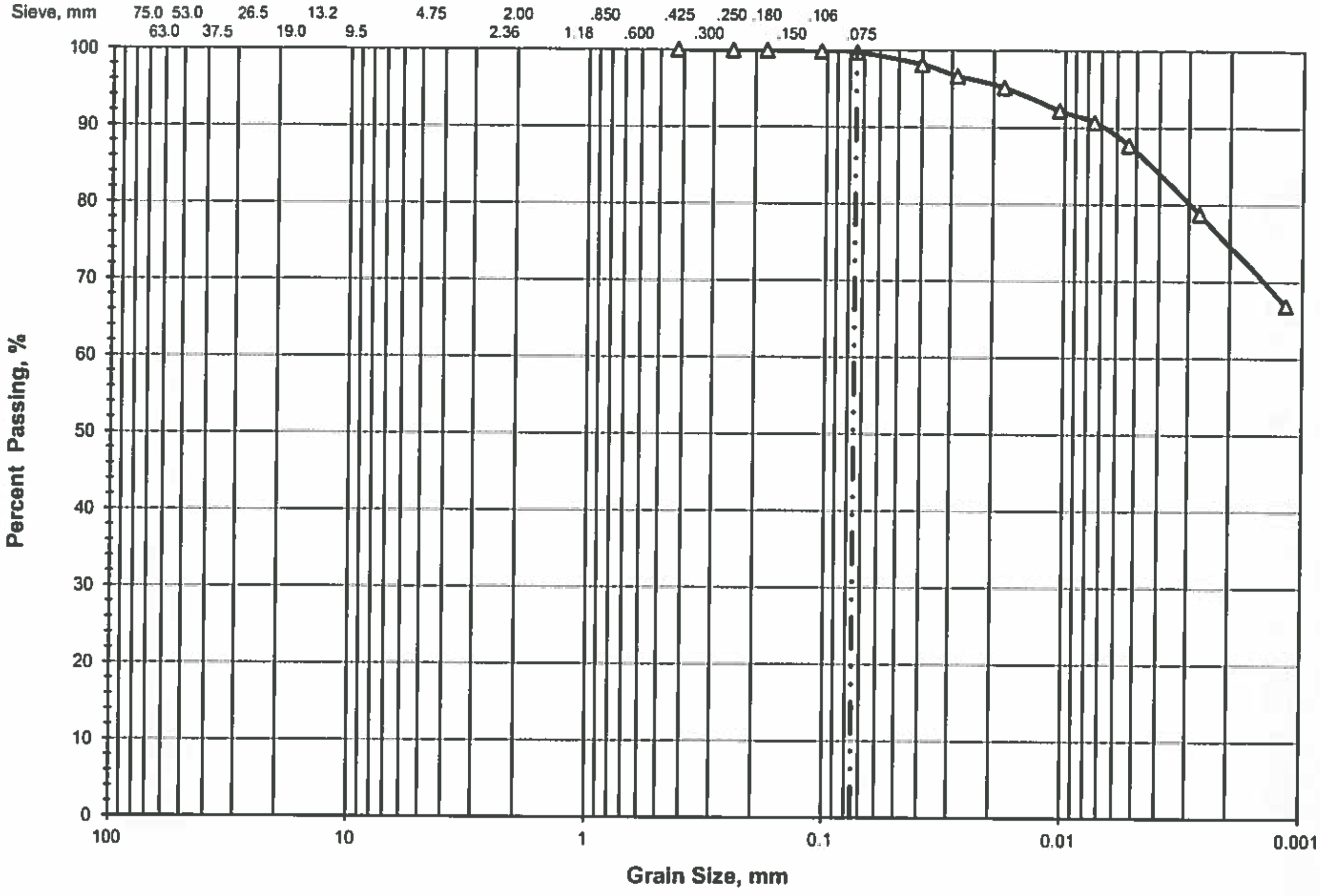
LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client: Marcello's Market & Deli Inc.
Project: Geotechnical Investigation
Location: 1850 Walkley Road, Ottawa, ON.

File No.: 170757
Report No.: 2
Date: January 22, 2018



Unified Soil Classification System

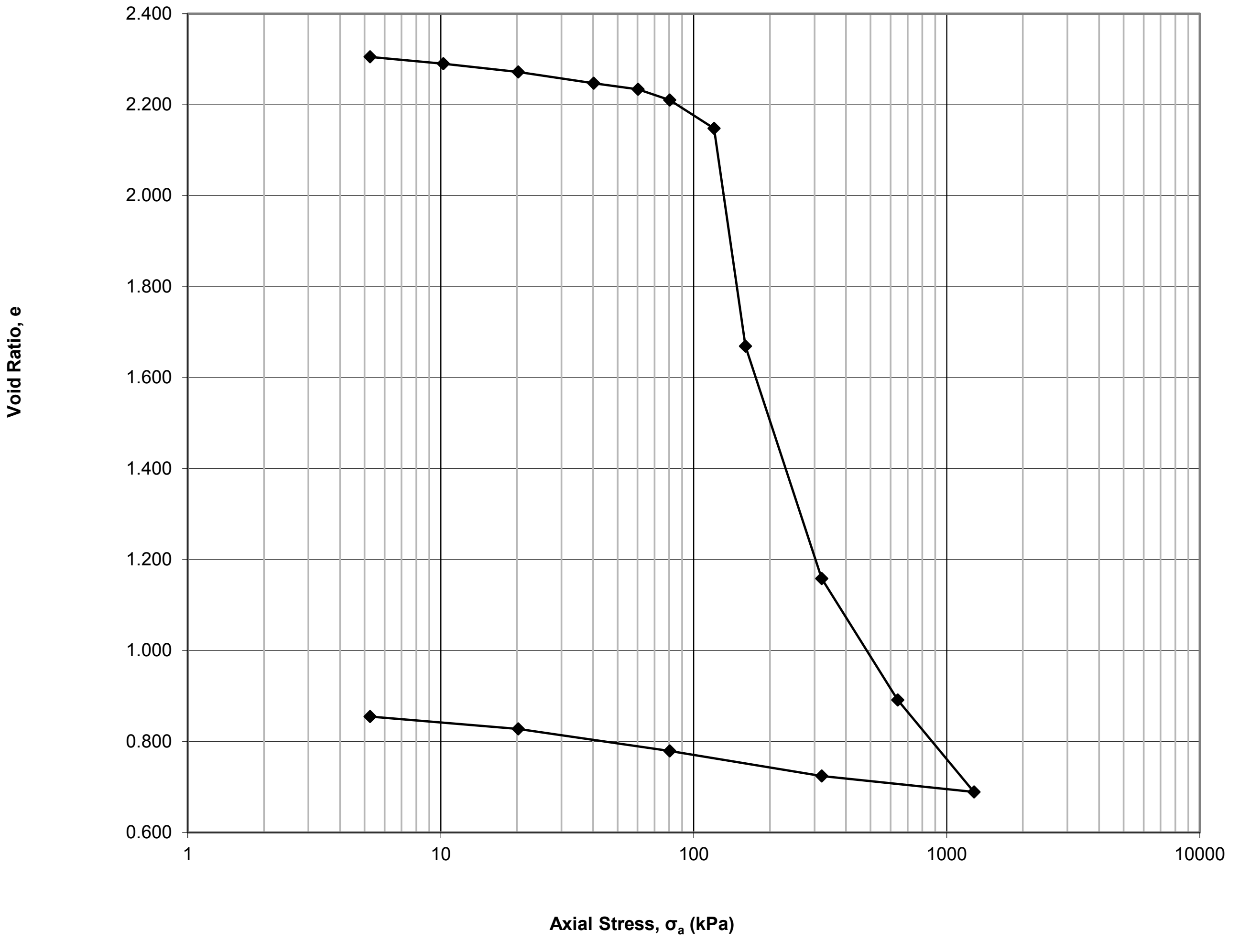
	> 75 mm	% GRAVEL		% SAND			% FINES	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
△	0.0	0.0	0.0	0.0	0.0	0.3	26.4	73.3

	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH1	SS-3	1.52 - 1.98							

One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D2435/D2435M - 11

Project	**LRL Associates Engineers, File# 170757**
Project No.	**122410526**
Borehole No.	**BH 2**
Sample No.	**BH 2**
Sample Depth	**10-12 ft.**



Checked by: D. Boateng Date: 21-Feb-18
Approved by: R. Hache Date: 21-Feb-18

Stantec Consulting Ltd.

One-Dimensional Consolidation Test using Incremental Loading

ASTM D2435/D2435M - 11

Specimen Details

Project Name	LRL Associates Engineers, File# 170757
Project Location	1850 Walkley Road, Ottawa, ON
Borehole	BH 2
Sample No.	BH 2
Depth	10-12 ft.
Sample Date	January 22, 2018
Test Number	One
Technician Name	Daniel Boateng

Soil Description & Classification

Silty Clay, Brown/Grey, Fissured, Wet

Specific Gravity of Solids		2.750
Average water content of trimmings	%	79

Additional Notes (information source, occurence and size of large isolated particles etc.)

Specific Gravity of Solids Assumed

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm^2	1963
Volume	mm^3	39270
Mass	g	57.31
Dry Mass	g	31.99
Density	Mg/m^3	1.459
Dry Density	Mg/m^3	0.815
Water Content	%	79.15
Degree of Saturation	%	91.6
Height of Solids	mm	5.92
Initial Void Ratio		2.376

Final Specimen Conditions

Water Content	%	45.30
Final Void Ratio D		0.855

Stantec

Stantec Consulting Ltd.

One-Dimensional Consolidation Test using Incremental Loading

ASTM D2435/D2435M - 11

Specimen Details

Project Name	LRL Associates Engineers, File# 170757
Project Location	1850 Walkley Road, Ottawa, ON
Borehole	BH 2
Sample No.	BH 2
Depth	10-12 ft.
Sample Date	January 22, 2018
Test Number	One
Technician Name	Daniel Boateng

Test Procedure

Date Started	February 2, 2018
Date Finished	February 17, 2018
Machine Number	Frame E
Cell Number	E
Ring Number	E
Trimming Procedure	Turntable
Moisture Condition	Inundated
Axial Stress at Inundation kPa	5
Water Used	Distilled
Test Method	A
Interpretation Procedure for c_v	2

All Departures from Outlined ASTM D2435/D2435M-11 Procedure

Calculations

Load Increment	Increment Duration min	Axial Stress σ_a kPa	Corrected Deformation ΔH mm	Specimen Height H mm	Axial Strain ε_a %	Void Ratio e
Seating	0.0	5	0.0000	20.0000	0.00	2.376
1	1440.0	5	0.4195	19.5805	2.10	2.305
2	1440.0	10	0.5063	19.4937	2.53	2.290
3	1440.0	20	0.6169	19.3831	3.08	2.272
4	1440.0	40	0.7621	19.2379	3.81	2.247
5	1440.0	60	0.8408	19.1592	4.20	2.234
6	1440.0	80	0.9822	19.0178	4.91	2.210
7	1440.0	120	1.3522	18.6478	6.76	2.148
8	1440.0	160	4.1862	15.8138	20.93	1.669
9	1440.0	320	7.2137	12.7863	36.07	1.158
10	1440.0	640	8.7943	11.2057	43.97	0.891
11	1440.0	1280	9.9931	10.0069	49.97	0.689
12	1440.0	320	9.7847	10.2153	48.92	0.724
13	1440.0	80	9.4591	10.5409	47.30	0.779
14	1440.0	20	9.1711	10.8289	45.86	0.828
15	1440.0	5	9.0115	10.9885	45.06	0.855

Filename: V:\01216\active\laboratory_standing_offers\2018 Laboratory Standing Offers\122‹
Checked by: D. Boateng Date: 21-Feb-18
Date: 21-Feb-18 Approved by: R. Hache Date: 21-Feb-18

One-Dimensional Consolidation Test using Incremental Loading

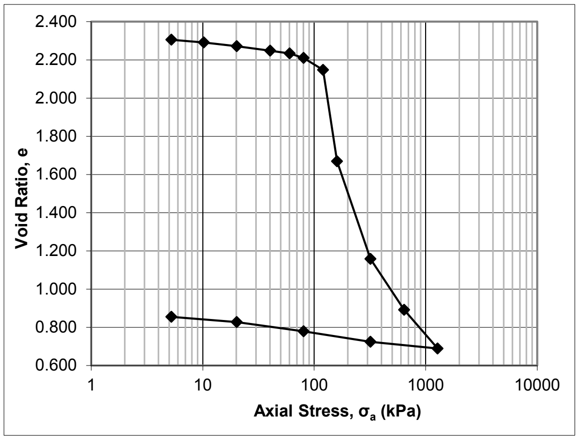
ASTM D2435/D2435M - 11

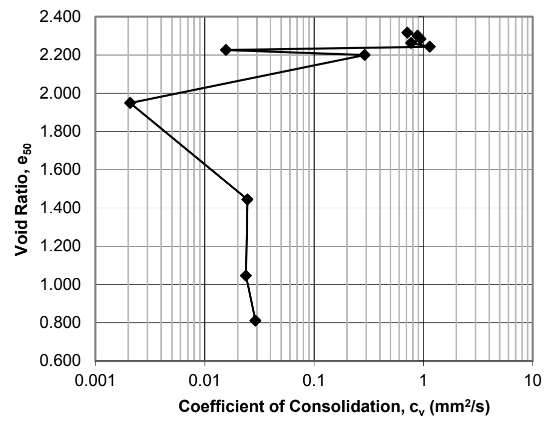
Specimen Details

Project Name	LRL Associates Engineers, File# 170757
Project Location	1850 Walkley Road, Ottawa, ON
Borehole	BH 2
Sample No.	BH 2
Depth	10-12 ft.
Sample Date	January 22, 2018
Test Number	One
Technician Name	Daniel Boateng

Calculations

		Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
Load Increment	Axial Stress $\sigma_{a,\ average}$ kPa	Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\varepsilon_{a,50}$ %	Void Ratio e_{50}	Time t_{50} sec	Coeff. Consol. c_v mm^2/s	Time t_{90} sec	Coeff. Consol. c_v mm^2/s
Seating	3								
1	5	0.3595	19.6405	1.80	2.315			114	7.16E-01
2	8	0.4454	19.5546	2.23	2.301			91	8.88E-01
3	15	0.5515	19.4485	2.76	2.283			85	9.42E-01
4	30	0.6771	19.3229	3.39	2.262			102	7.73E-01
5	50	0.7867	19.2133	3.93	2.243			68	1.15E+00
6	70	0.8910	19.1090	4.46	2.225			4938	1.57E-02
7	100	1.0480	18.9520	5.24	2.199			262	2.91E-01
8	140	2.5307	17.4693	12.65	1.949			31133	2.08E-03
9	240	5.5228	14.4772	27.61	1.444			1807	2.46E-02
10	480	7.8784	12.1216	39.39	1.046			1307	2.38E-02
11	960	9.2773	10.7227	46.39	0.810			837	2.91E-02
12	800	9.8707	10.1293	49.35	0.710				
13	200	9.6283	10.3717	48.14	0.751				
14	50	9.3568	10.6432	46.78	0.796				
15	13	9.1308	10.8692	45.65	0.835				





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Date: 21-Feb-18
Checked by: D. Boateng
Date: 21-Feb-18
Approved by: R. Hache
Date: 21-Feb-18

Stantec	Project No.: 122410526	Photo Log
	Project Name: LRL Associates, File# 170757	

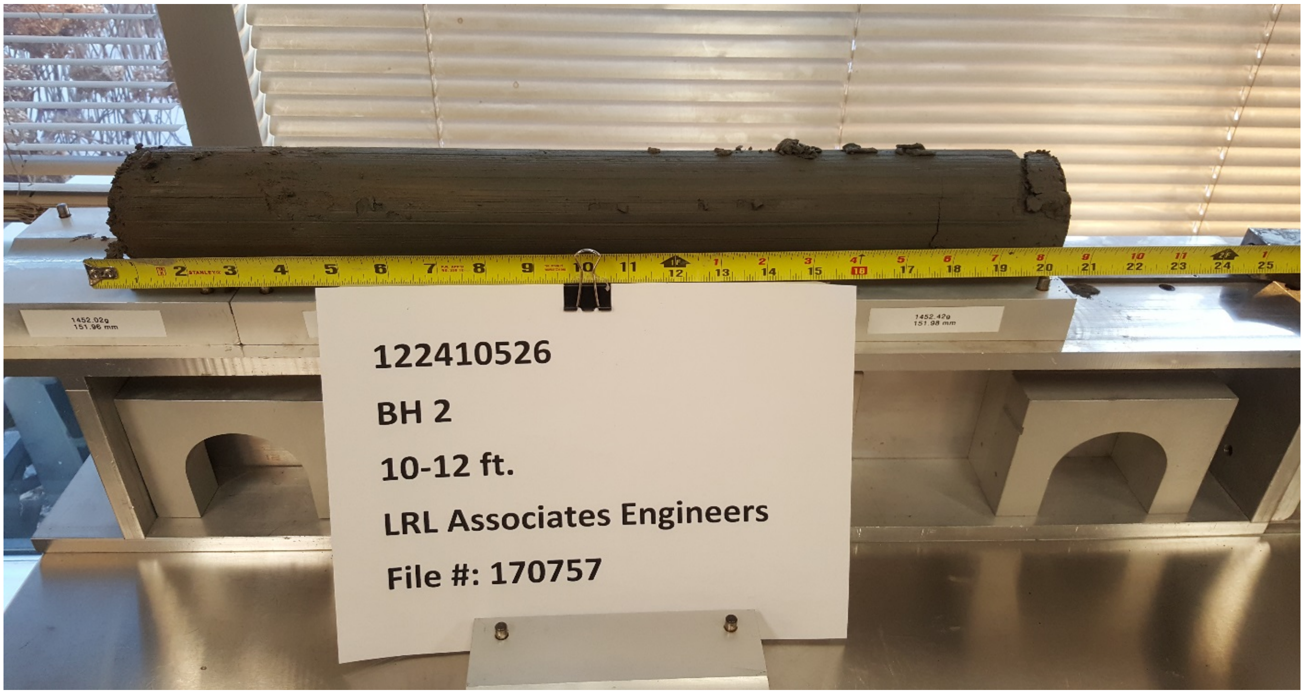

Photo No.: 1	Borehole: BH 2	Depth: 10 – 12 ft.

Photo No.: 2	Borehole: BH 2	Depth: 10 – 12 ft.

Stantec	Project No.: 122410526	Photo Log
	Project Name: LRL Associates, File# 170757	

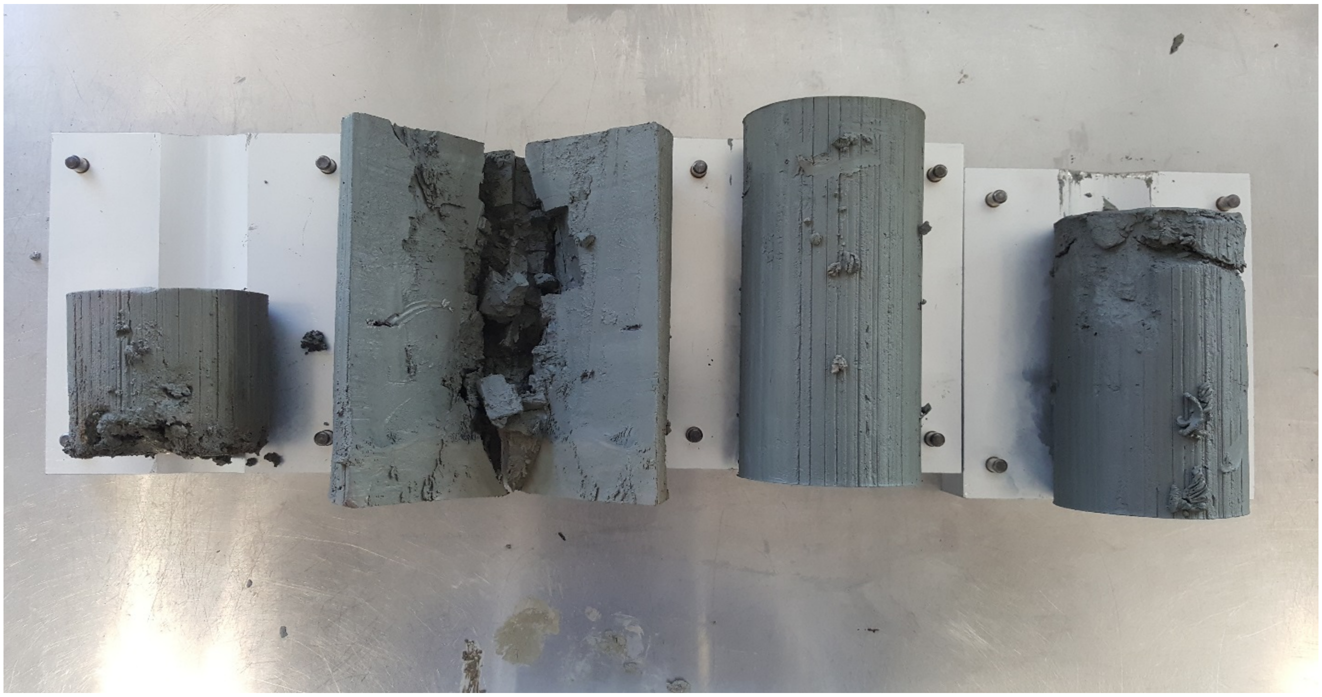

Photo No.: 3	Borehole: BH 2	Depth: 10 – 12 ft.

Photo No.: 4	Borehole: BH 2	Depth: 10 – 12 ft.

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Certificate of Analysis

LRL Associates Ltd.

5430 Canotek Road
Ottawa, ON K1J 9G2
Attn: Brad Johnson

Client PO:
Project: 170757
Custody: 114627

Report Date: 2-Mar-2018
Order Date: 26-Feb-2018

Order #: 1809037

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1809037-01	BH3: 5-6.5' SS3

Approved By:

Dale Robertson, BSc
Laboratory Director

Certificate of Analysis
Client: LRL Associates Ltd.
Client PO:

Report Date: 02-Mar-2018
Order Date: 26-Feb-2018
Project Description: 170757

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	27-Feb-18	27-Feb-18
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	28-Feb-18	28-Feb-18
Resistivity	EPA 120.1 - probe, water extraction	2-Mar-18	2-Mar-18
Solids, %	Gravimetric, calculation	1-Mar-18	1-Mar-18

Certificate of Analysis
Client: LRL Associates Ltd.
Client PO:

Report Date: 02-Mar-2018
Order Date: 26-Feb-2018
Project Description: 170757

	Client ID:	BH3: 5-6.5' SS3	-	-	-
	Sample Date:	22-Jan-18	-	-	-
	Sample ID:	1809037-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	64.4	-	-	-
General Inorganics					
pH	0.05 pH Units	6.49 [1]	-	-	-
Resistivity	0.10 Ohm.m	32.3	-	-	-
Anions					
Chloride	5 ug/g dry	78 [1]	-	-	-
Sulphate	5 ug/g dry	60 [1]	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	7.5	5	ug/g dry	8.2			9.1	20	
Sulphate	6.09	5	ug/g dry	6.27			2.9	20	
General Inorganics									
pH	5.18	0.05	pH Units	5.18			0.0	10	
Resistivity	33.3	0.10	Ohm.m	32.3			3.0	20	
Physical Characteristics									
% Solids	87.7	0.1	% by Wt.	90.1			2.7	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	101	5	ug/g	8.2	92.7	78-113			
Sulphate	112	5	ug/g	6.27	106	78-111			

Qualifier Notes:

Login Qualifiers :

Sample - One or more parameter received past hold time - Chloride, pH, Sulphate.
Applies to samples: BH3: 5-6.5' SS3

Sample Qualifiers :

1 : Holding time had been exceeded upon receipt of the sample at the laboratory.

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.