Geotechnical Investigation

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Mer Bleue Road
Orleans, Ontario

Prepared for:

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1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by Smart Centres to perform a geotechnical investigation for a proposed commercial development located east of Mer Bleue Road, in Orleans, 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 vacant land, it is irregular in shape, and has a total surface area of about 126,870 m^2 (31.35 acres). The site location is presented in Figure 1 included in **Appendix A**. Some trees were sparsely vegetated around central location of the site. Based on the presence of old corn stalks observed at the time of the investigation, it is assumed this area had been used for agricultural purposes in the past. The terrain at the proposed site is considered to be relatively flat.

It is our understanding that the proposed development will consist of a site to be sub-divided into seven (7) lots, labelled Lot 1 through Lot 7. Further, it is understood that the development will include several low-rise to high-rise buildings, local roads, access lanes and parking areas. This report must be considered as a preliminary investigation and was developed to provide a general overview of the subsurface conditions across the site. It is recommended to conduct a more thorough investigation on a lot by lot basis after they have been sub-divided and prior to any type of development.

3 PROCEDURE

The fieldwork for this investigation was carried out on April 5 and 6, 2018. Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of seven (7) boreholes, labelled BH1 through BH7, were drilled inside the property based on the proposed location of the future building developments and 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 3.35 to 7.62 m below ground surface (bgs). Dynamic Cone Penetration Test (DCPT) was advanced beyond the termination depth in BH1, BH2, BH3 and BH4 to depths ranging from 7.06 to 11.32 m bgs. Data was recorded for every 0.3 m of the cone being advanced (blows per 0.3 m). Upon completion, stand pipe piezometers were installed in all boreholes.

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 “T/G of Existing Sanitary #9 - South of Existing LCBO Building Adjacent to the Site.” as a Temporary Bench Mark (TBM). The TBM has an elevation of 87.86 m, obtained from the “Issued for Rezoning Application” drawing, dated December 12, 2016, generated by Stantec Consulting Ltd. 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 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 about 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 brownish grey to grey silty clay with trace sand was encountered at all boring locations, and it extended to depths ranging from 2.54 to 11.32 m bgs. Standard penetration tests were carried out in the silty clay material and the SPT "N" values were found ranging from 9 to WH. The natural moisture content was found varying between 37 and 91%. In-situ field vane shear test using a 150 x 65 mm tapered vane was carried-out in the silty clay deposit. The undrained shear strength value was calculated following the procedure **ASTM D2573**. The initial in-situ values were found ranging from 27 to greater than 100 kPa, and the remold values ranging between 1 and 6 kPa. The upper portion of the silty clay deposit with thickness of approximately 2.4 to 3.0 m is weathered to a stiff to very stiff brownish grey silty clay crust with undrained shear strength value of more than 100 kPa, estimated based on the soil penetrometer tests. Below the crust layer, the silty clay turns to a wet, soft to firm deposit with an average undrained shear strength value of 36 kPa, estimated based on the field vane shear tests.

According to the Canadian Engineering Foundation Manual (CFEM, 2006), the silty clay deposit is considered to be highly sensitive (or quick) with an average soil sensitivity value of 22.

Three (3) soil samples were collected from BH1, BH3 and BH4 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 samples can generally be classified as silty clay, trace sand. Details of laboratory analyses are reflected in **Table 1**.

Table 1: Gradation Analysis Summary – Silty Clay

Sample Location	Depth (m)	Percent for Each Soil Gradation					Estimated Hydraulic Conductivity K (cm/s)
		Sand			Silt (%)	Clay (%)	
		Coarse (%)	Medium (%)	Fine (%)			
BH1	1.2 – 1.8	0.0	0.1	0.6	35.3	64.0	1×10^{-8}
BH3	2.4 – 3.1	0.0	0.0	1.2	28.8	70.0	1×10^{-8}
BH4	5.8 – 6.4	0.0	0.0	0.1	36.6	63.3	1×10^{-8}

Atterberg limits and moisture contents were conducted on four (4) split spoon soil samples from BH1 – BH4. Based on the obtained values, it was determined 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
BH1	1.2 – 1.8	77	29	48	52	CH
BH2	6.1 – 6.7	72	28	44	75	CH
BH3	2.4 – 3.1	81	30	51	72	CH
BH4	5.8 – 6.4	64	28	36	83	CH

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

4.4 Glacial Till

Underneath the silty clay in BH3, BH5, BH6 and BH7, a deposit of glacial till was encountered, and extended to auger refusal depths ranging from of 3.35 to 6.15 m bgs.

The glacial till can be classified as silty gravel with sand (or silty sand with gravel) and trace of cobbles and boulders. The recorded SPT “N” values of this deposit were greater than 50 blows per 50 mm of sampler penetration, indicating the deposit is dense 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 14 and 21%.

One (1) soil sample was collected from BH6 from a depth of 3.1 – 3.5 m bgs for gradation sieve analysis. The result is summarized in **Table 3.** According to the Unified Soil Classification System, the soil samples can generally be classified as silty gravel, gravel-sand-silt mixture.

Table 3: Gradation Analysis Summary – Glacial Till

Sample Location	Depth (m)	Percent for Each Soil Gradation						Estimated Hydraulic Conductivity K (cm/s)
		Gravel		Sand			Fines	
		Coarse (%)	Fine (%)	Coarse (%)	Medium (%)	Fine (%)	Silt & Clay (%)	
BH6	3.1 – 3.5	4.7	33.1	13.3	10.5	11.1	27.3	5×10^{-3}

4.5 Refusal at Inferred Boulder or Bedrock

DCPT refusal over inferred large boulder or bedrock was encountered in three (3) boreholes including BH1, BH2 and BH4 at depths ranging from 7.06 to 11.32 m bgs. The DCPT refusal was considered when SPT “N” value 50 blows was recorded per 50 mm of cone penetration in first 0.15 m.

Bedrock was not encountered during this investigation, however, reviewing of geological maps and available publications indicated that the bedrock underlain the glacial till mostly consists of hard Ordovician dolostone (dolomite) and limestone.

4.6 Groundwater Conditions

For long term water level monitoring, a 19 mm standpipe piezometer was installed in all the boreholes. The piezometers were measured on April 19, 2018, and the water levels are summarized in **Table 4** below. In addition, the water level measurements are shown on the borehole logs presented in **Appendix B.**

Table 4: Water Level Readings

Location	Water Level (m bgs.)
BH1	N/A*
BH2	1.8
BH3	0.6
BH4	0.9
BH5	2.2
BH6	0.6
BH7	0.7

*Water level reading was not considered in this study. The recorded value was not consistent with the moisture contents of the split spoon samples obtained during the laboratory testing.

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 proposed development based on our interpretation of the information gathered from the boreholes performed at this site and from the project requirements.

Also, this section provides the general requirements and limitations with regard to foundation types, allowable foundation bearing pressure and depth, grade raise and size of the footings.

5.1 Foundations

The site under investigation is underlain by a silty clay deposit with variable depths ranging from 2.54 to 11.3 m bgs, as encountered in this borehole investigation. The thickness of the silty clay deposit is minimal in the middle portion of the site where shallow bedrock was encountered, as reported in previous geotechnical investigation completed by Paterson Group (2006) at this site. The approximate location of the shallow bedrock (i.e., less than about 1.5 m bgs) was identified on the drawing in Figure 2 appended to this report. From the middle portion toward the outer extents of the property, the bedrock became gradually deeper and reached to its deepest depth at the eastern extend of the property.

The primary concern for this site is the presence of relatively deep silty clay deposit, especially in the Northern and Eastern extends of the property close to boring location of BH1 to BH4. Based on the field vane shear test results and measured moisture contents, the unweathered silty clay deposit founded below the ground water table exhibits an average undrained shear strength value of 36 kPa and average moisture content of up to 80%. The silty clay deposit is relatively consistent across the site in terms of its shear strength and physical properties. The measured properties indicated that the silty clay deposit is relatively weak and compressible and has a limited capacity to support the combined loads from the shallow foundations and grade raise fills.

It is our understanding that the proposed development will consist of several high-rise (e.g., ten-story) buildings, commercial low-rise buildings, local roads, access lanes and parking areas.

Based on the subsurface soil conditions established at this site and type of the structures in the proposed development, several foundation options including conventional shallow foundations and deep foundations are considered feasible to support the loads and limit the potential post-construction settlements at this site, as summarized below:

- High-rise buildings can be supported by using shallow foundation footings (or raft foundations) founded over the sound bedrock surface; or by using deep foundation such as cast-in-place concrete caissons founded over the sound bedrock surface, or alternatively socketed into the bedrock, if required. There would be no grade raise restriction for buildings supported by deep foundations.
- Low-rise buildings can be supported by means of of shallow conventional foundations founded over the dense glacial till or sound bedrock surface with grade raise restriction of up to 2.5 m for glacial till subgrade and no grade raise restriction for the bedrock.
- If the low-rise buildings are to be located in the areas where relatively deep silty clay deposit is encountered (i.e., Northern and Southern portions of the site), then the consideration may be given to Geopier ground improvement system. Alternatively, a combination of shallow foundation footings founded over stiff silty clay crust and lightweight fills may be used to limit the post-constriction settlements.
- Low-rise buildings to be located in the Eastern portion of the site, where deep silty clay deposit was encountered can be supported by using a combination of shallow foundation footings founded over stiff silty clay crust and lightweight fills. Deep foundations such as steel pipe piles driven to refusal can also be considered as an alternative option.

The above proposed foundation types are further discussed in the following sections.

5.2 Shallow Foundation

Shallow conventional footings founded over the undisturbed stiff silty clay crust may be designed using a preliminary maximum allowable bearing pressure of **75 kPa** for serviceability limit state **(SLS)** and **110 kPa** for ultimate limit state **(ULS)** factored bearing resistance. The factored ULS value includes the geotechnical resistance factor of 0.5. This bearing capacity assumes no grade raise fill (or grade raise by using lightweight fills) and a maximum allowable founding depth of 0.9 m below existing grade. This bearing capacity also allows for a maximum strip footing width of 0.9 m and a maximum pad footing width of 1.8 m on any side.

A preliminary maximum allowable bearing pressure of **150 kPa** for serviceability limit state **(SLS)** and **225 kPa** for ultimate limit state **(ULS)** factored bearing resistance may be used for designing of shallow conventional footings founded over the undisturbed glacial till. This bearing capacity limits the allowable grade raise to 2.5 m. The factored ULS value includes the geotechnical resistance factor of 0.5. This bearing capacity allows for a strip footing of width maximum 2.0 m and a pad footing of width maximum 3.5 m on any side.

Any topsoil/organic material and incompetent native soil should be removed from any buildings footprint down to the stable native subgrade comprised of silty clay or glacial till.

Prior to pouring footing concrete, the subgrade should be inspected and approved by a geotechnical engineer or a representative of the geotechnical engineer. In-situ testing (e.g., dynamic cone penetration or field vane shear test) may be required to check the stability of the footing 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.

If the shallow foundation footings or raft foundations are to be founded over sound bedrock, a preliminary maximum allowable bearing pressure of **500 kPa** for serviceability limit state **(SLS)** and **750 kPa** for ultimate limit state **(ULS)** factored bearing resistance may be used. For footings set directly over sound bedrock, there are no restrictions for grade raise, and maximum footings widths. Prior to the placement of any concrete for the footings, it should be ensured no loose or fractured debris and/or weathered rock is present at the location of the proposed footings. Unshrinkable concrete of compressive strength 1.0 MPa shall be used to fill the voids resulted from sub-excavation at the footing area.

Furthermore, if footings are to be founded on different soil, partly on sound bedrock and glacial till, there will be differential settlement. A structural engineer should consider adequate precautions in structural design to compensate this differential settlement so as to avoid any adverse effect in the superstructure of the proposed building.

Prior to pouring footings concrete, the bedrock should be inspected and approved by a qualified geotechnical engineer or a representative of geotechnical engineer.

5.3 Geopier System Ground Improvements

The shallow foundations could be supported on conventional strip or spread footings resting on the existing silty clay material reinforced with Geopier System, operated/managed by Geosolv Design/Build Inc. Geopier ground improvement system is installed using the Armorpact System. The Armorpact method would be able to provide increased bearing capacity and settlement control for the footings, walls and floor slabs of the proposed structures.

The Geopier Armorpact elements are installed by driving a 200 – 600 mm diameter Armorpact sleeve to the design depth (up to 10 m deep) using a specially designed hollow driving mandrel. Once the design depth is reached the mandrel is raised and lowered to deliver and compact thin lifts of aggregate using crowd pressure and high frequency vertical impact energy. A specialized tamper head is used at the bottom of the mandrel to densify the aggregate vertically and increase the lateral stress in the soil matrix. The construction process results in a reinforced soil profile with less compressibility and higher shear strength than the existing soil. Design of the Geopier elements are typically performed as a design-build process by Geosolv Design/Build Inc. Further details on Geopier ground improvement could be provided upon request.

5.4 Deep Foundation (Driven Steel Piles)

The buildings of the proposed development 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 of steel piles are used for deep foundation of high bearing capacity. These are as follows:

i. Steel H piles; and

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

Steel H-pile or closed ended, concrete filled steel pipe 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 can also be considered if driven to practical refusal. For design example, a concrete filled steel pipe pile of 245 mm diameter with 12.5 mm pipe wall thickness can be designed for Service Limit State **(SLS) 1150 kN** and Ultimate Limit State **(ULS) 1380 kN**. This assumes that the steel has a minimum yield strength of 350 MPa and 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.

It should be noted that driven steel piles must be capable of providing required lateral and bending resistances if the piles are subjected to such loadings.

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. 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 project specifications 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. 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.

5.5 Deep Foundation (Concrete Caissons)

Consideration could also be given on supporting the multi-storey buildings on drilled pier/reinforced concrete caissons set below fractured or any highly weathered bedrock overlying relatively sound bedrock.

It should be noted that the caissons must have a depth/diameter ratio of equal to or greater than 3. Socketed piers/drilled caisson should have a socket length to diameter ratio at least 1.5. If the caissons are installed through the silty clay or glacial till deposit, temporary liners will be required to seal and to prevent silty clay soil from caving and thus minimize the possible formation of voids and to help control water seepage into the holes. During auguring, there is possibility to encounter cobbles and boulders in glacial till deposit and allowances should be made to break the boulder where necessary. The bottom of the hole should be properly cleaned, free of water and loose or remolded materials prior to pouring concrete. The minimum caisson size to allow access for cleaning and inspection is 600 mm.

It is recommended to conduct unconfined compressive strength test on select representative rock cores obtained from the bedrock formation encountered at this site to determine the allowable toe and skin resistances for the design of the concrete caissons.

5.6 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.

5.7 Bedrock Excavation

In view of the shallow bedrock at the middle portion of this site and considering the founding depths of the structures for the proposed development, it is assumed that some excavation of bedrock will be required. It is anticipated that some minor bedrock removal may be possible with the use of heavy excavation equipment, but that removal of most of the bedrock could be facilitated by means of a hoe ramming operation. Both horizontal and vertical overbreak of the bedrock excavation face/bottom can be expected due to the hoe ramming operation. If control of potential bedrock overbreak is required, line drilling at the proposed excavation face is recommended. The smaller the distance between the drill holes, the fewer overbreaks is expected. It is generally considered that the drilling at 150 mm horizontal spacing to the full depth of the excavation should control overbreak to an acceptable level. Considering the proximity of the existing structures adjacent to the site and the potential for vibration during excavating and removal of the bedrock, it is required that monitoring of the hoe ramming be carried out throughout the operation to ensure that the vibration limit is not exceeded. As outlined in **OPSS 120, Table 5** below summarizes the following vibration limits for the nearest existing structures. In addition, a pre-excavation condition survey of nearby structures should be carried out.

Table 5: Vibration Frequency and Limit

Frequency of Vibration (HZ)	Vibration Limit, PPV (Peak Particle Velocity) mm/sec
≤ 40	20
> 40	50

5.8 Grade Raise and Settlement

Based on the finished ground level of the existing developments that border the Western and Northern extends of the property, it is anticipated that a grade raise fill of up to 2.5 m would be required at this site. As noted before, a combination of grade raise fill and conventional shallow foundations founded over the deep compressible silty clay deposit would not be permitted, unless lightweight fill is being used. . A preliminary grade raise fill of up to 2.5 m can be considered feasible at this site provided that the buildings loads will be supported by deep foundations or the ground is being improved by the ground improvement technique, as recommended above.

It should be noted that the above recommended grade raise is a preliminary estimation and must be confirmed by conducting consolidation tests on select representative silty clay soil samples and subsequent settlement analysis.

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 in this report, will be less than 25 mm. The differential settlement between adjacent column footings is anticipated to be 15 mm or less.

5.9 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 **C** for the footings placed over glacial till or bedrock surface.

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.

Due to the variations of the silty clay thickness across the site, a specific seismic testing such as shear wave velocity test or approved equivalent test is recommended to classify the Seismic Site Response for the buildings will not be founded over the bedrock.

5.10 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 tests on the split spoon samples collected exhibit the ratio of water content to liquid limit ranging from 0.67 to 1.30, and I_p ranging between 36 and 51. Based on these test results, the silty clay deposit is not susceptible to liquefaction or cyclic mobility. However, there is still a possibility to encounter localized shallow groundwater, which is mostly perched water, and will be mitigated through appropriate sump pumping.

5.11 Frost Protection

All exterior shallow 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.

5.12 Foundation Drainage

A conventional, perforated corrugated polyethylene drainage pipe (100 mm minimum), pre-wrapped with geotextile knitted sock conforming to **OPSS 1840** should be embedded in a 300 mm layer of 19 mm clear crushed stone and set adjacent to the perimeter footings. The drainage pipe should be connected positively to a suitable outlet, such as a sump pit or storm sewer. 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.

5.13 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.

5.14 Slab-on-grade Construction

Conventional concrete slab-on-grade is considered feasible for the structures in the proposed development, provided certain precautions are undertaken. For predictable performance for the proposed slab-on-grade, it should rest over undisturbed competent native stiff 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 stiff 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 II 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.**

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage of the concrete. The saw cut depth should be about one quarter of the thickness of the slab. In order to minimize and control cracking, the floor slab should be provided with wire mesh reinforcement and construction or control joints. The construction or control joints should be spaced equal distance in both directions and should not exceed 4.5m. The wire mesh reinforcement should 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.

5.15 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 detail 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	3.53
Granular B Type I	20.0	31	0.49	0.32	3.12
Granular B Type II	23.0	32	0.47	0.31	3.25
Silty Clay	17.5	28	0.53	0.36	2.77
Glacial Till	21.5	40	0.36	0.22	4.59

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 this site is 0.31g 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)

5.16 Corrosion Potential and Cement Type

Two (2) soil samples labelled BH2 (SS3) and BH5 (SS1) were submitted for soil sulphate (SO_4) analysis. The laboratory analyses revealed a maximum measured sulphate concentration of 0.0046% (46 µg/g) and 0.0027% (27 µg/g) respectively in the soil samples. 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 results from soil samples were 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 in BH2 (SS3) and BH5

(SS1) was measured to be 3830 and 8270 ohm-cm respectively, indicating that the steel structures with exposed surface in contact with the silty clay soil encountered at the site can be subjected to a moderate to very low degree of 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.**

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation

It is anticipated that the depth of excavation for the shallow footings will not be extended below 2.0 m bgs. Most of the excavation being carried out will be through silty clay or in glacial till or bedrock in the middle portion of the site. 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 silty clay expected to be excavated 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.

For the excavations in the bedrock (dolomite or limestone), a vertical cut in the sound bedrock (free of fractured zone) is anticipated to be self-supporting for height up to 2.5 m.

In the event that the aforementioned slopes are not possible to achieve due to space restrictions or deeper excavation is required, 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 5.12** 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

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 development will be founded over silty clay, glacial till or bedrock. 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 II 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.

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 parking areas, access lanes and local public roads will mostly consist of silty clay or engineered fills. Also, it is anticipated that the subgrade will consist of glacial till or bedrock where the overburden is minimal within the middle portion of the site. The following recommendations are provided based on the silty clay subgrade mostly encountered at this site. It should be noted that the thickness of the subbase (and base) materials may be reduced to a lower value where the pavement structures are to be founded over the dense glacial till or bedrock surface.

The construction of paved areas over the undisturbed silty clay should be done once all debris, organic material, or otherwise deleterious material are removed from the subgrade area. Alternatively, if the paved areas are to be founded over the engineered fill, consideration must be given to properly prepare the subgrade conditions as outlined in the next section. Furthermore, the subgrade must be compacted using a suitable heavy duty compacting equipment and approved by a geotechnical engineer prior to placing any granular base material.

The following **Table 7** presents the recommended pavement structure to be constructed over a stable subgrade along the proposed parking areas, access lanes and local public roads as part of this development. The pavement structure recommended for the local public roads are calculated based on the subgrade California Bearing Ratio (CBR) of 4.5 for silty clay subgrade and recommended Granular Base Equivalency (GBE) thickness of 700 mm as per Ontario Ministry of Transportation's Pavement Design and Rehabilitation Manual (2013) for the assumed Equivalent Single Axle Loads of $0.2x10^6$ per year (ESALs/yr). These assumptions and recommendations must be further verified once more details with regard to the pavement construction are available.

Table 7: Recommended Pavement Structure

Course	Material	Thickness (mm)		
		Light Duty Parking Area (mm)	Heavy Duty Parking Area, Fire Routes and Access Lanes (mm)	Local Public Roads (mm)
GBE		450	630	705
Surface	HL3 A/C	50	40	40
Binder	HL8 A/C	-	50	50
Base course	Granular A	150	150	150
Sub base	Granular B Type II	300	450	500
Total:		500	690	740

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 local public roads, 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.

LICENSED PROFESSIONAL ENGINEER
A.Ghirian
A. GHIRIAN
100501824
27.04.2018
PROVINCE OF ONTARIO

Brad Johnson, B.Sc. Eng
Geotechnical Services

Alireza Ghirian, P. Eng.
Senior Geotechnical Engineer

APPENDIX A

Site and Borehole Location Plan

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

PROJECT
GEOTECHNICAL INVESTIGATION
SMARTREIT (ORLEANS II) INC.
MER BLEUE ROAD
ORLEANS, ONTARIO

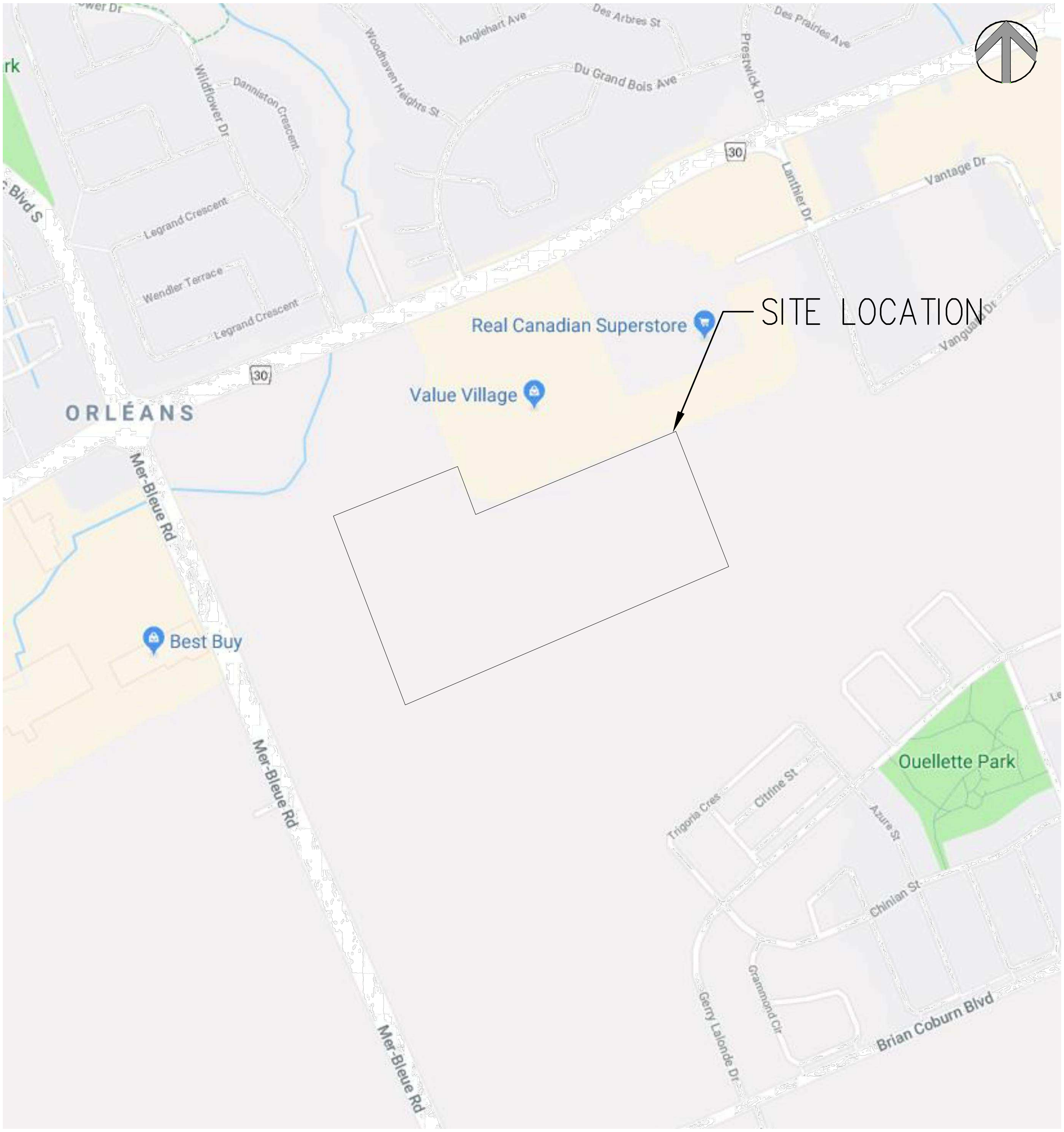
DRAWING TITLE
SITE LOCATION
SOURCE: Map Data 2018 Google

CLIENT
SMART CENTRES

DATE
APRIL 10, 2018

PROJECT
180015

FIGURE 1



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

PROJECT
GEOTECHNICAL INVESTIGATION
SMARTREIT (ORLEANS II) INC.
MER BLEUE ROAD
ORLEANS, ONTARIO

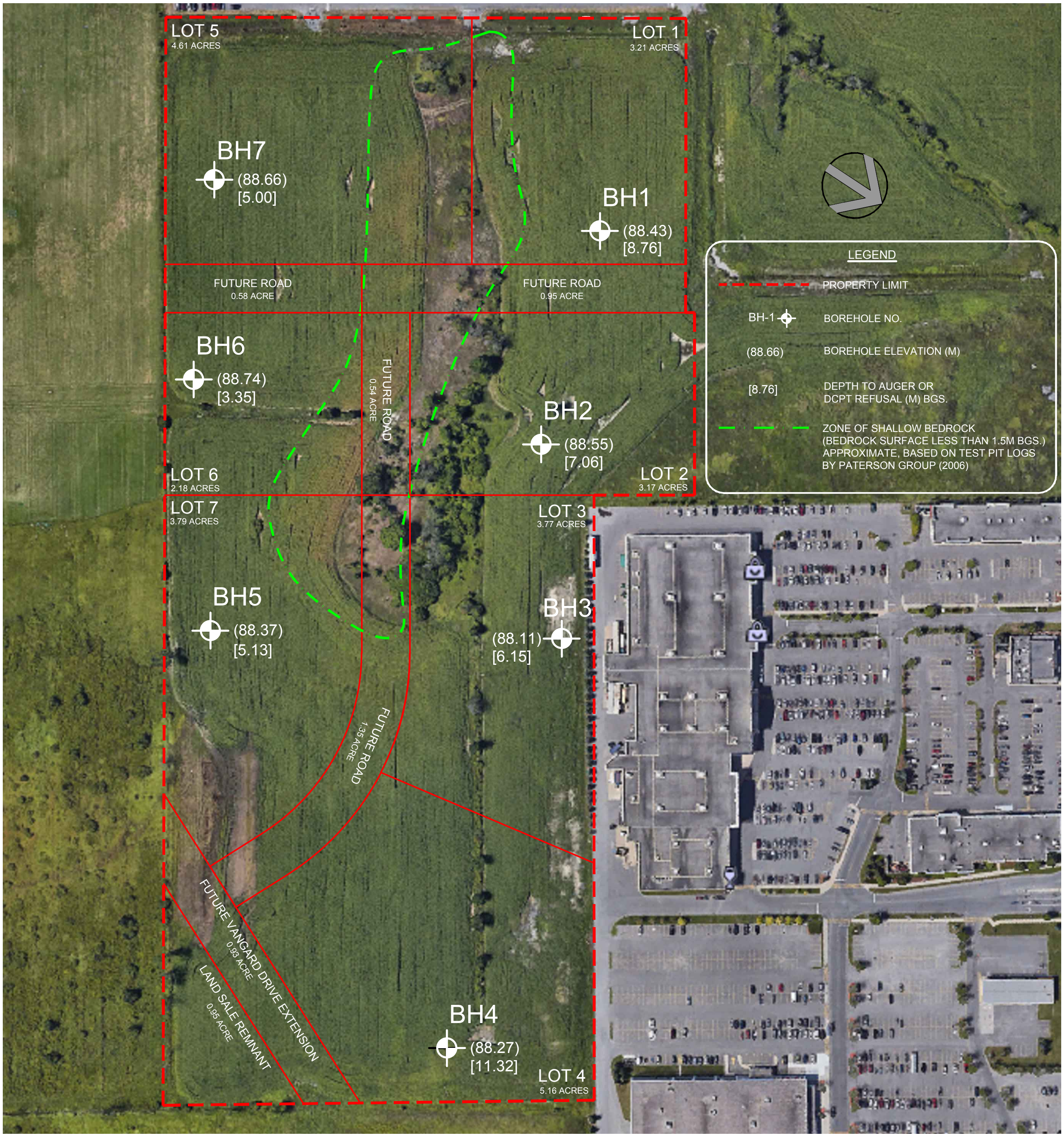
DRAWING TITLE
BOREHOLE LOCATION
SOURCE: Imagery 2018 Google, DigitalGlobe Map Data

CLIENT
SMART CENTRES

DATE
APRIL 10, 2018

PROJECT
180015

FIGURE 2



APPENDIX B

Borehole Logs

Borehole Log: BH1

Project No.: 180015

Project: SmartREIT (Orleans II) Inc.

Client: Smart Centres

Location: Mer Bleue Road, Orleans ON

Date: April 6, 2018

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 55

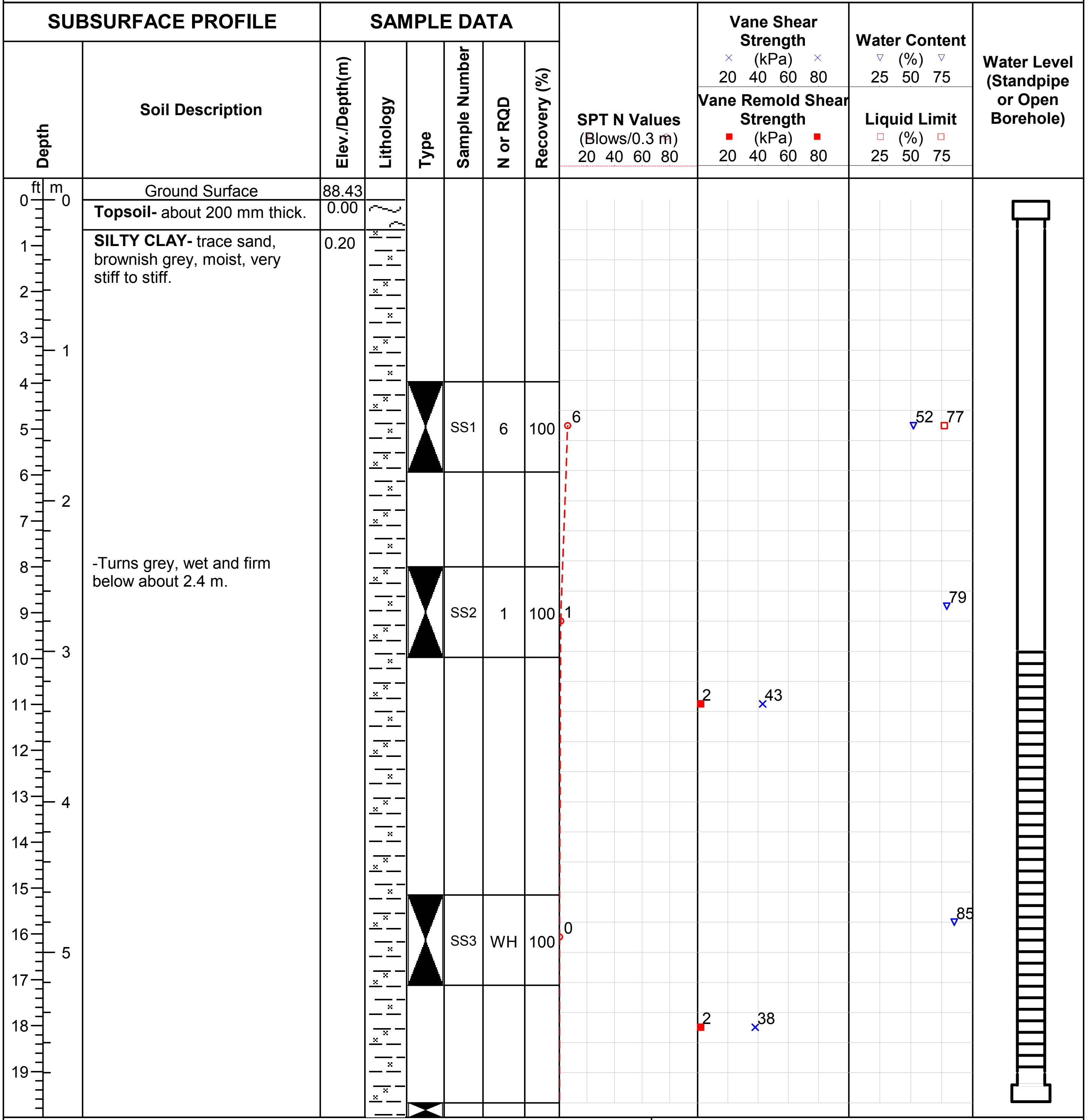
Drilling Method: HSA

SUBSURFACE PROFILE

Soil Description	Elev./Depth(m)
Ground Surface	88.43
Topsoil- about 200 mm thick.	0.00
SILTY CLAY- trace sand, brownish grey, moist, very stiff to stiff. -Turns grey, wet and firm below about 2.4 m.	0.20

SAMPLE DATA

Type	Sample Number	N or RQD	Recovery (%)
⧗	SS1	6	100
⧗	SS2	1	100
⧗	SS3	WH	100



Easting: 460899

Northing: 5033559

Site Datum: T/G of Existing Sanitary #9 - South of Existing LCBO Building (87.86 m).

Groundsurface Elevation: 88.43

Top of Riser Elev.: 89.48

Hole Diameter: 200 mm

Borehole Log: BH1

Project No.: 180015

Project: SmartREIT (Orleans II) Inc.

Client: Smart Centres

Location: Mer Bleue Road, Orleans ON

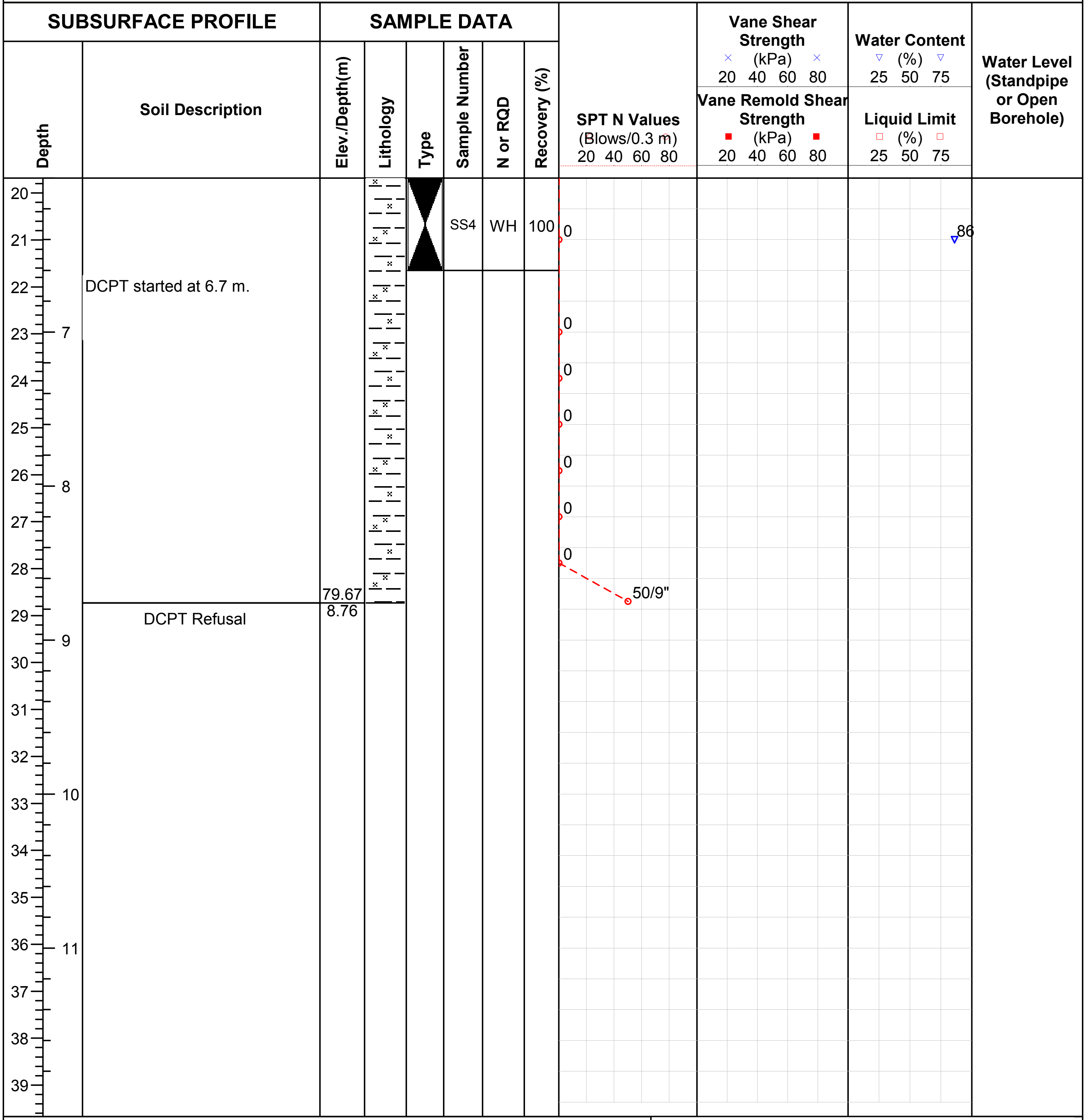
Date: April 6, 2018

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 55

Drilling Method: HSA



Easting: 460899

Northing: 5033559

Site Datum: T/G of Existing Sanitary #9 - South of Existing LCBO Building (87.86 m).

Groundsurface Elevation: 88.43

Top of Riser Elev.: 89.48

Hole Diameter: 200 mm

NOTES:
Water level was measured at 5.9 m bgs, however the result was not representative to the moisture contents determined in the laboratory.

Borehole Log: BH2

Project No.: 180015

Project: SmartREIT (Orleans II) Inc.

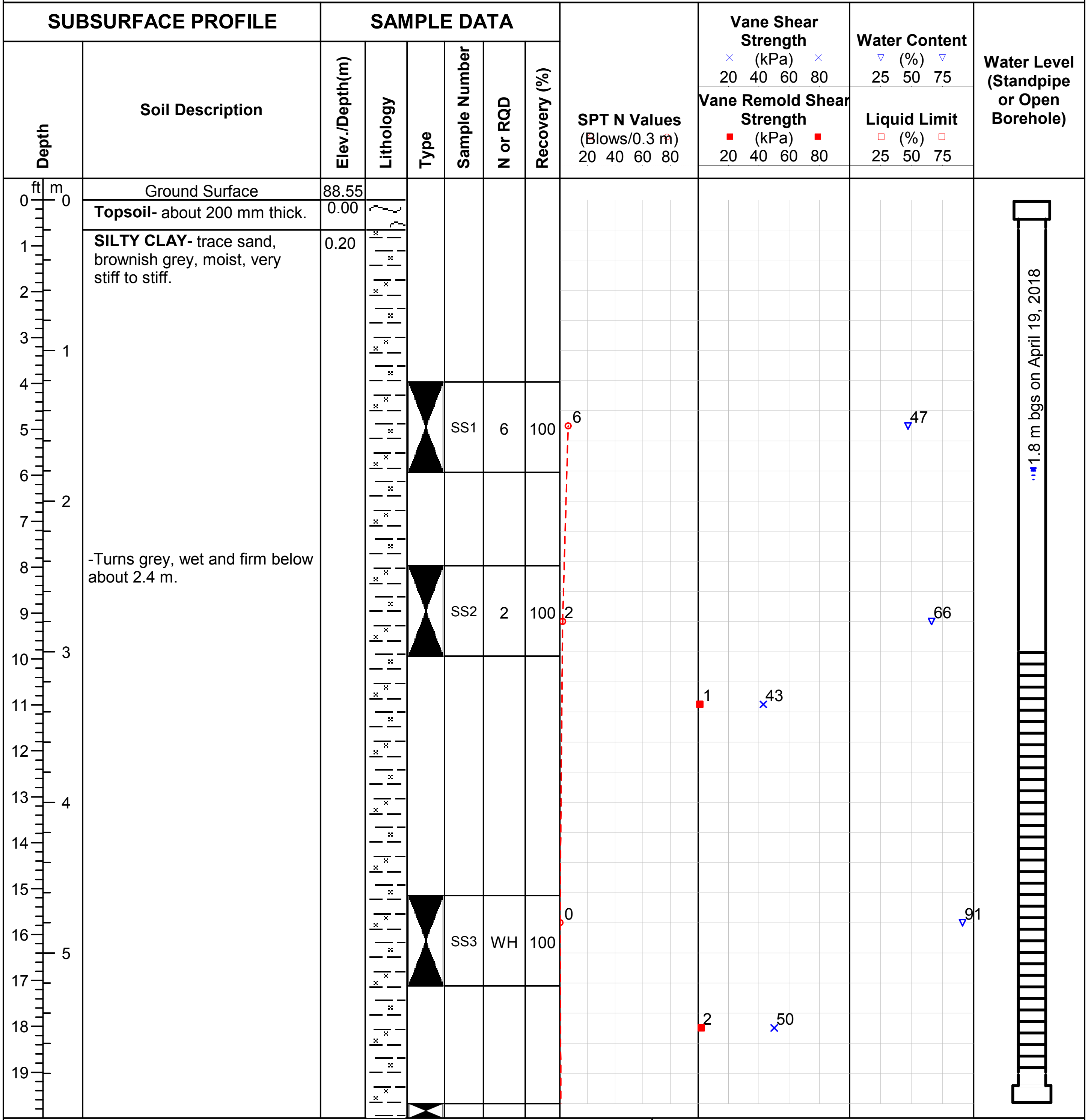
Client: Smart Centres

Location: Mer Bleue Road, Orleans ON

Date: April 6, 2018

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd. **Drilling Equipment:** Track Mount CME 55 **Drilling Method:** HSA



Soil Description	Elev./Depth(m)	Sample Number	N or RQD	Recovery (%)
Ground Surface	88.55			
Topsoil- about 200 mm thick.	0.00			
SILTY CLAY- trace sand, brownish grey, moist, very stiff to stiff.	0.20	SS1	6	100
-Turns grey, wet and firm below about 2.4 m.		SS2	2	100
		SS3	WH	100

Easting: 461010 **Northing:** 5033570

Site Datum: T/G of Existing Sanitary #9 - South of Existing LCBO Building (87.86 m).

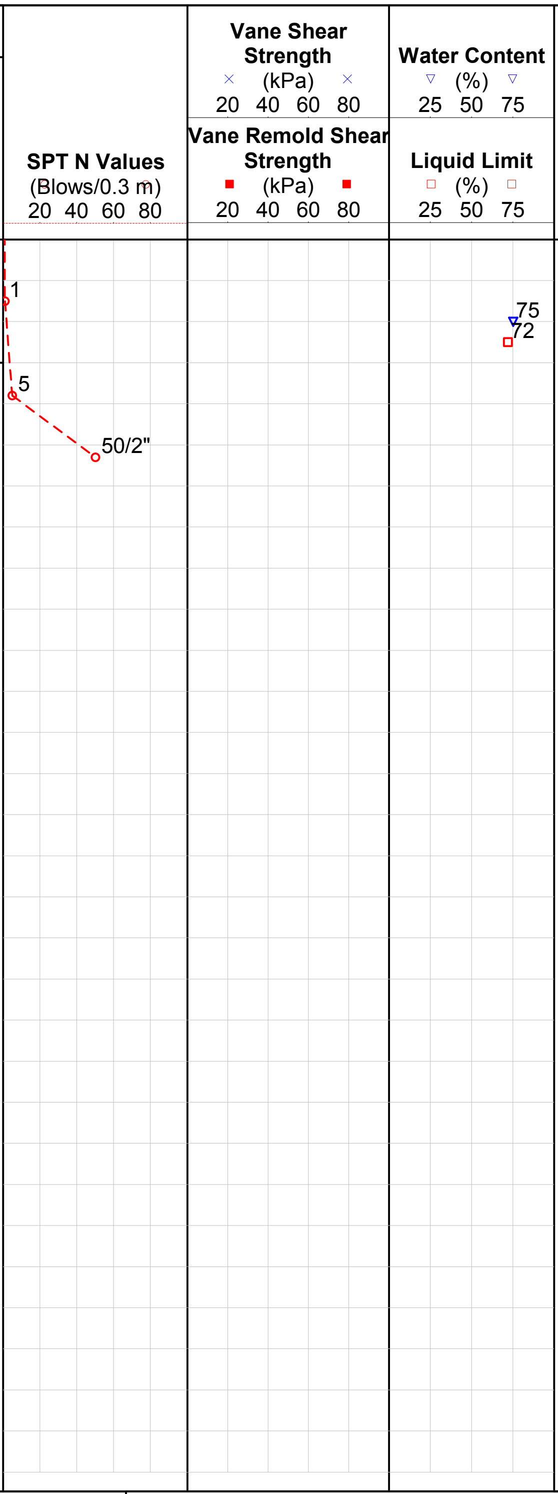
Groundsurface Elevation: 88.55 **Top of Riser Elev.:** 89.45

Hole Diameter: 200 mm

Borehole Log: BH2

Project No.: 180015
Project: SmartREIT (Orleans II) Inc.
Client: Smart Centres
Location: Mer Bleue Road, Orleans ON
Date: April 6, 2018
Field Personnel: BJ
Driller: George Downing Estate Drilling Ltd.
Drilling Equipment: Track Mount CME 55
Drilling Method: HSA

SUBSURFACE PROFILE			SAMPLE DATA						Water Level (Standpipe or Open Borehole)
Depth (ft)	Depth (m)	Soil Description	Elev./Depth(m)	Lithology	Type	Sample Number	N or RQD	Recovery (%)	
20–22						SS4	1	100	
22		DCPT started at 6.86 m.							
23	7	DCPT Refusal	81.49 7.06						
24–39	8, 9, 10, 11								



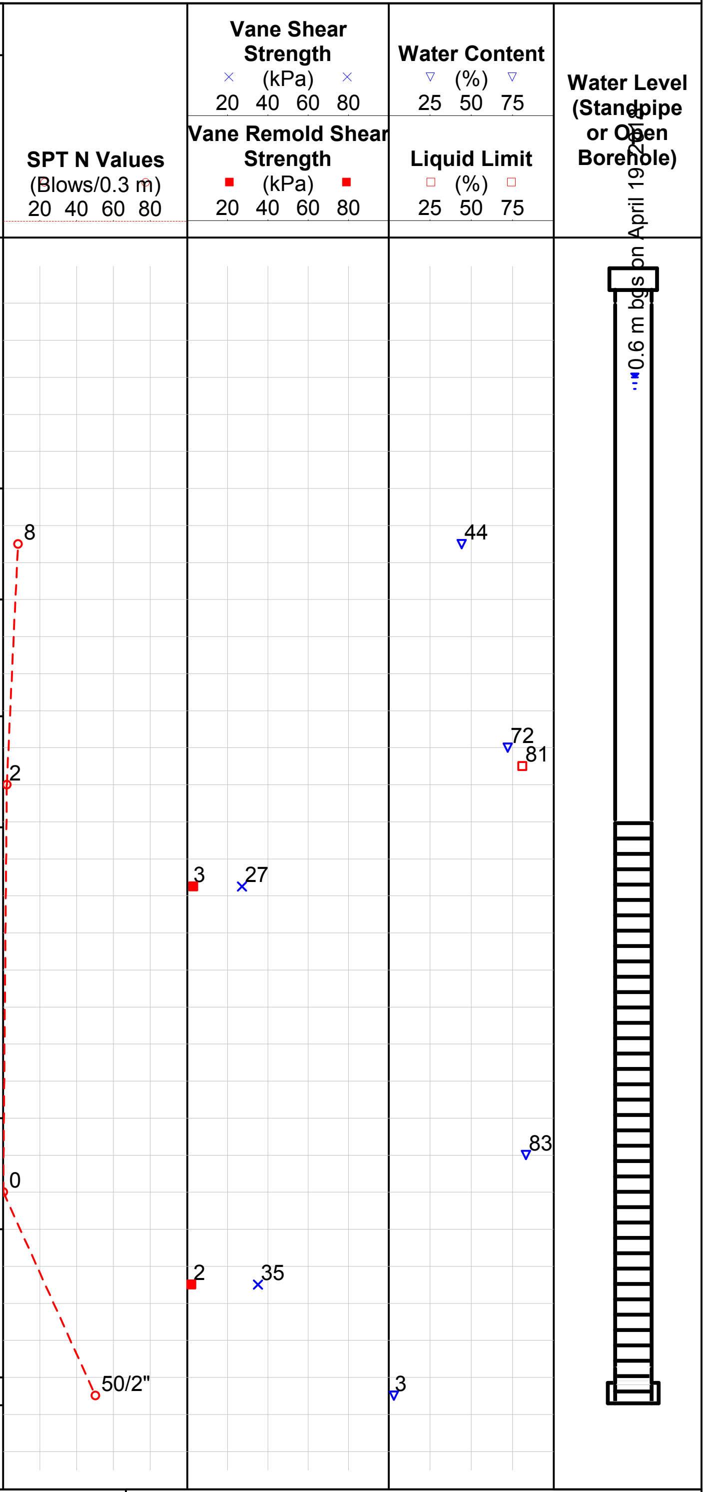
Easting: 461010
Northing: 5033570
Site Datum: T/G of Existing Sanitary #9 - South of Existing LCBO Building (87.86 m).
Groundsurface Elevation: 88.55
Top of Riser Elev.: 89.45
Hole Diameter: 200 mm

Borehole Log: BH3

Project No.: 180015
Project: SmartREIT (Orleans II) Inc.
Client: Smart Centres
Location: Mer Bleue Road, Orleans ON
Date: April 6, 2018
Field Personnel: BJ
Driller: George Downing Estate Drilling Ltd.
Drilling Equipment: Track Mount CME 55
Drilling Method: HSA

SUBSURFACE PROFILE / SAMPLE DATA

Soil Description	Elev./Depth(m)	Type	Sample Number	N or RQD	Recovery (%)
Ground Surface	88.11				
Topsoil- about 200 mm thick.	0.00				
SILTY CLAY- trace sand, brownish grey, moist, very stiff to stiff.	0.20				
		⧗	SS1	8	83
-Turns grey, wet, firm below about 2.4 m.		⧗	SS2	2	100
		⧗	SS3	WH	100
GLACIAL TILL- silty gravel with sand, wet, dense to very dense.	82.11 6.00	⧗	SS4	50+	100
Auger Refusal					



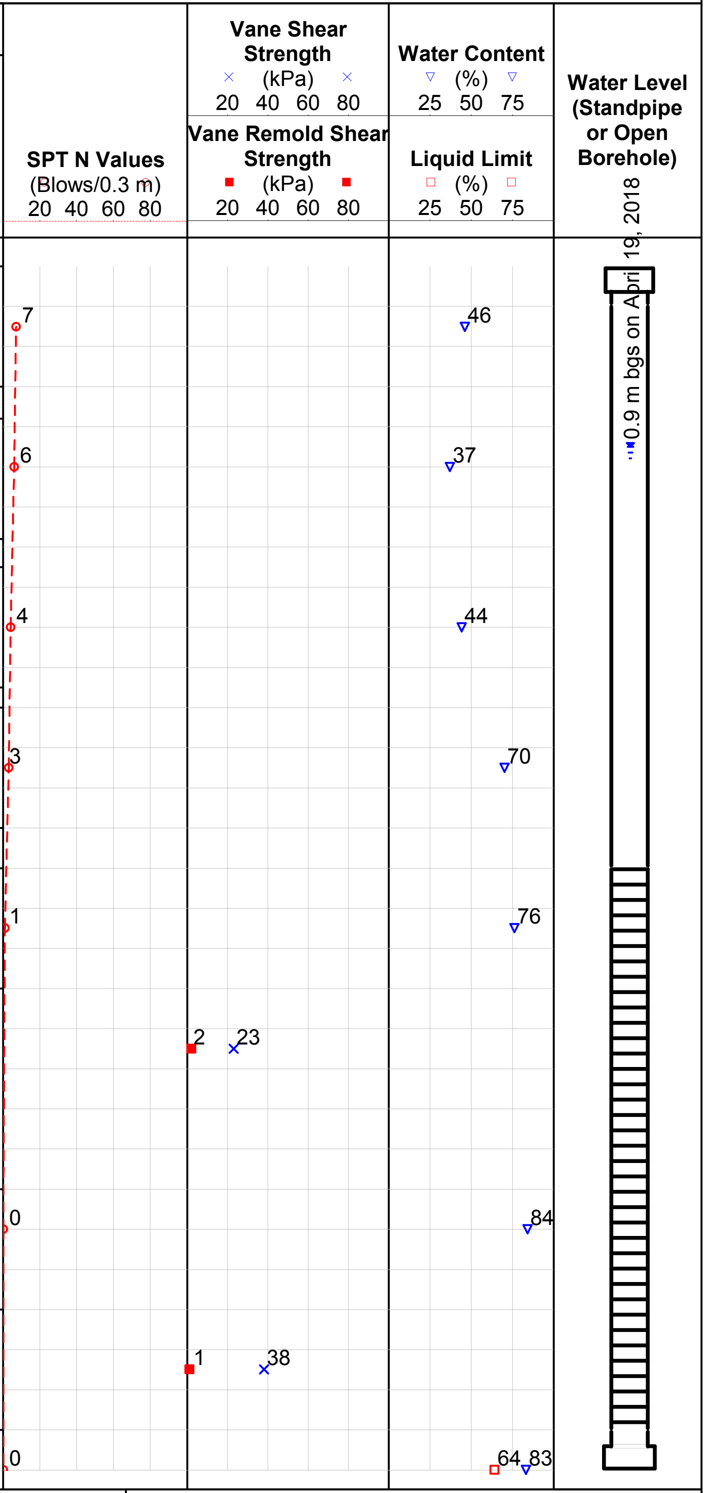
Depth scale: ft 0–21; m 0–6

Easting: 461098
Northing: 5033615
Site Datum: T/G of Existing Sanitary #9 - South of Existing LCBO Building (87.86 m).
Groundsurface Elevation: 88.11
Top of Riser Elev.: 89.21
Hole Diameter: 200 mm

Borehole Log: BH4

Project No.: 180015
Project: SmartREIT (Orleans II) Inc.
Client: Smart Centres
Location: Mer Bleue Road, Orleans ON
Date: April 5, 2018
Field Personnel: BJ
Driller: George Downing Estate Drilling Ltd.
Drilling Equipment: Track Mount CME 55
Drilling Method: HSA

SUBSURFACE PROFILE			SAMPLE DATA				
Depth	Soil Description	Elev./Depth(m)	Lithology	Type	Sample Number	N or RQD	Recovery (%)
ft / m	Ground Surface	88.27					
	Topsoil- about 200 mm thick.	0.00					
	SILTY CLAY- trace sand, brownish grey, moist, very stiff to stiff.	0.20			SS1	7	0
					SS2	6	75
					SS3	4	100
					SS4	3	100
	-Turns grey, wet, firm to soft below about 2.6 m.						
					SS5	1	100
					SS6	WH	100
					SS7	WH	



Easting: 461316
Northing: 5033636
Site Datum: T/G of Existing Sanitary #9 - South of Existing LCBO Building (87.86 m).
Groundsurface Elevation: 88.27
Top of Riser Elev.: 89.27
Hole Diameter: 200 mm

Borehole Log: BH4

Project No.: 180015

Project: SmartREIT (Orleans II) Inc.

Client: Smart Centres

Location: Mer Bleue Road, Orleans ON

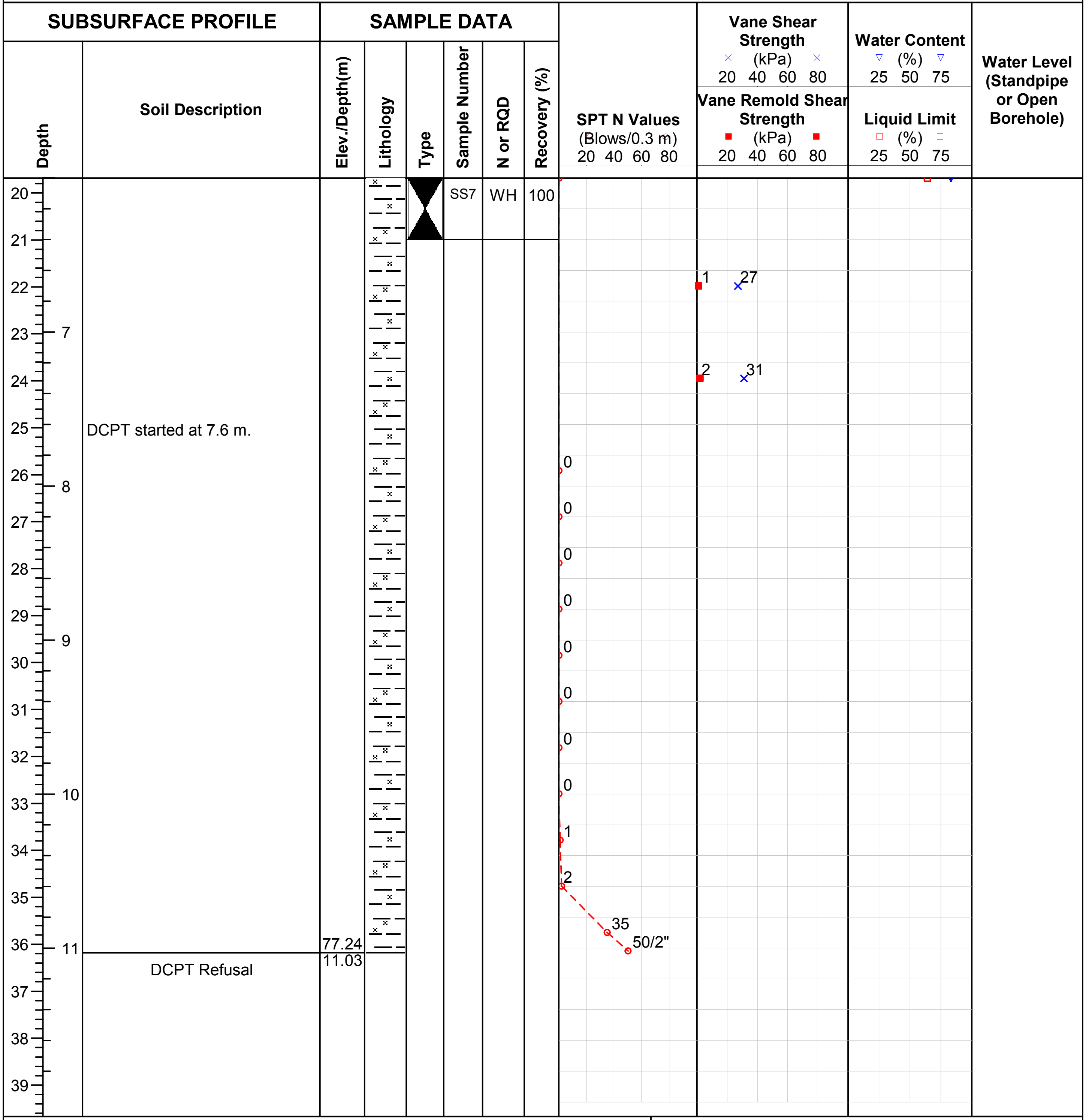
Date: April 5, 2018

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 55

Drilling Method: HSA



Easting: 461316

Northing: 5033636

Site Datum: T/G of Existing Sanitary #9 - South of Existing LCBO Building (87.86 m).

Groundsurface Elevation: 88.27

Top of Riser Elev.: 89.27

Hole Diameter: 200 mm

Borehole Log: BH5

Project No.: 180015

Project: SmartREIT (Orleans II) Inc.

Client: Smart Centres

Location: Mer Bleue Road, Orleans ON

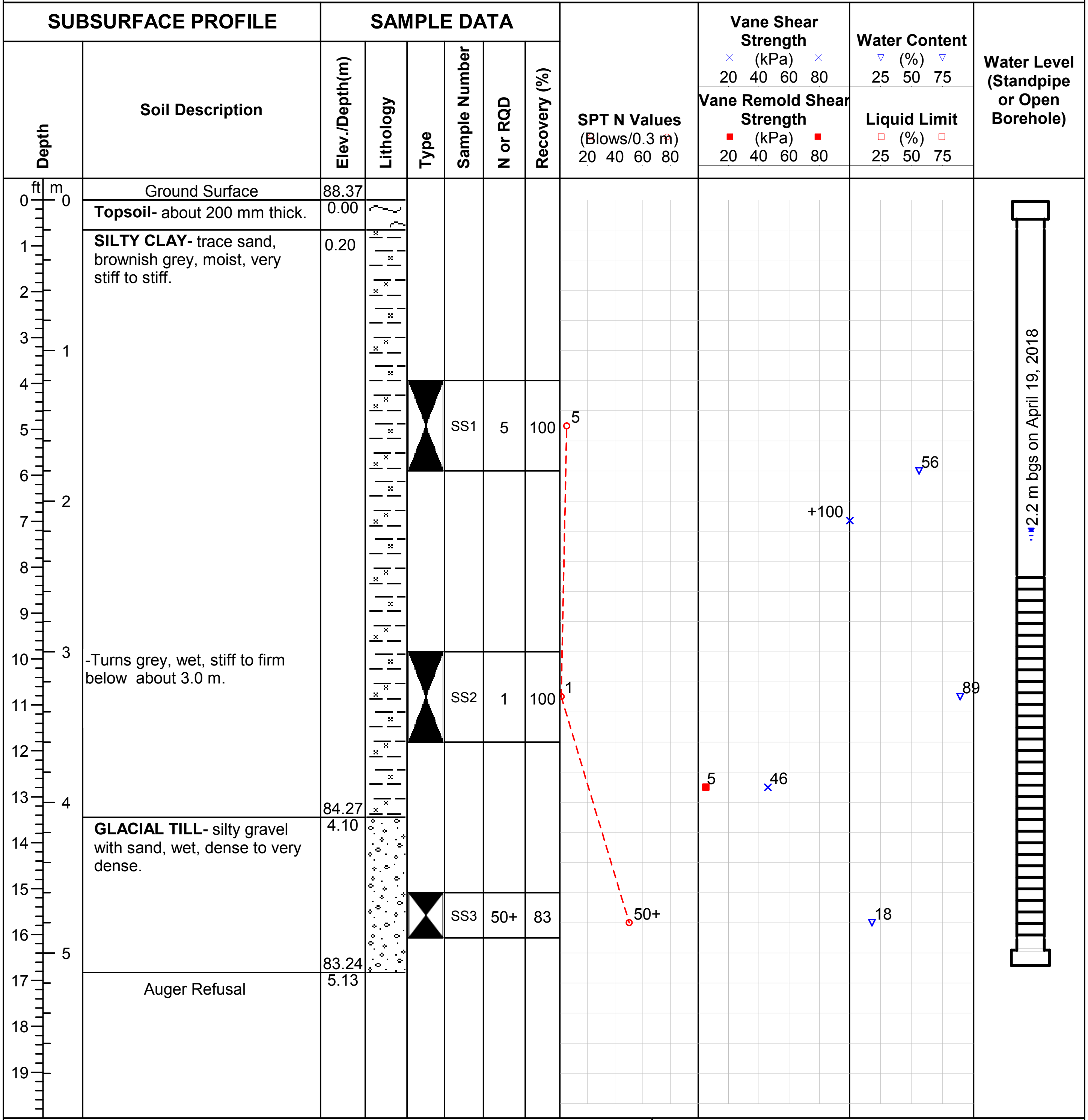
Date: April 5, 2018

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 55

Drilling Method: HSA



Elev./Depth(m)	Soil Description
88.37	Ground Surface
0.00	**Topsoil-** about 200 mm thick.
0.20	**SILTY CLAY-** trace sand, brownish grey, moist, very stiff to stiff. -Turns grey, wet, stiff to firm below about 3.0 m.
84.27 4.10	**GLACIAL TILL-** silty gravel with sand, wet, dense to very dense.
83.24 5.13	Auger Refusal

Sample Number	N or RQD	Recovery (%)
SS1	5	100
SS2	1	100
SS3	50+	83

Easting: 461169

Northing: 5033446

Site Datum: T/G of Existing Sanitary #9 - South of Existing LCBO Building (87.86 m).

Groundsurface Elevation: 88.37

Top of Riser Elev.: 89.47

Hole Diameter: 200 mm

Borehole Log: BH6

Project No.: 180015

Project: SmartREIT (Orleans II) Inc.

Client: Smart Centres

Location: Mer Bleue Road, Orleans ON

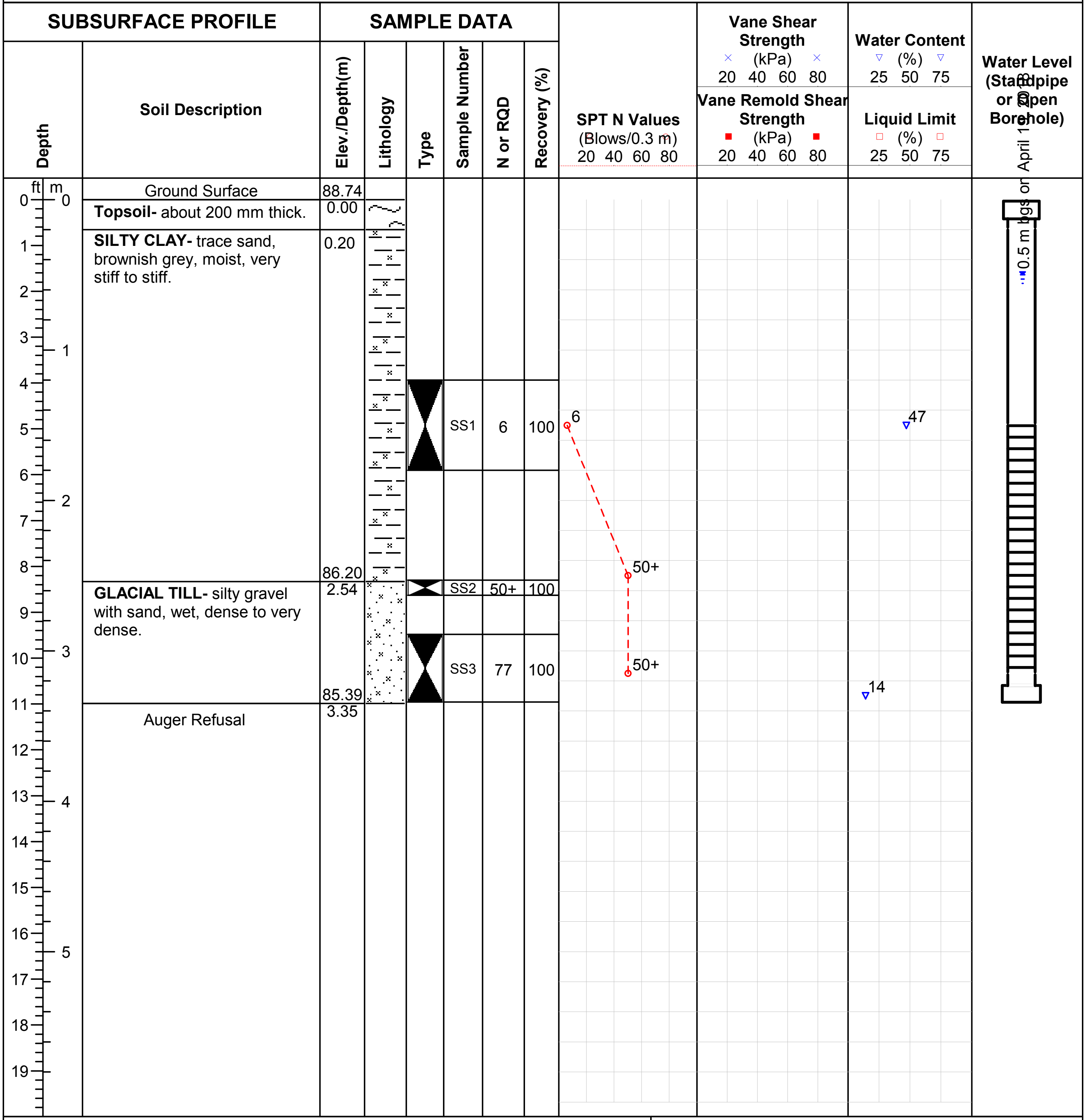
Date: April 5, 2018

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	88.74
Topsoil- about 200 mm thick.	0.00
SILTY CLAY- trace sand, brownish grey, moist, very stiff to stiff.	0.20
GLACIAL TILL- silty gravel with sand, wet, dense to very dense.	86.20 2.54
Auger Refusal	85.39 3.35

Sample Number	N or RQD	Recovery (%)
SS1	6	100
SS2	50+	100
SS3	77	100

Easting: 461042

Northing: 5033405

Site Datum: T/G of Existing Sanitary #9 - South of Existing LCBO Building (87.86 m).

Groundsurface Elevation: 88.74

Top of Riser Elev.: 89.54

Hole Diameter: 200 mm

Borehole Log: BH7

Project No.: 180015

Project: SmartREIT (Orleans II) Inc.

Client: Smart Centres

Location: Mer Bleue Road, Orleans ON

Date: April 5, 2018

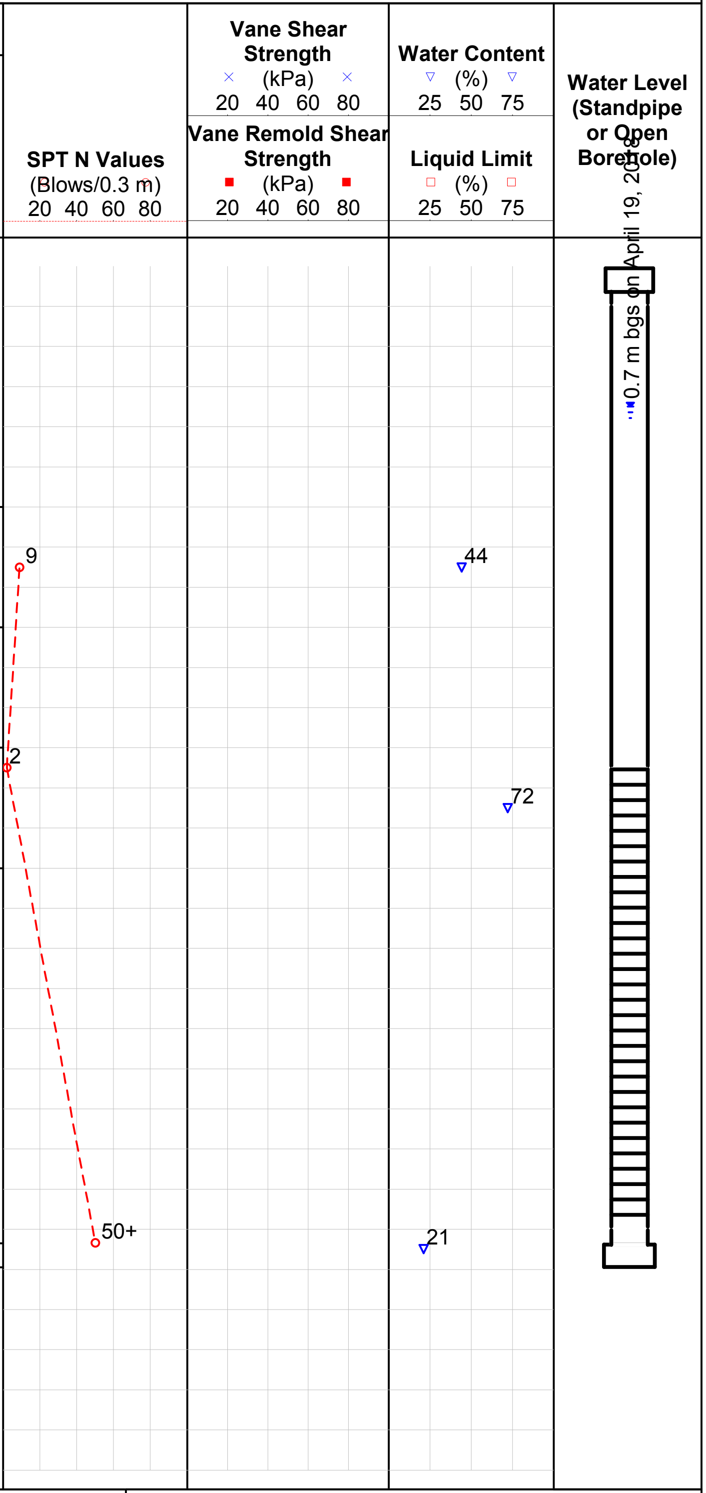
Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd.

Drilling Equipment: Track Mount CME 55

Drilling Method: HSA

SUBSURFACE PROFILE		SAMPLE DATA				
Depth (ft / m)	Soil Description	Elev./Depth(m)	Type	Sample Number	N or RQD	Recovery (%)
0 / 0	Ground Surface	88.66				
	Topsoil- about 200 mm thick.	0.00				
	SILTY CLAY- trace sand, brownish grey, moist, very stiff to stiff.	0.20				
4.5 / 1.4			⧗	SS1	9	100
8 / 2.4	-Turns grey, wet below about 2.4 m.		⧗	SS2	2	100
11 / 3.35	**GLACIAL TILL-** silty gravel with sand, wet, dense to very dense.	85.31 3.35				
16 / 4.9			⧗	SS3	53+	
16.4 / 5	Auger Refusal	83.66 5.00				



Easting: 460945

Northing: 5033369

Site Datum: T/G of Existing Sanitary #9 - South of Existing LCBO Building (87.86 m).

Groundsurface Elevation: 88.66

Top of Riser Elev.: 90.06

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

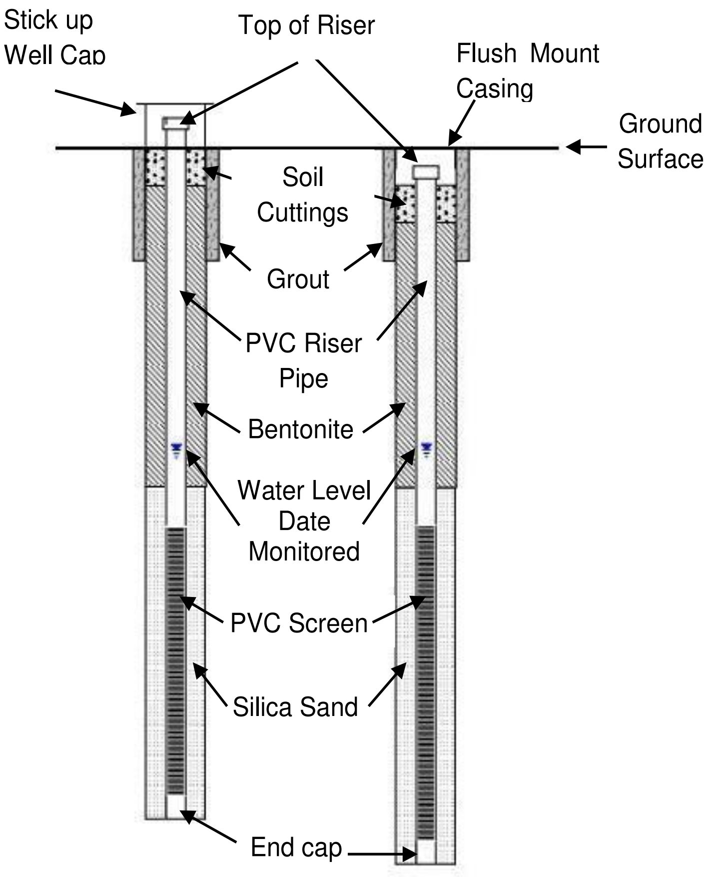
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given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

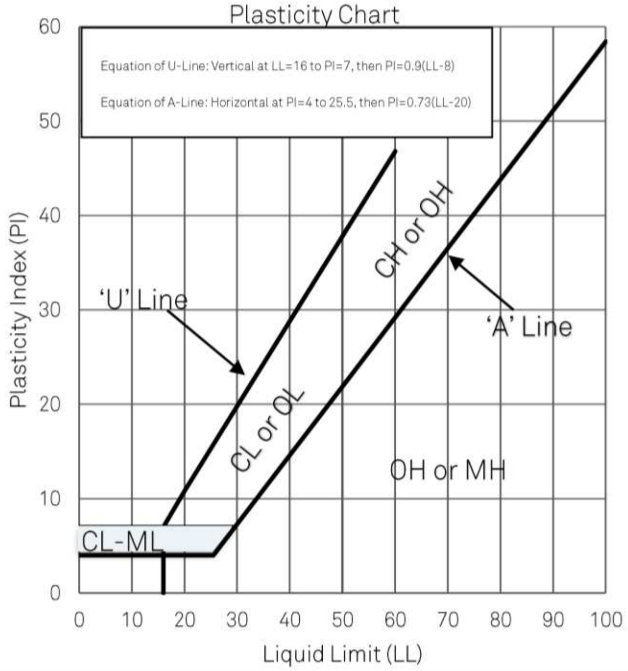
Strength classification of rock is presented below.

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 appropriate. 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

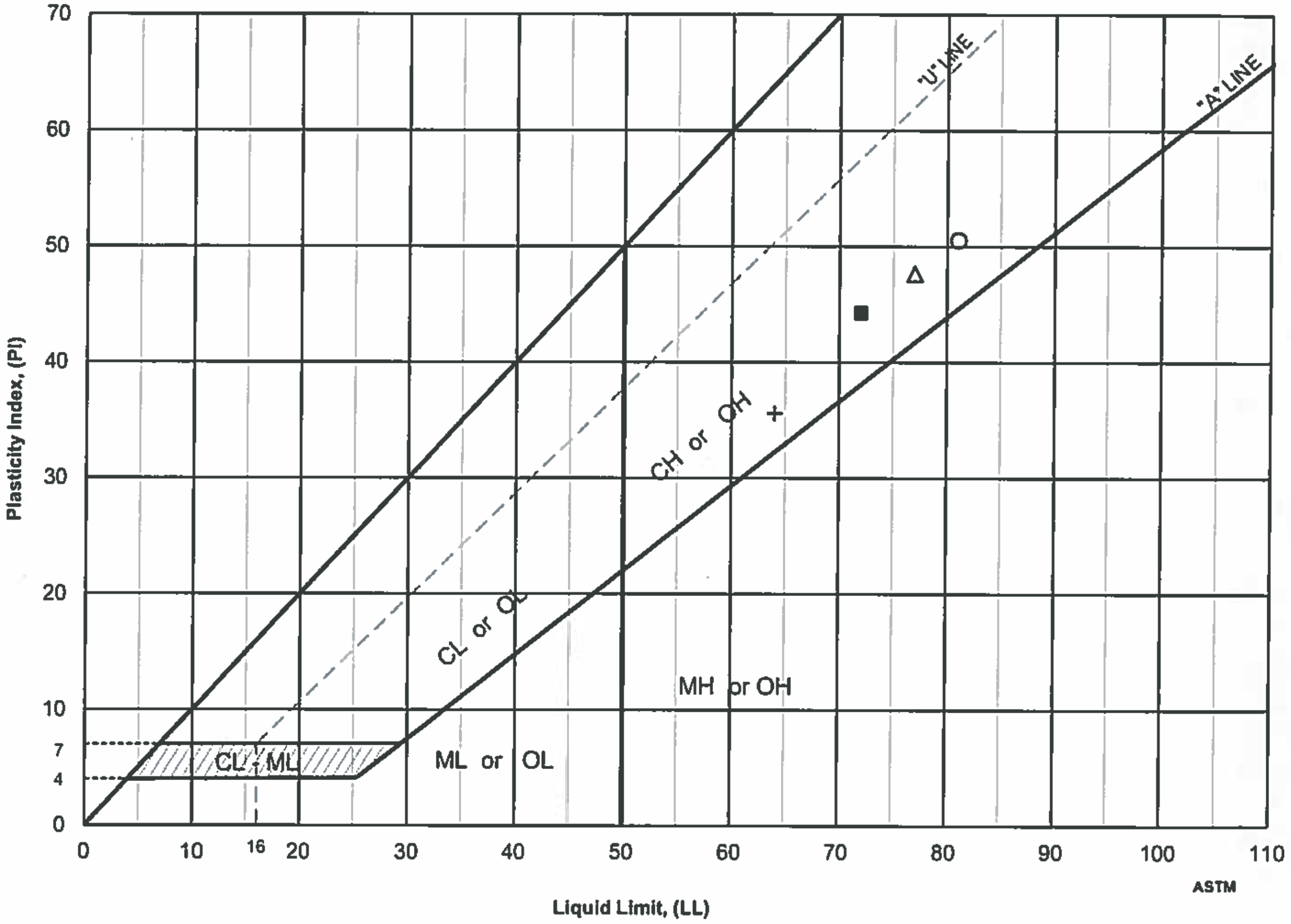
LRL Associates Ltd.

PLASTICITY INDEX

ASTM D 4318 / LS-703/704

Client:	SmartCentre	**File No.:**	180015
Project:	Geotechnical Investigation	**Report No.:**	1
Location:	Innes Road & Mer-Bleue Road, Orleans, ON.	**Date:**	April 6, 2018

Plasticity Chart



	Location	Sample	Depth, m	Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Activity Number	USCS
△	BH 1	SS-1	1.22 - 1.83	52	77	29	48	0.48	0.74	CH
■	BH 2	SS-4	6.10 - 6.71	75	72	28	44	1.07	n/d	CH
○	BH 3	SS-2	2.44 - 3.05	72	81	30	51	0.83	0.72	CH
+	BH 4	SS-7	5.79 - 6.40	83	64	28	36	1.54	0.56	CH

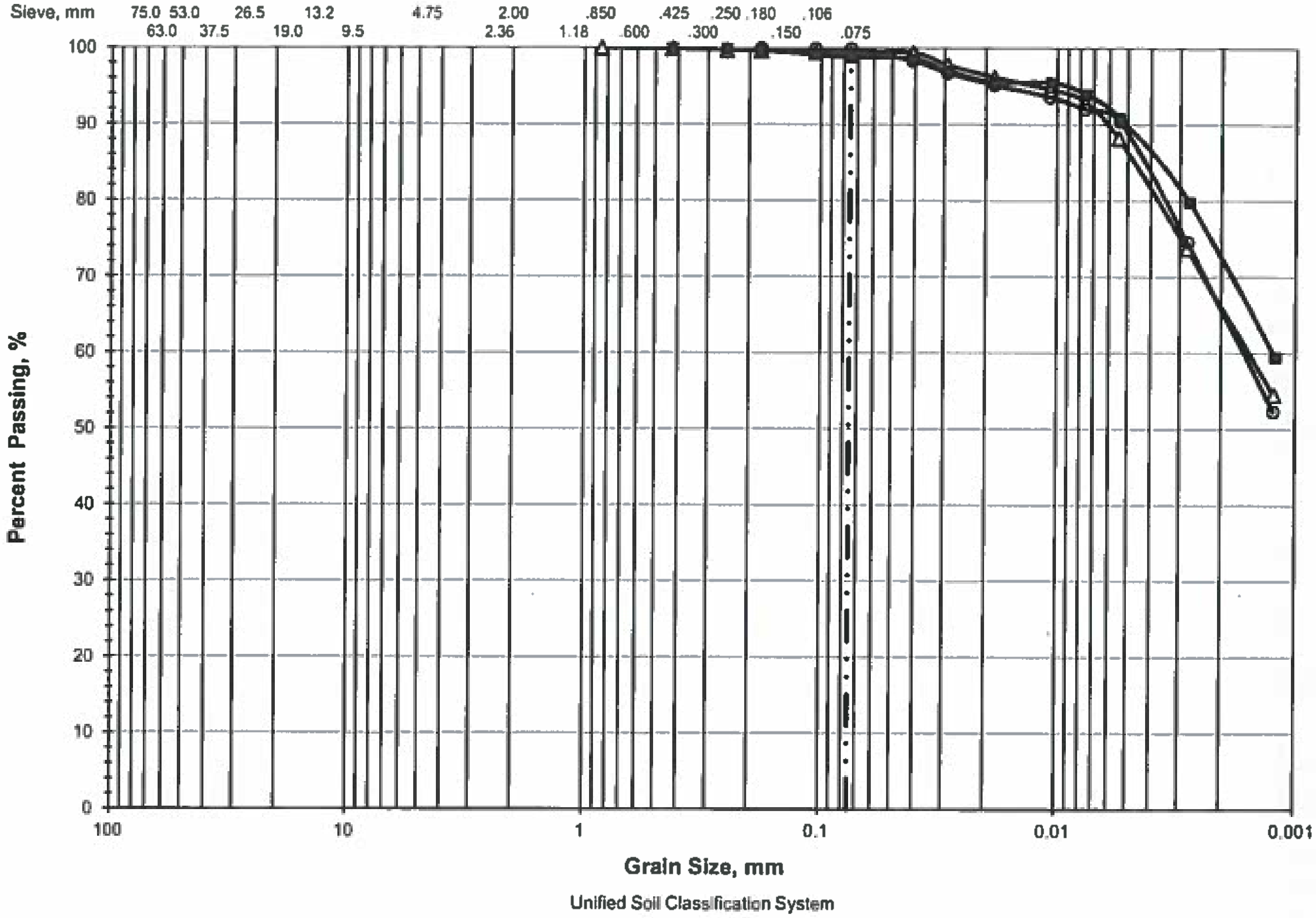
LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client: SmartCentre
Project: Geotechnical Investigation
Location: Innes Road & Mer-Bleue Road, Orleans, ON.

File No.: 180015
Report No.: 2
Date: April 6, 2018



	> 75 mm	% GRAVEL		% SAND			% FINES	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
△	0.0	0.0	0.0	0.0	0.1	0.6	35.3	64.0
■	0.0	0.0	0.0	0.0	0.0	1.2	28.8	70.0
○	0.0	0.0	0.0	0.0	0.0	0.1	36.6	63.3

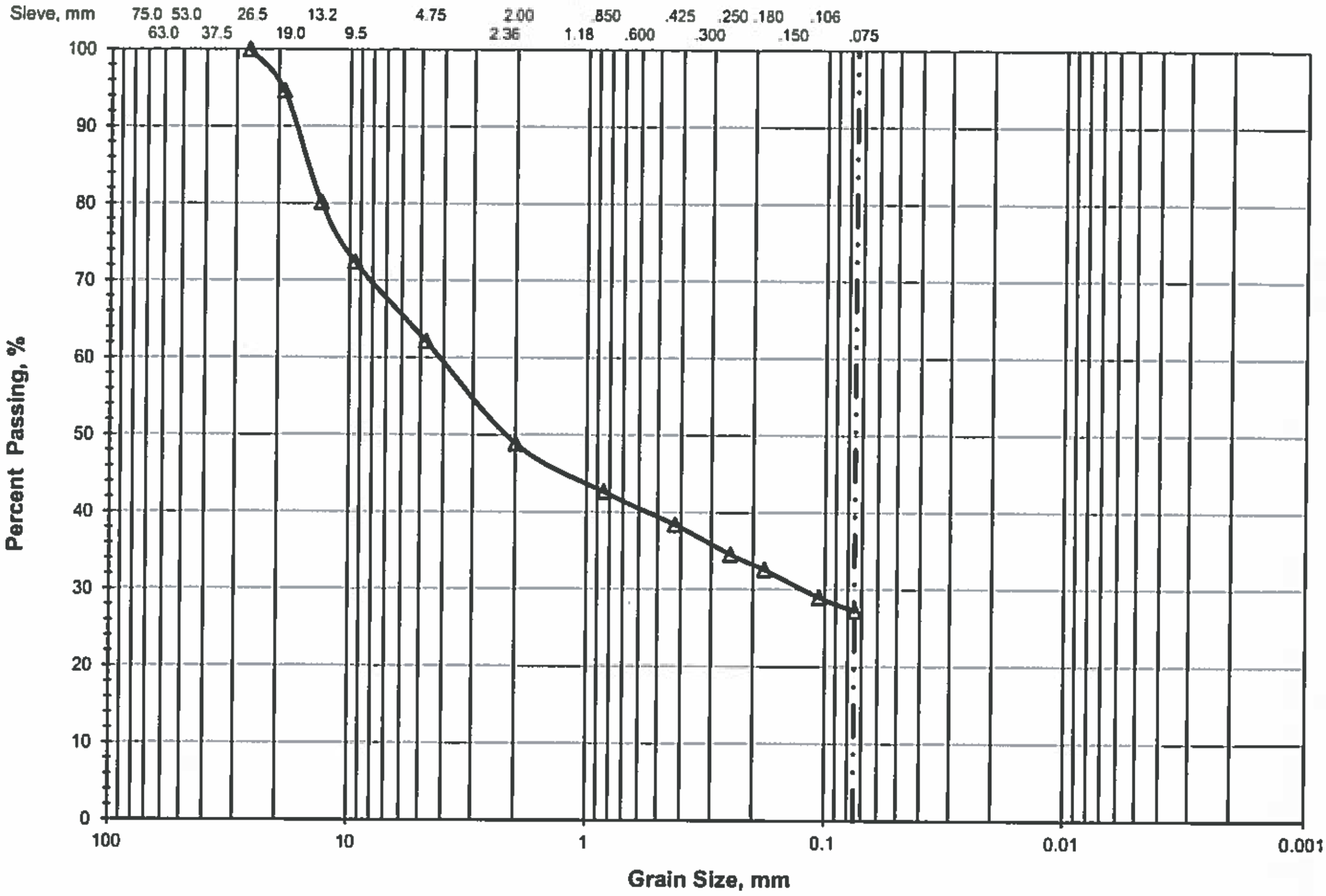
	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 1	SS-1	1.22 - 1.83	0.0017						
■	BH 3	SS-2	2.44 - 3.05	0.0012						
○	BH 4	SS-7	5.79 - 6.40	0.0018						

LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client:	SmartCentre	**File No.:**	180015
Project:	Geotechnical Investigation	**Report No.:**	3
Location:	Innes Road & Mer-Bleue Road, Orleans, ON.	**Date:**	April 6, 2018



Unified Soil Classification System

	> 75 mm	% GRAVEL		% SAND			% FINES
		Coarse	Fine	Coarse	Medium	Fine	Silt & Clay
△	0.0	4.7	33.1	13.3	10.5	11.1	27.3

	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 6	SS-2	3.05 - 3.51	4.2955	2.2230	0.1256				

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Certificate of Analysis

LRL Associates Ltd.

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

Client PO:
Project: 180015
Custody:

Report Date: 16-Apr-2018
Order Date: 10-Apr-2018

Order #: 1815172

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

Paracel ID	Client ID
1815172-01	BH 2 SS3 (15-17')
1815172-02	BH 5 SS1 (4 - 6')

Approved By:

Dale Robertson, BSc
Laboratory Director

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

Report Date: 16-Apr-2018
Order Date: 10-Apr-2018
Project Description: 180015

	Client ID: **Sample Date:** **Sample ID:** **MDL/Units**	BH 2 SS3 (15-17') 04/06/2018 09:00 1815172-01 Soil	BH 5 SS1 (4 - 6') 04/05/2018 13:00 1815172-02 Soil	- - - -	- - - -
Physical Characteristics					
% Solids	0.1 % by Wt.	53.0	65.4	-	-
General Inorganics					
pH	0.05 pH Units	8.41	7.76	-	-
Resistivity	0.10 Ohm.m	38.3	82.7	-	-
Anions					
Chloride	5 ug/g dry	14	11	-	-
Sulphate	5 ug/g dry	46	27	-	-