Geotechnical Investigation

Commercial Development
8015 Russell Road
Ottawa, Ontario

Prepared for:

2245040 Ontario Inc.
2142 Dunning Road
Cumberland, Ontario
K4C 1M1

Attention: Mr. Denis Desjardins

LRL File No.: 150570.02

December, 2015

TABLE OF CONTENTS

LIST OF TABLES

APPENDICES

1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by 2245040 Ontario Inc. to perform a geotechnical investigation for a proposed Commercial Development, located at 8015 Russell Road, Ottawa Ontario.

The purpose of the investigation was to identify the subsurface conditions across the site by the completion of a limited 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 DESCRIPTION

The site under investigation is currently a vacant lot. It is irregular in shape, and has an approximate surface area of 55.7 acres (22.5 hectares). The east portion of the site is a wooded area, and the west portion has been stripped of vegetation, and approximately 2.0 meters of fill found placed throughout the site. A berm has been constructed parallel to Russell Road at the edge of the property, and is approximately 2.5 m high by 3.0 m wide. Access to the property comes by way of Russell Road, and the designated civic address is 8015 Russell Road, Ottawa, Ontario.

It is our understanding that the proposed Commercial Development will consist of a two storey, 1,858 m^2 (about 20,000 ft^2) building with two garage bays, an asphalted area with subsequent parking, and a large gravel yard. The intended use of the property is a scrap yard for vehicles. The property will have a septic bed towards the west edge of the property.

3 PROCEDURE

The fieldwork for this investigation was carried out on December 3, 2015. 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 building footprint and asphalted area, where it was possible to do so. The approximate locations of the boreholes are shown on a site plan included in **Appendix A**.

The boreholes were advanced using a Geoprobe 7822DT drill rig equipped with 200 mm diameter continuous flight hollow stem auger supplied and operated by Strata Drilling Group. 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 mm diameter drive open conventional spoon sampler in conjunction with standard penetration testing (“N” values). The boreholes were advanced to depths ranging from 5.95 to 6.17 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 located using a Garmin Etrex Legend GPS (Global Positioning System) receiver using NAD 83 datum (North American Datum). No topographic survey was conducted, and elevation of the boreholes was assumed to be 100.00 m.

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 glacial deposits: till; heterogeneous mixture of material ranging from clay to large boulders, generally sandy, grades downward into modified till.

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 Fill

Fill comprised of sand-silt-clay and gravel with trace to some organics material was encountered in all three boring locations. The surficial fill at BH1 of depth about 0.6 m is mostly organics mixed with granular fill. Thickness of fill found ranged from 1.5 to 2.1 m bgs.

4.3 Possible Fill

Underneath the fill in BH3, a deposit of clean sand-silt-clay was encountered and thickness of this deposit was about 0.6 m. Standard penetration test was carried out in the possible fill and the SPT “N” value was 6, indicating the deposit is loose in consistency. The natural moisture content of the split spoon sample was measured in laboratory and found to be 47%, indicating its saturated condition.

4.4 Silty Clay

Underlying the fill and possible fill material, a layer of silty clay was encountered at all three boring locations. The layer extended to depths ranging from 4.2 to 4.8 m bgs. Standard penetration tests were carried out within the silty clay layer, and "N" values were ranged from 6 to 8 blows per 0.3 m of sample penetration, indicating a firm to stiff consistency. Its moisture content was found ranging between 34 and 53%, indicating it to be in saturated or wet condition.

4.5 Silty Clay Till

Underlying the silty clay, dark grey silty clay till was encountered in BH3. The deposit generally consisted of interlayered clay, silty clay and silt, and extended to depth about 6.14 m bgs. The consistency of this layer as interpreted based on SPT "N" values ranged from 23 to 50 per 0.3 m of sample penetration, indicating this layer to be very stiff to hard in consistency. The natural water content of the silty clay till was 12%, indicating it to be in a moist condition.

4.6 Shale

Shale with mud seams was encountered underneath the silty clay in BH1 and BH2, and extended to depths 6.17 and 5.95 m bgs respectively. The SPT "N" values ranged in this layer were between 13 and 40 per 0.3 m of sample penetration, indicating this layer to be very stiff to hard in consistency.

4.7 Refusal/Bedrock

Refusal over inferred large boulder or shale bedrock was encountered in BH1, BH2 and BH3 at depths of 6.17, 5.95, 6.14 m bgs respectively. The bedrock is most likely to be weathered to faintly weathered at the surface.

4.8 Laboratory Analyses

A representative soil sample was collected from BH2 between depth 2.3 and 2.7 m bgs for a gradation analyses. Laboratory gradation analysis was conducted following the procedure **ASTM C136** and **ASTM D422**. Based on the sieve and hydrometer analyses test results, the borehole sample revealed that the soil matrix is comprised of about 2.2% sand, 88.3% silt, and 9.5% clay. According to the Unified Soil Classification System, the soil would be classified as silt, trace clay, trace sand. Details of laboratory analyses are reflected in **Table 1**.

Table 1: Gradation Analysis Summary

Sample Location	No.	Depth (m)	Percent for Each Soil Gradation			Hydraulic Conductivity K (cm/s)
			Sand (%)	Silt (%)	Clay (%)	
BH2	SS4	2.3 – 2.7	2.2	88.3	9.5	3.14×10^{-6}

Atterberg limits and moisture contents were conducted on the silty clay sample collected between 2.3 and 2.7 m in BH1. Based on the test result, the sample yielded a plastic limit of 32% and corresponding liquid limit of 55%. According to the Unified Soil Classification System, the soil in BH1 would be classified as inorganic clays of medium to high plasticity. Details of these laboratory analyses are reflected in **Table 2**.

Table 2: Summary of Atterberg Limits and Water Contents

Location (Sample)	Parameter					
	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Water Content (%)	USCS Group Symbol
BH1	2.4 – 2.7	55	32	23	34.2	OH

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

4.9 Groundwater Conditions

Groundwater was carefully monitored and measured during this field investigation. Immediately after completion of drilling, water was encountered in BH1, BH2, and BH3; at 3.65, 3.96, and 5.48 m bgs respectively. The boreholes were left open and water levels were further measured after 2.0 hours and found to be at 1.40, 1.45, 4.57 m bgs respectively. The water level measurements are 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.

It is our understanding that the proposed Commercial Development will consist of a two (2) storey, 1,858 m^2 (about 20,000 ft^2) building with two garage bays, an asphalted area with subsequent parking (34+1 P.D.), and a large gravel yard.

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

5.1 Foundations

Based on the subsurface soil conditions established at this site, it is recommended that the footings for the proposed building be founded over the undisturbed silty clay layer or shallow structural fill. Therefore, any fill material should be removed from the building's footprint down to relatively stable silty clay deposit.

5.1.1 Shallow Foundation on Native Silty Clay

Structural load may be supported on reinforced, spread and continuous strip footings for column and load bearing walls respectively. The footings founded over stiff silty clay at 2.1 m bgs (at boring locations) may be designed using a 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 also allows for a strip footing of width minimum 0.6 to maximum 0.75 m and pad footing of width minimum 1.2 to maximum 1.6 m on any side. It is noted that the weight of the footing and the backfill soil above the footing should be included when calculating footing bearing pressure. Footing shall be founded a minimum of 0.3 m above the high groundwater table.

Prior to pouring footing concrete, the subgrade comprised of the stiff silty clay deposit should be inspected and approved by geotechnical engineer or a representative of geotechnical engineer. In-situ (field vane shear test) 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. All the footings founded on silty 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. Footing and foundation walls shall be reinforced, especially at segments where footing 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 9.15.3.8 of OBC 2012**.

If the grade raise is greater than that mentioned above, the above allowable bearing pressure at serviceability limit state may have to be reduced.

5.1.2 Foundation on Structural Fill

Conventional strip and column footings set over properly compacted and approved engineered fill conforming to OPSS Granular B Type II or approved equivalent may be considered for a maximum allowable bearing pressure of **100 kPa** for Serviceability Limit State (**SLS**) and **150 kPa** for Ultimate Limit State (**ULS**) factored bearing resistance at depth 1.5 to 1.8 m bgs. The factored ULS value includes the geotechnical resistance factor of 0.5. The allowable bearing resistance is based on maximum footing width of 0.75 m for strip footing and 1.6 m for pad footings on any sides. Therefore, any fill material, including soft native deposit, should be removed from the proposed building’s footprint down to relatively competent native subgrade at depth about 2.1 m bgs. Prior to placing the approved engineered fill / structural fill, the subgrade comprises of silty clay should be inspected and proof-rolled with a heavy duty vibratory equipment (ten tonne or larger) under suitable (dry) conditions. Any incompetent subgrade areas as identified from proof rolling must be sub-excavated and backfilled with approved structural fill and compacted to 98% of its Standard Proctor Maximum Dry Density (SPMDD). Similarly, any soft or wet areas should also be sub-excavated and backfilled with suitably compacted structural fill. The above bearing capacity is contingent on founding the footing at minimum 0.3 m above the water table. If the strip footings need to be founded at different level, it is recommended to use the step footing as recommended in **Clause 9.15.3.9** of OBC 2012.

If the footings are wider or founded deeper, the maximum allowable bearing pressure for serviceability limit state may have to be reduced. Any fill needed to raise the footing subgrade of the proposed two storeys commercial building should consist of imported granular material meeting the Ontario Provincial Standard Specification (OPSS) 1010 Granular B type II gradation requirements or approved equivalent.

The bearing values of the silty clay or structural fill and the corresponding founding elevations at the borehole locations are summarized on **Table 3**.

Table 3: Bearing Values and Founding Levels of Footings

BH No.	UTM Easting (m)	UTM Northing (m)	Material	Minimum Depth below Existing Ground (m)	Footing Founding Elevation (m)	Note
BH1	470460	5024438	Silty clay Structural Fill	2.1 1.5 -1.8	97.9 98.5 – 98.2	Shallow ground-water is at about Elev. 98.6 m as recorded in BH1
BH2	470492	5024473	Silty clay Structural Fill	2.1 1.5 – 1.8	97.9 98.5 – 98.2	
BH3	470527	5024476	Silty clay Structural Fill	2.1 1.5 – 1.8	97.9 98.5 – 98.2	

5.2 Structural Fill

For foundations set over soil and where excavation below the underside of the footing 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 (silty clay) in layers not exceeding 200 mm and compacted to 98% of its SPMDD within 2% of its optimum moisture content. In order to allow the spread of load beneath the footings and to prevent under mining 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 was achieved. Approved bi-axial geogrid in two layers (one at 0.3 m below the underside of footing and the other one at 0.3 m above the approved subgrade are required to be installed to minimize any differential settlement).

5.3 Settlement

The estimated total settlement of the shallow 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.

During our site investigation and developing this report, no site grading or servicing plan are provided for this project. Therefore, the above given settlements have considered that the terrain will not be raised by more than 0.5 – 0.8 m above current elevations. If a greater grade raise are required, a more intensive geotechnical review will be required to ensure that the clayey soil will not exceed the settlement limit under this load.

5.4 Seismic

Based on the limited information of this geotechnical investigation and in accordance with the Ontario Building Code 2012 (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 noted that a greater seismic site response may be obtained by conducting a seismic velocity testing using a multichannel analysis of surface wave (MASW).

5.5 Potential for Soil Liquefaction

Referring to Canadian Foundation Engineering Manual, 2006, soil is not susceptible to liquefaction or cyclic mobility, if the ratio of moisture content to liquid limit is less than 80% (i.e. $w/w_L < 0.8$) and plasticity index, $I_p \geq 20$; but may undergo significant deformations if cyclic shear stresses > static undrained shear strength. Based on the subsurface soil conditions established at this site, the foundations will be founded over undisturbed native silty clay or structural fill. Laboratory results indicate that the representative silty clay sample at depth between 2.3 and 2.7 m bgs having a ratio of natural moisture content to liquid limit is about 62% (<80%) and the relative plasticity index, I_P is 23. As such, it is anticipated soil liquefaction will not be an issue for this site for constructing shallow foundation at depth between 2.1 m bgs. Shallow groundwater at depth about 1.4 m bgs was encountered during our field investigation which is mostly perched-water. However, soil liquefaction, if any, will possibly be mitigated through appropriate sump pumping/lowering groundwater table.

5.6 Frost Protection

All exterior footings located in any unheated portions of the proposed building 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 basements or whenever any open spaces located below the finish ground are being considered. A 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 wrapped in a geotextile 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.

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. The exterior grade should be sloped away from the building to promote water drainage away from the foundation walls.

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

For predictable performance for the proposed slab-on-grade, it should rest over undisturbed competent native soil (silty clay) or structural fill only. Therefore, all fill including organic or otherwise deleterious material shall be removed from the building's footprint. The exposed native subgrade surface should then be inspected and approved by 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 area of extensive exterior slab-on-grade (sidewalks, ramp etc.) shall be constructed using Granular B subbase of thickness 450 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 **14 MPa/m.**

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 5.0 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 4** 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.

Table 4: Material Properties for Shoring and Permanent Wall Design (Static)

Type of Material	Bulk Density (kN/m^3)	Pressure Coefficient		
		Active (K_a)	Passive (K_p)	At Rest (K_o)
Granular A	22.0	0.43	0.27	3.69
Granular B Type I	20.0	0.48	0.32	3.12
Granular B Type II	23.0	0.47	0.31	3.25
Sand	19.0	0.50	0.33	3.00
Silty clay	17.5	0.62	0.51	1.97
Till	23.0	0.43	0.27	3.70

The above values are for a flat surface behind the wall, a straight wall and a wall friction angle of 0 degree. 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 Canadian Building Code recommends the used of combined coefficients of static and seismic earth pressure, referred to as K_{AE} for active conditions and K_{PE} for passive conditions for routine design purposes.

The total active and passive loads under seismic conditions can be calculated using the following two equations;

$$P_{AE} = ½ K_{AE} \gamma H^2 (1\text{-}k_V)$$

$$P_{PE} = ½ K_{PE} \gamma H^2 (1\text{-}k_V)$$

Where;

K_{AE} = Combined static and seismic active earth pressure coefficient

K_{PE} = Combined static and seismic passive earth pressure coefficient

H = Total height of the wall (m)

K_h = horizontal acceleration coefficient

K_v = vertical acceleration coefficient

γ = bulk density (kg/m^3)

These equations are based on a horizontal slope behind the wall and a vertical back of the retaining wall and zero wall friction. For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values.

A = Zonal acceleration ratio = 0.2

K_h = Horizontal acceleration coefficient = 0.1

K_V = Vertical acceleration coefficient = 0.067

The above value of K_h corresponds to ½ of the A value and the value K_V of corresponds to 0.67 of the K_h value. The angle of friction between the soil and the wall has been set at 0^o to provide a conservative estimate.

The following **Table 5** provides the parameters for seismic design of retaining structures

Table 5: Material Properties for Shoring and Permanent Wall Design (Seismic)

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II
Bulk Unit Weight, γ (kN/m³)	20	23
Effective Friction Angle (degrees)	30	32
Angle of Internal Friction Between wall and Backfill (degrees)	0	0
Yielding Wall		
Active Seismic Earth Pressure Coefficient (K_{AE})	0.37	0.33 (Granular A) & 0.37 (Granular B Type II)
Height of the Application of P_{AE} from the base of the wall as a ration of its height (H)	0.36	0.37
Passive Seismic Earth Pressure Coefficient (K_{PE})	3.06	3.48
Height of the Application of P_{PE} from the base of the wall as a ration of its height (H)	0.30	0.30

5.11 Sulphate Attack and Corrosivity Analysis on Buried Concrete

No chemical analyses were conducted during our limited geotechnical investigation but should be confirmed prior to proceeding with proposed development works.

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation

It is anticipated that the depth of excavation for the building and underground services will not extend below 2.1 m bgs. Most of the excavation being carried out will be through fill and native silty clay. 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 can be cut at 1 horizontal to 1 vertical (1H: 1V) for a fully drained excavation 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 4 and Table 5** 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

Based on the subsurface conditions encountered at this site, groundwater seepage or infiltration from the native silty clay into shallow temporary excavations (less than 2.1 m) during construction should be minor to moderate. 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 temporary permit to take water (PTTW) from the Ministry of Environment (MOE) is recommended for this project, which would be required if more than 50,000 liters/day are to be pumped during excavation/construction. It generally takes 4-6 months for processing the application and issuance of the permit by the MOE.

6.3 Pipe Bedding Requirements

It is anticipated that the underground services required as part of this project will be founded over native silty 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 laid in loose lifts of no more than 200 mm thick and compacted to 95% of its SPMDD. Bedding, thickness of cover material and compaction requirements for watermains and sewers 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 below the shallow groundwater level. Where sandy soil will constitute the founding soil and is located 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 (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 at minimum to OPSS Granular B Type I.

Any boulders larger than 150 mm in size should not be used as trench backfill. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming at minimum to OPSS Granular B Type I or approved equivalent.

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% 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.

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% 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.

7 Reuse of On-Site Soils

The existing surficial overburden soils consist mostly of fill (sand & gravel mixed with organics) and sand-silt-clay. The fill and overburden silty soil 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 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 PAVEMENT DESIGN

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 firm to stiff silty clay once all debris, organic material, objectionable fill 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.

The following are the recommended pavement structures for light and heavy duty access roads and parking areas proposed as part of this project.

For light vehicle access lanes and parking area, the pavement should consist of:

50 mm of hot mix asphaltic concrete surface course, HL-3 over

150 mm of OPSS Granular A base over

350 mm of OPSS Granular B Type II subbase

For heavy duty access roads, the pavement should consist of:

40 mm of hot mix asphaltic concrete surface course, HL-3 over

50 mm of hot mix asphaltic concrete binder layer, HL-8 over

150 mm of OPSS Granular A base over

450 mm of OPSS Granular B, Type II subbase

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% SPMDD. Asphaltic concrete shall conform to **OPSS 1150** and be placed and compacted to at least 97% 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 proposed 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 Tandem axle, dual wheel dump truck (with tire pressure not less than 90% of the manufacturer’s recommended maximum inflation and minimum gross weight of the loaded truck shall be 37,000 kg) shall be used. Any resulting loose/soft areas should be sub-excavated down to an adequate bearing layer and replaced with approved backfill.

Any materials used as select subgrade should be approved by the geotechnical engineer before placement within the roadway. These materials should be placed in maximum 300 mm thick loose lifts and be compacted to at least 95% of its SPMDD using suitable compaction equipment. Following approval of the preparation of the subgrade, the pavement structure may be placed.

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 months.

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 engineered 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 months, 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 test locations only. Boundaries between zones presented on the borehole and test pit logs 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 report recommendations are applicable only to the project described in the 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.

Mohammed Zamshad, M.Eng., P.Eng.

Stéphane Leclerc, P.Eng.

W:\FILES 2015\150570\02-Reports\08 Geotechnical\8.01 Reports\150570.02Geotechnical Investigation-Commercial Development.docx

APPENDIX A

Site and Borehole Location Plan

ENGINEERING I INGÉNIERIE

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

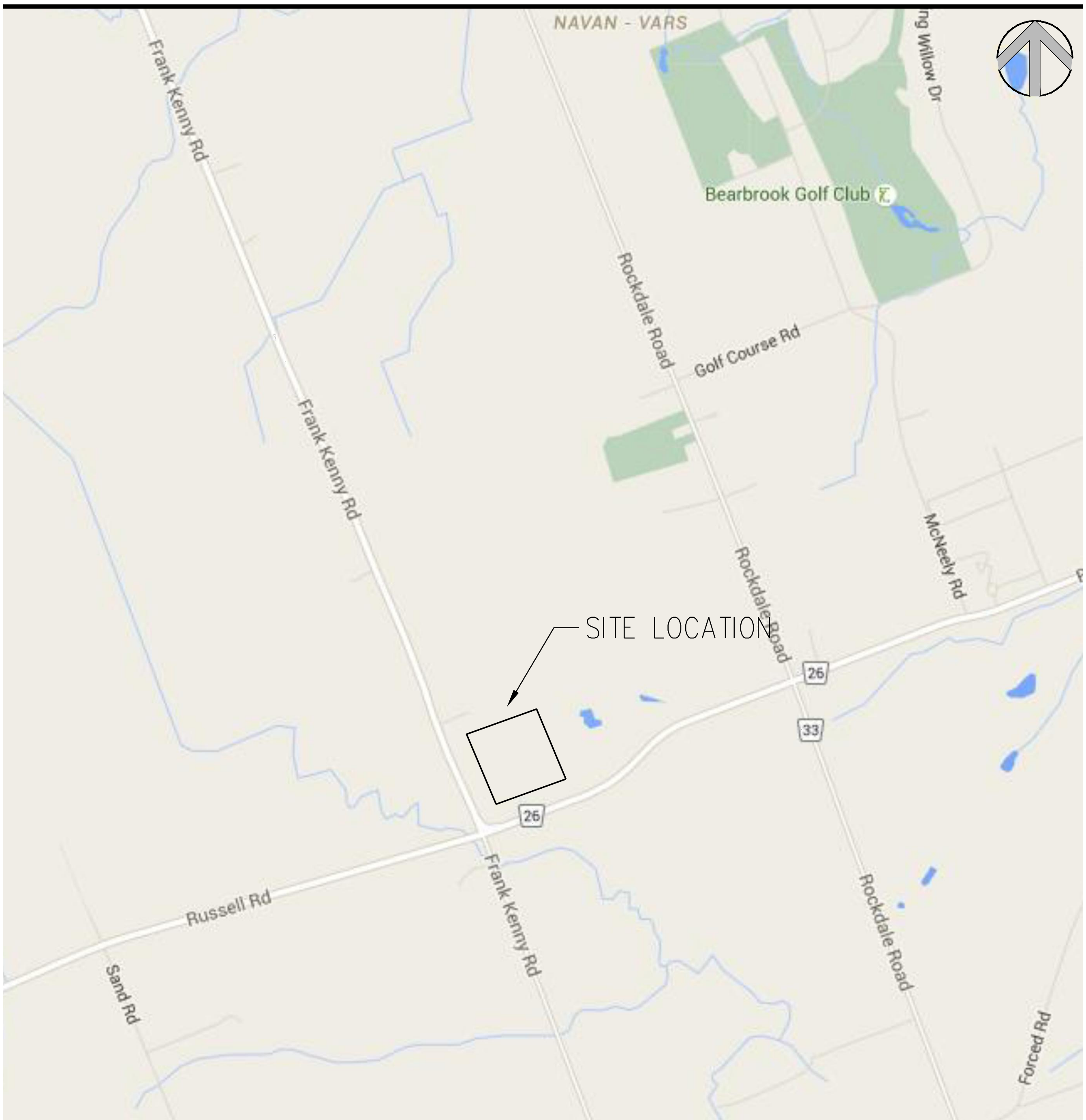
PROJECT

GEOTECHNICAL INVESTIGATION
COMERCIAL DEVELOPMENT
8015 RUSSELL ROAD
OTTAWA, ON

DRAWING TITLE

SITE LOCATION

CLIENT	DATE	PROJECT	
2245040 ONTARIO INC.	DECEMBER 10, 2015	150570.02	FIGURE 1



ENGINEERING I INGÉNIERIE

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

PROJECT

GEOTECHNICAL INVESTIGATION
COMMERCIAL DEVELOPMENT
8015 RUSSELL ROAD
OTTAWA, ON

DRAWING TITLE

BOREHOLE LOCATION

CLIENT	DATE	PROJECT	
2245040 ONTARIO INC.	DECEMBER 10, 2015	150570.02	FIGURE 2

APPENDIX B

Borehole Logs

Borehole Log: BH1

Project No.: 150570.02
Project: Commercial Development
Client: 2245040 Ontario Inc.
Location: 8015 Russell Road, Ottawa ON
Date: December 3, 2015
Field Personnel: BJ
Driller: Strata Drilling Group
Drilling Equipment: Geoprobe 7822DT
Drilling Method: HSA

SUBSURFACE PROFILE

Depth (m)	Soil Description	Elev./Depth(m)
0	Ground Surface	100.00
0.00–0.60	**Fill-** sand and gravel with trace to some orgaincs.	100.00 / 0.00 – 99.40
0.60–2.10	**Fill-** sand-silt-clay with some black orgaincs, very moist.	99.40 / 0.60 – 97.90
2.10–4.30	**SILTY CLAY-** trace sand, brownish grey, wet, stiff. -colour changes to grey below about 3.2 m.	97.90 / 2.10 – 95.70
4.30–6.17	**SHALE-** some mud seams, dark grey, moist, hard.	95.70 / 4.30 – 93.83
6.17	End of Borehole	93.83 / 6.17

SAMPLE DATA

Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)
1	9	75	9	24
2	11	89	11	18
3	9	75	9	33
4	8	100	8	34
5	8	83	8	40
6	40	100	40	14
7	50		50	12

Shear Strength (kPa): 50 100 150 200
SPT N Value (Blows/0.3 m): 20 40 60 80
Water Content (%): 25 50 75
Liquid Limit (%): 25 50 75

Water Level (Standpipe or Open Borehole)
1.40 m bgs 2 hours after drilling.
3.65 m bgs on completion of drilling.

Easting: 470460
Northing: 5024438
Site Datum: Assumed
Groundsurface Elevation: 100.00
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES: End of borehole at 6.17 m bgs.
Water level at 3.65 m on completion and 1.4 m bgs 2 hours after drilling.
Cave-in at 5.18 m bgs.
50 blows for 8 cm of sample penetration.

Borehole Log: BH2

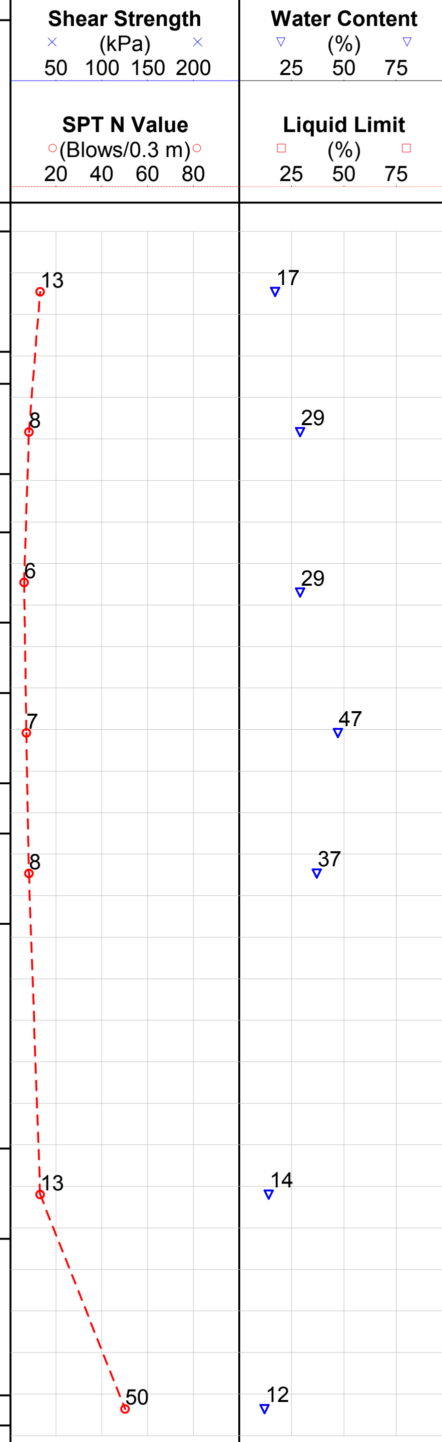
Project No.: 150570.02
Project: Commercial Development
Client: 2245040 Ontario Inc.
Location: 8015 Russell Road, Ottawa ON
Date: December 3, 2015
Field Personnel: BJ
Driller: Strata Drilling Group
Drilling Equipment: Geoprobe 7822DT
Drilling Method: HSA

SUBSURFACE PROFILE

Elev./Depth (m)	Soil Description
100.00	Ground Surface
0.00 – 98.80 / 1.20	**Fill-** sand-silt with some black organics, some clay, ocassional gravel sized stone, very moist.
1.20 – 97.90 / 2.10	**Fill-** sand-silt-clay, trace rootlets, wet.
2.10 – 95.20 / 4.80	**SILTY CLAY-** trace sand, grey, wet, firm. -becomes stiff below about 3.3 m.
4.80 – 94.05 / 5.95	**SHALE-** weathered, some mud seams, moist, very stiff.
5.95	End of Borehole

SAMPLE DATA

Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)
1	13	63	13	17
2	8	67	8	29
3	6	44	6	29
4	7	100	7	47
5	8	100	8	37
6	13	50	13	14
7	50		50	12



Water Level (Standpipe or Open Borehole)

1.45 m bgs 2 hours after drilling.

3.96 m bgs on completion of drilling.

Easting: 470492
Northing: 5024473
Site Datum: Assumed
Groundsurface Elevation: 100.00
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES: End of borehole at 5.95 m bgs.
Water level at 3.96 m bgs on completion of drilling and 1.45 m bgs 2 hours after drilling.
Cave-in at 4.87 m bgs.
50 blows for 10 cm of sample penetration.

Borehole Log: BH3

Project No.: 150570.02
Project: Commercial Development
Client: 2245040 Ontario Inc.
Location: 8015 Russell Road, Ottawa ON
Date: December 3, 2015
Field Personnel: BJ
Driller: Strata Drilling Group
Drilling Equipment: Geoprobe 7822DT
Drilling Method: HSA

SUBSURFACE PROFILE / SAMPLE DATA

Soil Description	Elev./Depth(m)	Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)
Ground Surface	100.00					
Fill- sand-silt mixed with black orgaincs.	0.00	1	21	83	21	15
		2	7	28	7	25
Possible fill- sand-silt-clay, brownish grey, wet.	98.50 / 1.50	3	6	100	6	47
SILTY CLAY- grey, wet, firm.	97.90 / 2.10	4	7	67	7	50
		5	6	100	6	53
SILTY CLAY TILL- trace fine gravel, trace sand, occasional gravel sized stone, dark grey, moist, compact.	95.80 / 4.20	6	23	56	23	12
		7	50		50	
End of Borehole	93.86 / 6.14					

Shear Strength (kPa): 50 100 150 200
Liquid Limit (%): 25 50 75

Water Level (Standpipe or Open Borehole):
5.48 m bgs on completion of drilling
4.57 m bgs 2 hours after drilling

Easting: 470527
Northing: 5024476
Site Datum: Assumed
Groundsurface Elevation: 100.00
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES: End of borehole at 6.14 m bgs.
Water level at 5.48 m bgs on completion and 4.57 m bgs 2 hours after drilling.
Cave-in at 5.48 m bgs.
50 blows for 5 cm of sample penetration.

APPENDIX C

Symbols and Terms used in Borehole Logs

Symbols and Terms Used on Borehole and Test Pit Logs

The following explains the data presented in the 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 Test. See Section 2c for more details. 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”
Very loose	0 – 4
Loose	4 – 10
Compact or medium	10 - 30
Dense	30 - 50
Very dense	over - 50

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

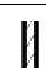
Consistency Cohesive Soils	Undrained Shear Strength (Cu) (kPa)
Very soft	under 10
Soft	10 - 25
Medium or firm	25 - 50
Stiff	50 - 100
Very stiff	100 - 200
Hard	over - 200

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. Blows (N) or RQD

This column indicates the Standard Penetration Number (N) as per ASTM D-1586. This is used to determine the state of compactness of the soil sampled. It corresponds to the number of blows required to drive 300 mm of the split spoon sampler using a 622 kg*m/s^2 hammer falling freely from a height of 760 mm. For a 600 mm long split spoon, the blow counts are recorded for every 150 mm. The "N" index is obtained by adding the number of blows from the 2nd and 3rd count. Technical refusal indicates a number of blows greater than 50.

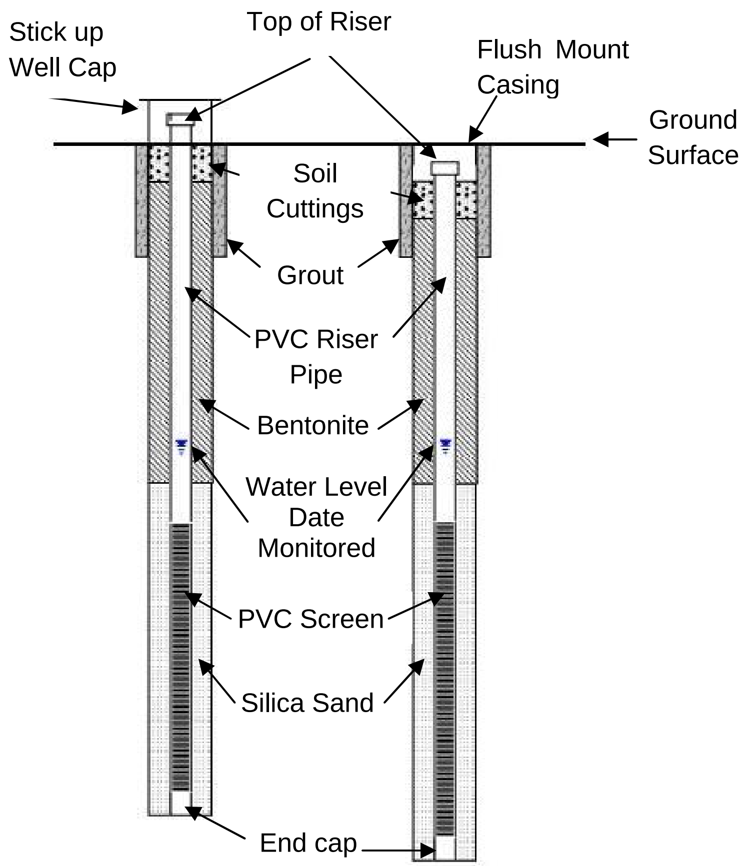
In the case of rock, this column presents the Rock Quality Designation (RQD). The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 10 cm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given below.

Rock Quality Designation (RQD) (%)	Description of Rock Quality
0 –25	very poor
25 – 50	poor
50 – 75	fair
75 – 90	good
90 – 100	excellent

e. 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. General Monitoring Well Data



APPENDIX D

Lab Results

Sieve and Hydrometer Analysis

5430 Canotek Road
Ottawa, Ontario
K1J 9G2

Tel: (613) 842-3434 or (877) 632-5664
Website: www.LRL.ca
Fax: (613) 842-4338

Project Information

Client: 2245040 Ontario Inc.
Project: Commercial Development
Date Sampled: 03/12/2015

Project No.: 150570.02
Report No.: 1
Sample No.: BH2: 2.3-2.7 m

Test Results

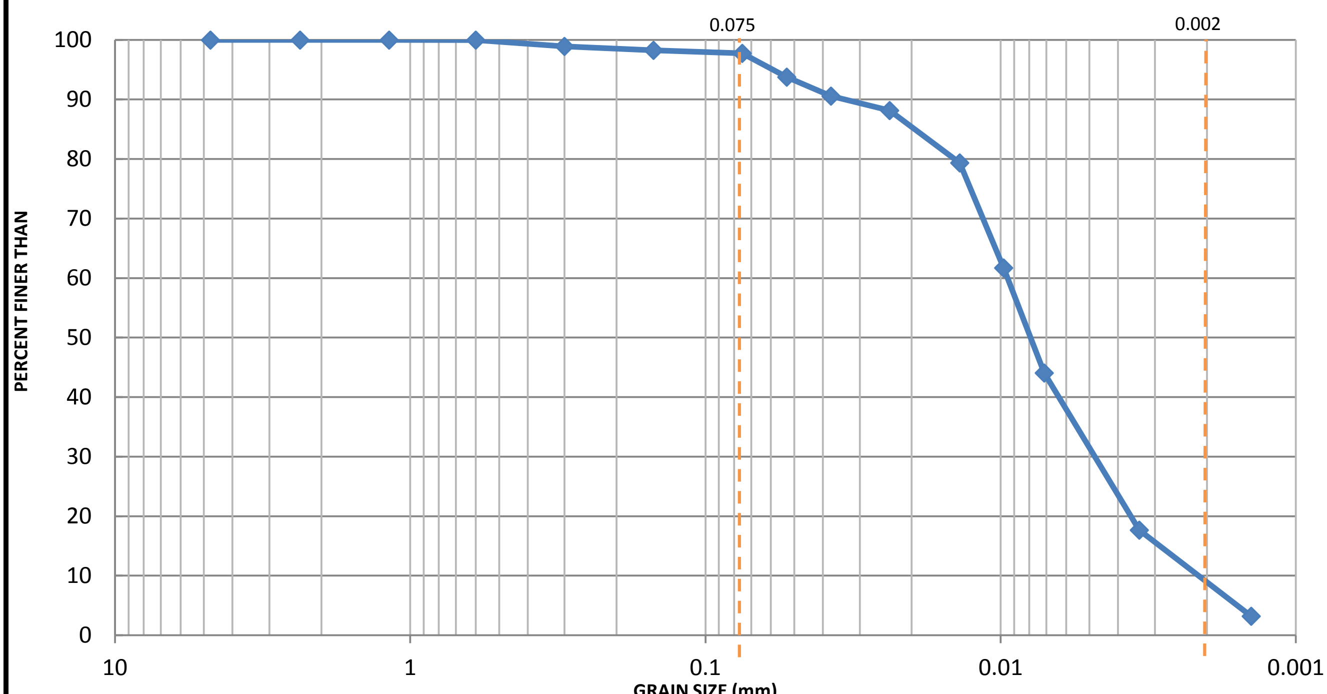
Small Sieve Results

Sieve #	Sieve size	Percent finer (%)
4	4.75	100.00
8	2.36	100.00
16	1.18	100.00
30	0.6	99.95
50	0.3	98.92
100	0.15	98.25
200	0.075	97.77

Hydrometer Results

Particle size (mm)	Percent finer (%)
0.0530	93.75
0.0375	90.55
0.0238	88.14
0.0138	79.33
0.0098	61.70
0.0071	44.07
0.0034	17.63
0.0014	3.21

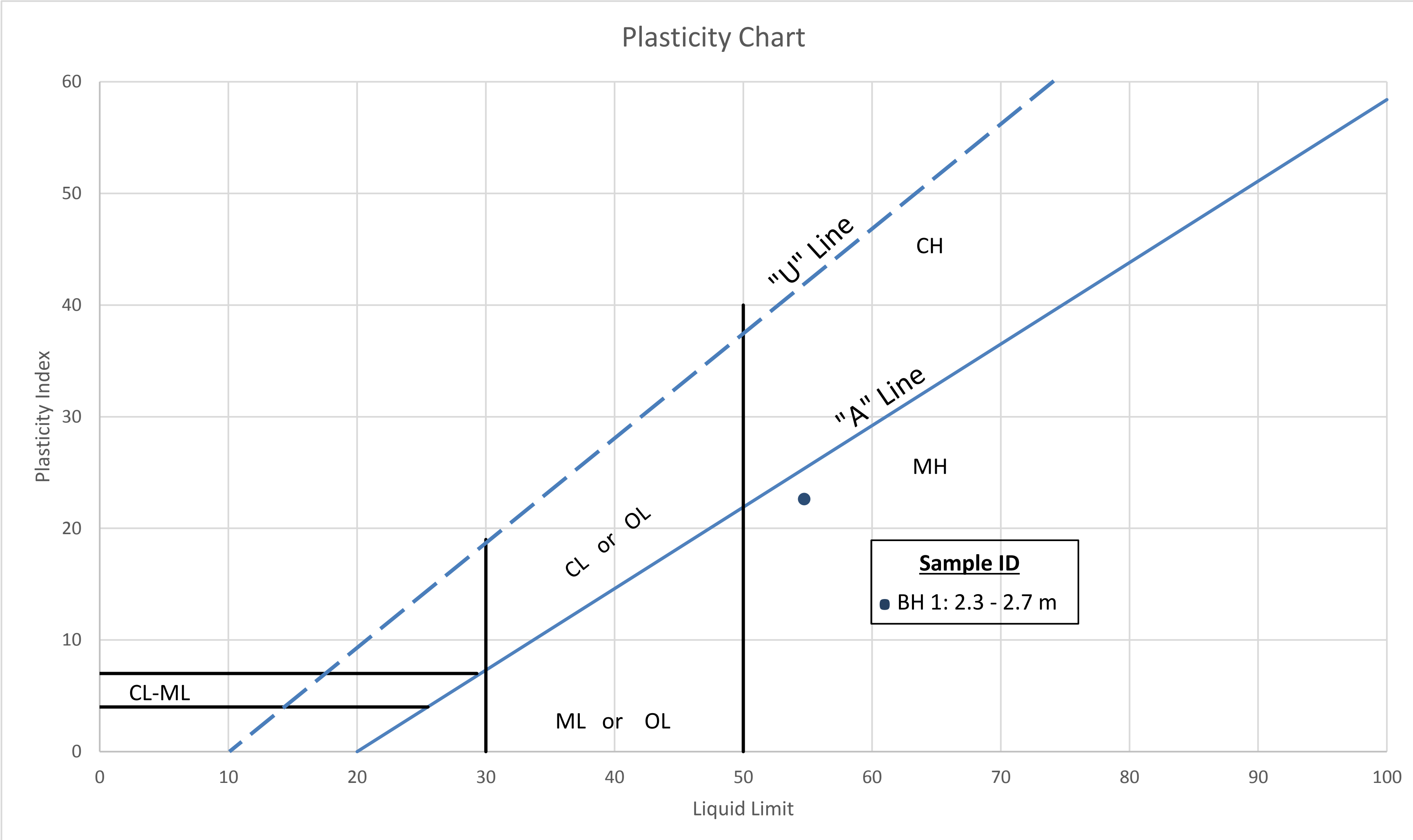
Grain Size Distribution



% Sand	% Silt	% Clay
2.23%	88.27%	9.50%

Tested By: Norman Johnson

Approved By: Joel Lajeunesse



PLASTICITY CHART

Client: 2245040 Ontario Inc.
LRL File Number: 150570.02
Date: December 17, 2015