Geotechnical Investigation

Proposed Four (4) Storey Low-Rise Apartment
10-20 Empress Avenue North
Ottawa, Ontario

Prepared for:

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LRL File No.: 240202

May 2024

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1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by Dean and Denis Michaud on behalf of Dalhousie Non-Profit Cooperative Inc. to perform a geotechnical investigation for a four (4) Storey Low-Rise Apartment, to be located at 10 Empress Avenue North, Ottawa, Ontario.

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

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

2 SITE AND PROJECT DESCRIPTION

The site under investigation is located at 10-20 Empress Avenue North, Ottawa, ON. Currently the site is occupied by an abandoned two (2) storey (with basement) multi-unit residential building. On grade parking is available at the rear of the site; fronting Perkins Street. The site is bound by Empress Avenue North to the east, 6 Empress Avenue North to the north, 22 Empress Avenue North to the south, and Perkins Street to the west. The topography of the site is relatively flat. The site is accessible from both Empress Avenue North and Perkins Street. The site location is presented in Figure 1 included in **Appendix A**.

At the time of generating this report, it is understood the development will consist of demolition of all structures onsite, and construction of a four (4) storey low-rise apartment building, complete with one (1) level below grade for a combination of living space and underground parking.

3 PROCEDURE

The fieldwork for this investigation was carried out on May 03, 2024. Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of four (4) boreholes, labelled BH1 through BH4, were drilled onsite at pre-determined locations; as agreed upon by LRL and client representative. The approximate locations of the boreholes are shown in Figure 2 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 George Downing Estate Drilling Ltd. 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 augered and sampled to a depth of 8.23 m below (existing) ground surface (bgs). In BH2, a Dynamic Cone Penetration (DCP) Test was carried out to a depth of 10.06 m bgs; after encountering refusal. Upon completion, piezometers were installed in three (3) of the boreholes, the remaining borehole was backfilled using the 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.

Furthermore, all boreholes were 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 Site Benchmark (Nail in Utility Pole: 63.43 m). Ground surface elevations of the boring locations are shown on their respective borehole 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 is Glacial Deposits consisting of a till material; a heterogenous mixture of material ranging from clay to large boulders.

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

4.2 Topsoil

Topsoil of thickness ranging from about 75 to 200 mm was found at the surface of BH1 through to BH3.

It 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 Fill

At the surface of BH4, and below the topsoil in BH1 through BH3, a layer of fill material was encountered and extended to depths ranging between 1.20 and 1.68 m bgs. This material generally was comprised of silt-sand-clay, some gravel sized stone, brown to dark brown, and moist. Standard Penetration Tests (SPTs) were carried out in the fill material and the "N" values were found to range between 2 and 7, indicating the material is loose to very loose. The natural moisture contents were found to range between 10 and 31%.

4.4 Glacial Till

Underlying the fill material at all boring locations, a layer of glacial till was encountered and extended to a depth of 8.23 m bgs (end of exploration), and an inferred depth of 10.06 m bgs. The material can be described as a mixture of silt-sand, trace clay, trace to some gravel sized stone, brown, and moist, becoming grey and wet with increased depths. The "N" values were found to range between 2 and 50+ indicating the material is very loose to very dense. The natural moisture contents were found to range between 7 and 20%.

4.5 Refusal

Refusal by way of the DCP test was encountered on a large boulder within the glacial till material, or possible bedrock, at a depth of 10.06 m bgs.

4.6 Laboratory Analysis

Four (4) soil samples were collected for laboratory gradation analyses. The gradation analyses comprised of sieve and hydrometer were conducted following the procedure **ASTM D422.** Details of laboratory analyses are reflected in **Table 1**.

Table 1: Gradation Analysis Summary

Sample Location	Depth (m)	Percent for Each Soil Gradation							Estimated Hydraulic Conductivity K (m/s)
		Gravel		Sand			Silt (%)	Clay (%)	
		Coarse (%)	Fine (%)	Coarse (%)	Medium (%)	Fine (%)			
BH1	1.5-2.1	0.0	8.7	6.1	15.9	31.4	29.8	8.1	5×10^{-6}
BH2	3.1-6.7	7.7	10.7	5.4	13.5	29.1	27.5	6.1	5×10^{-6}
BH3	4.6-5.2	0.0	13.1	7.8	13.5	30.6	28.9	6.1	5×10^{-6}
BH4	6.1-6.7	5.9	7.4	5.6	13.2	29.6	31.8	6.5	5×10^{-6}

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

4.7 Groundwater Conditions

Three (3) piezometers were installed in BH1, BH3, and BH4 to measure the static groundwater level. The piezometers consisted of a 19 mm diameter PVC pipe with slotted bottoms to allow for groundwater infiltration, backfilled with silica sand, and sealed with bentonite.

The water was measured on May 16, 2024 and found to be at 2.5 m, 2.5 m, and 2.4 m bgs in BH1, BH3, and BH4 respectively.

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 near the vicinity of the site.

5 Geotechnical Considerations

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

This section will detail design parameters for 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, two (2) possible foundation options to support to building structure may be conventional strip and column footings; or a raft slab foundation.

5.1.1 Shallow Foundations – on Conventional Strip and Column Footings

Conventional strip and column footings founded over the undisturbed native glacial till may be designed using a maximum allowable bearing pressure of **90 kPa** for serviceability limit state **(SLS)** and **135 kPa** for ultimate limit state **(ULS)** factored bearing resistance. The factored ULS value includes the geotechnical resistance factor of 0.5. There are no maximum footing width nor grade raise restrictions for this site.

In-situ field testing is required to check the strength and stability of the footings subgrade. Any incompetent subgrade areas as identified from in-situ testing must be sub-excavated and backfilled with approved structural fill. Similarly, any soft or wet areas should also be sub-excavated and backfilled with approved structural fill only. Prior to placing any approved structural fill, the subgrade should be inspected and approved by a geotechnical engineer or qualified geotechnical personnel.

The bearing pressure is contingent on the water level being 0.3 m below the underside footing elevation in order to have a stable and dry subgrade during construction. This shall be done by pumping from open sump pits, extending below the underside of footing elevation.

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

5.1.2 Raft Foundation

A raft foundation is a large structurally designed and reinforced concrete slab, typically constructed on a soft subgrade material, or where sites have a relatively shallow groundwater level. The slab is spread under the whole building footprint, which spreads the load over a wide footprint to the subgrade soils.

A preliminary founding depth of the basement level was assumed and taken as 3.0 m below grade. An excavation of 3.0 m of overburden material would result in unloading of the underlying soils by about 65 kPa. Therefore, the raft foundation can be designed using a maximum allowable bearing capacity of **155 kPa** for serviceability limit state **(SLS)** and **230 kPa** for ultimate limit state **(ULS)** factored bearing resistance.

These values are dependent on the slab being poured directly over a non-disturbed subgrade.

The differential settlements will depend on the stiffness between the slab and the subgrade. The deflections in the slab to be used in the structural design should be determined by a structural analysis using the modulus of subgrade reaction (k_s). For this site, the modulus of subgrade reaction may be taken as **24 MPa/m.**

5.2 Structural Fill

For foundations set over undisturbed native soil and where excavation below the underside of the footings is performed in order to reach a suitable founding stratum, consideration should also be given to support the footings on structural fill. The structural fill should be placed over undisturbed native soils in layers not exceeding 300 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.

5.3 Basement/Underground Parking Level Construction

An underground level made up of a combination of a basement and underground parking is being proposed for this site. All basement walls shall be damp-proofed as per Ontario Building Code Requirements.

For bedding and to serve as moisture barrier underneath the basement floor slab, a minimum of 300 mm thick layer of 19 mm clear stone should be placed. It is also recommended to place a 10 mil poly vapour barrier overlying the granular material, prior to placement of basement slab.

An under-floor drainage system with the invert located a minimum of 300 mm below the underside of basement slab is recommended to be installed. This shall be comprised of 100 mm diameter weeping tile pre-wrapped with geotextile knitted sock, embedded in a 300 mm surround layer of 19 mm clear stone.

Due to the fine-grained composition of the site’s underlying soils, the clear stone surround shall be wrapped in a geotextile fabric.

The drainage system shall be installed in one direction below the slab and connected to sump/frost-free outlet from which water is pumped to the nearby ditches or storm sewer line, if available.

5.4 Lateral Earth Pressure on Basement Wall

The following equation should be used to estimate the intensity of the lateral earth pressure against any earth retaining structure/foundation walls.

$$P = K(\gamma h + q)$$

Where;

P = Earth pressure at depth h;

K = Appropriate coefficient of earth pressure;

γ = Unit weight of compacted backfill, adjacent to the wall;

h = Depth (below adjacent to the highest grade) at which P is calculated;

q = Intensity of any surcharge distributed uniformly over the backfill surface (usually surcharge from traffic, equipment or soil stockpiled and typically considered 10 kPa).

The coefficient of earth pressure at rest (K_0) should be used in the calculation of the earth pressure on the storm water manhole/basement walls, which are expected to be rather rigid and not to deflect.

The above expression assumes that perimeter drainage system prevents the build-up of any hydrostatic pressure behind the foundation wall.

5.5 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.

5.6 Liquefaction Potential

For foundations constructed on a well graded glacial till, **liquefaction is not a concern.**

5.7 Seismic

Based on the results 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 as **Class "D"** as per the Site Classification for Seismic Site Response.

It should be noted that a greater seismic site response class may be obtained by conducting seismic velocity testing using a multichannel analysis of surface waves (MASW).

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**.**

5.8 Frost Protection

All exterior footings for any heated structure exposed to frost conditions should have a minimum of 1.5 m of earth cover. Footings for any unheated structures, signage or lighting, and where snow will be cleared, 1.8 m of earth cover is required. 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.

5.9 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 surround layer of 19 mm clear stone and set adjacent to the perimeter footings. The drainage pipe may be tied into the basement drainage system, and 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.

5.10 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 II or I, or a Select Subgrade Material (SSM).

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.

5.11 Slab-on-grade Construction

All organic or otherwise deleterious material shall be removed from the proposed building's footprint. The exposed subgrade should then be inspected and approved by a qualified geotechnical personnel.

Any underfloor fill needed to raise the general floor grade shall consist of OPSS Granular B Type II or I, SSM or approved on-site earth borrow, compacted to 98% of its SPMDD. A 200 mm Granular A meeting the **OPSS 1010** shall be placed underneath the slab and compacted to 98% 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.

It is also recommended that the area of extensive exterior slab-on-grade (sidewalks, ramp etc.) shall be constructed using 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 **24 MPa/m.**

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.

5.12 Retaining Walls and Shoring

Based on the subsurface soil conditions encountered on this site, the measured groundwater depths, and the depth of excavation; shoring is recommended to be installed in order to stabilize the excavation side walls.

It is recommended a shoring design/build company is retained to design the shoring system based on parameters contained in this report.

The following **Table 2** 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 2: Material Properties for Shoring and Permanent Wall Design (Static)

Type of Material	Bulk Density (kN/m^3)	Angle of internal friction	Pressure Coefficient			Combined static and seismic active earth pressure coefficient (K_{AE})
			At Rest (K_o)	Active (K_a)	Passive (K_p)	
Granular A	22.0	35	0.43	0.27	3.69	0.33
Granular B Type I	20.0	31	0.48	0.32	3.12	0.38
Granular B Type II	23.0	32	0.47	0.31	3.25	0.37
Till	23.0	30	0.50	0.33	3.00	0.39

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 Canadian Building Code recommends the use 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;

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

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

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 3** provides the parameters for seismic design of retaining structures

Table 3: 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/m^3)	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})	0.38	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

5.13 Corrosion Potential and Cement Type

Two (2) soil samples were submitted to Paracel Laboratories Ltd. for chemical testing. The following **Table 4** below summarizes the results.

Table 4: Results of Chemical Analysis

Sample Location	Depth (m)	pH	Sulphate (µg/g)	Chloride (µg/g)	Resistivity (Ohm.cm)
BH2	2.3 – 2.9	7.17	46	<10	6770
BH4	4.6 – 5.1	6.91	30	20	6410

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

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. Based on the above results, the soil resistivity falls within the moderate corrosive range.

5.14 Tree Planting

No sensitive marine clay soils were encountered onsite, trees being planted onsite do not have to follow the "Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines" document.

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation

It is anticipated that depth of excavation onsite will be +/- 3.0 m bgs. Excavation must be carried out in accordance with Occupational Health and Safety Act and Regulations for construction Projects.

According to the Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 213/91 and its amendments, the surficial overburden expected to be excavated into at this site can be classified as Type 3. Therefore, shallow temporary excavations 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.

Due to site constraints, sloping of excavation side walls will most likely not be possible. Therefore, as previously stated, shoring is recommended for this site.

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 soils into the temporary excavations during construction is expected. However, it is anticipated that pumping from open sumps should be sufficient to control groundwater inflow. 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 if possible.

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, rate of excavation, the size of excavation, depth below the groundwater level, and at the time of year which the excavation is executed. It is expected that pumping rates may exceed 50,000 litres per day. As such, EASR registration may be required for the construction at this site.

This could be confirmed by undertaking a Hydrogeological Study to better understand the infiltration rates of the site's underlying soils.

6.3 Pipe Bedding Requirements

It is anticipated that the subgrade material for any underground services required as part of this project will be founded over the glacial till material. Any sub-excavation of disturbed soil should be removed and replaced with a Granular A, Granular B Type II or I or approved equivalent, laid in loose lifts of thickness not exceeding 300 mm and compacted to 95% of its SPMDD. Bedding, thickness of cover material and compaction requirements for any 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. At minimum, a 150 mm thick layer of Granular A shall be used as pipe bedding, at the springline of the pipe, and a 300 mm thick layer above the obvert of the pipe.

If sewers are required to be founded below the groundwater table the native materials may be sensitive to disturbances. 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 (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 any sewers and structures.

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 300 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 II or I. 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 are 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 glacial till. This material is considered to be frost susceptible and should not be used as backfill material, except for landscaping purposes where no loads will be applied.

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.

Any imported material shall conform to OPSS Granular B – Type II or I, SSM, or an approved equivalent.

8 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.

9 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 ensure 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
B. W. JOHNSON
100510537
2024.05.22
PROVINCE OF ONTARIO

Brad Johnson, P.Eng.
Geotechnical Engineer

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APPENDIX A

Site and Borehole Location Plan

PROJECT

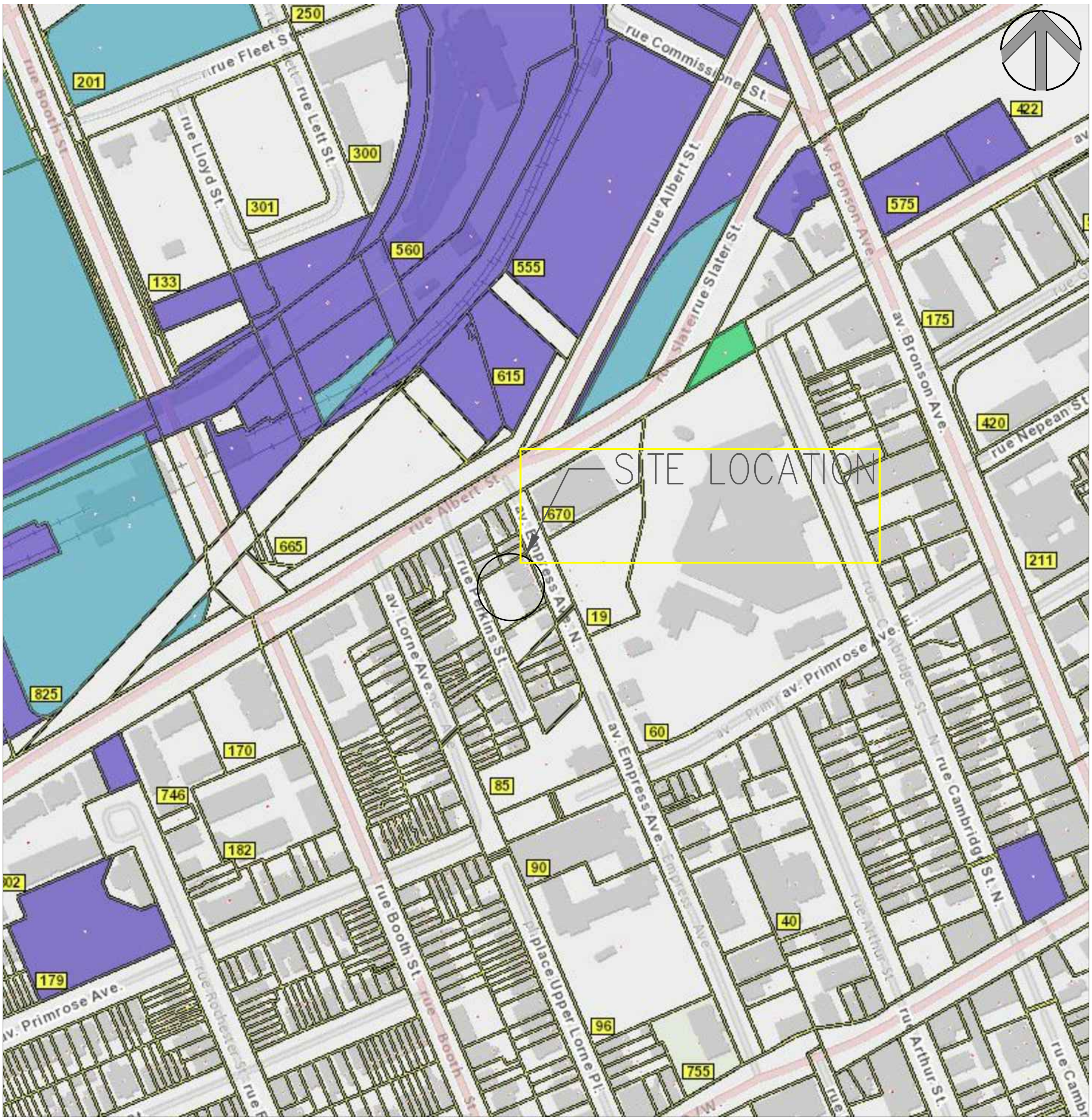
GEOTECHNICAL INVESTIGATION
PROPOSED 4-STOREY LOW-RISE APARTMENT
10 EMPRESS AVE NORTH
OTTAWA ONTARIO

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

DRAWING TITLE

SITE LOCATION
SOURCE: GEOOTTAWA

CLIENT	DATE	PROJECT	
DALHOUSIE NON-PROFIT COOPERATIVE INC.	MAY 2024	240202	FIGURE 1



LRL

ENGINEERING | INGÉNIERIE

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

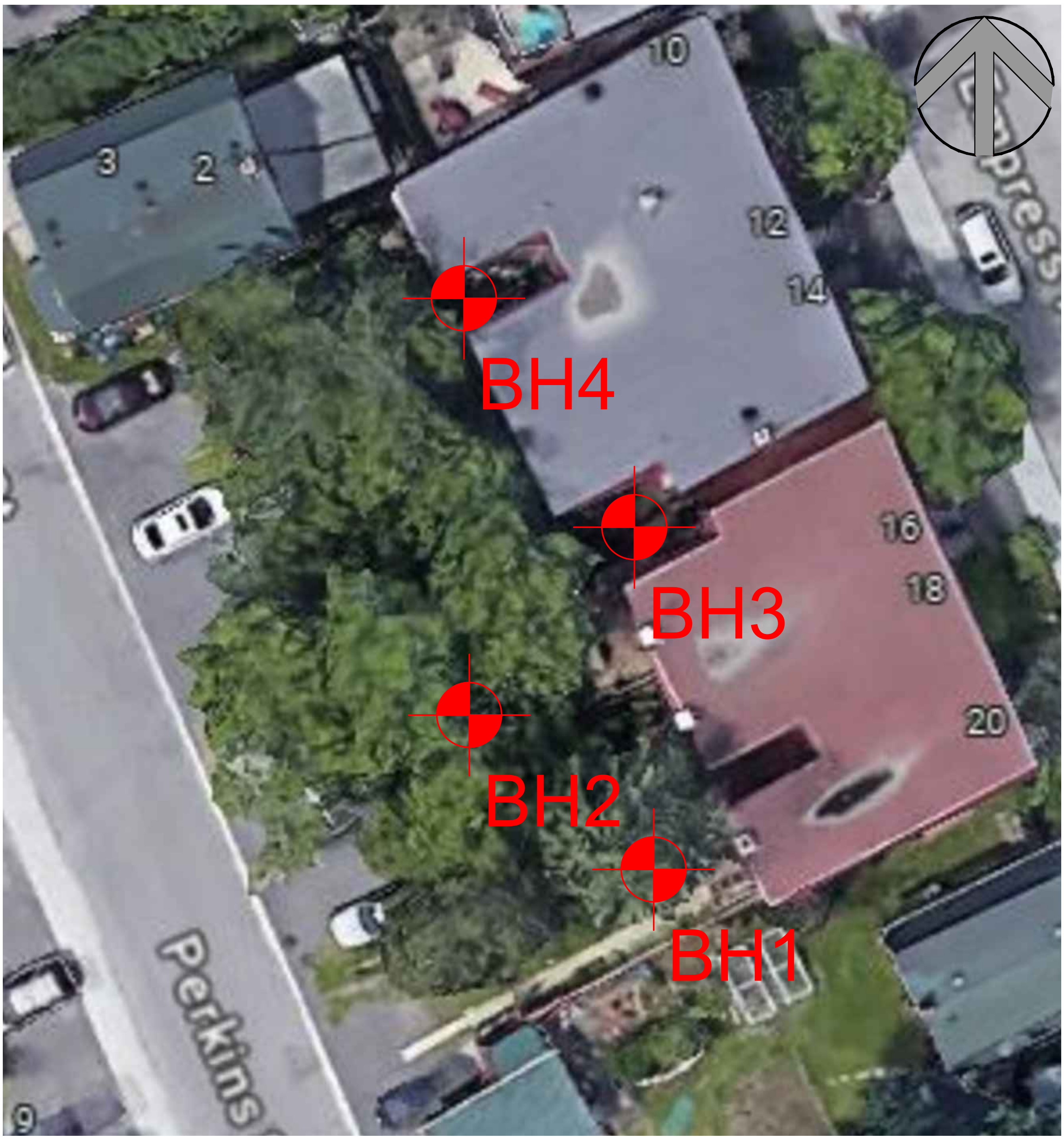
PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED 4-STOREY LOW-RISE APARTMENT
10 EMPRESS AVE NORTH
OTTAWA ONTARIO

DRAWING TITLE

BOREHOLE LOCATION
SOURCE: GOOGLE AERIAL VIEW

CLIENT	DATE	PROJECT	
DALHOUSIE NON-PROFIT COOPERATIVE INC.	MAY 2024	240202	FIGURE 2



APPENDIX B
Borehole Logs

Borehole Log: BH-1

Project No.: 240202
Project: GEO Investigation - Proposed 4-Storey Apartment
Client: Dalhousie Non-Profit Cooperative Inc.
Location: 10 Empress Ave. N, Ottawa ON
Date: May 3, 2024
Field Personnel: BJ

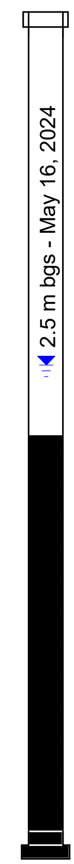
Driller: George Downing Estate Drilling. **Drilling Equipment:** Geoprobe 7822DT **Drilling Method:** Hollow Stem Auger

SUBSURFACE PROFILE / SAMPLE DATA

Depth (m)	Soil Description	Elev./Depth(m)	Type	Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)
0	Ground Surface	63.35 / 0.00						
0.0–0.2	**TOPSOIL** About 200 mm thick		SS	SS1	2	50	2	18
0.2–1.68	**FILL MATERIAL** silt-sand-clay, dark brown, moist, very loose to loose.		SS	SS2	5	67	5	21
1.68	**GLACIAL TILL** silt-sand, trace clay, trace gravel sized stone, brown, moist, loose to very dense.	61.67 / 1.68	SS	SS3	21	67	21	12
			SS	SS4	50+	50	50+	11
~3.0	-becomes wet and grey at about 3.0 m bgs.		SS	SS5	14	100	14	9
			SS	SS6	12	50	12	10
			SS	SS7	7	50	7	11
			SS	SS8	7	100	7	11
8.23	End of Borehole	55.12 / 8.23						

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

Water Level (Standpipe or Open Borehole)



Easting: 444398 **Northing:** 5029068

Site Datum: Site Benchmark - Nail in Utility Pole: 63.43 m

Groundsurface Elevation: 62.79 m **Top of Riser Elev.:** 63.35 m

Hole Diameter: 200mm

NOTES:

Borehole Log: BH-2

Project No.: 240202
Project: GEO Investigation - Proposed 4-Storey Apartment
Client: Dalhousie Non-Profit Cooperative Inc.
Location: 10 Empress Ave. N, Ottawa ON
Date: May 3, 2024
Field Personnel: BJ
Driller: George Downing Estate Drilling.
Drilling Equipment: Geoprobe 7822DT
Drilling Method: Hollow Stem Auger

SUBSURFACE PROFILE

Depth	Soil Description	Elev./Depth(m)
0	Ground Surface	62.89
0	**TOPSOIL** About 75 mm thick.	0.00
	FILL MATERIAL silt-sand-clay, some gravel sized stone, some organics, dark brown, moist, very loose to loose.	61.44 / 1.45
	GLACIAL TILL silt-sand, trace clay, some gravel sized stone, brown, moist, loose to very dense.	
	-becomes wet and grey at about 2.7 m bgs.	
	Start of Dynamic Cone Penetration (DCP) Test	

SAMPLE DATA

Sample Number	N or RQD	Recovery (%)	SPT N Value	Water Content (%)
SS1	2	13	2	10
SS2	5	50	5	17
SS3	54	42	54	10
SS4	15	50	15	9
SS5	7	67	7	9
SS6	7	50	7	11
SS7	10	50	10	8

DCP values: 3, 11, 8, 11, 8, 6

Shear Strength (kPa): 50, 150
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)

Easting: 444386
Northing: 5029102
Site Datum: Site Benchmark - Nail in Utility Pole: 63.43 m
Groundsurface Elevation: 62.89 m
Top of Riser Elev.: NA
Hole Diameter: 200mm

NOTES:

Borehole Log (continued): BH2

Project No.: 240202

Project: GEO Investigation - Proposed 4-Storey Apartment

Client: Dalhousie Non-Profit Cooperative Inc.

Location: 10 Empress Ave. N, Ottawa ON

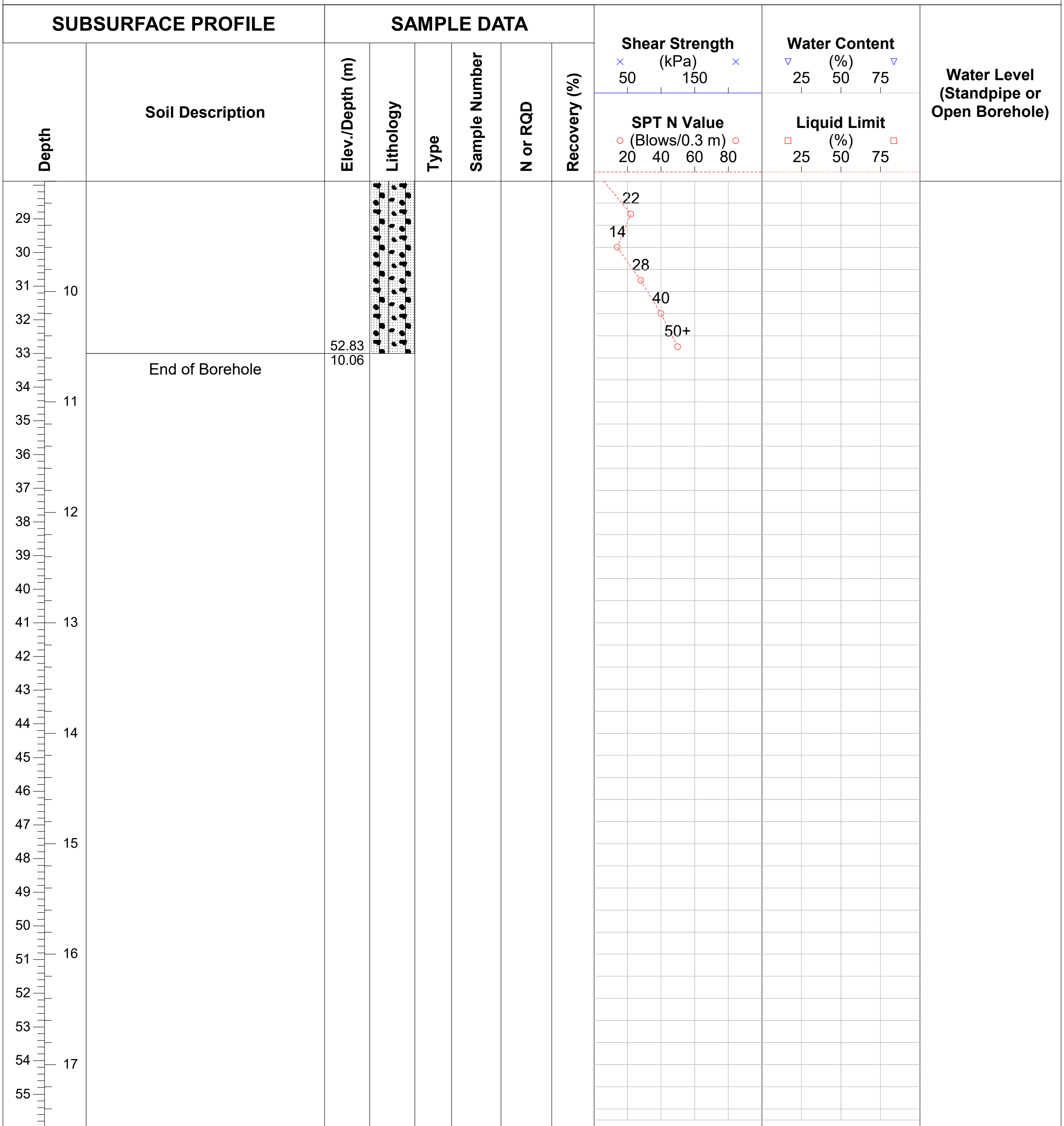
Date: May 3, 2024

Field Personnel: BJ

Driller: George Downing Estate Drilling.

Drilling Equipment: Geoprobe 7822DT

Drilling Method: Hollow Stem Auger



NOTES

Borehole Log: BH-3

Project No.: 240202

Project: GEO Investigation - Proposed 4-Storey Apartment

Client: Dalhousie Non-Profit Cooperative Inc.

Location: 10 Empress Ave. N, Ottawa ON

Date: May 3, 2024

Field Personnel: BJ

Driller: George Downing Estate Drilling. **Drilling Equipment:** Geoprobe 7822DT **Drilling Method:** Hollow Stem Auger

SUBSURFACE PROFILE

Elev./Depth (m)	Soil Description
63.45	Ground Surface
0.00	**TOPSOIL** About 150 mm thick
	FILL MATERIAL silt-sand-clay, some gravel, brown, moist, very loose.
62.25 / 1.20	**GLACIAL TILL** silt-sand, trace clay, some gravel sized stone, brown, moist, loose. -becomes wet and grey at about 2.7 m bgs.
55.22 / 8.23	End of Borehole

SAMPLE DATA

Sample Number	Type	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)
SS1	Split spoon	2	50	2	31
SS2	Split spoon	3	42	3	14
SS3	Split spoon	7	50	7	20
SS4	Split spoon	6	75	6	10
SS5	Split spoon	10	42	10	11
SS6	Split spoon	3	100	3	11
SS7	Split spoon	7	50	7	11
SS8	Split spoon	11	25	11	11

Shear Strength (kPa): 50, 150

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)

2.5 m bgs - May 16, 2024

Easting: 444394 **Northing:** 5029084

Site Datum: Site Benchmark - Nail in Utility Pole: 63.43 m

Groundsurface Elevation: 62.81 m **Top of Riser Elev.:** 63.42 m

Hole Diameter: 200mm

NOTES:

Borehole Log: BH-4

Project No.: 240202
Project: GEO Investigation - Proposed 4-Storey Apartment
Client: Dalhousie Non-Profit Cooperative Inc.
Location: 10 Empress Ave. N, Ottawa ON
Date: May 3, 2024
Field Personnel: BJ

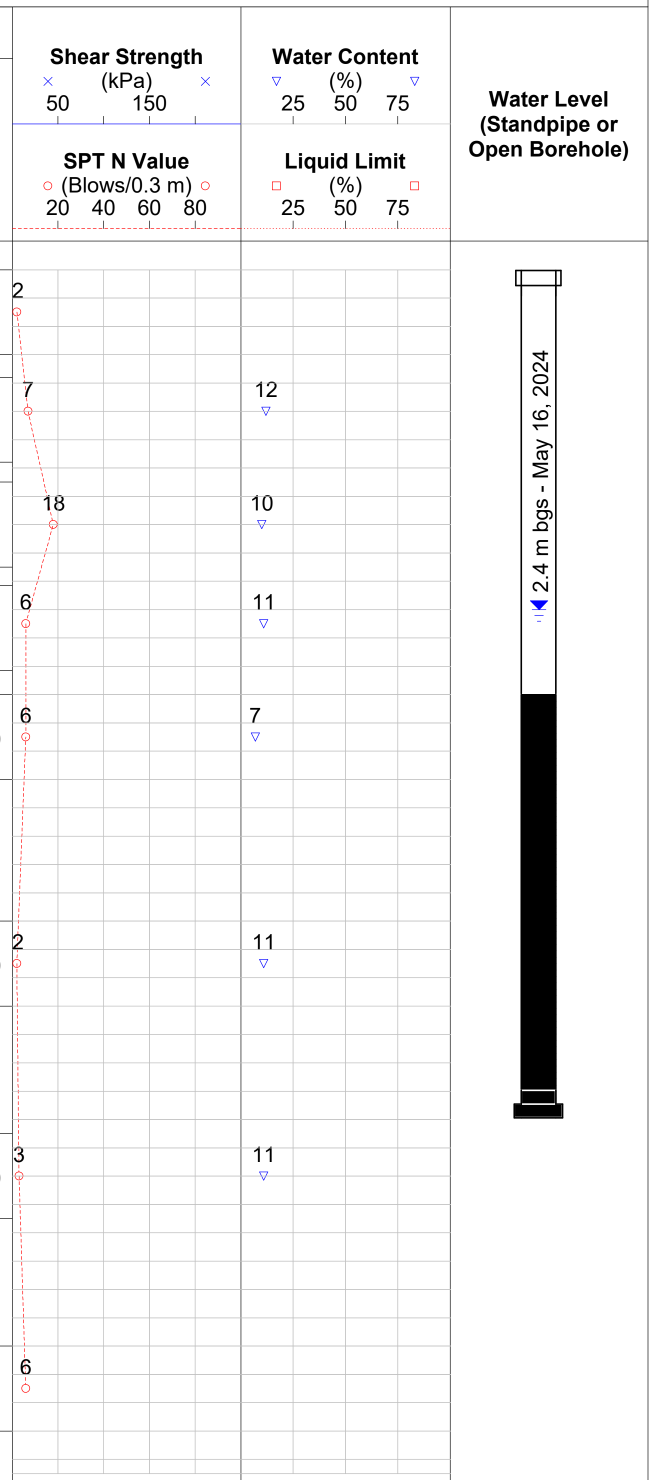
Driller: George Downing Estate Drilling. **Drilling Equipment:** Geoprobe 7822DT **Drilling Method:** Hollow Stem Auger

SUBSURFACE PROFILE

Depth (m)	Soil Description	Elev./Depth(m)
0	Ground Surface	62.88
0.00 – 1.20	**FILL MATERIAL** silt-sand, brown, moist, very loose to loose.	62.88 / 0.00
1.20 – 8.23	**GLACIAL TILL** silt-sand, trace clay, some gravel sized stone, brown, moist, loose to compact. -becomes wet and grey at about 2.7 m bgs.	61.68 / 1.20
8.23	End of Borehole	54.65 / 8.23

SAMPLE DATA

Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)
SS1	2	0	2	
SS2	7	60	7	12
SS3	18	60	18	10
SS4	6	75	6	11
SS5	6	100	6	7
SS6	2	100	2	11
SS7	3	100	3	11
SS8	6	0	6	



Easting: 444386 **Northing:** 5029096

Site Datum: Site Benchmark - Nail in Utility Pole: 63.43 m

Groundsurface Elevation: 62.88 m **Top of Riser Elev.:** 63.45 m

Hole Diameter: 200mm

NOTES:

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

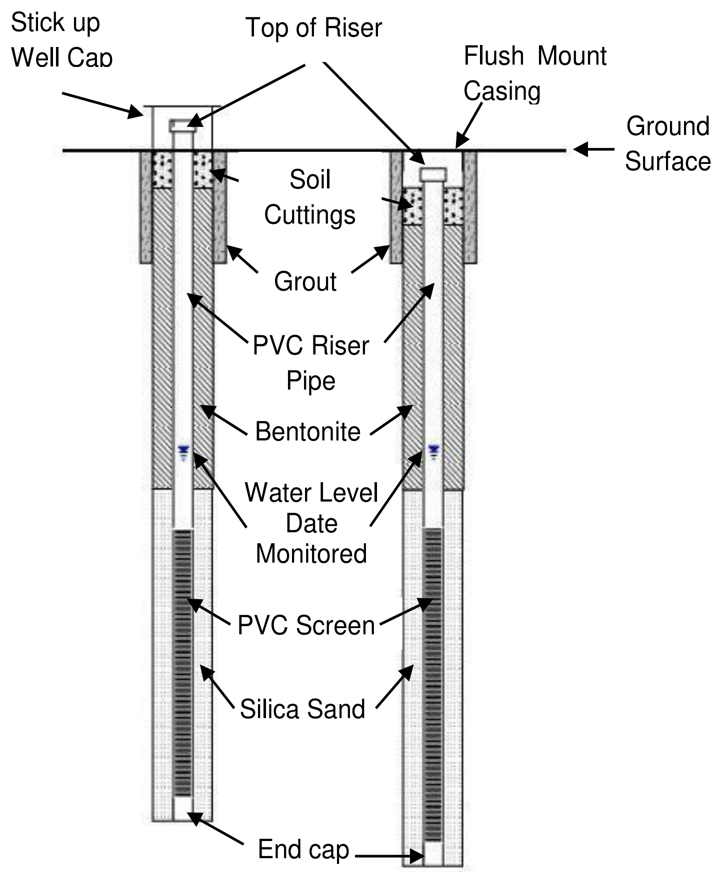
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given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

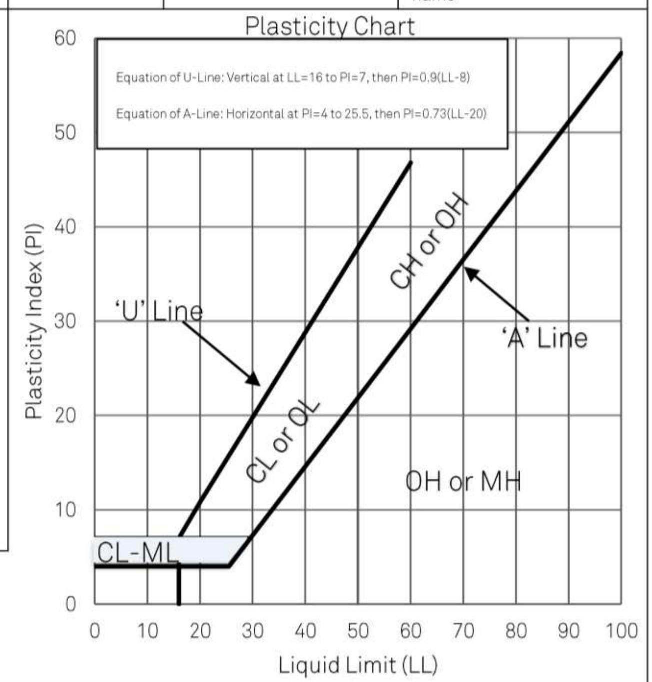
Strength Classification	Range of Unconfined Compressive Strength (MPa)
Extremely weak	< 1
Very weak	1 – 5
Weak	5 – 25
Medium strong	25 – 50
Strong	50 – 100
Very strong	100 – 250
Extremely strong	> 250

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
			GC	Clayey gravel			Atterberg limits on or above "A" line and PI > 7	If fines are organic add "with orgnic fines" to group name
	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
			SC	Clayey sand			Atterberg limits on or above "A" line and PI > 7	If fines are organic add "with orgnic fines" to group name
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.	Plasticity Chart		
			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

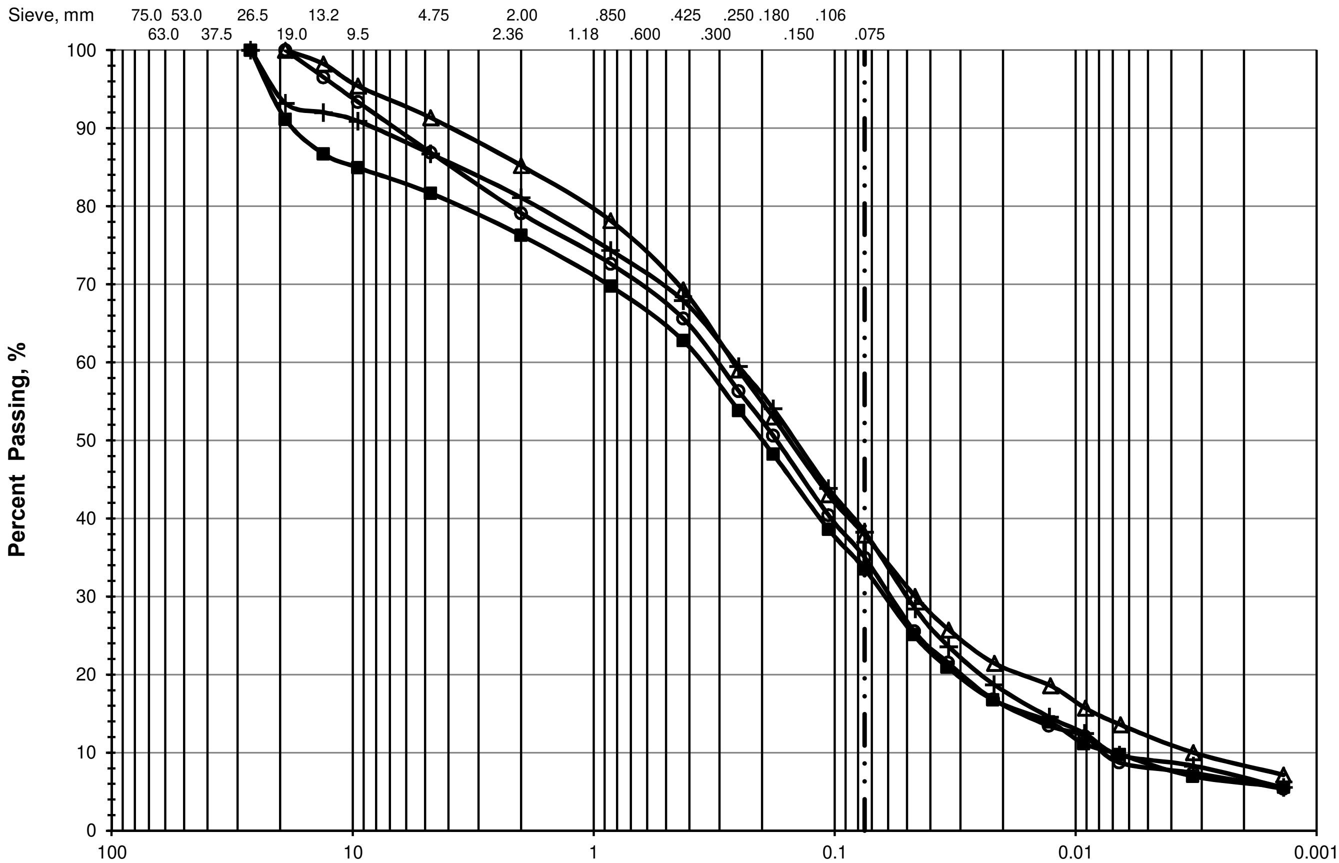
Laboratory Results

LRL Associates Ltd.

Particle Size Analysis

ASTM D 422 / LS-702

Client:	Dalhousie Non-Profit Cooperative Inc.	**File No.:**	240202
	Geotechnical Investigation	**Report No.:**	1
	210 Empress Ave. N, Ottawa, ON.	**Date:**	May 3, 2024



Unified Soil Classification System

	> 75 mm	% GRAVEL		% SAND			% FINES	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
△	0.0	0.0	8.7	6.1	15.9	31.4	29.8	8.1
■	0.0	7.7	10.7	5.4	13.5	29.1	27.5	6.1
○	0.0	0.0	13.1	7.8	13.5	30.6	28.9	6.1
+	0.0	5.9	7.4	5.6	13.2	29.6	31.8	6.5

	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 1	SS-3	1.52 - 2.13	0.2669	0.1578	0.0463	0.0082	0.0032	2.5	83.4
■	BH 2	SS-5	3.05 - 3.66	0.3705	0.2019	0.0633	0.0162	0.0068	1.6	54.5
○	BH 3	SS-6	4.57 - 5.18	0.3192	0.1754	0.0600	0.0171	0.0076	1.5	42.0
+	BH 4	SS-7	6.10 - 6.71	0.2610	0.1505	0.0508	0.0138	0.0069	1.4	37.8