Geotechnical Investigation

Proposed Apartment Building
161 Hinchey Avenue
Ottawa, Ontario

Prepared for:

Praveen Muppalla
450 Creekview Way
Ottawa, Ontario
K1T 0J5

LRL File No.: 200295

July, 2020

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1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by Fotenn on behalf of Praveen Muppalla to perform a geotechnical investigation for a proposed four (4) storey apartment building, located at 161 Hinchey Avenue, 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 currently encompasses a two (2) storey single family home. The site has about 15 m of frontage along Hinchey Avenue, and has a total surface area of about 460 m^2. The site is made up of the single family home, an asphalt driveway, manicured grasses, and some trees around the north, east, and west perimeter of the site. The topography of the site is considered to be relatively flat. Site access comes by way of Hinchey Avenue, and is civically located at 161 Hinchey Avenue, Ottawa ON. The location is presented in Figure 1 included in **Appendix A**.

This development is an infill project; the existing single family home will be demolished. A new four (4) storey apartment building complete with a basement will be constructed. The building will have a footprint of approximately 230 m^2. The apartment building will be serviced with municipal services.

3 PROCEDURE

The fieldwork for this investigation was carried out on July 13, 2020. Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of three (3) boreholes were drilled onsite where possible to do so, and labelled BH1 through BH3. The approximate locations of the boreholes are shown in Figure 2 included in **Appendix A**.

The boreholes were advanced using a truck mounted CME 75 drill rig equipped with 200 mm diameter continuous flight hollow stem auger supplied and operated by CCC Geotechnical and Environmental 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.

All boreholes were advanced until practical auger refusal over bedrock, two (2) of the boreholes consisted of NQ-size (Ø47.6mm) rock coring. The boreholes were terminated at depths ranging from 0.8 to 4.3 m below ground surface (bgs). Upon completion, the boreholes were backfilled and compacted using a combination of bentonite, overburden cuttings, and topped with asphalt cold patch.

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 were transported back to our office for further evaluation. 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 a temporary site bench mark (center of catch basin lid on sidewalk in front of 162 Hinchey Avenue), given an elevation of 100.00 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 bedrock, consisting of limestone with shaly partings.

The subsurface conditions encountered in the boreholes were classified based on visual and tactile examination of the materials recovered from the boreholes. 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 are given in their respective borehole 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 Asphalt

Asphalt having a thickness of 40 mm was present at the surface of all boring locations.

4.3 Fill

Underlying the asphalt in all boring locations, a layer of fill material was encountered, and extended to depths ranging between 0.60 and 1.20 m bgs. In BH1 and BH2, the fill can generally be described as a mixture of silt and sand, trace gravel, moist, and brown in colour. In BH3, the fill can be described as gravelly sand, some silt, moist, and brown in colour. Standard Penetration Tests (SPT) were carried out in this layer, and the SPT “N” values were found ranging between 14 and 59. However, the ”N” values for SS2 in BH1 and BH3 indicated bedrock was encountered, and does not demonstrate the state of compactness, and the fill material is considered to be in a compact state. The natural moisture contents were found to range between 5 and 10%.

Two (2) samples were collected for laboratory sieve analyses. The results are summarized below in **Table 1.**

Table 1: Sieve Analysis Summary

Sample Location	Depth (m)	Percent for Each Soil Gradation					
		Gravel		Sand			Fines
		Coarse (%)	Fine (%)	Coarse (%)	Medium (%)	Fine (%)	Silt & Clay (%)
BH1	0.0 – 0.6	0.0	8.2	7.3	18.6	35.5	30.4
BH3	0.0 – 0.6	0.0	32.2	13.3	17.4	19.1	18.3

4.4 Bedrock

Underlying the fill material in all boring locations, bedrock was encountered. The bedrock was encountered at very shallow depths, ranging from pavement structure in BH1, the topsoil in BH2, and the fill in BH3 – BH5, refusal over bedrock encountered. The bedrock was encountered at shallow depths, ranging from 0.6 – 1.2 m bgs.

The bedrock formation in this area can be described as consisting of limestone, with shaly partings, and grey to dark grey in colour.

The Rock Quality Designation (RQD) was determined after the rock was cored, this is done by summing the lengths of the intact recovered cores which are greater than 100 mm in length, and dividing by the total length of the core run. The RQD values, expressed as a percent, ranged from 42 to 100%, indicating the rock was poor to excellent quality, with the majority of the values being in the good to excellent range.

Four (4) rock core samples were selected to determine the unconfined compressive strengths at various depths. The results are summarized below in **Table 2.**

Table 2: Unconfined Compressive Strength of Select Rock Cores

Sample			Bedrock Type	Strength (MPa)
Borehole	Core ID	Depth (m)		
BH1	Run 2	1.57 – 1.68	Limestone	113.7
BH1	Run 3	2.82 – 2.92	Limestone	72.7
BH3	Run 1	1.40 – 1.50	Limestone	93.4
BH3	Run 2	3.73 – 3.84	Limestone	94.3

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

4.5 Groundwater Conditions

Groundwater conditions were carefully monitored during the field investigation. During drilling within the overburden material no water was encountered, and it is believed the groundwater table is found at deeper depths, within the bedrock.

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

5.1 Foundations

Based on the subsurface soil conditions established at this site, it is recommended that the footings for the proposed apartment building be founded over bedrock, or structural fill overlying the bedrock. Therefore, all material, and any construction debris from the existing home should be removed from the proposed footprint down to the required founding depth.

5.2 Shallow Foundation on Bedrock

Conventional strip and column footings set over sound bedrock may be designed using a maximum allowable bearing pressure of **750 kPa** for Ultimate Limit State **(ULS)** factored bearing resistance. Serviceability Limit State **(SLS)** does not apply for footings founded on bedrock since failure of the concrete would occur before unacceptable settlement of the foundation. For footings founded on sound bedrock, there are no restrictions for maximum footing sizes and grade raise fill thickness. Prior to pouring the footing, the rock should be free of any soil, debris or deleterious substances and should be inspected by a geotechnical engineer.

Considering there is a potential for the bedrock to consist of shale, it is recommended that if shale bedrock is encountered, a 50 mm thick mud slab (consisting of 10 MPa lean concrete) be poured within 24 hours of uncovering the bedrock surface. Requirements of a mud slab can be decided after conducting an inspection of the exposed bedrock surface by a qualified geotechnical engineer during construction.

The footing for the building must rest entirely over bedrock and not two (2) different founding strata (e.g., bedrock or structural fill) in order to limit differential settlements.

The footings should be constructed on a relatively flat bedrock surface (10 degrees or less from the horizontal). If the footings will be founded on bedrock that is sloped greater than 10 degrees, and less than 30 degrees, rock anchors should be considered. For angles greater than 30 degrees, the bedrock must be levelled and step footings should be constructed.

Any excavations below the underside of footing for the proposed building to be founded on bedrock should be backfilled using lean concrete only, having a minimum compressive strength of 10 MPa at 28 days.

5.2.1 Rock Anchors

If the need for rock anchors is required, they should be designed by a structural engineer. The engineer will design the rock anchors based on the type of bedrock and strength parameters.

Grouted rock anchor may fail in one or more of the following modes:

- Failure within the rock mass;
- Failure of the rock/grout bond;
- Failure of the grout/tendon bond; or
- Failure of the steel tendon, or top anchorage.

The capacity of rock anchors is dependent on the bond between the rock and grout. The method of installation will also affect the capacity of the bond between the rock and the grout. An invert cone angle of 90º may be used in the design of the anchors. Pull out testing should be carried out on the anchors to verify installations and to design load capacities. If bedrock is removed through mechanical hydraulic hammers (i.e. for levelling and installation of anchors), it is not expected to affect the contribution of the upper level of rock in the calculation of anchors capacity.

The bond length (grouted portion of the dowel) should be a minimum of 3.0 m. Generally, the bond between the grout and dowel are twice the bond developed between the grout and the bedrock. Therefore, the design should be based on failure between the grout and the bedrock.

Straight-shafted dowels anchor force is dependent on the ultimate bond stress of the bedrock or the grout. Typically, the ultimate bond force is taken as 10% of the average unconfined compressive strength of the bedrock, or the compressive strength of the grout, whichever is less (but not more than 3.1 MPa). The allowable bond stress is taken as 50% of the ultimate bond stress.

The required bond length can be determined using the following equation:

$$L(m) = P/(\pi \times d \times T_b)$$

Where;

P = Working Capacity of anchor (kg);

T_b = working bond stress (kg/m^2);

d = Core hole diameter (m).

5.3 Shallow Foundation on Structural Fill

Conventional strip and column footings set over properly compacted and approved structural fill conforming to OPSS Granular B Type II or approved equivalent may be designed for a maximum allowable bearing pressure of **150** kPa for Serviceability Limit State (SLS) and **225** kPa for Ultimate Limit State (ULS) factored bearing resistance. The factored ULS value includes the geotechnical resistance factor of 0.5. For footings founded on properly compacted structural fill (having a minimum thickness 300 mm) overlying bedrock, there are no restrictions for maximum footing sizes and grade raise fill thickness.

Prior to placing the approved structural fill, the subgrade at bedrock level should be inspected and assessed by a geotechnical engineer, or a representative to identify any localised incompetent/unstable areas of the subgrade. Any incompetent subgrade areas as identified must be sub-excavated and backfilled with approved structural fill and compacted to 98% of its SPMDD. 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.

5.4 Bedrock Excavation

It is expected that some bedrock excavation will be required as part of this development. It is anticipated that bedrock removal will be possible with the use of heavy excavation equipment, but that removal of most of the bedrock could be facilitated by means of a hoe ramming operation. Both horizontal and vertical overbreak of the bedrock excavation face/bottom can be expected due to the hoe ramming operation. If control of potential bedrock overbreak is required, line drilling at the proposed excavation face is recommended. The smaller the distance between the drill holes, the fewer overbreaks is expected. It is generally considered that the drilling at 150 mm horizontal spacing to the full depth of the excavation should control overbreak to an acceptable level. Considering the proximity of the existing structures adjacent to the site and the potential for vibration during excavating and removal of the bedrock, monitoring of the hoe ramming shall be carried out throughout the operation on nearby buildings to ensure that the vibration limit is not exceeded. As outlined in **OPSS 120, Table 3** below summarizes the following vibration limits for the nearest existing structures.

In addition, a pre and post construction excavation condition survey of nearby structures is required to be carried out.

Table 3: Vibration Frequency and Limit

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

5.5 Lateral Earth Pressure

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.

Table 4 below provides various material types and their respective earth pressure properties.

Table 4: Material and Earth Pressure Properties

Type of Material	Bulk Density (kN/m^3)	Friction Angle (Φ)	Pressure Coefficient		
			At Rest (K_0)	Active (K_A)	Passive (K_P)
Granular A	23.0	34	0.44	0.28	3.53
Granular B Type I	20.0	31	0.49	0.32	3.12
Granular B Type II	23.0	32	0.47	0.31	3.25

5.6 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.

5.7 Liquefaction

For buildings founded over bedrock or structural fill, the potential of soil liquefaction is not considered to be a concern.

5.8 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 “C” 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.

5.9 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.

5.10 Foundation Walls Backfill

To prevent possible 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, II or 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

Concrete slab-on-grade should rest over compacted, free draining and well graded structural fill only. Therefore, all overburden soils should be removed from the proposed building's footprint down to the bedrock surface. The exposed undisturbed bedrock should then be inspected and approved by qualified geotechnical personnel.

Any underfloor fill needed to raise the general floor grade shall consist of OPSS Granular B Type I, II, or SSM, compacted to 95% of its SPMDD. The final lift shall Granular B Type II, and 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.

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. The modulus of subgrade reaction (ks) for the design of the slabs set over structural fill is **18 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.

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.

5.12 Corrosion Potential and Cement Type

A soil sample was submitted to Paracel Laboratories Ltd. for chemical testing. The following **Table 5** below summarizes the results.

Table 5: Results of Chemical Analysis

Sample Location	Depth (m)	pH	Sulphate (µg/g)	Chloride (µg/g)	Resistivity (Ohm.cm)
BH3	0.8 – 1.4	7.4	423	90	1540

The above results revealed a measured sulphate concentration of 423 µg/g in the sample. 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 results from soil samples were below the noted threshold. As such, buried concrete for footings and foundations walls will not require any special additive to resist sulphate attack and the use of normal Portland cement is acceptable.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil resistivity was measured to be 1540 ohm.cm, which falls between severe range for soil resistivity (0 – 2000 Ohm.cm) indicating a corrosive environment.

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation

It is anticipated that the depth of excavation for the building or any proposed services will not extend below 2.4 m. Excavation must be carried-out in accordance with the 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 for fully drained excavations. Therefore, shallow temporary excavations in the overburden soil can be cut at 1 horizontal to 1 vertical, for a fully drained excavation starting from the base of the excavation and as per requirements of the OHSA regulations. When excavating into bedrock, the side of the excavation does not need to be sloped, and can be cut vertically from the base of excavation.

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 into the temporary excavations during construction is expected to be minor in nature, if any. This will be able to be controlled by pumping with sump pumps. Surface water runoff into the excavation should be minimized and diverted away from the excavation.

A permit to take water (PTTW) is required from Ministry of Environment and Climate Change (MOECC), Ontario Reg. 387/04, if more than 400,000 litres per day of groundwater will be pumped during a construction period less than 30 days. Registration in the Environmental Activity and Sector Registry (EASR) is required when water takings 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 will be less than 50,000 litres per day. As such, EASR registration is not required for the construction at this site.

6.3 Pipe Bedding Requirements

It is anticipated that any underground services required as part of this project will be founded over properly prepared and approved structural fill. Consequently all organic material should be removed down to a suitable bearing layer. Any sub-excavation of disturbed soil should be removed and replaced with a Granular B Type II or I, or an 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 watermains and sewer pipes should conform to the manufacturer's design requirements and to the detailed installations outlined in the Ontario Provincial Standard Specifications (OPSS) or any other applicable standards.

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. Any boulders larger than 150 mm in size should not be used as trench backfill.

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

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

7 Reuse of On-Site Soils

The existing surficial overburden materials consists of fill material. This material is considered to be frost susceptible and should not be used as backfill material directly against foundation walls or underneath unheated concrete slabs. However, it could be reused as general backfill material (service trenches, general landscaping/backfilling) if it can be compacted according to the specifications outlined herein at the time of construction and found free from any waste, organics and debris.

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, and approved for reuse by a geotechnical engineer.

8 PAVEMENT REINSTATEMENT

There are no access roads or municipal streets proposed to be constructed as part of this project. However, there may be some street reinstatement from connecting to the municipal services.

The reinstatement of any pavement structure within the existing street should be conducted as recommended in **Section 6.4** and the pavement structure should be reinstated to match at minimum what already exists.

Where the existing asphaltic concrete surface of a roadway is affected by the excavating process, the damaged zones should be saw cut and any damaged or loose pieces of asphaltic concrete should be removed down to the binder course or its entire depth, where only one layer exist. The existing base should be scarified and proof-rolled with any soft areas excavated and replaced to the proper level with OPSS Granular A. Where two layers of asphalt exist on an access lane, the surface course should be grinded over a width of 150mm to allow the new surface course to overlap the binder layer and not create one straight vertical joint. On existing streets, the overlap should be increased to 300mm.

9 INSPECTION SERVICES

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

All footing areas and any structural fill areas for the proposed building 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 required gradation and compaction specifications.

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

10 REPORT CONDITIONS AND LIMITATIONS

It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document or its use by a third party beyond the client specifically listed in the report is neither intended nor authorized by LRL Associates Ltd. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

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

The recommendations provided in this report are based on subsurface data obtained at the specific test pit locations only. Boundaries between zones presented on the 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 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.

Brad Johnson, P. Eng.
Geotechnical Engineer

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APPENDIX A

Site and Borehole Location Plan

ENGINEERING | INGÉNIERIE

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

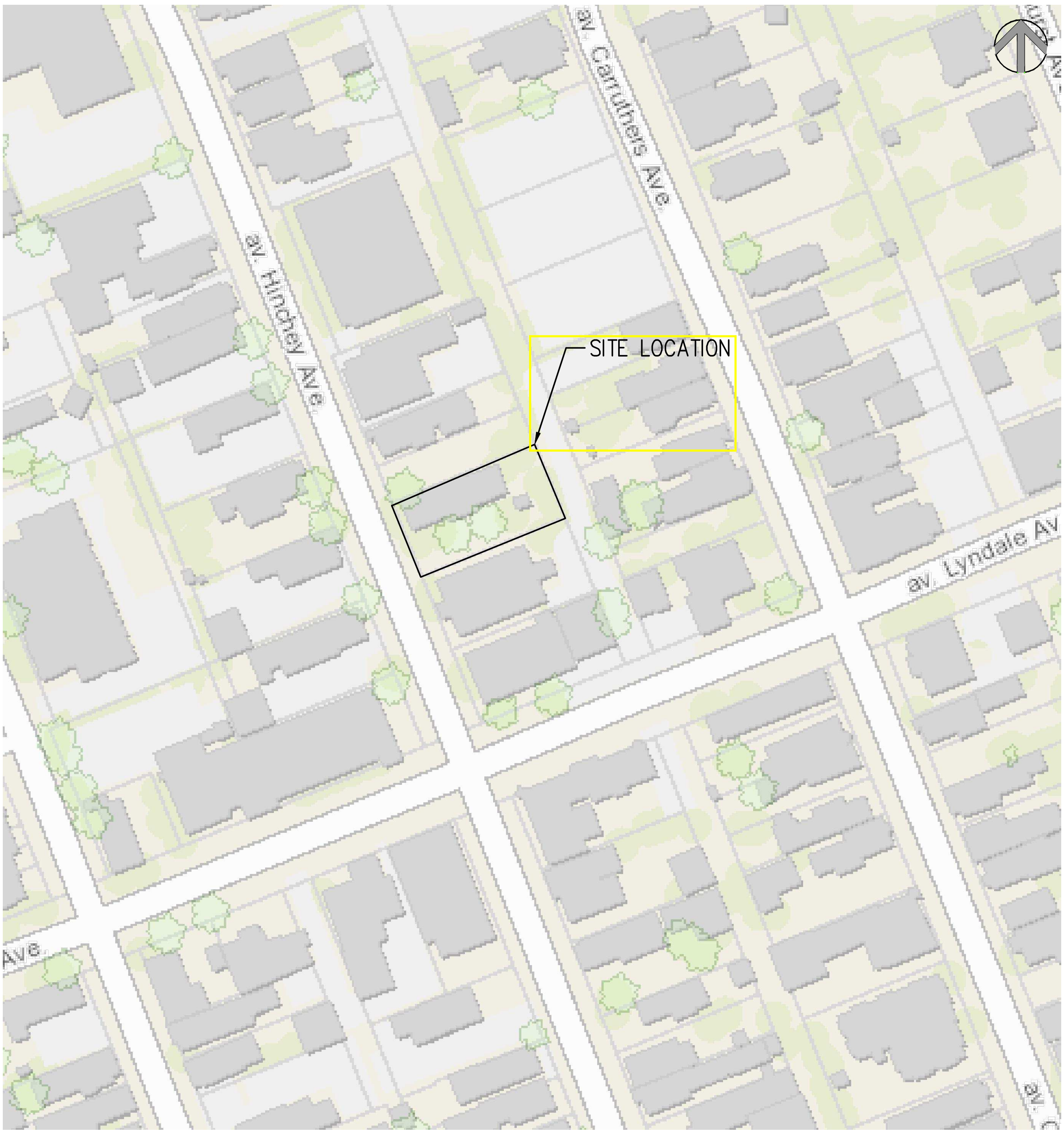
PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED FOUR STOREY APARTMENT BUILDING
161 HINCHEY AVE
OTTAWA, ONTARIO

DRAWING TITLE

SITE LOCATION
SOURCE: GEO-OTTAWA

CLIENT	DATE	PROJECT	
PRAVEEN MUPPALLA	AUGUST 2020	200295	FIGURE 1



ENGINEERING | INGÉNIERIE

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

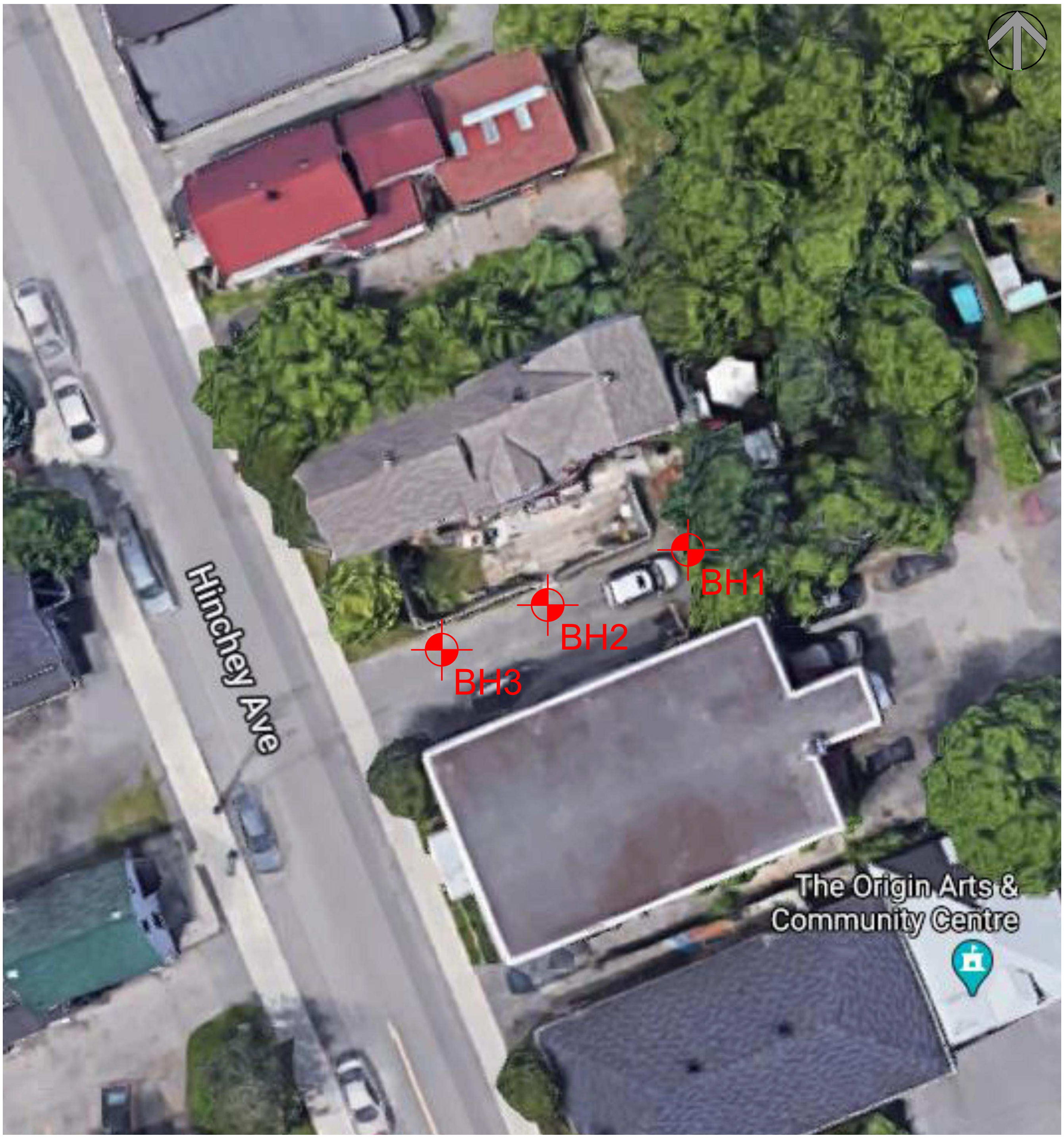
PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED FOUR STOREY APARTMENT BUILDING
161 HINCHEY AVE
OTTAWA, ONTARIO

DRAWING TITLE

BOREHOLE LOCATION
SOURCE: Imagery 2020 Google, Digital Globe Map Data

CLIENT	DATE	PROJECT	
PRAVEEN MUPPALLA	AUGUST 2020	200295	FIGURE 2



APPENDIX B

Borehole Logs

Borehole Log: BH1

Project No.: 200295

Client: Praveen Muppalla

Date: July 13, 2020

Project: Proposed Four Storey Apartment Building

Location: 161 Hinchey Ave, Ottawa ON

Field Personnel: BJ

Driller: CCC Geotech and Enviro Drilling **Drilling Equipment:** Truck Mount CME 75 **Drilling Method:** Hollow Stem Auger

SUBSURFACE PROFILE		SAMPLE DATA						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)
Depth	Soil Description	Elev./Depth(m)	Lithology	Type	Sample Number	N or RQD	Recovery (%)			
0 ft / 0 m	Ground Surface	99.90								
	Asphalt- 40 mm thick.	0.00								
	Fill- silt and sand, trace gravel, brown, loose.				SS1	59+	75	59	6	
	LIMESTONE BEDROCK- with shaly partings, grey. RQD: poor	99.30 / 0.60			Run 1	42	100			
	LIMESTONE BEDROCK- with shaly partings, grey. RQD: excellent	98.65 / 1.25			Run 2	92	100			
	LIMESTONE BEDROCK- with shaly partings, grey. RQD:excellent	97.16 / 2.74			Run 3	95	100			
	End of Borehole	95.61 / 4.29								

Easting: 442818 m **Northing:** 5028499 m

Site Datum: Center of Lid on Sidewalk in front of 162 Hinchey (100.00 m)

Groundsurface Elevation: 99.90 m **Top of Riser Elev.:** N/A

Hole Diameter: 200 mm

NOTES:

No water encountered while drilling.

Borehole Log: BH2

Project No.: 200295
Project: Proposed Four Storey Apartment Building
Client: Praveen Muppalla
Location: 161 Hinchey Ave, Ottawa ON
Date: July 13, 2020
Field Personnel: BJ
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Truck Mount CME 75
Drilling Method: Hollow Stem Auger

Depth (ft / m)	Soil Description	Elev./Depth(m)	Lithology	Type	Sample Number	N or RQD	Recovery (%)	Shear Strength (kPa) / SPT N Value (Blows/0.3 m)	Water Content (%) / Liquid Limit (%)	Water Level (Standpipe or Open Borehole)
0	Ground Surface	99.87								
0	**Asphalt-** 40 mm thick.	0.00								
0–0.76 m	**Fill-** silt and sand, trace gravel, brown, compact.	99.11 0.76			SS1	22	50	22	5	
0.76 m	End of Borehole Borehole terminated over inferred bedrock after practical auger refusal.									

Easting: 442810 m
Northing: 5028498 m
Site Datum: Center of Lid on Sidewalk in front of 162 Hinchey (100.00 m)
Groundsurface Elevation: 99.87 m
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES:
No water encountered while drilling.

Borehole Log: BH3

Project No.: 200295
Project: Proposed Four Storey Apartment Building
Client: Praveen Muppalla
Location: 161 Hinchey Ave, Ottawa ON
Date: July 13, 2020
Field Personnel: BJ
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Truck Mount CME 75
Drilling Method: Hollow Stem Auger

SUBSURFACE PROFILE		SAMPLE DATA						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)
Depth (ft / m)	Soil Description	Elev./Depth(m)	Lithology	Type	Sample Number	N or RQD	Recovery (%)			
0	Ground Surface	99.89								
	Asphalt- 40 mm thick.	0.00								
	Fill- Gravelly sand, some silt, brown, compact.				SS1	14	65	14	9	
					SS2	55+	50	55+	10	
1.20	**LIMESTONE BEDROCK-** with shaly partings, grey. RQD: good.	98.69 1.20			Run 1	80	100			
2.72	**LIMESTONE BEDROCK-** with shaly partings, grey. RQD: excellent	97.17 2.72			Run 2	100	100			
4.27	End of Borehole	95.62 4.27								

Easting: 442804 m
Northing: 5028496 m
Site Datum: Center of Lid on Sidewalk in front of 162 Hinchey (100.00 m)
Groundsurface Elevation: 99.89 m
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES:
No water encountered while drilling.

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

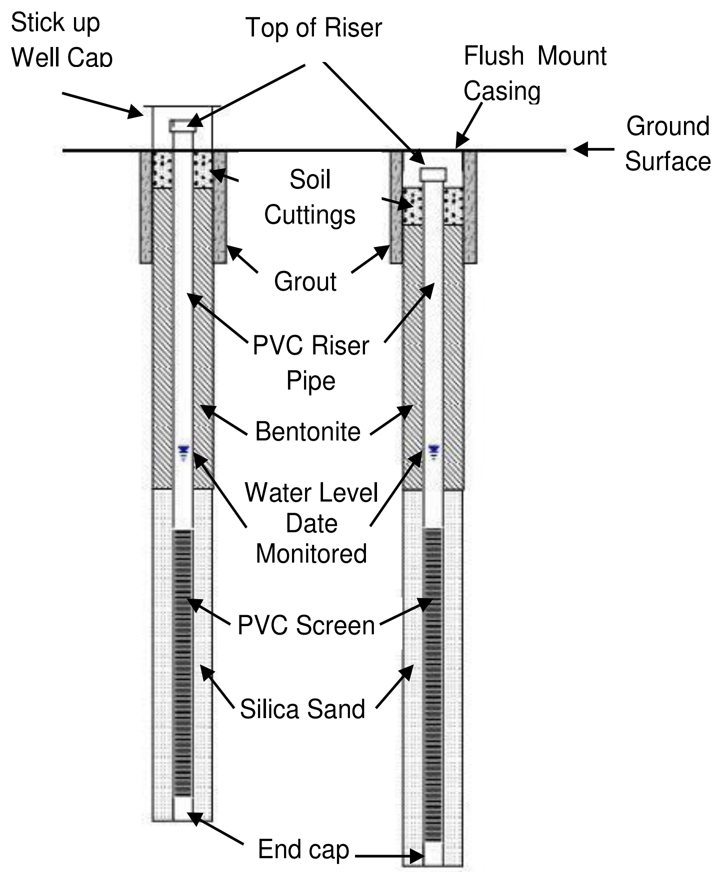
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given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

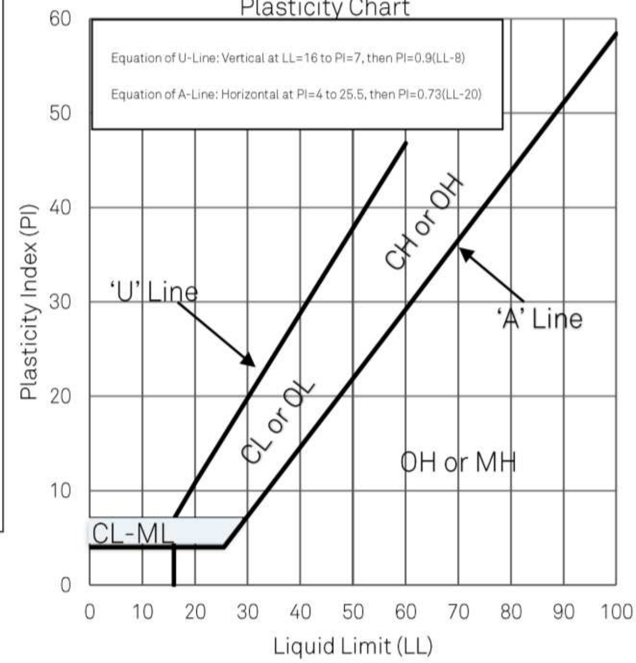
4. General Monitoring Well Data



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 (see below)		
			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				

Plasticity Chart



APPENDIX D

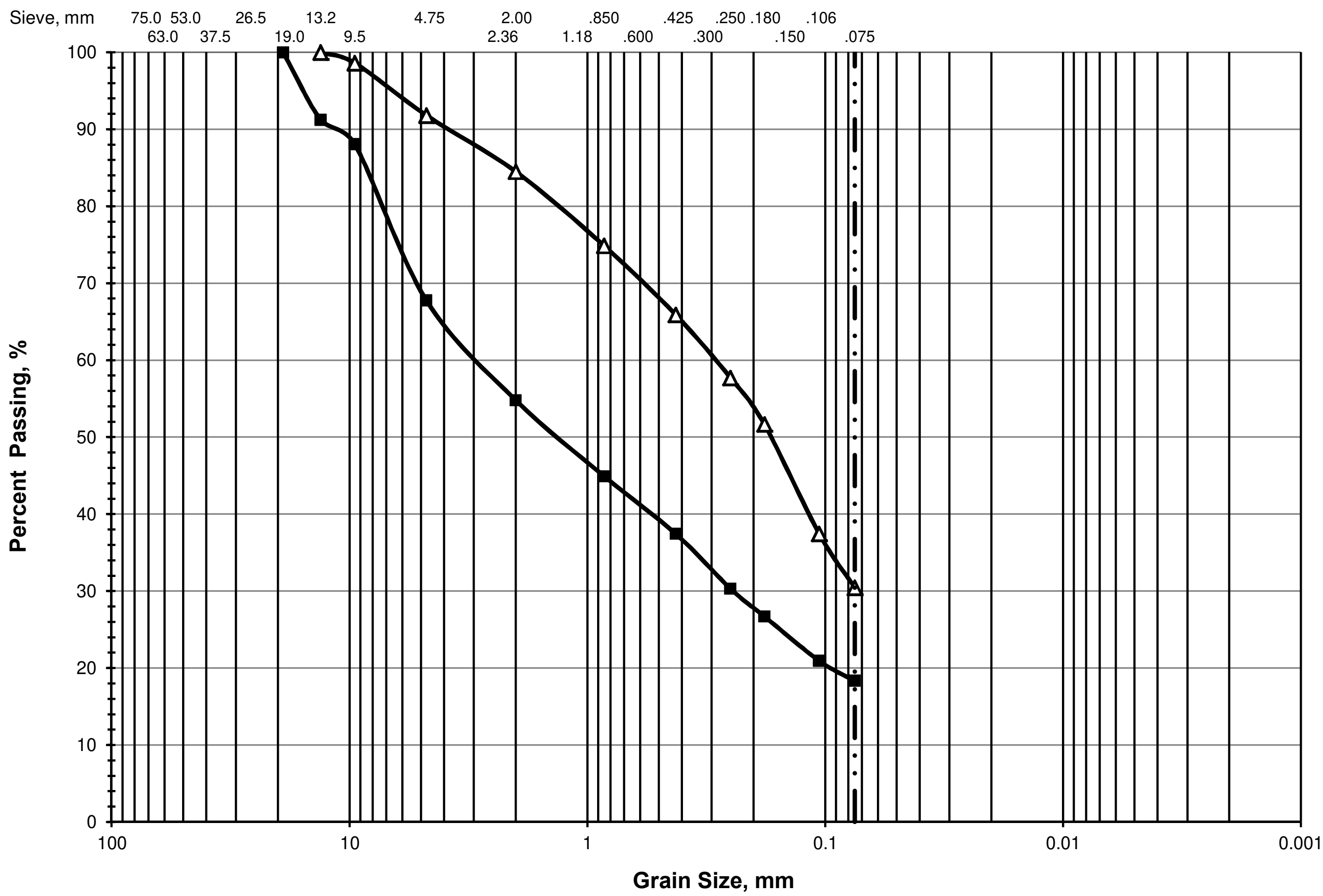
Laboratory Results

LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client:	Mupalla Praveen	**File No.:**	200295
Project:	Geotechnical Investigation	**Report No.:**	1
Location:	161 Hinchey Avenue, Ottawa, ON.	**Date:**	July 13, 2020



Unified Soil Classification System

	> 75 mm	% GRAVEL		% SAND			% FINES
		Coarse	Fine	Coarse	Medium	Fine	Silt & Clay
△	0.0	0.0	8.2	7.3	18.6	35.5	30.4
■	0.0	0.0	32.2	13.0	17.4	19.1	18.3

	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 1	SS-1	0.00 - 0.61	0.2992	0.1714	0.0731				
■	BH 3	SS-1	0.00 - 0.61	3.1042	1.4414	0.2439				

LRL Associates Ltd.

Unconfined Compressive Strength of Intact Rock Core

ASTM D 7012: Method C

Client:	Mupalla Praveen	**File No.:**	200295
Project:	Geotechnical Investigation	**Report No.:**	2
Location:	161 Hinchey Avenue, Ottawa, ON.	**Date:**	July 13, 2020

Drill Core Information

Date(s) Sampled: Jly 13, 2020

Sampled By: LRL Associates Ltd.

Date Received: July 13, 2020

Laboratory Identification	Core No.	Field Identification	Borehole	Run	Depth, m	Location / Description
C01175	1		BH 1	2	1.57 - 1.68	
C01176	2		BH 1	3	2.82 - 2.92	
C01177	3		BH 3	1	1.40 - 1.50	
C01178	4		BH 3	2	3.73 - 3.84	

Rock Core Unconfined Compressive Strength Test Data

	Core No.	Conditioning	Length, mm	Diameter, mm	Density, kg/m^3	MPa	Description of Failure
C01175	1	As received	88.5	45.0	2643	113.7	Columnar, relatively well formed cone on each end
C01176	2	As received	89.7	45.0	2653	72.7	Columnar with horizontal breaks along infill/shale seams, relatively well formed cone on each end
C01177	3	As received	86.5	44.7	2681	93.4	Columnar with horizontal breaks along infill/shale seams, relatively well formed cone on one end
C01178	4	As received	85.8	44.7	2709	94.3	Columnar with horizontal breaks along infill/shale seams, relatively well formed cone on one end

Comments:

Date Issued: July 17, 2020

Reviewed By: W.A.McLaughlin

W.A.M^{c}Laughlin, Geo.Tech., C.Tech.

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Certificate of Analysis

LRL Associates Ltd.

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

Client PO:
Project: 200295
Custody: 54508

Report Date: 21-Jul-2020
Order Date: 15-Jul-2020

Order #: 2029313

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

Paracel ID	Client ID
2029313-01	BH3 2.5'-4.5'

Approved By:

Dale Robertson, BSc
Laboratory Director

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

Report Date: 21-Jul-2020
Order Date: 15-Jul-2020
Project Description: 200295

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	20-Jul-20	20-Jul-20
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	20-Jul-20	20-Jul-20
Resistivity	EPA 120.1 - probe, water extraction	20-Jul-20	20-Jul-20
Solids, %	Gravimetric, calculation	20-Jul-20	20-Jul-20

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Order #: 2029313

Certificate of Analysis

Report Date: 21-Jul-2020

Client: LRL Associates Ltd.

Order Date: 15-Jul-2020

Client PO:

Project Description: 200295

	Client ID:	BH3 2.5'-4.5'	-	-	-
	Sample Date:	13-Jul-20 09:00	-	-	-
	Sample ID:	2029313-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	87.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.44	-	-	-
Resistivity	0.10 Ohm.m	15.4	-	-	-
Anions					
Chloride	5 ug/g dry	90	-	-	-
Sulphate	5 ug/g dry	423	-	-	-

Method Quality Control: Blank

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

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

Report Date: 21-Jul-2020
Order Date: 15-Jul-2020
Project Description: 200295

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	403	5	ug/g dry	385			4.7	20	
Sulphate	58.9	5	ug/g dry	55.8			5.4	20	
General Inorganics									
pH	6.84	0.05	pH Units	7.02			2.6	2.3	
Resistivity	11.3	0.10	Ohm.m	11.3			0.2	20	
Physical Characteristics									
% Solids	91.4	0.1	% by Wt.	89.1			2.6	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	481	5	ug/g	385	96.0	82-118			
Sulphate	157	5	ug/g	55.8	102	80-120			

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

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

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