Geotechnical Investigation

Proposed New Building
2020 Bantree Street
Ottawa, Ontario
Revision 2

Prepared for:

De Saulniers Construction Ltd.
1120 Marleau Avenue
Cornwall, ON
K6H 2W8

Attention: Christian Mercier

LRL File No.: 180357

February 2021; *Revised August, 2022*

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1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by De Saulniers Construction Ltd. to perform a geotechnical investigation for the proposed new building to be located at 2020 Bantree Street, in Ottawa, Ontario.

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

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

2 SITE AND PROJECT DESCRIPTION

The site under investigation is located at 2020 Bantree Street, in Ottawa, Ontario. The site has about 175 m of frontage, and a total surface area of about 20,500 m^2. The topography of the site is relatively flat. At the time of the field investigation, there was a thin layer of snow present, with some stockpiles of snow in the surrounding area of the site. There is an existing "L" shaped building located at the western half of the site. Access to the site comes by way of Bantree Street. The location is presented in Figure 1 included in **Appendix A**.

It is understood the new building will have a total area of 1858 m^2, and is to be located at the eastern portion of the site. The new building will be one (1) storey in height, and consist of a warehouse, showroom, and office spaces.

3 PROCEDURE

The fieldwork for this investigation was carried out on January 08, 2021. Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of five (5) boreholes were drilled within the proposed building footprint, and the proposed pavement area, and labelled BH1 through BH5. 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 SPTs 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 the "N" value.

Boreholes were advanced to depths ranging between 2.16 and 3.81 m below ground surface (bgs). Upon completion, the boreholes were backfilled and compacted 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 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 (bolts on flange of fire hydrant across from site (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 this site is located at a transition zone between two (2) different deposits, glacial till, and silt/silty clay.

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 Fill

At the surface of all boring locations, a layer of fill material consisting of greyish brown sand and gravel was encountered.

4.3 Silt

Underlying the fill in BH1, a layer of silt was encountered and extended to a depth of 2.74 m bgs. This layer can be described as having some clay, some silt, moist, and greyish brown in colour. The SPT "N" values were found ranging between 2 and 18 indicating the material is compact to loose. The natural moisture contents were found to be 13 and 23%.

4.4 Silty Clay

Underlying the fill in BH2, a layer of silty clay was encountered, and extended to a depth of 2.59 m bgs. The material had some gravel, greyish brown in colour, and moist. The SPT "N" values were found to be 3 and 15, indicating the material is stiff to soft. The natural moisture contents were found to range between 13 and 19%.

4.5 Silty Sand

Underlying the fill in BH4, a layer of greyish brown, moist, silty sand was encountered, and extended to a depth of 1.45 m bgs. The SPT “N” value was found to be 17, indicating it is compact. The moisture content was found to be 21%.

4.6 Glacial Till

Underlying the silt in BH1, the silty clay in BH2, the fill in BH3 and BH5, and the silty sand in BH4, a layer of glacial till was encountered, and extended to depths ranging between 2.16 and 3.81 m bgs (end of exploration depth). The material was generally composed of silty sand, some clay, some gravel sized stone, dry to moist, and grey in colour. The SPT “N” values were found ranging between 10 and 88, indicating the material is compact to very dense. The natural moisture contents were found to range between 2 and 25%.

4.7 Refusal

Practical auger refusal over large boulders within the glacial till material or possible bedrock was encountered in BH1 through BH3, at depths ranging between of 2.16 and 3.81 m bgs.

4.8 Laboratory Analysis

A soil sample was 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				
		Sand			Silt (%)	Clay (%)
		Coarse (%)	Medium (%)	Fine (%)		
BH2	1.52 – 2.13	0.0	5.6	11.0	63.5	19.9

A soil sample from BH4 was collected for laboratory sieve analyses. The results are summarized below in **Table 2.**

Table 2: Sieve Analysis Summary

Sample Location	Depth (m)	Percent for Each Soil Gradation					
		Gravel		Sand			Fines
		Coarse (%)	Fine (%)	Coarse (%)	Medium (%)	Fine (%)	Silt & Clay (%)
BH4	1.52 – 2.13	21.8	36.5	9.2	9.9	6.6	16.0

Atterberg limits and moisture contents were conducted on the spoon soil sample collected between depths 1.52 and 2.13 m in BH2. Based on the test result, the sample yielded a plastic limit of 19% and corresponding liquid limit of 32%. These values indicate that the subsoil contains inorganic clays of low plasticity. A summary of these values are provided below in **Table 3**.

Table 3: Summary of Atterberg Limits and Water Contents

Sample Location	Parameter					
	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Water Content (%)	USCS Group Symbol
BH2	1.52 – 2.13	32	19	13	19	CL

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

4.9 Groundwater Conditions

Groundwater was carefully monitored and measured during the field investigation. Immediately upon completion of drilling, groundwater was measured in all boreholes and was found to be dry.

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 details.

5.1 Foundations

Based on the subsurface soil conditions established at this site, it is recommended that the footings for the proposed building be founded below the frost penetration depth on the native, undisturbed material consisting of a combination of silt, silty clay, and/or glacial till. Therefore, any organic and any other deleterious material shall be stripped from the building footprint.

5.2 Shallow Foundation

Conventional strip and column footings founded over the undisturbed native soil 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. This bearing capacity limits the allowable grade raise to 2.0 m, and allows for a strip footing maximum width of 2.0 m, and a pad footing maximum width of 4.0 m on any side.

In-situ field testing is required to check the strength and stability of the footing subgrade prior to any placement of concrete. Any incompetent subgrade areas as identified from in-situ testing must be sub-excavated and backfilled with approved structural fill consisting of OPSS Granular B Type II. Similarly, any soft 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 geotechnical engineer or a qualified geotechnical personnel.

5.3 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, consisting of OPSS Granular B Type II, 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 1.2 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

5.4 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.

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 Seismic

Geophysics GPR International Inc. was retained in order to carry out Shear Wave Velocity Sounding for the purposes of Seismic Site Class determination.

It was concluded that this site can be classified as a Site Class B.

The report produced by Geophysics GPR International Inc. can be found attached in **Appendix E.**

5.7 Liquefaction

Based on the soil conditions encountered onsite during the field investigation, liquefaction is not considered to be a concern for this site.

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, lighting etc. 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

Permanent perimeter foundation drainage is recommended if any open spaces are present below the finished floor, or if the building has a basement. The foundation drainage shall consist of a conventional, perforated corrugated polyethylene drainage pipe (100 mm minimum), pre-wrapped with geotextile knitted sock conforming to **OPSS 1840,** embedded in a 300 mm layer of 19 mm clear stone and set adjacent to the perimeter footings. The drainage pipe should be connected positively to a suitable outlet, such as a sump pit or storm sewer.

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

To prevent possible lateral loading, the backfill material against the foundation walls 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 directly over a minimum 150 mm thick layer of OPSS Granular A, compacted to 98% of its SPMDD. Prior to the placement of Granular A, all fill or otherwise deleterious material shall be removed from the proposed building's footprint down to the native subgrade surface. The subgrade should then be inspected and approved by qualified geotechnical personnel prior to placement 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. The modulus of subgrade reaction (ks) for the design of the slabs 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.

5.12 Sulphate Attack and Corrosivity Analysis on Buried Concrete

A soil sample was 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	1.5 – 2.1	7.4	297	223	1,590

The above results revealed a measured sulphate concentration of 297 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 1,590 ohm.cm, which falls within “corrosive” range.

5.13 Other Considerations

Silty Clay was encountered in BH2 at a depth ranging from 0.3 – 2.6 m. Based on the in-situ field tests (“N” values) obtained in this layer, this silty clay is not considered to be sensitive marine clays.

Furthermore, there are no tree planting restrictions from a geotechnical perspective imposed on this site.

6 Excavation and Backfilling Requirements

6.1 Excavation

It is anticipated that the maximum depth of excavation for the building and any services will not extend below about 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.

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. If encountered, 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. 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 granular material. 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, storm 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 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 Recommended Pavement Structure

For predictable performance of the pavement areas, any organic, soft, and/or deleterious materials should be removed from the proposed pavement areas to expose native undisturbed subgrade soil. The exposed subgrade should be inspected and approved by geotechnical personnel and any evidently loose and unstable areas should be sub-excavated and replaced with suitable earth borrow approved by the geotechnical engineer. Following approval of the preparation of the subgrade, the granular subbase may be placed.

The recommended pavement structures for the proposed light and heavy duty access roads and parking areas are provided below.

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

50 mm of hot mix asphaltic concrete (HL3/SP12.5) over;

150 mm of OPSS Granular A base over;

350 mm of OPSS Granular B Type II subbase.

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

40 mm of hot mix asphaltic concrete surface layer (HL3/SP12.5) over;

50 mm of hot mix asphaltic concrete binder layer (HL8/SP19.0) over;

150 mm of OPSS Granular A base over;

450 mm of OPSS Granular B, Type II subbase.

The base and subbase granular materials should conform to **OPSS 1010** material specifications. Prior to importing any granular material onto the site, it should be tested and approved by a geotechnical engineer prior to delivery to the site and should be compacted to 98% SPMDD. Compaction of the granular pavement materials should be carried out in maximum 300 mm thick loose lifts.

Asphaltic concrete should conform to **OPSS 1150** and be placed and compacted to at least 95% of the Marshall Density. The mix and its constituents should be reviewed, tested and approved by a geotechnical engineer prior to delivery to the site.

7.1 Subgrade Preparation

Following the backfilling and satisfactory compaction of any underground service trenches up to the subgrade level, the subgrade should be shaped, crowned and proof-rolled using heavy roller with any resulting soft areas sub-excavated down to an adequate bearing layer and replaced with approved backfill. Following approval of the preparation of the subgrade, the pavement structure may be placed.

If the roadway subgrade is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or incorporate a non-woven geotextile separator between the roadway subgrade surface and the granular subbase material.

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

For areas of the site that require the subgrade to be raised, the material should consist of OPSS Granular B Type I, II, or approved equivalent. Any materials proposed for this use should be approved by the geotechnical engineer before placement. Materials used for raising the subgrade to the proposed roadway subgrade level should be placed in maximum 300 mm thick loose lifts and be compacted to at least 95% of the SPMDD using suitable compaction equipment.

The preparation of subgrade should be scheduled and carried out in such a manner that a protective cover of overlying granular material is placed as quickly as possible in order to avoid unnecessary circulation by heavy equipment over the subgrade. Frost protection of the surface should be implemented (i.e. insulated tarps, etc.), if works are carried out during the winter months.

Transitions should be constructed between new and existing pavement structures where new access lanes will meet with existing road. In areas where the new pavement structure will abut existing pavement structure, the depths of granular materials should be tapered up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement.

8 Reuse of On-Site Soils

The existing surficial overburden material for this site 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.

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 buildings 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

LRL ENGINEERING | INGÉNIERIE

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

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING
2020 BANTREE STREET
OTTAWA, ONTARIO

DRAWING TITLE

SITE LOCATION
SOURCE: GEO-OTTAWA

CLIENT

DE SAULNIERS CONSTRUCTION LTD.

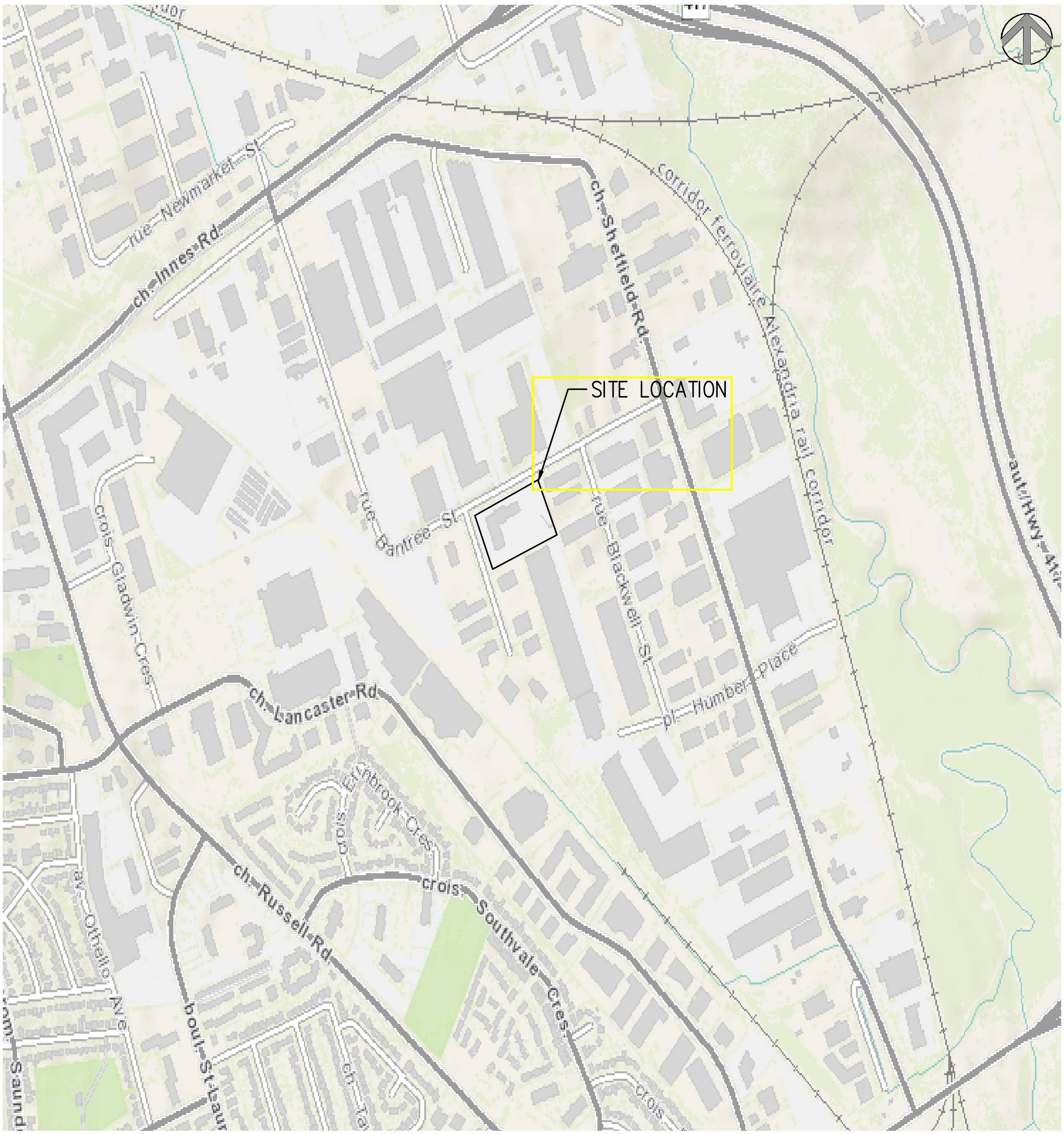
DATE

JANUARY 2021

PROJECT

180357

FIGURE 1



ENGINEERING | INGÉNIERIE

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

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING
2020 BANTREE STREET
OTTAWA, ONTARIO

DRAWING TITLE

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

CLIENT

DE SAULNIERS CONSTRUCTION LTD.

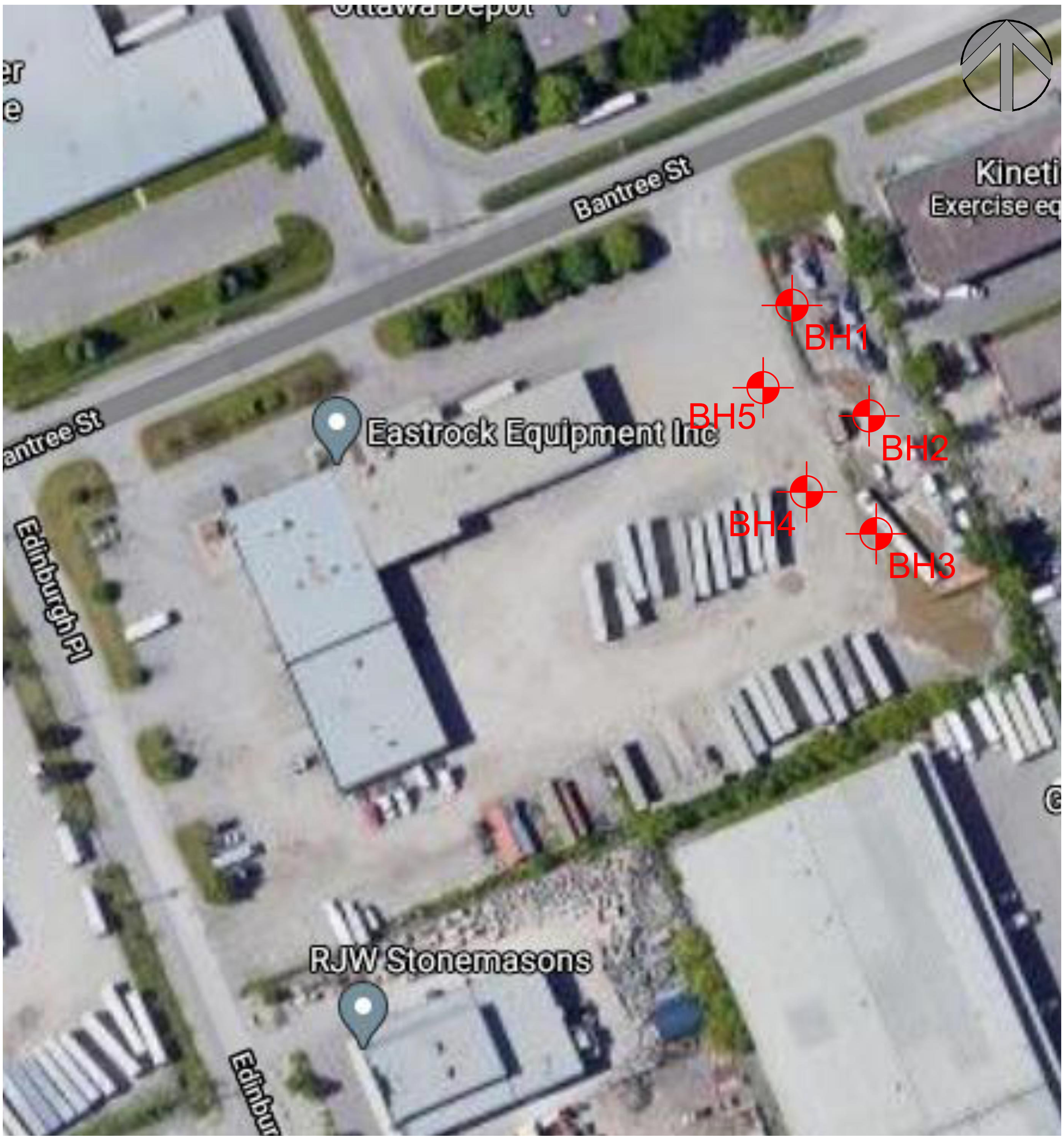
DATE

JANUARY 2021

PROJECT

180357

FIGURE 2



APPENDIX B

Borehole Logs

Borehole Log: BH1

Project No.: 180357
Project: Geotechnical Investigation - Proposed Building
Client: De Saulniers Construction Ltd.
Location: 2020 Bantree Street, Ottawa ON
Date: January 8, 2021
Field Personnel: BJ
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Truck Mount CME 75
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE

Depth (m)	Soil Description	Elev./Depth (m)
0	Ground Surface	99.57 / 0.00
0.00–0.30	**FILL** sand and gravel, greyish brown.	99.27 / 0.30
0.30–2.74	**SILT** some clay, some sand, greyish brown, moist, loose to compact.	96.83 / 2.74
2.74–3.71	**GLACIAL TILL** silty sand, some clay, some gravel sized stone, wet, grey, dense.	95.86 / 3.71
3.71	End of Borehole	

SAMPLE DATA

Type	Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)
SS	SS1	18	33	18	13
SS	SS2	5	21	5	23
SS	SS3	2	42	2	13
SS	SS4	50+	50	50+	12

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

Monitoring Well Details

Easting: 452136 m
Northing: 5028158 m
Site Datum: TBM - Bolts on flange of hydrant across the street from site (100.00 m)
Groundsurface Elevation: 99.574 m
Top of Riser Elev.: NA
Hole Diameter: 200 mm
Monitoring Well Diameter: N/A

NOTES:
Borehole terminated after practical auger refusal.

Borehole Log: BH2

Project No.: 180357
Project: Geotechnical Investigation - Proposed Building
Client: De Saulniers Construction Ltd.
Location: 2020 Bantree Street, Ottawa ON
Date: January 8, 2021
Field Personnel: BJ
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Truck Mount CME 75
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA							
Depth	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	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	Monitoring Well Details
ft 0 / m 0	Ground Surface	99.32 0.00							
	FILL sand and gravel, greyish brown.	99.02 0.30							
	SILTY CLAY some gravel, greyish brown, moist, stiff to soft.		SS	SS1	15	50	15	13	
			SS	SS2	3	50	3	19 / 32	
			SS	SS3	18	42	18	18	
	GLACIAL TILL silty sand, some clay, some gravel sized stone, dry to moist, grey, very dense.	96.73 2.59						5	
			SS	SS4	88	74	88	17	
	End of Borehole	95.51 3.81							

Easting: 452147 m
Northing: 5028134 m
Site Datum: TBM - Bolts on flange of hydrant across the street from site (100.00 m)
Groundsurface Elevation: 99.321 m
Top of Riser Elev.: NA
Hole Diameter: 200 mm
Monitoring Well Diameter: N/A

NOTES:
Borehole terminated after practical auger refusal.

Borehole Log: BH3

Project No.: 180357
Project: Geotechnical Investigation - Proposed Building
Client: De Saulniers Construction Ltd.
Location: 2020 Bantree Street, Ottawa ON
Date: January 8, 2021
Field Personnel: BJ
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Truck Mount CME 75
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE / SAMPLE DATA

Depth (ft / m)	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)	Monitoring Well Details
0 / 0	Ground Surface	99.44							
0–1 ft	**FILL** sand and gravel, greyish brown.	0.00							
1 ft	**GLACIAL TILL** silty sand, some clay, some gravel sized stone, dry to moist, grey, compact to very dense.	99.14 / 0.30							
3–4 ft			SS	SS1	18	67	18	15	
5.5–6.5 ft			SS	SS2	59	50	59	2	
7 ft	End of Borehole	97.28 / 2.16							

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

Easting: 452159 m
Northing: 5028107 m
Site Datum: TBM - Bolts on flange of hydrant across the street from site (100.00 m)
Groundsurface Elevation: 99.438 m
Top of Riser Elev.: NA
Hole Diameter: 200 mm
Monitoring Well Diameter: N/A

NOTES:
Borehole terminated after practical auger refusal.

Borehole Log: BH4

Project No.: 180357
Project: Geotechnical Investigation - Proposed Building
Client: De Saulniers Construction Ltd.
Location: 2020 Bantree Street, Ottawa ON
Date: January 8, 2021
Field Personnel: BJ
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Truck Mount CME 75
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE

Soil Description	Elev./Depth (m)
Ground Surface	99.52 / 0.00
FILL sand and gravel, greyish brown.	99.22 / 0.30
SILTY SAND greyish brown, moist, compact.	98.07 / 1.45
GLACIAL TILL sandy gravel, some silt and clay dry, grey, compact to very dense.	95.86 / 3.66
End of Borehole	

SAMPLE DATA

Type	Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)
SS	SS1	17	50	17	21
SS	SS2	51	46	51	4
SS	SS3	50+	50	50+	10
SS	SS4	50+	50	50+	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

Depth: ft 0–22; m 0–6

Monitoring Well Details

Easting: 452144 m
Northing: 5028115 m
Site Datum: TBM - Bolts on flange of hydrant across the street from site (100.00 m)
Groundsurface Elevation: 99.522 m
Top of Riser Elev.: NA
Hole Diameter: 200 mm
Monitoring Well Diameter: N/A

NOTES:

Borehole Log: BH5

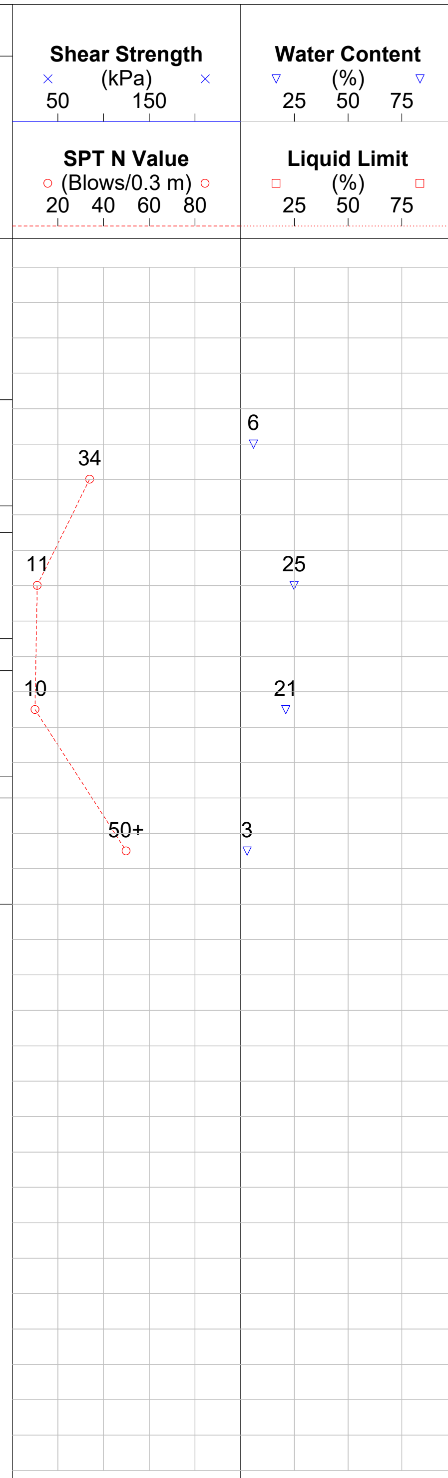
Project No.: 180357
Project: Geotechnical Investigation - Proposed Building
Client: De Saulniers Construction Ltd.
Location: 2020 Bantree Street, Ottawa ON
Date: January 8, 2021
Field Personnel: BJ
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Truck Mount CME 75
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE

Depth	Soil Description	Elev./Depth (m)
0 ft / 0 m	Ground Surface	99.68 / 0.00
	FILL sand and gravel, greyish brown.	99.38 / 0.30
	GLACIAL TILL sandy gravel, some silt and clay dry, grey, compact to very dense.	96.02 / 3.66
	End of Borehole	

SAMPLE DATA

Type	Sample Number	N or RQD	Recovery (%)	SPT N Value	Water Content (%)
SS	SS1	34	50	34	6
SS	SS2	11	46	11	25
SS	SS3	10	50	10	21
SS	SS4	50+	50	50+	3



Monitoring Well Details

Easting: 452131 m
Northing: 5028142 m
Site Datum: TBM - Bolts on flange of hydrant across the street from site (100.00 m)
Groundsurface Elevation: 99.679 m
Top of Riser Elev.: NA
Hole Diameter: 200 mm
Monitoring Well Diameter: N/A

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

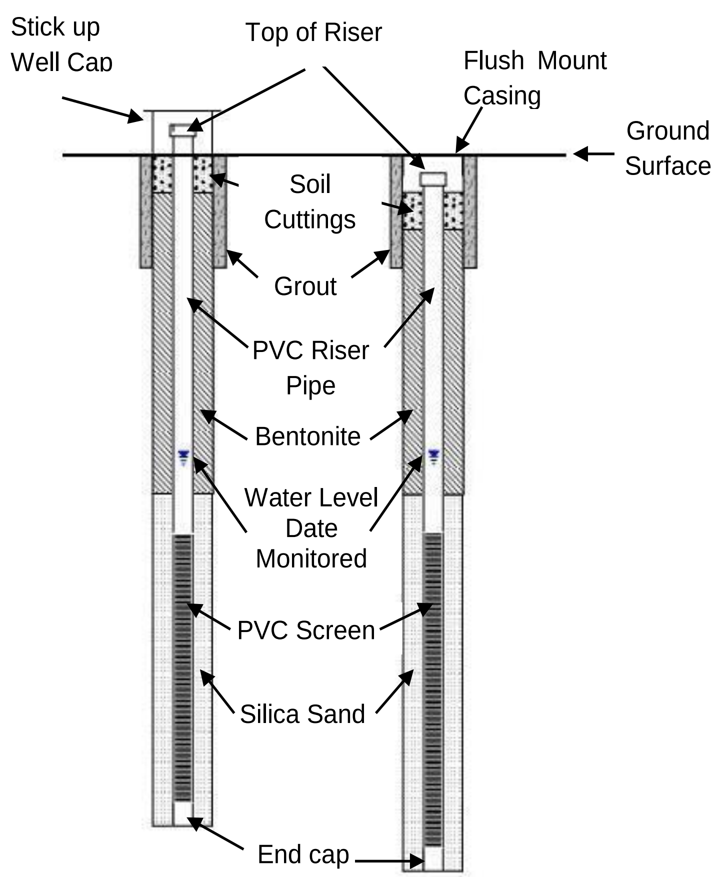
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given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

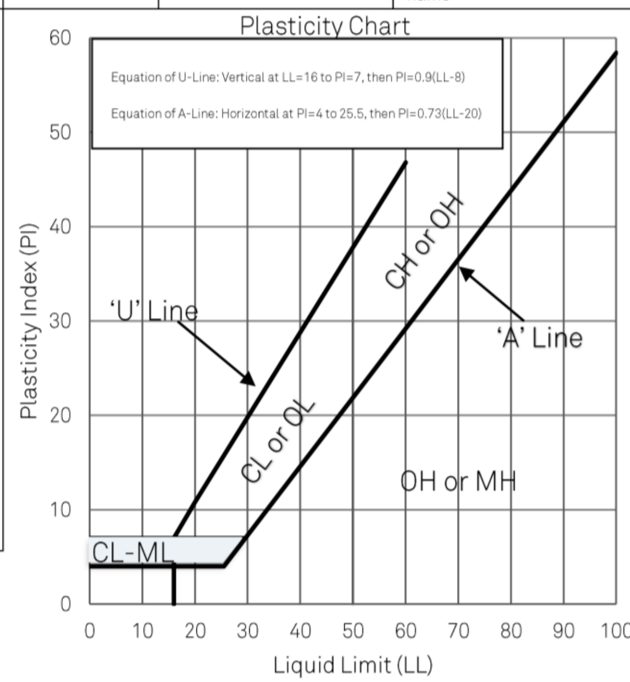
Strength Classification	Range of Unconfined Compressive Strength (MPa)
Extremely weak	< 1
Very weak	1 – 5
Weak	5 – 25
Medium strong	25 – 50
Strong	50 – 100
Very strong	100 – 250
Extremely strong	> 250

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 (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				



APPENDIX D

Laboratory Results

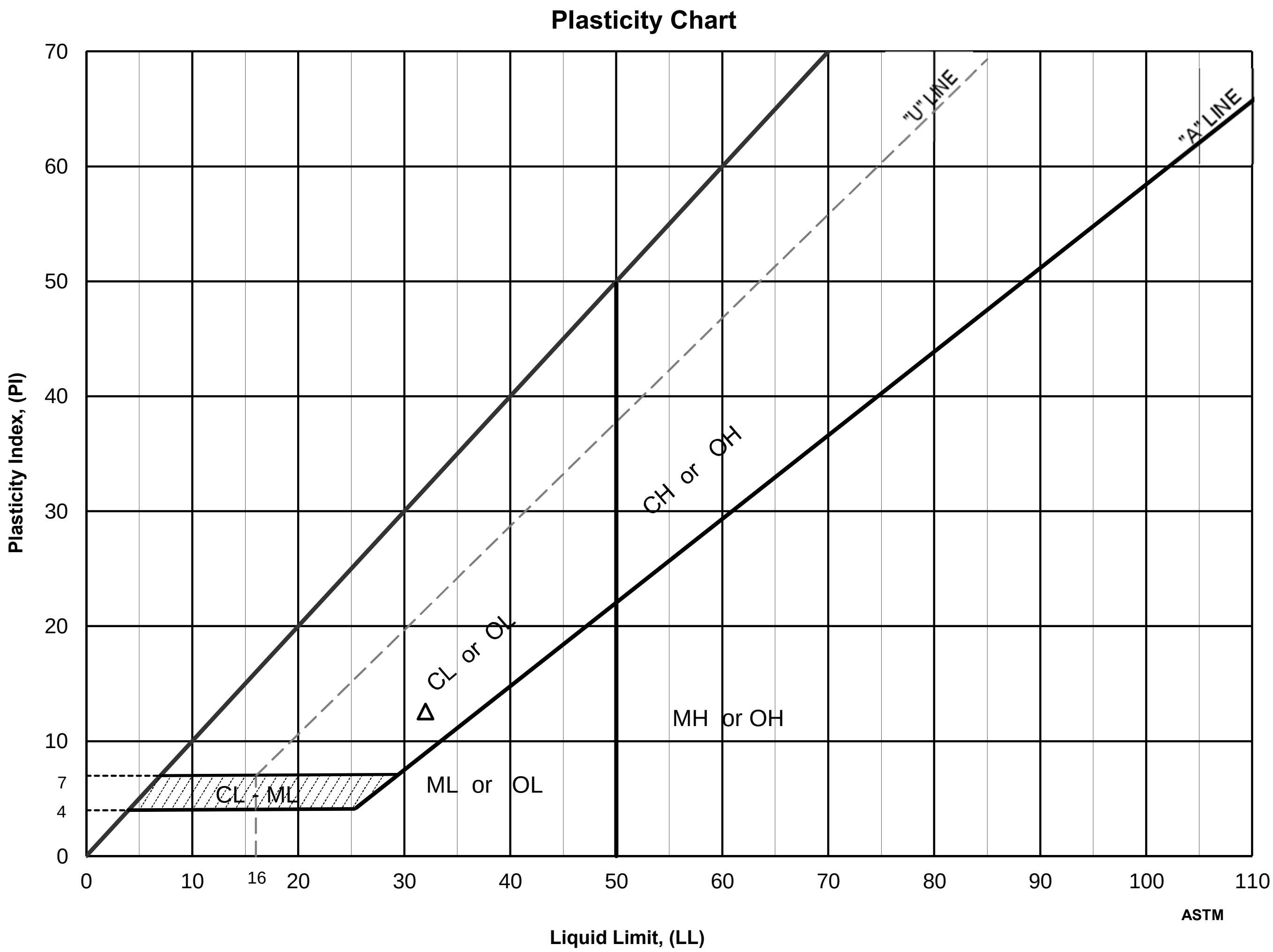
LRL Associates Ltd.

PLASTICITY INDEX

ASTM D 4318 / LS-703/704

Client:	DeSaulniers Construction Ltd.	**File No.:**	180357
Project:	Geotechnical Investigation	**Report No.:**	1
Location:	2020 Bantree Street, Ottawa, ON.	**Date:**	January 8, 2021

Plasticity Chart



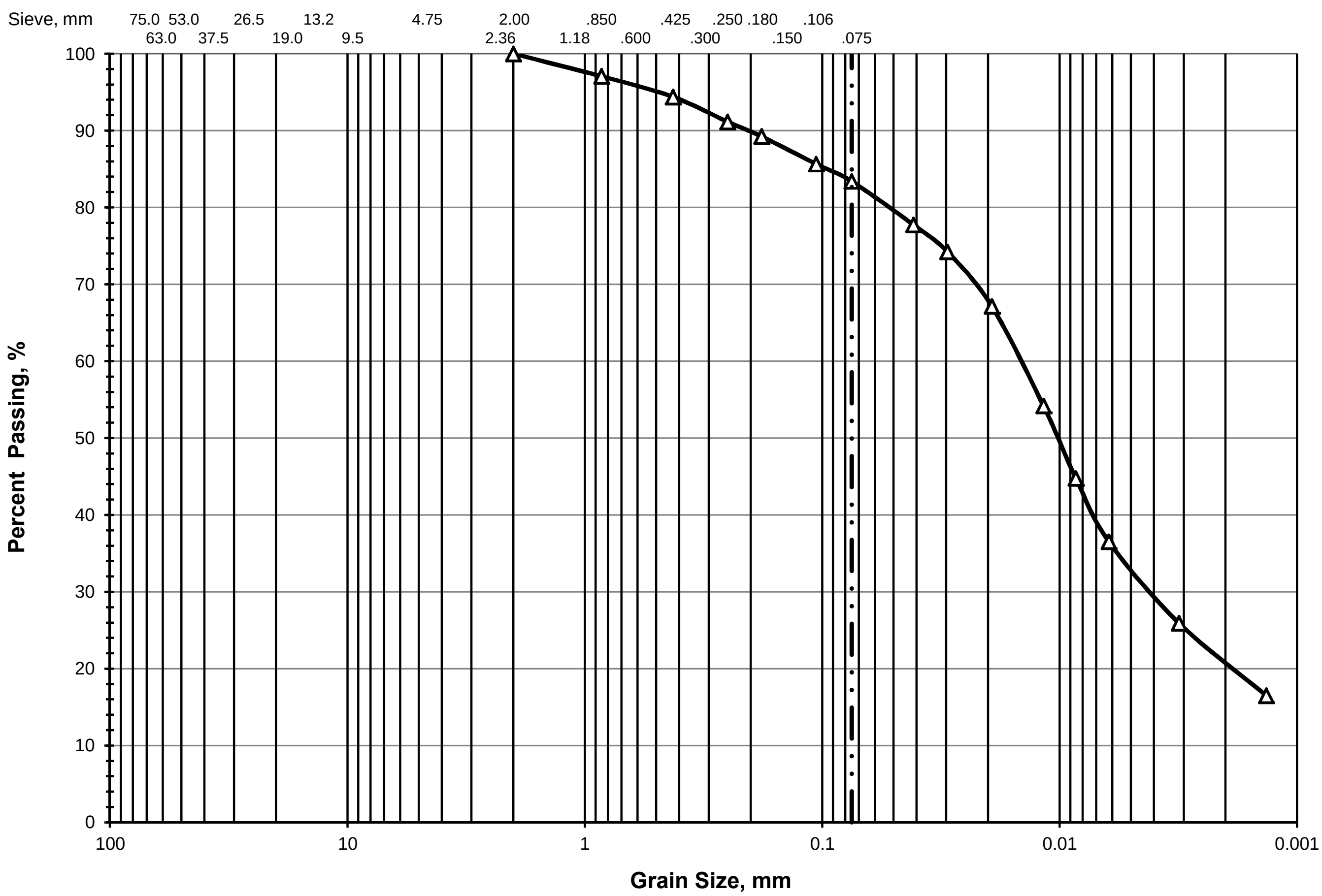
	Location	Sample	Depth, m	Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Activity Number	USCS
△	BH 2	SS-2	1.52 - 2.13	19	32	19	13	-0.02	n/d	CL

LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client:	DeSaulniers Construction Ltd.	**File No.:**	180357
Project:	Geotechnical Investigation	**Report No.:**	2
Location:	2020 Bantree Street, Ottawa, ON.	**Date:**	January 8, 2021



Unified Soil Classification System

	> 75 mm	% GRAVEL		% SAND			% FINES	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
△	0.0	0.0	0.0	0.0	5.6	11.0	63.5	19.9

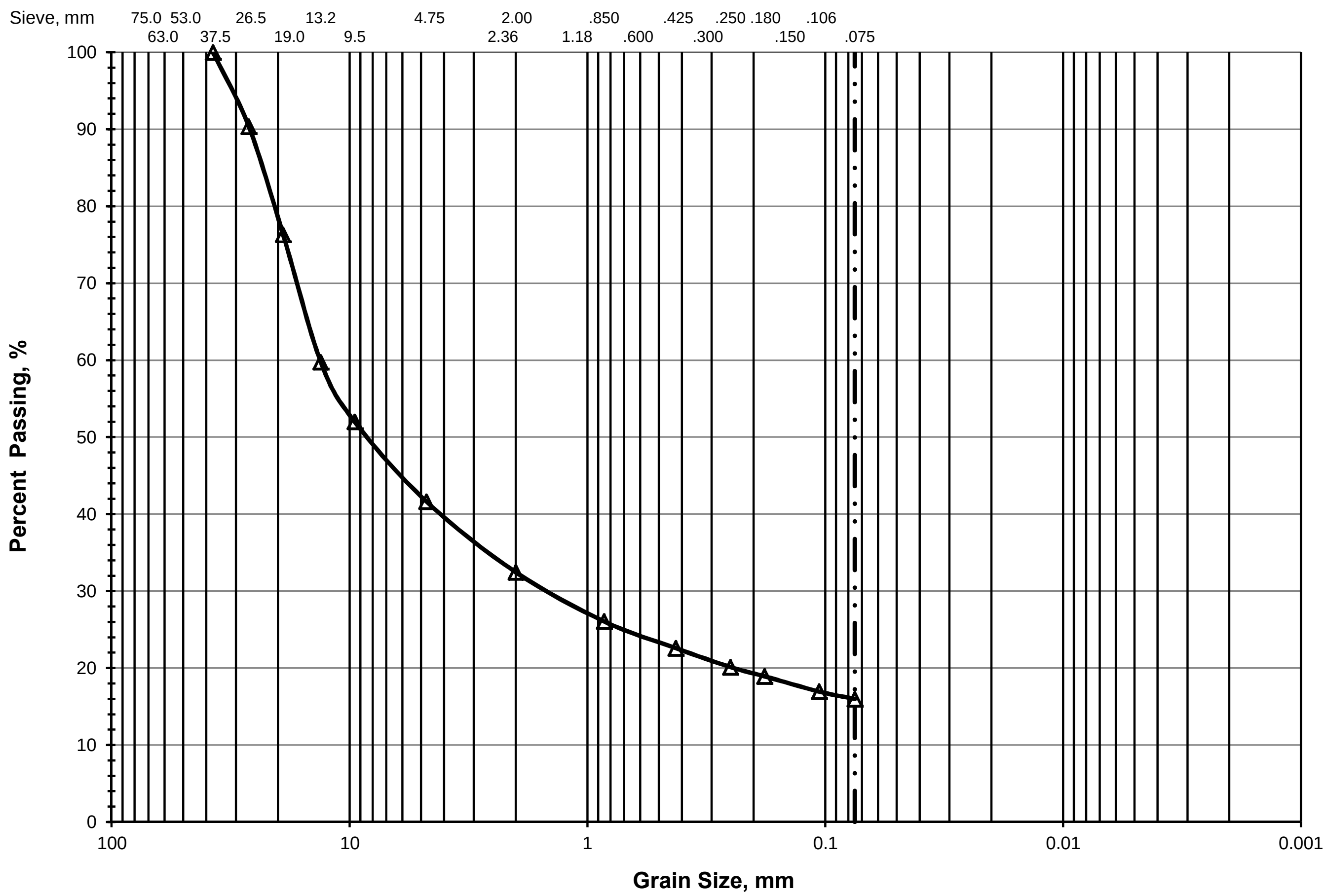
	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 1	SS-2	1.52 - 2.13	0.0151	0.0103	0.0043				

LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client:	DeSaulniers Construction Ltd.	**File No.:**	180357
Project:	Geotechnical Investigation	**Report No.:**	3
Location:	2020 Bantree Street, Ottawa, ON.	**Date:**	January 8, 2021



Unified Soil Classification System

	> 75 mm	% GRAVEL		% SAND			% FINES
		Coarse	Fine	Coarse	Medium	Fine	Silt & Clay
△	0.0	21.8	36.5	9.2	9.9	6.6	16.0

	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 4	SS-2	1.52 - 2.13	13.2912	8.5910	1.5577				

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Certificate of Analysis

LRL Associates Ltd.

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

Client PO:
Project: 180357
Custody: 52965

Report Date: 20-Jan-2021
Order Date: 15-Jan-2021

Order #: 2103488

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

Paracel ID	**Client ID**
2103488-01	180357: BH2: 5-7'

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis

Client: **LRL Associates Ltd.**

Client PO:

Report Date: 20-Jan-2021

Order Date: 15-Jan-2021

Project Description: 180357

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	19-Jan-21	19-Jan-21
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	18-Jan-21	18-Jan-21
Resistivity	EPA 120.1 - probe, water extraction	18-Jan-21	19-Jan-21
Solids, %	Gravimetric, calculation	15-Jan-21	18-Jan-21

Certificate of Analysis

Client: **LRL Associates Ltd.**

Client PO:

Report Date: 20-Jan-2021

Order Date: 15-Jan-2021

Project Description: 180357

	Client ID:	180357: BH2: 5-7'	-	-	-
	Sample Date:	08-Jan-21 09:00	-	-	-
	Sample ID:	2103488-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.35	-	-	-
Resistivity	0.10 Ohm.m	15.9	-	-	-
Anions					
Chloride	5 ug/g dry	223	-	-	-
Sulphate	5 ug/g dry	297	-	-	-

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

Report Date: 20-Jan-2021
Order Date: 15-Jan-2021
Project Description: 180357

Method Quality Control: Blank

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

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	206	5	ug/g dry	223			7.8	20	
Sulphate	326	5	ug/g dry	297			9.6	20	
General Inorganics									
pH	7.67	0.05	pH Units	7.65			0.3	2.3	
Resistivity	7.86	0.10	Ohm.m	7.89			0.5	20	
Physical Characteristics									
% Solids	84.8	0.1	% by Wt.	87.6			3.2	25	

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

Report Date: 20-Jan-2021
Order Date: 15-Jan-2021
Project Description: 180357

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	104	5	ug/g	12.6	91.7	82-118			
Sulphate	113	5	ug/g	15.0	98.5	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.

APPENDIX E

Multi-channel Analysis of Surface Waves (MASW) Report

GEOPHYSICS GPR INTERNATIONAL INC.

100 – 2545 Delorimier Street
Longueuil (Québec)
Canada J4K 3P7

Tel. : (450) 679-2400
Fax : (514) 521-4128
info@geophysicsgpr.com
www.geophysicsgpr.com

August 19th, 2022

Transmitted by email: bjohnson@lrl.ca
Our Ref.: GPR-22-04011

Mr. Brad Johnson, P.Eng.
Geotechnical Engineer
LRL Associates Ltd.
5430 Canotek Road
Ottawa, Ontario K1J 9G2

Subject: **Shear Wave Velocity Sounding for the Site Class Determination**
2020 Bantree Street, Ottawa (ON)

[Project: 180357]

Dear Sir,

Geophysics GPR International inc. has been mandated by LRL Associates Ltd. to carry out seismic shear wave surveys at 2020 Bantree Street, in Ottawa (ON). The geophysical investigation used the Multi-channel Analysis of Surface Waves (MASW), the Spatial AutoCorrelation (SPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocity values were calculated for the soil and the rock, to determine the Site Class.

The surveys were carried out on August 11th, 2022, by Mr. Timothy Ward-Laurier, tech. and Mr. Jean-Philippe Comtois, trainee. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendix.

The following paragraphs briefly describe the survey design, the principles of the testing methods, and the results presented in table and graph.

MASW PRINCIPLE

The *Multi-channel Analysis of Surface Waves* (MASW) and the *SPatial AutoCorrelation* (SPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface wave. The MASW is considered an "active" method, as the seismic signal is induced at known location and time in the geophones' spread axis. Conversely, the SPAC is considered a “passive” method, using the low frequency “signals” produced far away. The method can also be used with "active" seismic source records. The SPAC method generally allows deeper Vs soundings. Its dispersion curve can then be merged with the one of higher frequency from the MASW to calculate a more complete inversion. The dispersion properties are expressed as a change of velocities with respect to frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding).

Figure 3 schematically outlines the basic operating procedure for the MASW method. Figure 4 illustrates an example of one of the MASW/SPAC records, the corresponding spectrogram analysis and resulting 1D V_S model.

INTERPRETATION

The main processing sequence involved data inspection and edition when required; spectral analysis ("phase shift" for MASW, and "cross-correlation" for SPAC); picking the fundamental mode; and 1D inversion of the MASW and SPAC shot records using the SeisImagerSW™ software. The data inversions used a nonlinear least squares algorithm.

In theory, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, local surface seismic velocities variations, and/or dipping of overburden layers or rock. In general, the precision of the calculated seismic shear wave velocities (V_S) is of the order of 15% or better.

More detailed descriptions of these methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015.

SURVEY DESIGN

The seismic acquisition spreads were laid at the eastern extremity of the property (Figure 2). The geophone spacing was of 3.0 metres for the main spread, using 24 geophones. Two shorter seismic spreads, with geophone spacing of 0.5 and 1.0 metre, were dedicated to the near surface materials. The seismic records were produced with a seismograph Terraloc Pro 2 (from ABEM Instrument), and the geophones were 4.5 Hz. The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and 40 µs for the seismic refraction. The records included a pre-trigged portion of 10 ms. An 8 kg sledgehammer was used as the energy source, with impacts being recorded off both ends of the seismic spreads. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

The shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length.

RESULTS

From seismic refraction, the rock was calculated between 2.6 and 4.4 metres deep (±1 metre). The MASW calculated V_S results are illustrated at Figure 5. Some low seismic velocities were calculated from 1 to 2 metres deep. A geotechnical assessment of the corresponding materials could be required for the potential of liquefaction and other critical parameters.

The $\overline{V}_{S30}$ value results from the harmonic mean of the shear wave velocities, from the surface to 30 metres deep. It is calculated by dividing the total depth of interest (30 metres) by the sum of the time spent in each velocity layer from the surface down to 30 metres, as:

$$\overline{V}_{S30} = \frac{\sum_{i=1}^{N} H_i}{\sum_{i=1}^{N} H_i / V_i} \quad | \quad \sum_{i=1}^{N} H_i = 30 \text{ m}$$

(N: number of layers; H_i : thickness of layer "i" ; V_i : V_S of layer "i")

Thus, the $\overline{V}_{S30}$ value represents the seismic shear wave velocity of an equivalent homogeneous single layer response, between the surface and 30 metres deep.

The calculated $\overline{V}_{S30}$ value of the actual site is 1034.2 m/s (Table 1), corresponding to the Site Class "B". The Site Classes A and B are not to be used if there is 3 metres or more of unconsolidated material between the rock and the bottom of the spread footing or mat foundation. In the case the bottom of the foundation would be 1.3 metres or less from the rock, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, corresponding to the Site Class "A" (Table 2).

CONCLUSION

Geophysical surveys were carried out to identify the Site Class at 2020 Bantree Street, in Ottawa (ON). The seismic surveys used the MASW and the SPAC analysis, and the seismic refraction to calculate the $\overline{V}_{S30}$ value. Its calculation is presented at Table 1.

The $\overline{V}_{S30}$ value of the actual site is 1034 m/s, corresponding to the Site Class "B" ($760 < \overline{V}_{S30} \leq 1500$ m/s), as determined through the MASW and SPAC methods, Table 4.1.8.4.-A of the NBC, and the Building Code, O. Reg. 332/12. It must be noted that the Site Classes A and B are not to be used if there is 3 metres or more of unconsolidated material between the rock and the bottom of the spread footing or mat foundation.

In the case there would be 1.3 metres or less of unconsolidated material between the rock and the bottom of the foundation, the $\overline{V}_{S30}$* value would be greater than 1500 m/s, corresponding to the Site Class "A"

Some low seismic velocities were calculated from 1 to 2 metres deep. A geotechnical assessment of the corresponding materials could be required for the potential of liquefaction and other critical parameters.

It must also be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, very soft clays, high moisture content etc. (cf. Table 4.1.8.4.-A of the NBC) can supersede the Site classification provided in this report based on the $\overline{V}_{S30}$ value.

The V_S values calculated are representative of the in situ materials and are not corrected for the total and effective stresses.

Hoping the whole to your satisfaction, we remain yours truly,

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Figure 1: Regional location of the Site
(source: *OpenStreetMap©*)

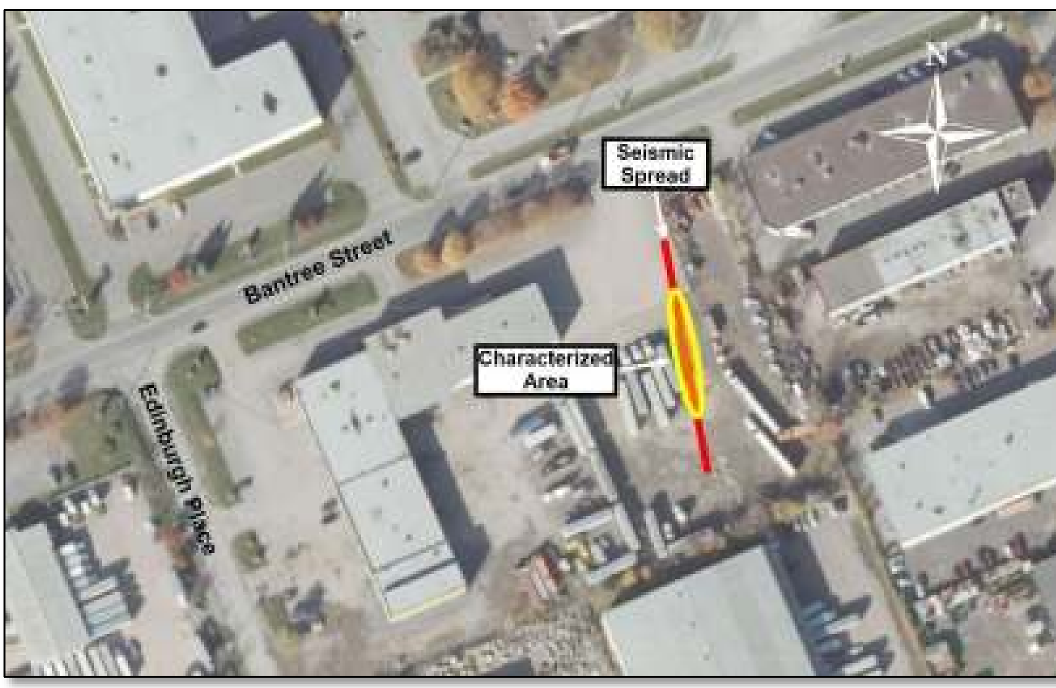

Figure 2: Location of the seismic spreads
(source: *geoOttawa*)

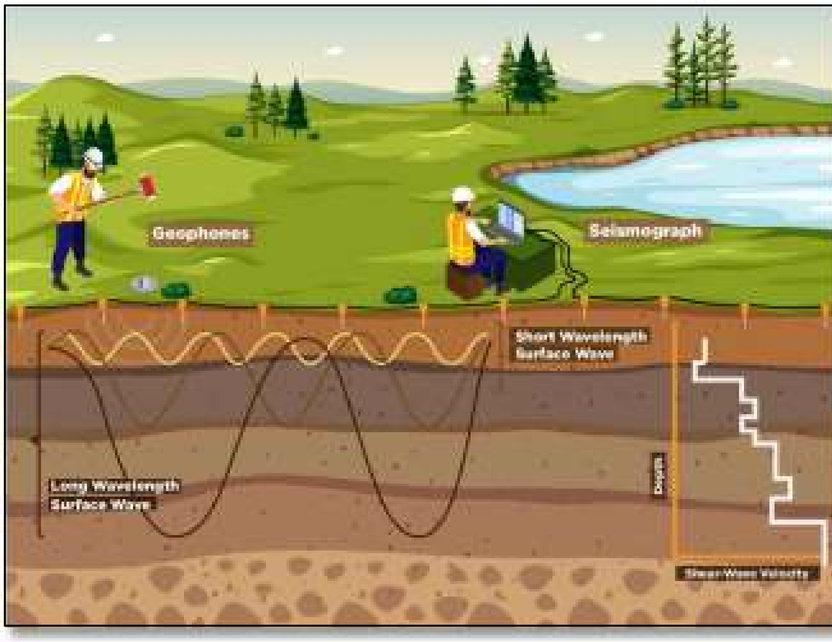

Figure 3: MASW Operating Principle

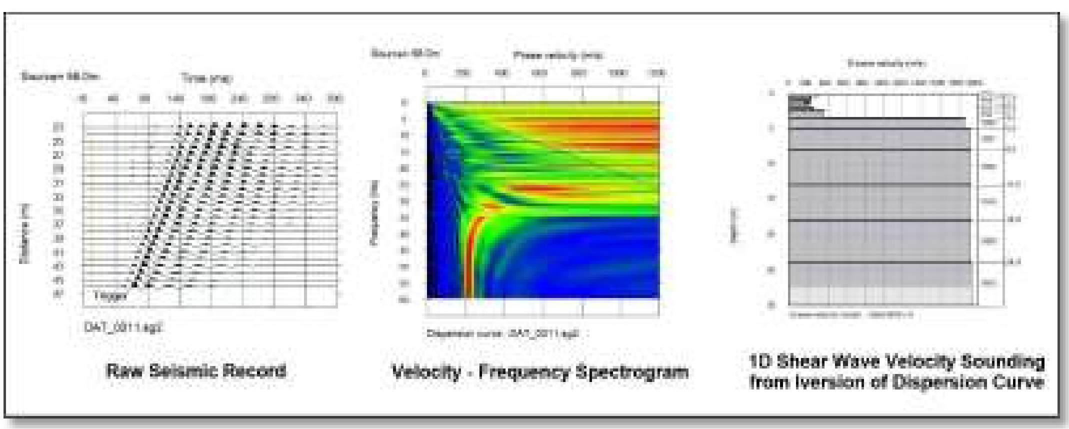

Figure 4: Example of a MASW/SPAC record, Phase Velocity - Frequency curve of the Rayleigh wave and resulting 1D Shear Wave Velocity Model

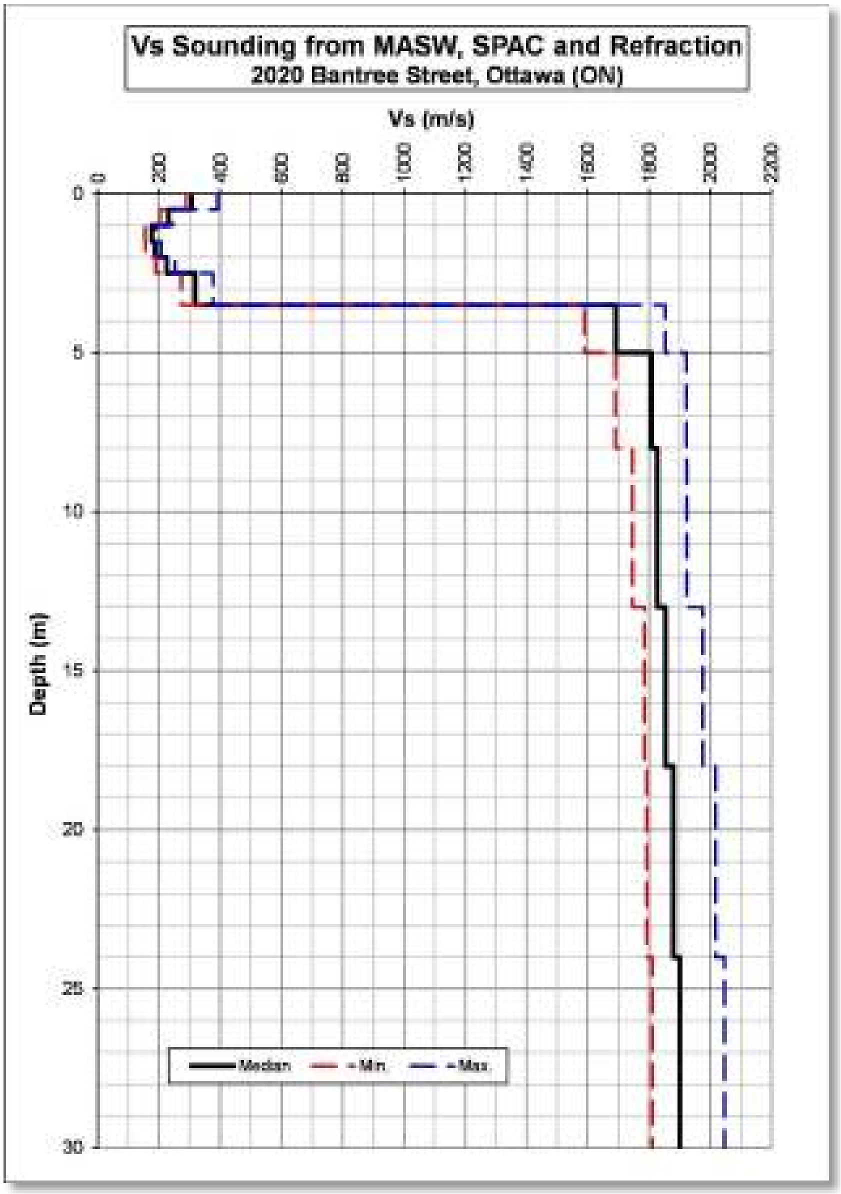

Figure 5: MASW Shear-Wave Velocity Sounding

TABLE 1

V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	288.7	306.7	392.2	**Grade Level (August 11, 2022)**				
0.5	203.9	231.4	245.2	0.50	0.50	0.001630	0.001630	306.7
1.0	**153.9**	**175.9**	189.3	0.50	1.00	0.002161	0.003791	263.8
1.5	**156.4**	186.6	205.9	0.50	1.50	0.002843	0.006634	226.1
2.0	189.5	225.3	252.3	0.50	2.00	0.002679	0.009313	214.8
2.5	272.6	317.7	376.5	0.50	2.50	0.002219	0.011532	216.8
3.5	1589.4	1692.4	1852.8	1.00	3.50	0.003147	0.014679	238.4
5.0	1690.3	1807.7	1921.8	1.50	5.00	0.000886	0.015565	321.2
8.0	1745.8	1826.9	1922.9	3.00	8.00	0.001660	0.017225	464.4
13.0	1785.2	1853.6	1976.0	5.00	13.00	0.002737	0.019962	651.2
18.0	1793.0	1878.8	2017.8	5.00	18.00	0.002698	0.022659	794.4
24.0	1810.4	1901.1	2046.4	6.00	24.00	0.003193	0.025853	928.3
30				6.00	30.00	0.003156	0.029009	1034.2

V_{S30} (m/s)	**1034.2**
Class	**B** (1), (2)

(1) The Site Classes A and B are not to be used if there is 3 metres or more of unconsolidated materials between the rock surface and the bottom of the spread footings or mat foundation.

(2) Some low seismic velocities were calculated from 1 to 2 metres deep. A geotechnical assessment of the corresponding material could be required.

TABLE 2

Limit for the Site Class A

Depth	Vs			Thickness	Cumulative Thickness	Delay for med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	288.7	306.7	392.2	**Limit for the Site Class A (1.3 metres of soil)**				
0.5	203.9	231.4	245.2					
1.0	153.9	175.9	189.3					
1.5	156.4	186.6	205.9					
2.0	189.5	225.3	252.3					
2.2	189.5	225.3	252.3					
2.5	272.6	317.7	376.5	0.30	0.30	0.001332	0.001332	225.3
3.5	1589.4	1692.4	1852.8	1.00	1.30	0.003147	0.004479	290.3
5.0	1690.3	1807.7	1921.8	1.50	2.80	0.000886	0.005365	521.9
8.0	1745.8	1826.9	1922.9	3.00	5.80	0.001660	0.007025	825.7
13.0	1785.2	1853.6	1976.0	5.00	10.80	0.002737	0.009761	1106.4
18.0	1793.0	1878.8	2017.8	5.00	15.80	0.002698	0.012459	1268.2
24.0	1810.4	1901.1	2046.4	6.00	21.80	0.003193	0.015652	1392.8
32.2				8.20	30.00	0.004313	0.019966	1502.6

V_{S30}* (m/s)	**1502.6**
Class	**A**