Geotechnical Investigation

Proposed Six (6) Storey Apartment Complex
374 McArthur Avenue
Ottawa, Ontario

Prepared for:

Castle Heights Development Inc.
286 Athabasca Drive
Maple, Ontario
L6A 3S1

August, 2020

LRL File No.: 200169

TABLE OF CONTENTS

LIST OF TABLES

APPENDICES

Appendix A	**Site and Borehole Location Plans**
Appendix B	**Borehole Logs**
Appendix C	**Symbols and Terms Used in Borehole Logs**
Appendix D	**Laboratory Results**

1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by Stas Dimos of Castle Heights Development Inc. to prepare a geotechnical report for a six (6) storey apartment complex, to be located at 374 McArthur Inc. This report is based off of the field work that was previously carried out on January 27, 2015 by LRL's geotechnical personnel.

The purpose of this mandate was to prepare a geotechnical report based on the findings from the previously completed borehole drilling program. 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 an abandoned 2-storey residential dwelling. The site is rectangular in shape, and has about 20 m of frontage, and a total surface area of about 11,170 m^2. The site is considered to be relatively flat, and is vegetated with overgrown grasses and some mature trees around the perimeter. The site is accessible from McArthur Avenue, and it civically located at 374 McArthur Avenue, Ottawa ON. The location is presented in Figure 1 included in **Appendix A**.

This development will consist of a six (6) storey apartment complex, including some retail space. The apartment building will have a basement level that will consist of underground parking. The apartment buildings will be serviced with municipal services.

3 PROCEDURE

The fieldwork for this investigation was carried out on January 27, 2015. The site was cleared for the presence of any underground services and utilities. A total of four (4) boreholes were drilled across the property to get a generally representation of the site's soil conditions, and labelled BH1 through BH4. The approximate locations of the boreholes are shown in Figure 2 included in **Appendix A**.

The boreholes were advanced using a Geoprobe drill rig equipped with direct push casing, supplied and operated by Downing Drilling. A "two man" crew experienced with geotechnical drilling operated the drill rig and equipment.

Sampling of the overburden materials encountered in the boreholes was carried out at regular depth intervals using a 50.8 mm diameter drive open conventional spoon sampler in conjunction with standard penetration testing (SPT) "N" values. The 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.

The boreholes were terminated at depths ranging from 3.9 to 5.1 m below ground surface (bgs). Upon completion, the boreholes were backfilled and compacted using a combination of bentonite, silica sand, and overburden cuttings.

The fieldwork was supervised throughout by a member of our engineering staff who oversaw the drilling activities, cared for the samples obtained and logged the subsurface conditions encountered within each of the boreholes. All soil samples 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.

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 (top of spindle on fire hydrant across from 374 McArthur (61.77 m)). Ground surface elevations of the boring locations are shown on their respective borehole logs.

4 SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

4.1 General

A review of local surficial geology maps provided by the Department of Energy, Mines and Resources Canada suggest that the surficial geology for this area is glacial till.

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 Topsoil

A thin layer (50mm to 200mm) of topsoil was found at the surface in all boreholes. The topsoil is described as a dark brown sandy loam.

The material was classified as topsoil based on colour and the presence of organic materials and is intended as identification for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustaining plant growth.

4.3 Fill

Underlying the topsoil, a fill material was encountered. The fill is described as fine to medium grained sand with some gravel, stones and organics. Clay was also found to be part of the fill matrix in BH2. It is brown in colour and in a loose state of packing. The fill extends to approximately 0.95 to 1.5 m bgs and was overlying glacial till.

4.4 Glacial Till

Underlying the fill layer, a glacial till deposit was encountered. The glacial till is described as a well graded mixed of silt, sand and gravel with presence of cobbles and traces of clay. It is noted that a distinct sand layer (100mm) was found in BH2 at around 2.8 m. The till is dark grey in colour and in a compact state of packing near its surface and becomes dense to very dense with depth. All boreholes were terminated within the glacial till deposit at depths ranging from 3.94 to 5.10 m bgs.

4.5 Refusal/Bedrock

Refusal over inferred large boulders or bedrock was encountered in BH3 and BH4 at depths of 3.98 m (Elev. 56.62 m) and 3.94 m (Elev. 56.79 m) respectively. No rock coring was performed to confirm the presence of bedrock.

4.6

4.6 Groundwater Conditions

One (1) 19 mm diameter piezometer was installed in BH2 to measure the static groundwater table. This piezometer was measured on January 30, 2015, and the water level was found to be at 1.8 m bgs.

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 on the glacial till deposit. Therefore, all fill, or other deleterious material shall be removed from the building footprint down to the required underside of footing elevation.

5.2

5.2 Shallow Foundation

Conventional strip and column footings set over the native, undisturbed glacial till 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. The bearing capacity is based on a minimum strip footing width of 0.6 m, and a minimum pad footings width of 0.9 m on any side. There are no restrictions for maximum footing sizes and grade raise fill thickness.

In-situ field testing may be required to check the strength and stability of the footings' subgrade. Any incompetent subgrade areas as identified from in-situ testing must be sub-excavated and backfilled with approved structural fill, consisting of OPSS Granular B Type II. Similarly, any soft or wet areas should also be sub-excavated and backfilled with approved structural fill only. Prior to placing any approved structural fill, the subgrade should be inspected and approved by geotechnical engineer or qualified geotechnical personnel. The bearing pressure is contingent on the water level being 0.3

m below the underside footing elevation in order to have a stable and dry subgrade during construction.

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

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 should be placed over undisturbed native soils in layers not exceeding 300 mm and compacted to 98% of its Standard Proctor Maximum Dry Density (SPMDD) within ±2% of its optimum moisture content. In order to allow the spread of load beneath the footings and to prevent undermining during construction, the structural fill should extend minimum 1.0 m beyond the outside edges of the footings and then outward and downward at 1 horizontal to 1 vertical profile (or flatter) over a distance equal to the depth of the structural fill below the footing. Furthermore, the structural fill must be tested to ensure proper compaction.

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

5.5 Retaining Walls and Shoring

The following **Table 1** below provides the suggested soil parameters for the design of retaining walls and/or shoring systems. For excavations near existing services and structures, the coefficient of earth pressure at rest (K_o) should be used.

Table 1: Materials Properties for Shoring and Permanent Wall Design (Static)

Types of Material	Bulk Density (kN/m^3)	Friction angle (degree)	Pressure Coefficient			Combined static and seismic active earth pressure coefficient (K_{AE})
			At Rest (K_O)	Active (K_A)	Passive (K_P)	
Fill	18.0	28	0.48	0.36	2.76	0.48
Glacial Till	22.0	35	0.43	0.27	3.69	0.39
Granular B Type I	20.0	31	0.49	0.32	3.12	0.44
Granular B Type II	23.0	32	0.47	0.31	3.25	0.43
Granular A	23.0	35	0.43	0.27	3.69	0.39

The above values are for a flat surface behind the wall, a straight wall and a wall friction angle of 0 degree. The designer should consider any difference between these coefficients and make appropriate corrections for a sloped surface behind the wall, angled wall or wall friction as required. The bearing capacity for the design of a retaining wall are the same as provided for the building structure provided it is founded over the same soil stratum.

Retaining walls should also be designed to resist the earth pressures produces under seismic conditions. The Canadian Building Code recommends the use of combined coefficients of static and seismic earth pressure, referred to as K_{AE} for active conditions and K_{PE} for passive conditions, for routine design purposes.

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

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

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

Where;

K_{AE} = Combined Static and Seismic Active Earth Pressure Coefficient

K_{PE} = Combined Static and Seismic Passive Earth Pressure Coefficient

H = Total Height of the Wall (m)

K_h = horizontal acceleration coefficient

K_v = vertical acceleration coefficient

γ = bulk density (kg/m^3)

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

A = Zonal acceleration ratio = 0.2

K_h = Horizontal acceleration coefficient = 0.1

K_v = Vertical acceleration coefficient = 0.067

The above value of K_h corresponds to ½ of the A value and the value K_v of corresponds to 0.67 of the K_h value. The angle of friction between the soil and the wall has been set at 0 degrees to provide a conservative estimate. The total active thrust PAE can be divided into a static component, P and a dynamic (seismic) component ΔPAE (i.e. PAE = P+ΔPAE) and may be considered to act at a height, h (m), from the base of the wall,

$$h = [P\ (H/3) + \Delta P_{AE}\ (0.6H)] /\ P_{AE}$$

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

Liquefaction is not a concern for the soils encountered onsite during the field investigation.

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 fill or otherwise deleterious material shall be removed from the proposed building's footprint down to the subgrade surface. The subgrade 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 150 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 Cement Type

A soil sample collected from BH4 (SS3 – 1.5m) was submitted to Paracel Laboratories Ltd., an accredited chemical testing laboratory, for analysis on sulphate content. This sample is representative of the glacial till near the founding depth level. The laboratory analysis revealed a sulphate concentration of 0.0324% (324 µg/g).

Based on the CAN/CSA - A23.1 standards (Concrete Materials and Methods of Concrete Construction), a sulphate concentration of 0.1% (1000 µg/g) or less in soil falls within the negligible category for sulphate attack on buried concrete. As such, buried concrete for footings and foundation walls will not require any special additive to resist sulphate attack and the use of normal Portland cement is acceptable.

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

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation

It is anticipated that the maximum depth of excavation will not extend below 3.0 m bgs. 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.

In the event that the aforementioned slopes are not possible to achieve due to space restrictions, the excavation shall be shored according to OHSA O. Reg. 213/91 and its amendments. The shoring shall be designed from the parameters provided in **Table 1** in **Section 5.5**.

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. 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 are provided to the excavation. Where asphaltic concrete is present, it should be cut back to a minimum of 150 mm from the edge of the excavation to allow for proper compaction between the new and existing pavement structures.

7 REUSE OF ON-SITE SOILS

The existing surficial overburden materials consists of fill and glacial till material. These materials are 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

8 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. The existing sand fill may be acceptable subgrade material if it is properly compacted as outlined herein. The subgrade should be shaped and crowned to promote drainage of the roadway. Following approval of the preparation of the subgrade, the granular subbase may be placed.

It is anticipated that the subgrade for the proposed road will consist of native glacial till or sand fill material. 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;

300 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;

400 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.

8.1 Paved Areas & Subgrade Preparation

The proposed access lanes and parking areas should be stripped of existing asphalt, vegetation, topsoil, debris and other obvious objectionable material. The existing pavement structures should not be reused unless its gradation is tested upon exposure and revealed to meet OPSS standards. Any existing asphaltic concrete could be pulverised with the granular base and could be used as select subgrade material for the new parking areas, if required.

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.

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.

LICENSED PROFESSIONAL ENGINEER
Aug 31, 2020
B. W. JOHNSON
100510537
PROVINCE OF ONTARIO

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 SIX STOREY APARTMENT COMPLEX
374 MCARTHUR AVE.
OTTAWA, ONTARIO

DRAWING TITLE

SITE LOCATION
SOURCE: GEO-OTTAWA

CLIENT

374 MCARTHUR INC

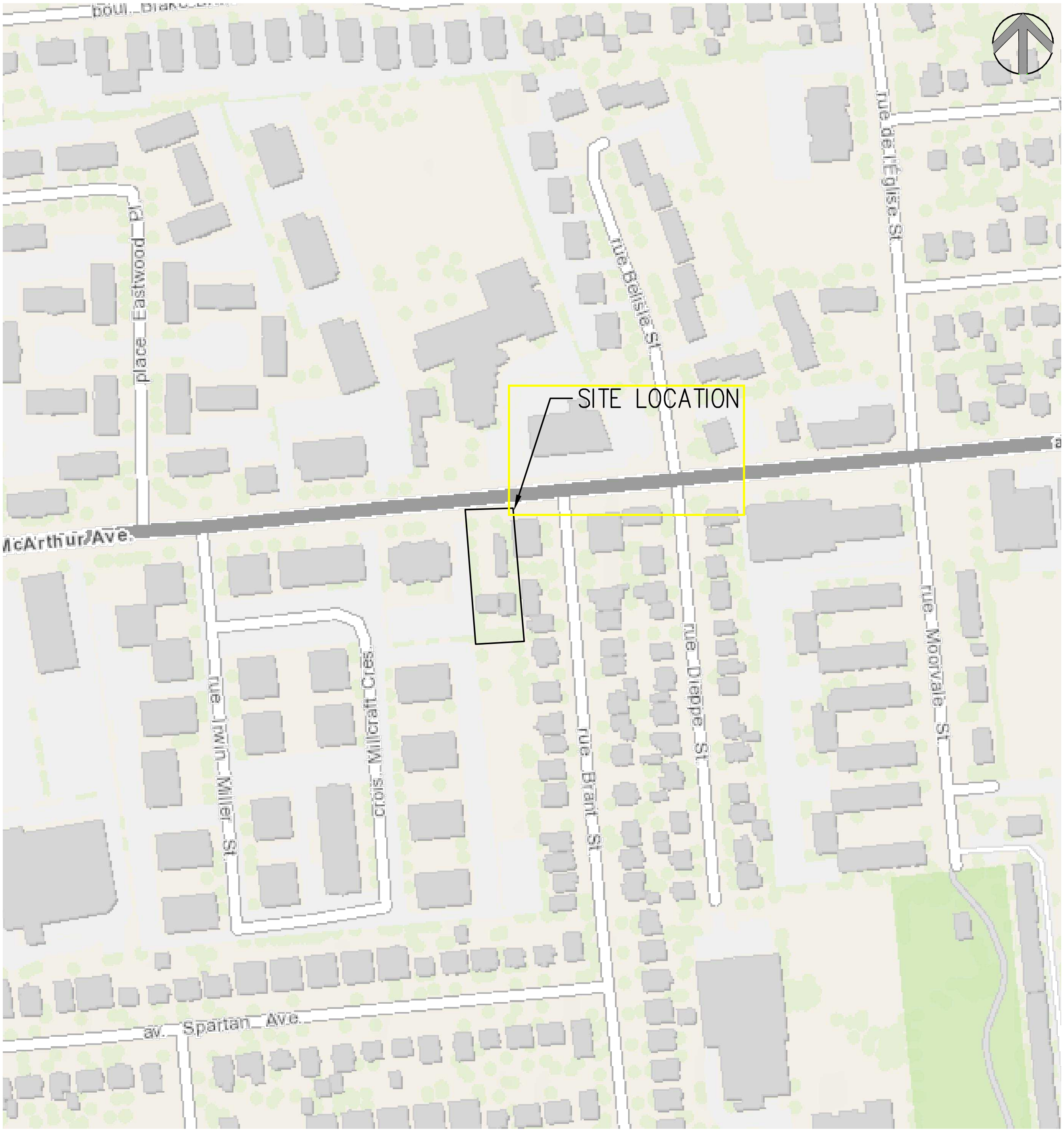
DATE

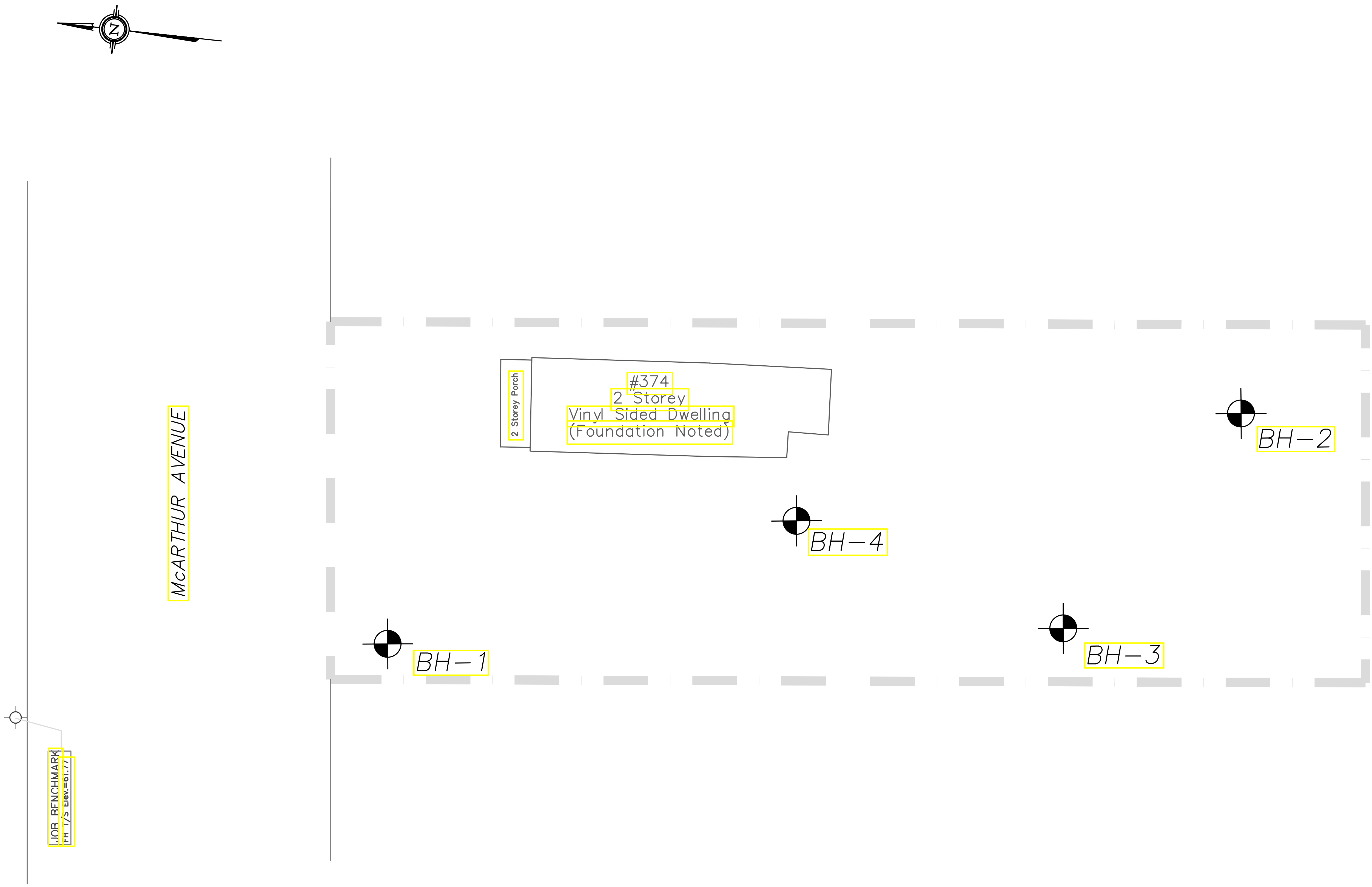
AUGUST 2020

PROJECT

200169

FIGURE 1





LEGEND:

— —	PROPERTY LINES
⌖ BH–3	BOREHOLE LOCATION

No.	REVISIONS	BY	DATE

LRL
ENGINEERING | INGÉNIERIE

5430, Canotek Road
Ottawa, Ontario
K1J 9G2

Tel: (613) 842-3434
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CLIENT
374 MCARTHUR INC.
1291 SQUIRE DRIVE
MANOTICK, ONTARIO K4M 1B3

DESIGNED BY:	DRAWN BY:	APPROVED BY:
M.E.	M.E.	W.B.

PROJECT
GEOTECHNICAL INVESTIGATION
PROPOSED APARTMENT COMPLEX
374 McARTHUR STREET
OTTAWA, ONTARIO

DRAWING TITLE
BOREHOLE LOCATION SKETCH
PAGE 1 of 1

PROJECT NO.
200169

DATE
JAN. 2015

FIGURE 1

APPENDIX B

Borehole Logs

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Borehole Log: BH-1

Project No.: 200169

Project: Proposed Six Storey Apartment Building

Client: 374 McArthur Inc.

Location: 374 McArthur Avenue, Ottawa, Ontario

Date: January 27, 2015

Field Personnel: N.V

Driller: Downing Drilling

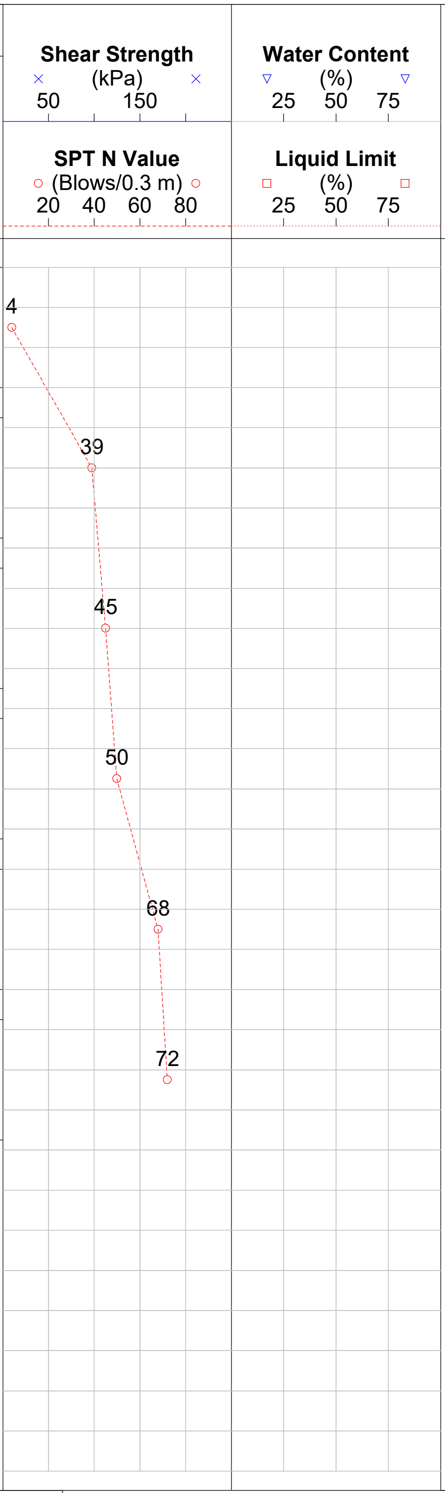
Drilling Equipment: Geoprobe

Drilling Method: Direct Push Casing

SUBSURFACE PROFILE / SAMPLE DATA

Depth ft: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19
Depth m: 0 1 2 3 4 5

Soil Description	Elev./Depth(m)	Lithology	Type	Sample Number	N or RQD	Recovery (%)	Water Level (Standpipe or Open Borehole)
Ground Surface	60.72						
TOPSOIL About 200 mm thick	0.00		⧗	SS1	4	65	
FILL Fine to medium grained sand with some gravel and stones and organics, brown in colour and loose.							
GLACIAL TILL Well graded mixed of silt, sand and gravel with presence of cobbles and traces of clay, dark grey in colour, very dense, moist, becoming wet below 2.0m	59.77 0.95		⧗	SS2	39	60	
			⧗	SS3	45	85	
			⧗	SS4	50	95	
			⧗	SS5	68	100	
			⧗	SS6	72	100	
End of Borehole	56.37 4.35						



Easting: NA

Northing: NA

Site Datum: Geodetic

Groundsurface Elevation: 60.72

Top of Riser Elev.: NA

Hole Diameter: 150mm

NOTES:

Borehole Log: BH-2

Project No.: 200169

Project: Proposed Six Storey Apartment Building

Client: 374 McArthur Inc.

Location: 374 McArthur Avenue, Ottawa, Ontario

Date: January 27, 2015

Field Personnel: N.V

Driller: Downing Drilling

Drilling Equipment: Geoprobe

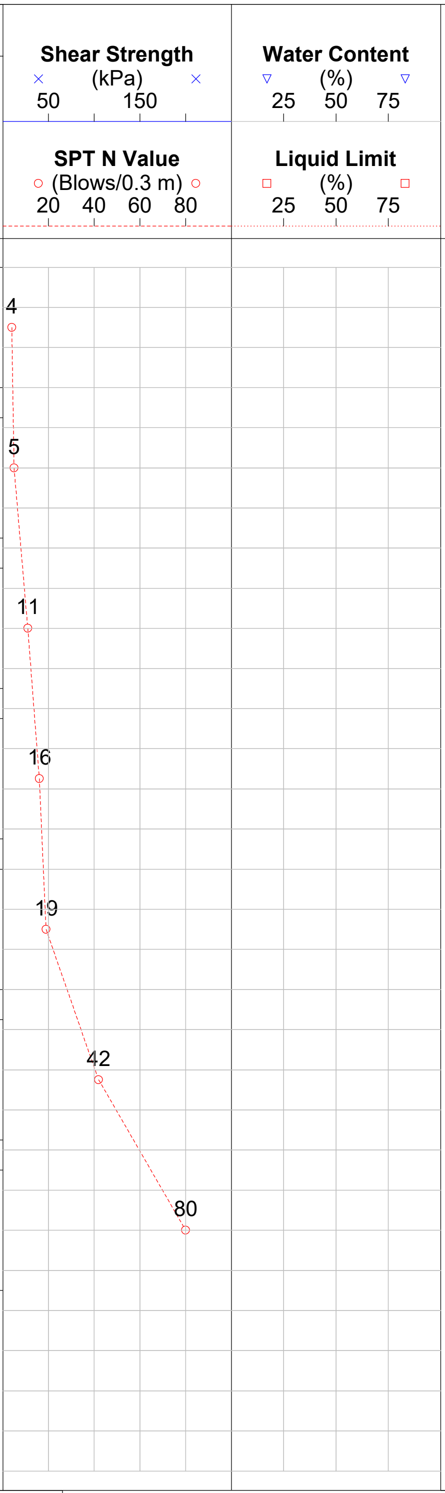
Drilling Method: Direct Push Casing

SUBSURFACE PROFILE

Soil Description	Elev./Depth(m)
Ground Surface	60.45 0.00
TOPSOIL About 200 mm thick	
FILL Mix of clay and sand with some gravel and stones, greyish brown in colour and loose.	
GLACIAL TILL Well graded mixed of silt, sand and gravel with presence of cobbles and traces of clay. A distinct sand layers at around 2.8m. Dark grey in colour, compact becoming very dense with depth, moist, becoming wet below 2.0m	59.40 1.05
End of Borehole	55.35 5.10

SAMPLE DATA

Sample Number	N or RQD	Recovery (%)
SS1	4	65
SS2	5	75
SS3	11	80
SS4	16	55
SS5	19	65
SS6	42	85
SS7	80	100



Water Level (Standpipe or Open Borehole)

1.8m (30-01-15)

Easting: NA

Northing: NA

Site Datum: Geodetic

Groundsurface Elevation: 60.45

Top of Riser Elev.: NA

Hole Diameter: 150mm

NOTES:

Borehole Log: BH-3

Project No.: 200169

Client: 374 McArthur Inc.

Date: January 27, 2015

Project: Proposed Six Storey Apartment Building

Location: 374 McArthur Avenue, Ottawa, Ontario

Field Personnel: N.V

Driller: Downing Drilling

Drilling Equipment: Geoprobe

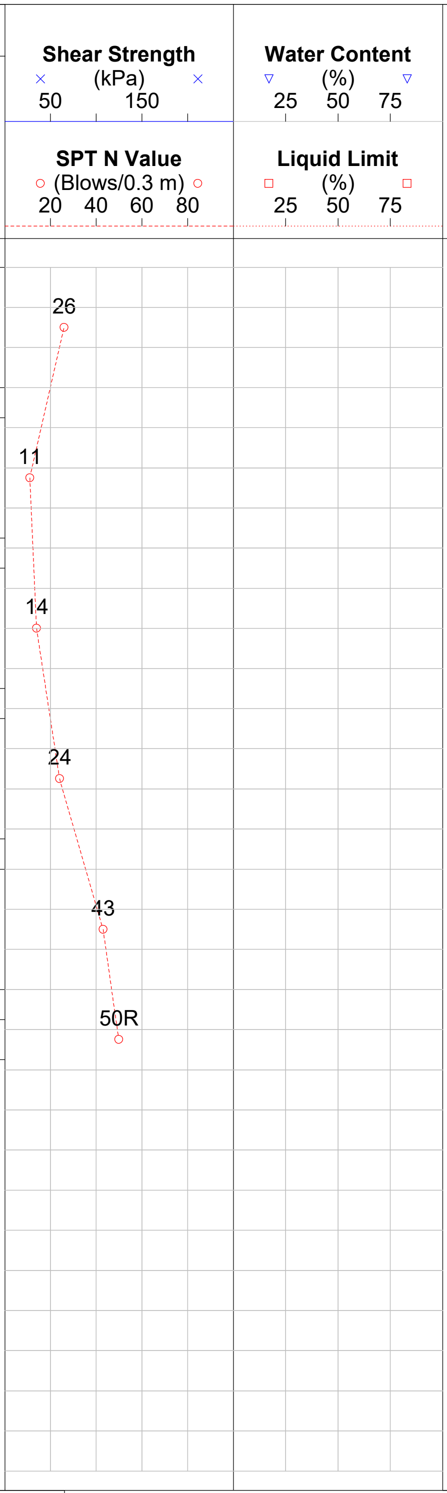
Drilling Method: Direct Push Casing

SUBSURFACE PROFILE

Depth (ft / m)	Soil Description	Elev./Depth(m)
0 / 0	Ground Surface	60.60
	TOPSOIL About 50 mm thick	0.00
	FILL Fine to medium grained sand with some gravel and stones and organics, brown in colour and compact.	
5 / 1.5	**GLACIAL TILL** Well graded mixed of silt, sand and gravel with presence of cobbles and traces of clay, dark grey in colour, compact becoming very dense with depth, moist, becoming wet below 2.0m Borehole refusal over inferred large boulder or bedrock. 50 Blows for 25mm	59.10 1.50
13 / 4	End of Borehole	56.62 3.98

SAMPLE DATA

Sample Number	N or RQD	Recovery (%)
SS1	26	65
SS2	11	75
SS3	14	80
SS4	24	55
SS5	43	65
SS6	50-0.25	85



Water Level (Standpipe or Open Borehole)

Easting: NA

Northing: NA

Site Datum: Geodetic

Groundsurface Elevation: 60.60

Top of Riser Elev.: NA

Hole Diameter: 150mm

NOTES:

Borehole Log: BH-4

Project No.: 200169

Project: Proposed Six Storey Apartment Building

Client: 374 McArthur Inc.

Location: 374 McArthur Avenue, Ottawa, Ontario

Date: January 27, 2015

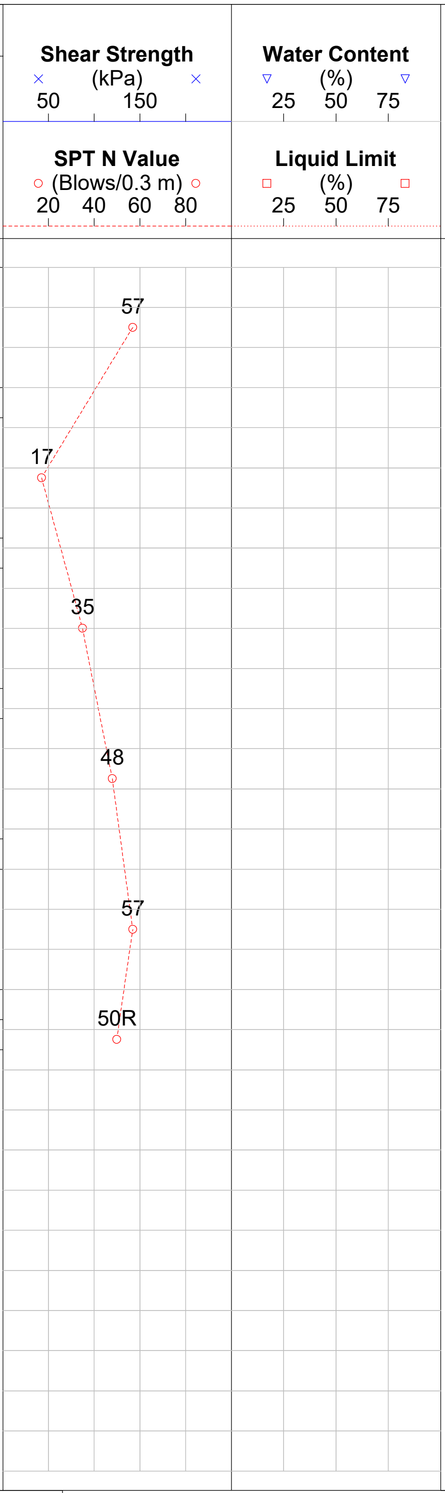
Field Personnel: N.V

Driller: Downing Drilling

Drilling Equipment: Geoprobe

Drilling Method: Direct Push Casing

SUBSURFACE PROFILE		SAMPLE DATA					
Depth (ft / m)	**Soil Description**	**Elev./Depth(m)**	**Lithology**	**Type**	**Sample Number**	**N or RQD**	**Recovery (%)**
0 / 0	Ground Surface	60.73					
	TOPSOIL About 50 mm thick	0.00		⧗	SS1	57	65
	FILL Fine to medium grained sand with some gravel and stones and organics, brown in colour and compact.			⧗	SS2	17	75
	GLACIAL TILL Well graded mixed of silt, sand and gravel with presence of cobbles and traces of clay, dark grey in colour, compact to very dense, moist, becoming wet below 2.0m Borehole refusal over an inferred large boulder or bedrock.	59.23 1.50		⧗	SS3	35	80
				⧗	SS4	48	55
				⧗	SS5	57	65
	50 Blows for 125 mm			⧗	SS6	50-125	85
	End of Borehole	56.79 3.94					



Water Level (Standpipe or Open Borehole)

Easting: NA

Northing: NA

Site Datum: Geodetic

Groundsurface Elevation: 60.73

Top of Riser Elev.: NA

Hole Diameter: 150mm

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

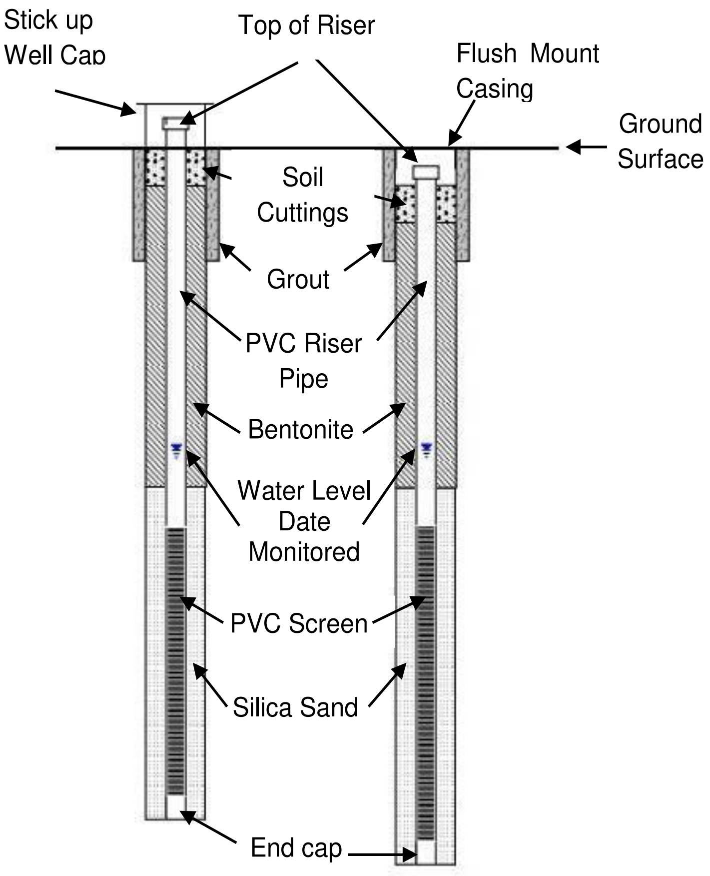
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given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

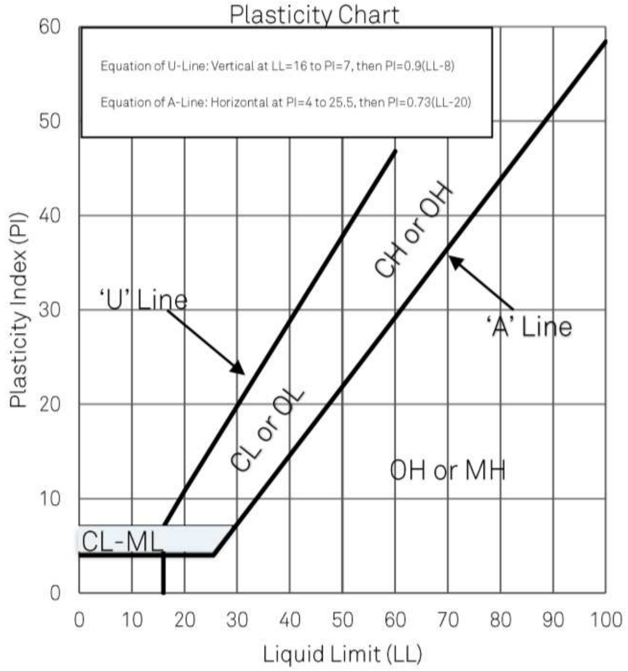
Strength classification of rock is presented below.

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 If fines are organic add "with orgnic fines" to group name
			GC	Clayey gravel			Atterberg limits on or above "A" line and PI > 7	
	Sands 50% or more of coarse fraction passes No. 4 sieve(<4.75 mm)	Clean sands <5% fines	SW	Well-graded sand	If 15% gravel add "with gravel to group name		$C_u = \frac{D_{60}}{D_{10}} \geq 6$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sand			Not meeting either Cu or C ccriteria for SW	
		Sands with >12% fines	SM	Silty sand			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols If fines are organic add "with orgnic fines" to group name
			SC	Clayey sand			Atterberg limits on or above "A" line and PI > 7	
Fine-grained soils50% or more passes No. 200 sieve* (<0.075 mm)	Silts and Clays Liquid Limit <50%	Inorganic	ML	Silt	If 15 to 29% coarse-grained, add "with sand" or "with gravel" as appropriate. If > 30% coarse-grained, add "sandy" or "gravelly" as appropriate. Class as organic when oven dried liquid limit is < 75% of undried liquid limit.			
			CL	Lean Clay -low plasticity				
		Organic	OL	Organic clay or silt (Clay plots above 'A' Line)				
	Silts and Clays Liquid Limit >50%	Inorganic	MH	Elastic silt				
			CH	Fat Clay -high plasticity				
		Organic	OH	Organic clay or silt (Clay plots above 'A' Line)				
	Highly Organic Soils		PT	Peat, muck and other highly organic soils				

APPENDIX D

Laboratory Results

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Certificate of Analysis

LRL Associates Ltd.
5430 Canotek Road
Ottawa, ON K1J 9G2
Attn: Michael Melaney

Phone: (613) 446-7777
Fax: (613) 446-1427

Client PO:
Project: 140805
Custody: 16237

Report Date: 4-Feb-2015
Order Date: 2-Feb-2015
Order #: 1506006

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

Paracel ID	**Client ID**
1506006-01	BH4 SS#3

Approved By: Mark Foto

Mark Foto, M.Sc. For Dale Robertson, BSc
Laboratory Director

Certificate of Analysis

Report Date: 04-Feb-2015
Client: **LRL Associates Ltd.**
Order Date:2-Feb-2015
Client PO:
Project Description: 140805

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	3-Feb-15	3-Feb-15
Solids, %	Gravimetric, calculation	2-Feb-15	2-Feb-15

Certificate of Analysis

Report Date: 04-Feb-2015
Order Date:2-Feb-2015

Client: **LRL Associates Ltd.**
Client PO:
Project Description: 140805

	Client ID:	BH4 SS#3	-	-	-
	Sample Date:	27-Jan-15	-	-	-
	Sample ID:	1506006-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.6	-	-	-
Anions					
Sulphate	5 ug/g dry	324	-	-	-

Certificate of Analysis

Report Date: 04-Feb-2015
Client: **LRL Associates Ltd.**
Order Date:2-Feb-2015
Client PO:
Project Description: 140805

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Sulphate	ND	5	ug/g						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Sulphate	378	5	ug/g dry	324			15.2	20	
Physical Characteristics									
% Solids	87.0	0.1	% by Wt.	84.5			2.9	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Sulphate	41.2		mg/L	32.4	88.0	78-111			

Certificate of Analysis

Report Date: 04-Feb-2015
Order Date:2-Feb-2015

Client: **LRL Associates Ltd.**
Client PO:
Project Description: 140805

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.

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.