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Geotechnical Investigation

Proposed 28-Storey Apartments
593, 601, 603 Laurier Avenue West
Ottawa, Ontario
Revision 2

Prepared for:

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LRL File No.: 240447

October, 2024; *Revised: November 2025*

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1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by Heritage Investments Ltd. to perform a geotechnical investigation for a proposed 28 storey apartment building located at 593, 601, 603 Laurier Avenue West, Ottawa, Ontario.

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

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

2 SITE AND PROJECT DESCRIPTION

The site under investigation currently encompasses a detached three (3) storey residential unit with an attached garage at 593 Laurier Ave W, a 3-storey apartment complex at 601 Laurier Ave W, and a 2-storey apartment complex at 603 Laurier Ave W. The site is irregular in shape, having frontage of approximately 107 m, and a total surface area of about 3,000 m^2. The majority of the site is covered with manicured grasses. The site is considered to have a relatively flat topography. Access to the site comes by way of Laurier Avenue West, and is civically located at 593, 601, and 603 Laurier Avenue West, Ottawa ON. The location is presented in Figure 1 included in **Appendix A**.

It is our understanding that the proposed development will consist of demolishing the existing garage at 593 Laurier Ave W, and the buildings on 601 and 603 Laurier Ave W. New construction will consist of a 28-storey apartment complex, with three (3) levels of underground parking.

3 PROCEDURE

The fieldwork for this investigation was carried out on September 03, 2024; in conjunction with the Phase II Environmental Site Assessment (ESA). Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of five (5) boreholes, labelled BH24-01 through BH24-05, were drilled across the property, where possible to do so, to get a general representation of the site's subsurface conditions. The approximate locations of the boreholes are shown in Figure 2 included in **Appendix A**.

The boreholes were advanced using a track mount CME 75 drill rig equipped with 200 mm diameter continuous flight hollow stem auger supplied and operated by George Downing Estate 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 SPT were conducted following the method **ASTM D1586** and the results of SPT, in terms of the number of blows per 0.3 m of split-spoon sampler penetration after first 0.15 m designated as "N" value.

All boreholes were advanced until practical auger refusal over inferred bedrock, two (2) of the boreholes consisted of HQ-size (Ø63.5mm) rock coring. The boreholes were terminated at depths ranging from 0.36 to 10.70 m below ground surface (bgs).

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 and rock cores were transported bag 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). Existing grade elevations at the borehole locations were determined by a topographic field survey using a temporary site bench mark; taken as the flange of on the fire hydrant located south of Laurier Avenue W, and assumed to have an elevation of 100.00 m. Ground surface elevations of the boring locations are shown on their respective borehole logs.

4 SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

4.1 General

A review of local surficial geology maps provided by the Department of Energy, Mines and Resources Canada suggest that the surficial geology for this area consist of bedrock, consisting of limestone with shaly partings.

The subsurface conditions encountered in the boreholes were classified based on visual and tactile examination of the materials recovered from the boreholes. The soil descriptions presented in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil were conducted according to the procedure **ASTM D2487** and judgement, and LRL does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

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

4.2 Pavement Structure

At the surface of boring location BH24-01, a pavement structure was encountered. This consisted of a 50 mm thick layer of asphaltic concrete, overlying a 310 mm thick layer of crushed stone granular material.

4.3 Fill

At the surface of all boring locations, a layer of fill material was encountered, and extended to depths ranging between 0.48 and 1.02 m bgs. (depths of auger reusal). Generally the fill material can be described as a brown silty to sand material mixed with some gravel. Standard penetration tests were carried out in the fill material and the SPT “N” values were found ranging from 16 to 56, indicating it is compact to very dense. The natural moisture content was found varying between 2 and 11%

4.4 Refusal/Bedrock

Underlying the pavement structure in BH24-01, and the fill material in BH24-02 through BH24-05, refusal over bedrock encountered. The bedrock was encountered at depths, ranging from 0.48 – 1.02 m bgs.

The Rock Quality Designation (RQD) was determined after the rock was cored, this is done by summing the lengths of the intact recovered cores which are greater than 100 mm in length, and dividing by the total length of the core run. The RQD values, expressed as a percent, ranged from 17 to 98%, indicating the rock was very poor to excellent quality.

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

Two (2) rock core samples were selected to determine the unconfined compressive strengths at various depths. The results are summarized below in **Table 1.**

Table 1: Unconfined Compressive Strength of Select Rock Cores

Sample			Bedrock Type	Strength (MPa)
Borehole	Core ID	Depth (m)		
BH24-02	R1	10.1	Limestone	94.9
BH24-05	R2	9.1	Limestone	90.1

4.5 Groundwater Conditions

Groundwater was carefully monitored during this field investigation. No water was encountered within the overburden during the borehole drilling.

The monitoring well was measured from BH24-05 on September 11, 2024, and water found to be at 7.11 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 in 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 building be founded over bedrock, or structural fill overlying the bedrock. Therefore, all other material should be removed from the building's footprint down to the sound bedrock.

5.2 Conventional Foundation on Bedrock

Conventional strip and column footings set over sound bedrock may be designed using a maximum allowable bearing pressure of **3,000 kPa** for Ultimate Limit State **(ULS)** factored bearing resistance. Serviceability Limit State **(SLS)** does not apply for footings founded on bedrock since failure of the concrete would occur before unacceptable settlement of the foundation. For footings founded on sound bedrock, there are no restrictions for maximum footing sizes and grade raise fill thickness. Prior to pouring the footing, the rock should be free of any soil, debris or deleterious substances and should be inspected by a geotechnical engineer.

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

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

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

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

An unfactored friction coefficient value of 0.6 between concrete and bedrock materials may be considered when calculating the sliding resistance between the two (2) materials.

5.2.1 Rock Anchors

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

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

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

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

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

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

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

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

Where;

P = Working Capacity of anchor (kg);

τ_b = working bond stress (kg/m^2);

d = Core hole diameter (m).

5.3 Shallow Foundation on Structural Fill

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

Prior to placing the approved structural fill, the subgrade at bedrock level should be inspected and assessed by a geotechnical engineer, or a representative to identify any localised incompetent/unstable areas of the subgrade. Any incompetent subgrade areas as identified must be sub-excavated and backfilled with approved structural fill and compacted to 98% of its SPMDD. In order to allow the spread of load beneath the footings and to prevent undermining during construction, the structural fill should extend minimum 1.0 m beyond the outside edges of the footings and then outward and downward at 1 horizontal to 1 vertical profile (or flatter) over a distance equal to the depth of the structural fill below the footing.

5.4 Bedrock/Deep Excavation

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

Furthermore, the proposed works are located near a 1524 mm high-pressure watermain; which is sensitive to settlement and construction-induced vibrations. To mitigate potential impacts, excavation support should be designed to limit settlement and lateral movement. Low-vibration methods should be employed where feasible.

It is recommended that a pre-condition survey of the watermain be completed, and monitoring (settlement and vibration) be implemented during construction. The below **Table 2,** as outlined in **OPSS 120** are the vibration limits for the watermain, and any other adjacent structures. Any exceedances should trigger immediate review of construction method.

Table 2: Vibration Frequency and Limit

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

5.5 Lateral Earth Pressure

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

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

Where;

P = Earth pressure at depth h;

K = Appropriate coefficient of earth pressure;

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

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

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

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

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

Table 3 below provides various material types and their respective earth pressure properties.

Table 3: Material and Earth Pressure Properties

Type of Material	Bulk Density (kN/m^3)	Friction Angle (Φ)	Pressure Coefficient			Combined Static & Seismic Active Earth Pressure Coefficient (K_{AE})
			At Rest (K_0)	Active (K_A)	Passive (K_P)	

Granular A	23.0	34	0.44	0.28	3.53	0.40
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
Limestone Bedrock	25.0	35	0.43	0.27	3.69	0.39

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 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 "A"** as per the Site Classification for Seismic Site Response.

5.8 Liquefaction Potential

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

5.9 Frost Protection

For foundations set directly on sound bedrock, free of cracks and fractures, the need for frost protection is not required.

5.10 Foundation Walls Backfill (Shallow Foundations)

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 II, Type I, or Select Subgrade Material (SSM).

The foundation wall backfill should be compacted to minimum 95% of its SPMDD using light compaction equipment, where no loads will be set over top. The compaction shall be increased to 98% of its SPMDD under walkways, slabs or paved areas close to the foundation or retaining walls. Backfilling against foundation walls should be carried out on both sides of the wall at the same time where applicable.

5.11 Slab-on-grade Construction

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

Any underfloor fill needed to raise the general floor grade shall consist of OPSS Granular B Type II material or an approved equivalent, compacted to 95% of its SPMDD. The final lift shall be compacted to 98% of its SPMDD. A 200 mm Granular A meeting the **OPSS 1010** shall be placed underneath the slab and compacted to 100% of its SPMDD.

It is also recommended that the area of extensive exterior slab-on-grade (sidewalks, ramp etc.) shall be constructed using Granular B subbase of thickness 300 mm and Granular A base of thickness 150 mm with incorporating subdrain facilities. The modulus of subgrade reaction (ks) for the design of the slabs set over structural fill is **24 MPa/m.**

In order to further minimize and control cracking, the floor slab shall be provided with wire or fibre mesh reinforcement and construction or control joints. The construction or control joints should be spaced equal distance in both directions and should not exceed 4.5 m. The wire or fibre mesh reinforcement shall be carried out through the joints.

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 Basement Slab Construction

Basement floor slabs shall be founded on a minimum of 200 mm thick layer of 19 mm clear stone meeting the **OPSS 1004** gradation requirements should be placed.

An under-floor drainage system with an invert located a minimum of 300 mm below the underside of basement slab is recommended to be installed. This shall be comprised of 100 mm diameter weeping tile pre-wrapped with geotextile knitted sock, embedded in a 150 mm layer of 19 mm clear stone. It should installed in one direction below the slab and connected to a sump/frost-free outlet of the exterior weeping tile from which water is pumped to the nearby ditches or storm sewer line, if available.

Proper moisture barrier with vapour retarder should be used for any slab on grade where the floor will be covered by moisture sensitive flooring materials/equipment or environment will exist.

5.13 Impact on Neighbouring Structures

It is understood the existing structures to the north of the site (retaining wall between property lines and building located at 140 Bronson Ave) have the potential to experience lateral loading from the proposed development planned for this site. Due to the difference in grades between the site and 140 Bronson Ave, the underside of footing (USF) is proposed to be at a higher elevation than the expected USF for 140 Bronson Ave and the retaining wall.

Foundation loading will be transferred downward to the bedrock surface at a 1:1 ratio. Therefore, adjacent structures could potentially be impacted by a lateral load from the proposed apartment complex if the horizontal distance between the foundations is equal to or less than the difference in elevations of the USF between the proposed apartment complex and 140 Bronson Ave.

This area shall be equipped with 100 mm diameter weeping tile, connected to a suitable outlet to ensure water is not ponding in the vicinity of the adjacent structure at 140 Bronson Ave.

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation and Shoring

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 at this site can be classified as Type 3. Therefore, shallow temporary excavations in overburden soil classified as Type 3 can be cut at 1 horizontal to 1 vertical (1H: 1V), for a fully drained excavation starting at the base of the excavation and as per requirements of the OHSA regulations.

Excavation side walls in sound bedrock can be excavated vertically, and sloping would not be required. It is recommended to bench the overburden material 1.0 m wide to avoid any cave-in of the overburden material into the excavation. Considering the thin layer of overburden overlying the bedrock, slope stability is not a concern for this site.

To ensure no bedrock fragments become loose from vibration during the construction phase, it is recommended to protect the bedrock side walls. Two possible options to protect the excavation walls are as follows:

- Protective Wire Mesh: Protective wire mesh is commonly used in construction excavations to manage the risk of loose rock or soil fragments detaching from exposed side walls. The mesh acts as a barrier to contain and control debris, thereby improving site safety and protecting workers and equipment at the excavation base. The mesh is draped and tensioned against the excavation face, anchored with rock anchors at the crest. Considering the low RQD values of the bedrock, it is also recommended to anchor at intermediate points, every 2 to 3 m.
- Shotcrete: Shotcrete is another viable option used to stabilize excavation walls in rock, providing both immediate surface support and longer-term protection. It is applied pneumatically onto the excavation face, where it bonds to the substrate and creates a continuous protective layer. A minimum thickness of 100 mm is recommended for this site.

In lieu of wire mesh and shotcrete, the excavation may also be shored. The design and approval of the shoring shall be the responsibility of the shoring contractor and subsequent shoring designer retained for the project. It shall be designed to ensure the shoring complies with safety measures, and appropriate dewatering measures.

The shoring is recommended to be designed using the Rock Mass Rating (RMR) system. This is used to quantify rock mass quality and guide the excavation and support design. RMR takes into account the intact rock strength, RQD, spacing and condition of discontinuities, and groundwater. The rating system produces a numerical rating value based on the above-mentioned parameters.

The RMR for rock on this site can be taken as 72, which falls in the "Good Rock" range (61-80).

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

As mentioned above in **Section 4.5,** groundwater was encountered at a depth of 7.11 m bgs. Based on this information, groundwater seepage or infiltration into shallow temporary excavations during construction should be minor in nature, and will be able to be pumped out with open sumps. 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 anticipated that pumping rates will be less than 50,000 litres per day. As such, EASR registration is not expected to be required for the construction at this site. **However, the exact pumping volumes could be confirmed by carrying out a Hydrology Study for this site.**

If EASR is required, a groundwater management plan generated by the contractor shall be submitted for review.

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 approved equivalent, laid in loose lifts of thickness not exceeding 300 mm and compacted to 95% of its SPMDD. Bedding, thickness of cover material and compaction requirements for watermains and sewer pipes should conform to the manufacturer's design requirements and to the detailed installations outlined in the Ontario Provincial Standard Specifications (OPSS) or any other applicable standards.

6.4 Trench Backfill

All service trenches should be backfilled using compactable material, free of organics, debris and large cobbles or boulders. Acceptable native materials (if encountered and where possible) should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetrations (i.e. 1.8 m below finished grade) in order to reduce the potential for differential frost heaving between the new excavated trench and the adjacent section of roadway. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type II. Any boulders larger than 150 mm in size should not be used as trench backfill.

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

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

7 Slope Stability Analysis

For this site, a slope stability analysis is not required, and global slope stability is not a concern for this site. The elevation changes to the east and the north of the property limits are constrained by a retaining wall, overlying bedrock, which is relatively resistant to erosion; thus, will remain stable. However, it is recommended for a structural engineer to assess to integrity of the wall on the east side of the property, as well as the retaining wall to the rear of the property at 140 Bronson Ave.

8 Reuse of On-Site Soils

The existing surficial overburden soils consist mostly of topsoil and fill material. This is considered to be highly organic and frost susceptible and should not be used as backfill material directly against foundation walls or underneath unheated concrete slabs. Any imported material shall conform to OPSS Granular B – Type II or approved equivalent.

It should be noted that the adequacy of any material for reuse as backfill will depend on its water content at the time of its use and on the weather conditions prevailing prior to and during that time. Therefore, all excavated materials to be reused shall be stockpiled in a manner that will prevent any significant changes in their moisture content, especially during wet conditions. Any excavated materials proposed for reuse should be stockpiled in a manner to promote drying and should be inspected and approved for reuse by a geotechnical engineer.

9 Recommended Pavement Structure

It is anticipated that the subgrade soil for the new parking areas and access lanes will consist mostly of fill material or bedrock. The construction of access lanes and parking areas will be acceptable over these materials, once all debris, organic material, or otherwise deleterious material are removed from the subgrade area. Furthermore, the subgrade must be compacted (with the exception where the subgrade consists of bedrock) using a suitable heavy duty compacting equipment and approved by a geotechnical engineer prior to placing any granular base material.

The following **Table 4** presents the recommended pavement structures to be constructed over a stable subgrade along the proposed parking areas and access lane or driveway as part of this project.

Table 4: Recommended Pavement Structure

Course	Material	Thickness (mm)	
		Light Duty Parking Area (mm)	Heavy Duty Parking Area (Access Roads, Fire Routes and Trucks) (mm)
Surface	HL3 A/C	50	40

Binder	HL8 A/C	-	50
Base course	Granular A	150	150
Sub base	Granular B Type II	350	450
Total:		500	690

Performance Graded Asphaltic Cement (PGAC) **58-34** is recommended for this project.

If the subgrade level is bedrock, the Granular B Type II thickness may be reduced to 300 mm for both light and heavy duty surfaces.

The base and subbase granular materials shall conform to **OPSS 1010** material specifications. Any proposed materials shall be tested and approved by a geotechnical engineer prior to delivery to the site and shall be compacted to 100% of its SPMDD. Asphaltic concrete shall conform to **OPSS 1150** and be placed and compacted to at least 93% of the Marshall Density. The mix and its constituents shall be reviewed, tested and approved by a geotechnical engineer prior to delivery to the site.

9.1 Paved Areas & Subgrade Preparation

The access lanes and parking areas shall be stripped of top soil, vegetation, debris and other obvious objectionable material. Following the backfilling and satisfactory compaction of any underground service trenches up to the subgrade level, the subgrade shall be shaped, crowned and proof-rolled. A loaded Tandem axle, dual wheel dump truck or approved equivalent heavy duty smooth drum roller shall be used for proof-rolling. Any resulting loose/soft areas should be sub-excavated down to an adequate bearing layer and replaced with approved backfill.

The preparation of subgrade shall be scheduled and carried out in manner so that a protective cover of overlying granular material (if required) is placed as quickly as possible in order to avoid unnecessary circulation by heavy equipment, except on unexcavated or protected surfaces. Frost protection of the surface shall be implemented if works are carried out during the winter season.

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

10 Inspection Services

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

All footing areas and any structural fill areas for the proposed building should be inspected by LRL to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundations and slab-on-grade should be inspected to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for the pavement areas and underground services should be inspected and approved by geotechnical personnel. In-situ density testing should be carried out on the pavement granular materials, pipe bedding and backfill to ensure the materials meet the specifications for required compaction.

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

11 REPORT CONDITIONS AND LIMITATIONS

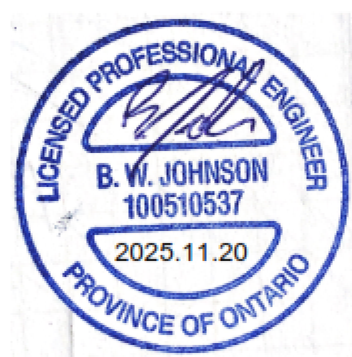
It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document or its use by a third party beyond the client specifically listed in the report is neither intended nor authorized 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

W:\FILES 2024\240447\05 Geotechnical\01 Investigation\05 Reports_2025.09.19.Geotechnical Investigation_593 601 603 Laurier Ave W_R1

APPENDIX A

Site and Borehole Location Plan

LRL

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

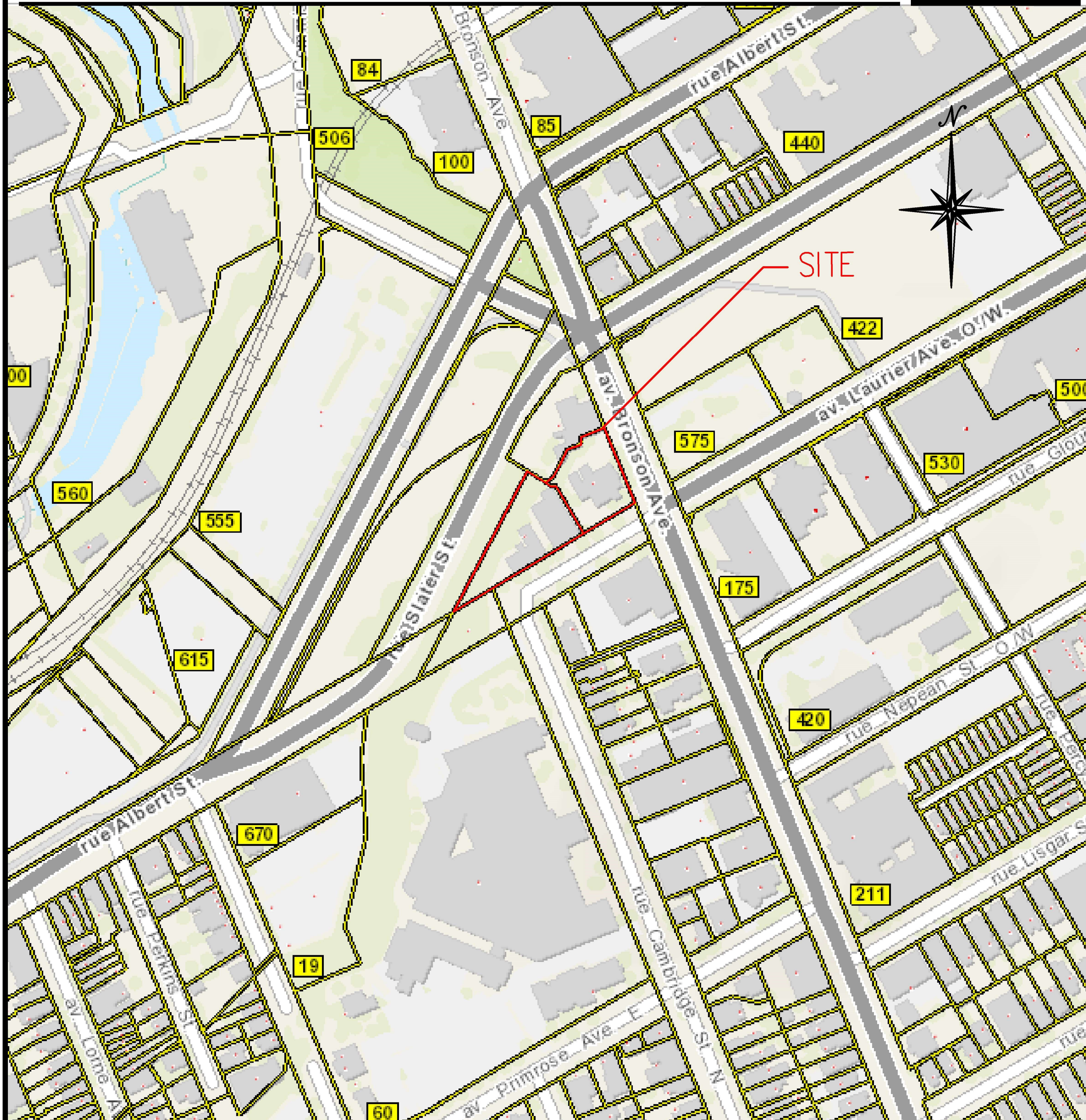
PROJECT

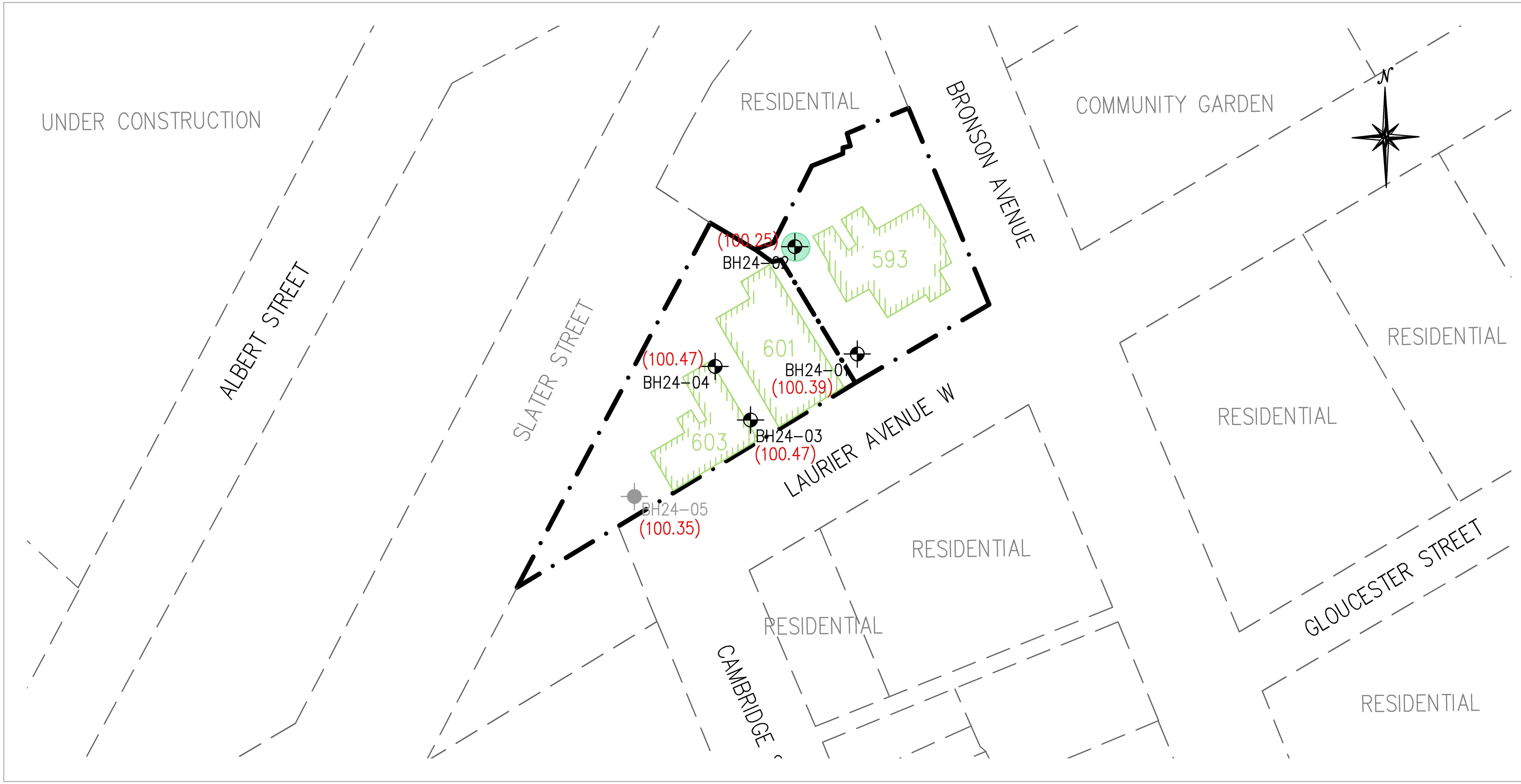
GEOTECHNICAL INVESTIGATION
PROPOSED 28-STOREY APARTMENT
593, 601 & 603 LAURIER AVENUE W,
OTTAWA, ONTARIO

DRAWING TITLE

SITE LOCATION
(NOT TO SCALE)
SOURCE: GEOOTTAWA

CLIENT	DATE	PROJECT	
HERITAGE INVESTMENTS LTD.	OCTOBER 2024	240447	FIGURE1





Legend

- Subject Sites
- Neighbouring Property Extents
- Existing Buildings
- (XX.XX) Ground Surface Elevation
- BH24-X Newly Advanced Borehole (2024)

10m 0 10 20 30m

SCALE: 1:750

No.	REVISIONS	BY	DATE
01	FINAL	BJ	10/22/2024

LRL

ENGINEERING | INGÉNIERIE

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

CLIENT

HERITAGE INVESTMENTS LTD.

DESIGNED BY:	DRAWN BY:	APPROVED BY:
--	BJ	BJ

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED 28-STOREY APARTMENT
593, 601 & 603 LAURIER AVENUE W
OTTAWA, ONTARIO

DRAWING TITLE

BOREHOLE LOCATION

PROJECT NO.
240447

DATE
OCTOBER 2024

FIGURE2

APPENDIX B

Borehole Logs

BOREHOLE LOG: BH24-01

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

PROJECT NO.: 240447

CLIENT: HERITAGE INVESTMENTS LTD.

DATE: SEPTEMBER 3, 2024

PROJECT: GEOTECHNICAL INVESTIGATION

LOCATION: 593, 601 & 603 LAURIER AVENUE WEST

FIELD PERSONNEL: ERIC LAVERGNE

DRILLER: GEORGE DOWNING ESTATE DRILLING LTD. **DRILLING EQUIPMENT:** TRUCK MOUNTED CME-55 **DRILLING METHOD:** SPLIT-BARREL AUGER

DEPTH (FT)	DEPTH (M)	SOIL DESCRIPTION	ELEV./DEPTH (m)	LITHOLOGY	TYPE	SAMPLE NUMBER	N OR RQD (%)	RECOVERY (%)	SPT N VALUE (Blows/0.3 m) 20 40 60 80	WATER CONTENT (%) 20 40 60 80	MONITORING WELL DETAILS
0.0	0.0		100.39								
		ASPHALT:	0.00								
1.0		**FILL:** Gravel, crushed stone, dry, gray.	0.05			SS1	50+	2	50+	1	
		End of Borehole	100.03 0.36								
2.0		Borehole terminated after auger refusal									
3.0	1.0										
4.0											
5.0											
6.0											
7.0	2.0										
8.0											
9.0											
10.0	3.0										
11.0											
12.0											
13.0	4.0										
14.0											
15.0											
16.0	5.0										
17.0											
18.0											
19.0											
20.0	6.0										

EASTING: 0444584 **NORTHING:** 5029286

SITE DATUM: Elevations measured from temporary benchmark established at the northeast upper rim of the fire hydrant located south of Laurier Avenue West (100.00 m).

GROUNDSURFACE ELEVATION: 100.39 m **TOP OF RISER ELEVATION:** --

HOLE DIAMETER: 50 mm **MONITORING WELL DIAMETER:** --

NOTES:
bgs: Below Ground Surface
N/A: Not applicable

BOREHOLE LOG: BH24-02

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

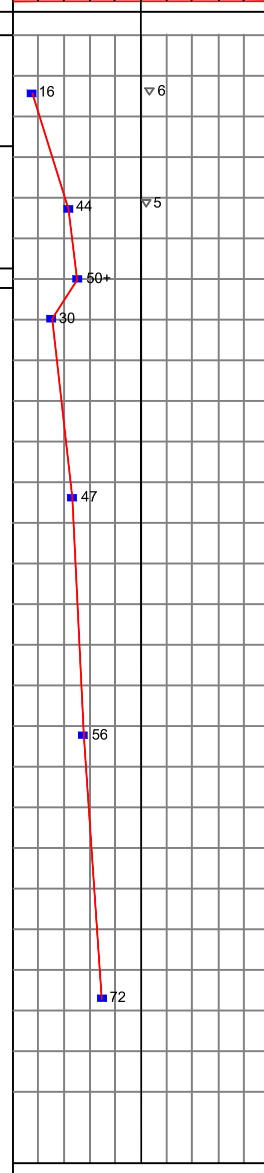
PROJECT NO.: 240447
PROJECT: GEOTECHNICAL INVESTIGATION
CLIENT: HERITAGE INVESTMENTS LTD.
LOCATION: 593, 601 & 603 LAURIER AVENUE WEST
DATE: SEPTEMBER 3, 2024
FIELD PERSONNEL: ERIC LAVERGNE

DRILLER: GEORGE DOWNING ESTATE DRILLING LTD. **DRILLING EQUIPMENT:** TRUCK MOUNTED CME-55 **DRILLING METHOD:** SPLIT-BARREL AUGER

DEPTH (FT / M)	SOIL DESCRIPTION	ELEV./DEPTH (m)	LITHOLOGY	TYPE	SAMPLE NUMBER	N OR RQD (%)	RECOVERY (%)
0.0 / 0.0	**FILL:** Sandy soil Gravel and rock with stone dust, brown-grey, dry.	100.25 / 0.00			SS1	16	15
					SS2	44	9
					SS3	50+	0
	BEDROCK	96.24 / 3.81				30	
						47	
						56	
						72	

Depth scale: FT 0.0–20.0; M 0.0–6.0

SPT N VALUE / RQD VALUE (Blows/0.3 m): 20 40 60 80
WATER CONTENT (%): 20 40 60 80
MONITORING WELL DETAILS



EASTING: 0444577 **NORTHING:** 5029309

SITE DATUM: Elevations measured from temporary benchmark established at the northeast upper rim of the fire hydrant located south of Laurier Avenue West (100.00 m).

GROUNDSURFACE ELEVATION: 100.25 m **TOP OF RISER ELEVATION:** --

HOLE DIAMETER: 50 mm **MONITORING WELL DIAMETER:** --

NOTES:
bgs: Below Ground Surface
N/A: Not applicable

BOREHOLE LOG: BH24-02

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

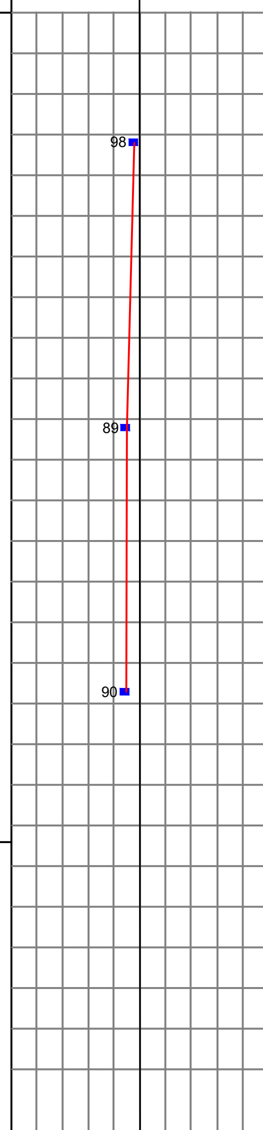
PROJECT NO.: 240447
CLIENT: HERITAGE INVESTMENTS LTD.
DATE: SEPTEMBER 3, 2024

PROJECT: GEOTECHNICAL INVESTIGATION
LOCATION: 593, 601 & 603 LAURIER AVENUE WEST
FIELD PERSONNEL: ERIC LAVERGNE

DRILLER: GEORGE DOWNING ESTATE DRILLING LTD. **DRILLING EQUIPMENT:** TRUCK MOUNTED CME-55 **DRILLING METHOD:** SPLIT-BARREL AUGER

DEPTH FT	DEPTH M	SOIL DESCRIPTION	ELEV./DEPTH (m)	LITHOLOGY	TYPE	SAMPLE NUMBER	N OR RQD (%)	RECOVERY (%)	SPT N VALUE / RQD VALUE (Blows/0.3 m) 20 40 60 80	WATER CONTENT (%) 20 40 60 80	MONITORING WELL DETAILS
20.0	6.1		94.15 6.1								
22.0							98		98		
23.0	7.0										
26.0	8.0										
27.0							89		89		
29.0	9.0										
32.0							90		90		
33.0	10.0										
35.0		End of Borehole	89.55 10.70								
36.0	11.0										
39.0	12.0										
40.0											

Depth scale (FT): 20.0, 21.0, 22.0, 23.0, 24.0, 25.0, 26.0, 27.0, 28.0, 29.0, 30.0, 31.0, 32.0, 33.0, 34.0, 35.0, 36.0, 37.0, 38.0, 39.0, 40.0



EASTING: 0444577 **NORTHING:** 5029309
SITE DATUM: Elevations measured from temporary benchmark established at the northeast upper rim of the fire hydrant located south of Laurier Avenue West (100.00 m).
GROUNDSURFACE ELEVATION: 100.25 m **TOP OF RISER ELEVATION:** --
HOLE DIAMETER: 78 mm **MONITORING WELL DIAMETER:** --

NOTES:
bgs: Below Ground Surface
N/A: Not applicable

BOREHOLE LOG: BH24-03

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

PROJECT NO.: 240447
CLIENT: HERITAGE INVESTMENTS LTD.
DATE: SEPTEMBER 3, 2024
PROJECT: GEOTECHNICAL INVESTIGATION
LOCATION: 593, 601 & 603 LAURIER AVENUE WEST
FIELD PERSONNEL: ERIC LAVERGNE

DRILLER: GEORGE DOWNING ESTATE DRILLING LTD. **DRILLING EQUIPMENT:** TRUCK MOUNTED CME-55 **DRILLING METHOD:** SPLIT-BARREL AUGER

DEPTH (FT / M)	SOIL DESCRIPTION	ELEV./DEPTH (m)	LITHOLOGY	TYPE	SAMPLE NUMBER	N OR RQD (%)	RECOVERY (%)	SPT N VALUE (Blows/0.3 m) 20 40 60 80	WATER CONTENT (%) 20 40 60 80	MONITORING WELL DETAILS
0.0 / 0.0	**FILL:** Silty soil, brown, dry for 0.13 m. Gravel with stone dust, gray, dry.	100.47 / 0.00			SS1	50+	8	50+	3	
	End of Borehole Borehole terminated after auger refusal	99.99 / 0.48								

Depth scale: FT 0.0 – 20.0; M 0.0 – 6.0

EASTING: 0444584
NORTHING: 5029286
SITE DATUM: Elevations measured from temporary benchmark established at the northeast upper rim of the fire hydrant located south of Laurier Avenue West (100.00 m).
GROUNDSURFACE ELEVATION: 100.47 m
TOP OF RISER ELEVATION: --
HOLE DIAMETER: 50 mm
MONITORING WELL DIAMETER: --

NOTES:
bgs: Below Ground Surface
N/A: Not applicable

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BOREHOLE LOG: BH24-04

PROJECT NO.: 240447
CLIENT: HERITAGE INVESTMENTS LTD.
DATE: SEPTEMBER 3, 2024

PROJECT: GEOTECHNICAL INVESTIGATION
LOCATION: 593, 601 & 603 LAURIER AVENUE WEST
FIELD PERSONNEL: ERIC LAVERGNE

DRILLER: GEORGE DOWNING ESTATE DRILLING LTD.
DRILLING EQUIPMENT: TRUCK MOUNTED CME-55
DRILLING METHOD: SPLIT-BARREL AUGER

DEPTH (FT / M)	SOIL DESCRIPTION	ELEV./DEPTH (m)	LITHOLOGY	TYPE	SAMPLE NUMBER	N OR RQD (%)	RECOVERY (%)	SPT N VALUE (Blows/0.3 m)	WATER CONTENT (%)	MONITORING WELL DETAILS
0.0 / 0.0	**FILL:** Silty soil, brown, dry for 0.10 m. Crushed stone and gravel, gray, dry.	100.47 / 0.00			SS1	50+	8	50+	2	
					SS2	50+	0	50+		
	End of Borehole Borehole terminated after auger refusal	99.61 / 0.86								

EASTING: 0444515
NORTHING: 5029275
SITE DATUM: Elevations measured from temporary benchmark established at the northeast upper rim of the fire hydrant located south of Laurier Avenue West (100.00 m).
GROUNDSURFACE ELEVATION: 100.47 m
TOP OF RISER ELEVATION: --
HOLE DIAMETER: 50 mm
MONITORING WELL DIAMETER: --

NOTES:
bgs: Below Ground Surface
N/A: Not applicable

BOREHOLE LOG: BH24-05

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PROJECT NO.: 240447
CLIENT: HERITAGE INVESTMENTS LTD.
DATE: SEPTEMBER 3, 2024

PROJECT: GEOTECHNICAL INVESTIGATION
LOCATION: 593, 601 & 603 LAURIER AVENUE WEST
FIELD PERSONNEL: ERIC LAVERGNE

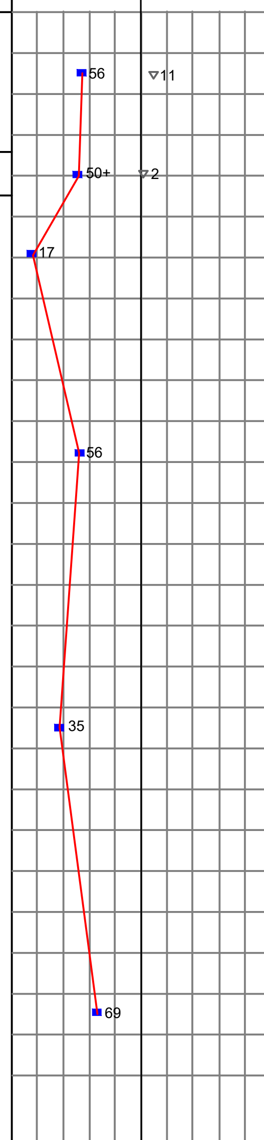
DRILLER: GEORGE DOWNING ESTATE DRILLING LTD.
DRILLING EQUIPMENT: TRUCK MOUNTED CME-55
DRILLING METHOD: SPLIT-BARREL AUGER & CORING

DEPTH (FT / M)	SOIL DESCRIPTION	ELEV./DEPTH (m)	LITHOLOGY	TYPE	SAMPLE NUMBER	N OR RQD (%)	RECOVERY (%)
0.0 FT / 0.0 M	**FILL:** Traces of grass and roots, sandy, silty soil.	100.35 / 0.00					
1.0 FT	Becomes more fine, traces of gravel, brown, dry.				SS1	56	15
2.0 FT	Gravel, gray, dry.						
3.0 FT					SS2	50+	6
1.0 M	BEDROCK	99.33 / 1.02					
4.0 FT						17	
8.0 FT						56	
13.0 FT						35	
18.0 FT						69	

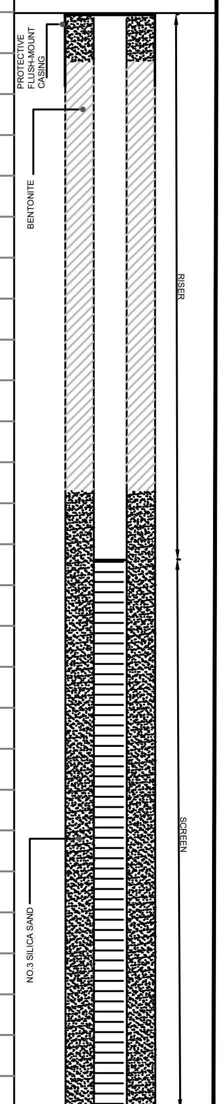
Depth scale: FT 0.0–20.0; M 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0

SPT N VALUE
RQD VALUE
(Blows/0.3 m)
20 40 60 80

WATER CONTENT
(%)
20 40 60 80



MONITORING WELL DETAILS



Groundwater samples collected were submitted for laboratory analysis of VOC, PHC, PAH, Heavy Metals (including CrVi and Hg), & PCB

EASTING: 0444543
NORTHING: 5029252
SITE DATUM: Elevations measured from temporary benchmark established at the northeast upper rim of the fire hydrant located south of Laurier Avenue West (100.00 m).
GROUNDSURFACE ELEVATION: 100.35 m
TOP OF RISER ELEVATION: 100.21 mm
HOLE DIAMETER: 78 mm
MONITORING WELL DIAMETER: 1 1/4 "

NOTES:
bgs: Below Ground Surface
N/A: Not applicable

BOREHOLE LOG: BH24-05

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5430 Canotek Road | Ottawa, ON, K1J 9G2
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PROJECT NO.: 240447
CLIENT: HERITAGE INVESTMENTS LTD.
DATE: SEPTEMBER 3, 2024

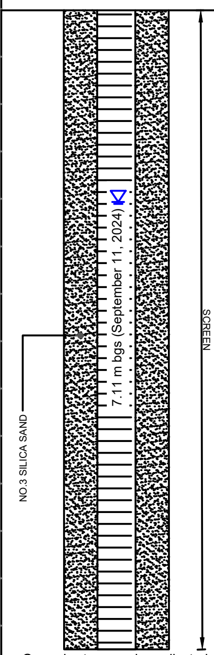
PROJECT: GEOTECHNICAL INVESTIGATION
LOCATION: 593, 601 & 603 LAURIER AVENUE WEST
FIELD PERSONNEL: ERIC LAVERGNE

DRILLER: GEORGE DOWNING ESTATE DRILLING LTD. **DRILLING EQUIPMENT:** TRUCK MOUNTED CME-55 **DRILLING METHOD:** SPLIT-BARREL AUGER

DEPTH (FT / M)	SOIL DESCRIPTION	ELEV./DEPTH (m)	LITHOLOGY	TYPE	SAMPLE NUMBER	N OR RQD (%)	RECOVERY (%)	SPT N VALUE / RQD VALUE (Blows/0.3 m) 20 40 60 80	WATER CONTENT (%) 20 40 60 80
20.0 / 6.1		94.25 6.1				90		90	
						79		79	
30.0 / 9.14	End of Borehole	91.21 9.14							

Depth scale: FT 20.0, 21.0, 22.0, 23.0, 24.0, 25.0, 26.0, 27.0, 28.0, 29.0, 30.0, 31.0, 32.0, 33.0, 34.0, 35.0, 36.0, 37.0, 38.0, 39.0, 40.0; M 6.1, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0

MONITORING WELL DETAILS



Groundwater samples collected were submitted for laboratory analysis of VOC, PHC, PAH, Heavy Metals & Inorganics, and Cyanide

EASTING: 0444515 **NORTHING:** 5029275

SITE DATUM: Elevations measured from temporary benchmark established at the northeast upper rim of the fire hydrant located south of Laurier Avenue West (100.00 m).

GROUNDSURFACE ELEVATION: 100.35 m **TOP OF RISER ELEVATION:** 100.21
HOLE DIAMETER: 78 mm **MONITORING WELL DIAMETER:** 1 ¼"

NOTES:
bgs: Below Ground Surface
N/A: Not applicable

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

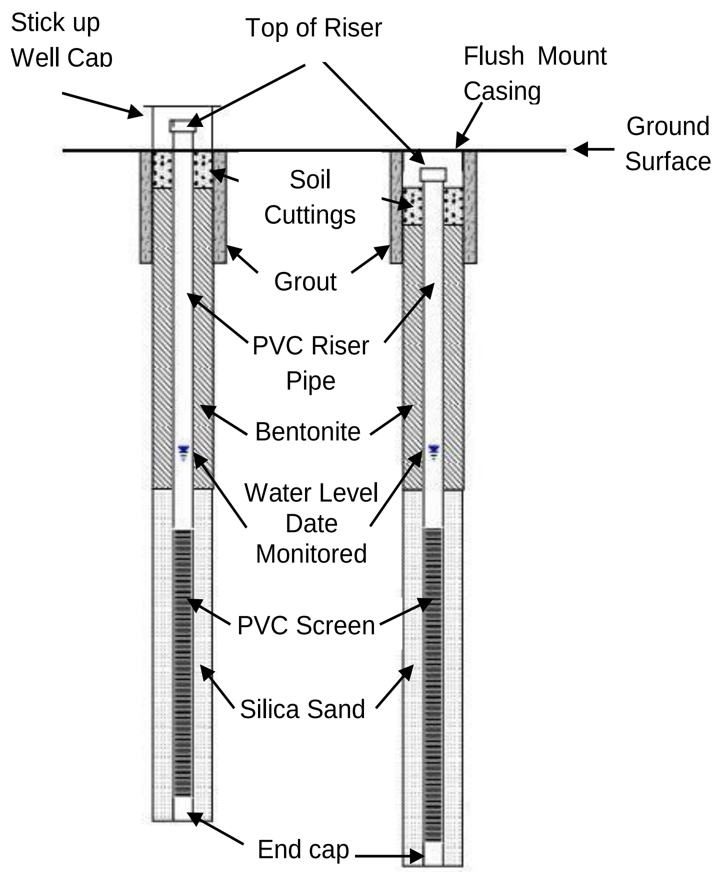
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given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