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# Geotechnical Investigation
## Proposed High-Rise Building
## 829 Carling Avenue
## Ottawa, Ontario

Prepared For

Claridge Homes

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Report: PG5744-1

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# Appendices

# 1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to complete a geotechnical investigation for the proposed high-rise building to be located at 829 Carling Avenue, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

☐ determine the subsoil and groundwater conditions at this site by means of test holes.

☐ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

# 2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a high-rise building with 6 levels of underground parking. Further, it is understood the footprint of the underground parking levels will occupy the majority of the subject site. The proposed building will be surrounded by landscaped margins.

Construction of the proposed development will involve demolition of the existing commercial structure on-site.

# 3.0 Method of Investigation

## 3.1 Field Investigation

### Field Program

The field program for the current investigation was carried out from April 20 to 22, 2021 and consisted of advancing 6 boreholes to a maximum depth of 23.9 m below the existing ground surface. A previous investigation was also completed at the subject site by others in April of 2016. At that time, 4 boreholes were advanced to a maximum depth of 7.6 m. The borehole locations for the current investigation were determined in the field by Paterson personnel taking into consideration site features and underground services. The locations of the boreholes are shown on Drawing PG5744-1 - Test Hole Location Plan in Appendix 2.

The boreholes were completed with a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The test hole procedure consisted of augering and rock coring to the required depths at the selected locations, and sampling and testing the overburden.

### Sampling and In-Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Bedrock samples were recovered using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1.

**Groundwater**

Monitoring wells were installed in the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the current sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

All monitoring wells should be decommissioned in accordance with Ontario Regulations O.Reg 903 by a qualified licensed well technician and prior to construction.

**Sample Storage**

All samples from the current investigation will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

## 3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5744-1 - Test Hole Location Plan in Appendix 2.

## 3.3 Laboratory Testing

Soil and bedrock samples recovered from the subject site were visually examined in our laboratory to review the field logs. Unconfined compressive strength testing of recovered rock cores was carried out on select bedrock core samples. The results of the unconfined compressive strength testing are discussed in Subsection 4.2.

## 3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed in Subsection 6.7.

# 4.0 Observations

## 4.1 Surface Conditions

The subject site is currently occupied by an existing single-storey commercial building, which is located on the eastern end of the site. The western half of the site generally consists of asphalt paved access lanes and parking areas with landscaped margins. The subject site is bordered to the north by Sidney Street, to the east by Preston Street, to the south by Carling Avenue, and to the west by a low-rise commercial building.

The ground surface across the subject site is relatively flat at approximate geodetic elevation 62 m, and is generally at-grade with the surrounding roadways.

## 4.2 Subsurface Profile

**Overburden**

Generally, the subsurface profile encountered at the test hole locations consists of an approximate 50 to 80 mm thick asphalt surface underlain by fill. The fill extended to the bedrock surface at approximate depths of 0.9 to 1.5 m below the existing ground surface, and was generally observed to consist of silty sand with clay, gravel, topsoil, and crushed stone. Construction debris including wood, brick and concrete were also observed within the fill at borehole BH 3-21.

**Bedrock**

Practical refusal to augering on the bedrock surface was encountered at approximate depths ranging from 0.9 to 1.5 m. The bedrock was observed to consist of grey limestone, and based on the RQDs of the recovered bedrock core, was generally weathered and of poor quality to approximate depths of 3 m, becoming good to excellent in quality with depth. At boreholes BH 1-21 to BH 3-21, the bedrock was cored to depths ranging from 22.6 to 23.9 m below the existing ground surface.

Unconfined compressive strength (UCS) was carried out on a total of 3 bedrock core samples. The results of the testing are presented in Table 1 on the next page.

**Table 1 - Unconfined Compressive Strength Testing Results**

| Borehole Number | Sample No. | Sample Depth (m) | Unconfined Compressive Strength (MPa) |
|---|---|---|---|
| BH 1-21 | RC14 | 21.2 - 21.3 | 15.7 |
| BH 2-21 | RC14 | 20.6 - 20.7 | 11.4 |
| BH 3-21 | RC14 | 20.7 - 20.8 | 11.6 |

Based on available geological mapping, the bedrock in this area consists of interbedded limestone and shale of the Verulam formation with a drift thickness of 1 to 10 m.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil and bedrock profiles encountered at each test hole location.

## 4.3 Groundwater

Groundwater levels were measured in the monitoring wells on April 28, 2021. The monitoring wells installed by others (MW-1 through MW-3) were also measured on April 18, 2016. The results are presented in Table 2 below.

**Table 2 - Summary of Groundwater Levels**

| Borehole Number | Measured Groundwater Level | | Recording Date |
|---|---|---|---|
| | Depth (m) | Elevation (m) | |
| BH 1-21 | 10.35 | 51.94 | April 28, 2021 |
| BH 2-21 | 23.24 | 39.13 | April 28, 2021 |
| BH 3-21 | 3.59 | 59.08 | April 28, 2021 |
| MW-1 | 3.45 | - | April 18, 2016 |
| | 2.03 | - | April 28, 2021 |
| MW-2 | 4.75 | - | April 18, 2016 |
| | 2.10 | - | April 28, 2021 |
| MW-3 | Dry | - | April 18, 2016 |
| | Dry | - | April 28, 2021 |

It should be noted that groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color, moisture content and consistency of the recovered soil samples.

Based on these observations, the long-term groundwater level is expected between a 3 to 4 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

# 5.0 Discussion

## 5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed high-rise building is recommended to be founded on conventional spread footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the underground parking levels. Hoe ramming is an option where the bedrock is weathered and/or where only small quantities of bedrock need to be removed. Line drilling and controlled blasting is recommended where large quantities of bedrock need to be removed. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

The above and other considerations are discussed in the following sections.

## 5.2 Site Grading and Preparation

### Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Due to the relatively shallow depth of the bedrock surface and the anticipated founding level for the proposed building, all existing overburden material should be excavated from within the proposed building footprint.

### Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be completed prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

### Vibration Considerations

Construction operations are the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed building.

### Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

## 5.3 Foundation Design

### Bearing Resistance Values

Footings placed on clean, surface sounded limestone bedrock can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **5,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings placed on clean, surface-sounded bedrock will be subjected to negligible post-construction total and differential settlements.

Footings placed on a sound limestone bedrock bearing surface at depth can also be designed using a higher factored bearing resistance value at ultimate limit states (ULS) of **6,000 kPa**, incorporating a geotechnical resistance factor of 0.5, if founded on limestone bedrock and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level.

This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footprint of the footings. At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant. Also, the above probing program can be omitted if the bedrock side profile in the excavation demonstrates and confirms that the limestone bedrock is sound.

### Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or shallower) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or shallower).

## 5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2.

### Field Program

The seismic array testing location was placed on the southeast corner of the site in an approximate southwest-northeast direction as presented in Drawing PG5744-1, attached to the present report. Paterson field personnel placed 18 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between 4 to 8 times at each shot location to improve signal to noise ratio.

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 10, 1.5, and 1 m away from the first geophone, 18 and 18.5 m away from the last geophone, and at the centre of the seismic array.

## Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, $V_{s30}$, of the upper 30 m profile, immediately below the proposed building foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

The $Vs_{30}$ was calculated using the standard equation for average shear wave velocity provided in the OBC 2012, and as presented below.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\sum\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2{,}439m/s}\right)}$$

$$V_{s30} = 2{,}439m/s$$

Based on the result, the average seismic shear wave velocity, $Vs_{30}$, for foundations at the subject site is **2,439 m/s**. Therefore, a **Site Class A** is applicable for seismic design of the proposed building as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

## 5.5 Basement Slab

For the proposed development, all overburden soil will be removed from the building footprint, leaving the bedrock as the founding medium for the basement floor slab. It is anticipated that the basement area for the proposed building will be mostly parking and the recommended pavement structures noted in Subsection 5.8 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

Any soft areas in the basement slab subgrade should be removed and backfilled with appropriate backfill material prior to placing fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions at the site, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the subfloor fill under the lower basement floor. This is discussed further in Subsection 6.1.

## 5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed building. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m$^3$ (effective unit weight 13 kN/m$^3$).

However, the majority of the basement walls are to be poured against a composite drainage blanket which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 23.5 kN/m$^3$ (effective 15.5 kN/m$^3$) where this condition occurs. Further, a seismic earth pressure component will not be applicable for the foundation wall which is poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective unit weight of the retained soil or bedrock should be utilized, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

**Lateral Earth Pressures**

The static horizontal earth pressure ($P_o$) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

$K_o$ = at-rest earth pressure coefficient of the applicable retained material
$\gamma$ = unit weight of fill of the applicable retained material (kN/m$^3$)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

**Seismic Earth Pressures**

The total seismic force ($P_{AE}$) includes both the earth force component ($P_o$) and the seismic component ($\Delta P_{AE}$).

The seismic earth force ($\Delta P_{AE}$) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

$a_c$ = $(1.45-a_{max}/g)a_{max}$
$\gamma$ = unit weight of fill of the applicable retained soil (kN/m$^3$)
H = height of the wall (m)
g = gravity, 9.81 m/s$^2$

The peak ground acceleration, ($a_{max}$), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component ($P_o$) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force ($P_{AE}$) is considered to act at a height, h (m), from the base of the wall, where:

$$h \quad = \quad \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

## 5.7 Rock Anchor Design

### Overview of Anchor Features

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

The anchor should be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed building, the rock anchors for this project are recommended to be provided with double corrosion protection.

## Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of limestone ranges between about 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

## Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

## Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 3 below:

**Table 3 - Parameters used in Rock Anchor Review**

| | |
|---|---|
| Grout to Rock Bond Strength - Factored at ULS | 1.0 MPa |
| Compressive Strength - Grout | 40 MPa |
| Rock Mass Rating (RMR) - Good quality Limestone<br>Hoek and Brown parameters | 65<br>m=0.575 and s=0.00293 |
| Unconfined compressive strength - Limestone bedrock | 50 MPa |
| Unit weight - Submerged Bedrock | 15.5 kN/m$^3$ |
| Apex angle of failure cone | 60° |
| Apex of failure cone | mid-point of fixed anchor length |

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 4 on the next page. The factored tensile resistance values given in Table 3 are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed building are determined.

**Table 4 - Recommended Rock Anchor Lengths - Grouted Rock Anchor**

| Diameter of Drill Hole (mm) | Anchor Lengths (m) | | | Factored Tensile Resistance (kN) |
|---|---|---|---|---|
| | Bonded Length | Unbonded Length | Total Length | |
| 75 | 2.0 | 0.8 | 2.8 | 450 |
| | 2.6 | 1.0 | 3.6 | 600 |
| | 3.2 | 1.3 | 4.5 | 750 |
| | 4.5 | 2.0 | 6.5 | 1000 |
| 125 | 1.6 | 1.0 | 2.6 | 600 |
| | 2.0 | 1.2 | 3.2 | 750 |
| | 2.6 | 1.4 | 4.0 | 1000 |
| | 3.2 | 1.8 | 5.0 | 1250 |

**Other considerations**

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout.

## 5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 5 on the next page. The flexible pavement structure presented in Table 6 should be used for at grade access lanes and heavy loading parking areas.

**Table 5 - Recommended Rigid Pavement Structure - Lower Parking Level**

| Thickness (mm) | Material Description |
|---|---|
| 150 | **Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment) |
| 300 | **BASE** - OPSS Granular A Crushed Stone |
| **SUBGRADE** - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock. | |

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

**Table 6 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas**

| Thickness (mm) | Material Description |
|---|---|
| 40 | **Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete |
| 50 | **Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete |
| 150 | **BASE** - OPSS Granular A Crushed Stone |
| 450 | **SUBBASE** - OPSS Granular B Type II |
| **SUBGRADE** - OPSS Granular B Type II overlying the Concrete Podium Deck. | |

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

## 5.9 Hydraulic Conductivity Testing

Hydraulic conductivity testing was completed at boreholes BH 3-21 and MW-1 which were outfitted with monitoring wells and screened within the bedrock. Rising head and falling head testing (“slug testing”) was completed within the limestone bedrock in accordance with ASTM Standard Test Method D4404 - Field Procedure for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers.

Following the completion of the slug testing, the test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter. The assumption regarding aquifer storage is considered to be appropriate for groundwater flow through the bedrock aquifer. The assumption regarding screen length and well diameter is considered to be met based on the screen lengths of 3 and 2.1 m and the well diameters of 0.032 and 0.038 m at boreholes BH 3-21 and MW-1, respectively.

While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site.

The Hvorslev analysis is based on the line of best fit through the field data (hydraulic head recovery vs. time), plotted on a semi-logarithmic scale. In cases where the initial hydraulic head displacement is known with relative certainty, such as in this case where a physical slug has been introduced/removed, the line of best fit is considered to pass through the origin.

Based on the above test methods, the monitoring wells screened in the bedrock displayed a hydraulic conductivity value ranging from **$9.96 \times 10^{-8}$ to $6.02 \times 10^{-7}$ m/sec**. The values measured within the monitoring wells are generally consistent with similar material Paterson has encountered on other sites and typical published values for good to excellent quality limestone bedrock. These values typically range from $1 \times 10^{-6}$ to $1 \times 10^{-10}$ m/sec. The range in hydraulic conductivity values is due to the variability of the bedrock quality. The results from the hydraulic conductivity testing are attached to the current report.

# 6.0 Design and Construction Precautions

## 6.1 Foundation Drainage and Backfill

It is anticipated that the portion of the proposed building foundation walls located below the long-term groundwater table will be blind poured and placed against a groundwater infiltration control system. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater which comes in contact with the proposed building's foundation walls.

For the groundwater infiltration control system for the foundation walls, the following is recommended:

☐ Line drill the excavation perimeter (usually at 150 to 200 mm spacing).
☐ Mechanical bedrock removal along the foundation walls can be undertaken up to 150 mm from the finished vertical excavation face.
☐ Grind the bedrock surface up to the outer face of the line drill holes to ensure a satisfactory surface for the below grade foundation drainage system.
☐ If bedrock overbreaks occur, shotcrete these areas to fill in cavities and to smooth out angular features at the bedrock surface, as required based on site inspection by Paterson.
☐ Place a suitable waterproofing membrane (such as Tremco Paraseal or approved equivalent) against the prepared bedrock surface. The membrane liner should extend from 4 m below existing grade down to footing level.
☐ Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
☐ Pour foundation wall against the composite drainage system.

It is recommended that 100 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of any water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

A waterproofing system should also be provided for the elevator pits (pit bottom and walls).

### Sub-slab Drainage System

Sub-slab drainage will be required to control water infiltration for the underground parking levels. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres underlying the lowest level floor slab. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

## 6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover in conjunction with adequate foundation insulation, should be provided.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

## 6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

### Unsupported Excavations

The excavation side slopes in the overburden and above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

**Bedrock Stabilization**

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for horizontal rock anchors should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Further, due to the depth of excavation at this site, groundwater infiltration through the vertical bedrock face is anticipated. During the winter season, ice may start to form along the excavation sidewalls at various locations. The following recommendations are suggested to manage the ice accumulation, where encountered. Ultimately, it is the responsibility of the excavation contractor to ensure that excavation remains a worker safe area.

☐ Ice build up on the excavation sidewalls, should it occur, would present a hazard for workers working below these areas. At the locations where ice is observed above head level, worker access should be restricted using approved barriers and signage for hazard areas, until such time that the ice has been removed

☐ At the locations where construction personnel will be working, any overhanging ice should be removed at the beginning of each day using either the excavator bucket, hoe-ram or rock grinder where the excavator can reach the ice. Once this equipment is no longer present on-site, a hydraulic lift may be required to remove the overhanging ice.

## Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The design and approval of the temporary shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures, and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system.

The temporary shoring system may consist of a soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure the stability.

The toe of the shoring is recommended to be adequately supported to resist toe failure by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is used.

The earth pressures acting on the shoring system may be calculated using the following parameters.

**Table 7 - Soil Parameters**

| Parameters | Values |
|---|---|
| Active Earth Pressure Coefficient ($K_a$) | 0.33 |
| Passive Earth Pressure Coefficient ($K_p$) | 3 |
| At-Rest Earth Pressure Coefficient ($K_o$) | 0.5 |
| Dry Unit Weight (γ), kN/m$^3$ | 21 |
| Effective Unit Weight (γ’), kN/m$^3$ | 13 |

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible.

The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

## 6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

## 6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

### Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

### Impacts to Neighbouring Properties

It is understood that 6 levels of underground parking are planned for the proposed building with the lower portion of the foundation walls having a groundwater infiltration control system in place. Due to the presence of a groundwater infiltration control system in place against the bedrock face, long-term groundwater lowering is anticipated to be negligible for the area. Therefore, no adverse effects to neighbouring properties are expected.

## 6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site mostly consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters, tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

## 6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive to very aggressive corrosive environment.

# 7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

☐ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.

☐ Review the bedrock stabilization and excavation requirements.

☐ Review proposed waterproofing and foundation drainage design and requirements.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Sampling and testing of the concrete and fill materials used.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

## 8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Claridge Homes or their agents is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

**Paterson Group Inc.**

Scott Dennis

Scott S. Dennis, P.Eng.

LICENSED PROFESSIONAL ENGINEER
May 12, 2021
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

**Report Distribution**

- ☐ Claridge Homes (1 digital copy)
- ☐ Paterson Group (1 copy)

# APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

STRATIGRAPHIC AND INSTRUMENTATION LOGS BY OTHERS

UNCONFINED COMPRESSIVE STRENGTH TESTING RESULTS

HYDRAULIC CONDUCTIVITY ANALYSIS

ANALYTICAL TESTING RESULTS

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed High-Rise Building - 829 Carling Avenue**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** Track-Mount Power Auger

**DATE** April 20, 2021

**FILE NO.** PG5744

**HOLE NO.** BH 1-21

| SOIL DESCRIPTION | Depth (m) |
|---|---|
| GROUND SURFACE | |
| Asphaltic concrete | 0.05 |
| **FILL:** Brown silty sand with crushed stone | 0.36 |
| **FILL:** Topsoil with silty clay | 0.76 |
| **FILL:** Brown silty sand with clay and gravel, trace topsoil | 1.17 |
| **BEDROCK:** Poor quality, grey limestone | 3.00 |
| **BEDROCK:** Good to excellent quality, grey limestone | |

| SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|
| AU | 1 | | |
| AU | 2 | | |
| SS | 3 | 50 | 28 |
| RC | 1 | 100 | 62 |
| RC | 2 | 100 | 88 |
| RC | 3 | 100 | 100 |
| RC | 4 | 100 | 100 |
| RC | 5 | 100 | 100 |
| RC | 6 | 100 | 100 |
| RC | 7 | 100 | 100 |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 62.29 |
| 1 | 61.29 |
| 2 | 60.29 |
| 3 | 59.29 |
| 4 | 58.29 |
| 5 | 57.29 |
| 6 | 56.29 |
| 7 | 55.29 |
| 8 | 54.29 |
| 9 | 53.29 |
| 10 | 52.29 |
| 11 | 51.29 |
| 12 | 50.29 |
| 13 | 49.29 |

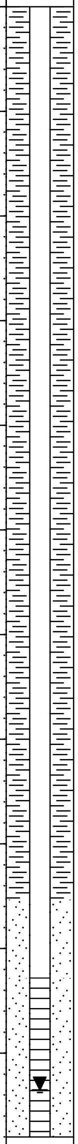
Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Monitoring Well Construction



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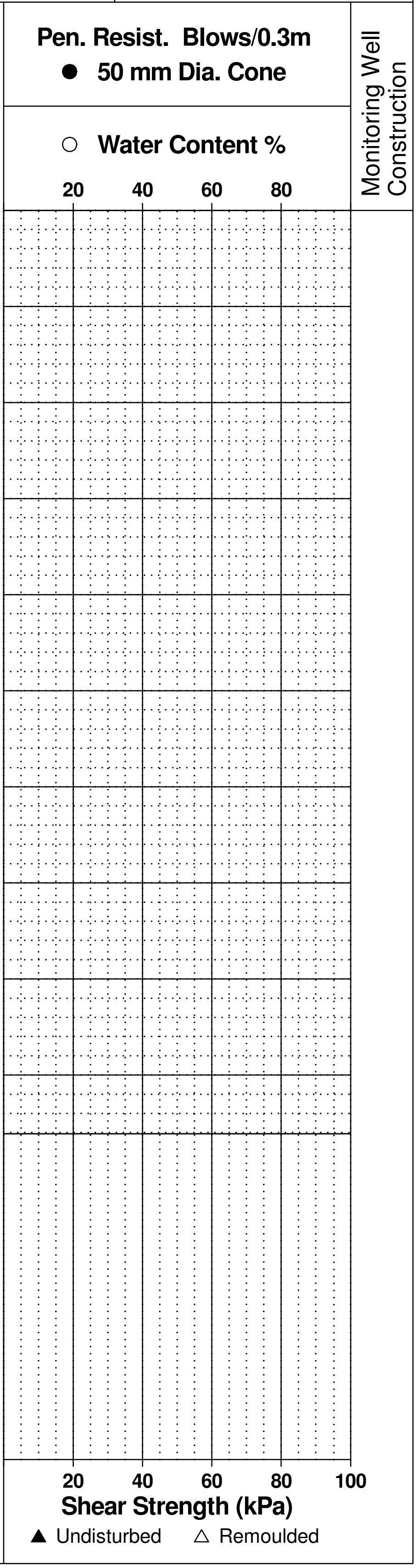
| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | | |
| | | RC | 8 | 100 | 100 | 13 | 49.29 |
| | | | | | | 14 | 48.29 |
| | | RC | 9 | 100 | 100 | | |
| | | | | | | 15 | 47.29 |
| | | RC | 10 | 100 | 100 | 16 | 46.29 |
| | | | | | | 17 | 45.29 |
| **BEDROCK:** Good to excellent quality, grey limestone | | RC | 11 | 100 | 100 | | |
| | | | | | | 18 | 44.29 |
| | | RC | 12 | 100 | 95 | 19 | 43.29 |
| | | | | | | 20 | 42.29 |
| | | RC | 13 | 100 | 90 | | |
| | | | | | | 21 | 41.29 |
| | | RC | 14 | 100 | 100 | 22 | 40.29 |
| 22.61 | | | | | | | |
| End of Borehole<br>(GWL @ 10.35m - April 28, 2021) | | | | | | | |



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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed High-Rise Building - 829 Carling Avenue**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** Track-Mount Power Auger

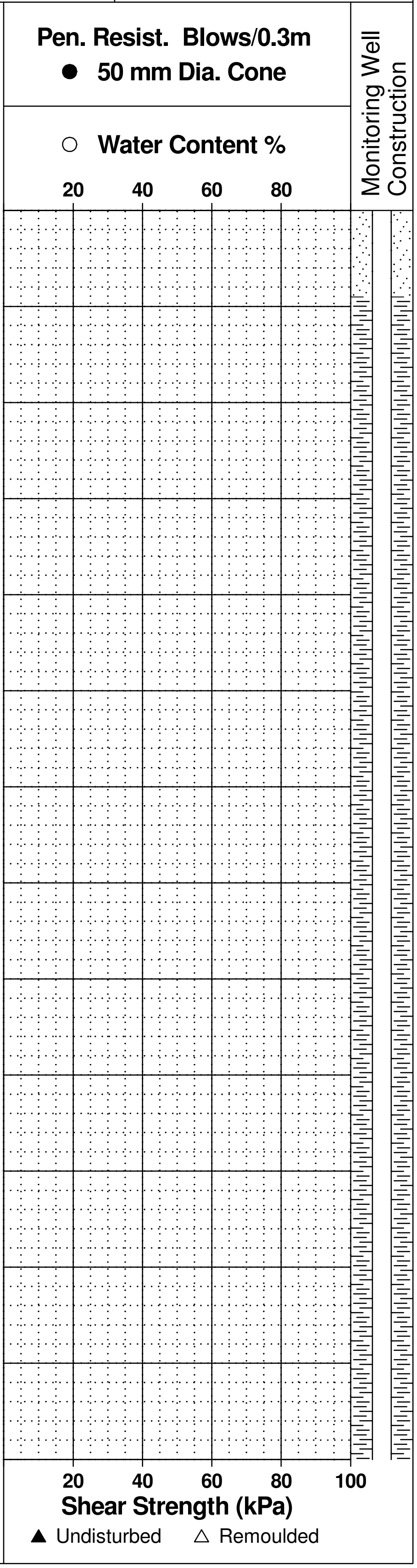
**DATE** April 21, 2021

**FILE NO.** PG5744

**HOLE NO.** BH 2-21

| SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|
| **GROUND SURFACE** | | | | |
| Asphaltic concrete 0.08 | AU | 1 | | |
| **FILL:** Brown silty sand with crushed stone, trace clay | AU | 2 | | |
| 1.22 | SS | 3 | 54 | 20 |
| | RC | 1 | 100 | 100 |
| | RC | 2 | 100 | 95 |
| | RC | 3 | 100 | 100 |
| | RC | 4 | 100 | 100 |
| **BEDROCK:** Excellent quality, grey limestone | RC | 5 | 100 | 100 |
| | RC | 6 | 100 | 100 |
| | RC | 7 | 100 | 100 |
| | RC | 8 | 100 | 100 |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 62.37 |
| 1 | 61.37 |
| 2 | 60.37 |
| 3 | 59.37 |
| 4 | 58.37 |
| 5 | 57.37 |
| 6 | 56.37 |
| 7 | 55.37 |
| 8 | 54.37 |
| 9 | 53.37 |
| 10 | 52.37 |
| 11 | 51.37 |
| 12 | 50.37 |
| 13 | 49.37 |



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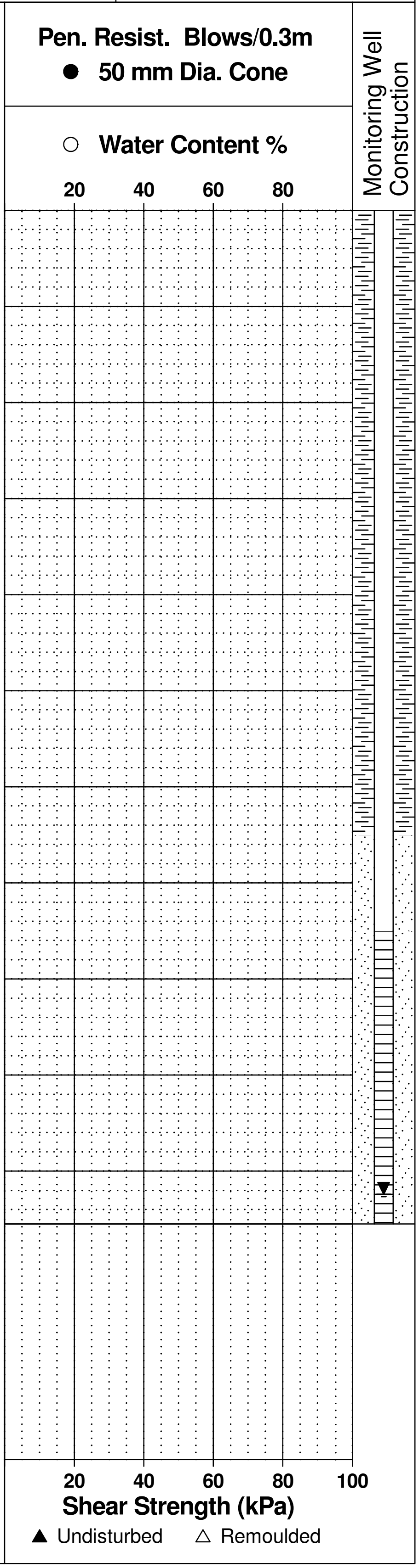
**HOLE NO.** BH 2-21

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|---|
| GROUND SURFACE | | | | | |
| **BEDROCK:** Excellent quality, grey limestone | | RC | 9 | 100 | 100 |
| | | RC | 10 | 100 | 100 |
| | | RC | 11 | 100 | 100 |
| | | RC | 12 | 100 | 97 |
| | | RC | 13 | 100 | 100 |
| | | RC | 14 | 100 | 90 |
| | | RC | 15 | 100 | 100 |
| 23.55 | | | | | |
| End of Borehole<br>(GWL @ 23.24m - April 28, 2021) | | | | | |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 13 | 49.37 |
| 14 | 48.37 |
| 15 | 47.37 |
| 16 | 46.37 |
| 17 | 45.37 |
| 18 | 44.37 |
| 19 | 43.37 |
| 20 | 42.37 |
| 21 | 41.37 |
| 22 | 40.37 |
| 23 | 39.37 |



# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed High-Rise Building - 829 Carling Avenue**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** Track-Mount Power Auger

**DATE** April 22, 2021

**FILE NO.** PG5744

**HOLE NO.** BH 3-21

**GROUND SURFACE**

| SOIL DESCRIPTION | Depth (m) |
|---|---|
| Concrete | 0.08 |
| **FILL:** Brown silty sand, trace crushed stone | 0.91 |
| **FILL:** Brown silty sand with gravel, some topsoil, trace wood, brick, mortar and concrete | 1.09 |
| **BEDROCK:** Poor quality, grey limestone<br>- vertical seam from 1.45 to 1.9m depth | 2.95 |
| **BEDROCK:** Excellent quality, grey limestone | |

| SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|
| AU | 1 | | |
| SS | 2 | 100 | 5 |
| RC | 1 | 100 | 47 |
| RC | 2 | 100 | 93 |
| RC | 3 | 100 | 98 |
| RC | 4 | 100 | 100 |
| RC | 5 | 100 | 95 |
| RC | 6 | 100 | 10 |
| RC | 7 | 100 | 100 |
| RC | 8 | 100 | 100 |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 62.67 |
| 1 | 61.67 |
| 2 | 60.67 |
| 3 | 59.67 |
| 4 | 58.67 |
| 5 | 57.67 |
| 6 | 56.67 |
| 7 | 55.67 |
| 8 | 54.67 |
| 9 | 53.67 |
| 10 | 52.67 |
| 11 | 51.67 |
| 12 | 50.67 |
| 13 | 49.67 |

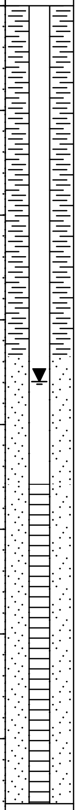
Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100
**Shear Strength (kPa)**
▲ Undisturbed △ Remoulded

Monitoring Well Construction



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## SOIL PROFILE AND TEST DATA

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**DATUM** Geodetic

**REMARKS**

**BORINGS BY** Track-Mount Power Auger

**DATE** April 22, 2021

**FILE NO.** PG5744

**HOLE NO.** BH 3-21

| SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|
| GROUND SURFACE | | | | |
| **BEDROCK:** Excellent quality, grey limestone | RC | 9 | 100 | 100 |
| | RC | 10 | 100 | 100 |
| | RC | 11 | 100 | 95 |
| | RC | 12 | 100 | 97 |
| | RC | 13 | 100 | 100 |
| | RC | 14 | 100 | 85 |
| | RC | 15 | 100 | 100 |
| 23.93 | | | | |
| End of Borehole | | | | |
| (GWL @ 3.59m - April 28, 2021) | | | | |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 13 | 49.67 |
| 14 | 48.67 |
| 15 | 47.67 |
| 16 | 46.67 |
| 17 | 45.67 |
| 18 | 44.67 |
| 19 | 43.67 |
| 20 | 42.67 |
| 21 | 41.67 |
| 22 | 40.67 |
| 23 | 39.67 |

**Pen. Resist. Blows/0.3m**
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100

**Shear Strength (kPa)**

▲ Undisturbed △ Remoulded

Monitoring Well Construction

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed High-Rise Building - 829 Carling Avenue**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** Track-Mount Power Auger

**DATE** April 20, 2021

**FILE NO.** PG5744

**HOLE NO.** BH 4-21

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) | Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80) | Monitoring Well Construction |
|---|---|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | | | | |
| Asphaltic concrete 0.05 | | AU | 1 | | | 0 | 62.61 | | |
| **FILL:** Crushed stone with topsoil, some sand 1.22 | | AU | 2 | | | | | | |
| | | SS | 3 | 33 | 14 | 1 | 61.61 | | |
| **BEDROCK:** Weathered grey limestone 1.40 | | | | | | | | | |
| End of Borehole<br>Practical refusal to augering at 1.40m depth. | | | | | | | | | |

20 40 60 80 100
**Shear Strength (kPa)**
▲ Undisturbed △ Remoulded

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed High-Rise Building - 829 Carling Avenue**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

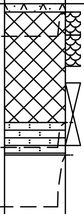
**BORINGS BY** Track-Mount Power Auger

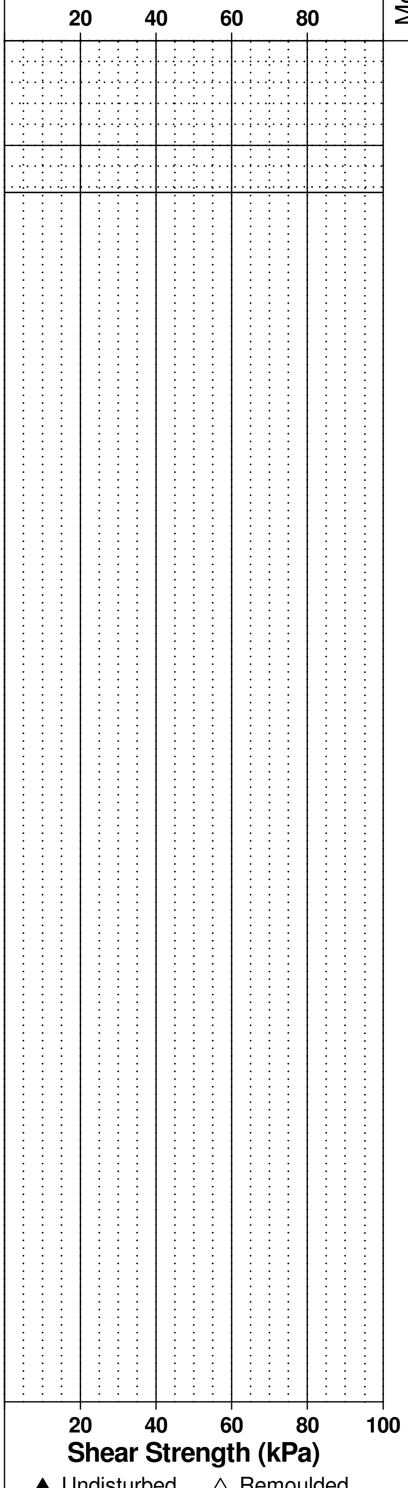
**DATE** April 20, 2021

**FILE NO.** PG5744

**HOLE NO.** BH 5-21

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE | | | | DEPTH (m) | ELEV. (m) | Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone<br>○ Water Content % | Monitoring Well Construction |
|---|---|---|---|---|---|---|---|---|---|
| | | TYPE | NUMBER | % RECOVERY | N VALUE or RQD | | | | |
| **GROUND SURFACE** | | | | | | 0 | 62.16 | | |
| Asphaltic concrete 0.08 | | AU | 1 | | | | | | |
| **FILL:** Brown silty sand with crushed stone<br>- some topsoil, trace clay and rock fragments by 0.8m depth 1.14 | | AU | 2 | | | | | | |
| | | SS | 3 | 25 | 31 | 1 | 61.16 | | |
| 1.45 | | | | | | | | | |
| **BEDROCK:** Weathered grey limestone | | | | | | | | | |
| End of Borehole<br><br>Practical refusal to augering at 1.45m depth. | | | | | | | | | |





**Shear Strength (kPa)**

▲ Undisturbed △ Remoulded

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed High-Rise Building - 829 Carling Avenue**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** Track-Mount Power Auger

**DATE** April 21, 2021

**FILE NO.** PG5744

**HOLE NO.** BH 6-21

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) | Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone<br>○ Water Content %<br>20 40 60 80 | Monitoring Well Construction |
|---|---|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | 62.44 | | |
| Asphaltic concrete 0.08 | | | | | | | | | |
| **FILL:** Brown silty sand with crushed stone 0.84 | | AU | 1 | | | | | | |
| **BEDROCK:** Weathered, grey limestone 0.91 | | SS | 2 | 8 | 50+ | | | | |
| End of Borehole<br>Practical refusal to augering at 0.91m depth. | | | | | | | | | |

20 40 60 80 100

**Shear Strength (kPa)**

▲ Undisturbed △ Remoulded

# SYMBOLS AND TERMS

## SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

| | | |
|---|---|---|
| Desiccated | - | having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc. |
| Fissured | - | having cracks, and hence a blocky structure. |
| Varved | - | composed of regular alternating layers of silt and clay. |
| Stratified | - | composed of alternating layers of different soil types, e.g. silt and sand or silt and clay. |
| Well-Graded | - | Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution). |
| Uniformly-Graded | - | Predominantly of one grain size (see Grain Size Distribution). |

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

| Relative Density | ‘N’ Value | Relative Density % |
|---|---|---|
| Very Loose | <4 | <15 |
| Loose | 4-10 | 15-35 |
| Compact | 10-30 | 35-65 |
| Dense | 30-50 | 65-85 |
| Very Dense | >50 | >85 |

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

| Consistency | Undrained Shear Strength (kPa) |
|---|---|
| Very Soft | <12 |
| Soft | 12-25 |
| Firm | 25-50 |
| Stiff | 50-100 |
| Very Stiff | 100-200 |
| Hard | >200 |

## SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

## ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

| **RQD %** | **ROCK QUALITY** |
|---|---|
| 90-100 | Excellent, intact, very sound |
| 75-90 | Good, massive, moderately jointed or sound |
| 50-75 | Fair, blocky and seamy, fractured |
| 25-50 | Poor, shattered and very seamy or blocky, severely fractured |
| 0-25 | Very poor, crushed, very severely fractured |

## SAMPLE TYPES

| | | |
|---|---|---|
| SS | - | Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT)) |
| TW | - | Thin wall tube or Shelby tube |
| PS | - | Piston sample |
| AU | - | Auger sample or bulk sample |
| WS | - | Wash sample |
| RC | - | Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits. |

# SYMBOLS AND TERMS (continued)

## GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

## CONSOLIDATION TEST

$p'_o$ - Present effective overburden pressure at sample depth
$p'_c$ - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below $p'_c$)
Cc - Compression index (in effect at pressures above $p'_c$)

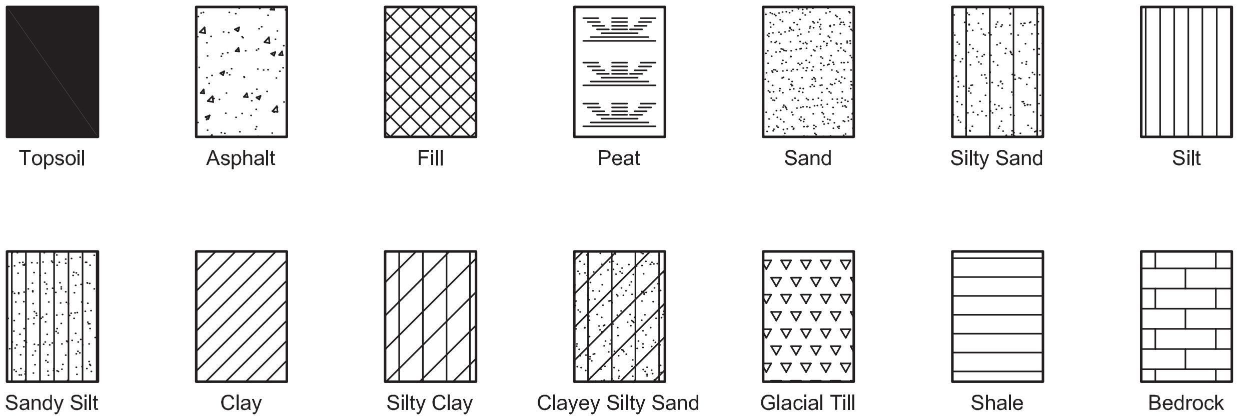
OC Ratio Overconsolidaton ratio = $p'_c / p'_o$
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

## PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

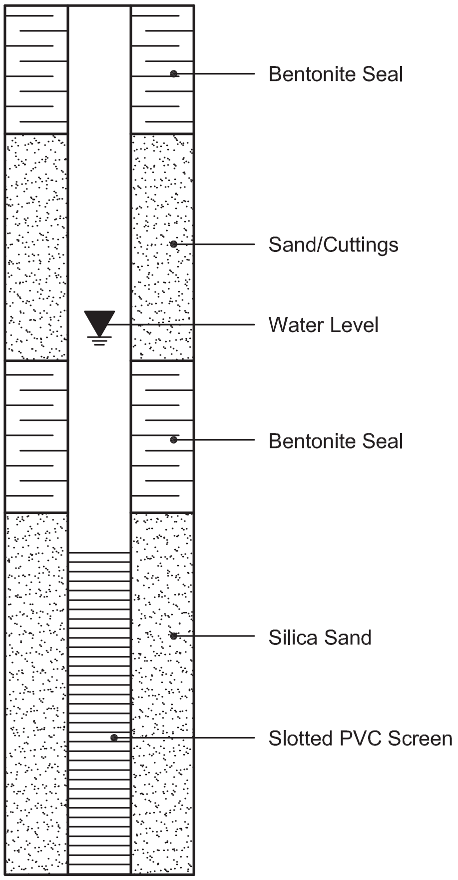
# SYMBOLS AND TERMS (continued)

## STRATA PLOT

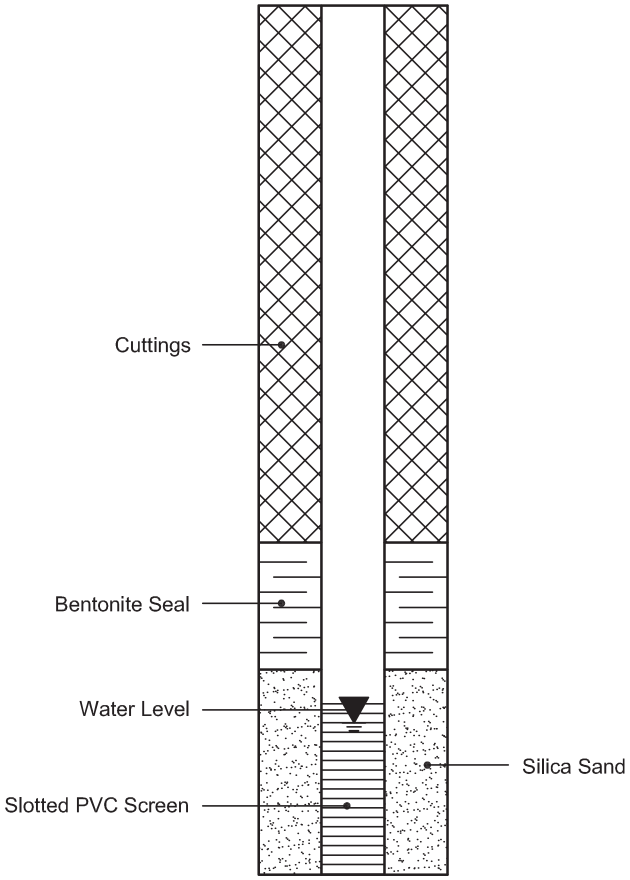


## MONITORING WELL AND PIEZOMETER CONSTRUCTION

### MONITORING WELL CONSTRUCTION



### PIEZOMETER CONSTRUCTION



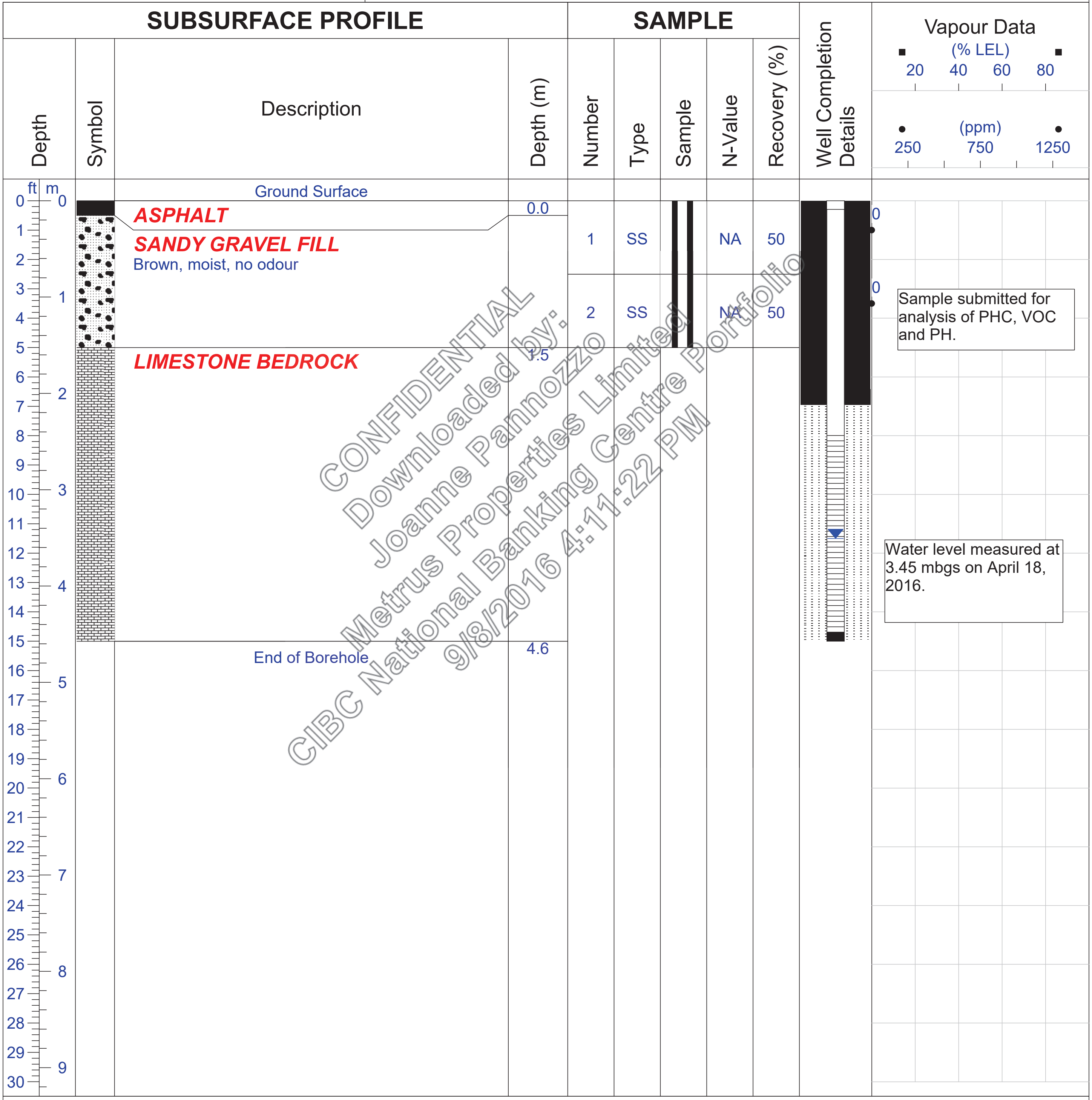
PINCHIN

**Pinchin Ltd.**
555 Legget Drive, Suite 1001
Kanata, Ontario

# Stratigraphic and Instrumentation Log: MW-1

**Project No.:** 111021.002
**Project:** Phase II ESA
**Client:** CIBC Corporate Real Estate
**Location:** 829 Carling Ave, Ottawa, ON

**Logged By:** RML
**Entered By:** RML
**Project Manager:** FD
**Drill Date:** April 15, 2016



| Description | Depth (m) | Number | Type | N-Value | Recovery (%) |
|---|---|---|---|---|---|
| Ground Surface | | | | | |
| **ASPHALT** | 0.0 | | | | |
| **SANDY GRAVEL FILL**<br>Brown, moist, no odour | | 1 | SS | NA | 50 |
| | | 2 | SS | NA | 50 |
| **LIMESTONE BEDROCK** | 1.5 | | | | |
| End of Borehole | 4.6 | | | | |

Sample submitted for analysis of PHC, VOC and PH.

Water level measured at 3.45 mbgs on April 18, 2016.

**Drilled By:** Strata Drilling Group
**Drill Method:** Geo-Machine
**Vapour Instrument:** Photoionization Detector
**Well Casing Size:** 38mm

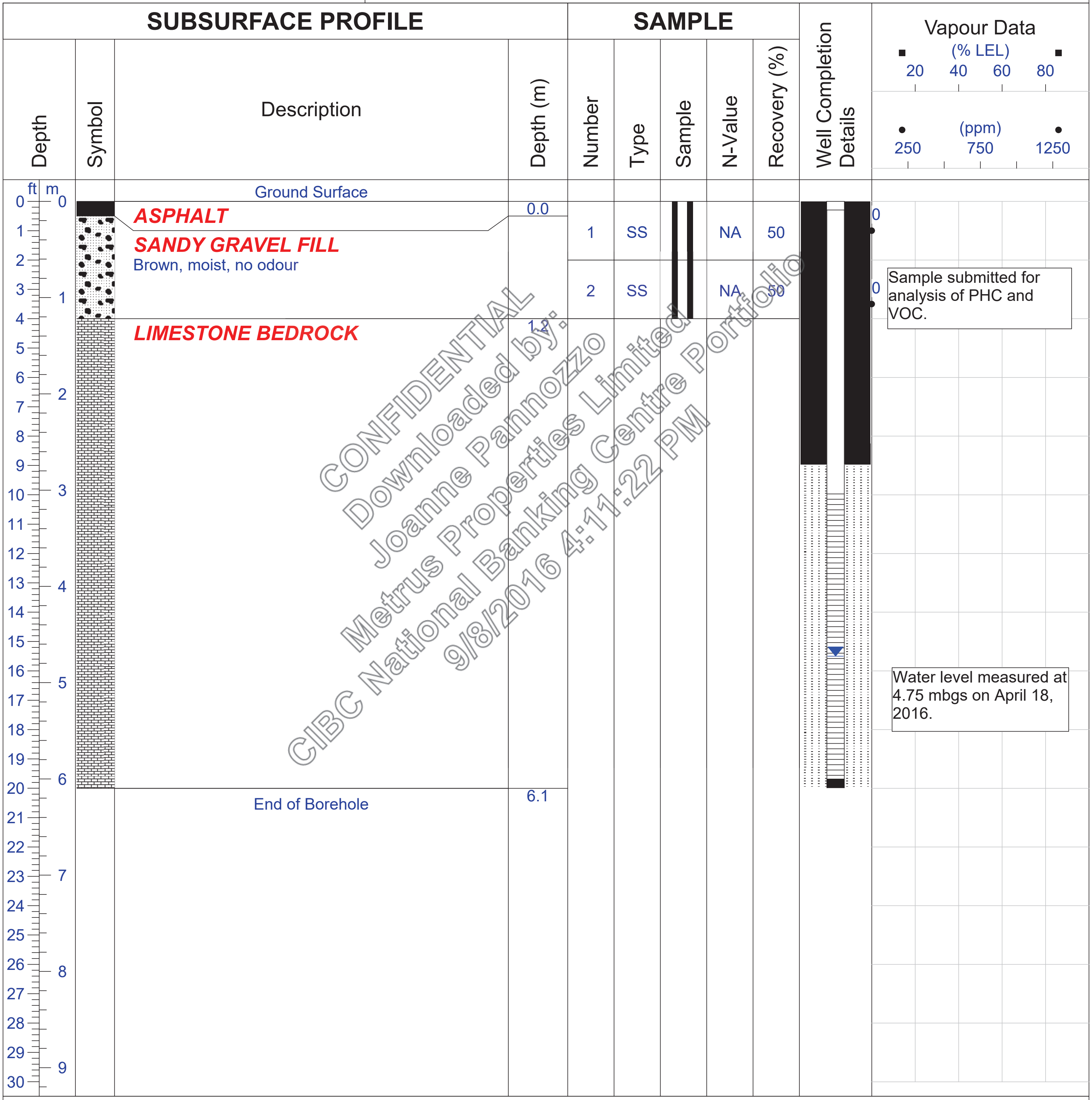
**Datum:** NA
**Casing Elevation:** NM
**Ground Elevation:** NM
**Sheet:** 1 of 1

# Stratigraphic and Instrumentation Log: MW-2

**Pinchin Ltd.**
555 Legget Drive, Suite 1001
Kanata, Ontario

**Project No.:** 111021.002
**Project:** Phase II ESA
**Client:** CIBC Corporate Real Estate
**Location:** 829 Carling Ave, Ottawa, ON

**Logged By:** RML
**Entered By:** RML
**Project Manager:** FD
**Drill Date:** April 15, 2016



| Depth (m) | Description | Number | Type | N-Value | Recovery (%) |
|---|---|---|---|---|---|
|  | Ground Surface |  |  |  |  |
| 0.0 | **ASPHALT** |  |  |  |  |
|  | **SANDY GRAVEL FILL**<br>Brown, moist, no odour | 1 | SS | NA | 50 |
|  |  | 2 | SS | NA | 50 |
| 1.2 | **LIMESTONE BEDROCK** |  |  |  |  |
| 6.1 | End of Borehole |  |  |  |  |

Sample submitted for analysis of PHC and VOC.

Water level measured at 4.75 mbgs on April 18, 2016.

**Drilled By:** Strata Drilling Group
**Drill Method:** Geo-Machine
**Vapour Instrument:** Photoionization Detector
**Well Casing Size:** 38mm

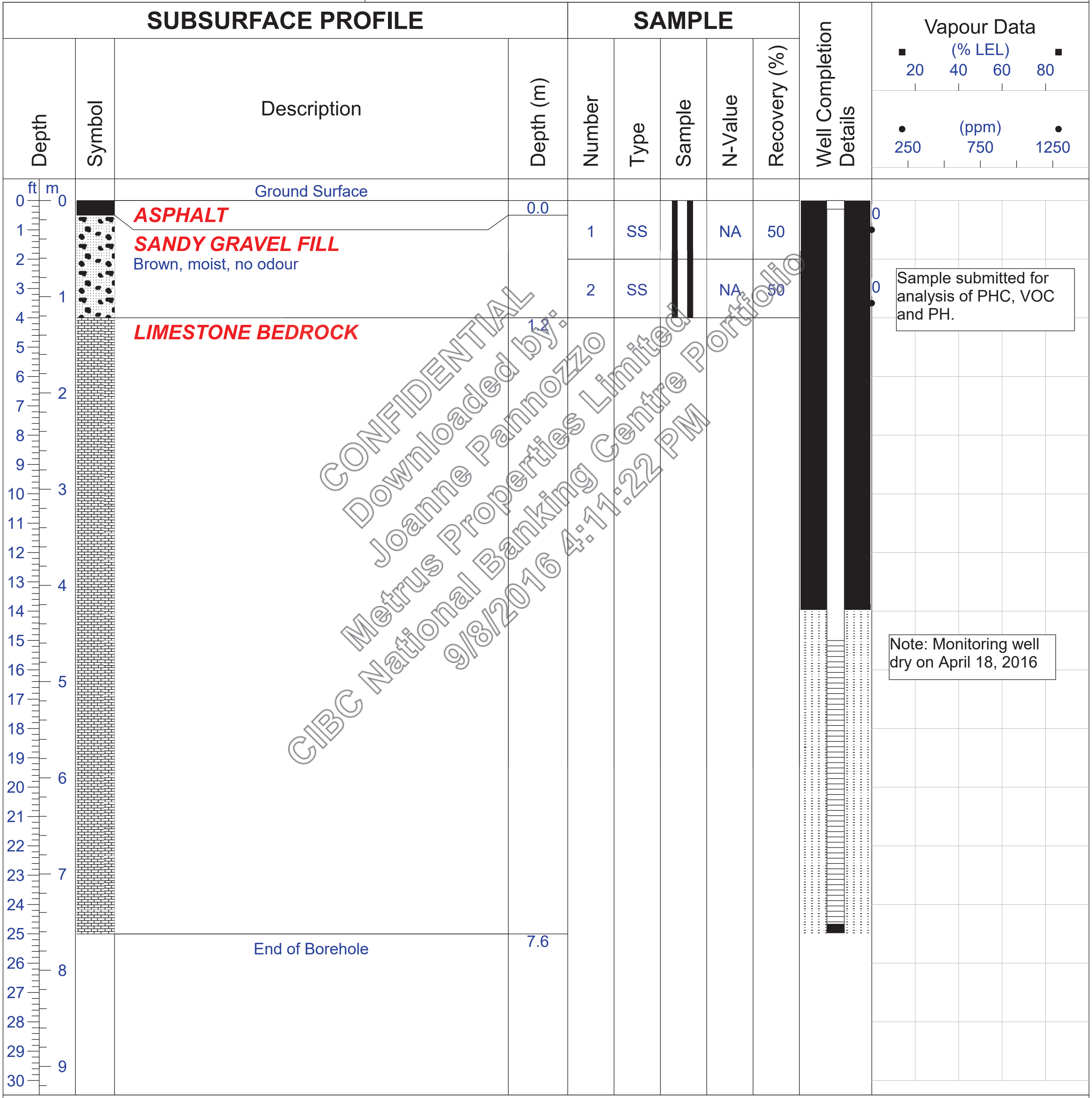
**Datum:** NA
**Casing Elevation:** NM
**Ground Elevation:** NM
**Sheet:** 1 of 1

# Stratigraphic and Instrumentation Log: MW-3

**Pinchin Ltd.**
555 Legget Drive, Suite 1001
Kanata, Ontario

**Project No.:** 111021.002
**Project:** Phase II ESA
**Client:** CIBC Corporate Real Estate
**Location:** 829 Carling Ave, Ottawa, ON

**Logged By:** RML
**Entered By:** RML
**Project Manager:** FD
**Drill Date:** April 15, 2016



| Depth (m) | Description | Number | Type | N-Value | Recovery (%) |
|---|---|---|---|---|---|
|  | Ground Surface |  |  |  |  |
| 0.0 | **ASPHALT** |  |  |  |  |
|  | **SANDY GRAVEL FILL**<br>Brown, moist, no odour | 1 | SS | NA | 50 |
|  |  | 2 | SS | NA | 50 |
| 1.2 | **LIMESTONE BEDROCK** |  |  |  |  |
| 7.6 | End of Borehole |  |  |  |  |

Sample submitted for analysis of PHC, VOC and PH.

Note: Monitoring well dry on April 18, 2016

**Drilled By:** Strata Drilling Group
**Drill Method:** Geo-Machine
**Vapour Instrument:** Photoionization Detector
**Well Casing Size:** 38mm

**Datum:** NA
**Casing Elevation:** NM
**Ground Elevation:** NM
**Sheet:** 1 of 1

patersongroup
consulting engineers

## BEDROCK CORE COMPRESSIVE STRENGTH ASTM D7012

| | | | |
|---|---|---|---|
| CLIENT: | **Claridge Homes** | FILE No.: | **PG5744** |
| PROJECT: | 829 Carling Ave. | REPORT No.: | 1 |
| SITE ADDRESS | **Proposed High-Rise Building** | DATE REPT'D: | 27-Apr-21 |
| STRUCTURE TYPE & LOCATION: | Bedrock | | |
| **SAMPLE INFORAMTION** | | | |
| LAB NO.: | **24013** | **24014** | **24015** |
| SAMPLE NO.: | BH1-21 RC14 | BH2-21 RC14 | BH3-21 RC14 |
| DEPTH: | 69'-5" to 69'-9" | 67'-5" to 67'-9" | 67'-10" to 68'2" |
| **SAMPLE DATES** | | | |
| DATE CORED | April 21st - 22nd | April 21st - 22nd | April 21st - 22nd |
| DATE RECEIVED | 22-Apr-21 | 22-Apr-21 | 22-Apr-21 |
| DATE TESTED | 27-Apr-21 | 27-Apr-21 | 27-Apr-21 |
| **SAMPLE DIMENSIONS** | | | |
| AVERAGE DIAMETER (mm) | 48.00 | 48.00 | 48.00 |
| HEIGHT (mm) | 95.00 | 95.00 | 95.00 |
| WEIGHT (g) | 460 | 460 | 440 |
| AREA (mm$^2$) | 1810 | 1810 | 1810 |
| VOLUME (cm$^3$) | 172 | 172 | 172 |
| **UNIT WEIGHT (kg/m$^3$)** | **2676** | **2676** | **2560** |
| **TEST RESULTS** | | | |
| H / D RATIO | 1.98 | 1.98 | 1.98 |
| CORRECTION FACTOR | 0.997 | 0.997 | 0.997 |
| LOAD (lbs) | 6402 | 4663 | 4713 |
| GROSS Mpa | 15.7 | 11.5 | 11.6 |
| **MPa CORRECTED** | **15.7** | **11.4** | **11.6** |
| FORM OF BREAK | **TYPE A** | **TYPE A** | **TYPE A** |
| DIRECTION OF LOADING | PARALLEL | PARALLEL | PARALLEL |
| **REMARKS** | | | |
| | | | |
| | | | |
| | | | |
| **TECHNICAL PERSONNEL** | | | |

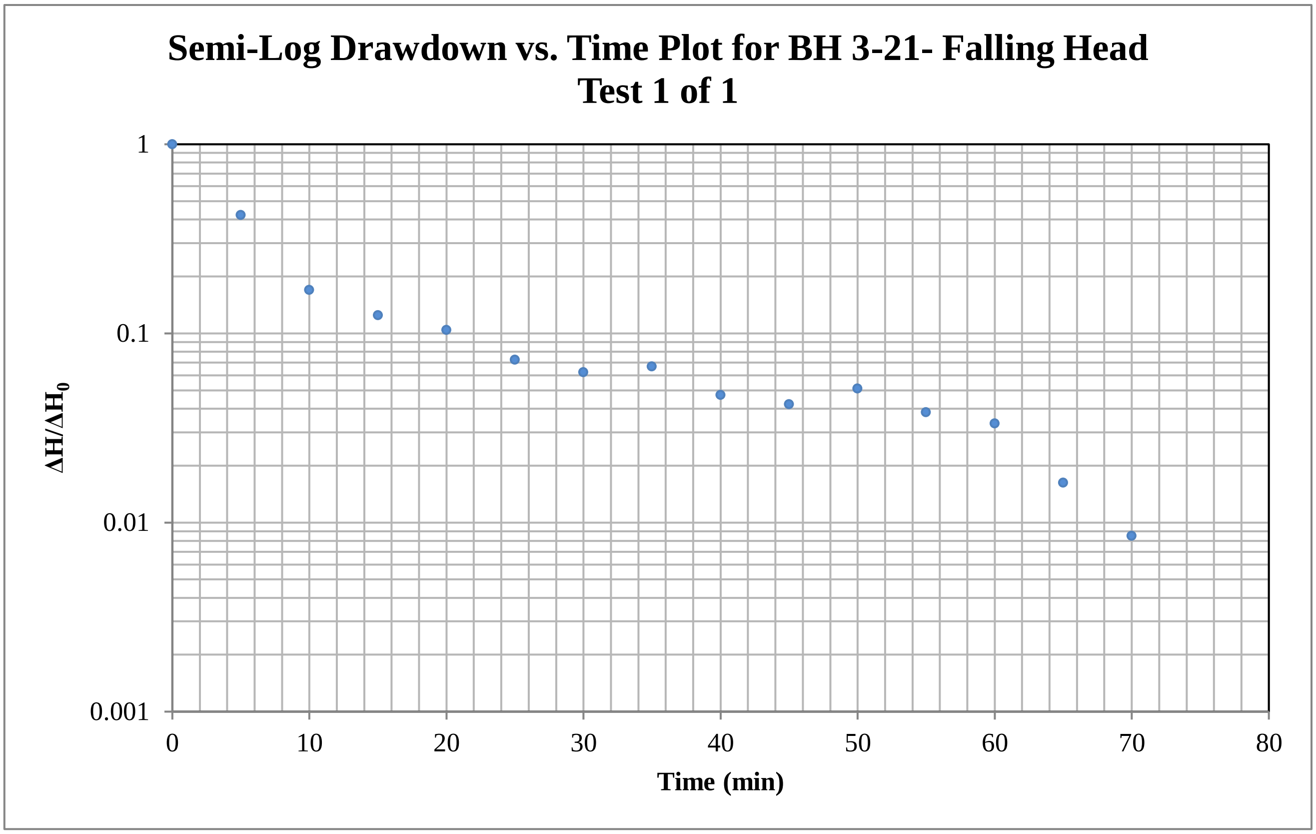
| | | | | |
|---|---|---|---|---|
| **TECHNICIAN:** | **VERIFIED BY:** | **C. Beadow** | **APPROVED BY:** | **Joe Forsyth, P. Eng.** |

**CERTIFIED LAB**

John D. Paterson & Associates Ltd., 28 Concourse Gate, Nepean, ON

# Hvorslev Hydraulic Conductivity Analysis

Project: Claridge Homes - 829 Carling Avenue
Test Location: BH 3-21
Test: 1 of 1 Rising Head
Date: April 28, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^2}{F} \frac{1}{t*} \ln\left(\frac{\Delta H *}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.59613

Well Parameters:

| | | | |
|---|---|---|---|
| L | 3 | m | Saturated length of screen or open hole |
| D | 0.03175 | m | Diameter of well |
| $r_c$ | 0.01588 | m | Radius of well |

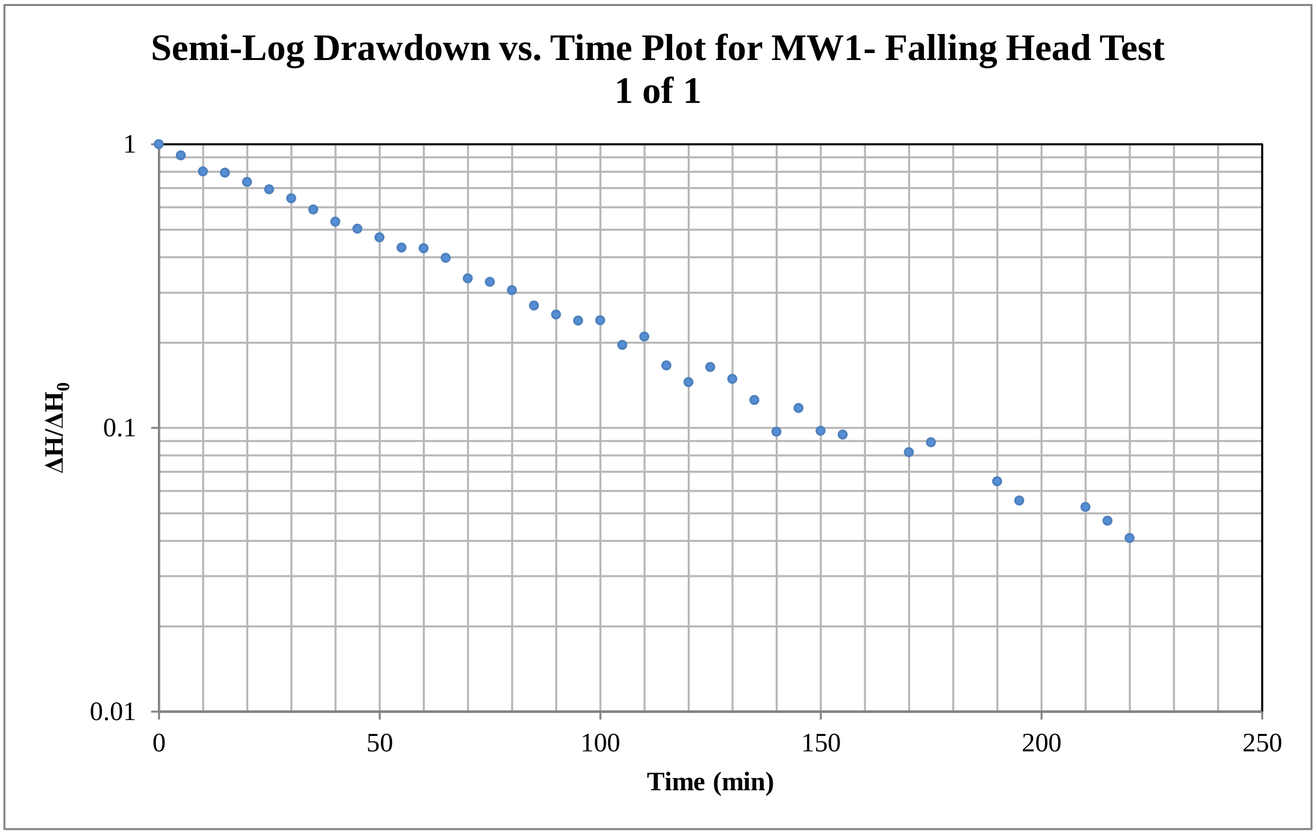
Data Points (from plot):

t*: 6.057 minutes $\Delta H^*/\Delta H_0$: 0.37

**Horizontal Hydraulic Conductivity**
**K = 6.02E-07 m/sec**

## Hvorslev Hydraulic Conductivity Analysis

Project: Claridge Homes - 829 Carling Avenue
Test Location: MW1
Test: 1 of 1 Falling Head
Date: April 27, 2021



Semi-Log Drawdown vs. Time Plot for MW1- Falling Head Test 1 of 1

Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^{\,2}}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.80425

Well Parameters:

| | | | |
|---|---|---|---|
| L | 2.1 | m | Saturated length of screen or open hole |
| D | 0.038 | m | Diameter of well |
| $r_c$ | 0.019 | m | Radius of well |

Data Points (from plot):

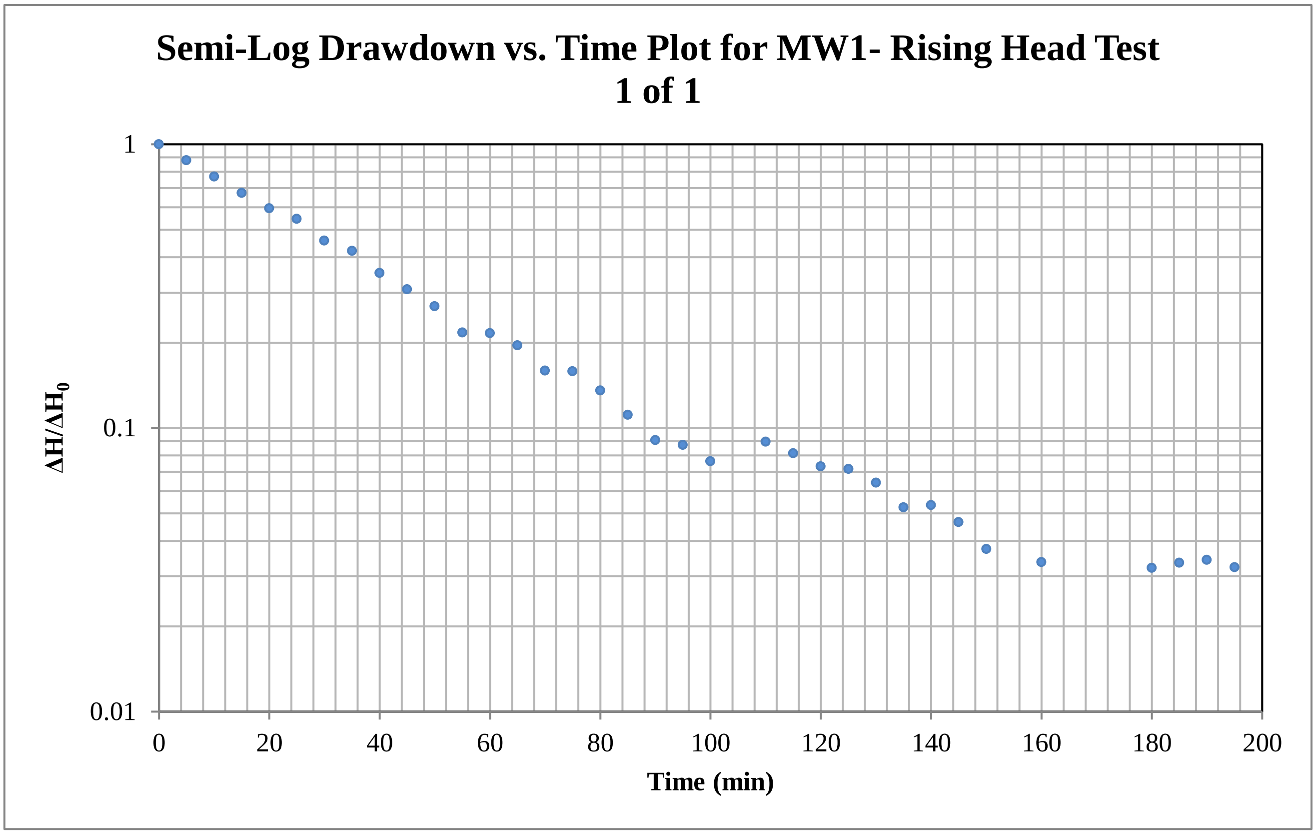
t*: 67.273 minutes

$\Delta H^*/\Delta H_0$: 0.37

**Horizontal Hydraulic Conductivity**
**K = 9.96E-08 m/sec**

## Hvorslev Hydraulic Conductivity Analysis

Project: Claridge Homes - 829 Carling Avenue
Test Location: MW1
Test: 1 of 1 Rising Head
Date: April 28, 2021



Semi-Log Drawdown vs. Time Plot for MW1- Rising Head Test 1 of 1

Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t*}\ln\left(\frac{\Delta H *}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.80425

Well Parameters:

| | | | |
|---|---|---|---|
| L | 2.1 | m | Saturated length of screen or open hole |
| D | 0.038 | m | Diameter of well |
| $r_c$ | 0.019 | m | Radius of well |

Data Points (from plot):

t*: 38.700 minutes $\Delta H*/\Delta H_0$: 0.37

**Horizontal Hydraulic Conductivity**
**K = 1.73E-07 m/sec**

**Order #: 2117544**

Certificate of Analysis

**Client: Paterson Group Consulting Engineers**

**Client PO: 29754**

Report Date: 27-Apr-2021

Order Date: 22-Apr-2021

**Project Description: PG5744**

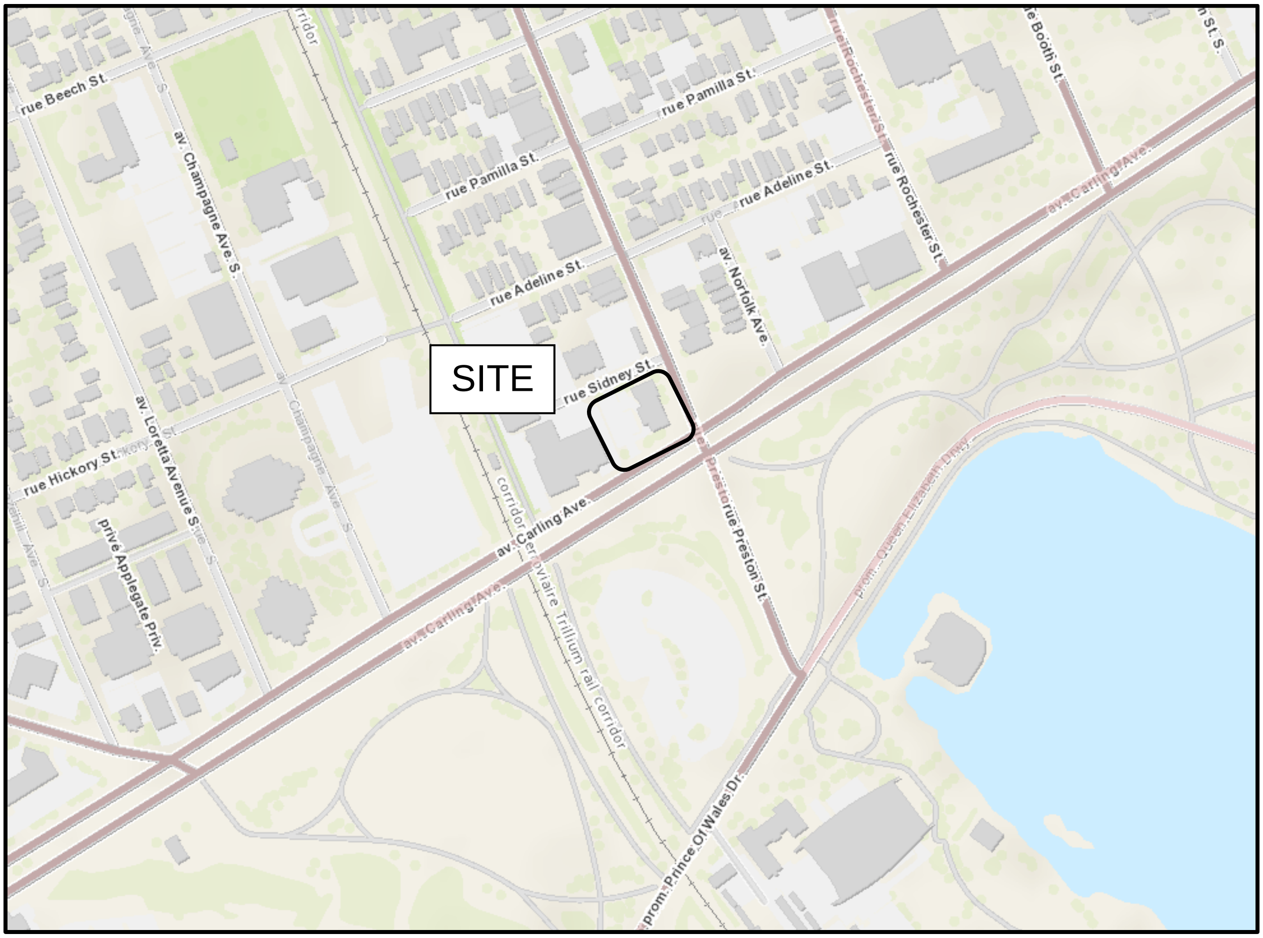
| | | | | | |
|---|---|---|---|---|---|
| | **Client ID:** | BH2-21 SS3 | - | - | - |
| | **Sample Date:** | 21-Apr-21 09:00 | - | - | - |
| | **Sample ID:** | 2117544-01 | - | - | - |
| | **MDL/Units** | Soil | - | - | - |
| **Physical Characteristics** | | | | | |
| % Solids | 0.1 % by Wt. | 90.4 | - | - | - |
| **General Inorganics** | | | | | |
| pH | 0.05 pH Units | 10.09 | - | - | - |
| Resistivity | 0.10 Ohm.m | 13.5 | - | - | - |
| **Anions** | | | | | |
| Chloride | 5 ug/g dry | 133 | - | - | - |
| Sulphate | 5 ug/g dry | 433 | - | - | - |

# APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 3 - SHEAR WAVE VELOCITY PROFILES

DRAWING PG5744-1 - TEST HOLE LOCATION PLAN



# FIGURE 1

## KEY PLAN

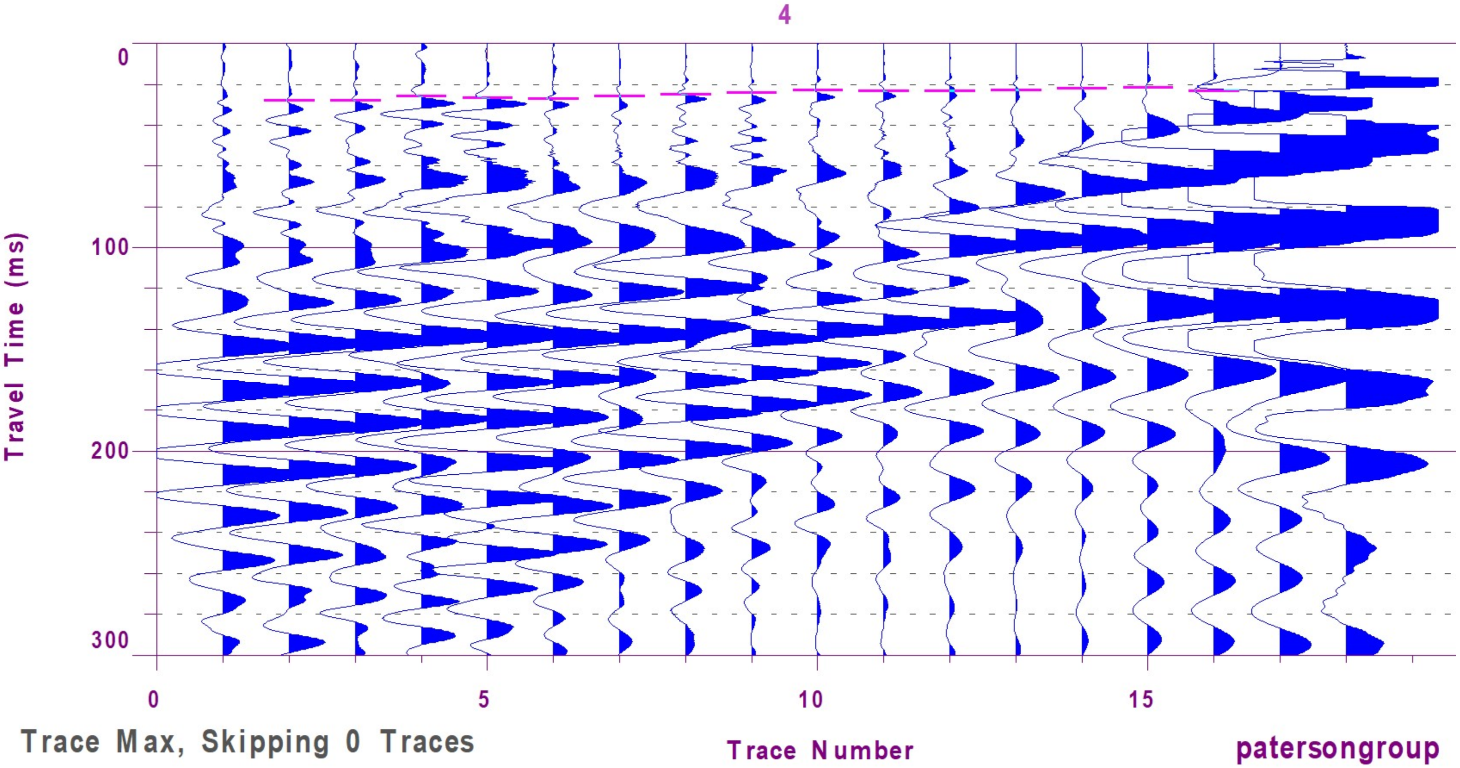


Figure 2 – Shear Wave Velocity Profile at Shot Location 18 m

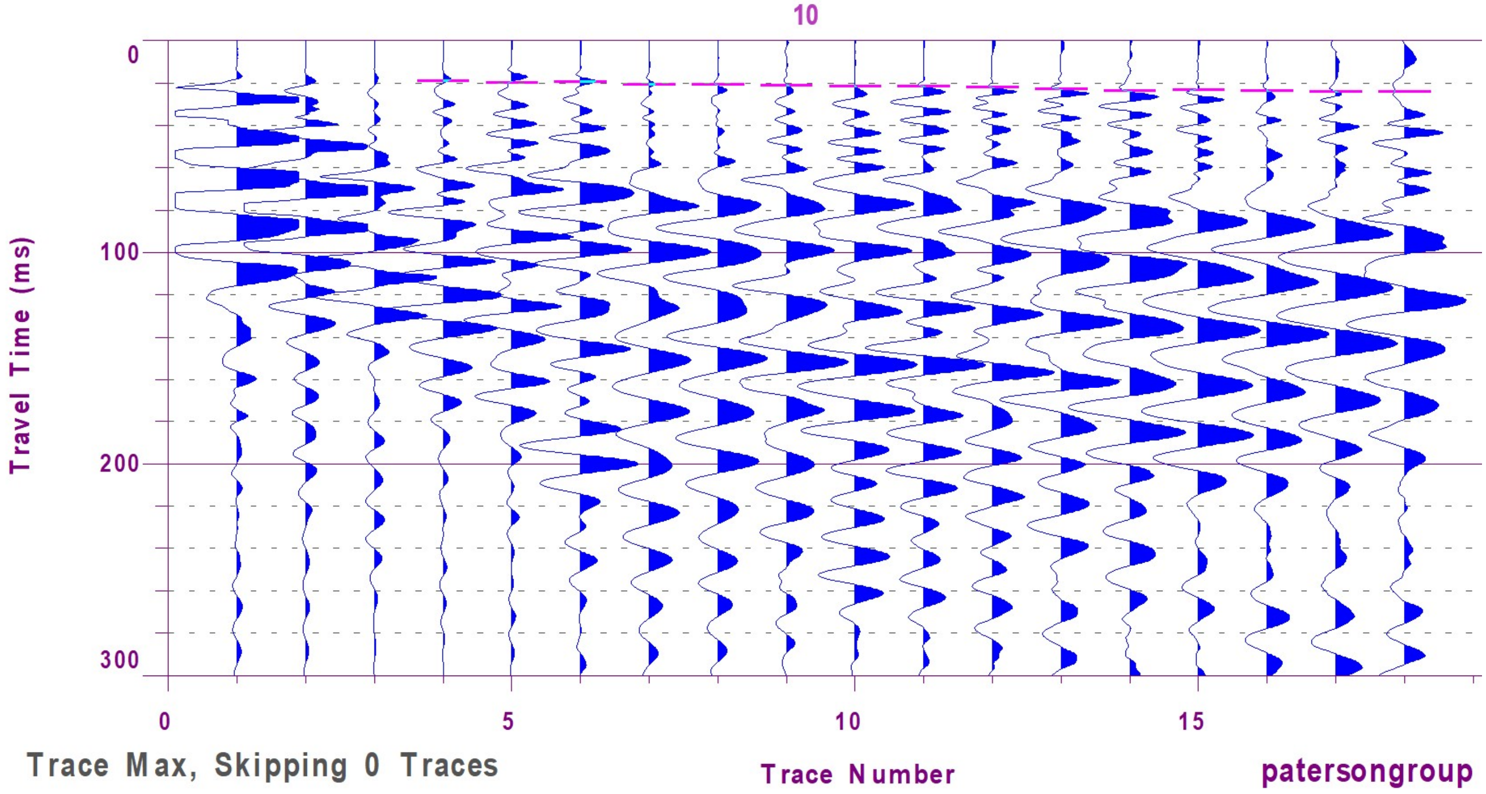
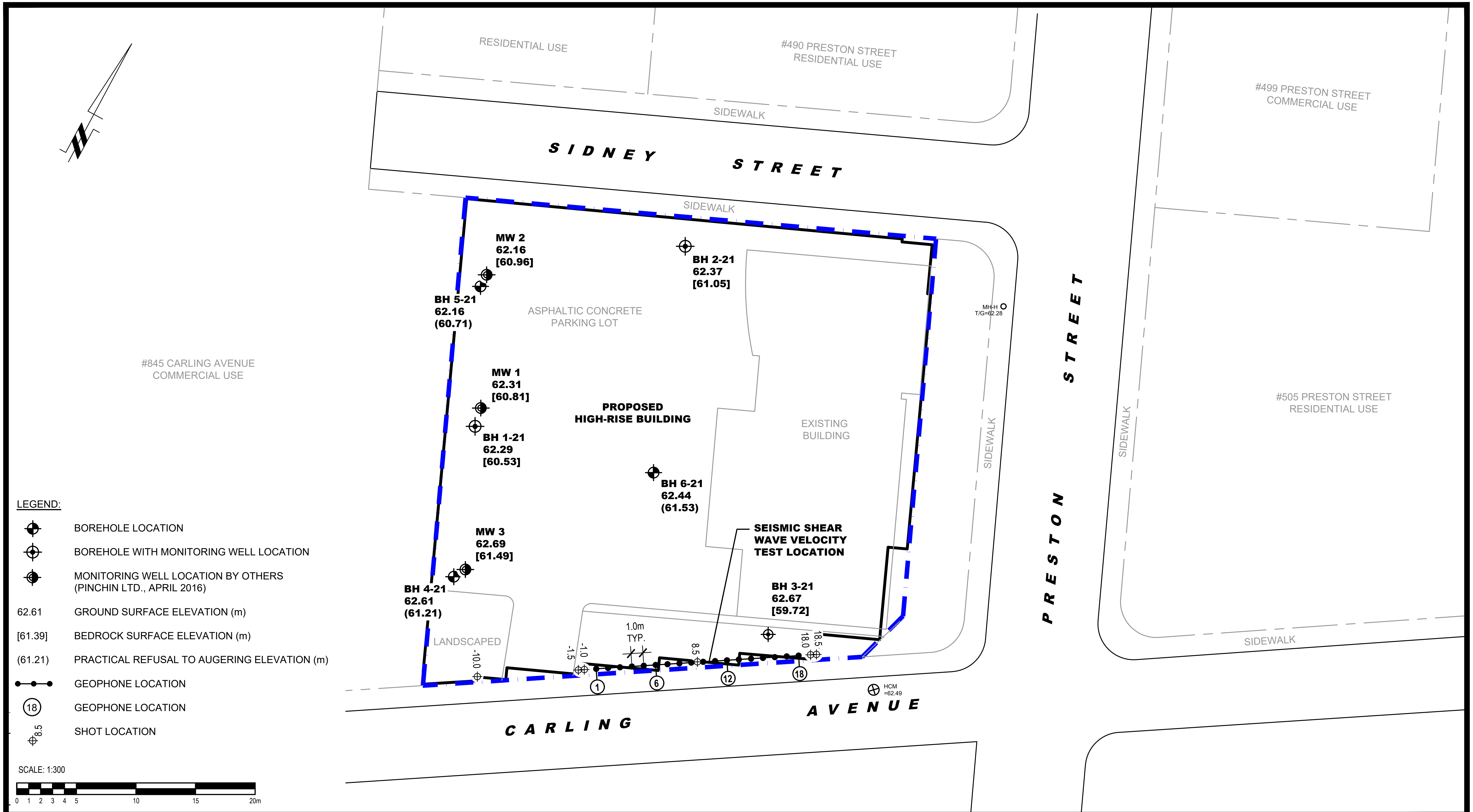


Figure 3 – Shear Wave Velocity Profile at Shot Location -1.5 m



LEGEND:

| | |
|---|---|
| ⊕ | BOREHOLE LOCATION |
| ⊕ | BOREHOLE WITH MONITORING WELL LOCATION |
| ⊕ | MONITORING WELL LOCATION BY OTHERS<br>(PINCHIN LTD., APRIL 2016) |
| 62.61 | GROUND SURFACE ELEVATION (m) |
| [61.39] | BEDROCK SURFACE ELEVATION (m) |
| (61.21) | PRACTICAL REFUSAL TO AUGERING ELEVATION (m) |
| ●—●—● | GEOPHONE LOCATION |
| ⑱ | GEOPHONE LOCATION |
| ⊕ 8.5 | SHOT LOCATION |

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

**paterson**group
**consulting engineers**

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| | | | |

CLARIDGE HOMES

GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE BUILDING - 829 CARLING AVENUE
OTTAWA, ONTARIO

Title:

# TEST HOLE LOCATION PLAN

Scale: 1:300
Date: 04/2021
Drawn by: JM
Report No.: PG5744-1
Checked by: SD
Dwg. No.: PG5744-1
Approved by: DJG
Revision No.:

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11x17