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Geotechnical Engineering

Environmental Engineering

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Archaeological Services

Geotechnical Investigation
Proposed Buildings C and D
Lebreton Flats
Ottawa, Ontario

Prepared For

Claridge Homes

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Report PG5203-1
Revision 1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to carry out a geotechnical investigation for the proposed Buildings C and D to be located at Lebreton Flats in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

❏ Determine the subsoil and groundwater conditions at this site by means of boreholes.

❏ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the 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's our understanding that the proposed development will consist of 2 high-rise structures (25 to 30 storeys) which will have 3 levels of shared underground parking. Further, it is understood that the footprint of the underground parking levels will extend to the property lines, and beyond the limits of the overlying structures. The proposed buildings will be surrounded by asphalt-paved access lanes with landscaped margins.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on February 24 and 25, 2020, and consisted of 4 boreholes which were advanced to a maximum depth of 12 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG5203-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed with a track-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.

Furthermore, 3 boreholes from previous investigations carried out by others, are also included to supplement our findings.

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. This testing was done in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

Rock 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 of this report.

Groundwater

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

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. The ground surface elevations at the borehole locations were referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5203-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and the results are discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant and generally grass covered, with a few gravel-surfaced areas around the perimeter. However, based on available aerial photos, the site has been used in recent years as a contractor staging area for adjacent developments. Reference should be made to the aerial photographs in Figure 2 - Aerial Photograph - 2011, Figure 3 - Aerial Photograph - 2014, and Figure 4 - Aerial Photograph - 2017 which illustrate the former and present site conditions. Further, it is understood that in 2003, an environmental remedial program involved the removal of all overburden from the site and its subsequent replacement with fill material.

The site is bordered by Fleet Street to the north, Lloyd Street to the west, and Lett Street to the south and east. An aqueduct is located beyond Lett Street to the south. The existing ground surface across the site is relatively level at approximate geodetic elevation 55 to 56 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of an approximate 2.1 to 3 m thickness of fill underlying the topsoil. The fill was generally observed to consist of a silty sand to sand and gravel.

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

Bedrock

Practical refusal to augering was encountered at depths ranging from 2.1 to 3 m below the existing ground surface. Bedrock was cored at the borehole locations, and was observed to consist of limestone with interbedded shale. Based on the RQDs of the recovered rock core, the upper 1.5 m of the bedrock can be classified as poor to fair in quality, becoming good to excellent in quality at depth.

Based on available geological mapping, the bedrock at the subject site consists of limestone of the Verulam formation.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells on March 2, 2020. The observed groundwater levels are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1	56.13	2.48	53.65	March 2, 2020
BH 2	55.66	2.39	53.27	March 2, 2020
BH 3	55.49	4.07	51.42	March 2, 2020
BH 4	55.39	4.66	50.73	March 2, 2020

Note:
- The ground surface elevations at the borehole locations are referenced to a geodetic datum.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 3 to 4 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered suitable for the proposed development, from a geotechnical perspective. The proposed multi-storey buildings are 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 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.

Due to the depth of the proposed underground parking garage, a water suppression system is recommended to lessen the volume of water infiltration over the long term during post-construction.

The above and other considerations are further 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.

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

Based on the volume of the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by 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 to 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 velocities (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 also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be excavated with almost vertical side walls. A minimum 1 m horizontal ledge should remain between the overburden excavation and the bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden shoring system.

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.

Bedrock Reinforcement and Stabilization

Due to the founding depth of the proposed buildings, bedrock stabilization may be required where the proposed foundation extends into the limestone bedrock.

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.

Fill Placement

Fill used for grading beneath the proposed buildings 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

It's expected that a mass excavation will take place and the bottom of the excavation will be relatively uniform to accept a concrete mud slab. Footings will be poured over this concrete mud slab which will also be acting as a horizontal hydraulic barrier for the water suppression system.

Concrete Hydraulic Barrier

To create a horizontal hydraulic barrier at depth, it's recommended that a concrete mud slab be placed on the bedrock surface which has been subexcavated 150 mm to accommodate this additional concrete thickness. The bearing surface should be inspected by the geotechnical engineer prior to concrete placement. The concrete mud slab should consist of a 150 mm thick layer with a minimum 25 MPa compressive strength.

Bearing Resistance Values - Auxiliary Structures

Footings for auxiliary structures such as canopies and vent shafts placed on a clean, surface sounded limestone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa**, incorporating a geotechnical resistance factor of 0.5. Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

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.

Bearing Resistance Values - Parking Garage and High-Rise Buildings

A factored bearing resistance value at ULS of **6,000 kPa** can be used for footings founded on the concrete mud slab overlying a sound limestone bedrock at the proposed founding elevation for the lower parking garage, provided the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. The bedrock vertical face, along the excavation sides and within depressed areas such as the elevator pit, can be assessed by the geotechnical engineer to confirm the soundness of the bedrock at depth.

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 horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock or soil bearing medium will require a lateral support zone of 1H:1V (or shallower).

5.4 Design for Earthquakes

A seismic shear wave velocity test was completed for the subject site to accurately determine the applicable seismic site classification for the proposed buildings based on Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity test was completed by Paterson personnel. Two seismic shear wave velocity profiles from the on-site testing are presented in Appendix 2.

Field Program

The seismic shear wave test was completed through the centre of the property, as presented in Drawing PG5203 -1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 18 horizontal geophones in a straight line in roughly a north-south orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was connected to a 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 strikes an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four to eight times at each shot location to improve signal to noise ratio. The shot locations are completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the center of the geophone array, as well as 3, 4.5 and 25 m away from the first and last geophones.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was completed by 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, Vs_{30}, of the upper 30 m below the structures 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 by 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. As bedrock quality increases, the bedrock shear wave velocity also increases.

Based on the testing results and the boreholes completed at the site, the footings for the proposed buildings will be founded directly on bedrock.

The Vs_{30} was calculated using the standard equation for average shear wave velocity from the Ontario Building Code (OBC) 2012.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\sum\left(\frac{(Depth_i(m))}{Vs_i(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2{,}557m/s}\right)}$$

$$V_{s30} = 2{,}557m/s$$

Based on the seismic test results, the average shear wave velocity, Vs_{30}, for foundations at the subject site is **2,557 m/s**. Therefore, a **Site Class A** is applicable for design of the proposed buildings, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not considered to be susceptible to liquefaction.

5.5 Basement Slab

For the proposed development, all overburden soil will be removed from the building footprints, leaving the concrete mud slab over the bedrock as the founding medium for the basement floor slab. It is anticipated that the basement area for the proposed buildings 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.

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 encountered at the time of the field investigation, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a sump pit, should be provided in the subfloor fill under the lower basement floor (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 buildings. 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 of 13 kN/m^3).

It is expected that 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 unit weight of 15.5 kN/m^3) where this condition occurs. Further, a seismic earth pressure component will not be applicable for foundation walls which are 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 and 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
γ = 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 (ΔP_{AE}).
The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = 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:

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

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 buildings, 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 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 **0.821 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2.

Table 2 - 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 Hoek and Brown parameters	65 m=0.821 and s=0.00293
Unconfined compressive strength - Limestone	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 3. 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 buildings are determined.

Table 3 - 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	1.2	4.4	750
	4.5	2.0	6.5	1000
125	1.6	0.6	2.2	600
	2.0	1.0	3.0	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 Structure

Access lanes and lower level parking areas are anticipated at this site. The proposed pavement structures are presented in Tables 4 and 5.

Table 4 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	**Material Description**
40	Wear Course - Superpave 12.5 Asphaltic Concrete
50	Binder Course - Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 5 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	**Material Description**
150	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over concrete mud slab/bedrock.	

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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Water Suppression System and Foundation Drainage

To manage and control groundwater water infiltration over the long term, the following water suppression system is recommended to be installed for the exterior foundation walls and underfloor drainage:

- ☐ The concrete mud slab will create a horizontal hydraulic barrier to lessen the water infiltration at the base of the excavation and will consist of a 150 mm thick layer of 25 MPa compressive strength concrete. The 150 mm minimum thickness is required to enable the support of construction traffic until the footings are poured and the area is backfilled.

- ☐ A waterproofing membrane will be required to lessen the effect of water infiltration for the underground parking levels starting at 4 m below finished grade. The waterproofing membrane will consist of bentonite panels fastened to the grinded bedrock surface. The membrane should extend to the bottom of the excavation at the founding level of the proposed footings over the concrete mud slab. Consideration can be given to doubling the bentonite panels in the lower P3 level where minor hydrostatic pressure will be created.

- ☐ A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the bottom of the foundation wall. It's expected that 150 mm diameter sleeves placed at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area. Water infiltration will result from two sources. The first will be water infiltration from the upper 4 m which is above the vertical waterproofed area. The second source will be groundwater breaching the waterproofing membrane.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration below the lowest underground parking level slab that breaches the horizontal hydraulic barrier (minimum 150 mm thick concrete mud slab). For design purposes, it's recommended that a 150 mm diameter perforated pipe be placed in each bay. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Water Infiltration Volumes

During the construction phase, it's expected that water infiltration should have a steady state volume of less than 400,000 L/day plus any surface water infiltration following a precipitation event. The initial influx will be greater once the excavation extends below the long term groundwater level. The zone of influence associated with the temporary dewatering during the construction excavation for 3 levels of underground parking will be approximately 15 m.

Based on the proposed water suppression system, it's expected that long term groundwater infiltration will be significantly reduced during post-construction. With a properly implemented water suppression system, it's expected that post-construction volumes will be less than 30,000 L/day.

Foundation Backfill

Where required, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I or OPSS Granular A granular material, should otherwise be used for this purpose.

Adverse Effects from Dewatering on Adjacent Structures

The temporary dewatering program during construction will have a limited zone of influence of less than 15 m from the foundation perimeter. The underlying subsurface conditions is essentially bedrock. In our opinion, no adverse effects to surrounding structures and infrastructure within the nearby roadway right of way are expected.

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 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.
As noted above, 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.

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.

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 shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services.

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 their stability. The toe of the shoring is recommended to be adequately supported to resist toe failure, if required, 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 6 - 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
Unit Weight (γ), kN/m^3	21
Submerged 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 used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used 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) Category 3 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 on Neighbouring Properties

It is understood that 3 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 suppression 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 a moderate to slightly 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 for water suppression system.

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

☐ Sampling and testing of the concrete and fill materials.

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

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

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

A report confirming the work has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials 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 for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Scott Dennis

Scott S. Dennis, P.Eng.

Carlos P. Da Silva, P.Eng., ing., QP_{ESA}

LICENSED PROFESSIONAL ENGINEER
Apr. 6, 2020
C. P. DA SILVA
PROVINCE OF ONTARIO

Report Distribution

☐ Claridge Homes (e-mail copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

BOREHOLE LOGS BY OTHERS

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Buildings C and D - Lebreton Flats
Ottawa, Ontario

DATUM Geodetic

REMARKS

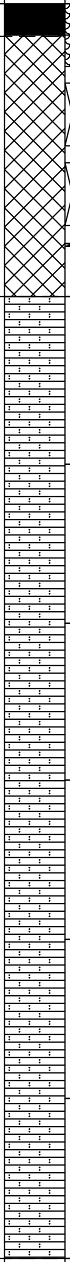
BORINGS BY CME 55 Power Auger

DATE 2020 February 24

FILE NO. PG5203

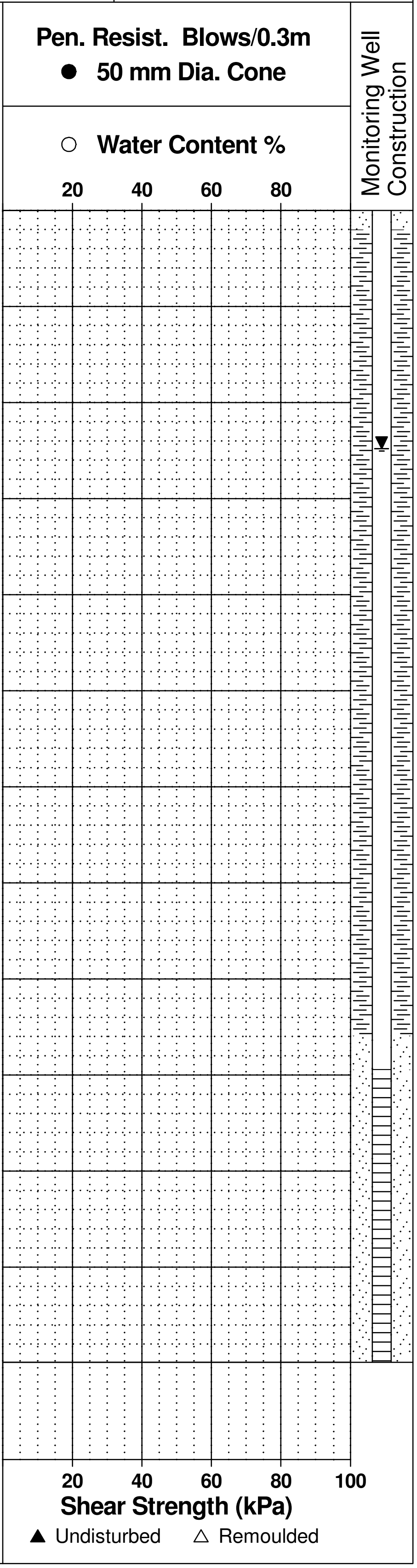
HOLE NO. BH 1

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.30
FILL: Brown silty sand with clay, gravel and organics, trace cobbles	2.80
BEDROCK: Fair to excellent quality, grey limestone with interbedded shale seams	11.99
End of Borehole (GWL @ 2.48m - March 2, 2020)	



SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	62	14
SS	3	83	52
SS	4	100	50+
RC	1	100	60
RC	2	100	100
RC	3	98	95
RC	4	100	100
RC	5	95	95
RC	6	100	93

DEPTH (m)	ELEV. (m)
0	56.13
1	55.13
2	54.13
3	53.13
4	52.13
5	51.13
6	50.13
7	49.13
8	48.13
9	47.13
10	46.13
11	45.13



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Buildings C and D - Lebreton Flats
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2020 February 24

FILE NO. PG5203

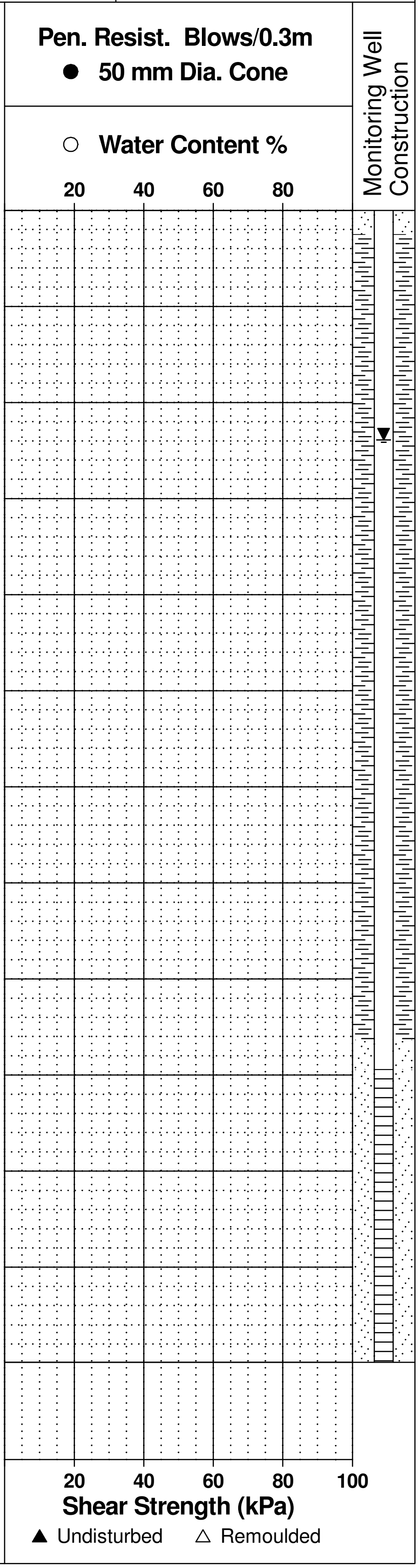
HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.30	AU	1		
FILL: Brown silty clay, trace gravel 0.99	SS	2	67	45
FILL: Brown sand with gravel, trace silt 3.02	SS	3	67	100
	SS	4		55
BEDROCK: Fair to excellent quality, grey limestone with interbedded shale seams 11.99	RC	1	100	53
	RC	2	100	100
	RC	3	100	68
	RC	4	100	80
	RC	5	100	95
	RC	6	100	100

End of Borehole

(GWL @ 2.39m - March 2, 2020)

DEPTH (m)	ELEV. (m)
0	55.66
1	54.66
2	53.66
3	52.66
4	51.66
5	50.66
6	49.66
7	48.66
8	47.66
9	46.66
10	45.66
11	44.66



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Buildings C and D - Lebreton Flats
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

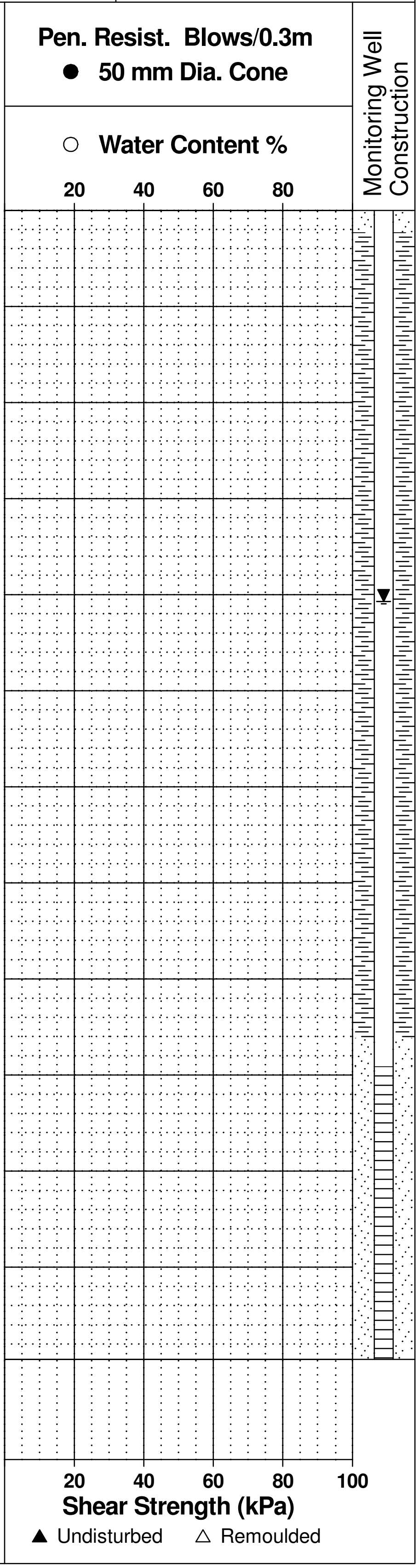
DATE 2020 February 25

FILE NO. PG5203

HOLE NO. BH 3

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30	AU	1		
FILL: Brown sand and gravel, trace silt		SS	2	75	103
		SS	3	14	50+
	2.13	SS	4	0	50+
BEDROCK: Weathered grey limestone	2.29	RC	1	90	34
BEDROCK: Poor to excellent quality, grey limestone with interbedded shale seams		RC	2	98	69
		RC	3	100	83
		RC	4	100	98
		RC	5	100	95
		RC	6	100	95
	11.96	RC	7	100	100
End of Borehole					
(GWL @ 4.07m - March 2, 2020)					

DEPTH (m)	ELEV. (m)
0	55.49
1	54.49
2	53.49
3	52.49
4	51.49
5	50.49
6	49.49
7	48.49
8	47.49
9	46.49
10	45.49
11	44.49



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Buildings C and D - Lebreton Flats
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2020 February 25

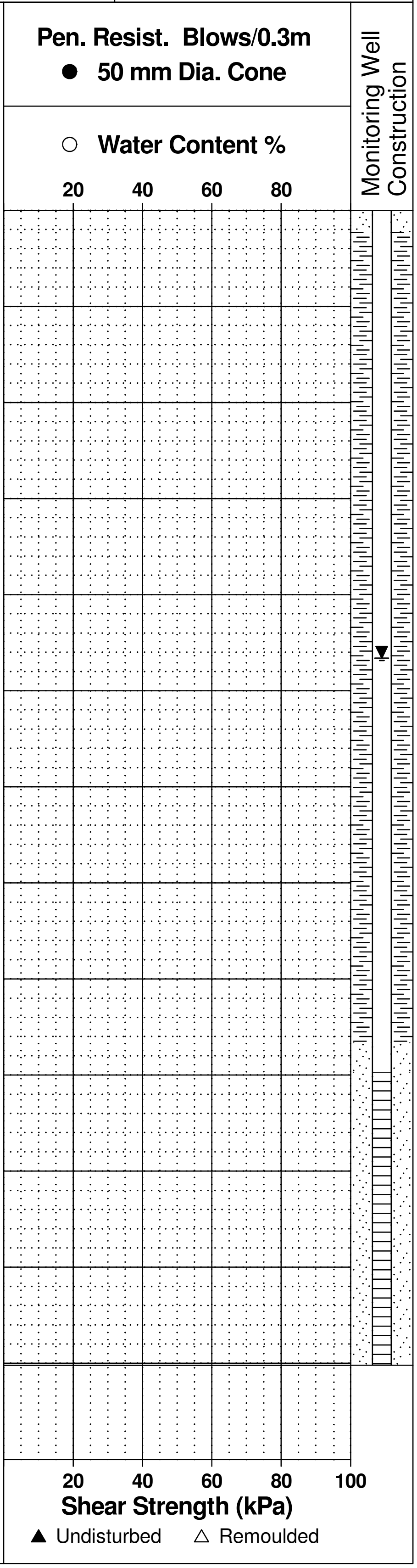
FILE NO. PG5203

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.30
FILL: Brown sand and gravel, trace silt	2.29
BEDROCK: Fair to excellent quality, grey limestone with interbedded shale seams	12.02
End of Borehole	
(GWL @ 4.66m - March 2, 2020)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	93	50+
SS	3	43	81
RC	1	97	57
RC	2	100	93
RC	3	98	95
RC	4	100	82
RC	5	98	85
RC	6	100	98
RC	7	100	100

DEPTH (m)	ELEV. (m)
0	55.39
1	54.39
2	53.39
3	52.39
4	51.39
5	50.39
6	49.39
7	48.39
8	47.39
9	46.39
10	45.39
11	44.39
12	43.39



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)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

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 = $(D30)^2 / (D10 \times D60)$
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)

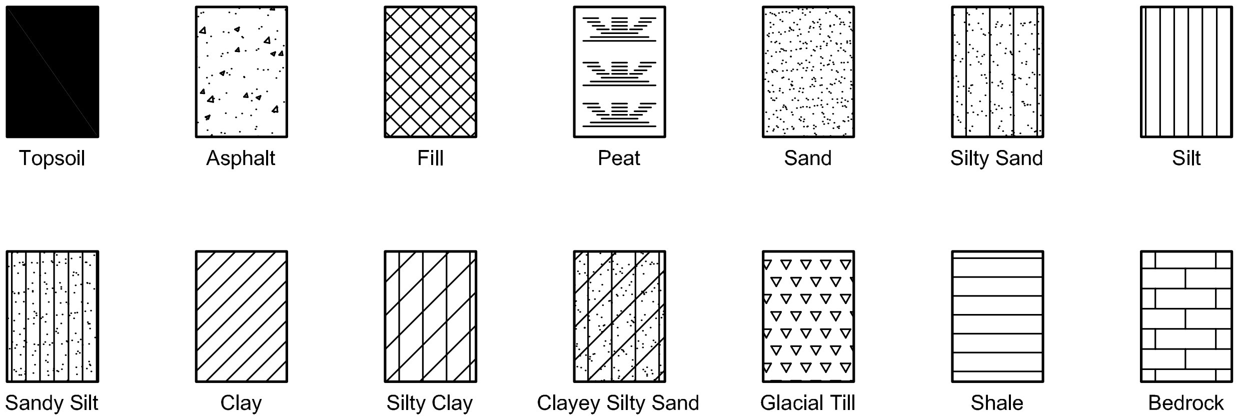
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

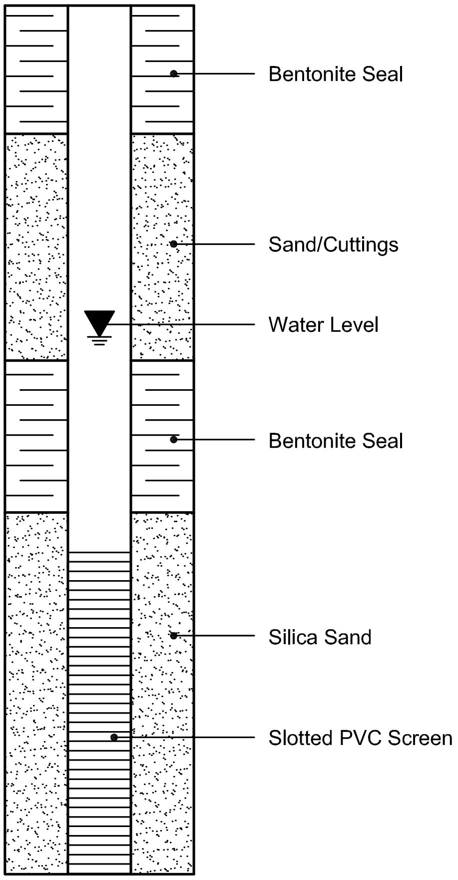
SYMBOLS AND TERMS (continued)

STRATA PLOT

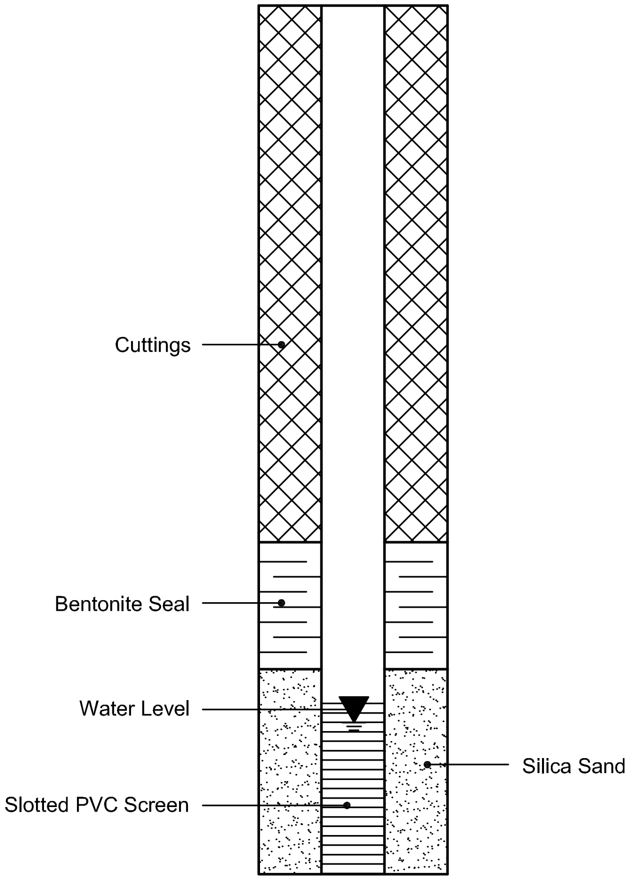


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEBRETON FLATS

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 179·0′ DATE APRIL 3, 1964

REMARKS SEE PLATE No. 2

HOLE No. 9

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	~~PROBING OR VANE TEST~~ ....LB. HAMMER ....INCH DROP NO CASING ....INCH DIA. ROD ~~BLOWS PER FOOT OR SHEAR STRENGTH IN KIPS PER FT.²~~
				GROUND SURFACE	0′	179·0′	
				FILL			← OVER-NIGHT WATER LEVEL 177·3′
		20	9.1	TILL WITH A LITTLE BRICK			
					3·9′	175·1′	
				ROCK			
				CORE RECOVERY 96%			
					9·6′	169·4′	
				BOTTOM OF HOLE			

% WATER CONTENT
NATURAL ○

PLATE

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CONSULTING ENGINEERS – INGÉNIEURS CONSEILS
OTTAWA CANADA

SOIL PROFILE & TEST SUMMARIES
PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

N.D.H.Q.
LEBRETON FLATS

ELEVATION OF GROUND SURFACE (ZERO DEPTH) / NIVEAU DU SOL (PROFONDEUR ZERO) 179·4'

DATE AUG. 10, 1966

HOLE / FORAGE No. 508

NOTES SEE PLATE No. 2

Compressive Strength K.S.F. / Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. / Petit Pénétromètre K/Pd.2	Essai-Standard Penetration Blows/ft.-Coups/pd	Sample No. Echantillon	DESCRIPTION OF SOIL / DU SOL Ground Surface → Niveau du Sol	Depth in Feet / Profondeur-Pied	Elevation / Niveau	PROBING OR VANE TEST / SONDAGE OU ESSAI AU MOULINET MARTEAU----HAMMER CHUTE LIBRE---DROP NO CASING / SANS TUBAGE BARRE-----DIA. ROD BLOWS/FOOT OR SHEAR STRENGTH K.S.F. COUPS/PIED OU RÉSISTANCE AU CISAILLEMENT K/PD.2
		29	508	FILL BROKEN ROCK	0'	179·4'	
				WEATHERED OR FRACTURED ROCK CORE RECOVERY 58%	2'	177·4'	
				ROCK CORE RECOVERY 92%	4'	175·4'	
				BOTTOM OF HOLE	8·9'	170·5'	← OVER-NIGHT WATER LEVEL BELOW 170·5'

R = REMOULDED-REMANIE

WATER CONTENT
% TENEUR EN EAU
NATURAL / NATURELLE ———— ⊙
LIQUID LIMIT / LIMITE DE LIQUIDITÉ ———— ⊡

PLATE / PLAQUE No.

Certificate of Analysis

Report Date: 05-Mar-2020

Client: Paterson Group Consulting Engineers

Order Date: 28-Feb-2020

Client PO: 29590

Project Description: PG5203

	Client ID:	BH2-SS3A	-	-	-
	Sample Date:	25-Feb-20 13:00	-	-	-
	Sample ID:	2009575-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	90.8	-	-	-
General Inorganics					
pH	0.05 pH Units	10.93	-	-	-
Resistivity	0.10 Ohm.m	8.61	-	-	-
Anions					
Chloride	5 ug/g dry	52	-	-	-
Sulphate	5 ug/g dry	1230	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 4 - AERIAL PHOTOGRAPHS (2011, 2014 and 2017)

FIGURES 5 TO 6 - SHEAR WAVE VELOCITY PROFILES

FIGURE 2 - BOREHOLE LOCATION PLAN BY OTHERS

DRAWING PG5203-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

patersongroup

FIGURE 2

Aerial Photograph - 2011

FIGURE 3

Aerial Photograph - 2014

FIGURE 4

Aerial Photograph - 2017

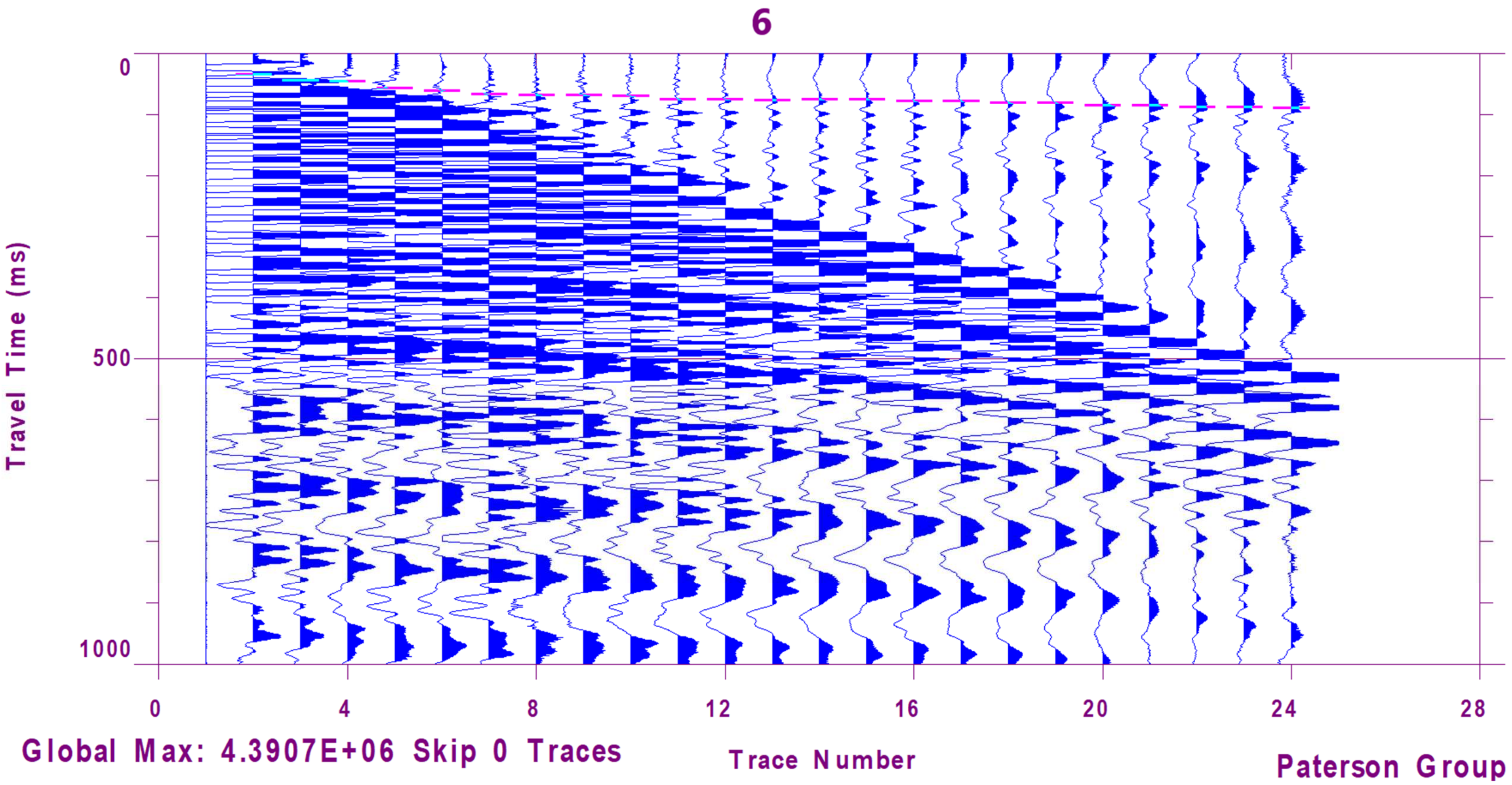


FIGURE 5 – Shear Wave Velocity Profile at Shot Location -4.5 m

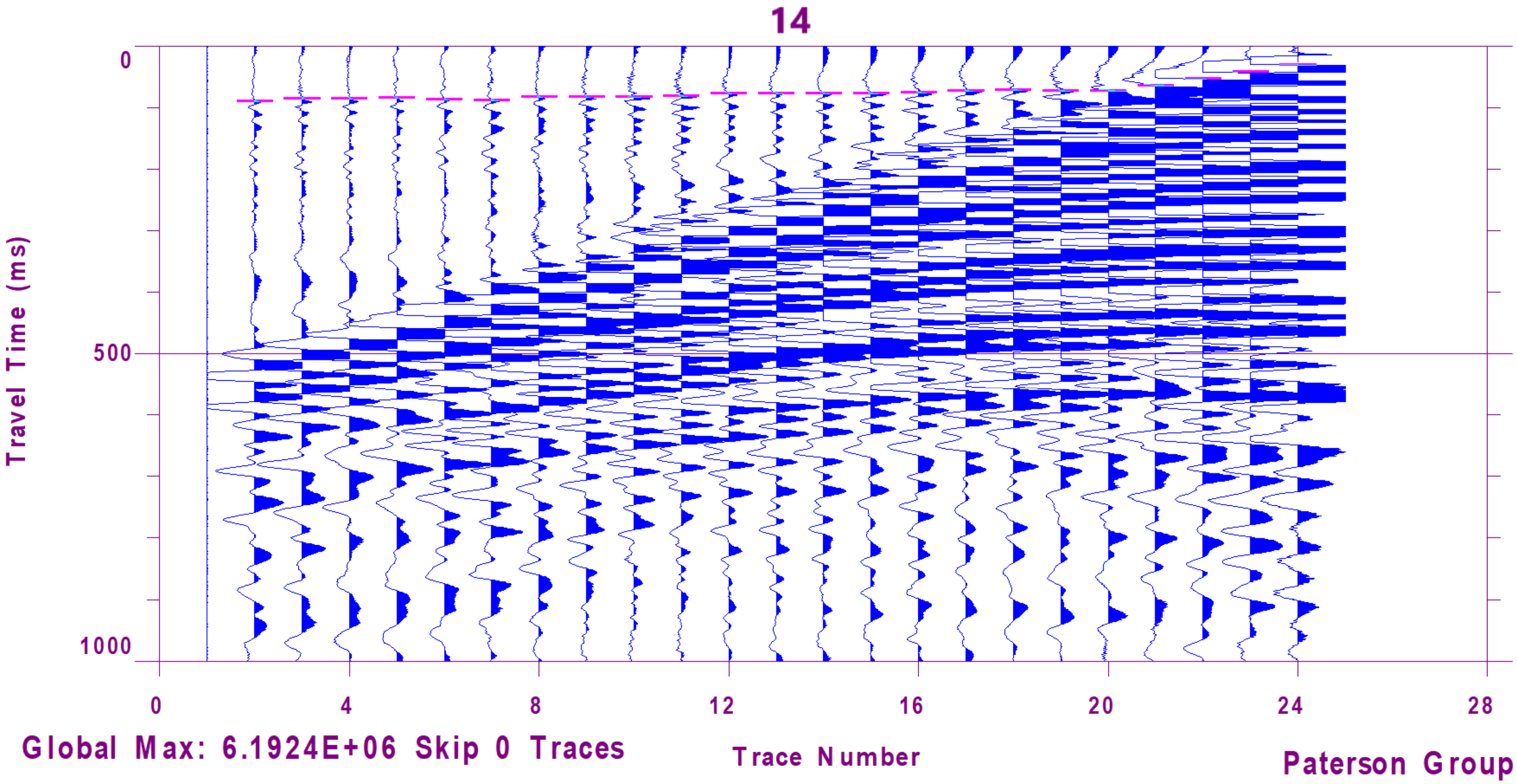
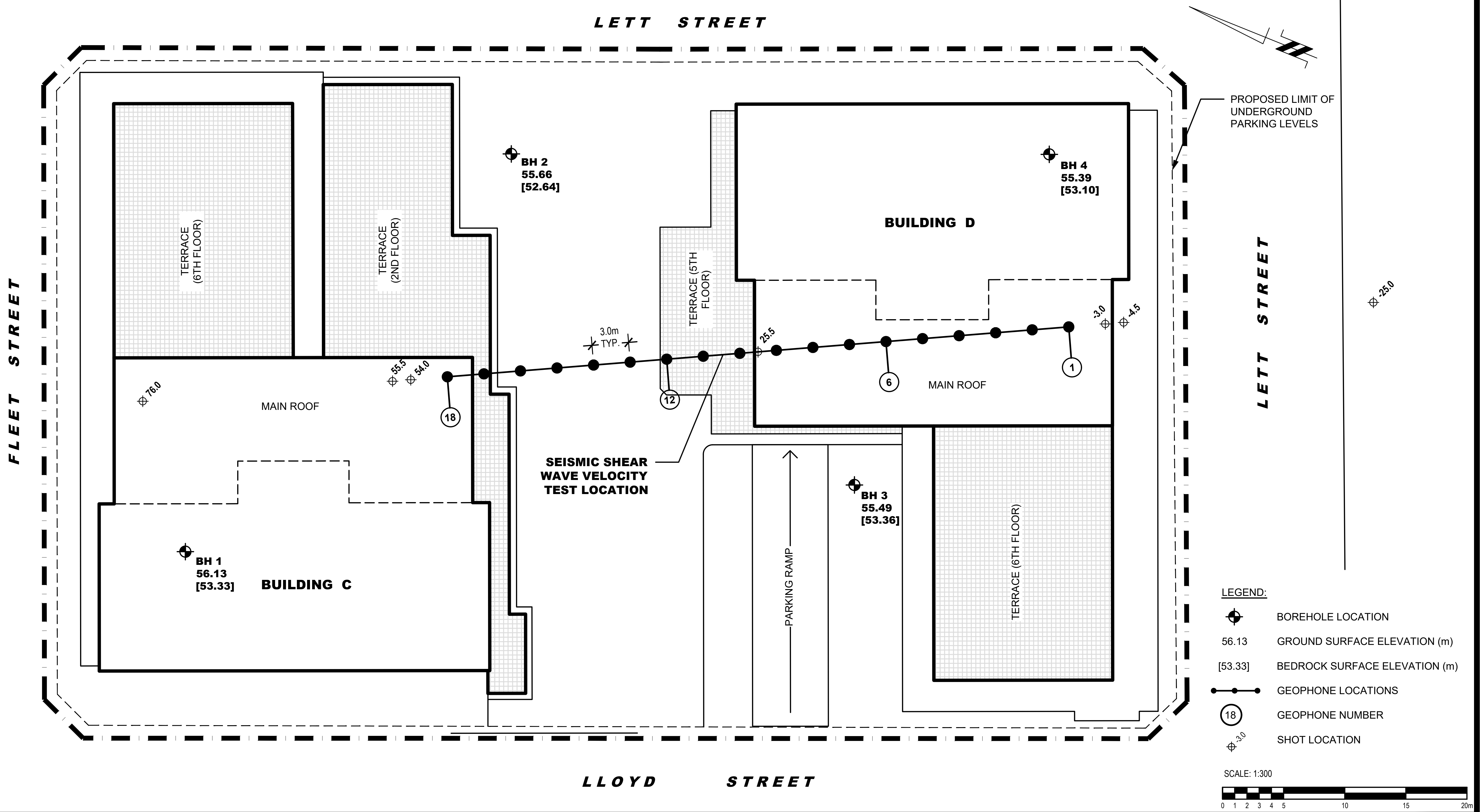


FIGURE 6 – Shear Wave Velocity Profile at Shot Location +73.5 m



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NO.	REVISIONS	DATE	INITIAL

CLARIDGE HOMES

GEOTECHNICAL INVESTIGATION
LEBRETON FLATS- PROPOSED BUILDINGS C AND D

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:300
Date: 03/2020
Drawn by: YA
Report No.: PG5203-1
Checked by: KP
Dwg. No.: PG5203-1
Approved by: SD
Revision No.:

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