PATERSON GROUP

Geotechnical Investigation

Proposed Multi-Storey Building

1500 Merivale Road – Phase 2
Ottawa, Ontario

Prepared for Claridge Homes
Report PG5561-3 dated September 2, 2026

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the proposed multi-storey building to be located within Phase 2 of the 1500 Merivale Road site in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- Determine the existing subsoil and groundwater information at this site by means of boreholes.
- Provide geotechnical recommendations for the design of the proposed development, including construction considerations which may affect its design.

This 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 available drawings, it is understood that Phase 2 of the proposed development will consist of a multi-storey building with 2 levels of underground parking which will extend beyond the building footprint, across the majority of the current phase.

It is further understood that the proposed building will include asphalt-paved access lanes and landscaped areas. It is also anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out from May 14 to 15, 2026, and consisted of 4 boreholes advanced to a maximum depth of 10.6 m below the existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the Phase 2 site and its vicinity.

A previous field program was conducted by Paterson within the vicinity of the subject site in November 2020, and by others in March 2013. A total of 6 boreholes were advanced to a maximum depth of 6.7 m during the previous field investigations. The approximate locations of the boreholes are shown on Drawing PG5561-5 - Test Hole Location Plan and included in Appendix 2.

The boreholes were advanced using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling 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.

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

Flexible polyethylene standpipes were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in Section 4.3 and presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and ground surface elevations at each borehole location completed during the current investigation, were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The location of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG5561-5 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

4.0 Observations

4.1 Surface Conditions

The Phase 2 site is located in the northeast portion of the overall 1500 Merivale Road property. The northern portion of the site is currently occupied by two single-story commercial buildings with asphalt and gravel surfaces along Baseline Road. The southern portion of Phase 2 consists of mature trees and vegetation.

Based on available aerial photos, additional buildings were historically located in the eastern portion of the current phase. These structures can be seen in the northern portion along Baseline Road as early as 1958 and appear to have been demolished between 1991 and 1999. Reference should be made to the aerial photographs in Figure 2 - Aerial Photograph - 1958, Figure 3 - Aerial Photograph - 1991, and Figure 4 - Aerial Photograph - 1999 in Appendix 2, which illustrate former and present site conditions, respectively.

The existing ground surface across the subject site slopes gently downward from north to south at approximate geodetic elevations of 101 to 99 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile across the site consists of topsoil or an asphaltic concrete underlain by an approximate 0.2 to 3.0 m thickness of fill followed by a glacial till deposit. The fill was generally observed to consist of a brown silty sand with gravel, crushed stone and trace topsoil and clay.

A glacial till deposit was encountered underlying the fill at approximate depths of 0.3 to 2.2 m below the existing ground surface. The glacial till was generally observed to consist of a dense to very dense, brown silty sand with gravel, cobbles, and boulders.

Bedrock

Auger refusal was encountered at the Phase 2 site at approximate depths ranging from 0.4 to 3.0 m below the existing ground surface. Bedrock was cored in all boreholes and was observed to consist of grey limestone. Based on the RQDs of the recovered rock core, the bedrock was generally fair to excellent in quality, generally improving with depth. At borehole BH 9-20 the upper 1.6 m of bedrock was observed to be weathered and of poor quality.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the subsurface profiles encountered at the borehole locations, and to Drawing PG5561-6 – Bedrock Surface Contour Plan in Appendix 2.

4.3 Groundwater

The groundwater level readings in the monitoring wells and standpipe piezometers are provided below in Table 1 below, and on the Soil Profile and Test Data sheet in Appendix 1.

Table 1 – Summary of Groundwater Level Readings

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-26	99.45	4.58	94.87	June 15, 2026
BH 2-26	98.98	3.70	95.28	June 15, 2026
BH 3-26	100.15	4.95	95.20	June 15, 2026
BH 4-26	100.89	5.00	95.89	June 15, 2026
BH 9-20*	100.22	4.92	95.30	November 10, 2020

Note: The ground surface elevation at each borehole location was surveyed by Paterson and was referenced to a geodetic datum.
* Denotes a monitoring well

Average groundwater levels were found to range from approximately **4 to 5 m** below the ground surface at the time of the geotechnical investigation. The recorded groundwater levels are also provided 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 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 building at the subject site is recommended to be founded on conventional spread footings bearing on the clean, surface sounded bedrock.

Where the clean, surface sounded bedrock is encountered below the underside of footing (USF) elevation, the overburden soils should be sub-excavated to the clean, surface sounded bedrock and lean concrete should then be placed up to the USF elevation.

Bedrock removal will be required to complete the excavation of the underground parking levels. Foundations and other debris from previously demolished structures may also be present at this site, and the need for their removal should be anticipated by the contractor.

Due to the presence of a silty clay layer, the proposed development will be subjected to grade raise restrictions. Our permissible grade raise recommendations are discussed in Section 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement-sensitive structures. Existing foundation walls and other construction debris should be entirely removed from within the building perimeter and within the lateral support zones of the foundations. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Bedrock Removal

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

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also 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 to 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.

Lean Concrete Placement

Where bedrock is encountered below the USF elevation, or where rock overbreak occurs at the USF elevation, lean concrete (minimum **17 MPa** compressive strength at 28 days) can be used to reinstate the subgrade from the bedrock surface up to the USF elevation. Typically, the excavation side walls will be used as the form to support the concrete. The lean concrete placement should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured will suffice in providing a direct transfer of the footing load to the underlying bedrock.

5.3 Foundation Design

Conventional Spread Footings bearing on Bedrock

Footings placed on a clean, surface sounded limestone bedrock surface, or on lean concrete which is placed directly over the clean, surface sounded bedrock, can be designed using a factored bearing resistance value at serviceability limit states (SLS) and ultimate limit states (ULS) of **3,000 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. The bedrock sounding should be carried out by the geotechnical consultant.

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.

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 flatter) passes only 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 flatter).

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site designation for the proposed structure in accordance with the Ontario Building Code (OBC) 2024. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 5 and 6 attached to the present report.

Field Program

The seismic array was located as presented in Drawing PG5561-5 - Test Hole Location Plan attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the surface by means of two 75 mm ground spike attached to the geophone land case. The geophones were spaced at 2 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a laptop computer 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 polarised 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 15.0, 3.0 and 2.0 m away from the first and last geophone, and at the centre of the geophone array.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct, reflected 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 soil profile below the 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.

Based on our testing results, the average overburden shear wave velocity is **309 m/s**, while the bedrock shear wave velocity is **2,443 m/s**. Further, the depth of the bedrock was interpreted at approximately 2 m depth.

The V_{s30} was calculated using the standard equation for average shear wave velocity and as presented below:

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{S_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{S_{Layer2}}(m/s)}\right)}$$

$$V_{s30=} \frac{30\ m}{\left(\frac{2\ m}{309\ m/s} + \frac{28\ m}{2{,}443\ m/s}\right)}$$

$$V_{s30=}\ 1{,}672\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30} , for the proposed building is 1,672 m/s. It should be noted that the bedrock is more than 3 m from the ground surface. Therefore, as per OBC 2024, a **Site Designation X_{1672}** is applicable for the design of the proposed structure at the subject site.

Based on Paterson's review of the in-situ soil characteristics, the soils underlying the subject site are not considered susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2024 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

Due to the anticipated founding elevation and relatively shallow bedrock, it is anticipated that 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 multi-storey building will be mostly parking and the recommended pavement structures noted in Section 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.

In consideration of the groundwater conditions encountered during the geotechnical investigation, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone layer under the lower-level floor slabs.

5.6 Basement Wall

There are several combinations of 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.

Two distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Lateral Earth Pressures

The static horizontal earth pressure (po) can be calculated using a triangular earth pressure distribution equal to Ko·γ·H where:

K_o = at-rest earth pressure coefficient of the applicable retained material (0.5)
γ = unit weight of fill of the applicable retained soil (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 this site is 0.309g according to OBC 2024 for a Site Class X_{1672}. 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 2024.

5.7 Rock Anchor Design

Where required for foundation uplift resistance, rock anchors could be used. The geotechnical design of grouted rock anchors in bedded sedimentary bedrock is based upon two possible failure modes.

The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. Both modes of failure have to be examined, as described in the following section. 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.

Anchors can be of the “passive” or the “prestressed/post-tensioned” type, depending on whether the anchor tendon is provided with prestress load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the prestressed type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well

an unbonded length, or free anchor length, between the rock surface and the start of the bonded length.

As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a shallower cone, and therefore less geotechnical resistance, than one where the bonded length was just the bottom part of the overall anchor.

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 60 and 90 MPa, which is stronger than most routine grouts. A factored at ULS tensile grout to rock bond resistance value of **1.0 MPa**, incorporating a resistance factor of 0.4, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

Based on the bedrock quality observed in the boreholes, and assuming that the bond zone will be in competent bedrock located 1 m below the bedrock surface, 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 2 on the following page:

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	62 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone	50 MPa
Unit weight - Submerged Bedrock	15.2 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75- and 125-mm diameter holes are provided in Table 3 below:

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

Other Considerations

The anchor drill holes should be a maximum of 1.5 to 2 times the rock anchor diameter, inspected by Paterson personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor hole. 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 and reviewed at the time of testing by Paterson field personnel. More information on proof testing can be provided upon request.

5.8 Pavement Structure

Lowest Underground Parking Level

For design purposes, it is recommended that the rigid pavement structure for the lowest 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 4 on the next page.

Table 4 – 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 lower 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.

Pavement Structure on Podium Deck

The pavement structures presented in Tables 5 and 6 should be used for car only parking areas, at grade access lanes, and heavy loading parking areas over the top of the podium structure.

Table 5 – Recommended Pavement Structure – Car Only Parking Areas Over Podium Deck	
Thickness (mm)	**Material Description**
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
200	**BASE** – OPSS Granular A Crushed Stone
See below*	**Thermal Break**** – Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced concrete podium deck *If specified by others, not required from a geotechnical perspective	

Table 6 – Recommended Pavement Structure – Access Lanes, Fire Truck Lane, Ramp, and Heavy Loading Areas Over Podium Deck	
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
300	**BASE** – OPSS Granular A Crushed Stone
See below*	**Thermal Break**** – Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced concrete podium deck *If specified by others, not required from a geotechnical perspective	

Pavement Structure on Overburden Soils

The flexible pavement structure presented in Table 7 should be used for access lanes and heavy loading areas.

Table 7 – Recommended Pavement Structure – Access Lanes and Heavy Loading Area	
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
300	**SUBBASE** – OPSS Granular B Type II
SUBGRADE – OPSS Granular B Type I or II material placed over in situ soil or engineered fill.	

Other Considerations

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 99% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage & Waterproofing

It is recommended that the proposed building's foundation walls be blind-poured against a composite drainage board which is fastened to the temporary shoring system or vertical bedrock face. Alternatively, where there is space to slope the excavation, the composite drainage board can be fastened to the foundation walls after they are poured.

It is also recommended that 100 mm diameter sleeves at 3 m centres be cast at the foundation wall/footing interface to allow for the infiltration of water from the composite drainage board to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area. Elevators and any other pits located below the sub-slab drainage system should be waterproofed.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration below the underground parking levels. For preliminary design purposes, we recommend that 100 mm perforated pipes be placed at approximately 6 m centers. 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.

Foundation Backfill

Where sufficient space is available for conventional backfilling, the backfill against the foundation walls within 1.8 m below finished grade should consist of clean sand or OPSS Granular B Type I granular material. Below the frost depth (approximately 1.8 m), the foundation walls can be backfilled with the site excavated fill, provided that this material is maintained in an unfrozen state and at a suitable moisture content for compaction.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

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

However, foundations which are founded directly on clean, surface-sounded bedrock, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

The underground parking area should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

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.

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

Due to the proximity of the proposed building to the property boundaries, temporary shoring may be required to support the overburden soils of the adjacent properties,

particularly along Baseline Road. The design and approval of the shoring system, where required, 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. Any changes to the approved shoring design system should be reported immediately to the owner's structural designer prior to implementation.

The temporary shoring system may consist of a soldier pile and lagging system which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures provided in Table 8. The earth pressures acting on the shoring system may be calculated using the parameters below

Table 8 – Soil Parameters for Shoring System Design

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
Total 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 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 is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil 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.

Bedrock Stabilization

Excavation side slopes in sound bedrock can be carried out using 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.

Rock bolts, chainlink fencing, and/or shotcrete 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 temporary chainlink fencing, shotcrete, and/or rock bolts should be evaluated by Paterson during the excavation operations and should be discussed with the structural engineer during the design stage of the project.

6.4 Pipe Bedding and Backfill

Bedding and backfill material should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Service 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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

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

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of 3 to 4 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. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

Impacts on Neighbouring Properties

It is understood that two levels of underground parking are planned for the proposed multi-storey building. Due to the presence of relatively shallow bedrock at, and in the vicinity of, the subject site, no adverse effects to neighbouring properties are expected as a result of dewatering which may be required.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site 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 using straw, propane heaters and 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.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

7.0 Recommendations

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

- ❒ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❒ Review the bedrock stabilization and excavation requirements.
- ❒ Observation of all bearing surfaces prior to the placement of concrete.
- ❒ Sampling and testing of the concrete and fill materials used.
- ❒ Observation of all subgrades prior to backfilling materials.
- ❒ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soils must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests 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 the 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 alternative use of the report.

Paterson Group Inc.







Jacob Priest, B.Eng.

Kevin A. Pickard, P.Eng.

Report Distribution:

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

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1500 Merivale Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364572.45 **NORTHING:** 5024937.38 **ELEVATION:** 99.45

PROJECT: Proposed Development - Phase 2

ADVANCED BY: CME-55 Low Clearance Dril

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** May 14, 2026

FILE NO. : PG5561

HOLE NO. : BH 1-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
TOPSOIL 0.23m [99.22m]			AU 1			19			99
FILL: Brown silty sand, some gravel, trace clay and topsoil 0.97m [98.48m]		1	SS 2	48	2-50-/-/ 50/0.05	22			98
GLACIAL TILL: Very dense, brown silty sand, with gravel, cobbles and boulders 1.93m [97.52m]			SS 3		50-/-/-/ 50/0.23	7			
BEDROCK: Good to excellent quality limestone		2	RC 1	95	RQD 81				97
		3	RC 2	100	RQD 95				96
		4						4.56 m 2026-06-15	95
		5	RC 3	100	RQD 100				94
		6							
		7	RC 4	100	RQD 100				93
		8	RC 5	100	RQD 93				92
		9							91
		10	RC 6	100	RQD 100				90
10.49m [88.96m]									89
End of Borehole		11							88
(GWL at 4.58 m depth - June 15, 2026)		12							

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1500 Merivale Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364603.62 **NORTHING:** 5024940.32 **ELEVATION:** 98.98

PROJECT: Proposed Development - Phase 2

ADVANCED BY: CME-55 Low Clearance Dril

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** May 14, 2026

FILE NO. : PG5561

HOLE NO. : BH 2-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
TOPSOIL 0.05m [98.93m]			SS 1	25	2-3-37-50 40	11			98
FILL: Brown silty sand, with gravel, trace clay and topsoil 0.28m [98.70m]		1	RC 1	100	RQD 60	1			97
GLACIAL TILL: Very dense, brown silty sand, with gravel, cobbles and boulders 0.43m [98.55m]		2	RC 2	100	RQD 93				96
BEDROCK: Fair quality limestone - Excellent quality by 1.52 m depth		3	RC 3	100	RQD 100			3.70 m 2026-06-15	95
		4	RC 4	100	RQD 100				94
		5	RC 5	98	RQD 96				93
- Vertical fracture from 7.52 m to 7.82 m depth		6	RC 6	10	RQD 92				92
10.52m [88.46m]		7	RC 7	100	RQD 100				91
End of Borehole		8							90
(GWL at 3.70 m depth - June 15, 2026)		9							89
		10							88
		11							87
		12							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1500 Merivale Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364635.26 **NORTHING:** 5024968.62 **ELEVATION:** 100.15

PROJECT: Proposed Development - Phase 2

ADVANCED BY: CME-55 Low Clearance Dril

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** May 15, 2026

FILE NO. : PG5561

HOLE NO. : BH 3-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
TOPSOIL 0.03m [100.12m]									100
FILL: Brown silty sand, with gravel, trace topsoil		1	SS 1	25	3-8-10-10 18				99
			SS 2	4	10-11-7-9 18				
2.21m [97.94m]		2	SS 3	42	1-2-3-2 5	17			98
GLACIAL TILL: Very dense, brown silty sand, with gravel, cobbles and boulders 2.74m [97.41m]			SS 4	50	14-18-50-/ 68/0.2	11			
BEDROCK: Poor quality limestone - Excellent quality by 3.12 m depth		3	RC 1	100	RQD 33				97
		4	RC 2	100	RQD 92				96
		5	RC 3	100	RQD 90				95
		6	RC 4	100	RQD 100				94
		7							93
		8	RC 5	100	RQD 100				92
		9							91
		10	RC 6	100	RQD 100				90
10.62m [89.53m] End of Borehole		11							89
(GWL at 4.95 m depth - June 15, 2026)		12							

4.95 m 2026-06-15

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1500 Merivale Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364596.80 **NORTHING:** 5025005.38 **ELEVATION:** 100.89

PROJECT: Proposed Development - Phase 2

ADVANCED BY: CME-55 Low Clearance Dril

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** May 15, 2026

FILE NO. : PG5561

HOLE NO. : BH 4-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / REMOULDED SHEAR STRENGTH (kPa) / UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
FILL: Brown silty sand, with gravel, some cruhsed stone 0.69m [100.20m]			SS 1	42	6-8-11-6 19	5			
FILL: Brown silty sand 1.45m [99.44m]		1	SS 2	21	12-10-6-4 16	11			100
FILL: Brown silty sand , with gravel 1.83m [99.06m]			SS 3	58	3-4-6-8 10	15			
GLACIAL TILL: Very dense, brown silty sand, with gravel, cobbles and boulders 2.44m [98.45m]		2	SS 4	25	16-22-50-/ 72/0.23				99
BEDROCK: Good to excellent quality limestone			RC 1	100	RQD 86				
		3							98
		4	RC 2	100	RQD 93				97
		5	RC 3	100	RQD 100			5.00 m 2026-06-15	96
		6							95
		7	RC 4	100	RQD 100				94
		8							93
			RC 5	100	RQD 95				
		9							92
		10	RC 6	100	RQD 98				91
10.54m [90.35m]									
End of Borehole		11							90
(GWL at 5.00 m depth - June 15, 2026)		12							89

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patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Complex - 1500 Merivale Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5561

REMARKS

HOLE NO. BH 9-20

BORINGS BY CME-55 Low Clearance Drill **DATE** 2020 November 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.22
TOPSOIL 0.10		AU	1				
GLACIAL TILL: Dense, brown silty sand with clay, gravel, cobbles and boulders 1.42		SS	2	80	50+	1	99.22
		RC	1	48	35	2	98.22
		RC	2	94	47		
BEDROCK: Poor to excellent quality, grey limestone		RC	3	100	90	3	97.22
		RC	4	78	57	4	96.22
		RC	5	100	86	5	95.22
6.68						6	94.22
End of Borehole							
(GWL @ 4.92m - Nov. 10, 2020)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

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)	'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 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

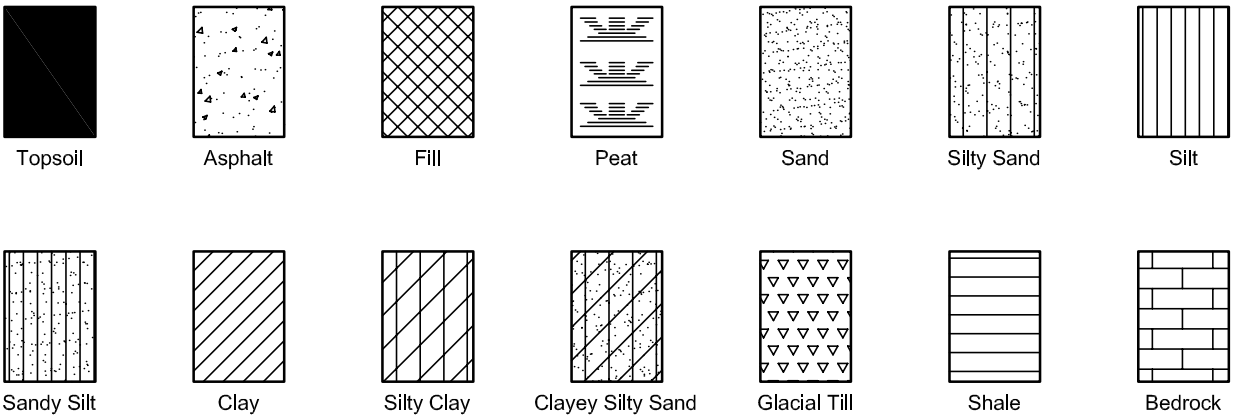
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

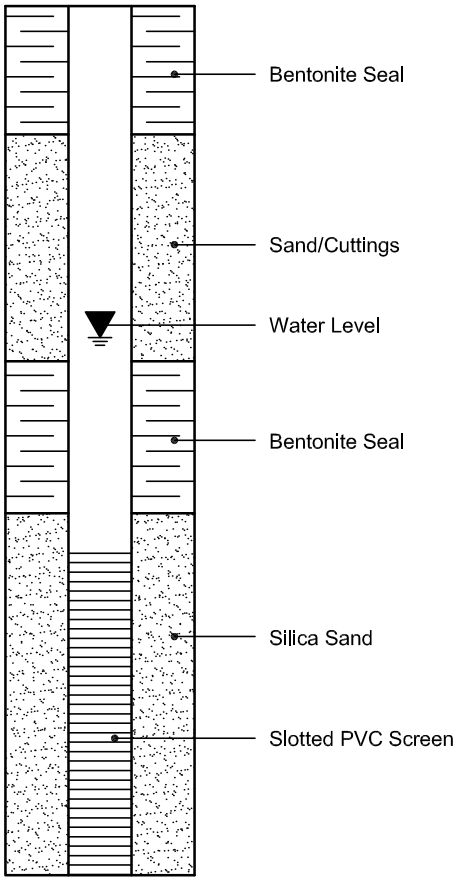
SYMBOLS AND TERMS (continued)

STRATA PLOT

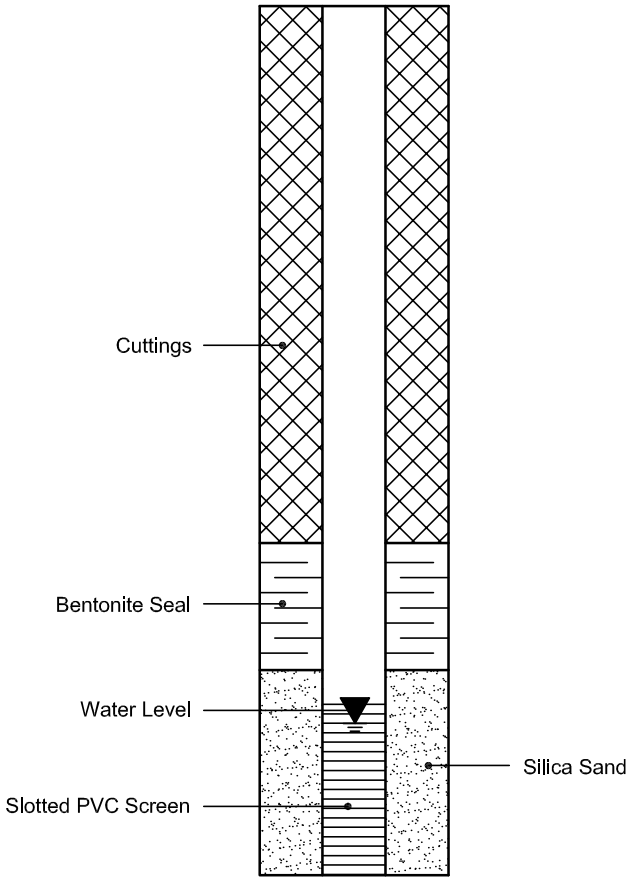


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Pinchin Environmental Ltd.
555 Legget Drive, Tower A
Suite 1001
Kanata, Ontario

Stratigraphic and Instrumentation Log: BH13-03

Project No.: 75040.001
Project: Phase II ESA
Client: Kempest Property Management Inc.
Location: Merivale and Baseline Road, Ottawa, Ontario

Logged By: M. Kosiw
Entered By: L. DiAngelo
Project Manager: M. Ryan
Drill Date: March 25, 2013

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation	Number	Type	Sample	N-Value	Recovery (%)	Vapour Reading (ppm) ppm 30 60 90 120	Monitoring Well
0		Ground Surface	0.0							
		Fill - Sand and Gravel damp, brown, no PHC odour or staining		SS1	Grab	SS	NA	20	0.0	
1				SS2	Grab	SS	NA	15	0.0	
			-1.5							
		Sand with gravel, moist, brown, no PHC odour or staining		SS3	Grab	SS	NA	50	0.0	
2			-2.3							
		Fill - Sand and Gravel moist to wet, brown, no PHC odour or staining		SS4	Grab	SS	NA	10	0.0	
3			-3.0							
		End of Borehole Refusal on Bedrock								
4										
5										
6										
7										
8										
9										

Drilled By: Downing (CME 55)
Drill Method: Spilt Spoon / Hollow Stem Auger
Vapour Instrument: Photoionization Detector
Well Casing Size: 38 mm

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

Pinchin Environmental Ltd.
555 Legget Drive, Tower A
Suite 1001
Kanata, Ontario

Stratigraphic and Instrumentation Log: MW13-05

Project No.: 75040.001
Project: Phase II ESA
Client: Kempest Property Management Inc.
Location: Merivale and Baseline Road, Ottawa, Ontario

Logged By: M. Kosiw
Entered By: L. DiAngelo
Project Manager: M. Ryan
Drill Date: March 26, 2013

Depth (m)	Symbol	Description	Elevation	Number	Type	Sample	N-Value	Recovery (%)	Vapour Reading (ppm) (ppm: 30, 60, 90, 120)	Monitoring Well
0		Ground Surface	101.7							
		Asphalt								
		Fill - Sand and Gravel damp, brown, no PHC odour or staining		SS1	Grab	SS	30	30	0.8	
1				SS2	Grab	SS	33	60	0.8	
2			99.6	SS3	Grab	SS	29	80	2.4	
		End of Borehole Refusal on Bedrock								
3										
4										
5										
6										
7										
8										
9										

Drilled By: Downing (CME 55)
Drill Method: Spilt Spoon / Hollow Stem Auger
Vapour Instrument: Photoionization Detector
Well Casing Size: 38 mm

Datum: Geodetic
Casing Elevation: NA
Ground Elevation: 101.73
Sheet: 1 of 1

Pinchin Environmental Ltd.
555 Legget Drive, Tower A
Suite 1001
Kanata, Ontario

Stratigraphic and Instrumentation Log: BH13-07

Project No.: 75040.001
Project: Phase II ESA
Client: Kempest Property Management Inc.
Location: Merivale and Baseline Road, Ottawa, Ontario

Logged By: M. Kosiw
Entered By: L. DiAngelo
Project Manager: M. Ryan
Drill Date: March 26, 2013

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation	Number	Type	Sample	N-Value	Recovery (%)	Vapour Reading (ppm) ppm 30 60 90 120	Monitoring Well
0		Ground Surface	0.0							
		Asphalt								
		Fill - Sand and Gravel damp, brown, no PHC odour or staining		SS1	Grab	SS	39	40	1.7	
1				SS2	Grab	SS	33	40	2.4	
2				SS3	Grab	SS	32	40	4.7	
			-2.6	SS4	Grab	SS	66	0	0.8	
		End of Borehole Refusal on Bedrock								
3										
4										
5										
6										
7										
8										
9										

Drilled By: Downing (CME 55)
Drill Method: Spilt Spoon / Hollow Stem Auger
Vapour Instrument: Photoionization Detector
Well Casing Size: 38 mm

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

Pinchin Environmental Ltd.
555 Legget Drive, Tower A
Suite 1001
Kanata, Ontario

Stratigraphic and Instrumentation Log: MW13-08

Project No.: 75040.001
Project: Phase II ESA
Client: Kempest Property Management Inc.
Location: Merivale and Baseline Road, Ottawa, Ontario

Logged By: M. Kosiw
Entered By: L. DiAngelo
Project Manager: M. Ryan
Drill Date: March 26, 2013

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation	Number	Type	Sample	N-Value	Recovery (%)	Vapour Reading (ppm) ppm 30 60 90 120	Monitoring Well
0		Ground Surface	101.9							
		Organics								
		Fill - Sand and Gravel damp, brown, no PHC odour or staining		SS1	Grab	SS	11	60	0.8	
1				SS2	Grab	SS	17	50	0.5	
2				SS3	Grab	SS	10	10	1.7	
			99.2	SS4	Grab	SS	NA	50	2.4	
3		End of Borehole Refusal on Bedrock								
4										
5										
6										
7										
8										
9										

Drilled By: Downing (CME 55)
Drill Method: Spilt Spoon / Hollow Stem Auger
Vapour Instrument: Photoionization Detector
Well Casing Size: 38 mm

Datum: Geodetic
Casing Elevation: NA
Ground Elevation: 101.92
Sheet: 1 of 1

Pinchin Environmental Ltd.
555 Legget Drive, Tower A
Suite 1001
Kanata, Ontario

Stratigraphic and Instrumentation Log: BH13-09

Project No.: 75040.001
Project: Phase II ESA
Client: Kempest Property Management Inc.
Location: Merivale and Baseline Road, Ottawa, Ontario

Logged By: M. Kosiw
Entered By: L. DiAngelo
Project Manager: M. Ryan
Drill Date: March 26, 2013

SUBSURFACE PROFILE				SAMPLE						
Depth (m)	Symbol	Description	Elevation	Number	Type	Sample	N-Value	Recovery (%)	Vapour Reading (ppm) ppm 30 60 90 120	Monitoring Well
0		Ground Surface	0.0							
		Asphalt								
		Fill - Sand and Gravel damp, brown, no PHC odour or staining		SS1	Grab	SS	30	40	0.5	
1				SS2	Grab	SS	33	50	0.5	
2				SS3	Grab	SS	27	5	1.2	
				SS4	Grab	SS	49	60	1.8	
3			-3.0							
		End of Borehole Refusal on Bedrock								
4										
5										
6										
7										
8										
9										

Drilled By: Downing (CME 55)
Drill Method: Spilt Spoon / Hollow Stem Auger
Vapour Instrument: Photoionization Detector
Well Casing Size: 38 mm

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

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - AERIAL PHOTOGRAPH - 1958

FIGURE 3 - AERIAL PHOTOGRAPH - 1991

FIGURE 4 - AERIAL PHOTOGRAPH - 1999

FIGURES 5 & 6 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5561-5 – TEST HOLE LOCATION PLAN

DRAWING PG5561-6 – BEDROCK SURFACE CONTOUR PLAN



FIGURE 1

KEY PLAN

PATERSON GROUP

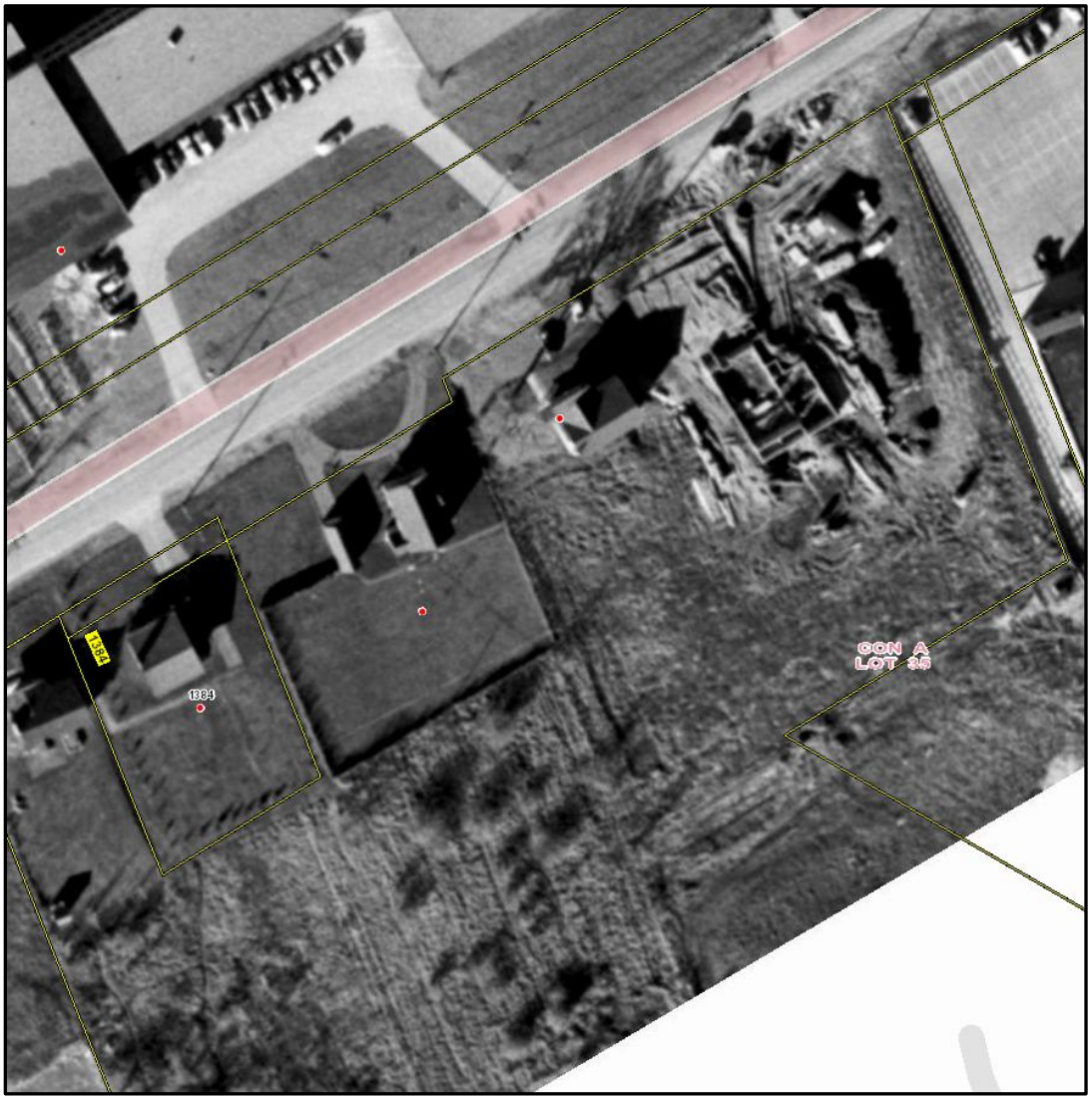


FIGURE 2

Aerial Photograph - 1958

PATERSON GROUP



FIGURE 3

Aerial Photograph - 1991

PATERSON GROUP

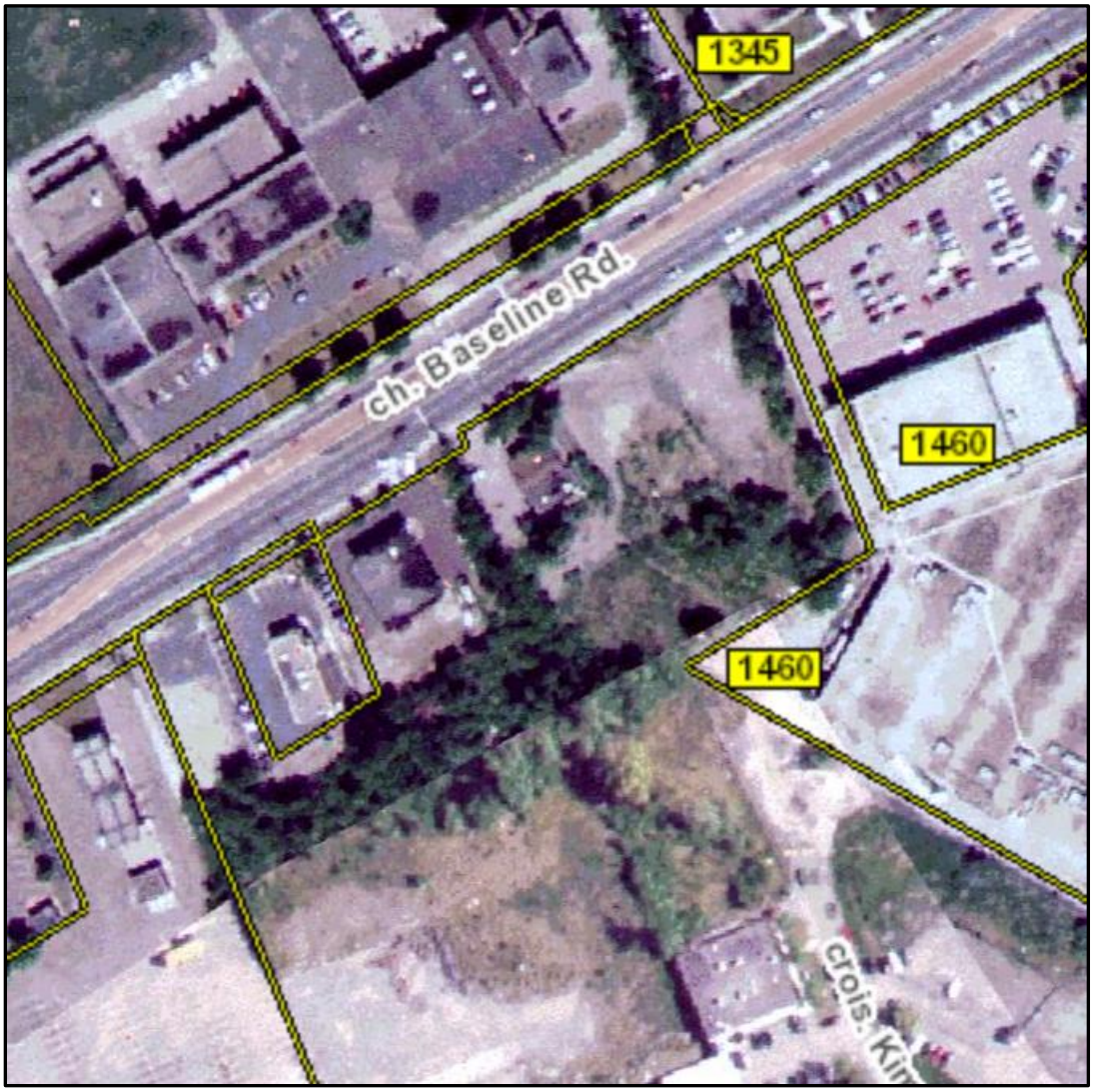


FIGURE 4

Aerial Photograph - 1999

PATERSON GROUP

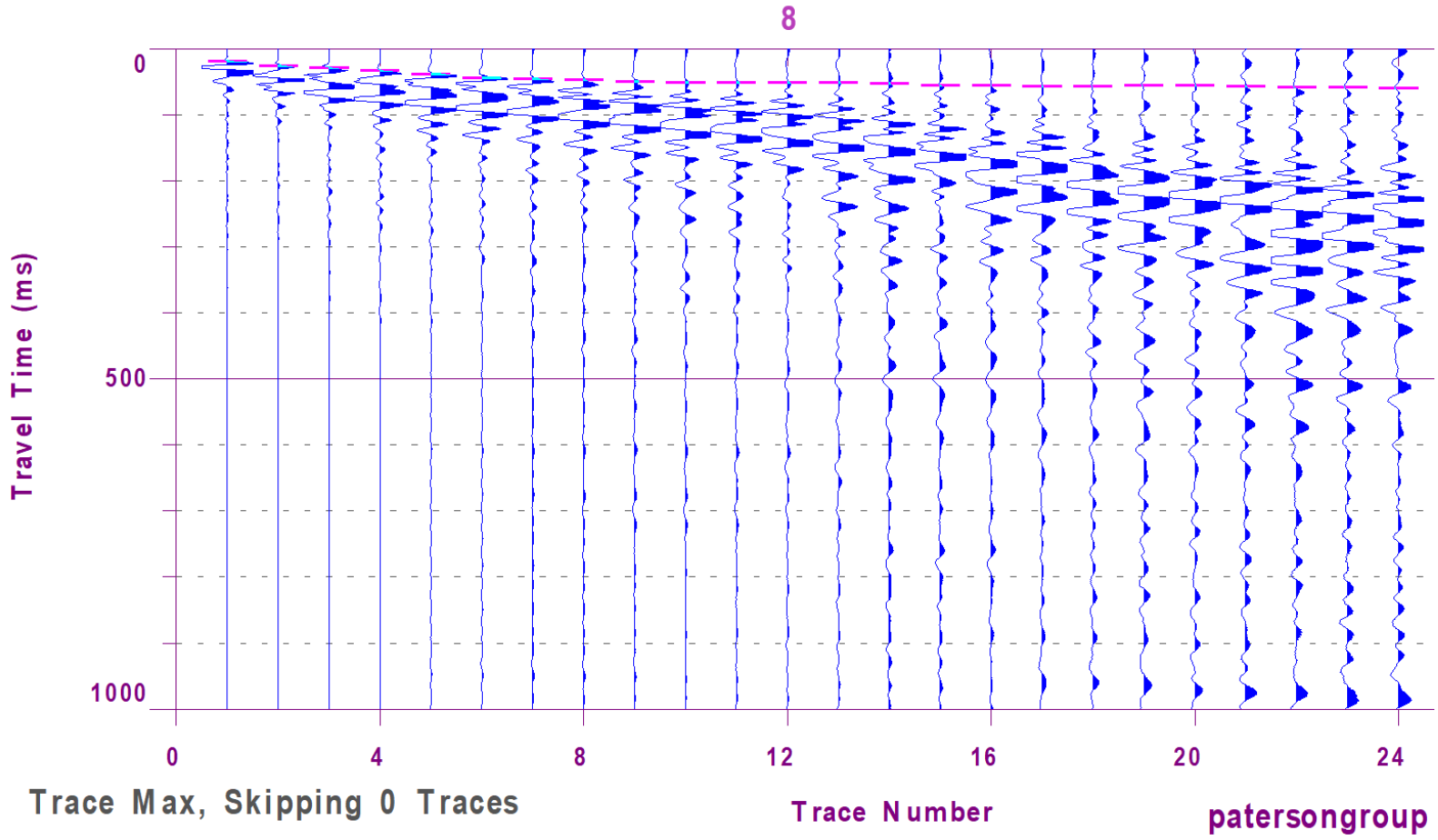


Figure 5 – Shear Wave Velocity Profile at Shot Location -3 m

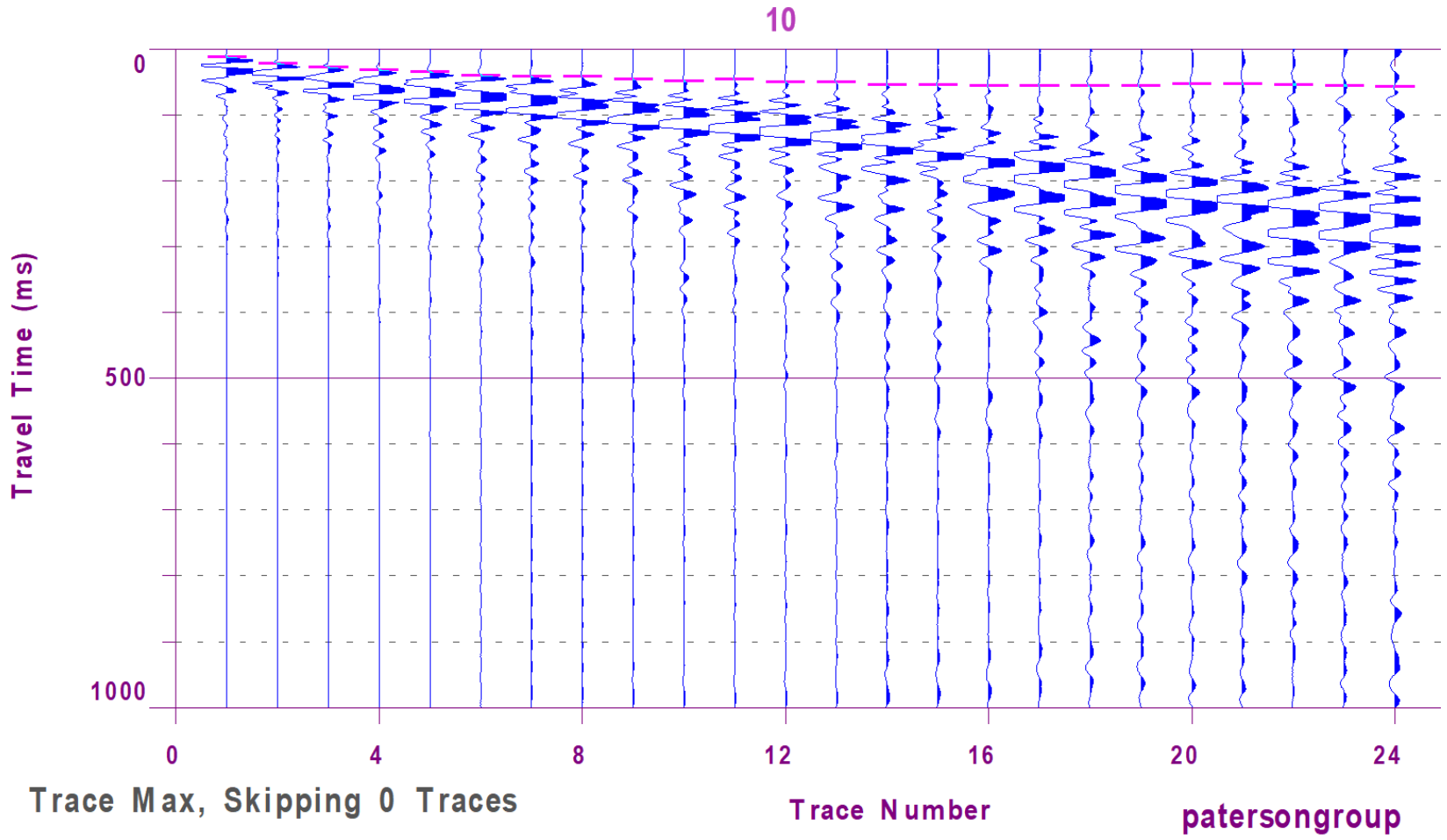
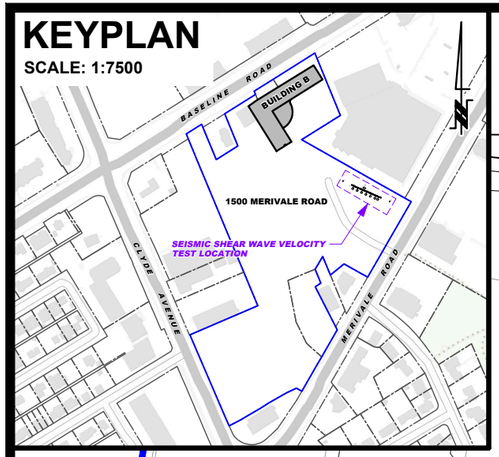


Figure 6 – Shear Wave Velocity Profile at Shot Location -2 m

KEYPLAN

SCALE: 1:7500



BASELINE ROAD

MW13-05

BH13-07

MW13-08

BH13-09

BH 1-26
99.45
[97.51]

EXISTING BUILDING

EXISTING BUILDING

BUILDING B
PROPOSED 9 STOREY
MIDRISE BUILDING

1 STOREY

BH 4-26
100.89
[98.45]

BH13-03

BH 3-26
100.15
[97.41]

BH 2-26
98.98
[98.55]

UNDERGROUND PARKING LIMIT

LEGEND:

- BOREHOLE LOCATION (CURRENT INVESTIGATION)
- BOREHOLE WITH MONITORING WELL LOCATION (2020)
- APPROX. MONITORING WELL LOCATION BY PINCHIN ENVIRONMENTAL
- APPROX. BOREHOLE LOCATION BY PINCHIN ENVIRONMENTAL
- 100.89 GROUND SURFACE ELEVATION (m)
- [97.51] BEDROCK SURFACE ELEVATION (m)

BH 9-20
100.22
[98.80]

CONCEPTUAL PLAN PROVIDED BY EVOQ ARCHITECTURE INC.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:500

0 5 10 15 20 25 30m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DD/MM/YYYY	INITIAL

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY DEVELOPMENT - PHASE 2
1500 MERIVALE ROAD
OTTAWA, ONTARIO

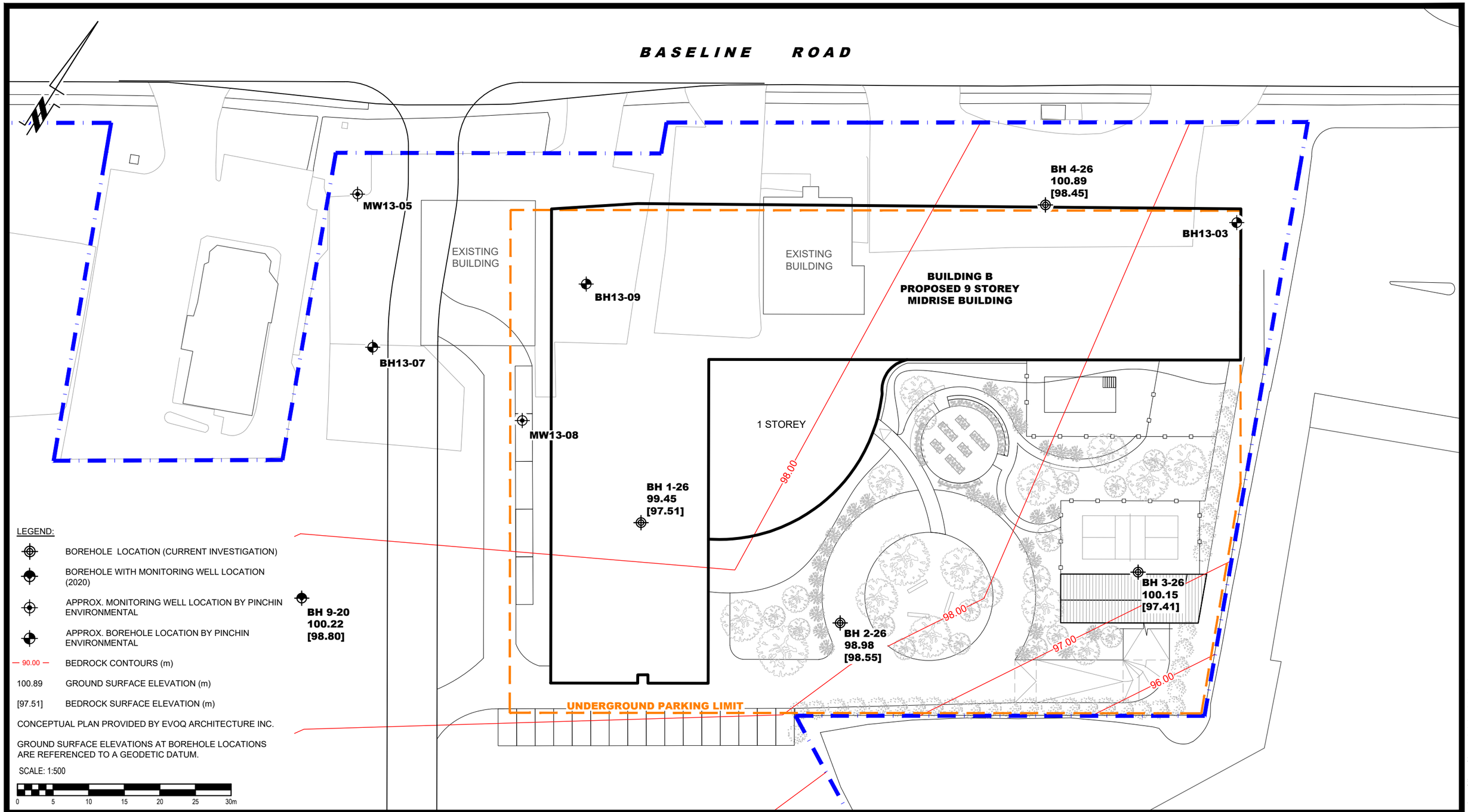
Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 06/2026
Drawn by: ZS	Report No.: PG5561-3
Checked by: JP	Dwg. No.: PG5561-5
Approved by: KP	Revision No.:

11x17

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LEGEND:

- BOREHOLE LOCATION (CURRENT INVESTIGATION)
- BOREHOLE WITH MONITORING WELL LOCATION (2020)
- APPROX. MONITORING WELL LOCATION BY PINCHIN ENVIRONMENTAL
- APPROX. BOREHOLE LOCATION BY PINCHIN ENVIRONMENTAL
- 90.00 — BEDROCK CONTOURS (m)
- 100.89 GROUND SURFACE ELEVATION (m)
- [97.51] BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY EVOQ ARCHITECTURE INC.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:500

0 5 10 15 20 25 30m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DD/MM/YYYY	INITIAL

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDINGS
1500 MERIVALE ROAD
OTTAWA, ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:500	Date: 06/2026
Drawn by: ZS	Report No.: PG5561-1
Checked by: JP	Dwg. No.: PG5561-6
Approved by: KP	Revision No.:

11x17

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