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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

Geotechnical Investigation

Proposed Multi-Storey Building
50 The Driveway
Ottawa, Ontario

Prepared For

Main and Main

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Report: PG5880-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Main and Main to conduct a geotechnical investigation for the proposed development to be located at 50 The Driveway in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

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

- Provide geotechnical recommendations pertaining to 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.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a multi-storey mixed-use structure with two levels of underground parking which will occupy the majority of the subject site. It is also understood that portions of the east and south existing building facades will be retained and integrated as part of the proposed building. However, the structure is expected to be demolished as part of the proposed development.

The proposed building will generally be surrounded by walkways and landscaped areas. It is also expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out during the period of June 30 through July 5, 2021. At that time three (3) boreholes and two (2) test pits were advanced to maximum depth of 20.5 m and 4.7 m below the existing ground surface, respectively. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration the location of underground utilities and site features. The test hole locations are shown on Drawing PG5880-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a low-clearance drill rig operated by a two-person crew. The test pits were excavated using a rubber-tired back-hoe. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of advancing each test hole to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. Grab samples were collected from the test pit sidewalls and by hand-auger recovery at selected intervals. The samples were classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger, split spoon and grab samples were recovered from the boreholes are shown as SS, AU and G, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The 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.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 1-21 and BH 5-21. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Monitoring wells were installed at boreholes BH 1-21, BH 4-21, and BH 5-21. Boreholes BH 2-21, BH 3-21 and BH 5-21 were fitted with flexible standpipe piezometers to allow for groundwater level monitoring. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- 3.0 m of slotted 51 mm PVC screen at the base of the boreholes.
- 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No. 3 silica sand backfill within annular space around screen.
- 300 mm thick bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson personnel using a handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5880-1 - 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 (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

Existing Conditions

The subject site is currently occupied by a three-storey institutional building with associated landscaped areas, parking areas and access lanes. The ground surface is relatively flat throughout the parking area. The ground surface around the eastern portion of the site slopes downwards gradually form north to south and between geodetic elevations of 68.5 to 66.0 m.

The site is bordered to the east by a paved pedestrian pathway and further by Queen Elizabeth Driveway, to the south by the Embassy of Germany and residential dwellings, to the west by townhouses and to the north by Lewis Street and further by a high-rise apartment building and the associated above-ground parking structure.

Historical Conditions

It should be noted Neville's Creek historically transected the southern portion of the subject site, which is understood to have been infilled in the late 19th century. The existing surface conditions have been completely altered since that time and are not considered representative of its previous footprint due to notable in-filling of the creek.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consisted of an asphalt pavement structure or topsoil underlain by a variable layer of fill. The fill was observed to generally consist of brown and/or grey silty clay or sand with varying amounts of gravel, cobbles, concrete, wood debris and organics. The fill was observed to extend to depths ranging between of 0.7 m to 6.7 m below the existing ground surface.

The fill layers were observed to be underlain by a deposit of silty clay. This deposit was generally observed to consist of a very stiff to stiff, brown silty clay crust underlain by a layer of stiff grey silty clay. It should be noted the crust layer was not encountered in the areas where the fill layer was encountered above the grey silty clay at BH 2-21 and BH 5 -21.

Practical refusal to DCPT was encountered at an approximate depth of 20.5 m and 22.1 m at the location of boreholes BH 1-21 and BH 5-21, respectively.

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

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic Shale of the Carlsbad formation, with an overburden drift thickness of 15 to 25 m depth.

Existing Building Foundation

Two test pits were advanced against portions of the existing building that are anticipated to be incorporated as part of the proposed development. The foundation wall was generally observed to consist of damp-proofed concrete and backfilled against by fill containing variable amounts of clay, silt, sand, gravel and inorganic debris. The top of the footing was encountered at an elevation of 63.3 and 62.2 m at TP 1-21 and TP 2-21, respectively. The underside of footing was encountered at an elevation of 63.0 m at TP 1-21 along with a clay drainage pipe.

The underside of footing was not encountered at TP 2-21 due to a combination of groundwater ingress and loose foundation backfill sidewalls unable to remain open. The top of the footing was inferred at an elevation of 62.2 m based on auger-probes carried out prior to in-filling the test pit at that time.

Based on our review of structural drawings prepared for The Canadian Nurses Association and dated October 1986, the southwestern and southeastern building addition is understood to be founded on piles anticipated to have been driven to refusal.

4.3 Groundwater

Groundwater levels were measured on July 6, 2021 within the installed monitoring wells and piezometers. Also, groundwater infiltration levels were recorded within the open holes during the excavation of the test pits. The measured groundwater levels and observed depth of infiltration are presented in Table 1 below:

Table 1 – Summary of Groundwater Levels					
Test Hole Number	**Groundwater Measuring Medium**	**Ground Surface Elevation (m)**	**Measured Groundwater Level / Groundwater Infiltration for Test Pits**		**Dated Recorded**
			Depth (m)	**Elevation (m)**	
BH 1-21	Monitoring Well	68.36	Dry	Dry	July 6, 2021
BH 2-21	Piezometer	68.21	10.56	57.65	July 6, 2021
BH 3-21	Piezometer	68.69	4.13	64.56	July 6, 2021
BH 4-21	Monitoring Well	66.10	4.03	62.57	July 6, 2021
BH 5-21	Monitoring Well	66.18	3.82	62.36	July 6, 2021
BH 5-21	Piezometer	66.18	9.72	56.46	July 6, 2021
TP 1-21	Sidewall Infiltration	65.98	Dry	Dry	June 30, 2021
TP 2-21	Sidewall Infiltration	66.18	3.0	63.18	June 30, 2021
Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.					

It should be noted that 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 approximate depths of 3.5 to 4.5 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

However, 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 considered suitable for the proposed development. It is anticipated the proposed building will have two levels of underground parking. Based on the results of the field program, it is recommended that the proposed building be founded by a raft foundation placed on an undisturbed, stiff, grey silty clay bearing surface.

Due to the presence of the silty clay deposit, a permissible grade raise restriction will be required for the proposed grading throughout the subject site.

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.

It is expected that the proposed underground parking levels will extend to a depth well within the native soils. Therefore, all surface soils will be removed as part of the excavation for the proposed structure. Existing foundation walls and other remnants of construction debris from existing structures should be entirely removed within the building perimeter. Below the proposed buildings foundation, existing construction remnants such as piles and foundations should excavated to a minimum of 300 mm below the excavation depth. Below paved areas, existing construction remnants should excavated to a minimum of 1 m below finished grade.

Fill Placement

Fill used for grading beneath the proposed development 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), unless noted otherwise throughout this report.

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.

Protective Mud Slab

It is recommended that a lean-concrete mud slab be placed on the undisturbed silty clay subgrade surface to protect it from disturbance due to worker traffic. A minimum 50 mm thick lean concrete mud slab (minimum 15 MPa 28-day compressive strength) is recommended to be poured over the undisturbed silty clay surface once exposed.

Excess Soils

All excess soils generated by construction activities that will be transported off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

5.3 Foundation Design

Raft Foundation

For 2 levels of underground parking, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between geodetic elevations of 60.0 to 59.0 m. The bearing medium will consist of a stiff grey silty clay which is susceptible to disturbance under construction traffic. The bearing surface should be protected to prevent disturbance as described above.

The contact pressure provided considers the stress relief associated with the soil removal required for 2 levels of underground parking. The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

For 2 levels of underground parking, a bearing resistance value at SLS (contact pressure) of **200 kPa** will be considered acceptable for a raft supported on the undisturbed, stiff grey silty clay bearing surface. It should be noted that the weight of the raft slab and everything above must be included when designing with this value. The factored bearing resistance (contact pressure) at ULS can be taken as **300 kPa**. For this case, the modulus of subgrade reaction was calculated to be **8.0 MPa/m** for a contact pressure of **200 kPa**.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

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 silty clay bearing medium when a plane extending down and out from the bottom edges of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as that of the bearing medium soil.

Permissible Grade Raise

Based on the existing borehole coverage and results of the undrained shear strength testing completed within the underlying cohesive soils, a permissible grade raise restriction of **2.0 m** is provided for design purposes for the subject site.

5.4 Design for Earthquakes

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

Field Program

The seismic array testing location was placed as presented in Drawing PG5880-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 24 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations were 2, 3 and 20 m away from the last geophone and at the centre of the seismic array.

Data Processing and Interpretation

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

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the foundation of the building. 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 **170 m/s**, while the bedrock shear wave velocity is **1,736 m/s**. Provided that the raft foundation base of the building will be at about 7 to 8 m below the ground surface (approximate geodetic elevation of 60.0 to 59.0 m), there will be approximately 12 m of overburden below the raft slab foundation.

Based on this, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30} = \frac{30\ m}{\left(\frac{12\ m}{170\ m/s} + \frac{18\ m}{1{,}736\ m/s}\right)}$$

$$V_{s30} = 370\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed building bearing on a raft foundation at an approximate geodetic elevation between 60.0 to 59.0 m is **370 m/s**. Therefore, a **Site Class C** is applicable for design of the proposed building as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

For a raft slab foundation, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the popping requirements.

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.

A sub-floor drainage system, consisting of a series of perforated drainage pipes connected to a positive outlet, should be provided in the granular fill layer below the lower basement floor (discussed 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 subject structure. 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 bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Static 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 soil (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 = $(1.45 - a_{max}/g) \cdot a_{max}$
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 0.32 g 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

$P_o = 0.5\, K_o \cdot \gamma \cdot H^2$, 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 Pavement Design

Car only parking areas, heavy truck parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 2 and 3.

Table 2 – Recommended Flexible Pavement Structure – Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**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 raft slab.	

Table 3 – Recommended Flexible Pavement Structure – Access Lanes, Ramp and Heavy Truck Parking and Loading Areas

Thickness (mm)	Material Description
40	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course –** HL-8 or Superpave 19 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 raft slab.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granulars (base and subbase) should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on maintaining the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is expected that insufficient room will be available for exterior backfill along the majority of the foundation walls, therefore, the majority of the foundation wall is anticipated to be blind-side poured against a drainage system placed against the shoring face. The following is recommended to be implemented for the proposed development:

Water Suppression System and Foundation Drainage

Based on the information provided, it is expected that a portion of the proposed building foundation walls will be located below the long-term groundwater table. To mitigate long-term groundwater table lowering, it is recommended that a groundwater infiltration suppression system be provided for the proposed building. Further, a perimeter foundation drainage system will be required to provide an outlet to surface water that may accumulate by heavy rainfall and snow-melt events.

The groundwater infiltration control system should extend at least 1 m above the long-term groundwater level (i.e., a geodetic elevation of 65.5 m) and the following is suggested for preliminary design purposes:

- Place a suitable waterproofing membrane against the temporary shoring surface. This membrane should consist of a dual-layer product consisting of a granular bentonite surface laminated by an HDPE membrane. The granular bentonite surface should face the shoring face. The membrane liner should extend down to founding elevation. The membrane liner should also extend horizontally a minimum of 600 mm below the perimeter of the raft slab.
- Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane, as a secondary system. The composite drainage layer should extend from finished grade to the top of the raft slab.
- Pour the foundation wall against the composite drainage system.

It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the top of the raft slab surface. It is recommended that 150 mm diameter PVC sleeves at 3 m centres be cast in the foundation wall/raft interface to allow the infiltration of water captured by the composite drainage layer to flow to the interior perimeter drainage pipe. The interior perimeter drainage pipe, which further connects to an underfloor drainage pipe system, should direct water to the buildings sump pit(s) within the lowest basement area.

It is important to note that the building's sump pit and elevator pit be also considered for waterproofing in a similar fashion. A detail can be provided by Paterson once the design drawings are available for the elevator and sump pits.

Foundation Raft Slab Construction Joints

It is expected that the raft slab, where utilized, will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab. The raft slab cold joints should also be overlapped in all directions and cast upon a waterproofing membrane across the length of the cold-joints.

Underfloor Drainage

Underfloor drainage is recommended to control water infiltration below the lowest underground parking level slab. For design purposes, the interior perimeter and underfloor drainage pipes should consist of a 150 mm diameter corrugated perforated pipe surrounded by a geosock and a minimum of 150 mm of 19 mm clear crushed stone on all of its sides. The underfloor drainage layout should be detailed by the geotechnical consultant once the structures basement layout has been completed by the structural engineer. The spacing of the underfloor 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, 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 granular material, should otherwise be used for this purpose.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas throughout the remainder of the subject site should be provided with a minimum 450 mm thick layer of OPSS Granular A or OPSS Granular B Type II. The subgrade material should be shaped to promote positive drainage towards the buildings perimeters drainage system.

This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of the materials SPMDD under dry and above-freezing conditions.

Adverse Effects of Dewatering on Adjacent Properties

Due to low permeability of the subsoil profile, any minor dewatering will be considered temporary and limited to the local area of the proposed building during the construction period. Therefore, adverse effects to the surrounding buildings or properties are not expected with respect to short-term temporary groundwater lowering.

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 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard, or a minimum of 0.6 m of soil cover in conjunction with foundation insulation should be provided in this regard.

Other 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 structure proper and require additional protection. These should be provided with a minimum 2.1 m thick soil cover or a combination of soil cover and foundation insulation.

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided to entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided for these areas.

6.3 Excavation Side Slopes and Temporary Shoring

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Given the proximity of the underground parking levels to the property lines, it is expected that a temporary shoring will be required to support the excavation for this proposed development.

Unsupported Excavations

The excavations for the proposed development will be mostly through a stiff silty clay. The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level.

The subsoil at this site is considered to be mainly 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. Excavation side slopes should also be protected from erosion by surface water and rainfall events by the use of tarpaulins or other means of erosion protection along their footprint.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Temporary shoring is anticipated to be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements, designed by a structural engineer specializing in those works, will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services.

The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer.

Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should 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 design prior to implementation.

For design purposes, the temporary shoring system may consist of a soldier pile and lagging system or interlocking steel sheet piling. 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, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure by means of rock bolts or extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is used.

It should be noted, if consideration is being given to utilizing a raker style support for the shoring system, that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 4 – Soils Parameter 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
Unit Weight (γ), kN/m^3	20
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 and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes when placed on a soil subgrade. If the bedding subgrade consists of grey silty clay 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 at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's SPMDD.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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 minimize 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 material's SPMDD.

6.5 Groundwater Control

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. The rate of flow of groundwater into the excavation through the overburden should be low to moderate for the conditions expected at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

Permit to Take Water

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long-Term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Surface water which encounters the building's perimeter infiltration control system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e., less than 20,000 L/day) with peak periods noted after rain events.

Impacts on Neighbouring Properties

Based on our observations, the groundwater level is anticipated at a 3.5 to 4.5 m depth and within the silty clay layer. Therefore, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. Since the proposed development will be founded below the long-term groundwater table, a groundwater infiltration control system has been recommended to lessen the effects of water infiltration. The short-term dewatering during the excavation program will be managed by the excavation contractor, as discussed above.

Further, based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighboring structures provided the implementation of the groundwater system noted in Subsection 6.1 Is carried out to the satisfaction of the geotechnical consultant at the time of construction.

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 by the use of 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 should be carried in a manner to avoid the introduction of frozen materials, snow, or ice into the trenches.

Precautions must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soils. Provisions should be made in the contract documents to protect the walls of the excavations from freezing, if applicable.

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 aggressive corrosive environment.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

- Review of the grading plan from a geotechnical perspective.
- Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction.
- Review of the geotechnical aspects of the integration of the heritage structure façade.
- Review of waterproofing details for raft slab construction, including elevator shafts and sump pits.
- Review and inspection of the groundwater suppression system and details related to the implementation of the 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.
- Observation of all subgrades prior to backfilling and placement of mud slabs.
- 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.

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 Main and Main 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.

LICENSED PROFESSIONAL ENGINEER
July 16, 2021
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng. David J. Gilbert, P.Eng.

Report Distribution:

- ❑ Main and Main (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 50 The Driveway
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5880

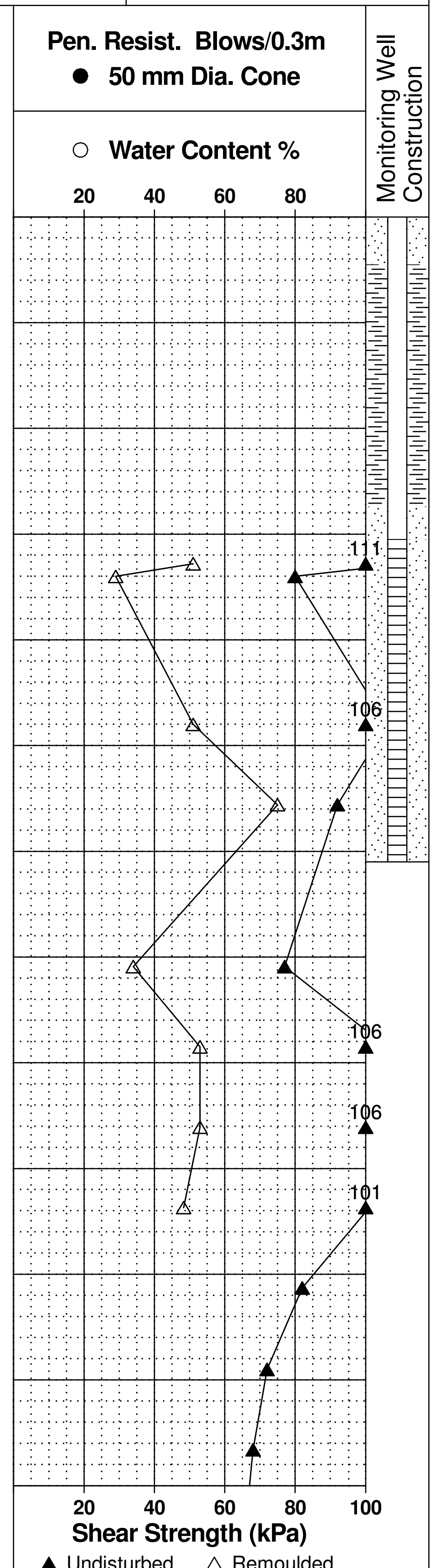
REMARKS

HOLE NO. BH 1-21

BORINGS BY CME-55 Low Clearance Drill

DATE June 30, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.36
Asphaltic concrete 0.08		AU	1				
FILL: Brown silty sand with crushed stone 0.36 – 0.76							
FILL: Brown silty clay, some sand and crushed stone		SS	2	100	15	1	67.36
		SS	3	100	12	2	66.36
		SS	4	100	4		
Very stiff to stiff, brown **SILTY CLAY**		SS	5	100	P	3	65.36
- grey by 3.8m depth						4	64.36
- silt content increasing with depth						5	63.36
		SS	6	100	1	6	62.36
						7	61.36
						8	60.36
						9	59.36
						10	58.36
- trace sand by 11.4m depth						11	57.36
						12	56.36



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 50 The Driveway
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 30, 2021

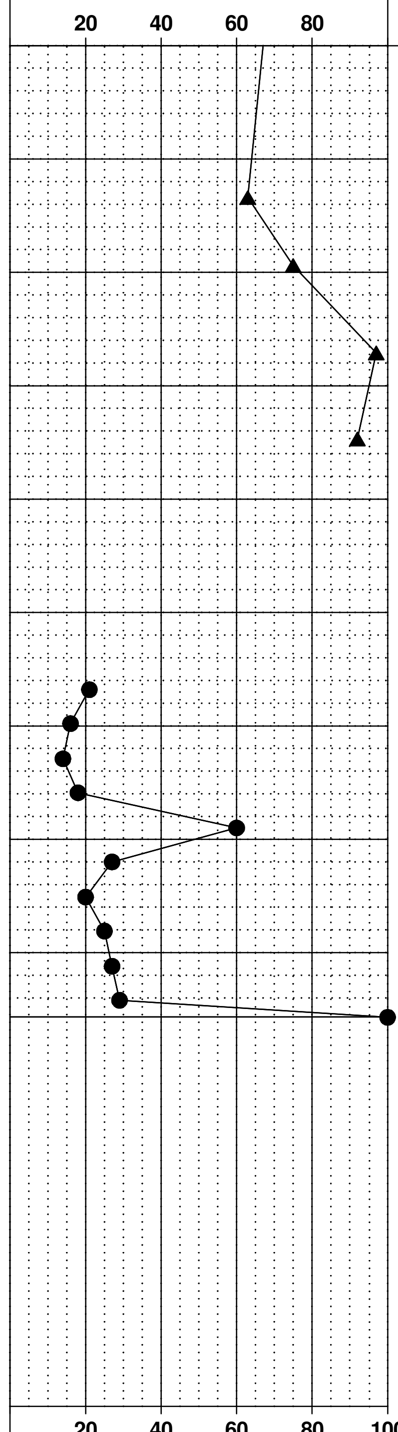
FILE NO. PG5880

HOLE NO. BH 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						12	56.36
		SS	7	100	1		
Stiff, grey **SILTY CLAY**						13	55.36
						14	54.36
15.70						15	53.36
Dynamic Cone Penetration Test commenced at 15.70m depth. Cone pushed to 17m depth.						16	52.36
						17	51.36
						18	50.36
						19	49.36
20.57						20	48.36
End of Borehole Practical DCPT refusal at 20.57m depth. (BH dry - July 6, 2021)							

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 50 The Driveway
Ottawa, Ontario

DATUM Geodetic

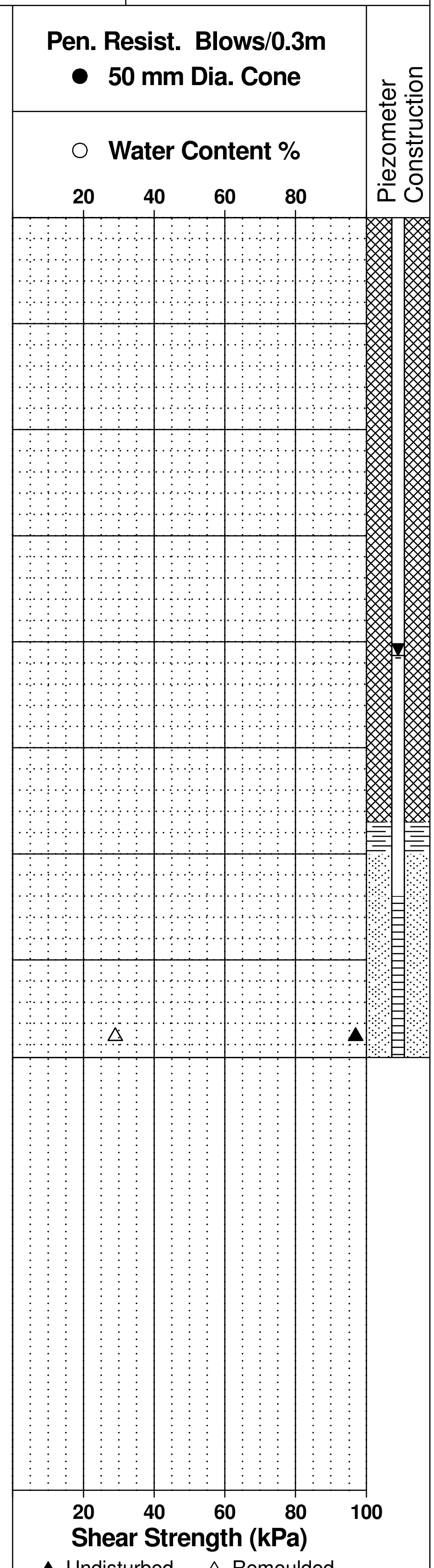
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** June 30, 2021

FILE NO. PG5880

HOLE NO. BH 2-21

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.21
Asphaltic concrete	0.08	AU	1				
FILL: Brown silty sand with crushed stone	0.46						
		SS	2	100	32	1	67.21
FILL: Brown silty sand with gravel, trace clay and topsoil, occasional cobbles		SS	3	75	40	2	66.21
		SS	4	0	50+		
	3.50	SS	5	27	50+	3	65.21
		SS	6	12	3	4	64.21
FILL: Grey silty clay with sand, trace gravel, organics and wood		SS	7	17	4	5	63.21
		SS	8	75	4	6	62.21
	6.78	SS	9	50	8		
Stiff, grey **SILTY CLAY**	7.92	SS	10	100	2	7	61.21
End of Borehole							
(GWL @ 4.13m - July 6, 2021)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 50 The Driveway
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5880

REMARKS

HOLE NO. BH 3-21

BORINGS BY CME-55 Low Clearance Drill **DATE** July 2, 2021

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone	0.46
FILL: Brown silty clay with sand and gravel, trace topsoil - trace coal by 2.3m depth	2.59
Hard to very stiff, brown **SILTY CLAY** - stiff and grey by 5.6m depth - trace sand by 11.4m depth	

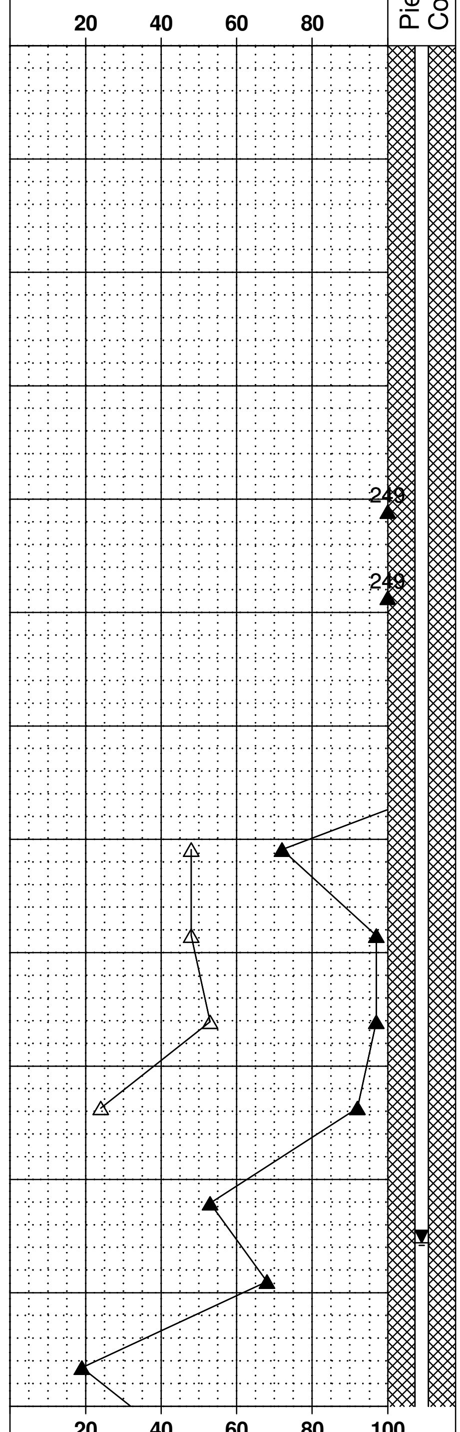
TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	10
SS	3	100	10
SS	4	75	6
SS	5	83	11
SS	6	100	P
SS	7	83	5
SS	8	4	12
SS	9	100	1
SS	10	100	P

DEPTH (m)	ELEV. (m)
0	68.69
1	67.69
2	66.69
3	65.69
4	64.69
5	63.69
6	62.69
7	61.69
8	60.69
9	59.69
10	58.69
11	57.69
12	56.69

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 50 The Driveway
Ottawa, Ontario

DATUM Geodetic

REMARKS

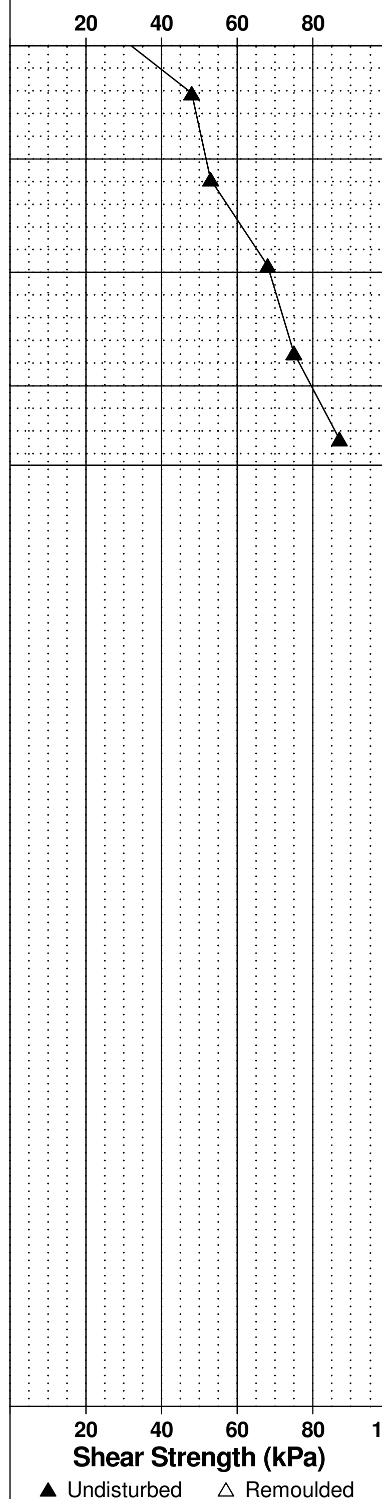
BORINGS BY CME-55 Low Clearance Drill

DATE July 2, 2021

FILE NO. PG5880

HOLE NO. BH 3-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE									
						12	56.69		
						13	55.69		
Stiff, grey **SILTY CLAY** - silt content increasing with depth						14	54.69		
						15	53.69		
15.70									
End of Borehole (GWL @ 10.56m - July 6, 2021)									



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 50 The Driveway
Ottawa, Ontario

DATUM Geodetic

REMARKS

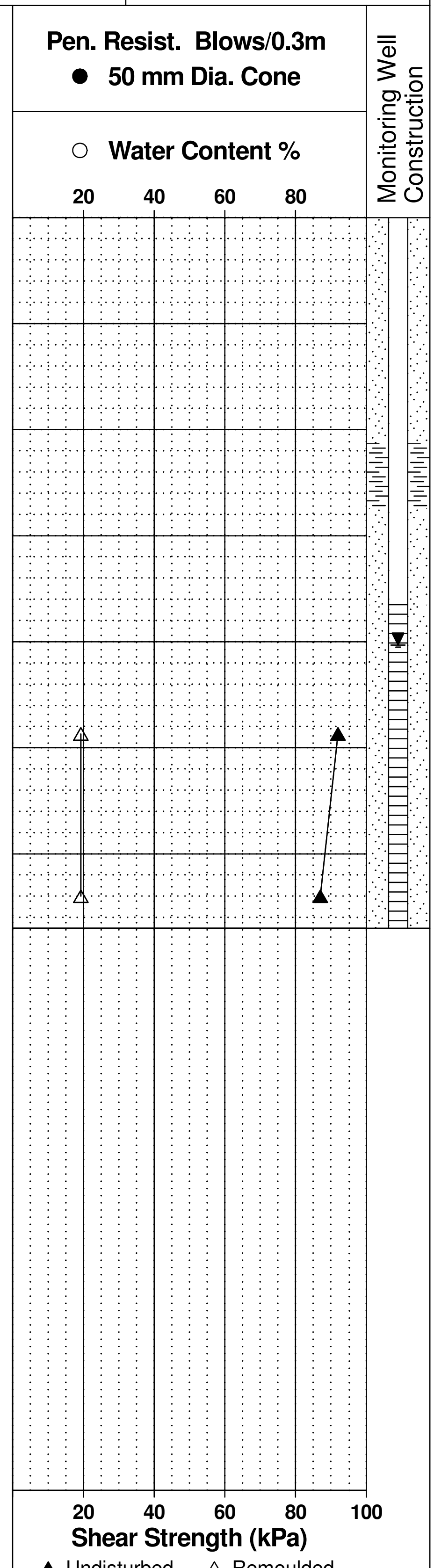
BORINGS BY CME-55 Low Clearance Drill

DATE July 2, 2021

FILE NO. PG5880

HOLE NO. BH 4-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.60
TOPSOIL 0.30		AU	1				
FILL: Brown silty clay with sand and gravel, some cobbles, concrete, trace wood		SS	2	100	50+	1	65.60
		SS	3	4	4	2	64.60
		SS	4	50	11		
3.96		SS	5	67	4	3	63.60
Stiff, brown **SILTY CLAY**		SS	6	67	4	4	62.60
		SS	7	100	P	5	61.60
- grey by 5.3m depth		SS	8	100	1	6	60.60
6.70		SS	9	100	P		
End of Borehole							
(GWL @ 4.03m - July 6, 2021)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 50 The Driveway
Ottawa, Ontario

DATUM Geodetic

REMARKS

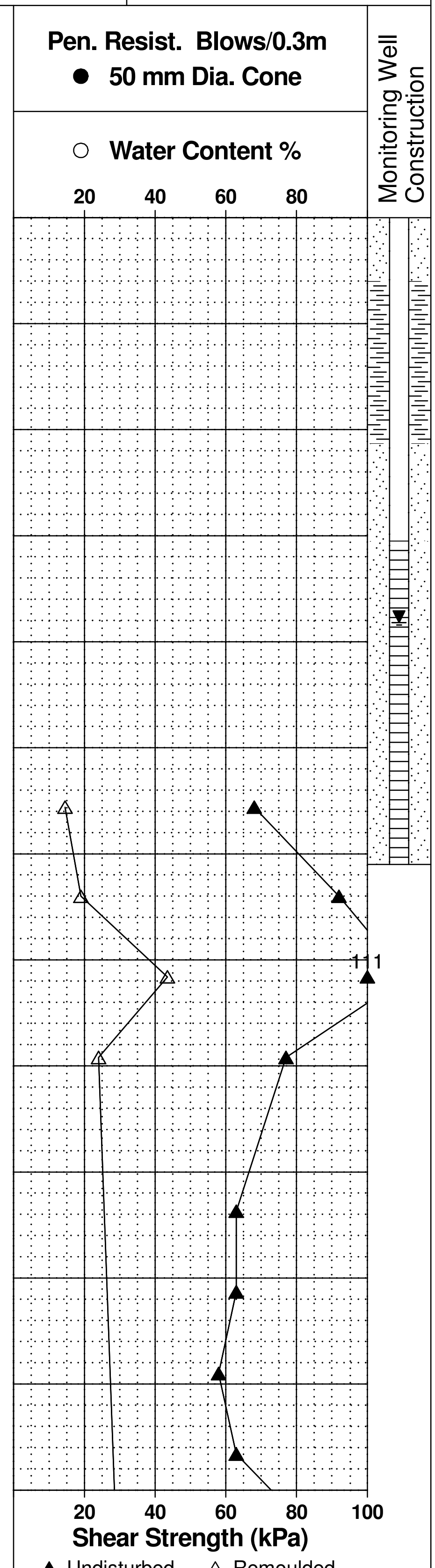
BORINGS BY CME-55 Low Clearance Drill

DATE July 5, 2021

FILE NO. PG5880

HOLE NO. BH 5-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.18
TOPSOIL	0.18						
FILL: Brown silty sand with clay	0.90	AU	1				
FILL: Brown silty clay with sand and gravel, trace topsoil	1.52	SS	2	17	12	1	65.18
FILL: Brown silty sand, trace gravel	2.29	SS	3	42	10	2	64.18
FILL: Grey silty clay with sand, trace organics, brick and gravel	4.50	SS	4	75	9	3	63.18
		SS	5	75	4	4	62.18
		SS	6	17	9		
Very stiff to stiff, grey **SILTY CLAY**		SS	7	83	3	5	61.18
- silt content increasing with depth		SS	8	83	P	6	60.18
		SS	9	83	P	7	59.18
		SS	10	83	P	8	58.18
		SS	11	83	1	9	57.18
						10	56.18
						11	55.18
						12	54.18



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 50 The Driveway
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5880**

REMARKS

HOLE NO. **BH 5-21**

BORINGS BY CME-55 Low Clearance Drill

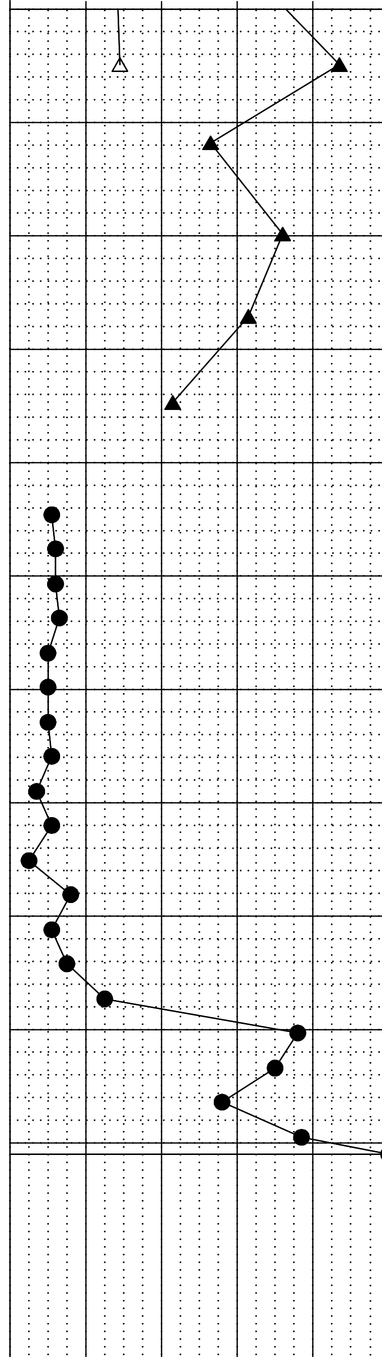
DATE July 5, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						12	54.18
		SS	12	83	P		
Stiff, grey **SILTY CLAY**						13	53.18
						14	52.18
						15	51.18
15.70							
Dynamic Cone Penetration Test commenced at 15.70m depth.						16	50.18
						17	49.18
						18	48.18
						19	47.18
						20	46.18
						21	45.18
						22	44.18
22.10							
End of Borehole Practical DCPT refusal at 22.10m depth. (GWL @ 3.82m - July 6, 2021)							

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

○ Water Content %

20 40 60 80



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

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 50 The Driveway
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe/Hand Auger

DATE June 30, 2021

FILE NO. PG5880

HOLE NO. TP 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	65.98
Decorative river rock (0.10)							
FILL: Brown silty sand with crushed stone, trace clay (0.30)		G	1				
		G	2				
						1	64.98
FILL: Brown silty sand with clay, gravel, crushed stone, trace concrete and brick		G	3				
		G	4				
						2	63.98
		G	5				
(2.95)		G	6				
FILL: Crushed stone (3.05)						3	62.98
- underside of footing at 2.95m depth		G	7				
Stiff, grey **SILTY CLAY**						4	61.98
(4.70)							
End of Test Pit							

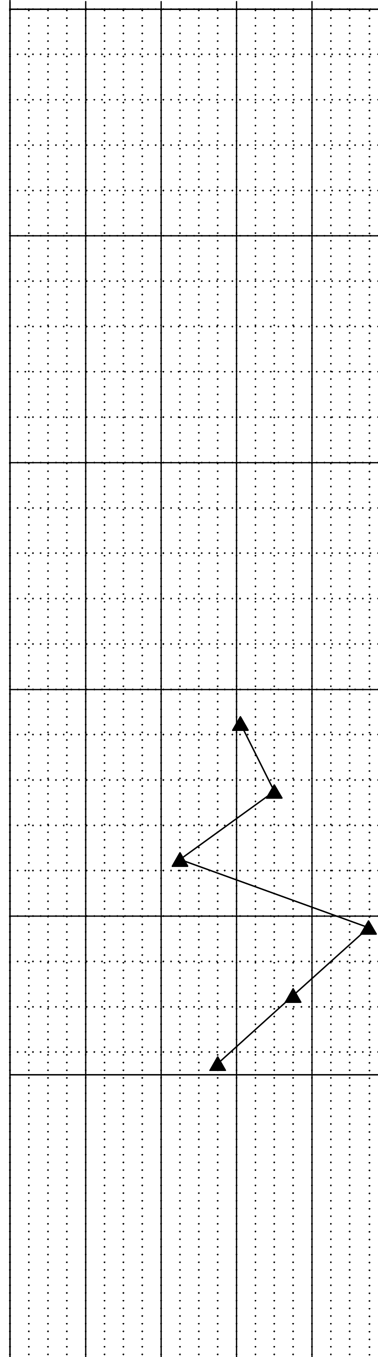
310mm deep footing encountered at 2.64m below existing ground surface, extending 230mm from foundation wall.

(TP dry upon completion)

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

○ Water Content %

20 40 60 80



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

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 50 The Driveway
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5880

REMARKS

HOLE NO. TP 2-21

BORINGS BY Backhoe/Hand Auger

DATE June 30, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	66.18		
Decorative river rock — 0.10									
FILL: Brown silty sand with gravel and crushed stone — 0.30		G	1						
FILL: Brown silty sand with clay, gravel and crushed stone, trace brick, concrete and organics		G	2						
		G	3			1	65.18		
		G	4						
		G	5			2	64.18		
— 3.05						3	63.18		∇
FILL: Brown silty sand to sandy silt		G	6						
— 3.98									

End of Test Pit

Inferred top of footing encountered at 3.98m below existing ground surface.

(Groundwater infiltration at 3.0m depth)

Shear Strength (kPa): 20 40 60 80 100

▲ 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 = $(D30)^2 / (D10 \times D60)$
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

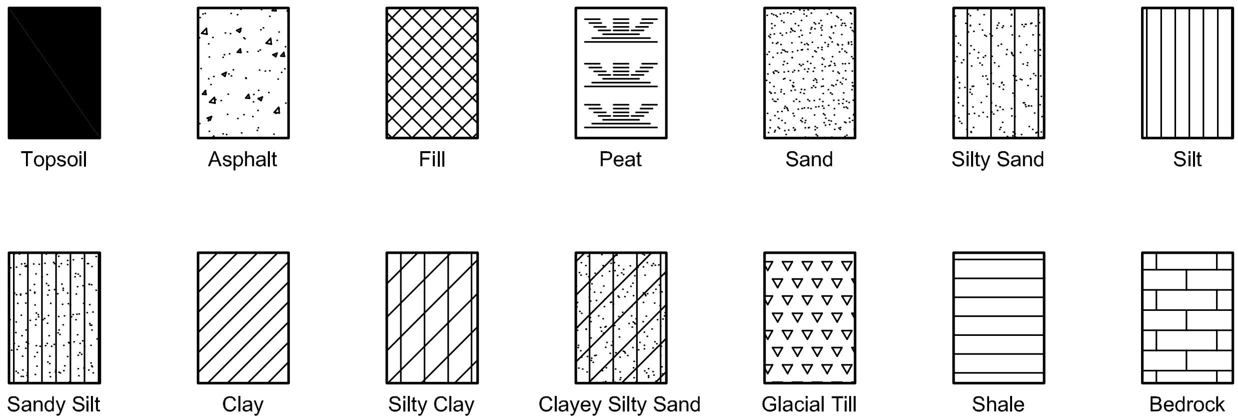
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

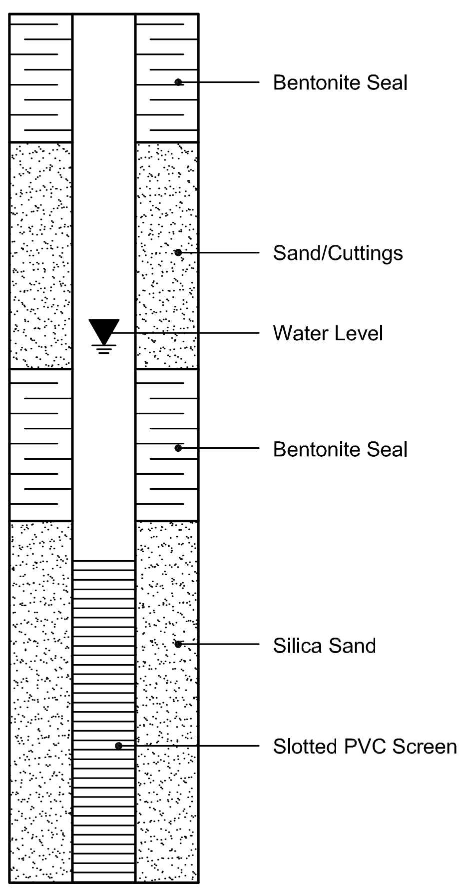
SYMBOLS AND TERMS (continued)

STRATA PLOT

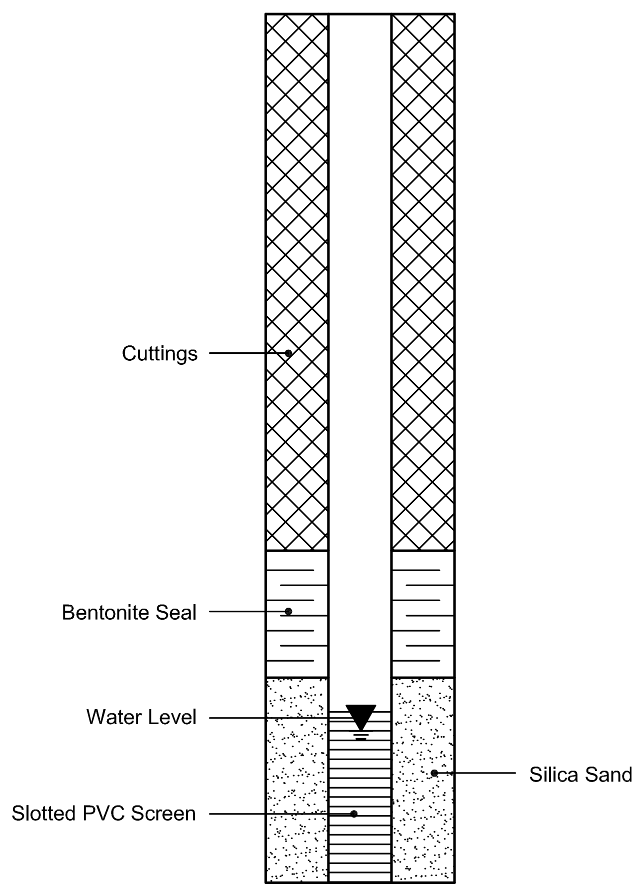


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 2128120

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 32281

Report Date: 08-Jul-2021

Order Date: 5-Jul-2021

Project Description: PG5880

	MDL/Units				
Client ID:		BH3-21 SS9	-	-	-
Sample Date:		02-Jul-21 09:00	-	-	-
Sample ID:		2128120-01	-	-	-
		Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	64.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.67	-	-	-
Resistivity	0.10 Ohm.m	32.1	-	-	-
Anions					
Chloride	5 ug/g dry	58	-	-	-
Sulphate	5 ug/g dry	79	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5880-1 – TEST HOLE LOCATION PLAN

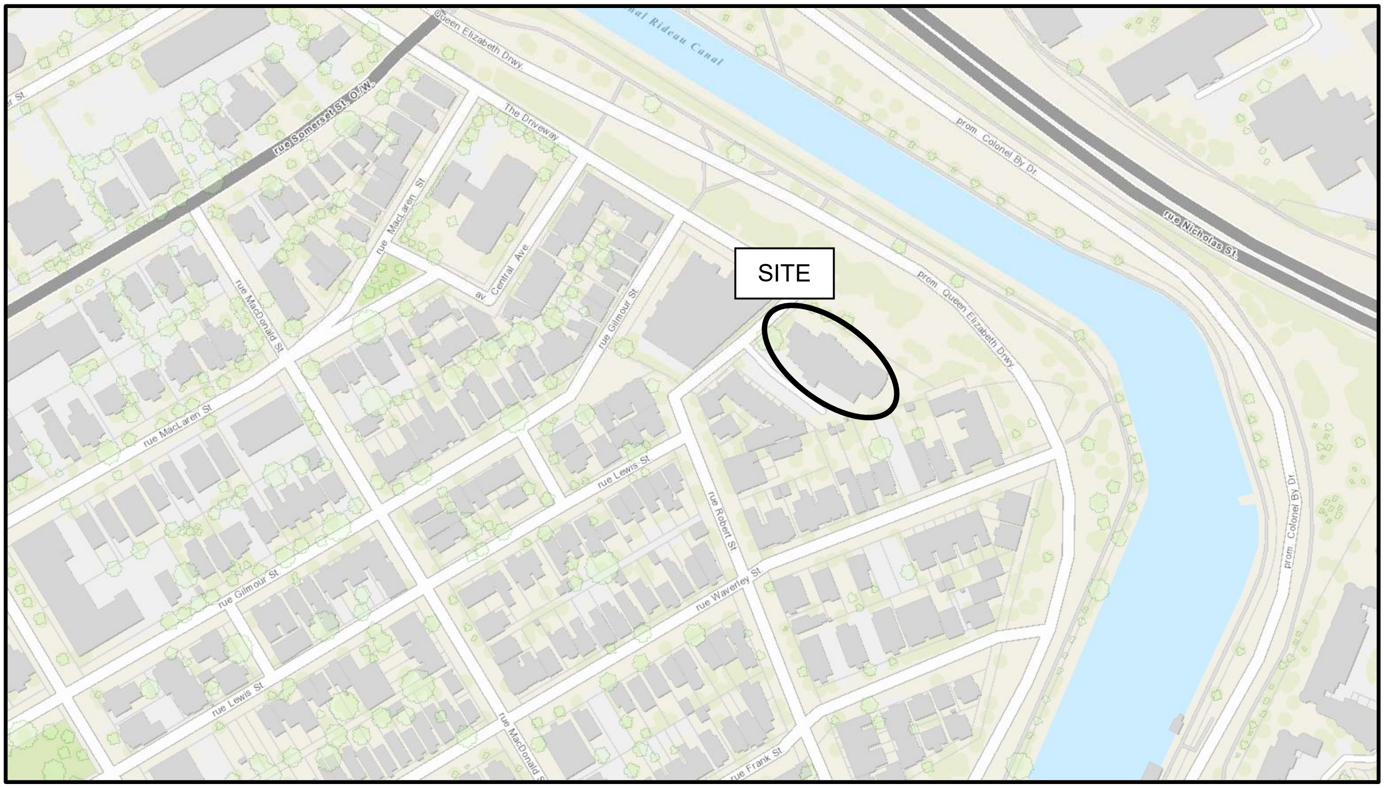


FIGURE 1

KEY PLAN

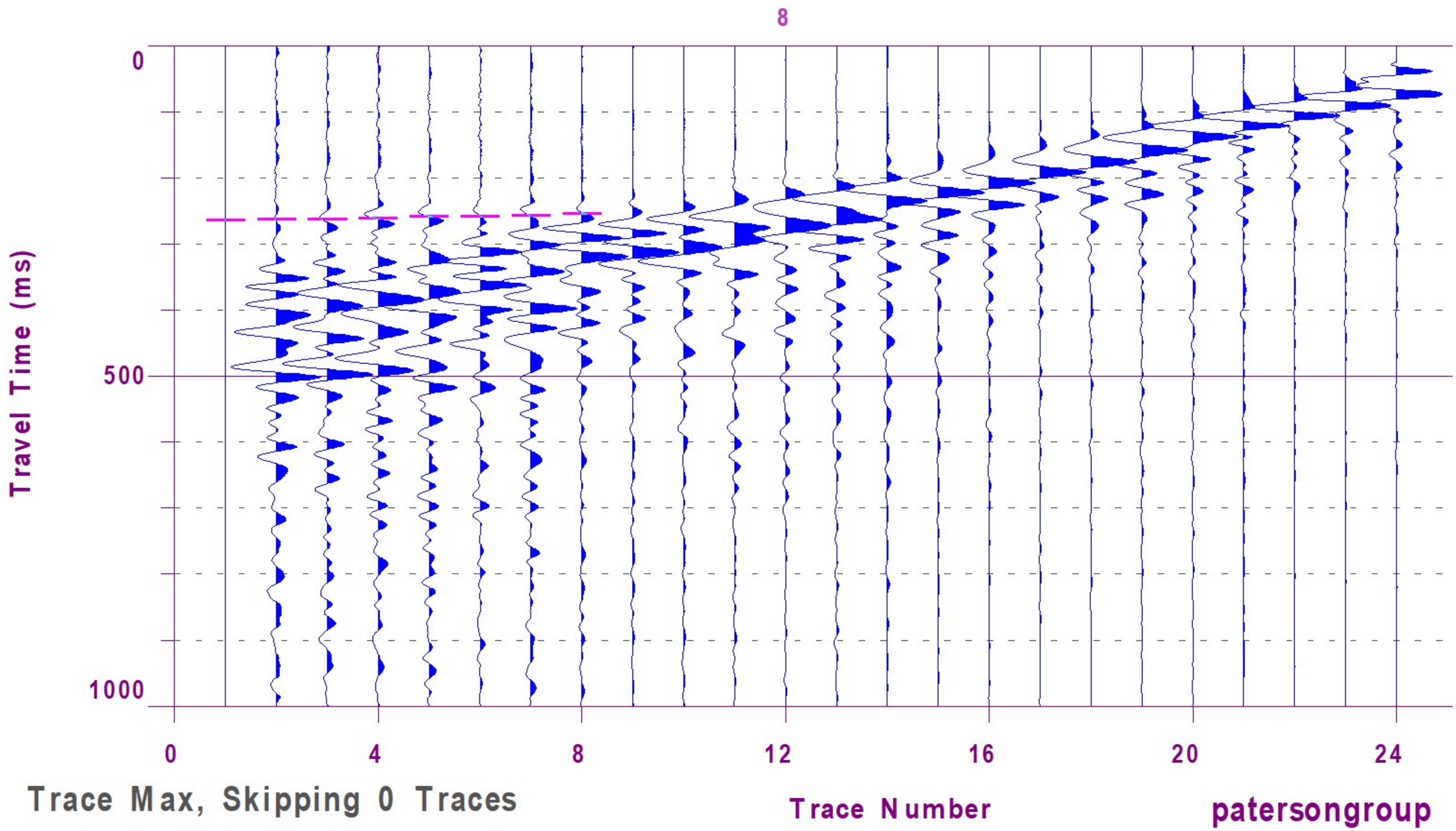


Figure 2 – Shear Wave Velocity Profile at Shot Location 49 m

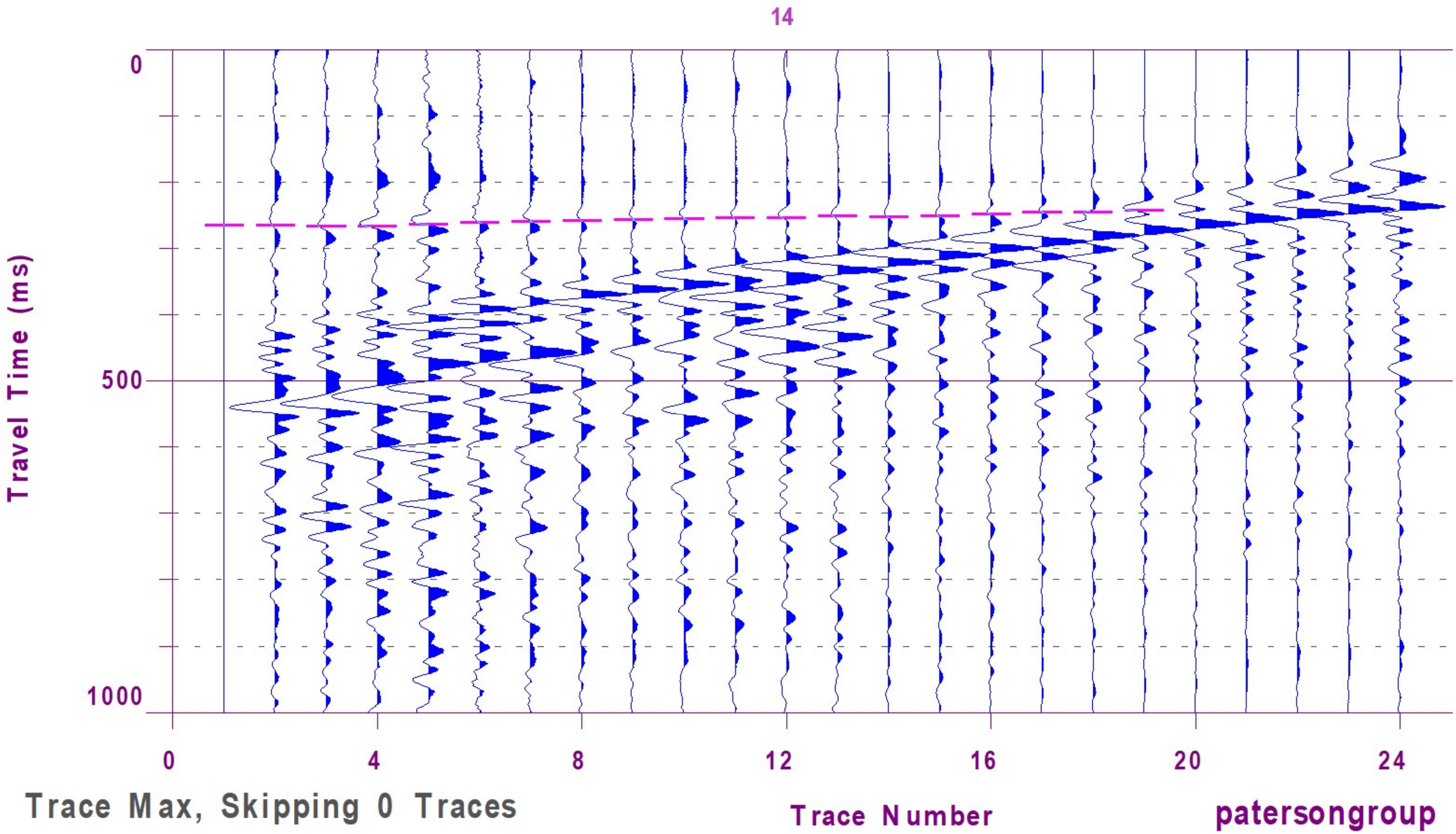
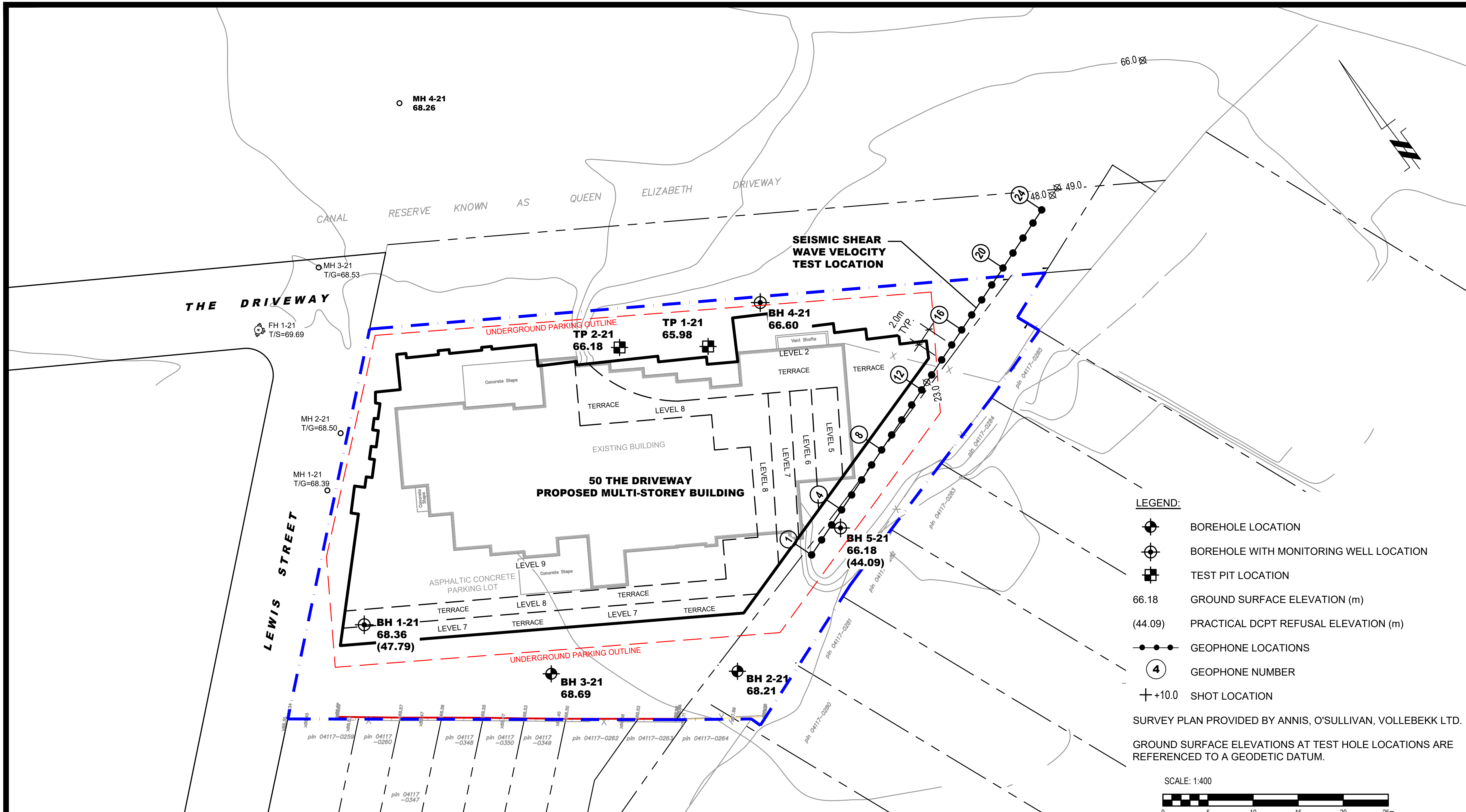


Figure 3 – Shear Wave Velocity Profile at Shot Location 66 m



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NO.	REVISIONS	DATE	INITIAL

MAIN AND MAIN
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
50 THE DRIVEWAY

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:400	Date:	07/2021
Drawn by:	YA	Report No.:	PG5880-1
Checked by:	FA	Dwg. No.:	PG5880-1
Approved by:	DJG	Revision No.:	

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