PATERSON GROUP

Geotechnical Investigation

Proposed Multi-Storey Building

178-200 Isabella Street and
205 Pretoria Avenue
Ottawa, Ontario

Minto Communities

Report PG5043-1 Revision 2 dated August 3, 2023

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Minto Communities to complete a geotechnical investigation for the proposed multi-storey building, which is to be located at 178-200 Isabella Street and 205 Pretoria Avenue in the City of Ottawa, Ontario (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 test holes.

- Provide geotechnical recommendations based on existing soils information for the design of the proposed buildings, 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. This report contains the geotechnical findings and includes recommendations pertaining to the design and construction of the proposed buildings as understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject site was not part of the scope of work of this present investigation. A report addressing environmental issues has been prepared under a separate cover.

2.0 Proposed Development

It is understood that the proposed development consists of a 19-storey residential building. The number of underground levels for the proposed building is not determined at the time of preparing this report. However, it is expected that the proposed tower will be constructed over 3 or 4 levels of underground parking. The excavation footprint for the underground parking structure is anticipated to encompass the majority of the site. The site is also anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the previous geotechnical investigation was carried out by Paterson on October 7, 2016. At that time, three (3) boreholes were advanced to a maximum depth of 9.75 m below existing grade. A supplemental investigation was completed on October 6 and 7, 2020, which consisted of 2 boreholes extended to a maximum 19 m depth. The borehole locations were distributed in a manner to provide general coverage of the site. The approximate locations of the boreholes are shown on Drawing PG5043-2 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, 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) at two borehole locations. 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 presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

A 51 mm diameter PVC groundwater monitoring well was installed in all borehole locations to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- 3.0 m of slotted 51 mm diameter PVC screen at base 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 directed otherwise.

3.2 Field Survey

The borehole locations and ground surface elevations were surveyed by Paterson field personnel. Ground surface elevations at the borehole locations were referenced to a geodetic datum. The location of the boreholes and the ground surface elevation at the boreholes are presented on Drawing PG5043-2 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were visually examined in our laboratory to review the results of the field logging.

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 is analyzed to determine the concentrations of sulphate and chloride, the resistivity and the pH of the sample. The results are included in Appendix 1 and are further discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site consists of five (5) individual properties (178, 180, 182 and 200 Isabella Street and 205 Pretoria Avenue). The properties from 178 to 200 Isabella Street were formerly occupied by several low-rise buildings. The east and west portions of this section of the site are currently paved and used for parking, while the middle portion of this section is covered with low vegetation and visible demolition debris throughout. The property at 205 Pretoria Avenue is currently occupied by an existing residential building.

The subject site has a moderate slope down towards Isabella Street and is approximately at grade with Isabella Street. It should be noted that a retaining wall is present along the west property boundary of the Isabella properties, which provides an elevation difference of a minimum of 1.5 m above the adjacent property, including 205 Pretoria Avenue.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of asphalt and/or fill consisting of brown silty sand to silty clay, some gravel, cobbles, boulders and construction debris extending to depths ranging from 1.4 to 2.3 m below the existing grade. A stiff brown to grey silty clay deposit was encountered below the above noted fill layers. A compact glacial till layer was encountered below the silty clay deposit at BH 1-20 and BH 2-20 at 13.7 and 13.4 m depth, respectively. Practical refusal to DCPT was encountered at 17.9 and 19.2 m in BH 1 and BH 2, respectively. Weathered bedrock was encountered at BH 1-20 and BH 2-20 at 18.3 and 15.2 m depth. Specific details of the subsurface profile at each borehole location are presented in the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of shale of the Billings Formation with an approximate drift thickness of 15 to 25 m.

4.3 Groundwater

Groundwater levels were recorded at each test hole location and presented in Table 1 below. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1.

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-20	67.37	4.80	62.57	October 9, 2020
BH 2-20	68.14	5.40	62.74	
BH 1	67.45	2.52	64.93	October 14, 2016
BH 2	67.90	4.74	63.16	
BH 3	67.95	6.79	61.16	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.

The Long-term groundwater levels can also be estimated based on the observed colour, consistency, and moisture content of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately **4.5 to 5.5 m** below ground surface. Groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

Groundwater monitoring wells were installed at BH 1-20 and BH 2-20. A falling head slug test was completed at each monitoring well to confirm the hydraulic conductivity of the soils. The results of our testing are presented in the data sheets in Appendix 1.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered acceptable for the proposed multi-storey building. Bearing resistance values have been provided for conventional shallow foundations, however, based on the proposed development, it is anticipated that these bearing resistance values will not be sufficient to support the expected building loads. Therefore, alternative options such as end bearing piled foundations, or a raft foundation would provide a suitable foundation for the proposed building.

Due to the presence of a silty clay deposit underlying the subject site, it is recommended that the site be subjected to permissible grade raise restrictions to prevent issues related to differential settlement in the underlying soils.

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

5.2 Site Grading and Preparation

Stripping Depth

Asphalt, topsoil, construction debris and any deleterious fill, such as those containing organic materials, should be removed from within the perimeter of the proposed buildings and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the proposed buildings, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 225 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified fill and/or Site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Conventional Shallow Footings

Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, placed over an undisturbed, stiff silty clay bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

Footings placed over an undisturbed, compact glacial till bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **350 kPa**.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings. The bearing resistance value given for footings at SLS will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Above the groundwater level, adequate lateral support is provided to a stiff silty clay when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil or engineered fill.

Piled Foundation

Consideration may be given to using concrete filled steel pipe piles driven to refusal on the bedrock surface where building loads exceed the bearing resistance values given above. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two to four piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data					
Pile Outside Diameter (mm)	**Pile Wall Thickness (mm)**	**Geotechnical Axial Resistance**		**Final Set (blows/ 12 mm)**	**Transferred Hammer Energy (kJ)**
		SLS (kN)	**Factored at ULS (kN)**		
245	9	925	1100	6	27
245	11	1050	1260	6	31
245	13	1200	1440	6	35

Raft Foundation

For our design calculations, a 19-storey building with three or four levels of underground parking were assumed.

Three Underground Parking Levels

For three underground parking levels, it is expected that the excavation will extend to a 11 m depth below existing ground surface and will be founded upon a very stiff to stiff grey silty clay. The maximum SLS contact pressure (includes the raft embedment compensation) can be taken to be **250 kPa**. It should be noted that the weight of the raft slab and everything above has to be included when designing with this value.

The modulus of subgrade reaction was calculated to be **10 MPa/m** for a contact pressure of **250 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. The proposed building can be designed using the above parameters and a total and differential settlement of 25 and 20 mm, respectively.

Four Underground Parking

For four underground parking levels, it is expected that the excavation will extend to a 14 m depth below existing ground surface and will be founded upon a compact to dense glacial till. The maximum SLS contact pressure (includes the raft embedment compensation) can be taken to be **360 kPa**. It should be noted that the weight of the raft slab and everything above has to be included when designing with this value.

The modulus of subgrade reaction was calculated to be **14.5 MPa/m** for a contact pressure of **360 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. The proposed building can be designed using the above parameters and a total and differential settlement of 25 and 20 mm, respectively.

Permissible Grade Raise Recommendations

From a geotechnical perspective, the subject site should be subjected to a permissible grade raise of **2 m** above existing ground surface.

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 from 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 attached to the present letter report.

Field Program

The seismic array testing location was placed as presented in Drawing PG5043-2 - Test Hole Location Plan, attached to the present letter report. Paterson field personnel placed 18 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 3 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 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 located at 20, 4.5 and 3 m away from the first geophone and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for 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 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 **225 m/s**, while the bedrock shear wave velocity is **2,045 m/s**. Further, the testing results indicate the average overburden thickness to be approximately 20 m. Provided the raft foundation base or pile cap for the proposed building will be at about 11 m below ground surface, the overburden thickness is conservatively assumed to be 9 m.

Based on this, the V_{s30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012, as presented below.

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

$$V_{s30} = \frac{30\ m}{\left(\frac{21\ m}{2{,}045\ m/s} + \frac{9\ m}{225 m/s}\right)}$$

$$V_{s30} = 597\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for foundations at the aforementioned site is **597 m/s**. Therefore, a Site **Class C** is applicable for the design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012 for raft foundation and or pile caps placed at a minimum depth of 11 m below existing grade.

5.5 Basement Slab

It is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD. It is also expected that a series of sub-floor drainage pipes connected to the building's sump pit will be incorporated to drain any water which enters the granular layer. A concrete mud slab should be poured to protect the native soil from construction activities.

5.6 Basement Wall

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 dry unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective 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.

Two distinct conditions, static and seismic, should 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 (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 = $(1.45 - a_{max}/g)a_{max}$
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 the OBC 2012. Note that the vertical seismic coefficient is assumed to be zero. The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Design

Asphalt pavement is not anticipated to be required at the subject site. However, should a flexible pavement be required for the project, the recommended flexible pavement structures shown in Tables 3 and 4 would be applicable.

Table 3 - 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 in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

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

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

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

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage

Foundation Drainage and Waterproofing

The following recommendations may be considered for the architectural design of the buildings foundation drainage systems. It is recommended that Paterson be engaged at the design stage of the future building (and prior to tender) to review and provide supplemental information for the building foundation drainage system design.

Supplemental details, review of architectural design drawings and additional information may be provided by Paterson for these items for incorporation in the building design packages and associated tender documents. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed structure. It is expected that insufficient room will be available for exterior backfill and the foundation wall will be cast as a blind-sided pour against a shoring system and the bedrock surface for the majority of the site. It is recommended that the groundwater suppression system consist of the following:

- For a blind-sided pour, a waterproofing membrane should be placed against the shoring system between underside of footing level and extending to 4 m below the existing ground surface. Where the membrane will extend against the shoring system, it is recommended to consist of a membrane with a bentonite-lined face for being paced against the shoring system. The membrane is recommended to overlap below the overlying perimeter foundation footprint by a minimum of 1 m inwards towards the building footprint and from the face of the overlying foundation. This will allow construction to proceed without imposing groundwater lowering within the surrounding area of the proposed buildings in the short and long term conditions.

- Where the foundation walls are constructed using a conventional double-sided pour and the drainage board is fastened directly to the foundation walls, the vertical overlaps should be completed such the top of the lower sheet of drainage board overlies the bottom of the upper sheet.

- A composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the HDPE face of the waterproofing membrane with the geotextile layer facing the waterproofing layer from finished ground surface to the top of the footing.

- The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant. It is highly recommended that the drainage board rolls be installed horizontally rather than vertically to minimize the number of vertical joints forming between the rolls.

- It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe system.

The top endlap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall.

Interior/Exterior Perimeter and Underfloor Drainage

An interior perimeter and underfloor drainage system will be required to redirect water from the building's foundation drainage system to the building's sump pit(s) if it will not discharge to an exterior catch basin structure. For preliminary design purposes, it is recommended that the interior perimeter and underfloor drainage pipes should consist of 100 or 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock, placed at approximately 6 m.

For locations where the foundation wall will be constructed with a conventional double sided pour, the interior perimeter drainage pipe must transition to an exterior perimeter drainage pipe by way of a sleeve placed through the foundation wall.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves.

The spacing of the underfloor drainage should be confirmed by Paterson at the time of excavation when water infiltration can be better assessed and once the foundation layout and sump system location has been finalized.

Transition from Blind Side to Double Sided Formwork

Based on our review, the construction of the exterior perimeter foundation walls above the temporary shoring system is anticipated to transition from a blind-sided pour into a conventional double sided pour at the north side of the excavation. The integrity of the drainage system should be maintained across this transition zone as per the following methodology:

- It is critical that the composite foundation drainage board is extended in a suitable manner through this transition zone to maintain the long-term performance of the foundation drainage system. It is recommended that the composite foundation drainage board be extended a minimum of 0.6 m beyond the limits of the temporary shoring system where a transition from blind side to double sided formwork is anticipated. As a result, the contractor should be prepared to provide supplemental temporary formwork to ensure this transition is completed in a suitable manner.

- The purpose of additional custom formwork is to provide a suitable surface on which to maintain the vertical continuity and application of the foundation drainage system across the transition zone from blind-side to double-sided pours. Based on our experience, the additional formwork will be required in areas where the bedrock/overburden interface has an unsuitable quantity of voids, jagged surfaces and/or fractures from blasting and bedrock removal procedures. This custom formwork typically consists of suitably prepared plywood, rigid insulation, or other concrete formwork materials as procured by the formwork contractor which is cut and sized to match the contours of the bedrock/overburden interface. This additional effort and material by the formwork contractor will mitigate the risks associated with over-pouring the foundation beyond the overburden/bedrock interface and with pouring concrete onto the system bridging gaps and voids. Lastly, carrying out this measure will ensure that the foundation drainage system is installed in a relatively flat and vertical fashion.

- It is **NOT** recommended to fold the composite foundation drainage board to accommodate temporary formwork and the placement of concrete at the transition zone. It is therefore recommended that the contractor modifies temporary forms to match the contour of the surface which the formwork will be placed upon (i.e., temporary shoring system) and secure the drainage board against the inside of the temporary forms.

- Since the bulk of the custom formwork will most likely remain in place, the foundation drainage board must extend a minimum of 1 m above the horizontal and vertical extent of the custom formwork and onto the inside of the prefabricated formwork. Once the foundation wall has been poured and the modified temporary formwork has been removed, the exposed portion of the composite foundation drainage board can be overlapped in a shingle fashion in accordance with the manufacturer's specifications and in general conformance with our geotechnical recommendations.

Foundation Backfill

Where applicable, 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 Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

Foundation backfill material should be compacted in maximum 300 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Podium Deck Waterproofing Tie-In

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Foundation Raft Slab Construction Joints

If applicable, it is anticipated the raft slab will be poured in several pour segments. For the construction joint at each pour, a PVC water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a PVC water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized structural and architectural drawings for the proposed building which includes the above noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double sided pours (if applicable), etc.

6.2 Protection of Footings Against Frost Action

Perimeter footings and pile caps of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided in this regard. 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, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp, may be required to insulate 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. Paterson should conduct a review of the ramp, footings located at the garage entrance and any footings not meeting the minimum frost cover requirements prior to construction to provide site specific frost protection/insulation recommendations.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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 subsurface soil 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 maintain safe working distance 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.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements 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.

The temporary system could consist of 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 included to the earth pressures described below. These systems could 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. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. 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 shoring system may be calculated with the following parameters.

Table 5 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
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 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/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications 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 bedding for sewer and water pipes when placed on soil subgrade. 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 (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% 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 reduce the potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Infiltration

It is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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.

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

For typical ground or surface water volumes, being pumped during the construction phase, 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. Any groundwater breaching the waterproofing system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low to moderate at the bottom of excavation with higher volumes during peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Adverse Effects of Dewatering on Adjacent Properties

Since the proposed development will be founded below the long term groundwater level, a waterproofing membrane system has been recommended to lessen the effects of water infiltration. Any long term dewatering of the site will be minimal and should have no adverse effect to the surrounding buildings or structures. The short term dewatering during the excavation program will be managed by the excavation contractor.

6.6 Winter Construction

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

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters 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.

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

Precautions must be taken where excavations are carried out 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 soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

One (1) sample was submitted for testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.01%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the resistivity indicate the presence of a moderate environment for exposed ferrous metals at this site, which is typical of silty clay samples submitted for the subject area. It is anticipated that standard measures for corrosion protection are sufficient for services placed within the silty clay deposit.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ❑ Review preliminary and detailed grading, servicing, landscaping and structural plan(s) from a geotechnical perspective.
- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if not design by Paterson, prior to construction, if applicable.
- ❑ Review of architectural plans pertaining to groundwater suppression system, underfloor drainage systems and waterproofing details for elevator shafts.

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

- ❑ Review and inspection of the installation of the foundation drainage systems.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Observation of driving and re-striking of all pile foundations.
- ❑ 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 follow-up field density tests to determine the level of compaction achieved.
- ❑ 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 soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

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

A geotechnical investigation is a limited sampling of a site. Should any conditions encountered during construction differ from the borehole locations, Paterson requests immediate notification to permit reassessment of the recommendations.

The recommendations provided should only be used by the design professionals associated with this project. The recommendations are not intended for contractors bidding on or constructing the project. The latter should evaluate the factual information provided in the report. The contractor should also determine the suitability and completeness for the intended construction schedule and methods. Additional testing may be required for the contractors purpose.

The present report applies only to the project described in the report. The use of the report for purposes other than those described above or by person(s) other than Minto Communities or their agents is not authorized without review by Paterson.

Paterson Group Inc.

Nicole R. L. Patey, B.Eng.

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❑ Minto Communities (Email Copy)
- ❑ Paterson Group (1 Copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY TEST RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Multi-Storey Building - 178-200 Isabella Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE October 6, 2020

FILE NO. PG5043

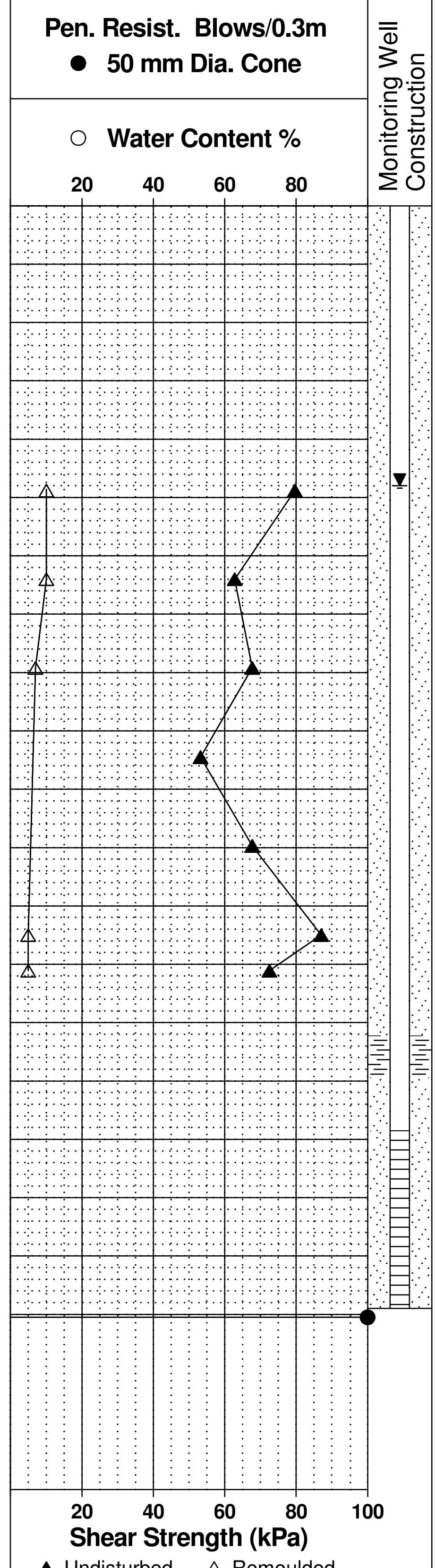
HOLE NO. BH 1-20

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.25	AU	1		
Stiff, brown **SILTY CLAY,** trace sand	3.05	SS	2	100	7
Stiff, grey **SILTY CLAY**	13.72	SS	3	100	2
GLACIAL TILL: Compact, grey silty sand, trace gravel, cobbles and boulders	18.30	SS	4	67	20
		SS	5	17	7
		SS	6	50	10
		SS	7	33	11
		SS	8	42	19
		SS	9	0	13
Weathered **BEDROCK**	19.05	SS	10	67	51
End of Borehole					

Practical DCPT refusal at 19.05m depth

(GWL @ 4.80m - Oct. 9, 2020)

DEPTH (m)	ELEV. (m)
0	67.37
1	66.37
2	65.37
3	64.37
4	63.37
5	62.37
6	61.37
7	60.37
8	59.37
9	58.37
10	57.37
11	56.37
12	55.37
13	54.37
14	53.37
15	52.37
16	51.37
17	50.37
18	49.37
19	48.37



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Multi-Storey Building - 178-200 Isabella Street
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5043

REMARKS

HOLE NO. BH 2-20

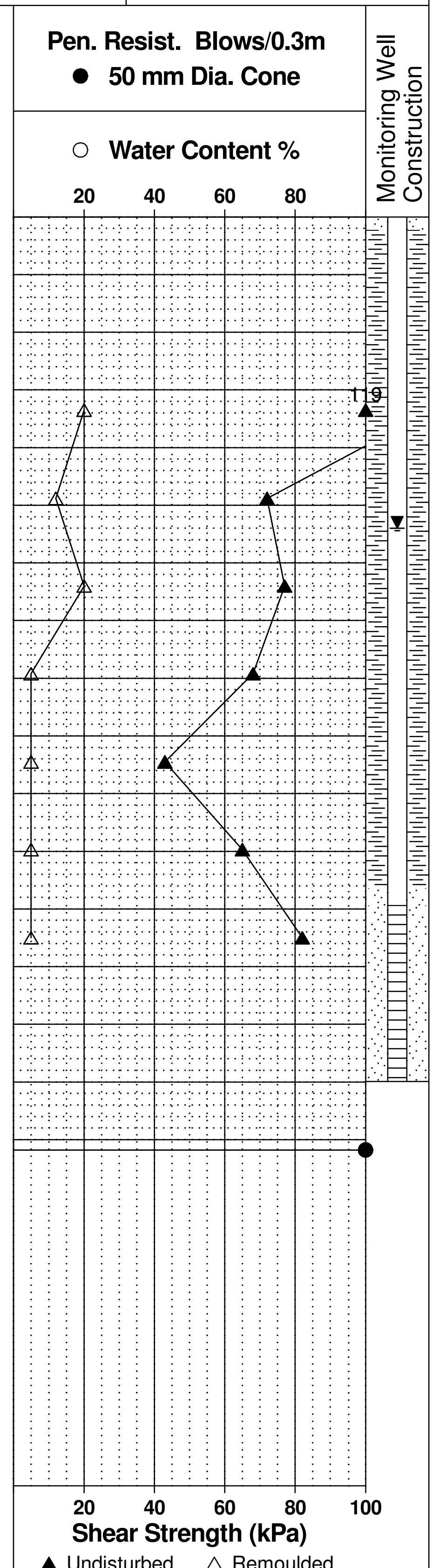
BORINGS BY Track-Mount Power Auger **DATE** October 7, 2020

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	68.14
Concrete pavement 0.07	AU	1				
FILL: Brown silty sand with gravel 0.60					1	67.14
Stiff, brown **SILTY CLAY**	SS	2	100	5	2	66.14
3.05					3	65.14
					4	64.14
					5	63.14
					6	62.14
					7	61.14
Stiff to firm, grey **SILTY CLAY**					8	60.14
					9	59.14
					10	58.14
					11	57.14
					12	56.14
13.36	SS	3	100	17	13	55.14
GLACIAL TILL: Grey silty sand, trace clay, gravel, cobbles and boudlers	SS	4	33	6	14	54.14
15.18	SS	5	100	3	15	53.14
Weathered **BEDROCK**	SS	6	42	23	16	52.14
16.18						

End of Borehole

Practical DCPT refusal at 16.18m depth

(GWL @ 5.40m - Oct. 9, 2020)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 178-200 Isabella Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

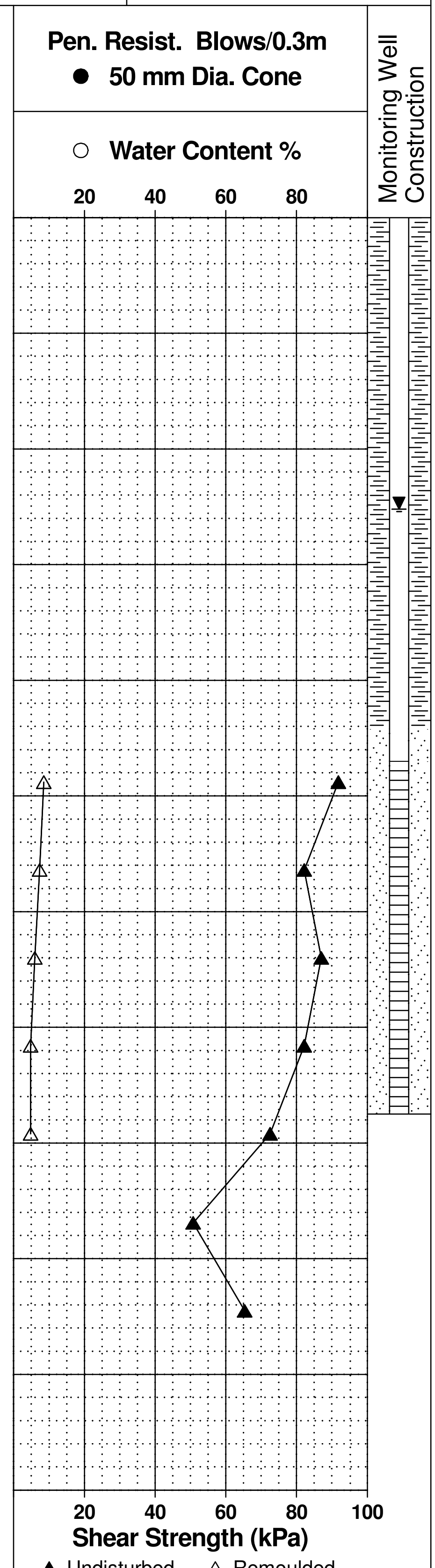
BORINGS BY CME-55 Low Clearance Drill

DATE October 7, 2016

FILE NO. PG3944

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	67.45
25mm Asphaltic concrete over brown silty sand with crushed stone, some clay, trace organics (to 0.60)	SS	1	83	10		
FILL: Brown silty clay with sand (0.60 to 1.37)	SS	2	67	4	1	66.45
Stiff, brown **SILTY CLAY** - grey by 3.7m depth (1.37 to 9.45)	SS	3	79	3	2	65.45
	SS	4	92	1	3	64.45
	SS	5	100	W	4	63.45
	SS	6	100	W	5	62.45
	G	1			6	61.45
	G	2			7	60.45
	G	3			8	59.45
					9	58.45
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 12.6m depth.					10	57.45
					11	56.45



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 178-200 Isabella Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 7, 2016

FILE NO. PG3944

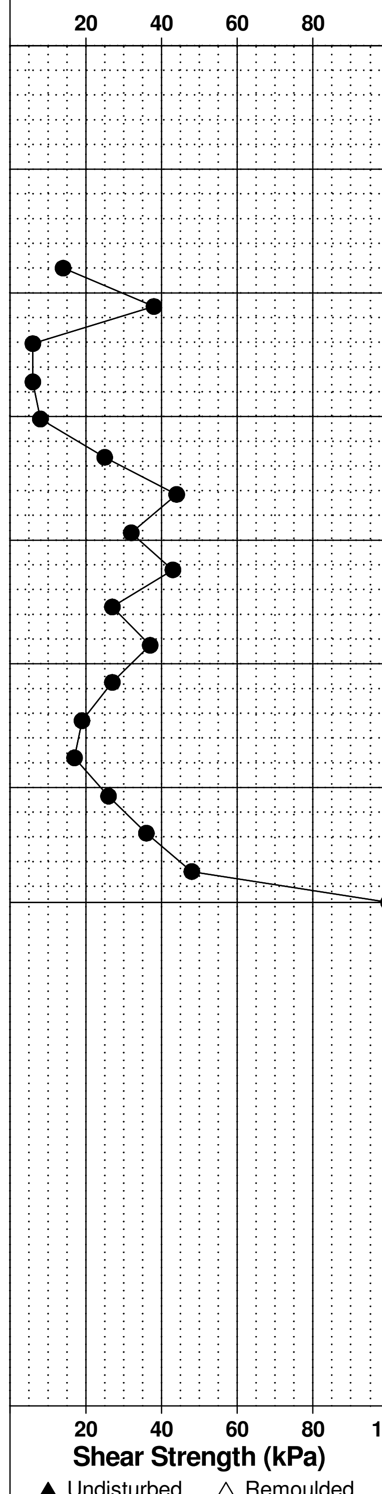
HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Monitoring Well Construction
GROUND SURFACE						11	56.45		
						12	55.45		
						13	54.45		
						14	53.45		
						15	52.45		
						16	51.45		
						17	50.45		
17.93									

End of Borehole

Practical DCPT refusal at 17.93m depth

(GWL @ 2.52m-October 14, 2016)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 178-200 Isabella Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 7, 2016

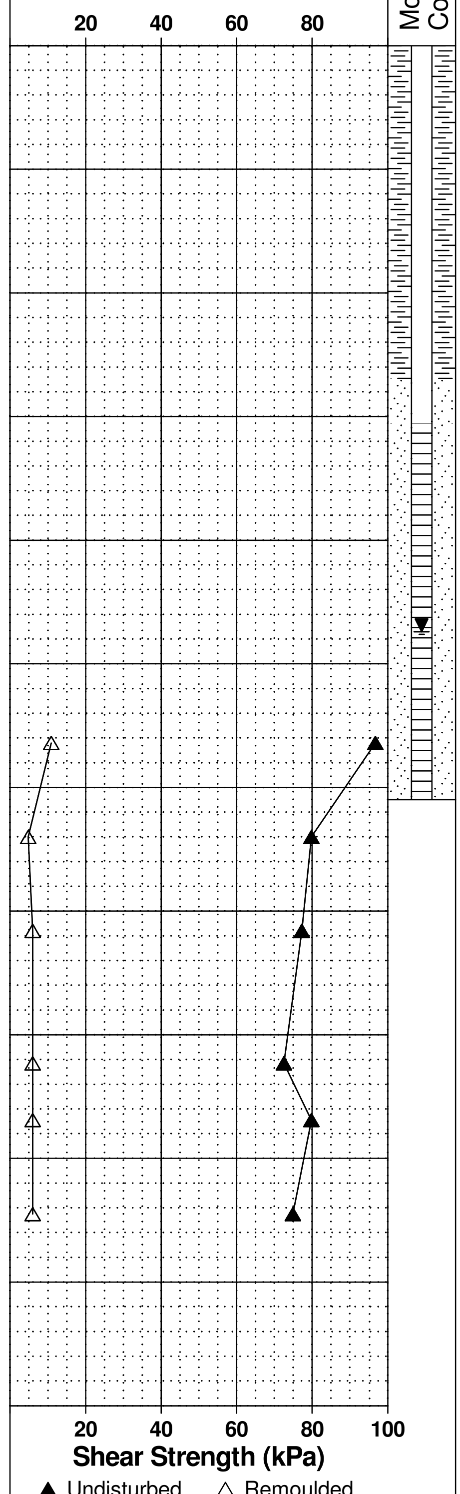
FILE NO. PG3944

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	67.90
FILL: Brown silty sand with crushed stone and construction debris		AU	1				
		SS	2	58	36	1	66.90
		SS	3	42	45	2	65.90
2.18							
Stiff, brown **SILTY CLAY,** trace sand		SS	4	4	4		
		SS	5	100	2	3	64.90
- grey by 3.8m depth		SS	6	100	2	4	63.90
		SS	7	100	2	5	62.90
						6	61.90
						7	60.90
						8	59.90
						9	58.90
9.45							
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 13.7m depth.						10	57.90
						11	56.90

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

○ Water Content %



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Building - 178-200 Isabella Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

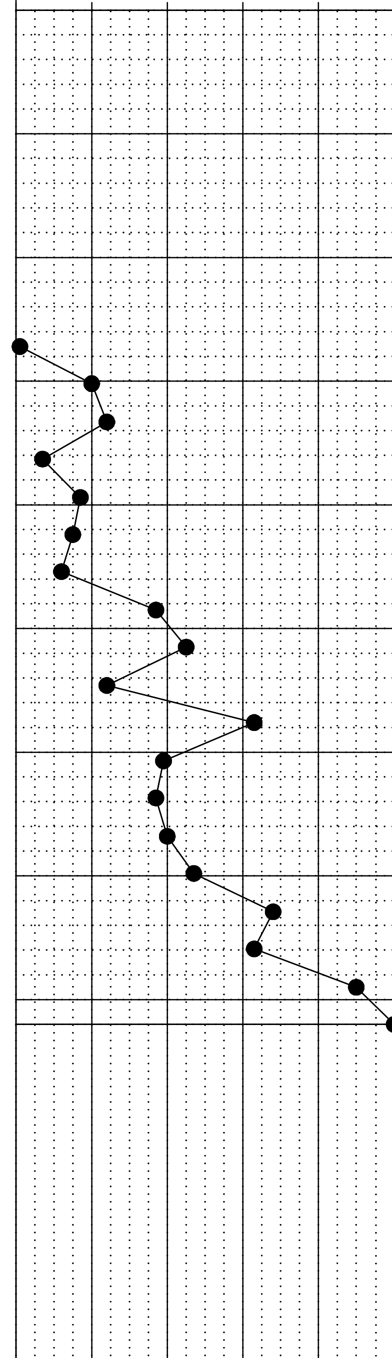
BORINGS BY CME-55 Low Clearance Drill

DATE October 7, 2016

FILE NO. PG3944

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Monitoring Well Construction
GROUND SURFACE								20 40 60 80	
						11	56.90		
						12	55.90		
						13	54.90		
						14	53.90		
						15	52.90		
						16	51.90		
						17	50.90		
						18	49.90		
						19	48.90		
19.20									
End of Borehole Practical DCPT refusal at 19.20m depth (GWL @ 4.74m-October 14, 2016)									



20 40 60 80 100
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 Building - 178-200 Isabella Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

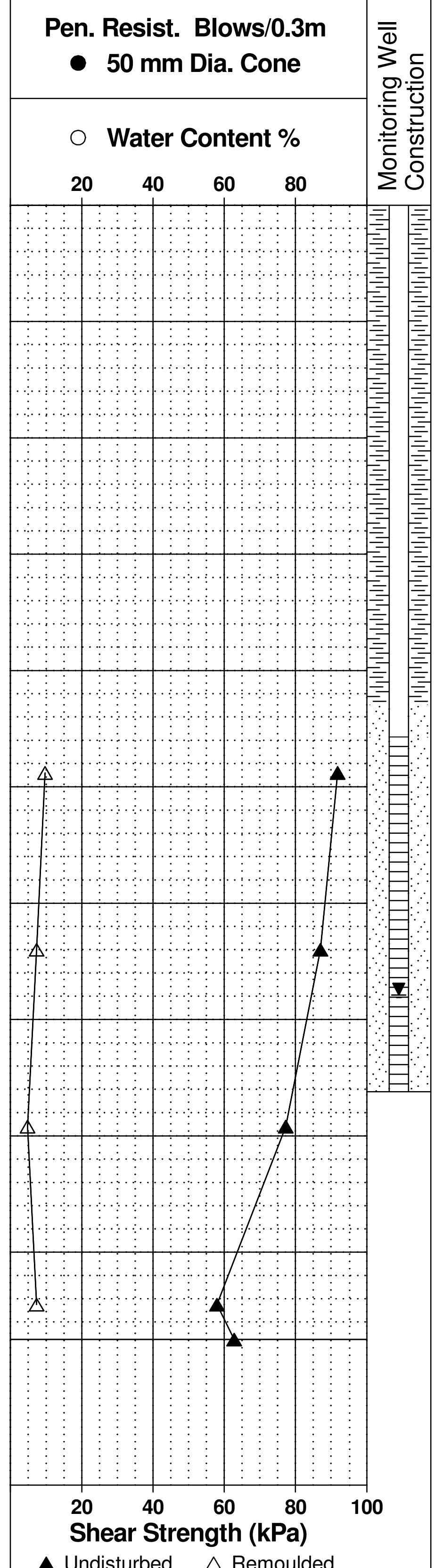
BORINGS BY CME-55 Low Clearance Drill

DATE October 7, 2016

FILE NO. PG3944

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	67.95
FILL: Brown silty clay with construction debris, some sand and gravel, trace organics	AU	1				
	SS	2	4	2	1	66.95
- styrofoam at 1.7m depth	SS	3	46	4		
2.29					2	65.95
Stiff, brown **SILTY CLAY,** trace organics	SS	4	100	4		
					3	64.95
	SS	5	100	2		
- grey by 4.1m depth	SS	6	100	1	4	63.95
					5	62.95
					6	61.95
					7	60.95
					8	59.95
					9	58.95
9.75						
End of Borehole						
(GWL @ 6.79m-October 14, 2016)						



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

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

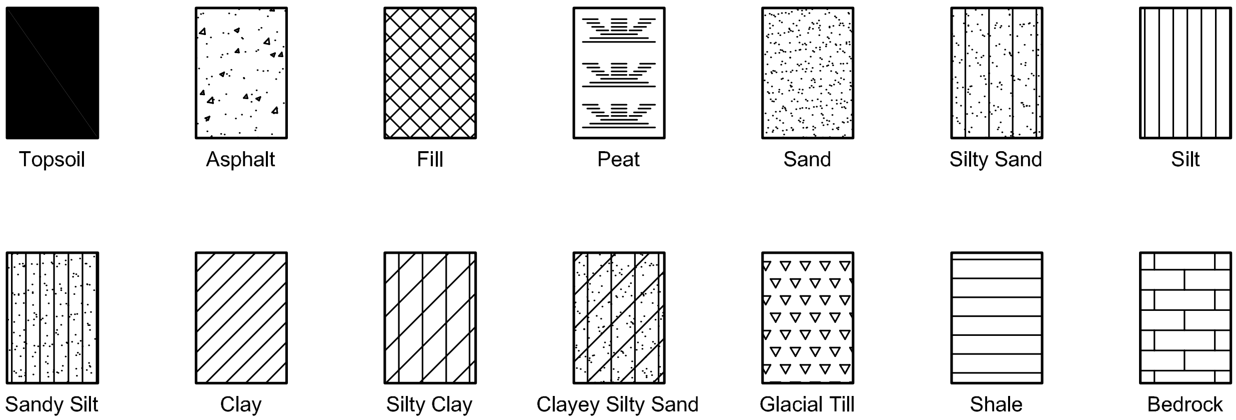
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

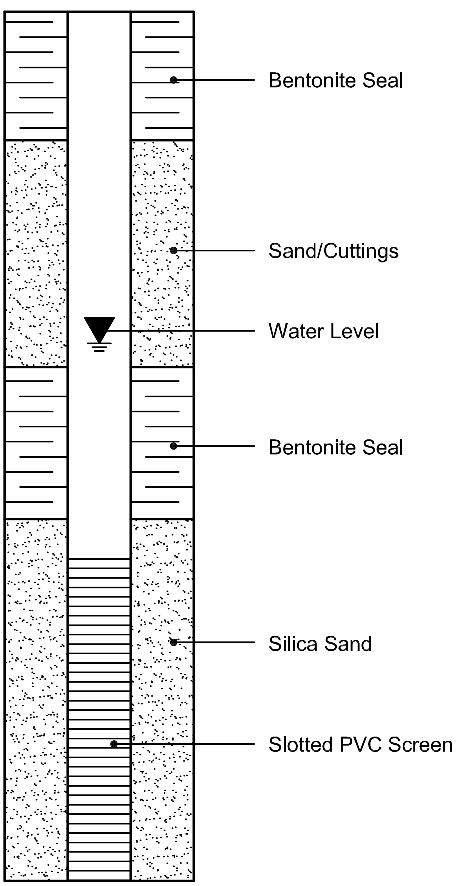
SYMBOLS AND TERMS (continued)

STRATA PLOT

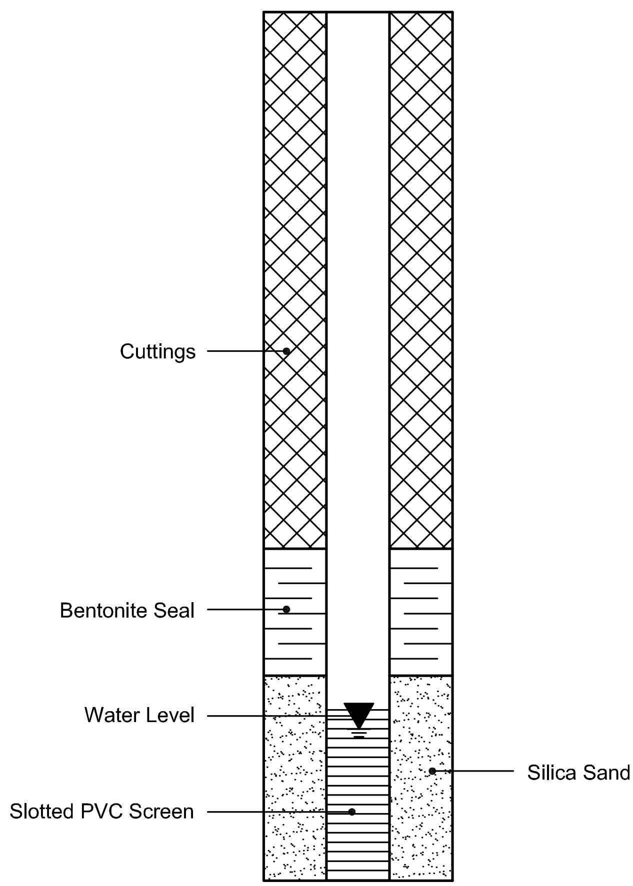


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 1642177

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 20955

Report Date: 14-Oct-2016

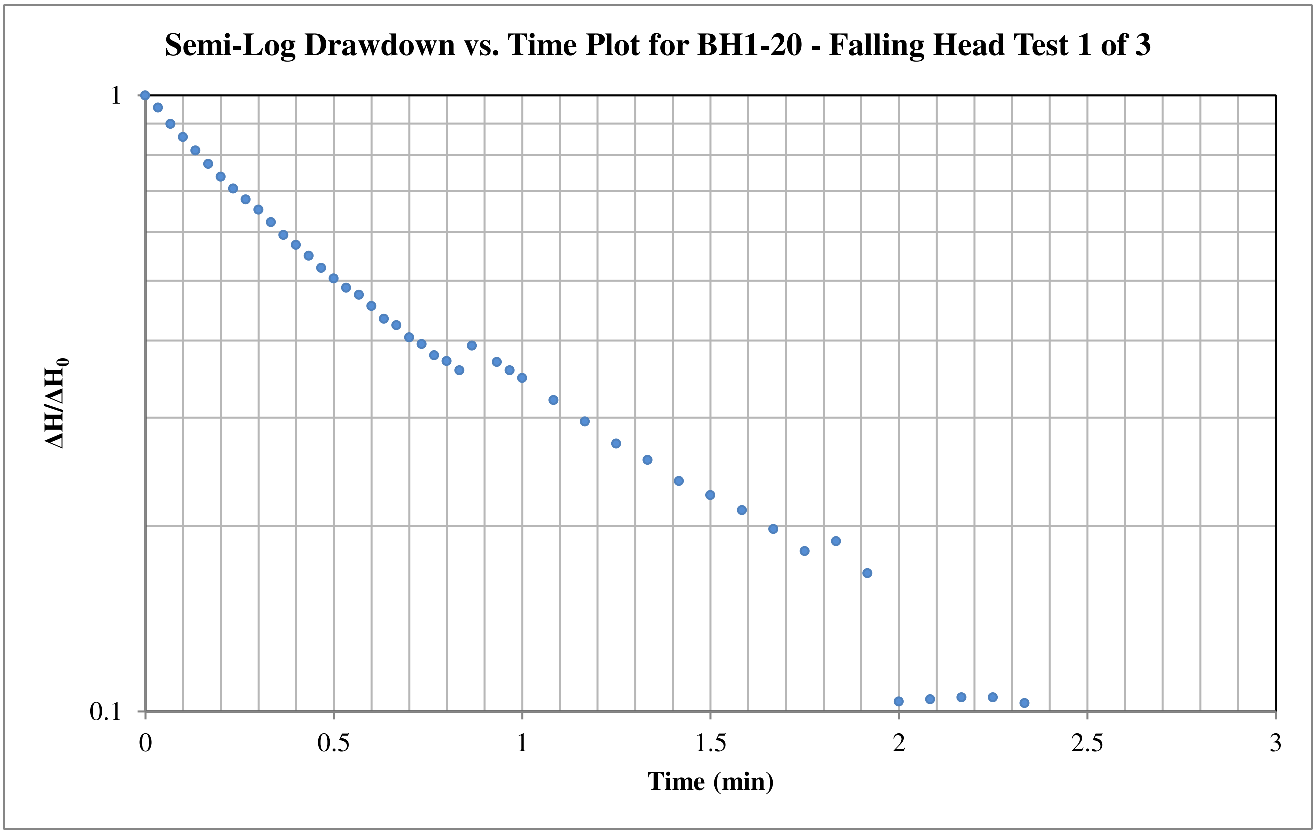
Order Date: 12-Oct-2016

Project Description: PG3944

	Client ID:	BH2-SS5	-	-	-
	Sample Date:	07-Oct-16	-	-	-
	Sample ID:	1642177-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	59.2	-	-	-
General Inorganics					
pH	0.05 pH Units	8.43	-	-	-
Resistivity	0.10 Ohm.m	23.1	-	-	-
Anions					
Chloride	5 ug/g dry	41	-	-	-
Sulphate	5 ug/g dry	112	-	-	-

Hvorslev Hydraulic Conductivity Analysis

Project: Minto Communities - 178-200 Isabella Street
Test Location: BH1-20
Test: Falling Head
Date: October 9, 2020



Hvorslev Horizontal Hydraulic Conductivity

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

Hvorslev Shape Factor

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

Valid for L>>D

Hvorslev Shape Factor F: 3.60152

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.032	m	Diameter of well
r_c	0.016	m	Radius of well

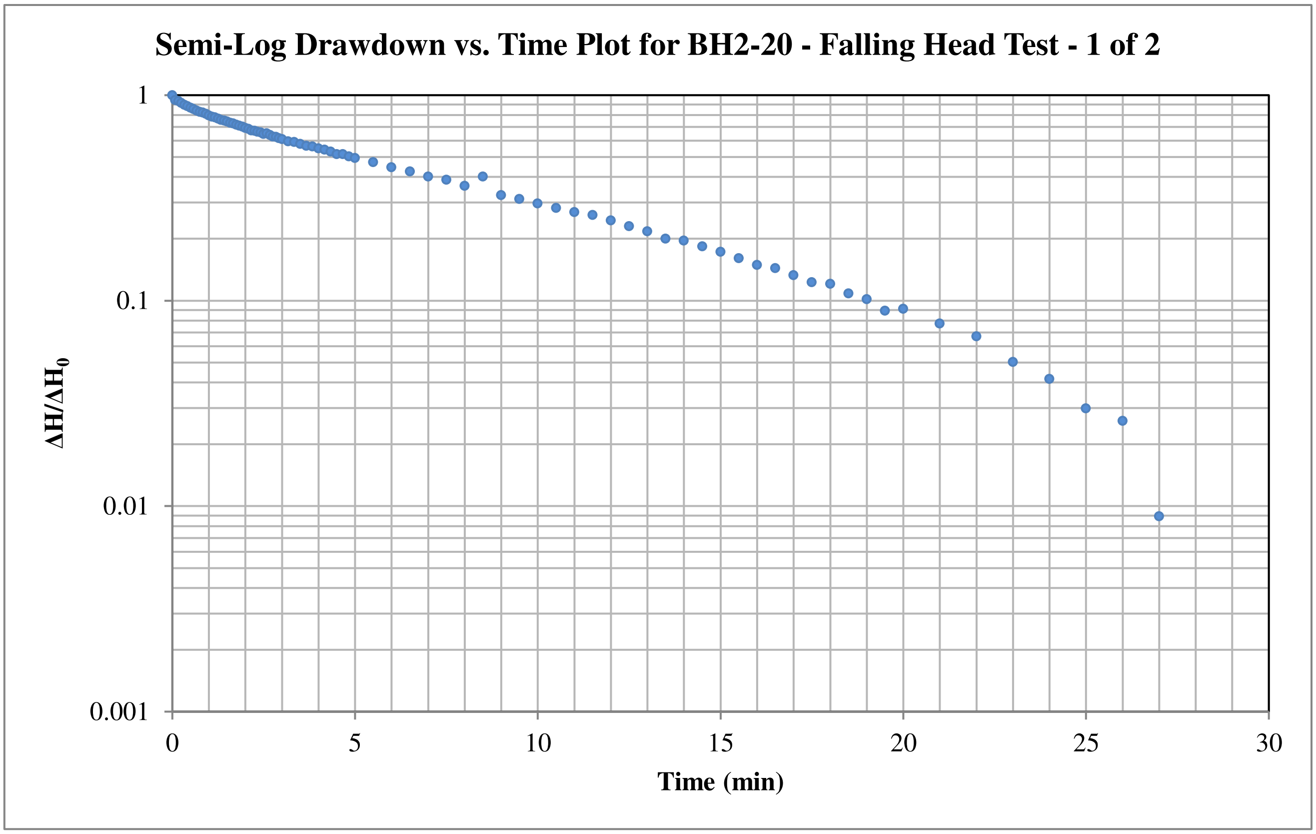
Data Points (from plot):

t^*: 0.802 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.62E-06 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Minto Communities - 178-200 Isabella Street
Test Location: BH2-20
Test: Falling Head
Date: October 9, 2020



Hvorslev Horizontal Hydraulic Conductivity

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

Hvorslev Shape Factor

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

Valid for L>>D

Hvorslev Shape Factor F: 3.60152

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.032	m	Diameter of well
r_c	0.016	m	Radius of well

Data Points (from plot):

t^*: 7.815 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.74E-07 m/sec

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5043-1 - TEST HOLE LOCATION PLAN

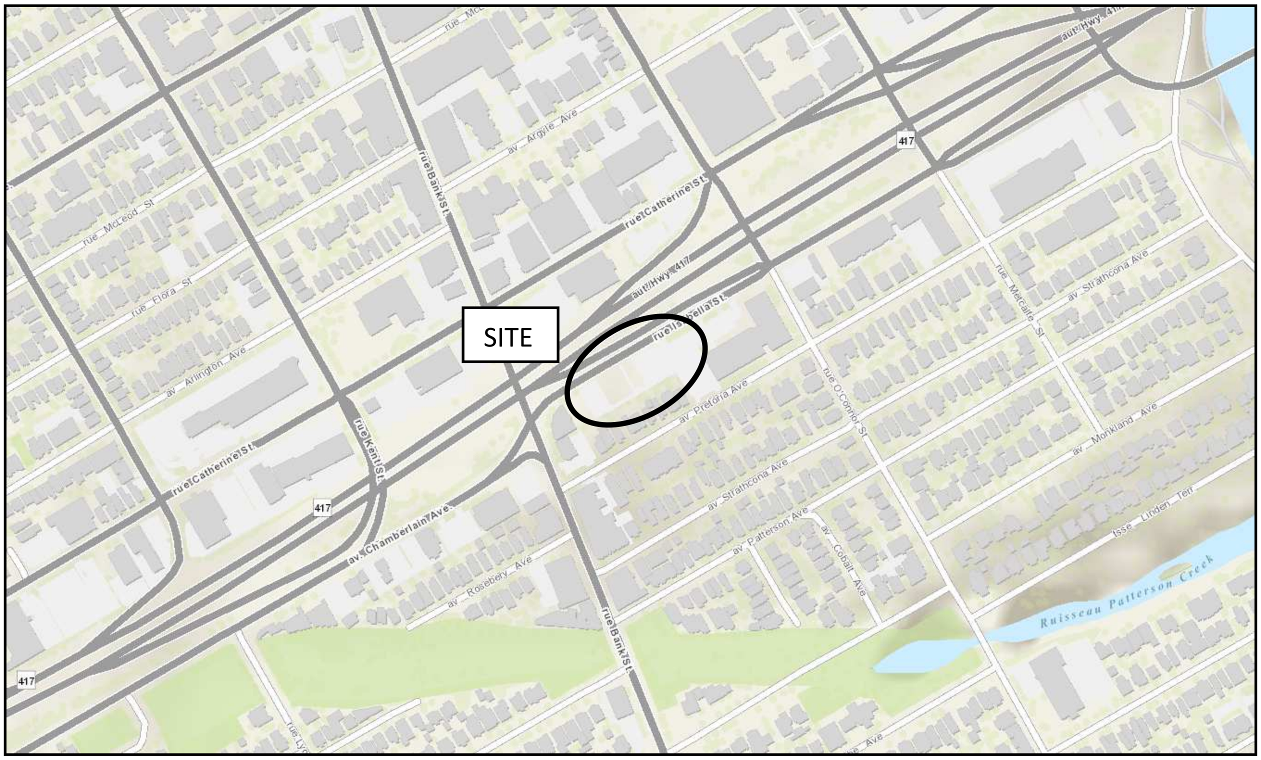


FIGURE 1

KEY PLAN

PATERSON GROUP

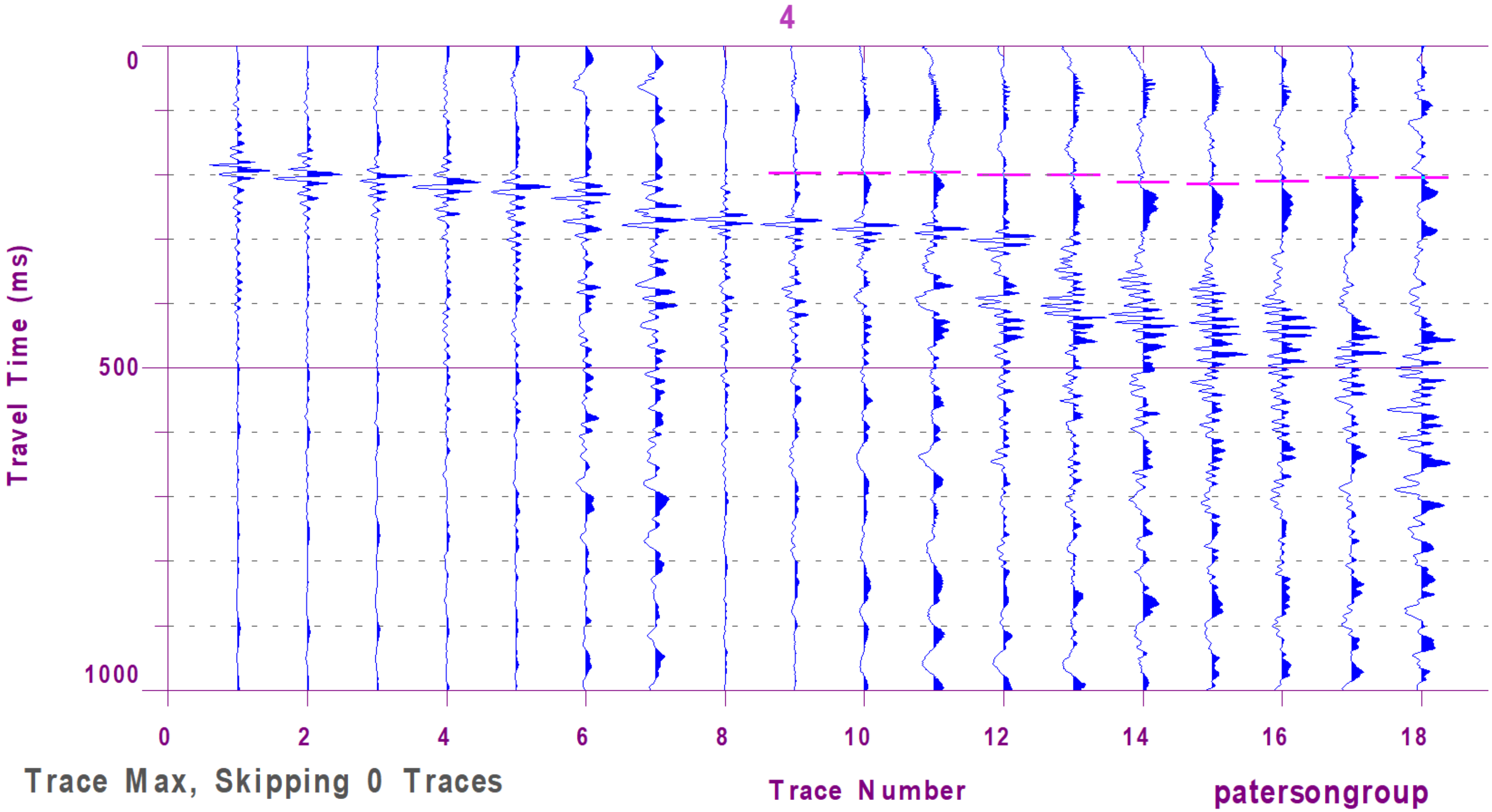


Figure 2 – Shear Wave Velocity Profile at Shot Location -20 m

PATERSON GROUP

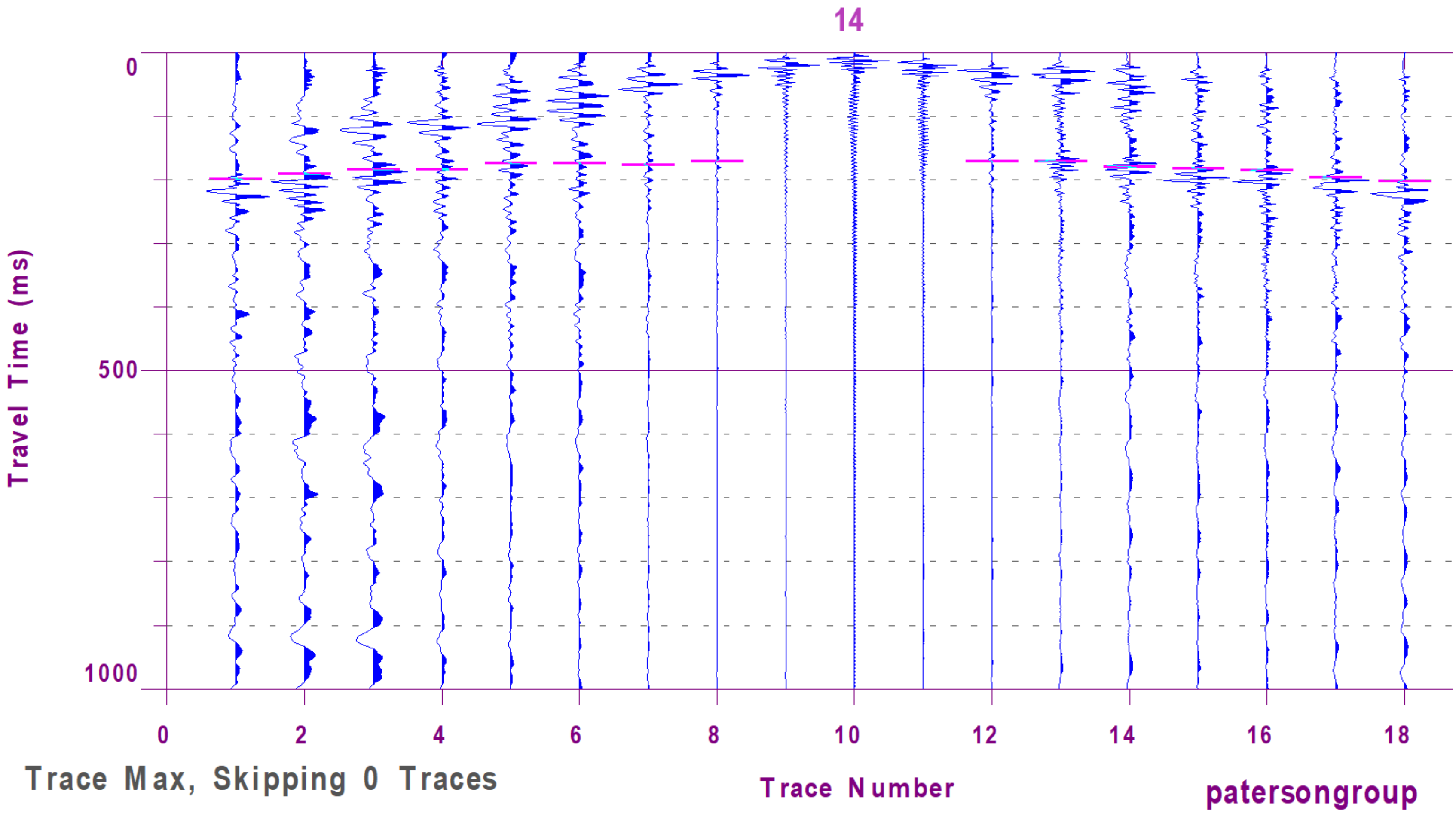
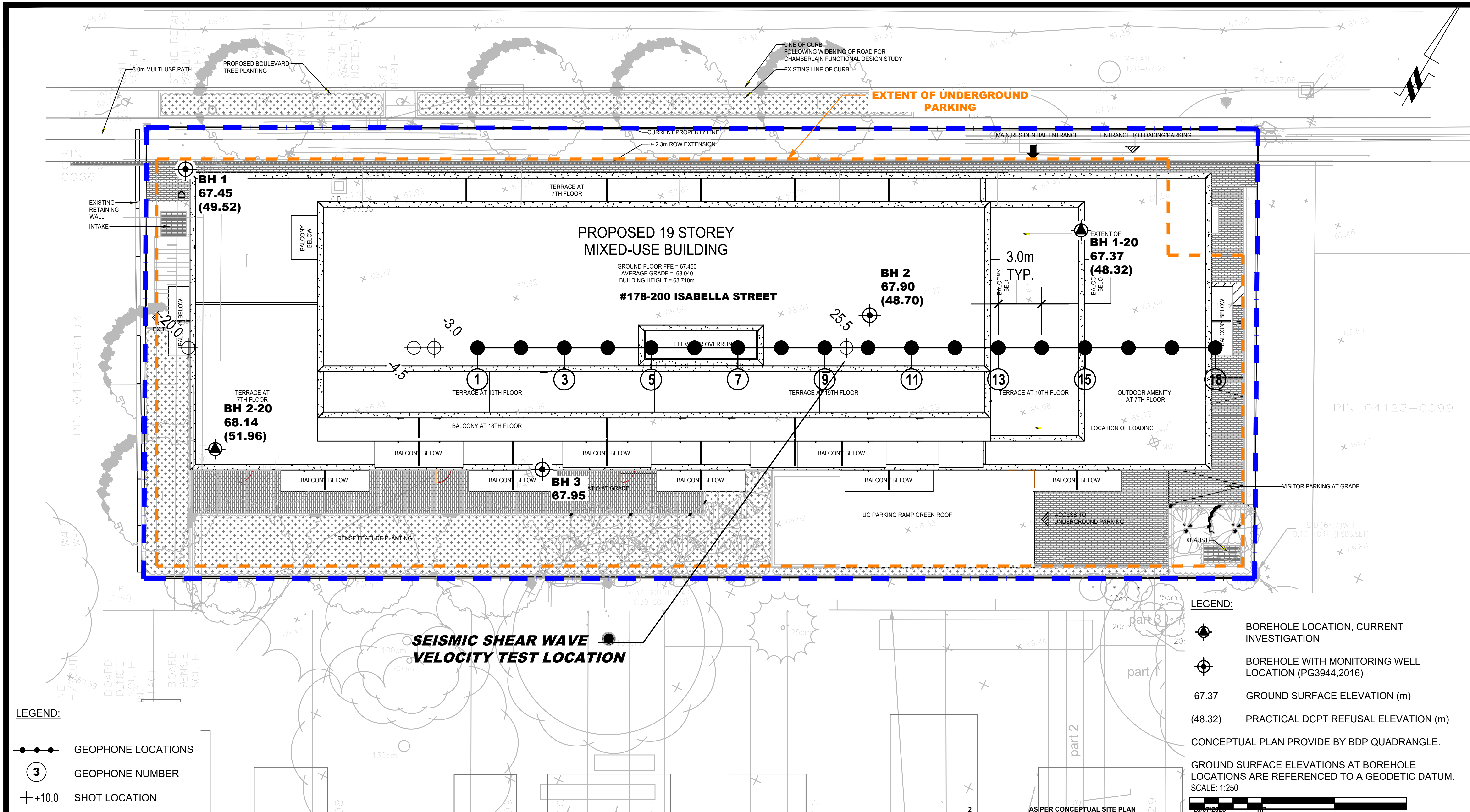


Figure 3 – Shear Wave Velocity Profile at Shot Location 25.5 m

PATERSON GROUP



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

NO.	REVISIONS	DATE	INITIAL
2	AS PER CONCEPTUAL SITE PLAN	28/07/2023	NP
1	SEISMIC SHEAR WAVE VELOCITY TEST ADDED TO PLAN	02/01/2022	FC

MINTO COMMUNITIES
GEOTECHNICAL INVESTIGATION
178-200 ISABELLA STREET & 205 PRETORIA AVENUE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 10/2020
Drawn by: YA	Report No.: PG5043-2
Checked by: KP	Dwg. No.: PG5043-2
Approved by: DJG	Revision No.: 2

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