# Geotechnical Investigation

## Proposed Multi-Storey Building

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

Isatoria Limited Partnership

**Report PG5043-1 Revision 3 dated December 10, 2025**

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# Appendices

# 1.0 Introduction

Paterson Group (Paterson) was commissioned by Isatoria Limited Partnership 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 (reference should be made 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 subsoils information for the design of the proposed building, 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

Based on the available drawings, it is understood that the proposed development will consist of a 19-storey residential building with 2 levels of underground parking that will extend over approximately the entire property.

Associated landscaped areas are also anticipated as part of the development. It is 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 geotechnical investigation was completed at the subject site by Paterson on October 7, 2016. At that time, three (3) boreholes were advanced to a maximum depth of 9.7 m below existing grade. A supplemental investigation was completed by this firm on October 6 and October 7, 2020, which consisted of 2 boreholes advanced to a maximum depth of 19.0 m below ground surface. The borehole locations were distributed in a manner to provide general coverage of the site and taking into consideration underground utilities and site features. The approximate locations of the boreholes are shown on Drawing PG5043-2 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted auger drill rig or a low clearance 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 from the auger flights or collected using a 50 mm diameter split-spoon sampler. The split-spoon and auger samples were inspected and 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**

All boreholes were fitted with monitoring wells to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. 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 diameter PVC screen at the base of each borehole.
- ❑ 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.

## 3.2 Field Survey

The borehole locations and ground surface elevations at each test hole location were surveyed by Paterson field personnel using a handheld GPS. Ground surface elevations at the borehole locations were referenced to a geodetic datum. The location of the boreholes and the ground surface elevation at each borehole location 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 approximately 1.5 m elevation difference above the adjacent property, including 205 Pretoria Avenue.

## 4.2 Subsurface Profile

**Overburden**

Generally, the subsurface profile at the borehole locations consists of topsoil, asphalt and/or fill material underlain by a thick silty clay deposit. The silty clay was underlain by a glacial till deposit and further underlain by bedrock.

The fill material was observed to generally consist 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 the time of the investigations at 17.9 and 19.2 m below ground surface at the location of BH 1 and BH 2, respectively. Weathered bedrock was encountered at 18.3 and 15.2 m below ground surface at the location of boreholes BH 1-20 and BH 2-20, respectively.

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

### Bedrock

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

## 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.

It should be noted that 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 and two basement level parking garage. Based on our review, foundation support may consist of either a raft foundation placed on an undisturbed, stiff, grey silty clay bearing surface, or an end-bearing deep foundation consisting of driven piles advanced to practical refusal.

Due to the presence of the silty clay layer, the subject site will have a permissible grade raise restriction. The permissible grade raise recommendations are discussed in Subsection 5.3.

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.

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

Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

**Fill Placement**

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

Under winter conditions, if snow and ice is present within the imported fill placed below future lower-level basement slabs, higher than tolerable amounts of settlement of the fill should be expected and support of a future basement slab and/or temporary supports for suspended slab pours will be negatively impacted and would undergo settlement during spring and summer time conditions. Paterson personnel should complete periodic inspections during fill import and placement to ensure that snow and ice quantities are minimized.

Any soft or poor performing areas should be removed and replaced with engineered fill consisting of OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm. The engineered fill should be placed in maximum 300 mm loose lifts and compacted to 98% SPMDD using suitable vibratory equipment.

Site-generated 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. Clayey workable fill must be compacted using a suitably sized vibratory sheepsfoot roller.

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 or adequate frost protection in areas overlain by settlement sensitive structures or hardscaping.

Care will need to be taken during storage, placement and compaction of the excavated native soils to maintain them in an unfrozen state and at a moisture content which is suitable for compaction. Soils intended for re-use which become frozen and/or which have excessive moisture contents will not be considered suitable for reuse at the subject site. Placement of site-generated material during winter months increases the risk of placing frozen material which may result in future poor performing areas that will require repair. Paterson field personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

**Vibration Considerations**

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

The following construction equipment could be the source of vibrations: pile driver, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by pile diving operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited. Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations.

As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz) and should comply to the City of Ottawa's S.P. No.: F-1201. The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed buildings.

**Protective Mud Slab (Raft Slab Foundation)**

Where a raft foundation is utilized, it is recommended that a minimum 50 mm thick lean concrete mud slab be placed on the undisturbed, silty clay subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance to the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay to potential disturbance due to drying, and immediately (i.e., within 48 hours) of exposing the clay bearing medium. It should be understood that the mud slab alone is not considered sufficient to mitigate the potential for the migration of frost within the clay bearing medium if construction is undertaken during winter conditions.

**Compacted Granular Fill Working Platform for Pile Foundation**

Since it is expected the proposed buildings may be supported on a deep foundation, the use of heavy equipment would be required to install piles (i.e., pile driving crane) or other deep foundation elements. It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance to the underlying soil.

It is recommended that a minimum 600 mm thick layer of OPSS Granular B Type II crushed stone or a combination of blast-rock and OPSS Granular B Type II crushed stone be placed as working platform throughout the building footprint which will support heavy equipment to facilitate deep foundation installations. The working pad granular should be compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in maximum 300 mm thick lifts.

Once the piles have been driven and cut off, the working platform can be re-graded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for the slab structure.

## 5.3 Foundation Design

### Raft Foundation

For our design calculations, a multi-storey building with two levels of underground parking was assumed. It is expected that the base of the raft foundation would be located at an approximate geodetic elevation of **60.20 to 60.70** m. <u>If the raft is anticipated to be founded at a different elevation, Paterson must be notified to review the applicability of the following bearing resistance values.</u> The bearing surface will consist of silty clay which is susceptible to disturbance under traffic condition and should be protected with the use of a mud slab, as described above.

The maximum serviceability limit states (SLS) contact pressure (includes the raft embedment compensation) can be taken to be **160 kPa**. It should be noted that the weight of the raft slab and all materials and loading overlying the raft has to be included when designing with this value. The factored bearing resistance (contact pressure) at ULS can be taken as **240 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS. The modulus of subgrade reaction was calculated to be **6 MPa/m** for a contact pressure of **160 kPa**.

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.

### 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 (2) to four (4) piles would be recommended and as undertaken and measured by Paterson at the time of pile testing.

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. A full-time field review program should be conducted by Paterson field personnel during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical).

**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 | 1,100 | 6 | 27 |
| 245 | 11 | 1,050 | 1,260 | 6 | 31 |
| 245 | 13 | 1,200 | 1,440 | 6 | 35 |

The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Down Drag Loads

Due to the presence of the clay deposit below the subject site, and potential long-term degradation of organics within the clay deposit, down drag loads should be considered during the final design of the piles.

Based on the available subsurface information, it is expected that the piles will be driven through approximately 7 to 8 m of clay. Assigning an adhesion factor of 1.0 (as per the Canadian Foundation Engineering Manual), the clay can be taken to have an ultimate adhesion of 45 kPa against the sides of the piles.

The down drag load is effectively applied to each pile at the location of the "neutral plane," where negative (i.e., down drag) skin friction becomes positive shaft resistance. In the case of the end-bearing piles at this site, the neutral plane will be located near the bedrock surface.

The down drag load is a structural pile capacity criterion and does not affect the geotechnical capacity of the piles. The structural axial capacity of the pile is governed by its structural strength at the neutral plane when subjected to the permanent load plus the down drag load. Transient live load is not to be included. At or below the pile cap, the structural strength of the embedded pile is determined as a short column subjected to the permanent load plus the transient live load, but down drag load is to be excluded.

At the depth of the neutral plane where the down drag load is applied, the pile structure is well confined. The 5$^{th}$ edition of the Canadian Foundation Engineering Manual recommends that the allowable structural axial capacity of piles at the neutral plane, for resisting permanent load plus the down drag load, can be determined by applying a factor of safety of 1.5 to the pile material strength (steel yield and concrete 28-day compressive strength).

**Conventional Shallow Footings for Auxiliary Structures**

The following bearing resistance values may be considered for auxiliary structures, such as for lightly loaded portions of the structure (i.e., canopies, parking garage wing-walls, etc.) and other potential external structures. These values are not considered applicable to the foundation support of the main structure at this time.

Strip footings, up to 2 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, stiff brown silty clay bearing surface (and more than 1.5 m above the grey silty clay layer) 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.

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.

Bearing mediums are 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.

**Permissible Grade Raise Recommendations**

While grade raises are not expected throughout the subject site due to the urban nature of the subject site and the building footprint taking up the majority of the subject sites footprint, a permissible grade raise of **2.0 m** above existing ground surface is provided for design purposes for the subject site for areas beyond a potential raft or deep foundation supported structure. It should be noted that a post-development long-term groundwater lowering of 0.5 m was conservatively applied to the permissible grade raise restriction.

## 5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site designation for the proposed building according to the Ontario Building Code 2024. 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 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. 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. Based on this, the $V_{s30}$ was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code 2024, as presented below.

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

$$V_{s30=} \frac{30\,m}{\left(\frac{20\,m}{225m/s} + \frac{10\,m}{2{,}045\,m/s} + \right)}$$

$$V_{s30=}\ 320\,m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, $V_{s30}$, for foundations at the aforementioned site is **320 m/s**. Therefore, a **Site Designation $X_{320}$** is applicable for the design of the proposed building, as per OBC 2024.

The soils underlying the subject site are not considered susceptible to liquefaction or cyclic softening based on the methods outlined in the current edition of the Canadian Foundation Engineering Manual.

## 5.5 Basement Slab

With the removal of all topsoil and deleterious materials within the footprint of the proposed building, the native silty clay bearing surface, approved by Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the basement slab construction.
If a raft slab is considered, 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 piping requirements.

For a building founded on piles, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone for areas located beyond the portions of the parking and access lane portions of the basement level (reference should be made to Section 5.7 of this report). 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.

A sub-floor drainage system, consisting of perforated drainage pipes connected to a positive outlet or to the building's sump pit, should be incorporated to drain any water that migrates within the sub-slab fill layer. The spacing of the sub-slab drainage pipes can be determined by Paterson during the design stage and prior to the pre-construction/tender phase, and as discussed in Section 6.1 of this report.

## 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 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. However, if a full drainage system is being implemented and approved by Paterson at the time of construction, hydrostatic pressure can be omitted in the structural design.

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)
$\gamma$ = 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 ($\Delta P_{AE}$).

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

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

The peak ground acceleration, ($a_{max}$), for the subject site is 0.36 g according to OBC 2024. 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 2024.

## 5.7 Pavement Design

### Flexible Pavement Structure for Soil Subgrade

The recommended flexible pavement structures shown in Tables 3 and 4 would be applicable for portions of pavement supported directly upon soil subgrade.

**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 |
| 450 | **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.

**Pavement Joint Tie-in and Road-Cut Reinstatement**

Where the proposed pavement structure meets an existing pavement structure, such as the existing road, or a road-cut is required to facilitate the connection to existing municipal services, the following recommendations should be followed:

- A 300 mm wide section of the existing asphalt roadway should be saw cut from the existing pavement edge to provide a sound surface to abut the proposed pavement structure.
- It is recommended to mill a 300 mm wide and 40 mm deep section of the existing asphalt at the saw cut edge.
- The proposed pavement structure subbase materials should be tapered no greater than 3H:1V to meet the existing subbase materials.
- Clean existing granular road subbase materials can be reused upon assessment by Paterson at the time of excavation (construction) as to its suitability.

Minimum Performance Graded (PG) 64-34 asphalt cement should be used for municipal right-of-way road-cut purposes. Cement asphalt should be compacted to a minimum average density of 93% and no more than 98%. The pavement granular 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.

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. All reinstatement efforts must be undertaken in accordance with the City of Ottawa's *Standard Detail R10 – Standard Trench Reinstatement in Paved Surface* dated March 2023 and other pertinent details, specifications and requirements identified by the City of Ottawa.

All subgrade fill is recommended to be placed in 300 mm maximum thick loose lifts and compacted to a minimum of 95% of the materials SPMDD (or as otherwise advised by the City of Ottawa) and reviewed and approved by Paterson personnel at the time of construction. All reinstatement efforts are recommended to be reviewed and approved by Paterson at the time of construction.

# 6.0 Design and Construction Precautions

## 6.1 Foundation Drainage

The following recommendations may be considered for the architectural design of the building's foundation drainage system. 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 during the detailed design and pre-construction phases. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

**Watertight Foundation/Groundwater Suppression Waterproofing Overview**

It is recommended that a water-tight foundation waterproofing system be implemented for the proposed buildings foundation walls. This is recommended to mitigate potential long-term dewatering of the surrounding clay deposit that may occur if a conventional foundation drainage system with associated drainage at the footing level is considered. Additional measures are not required throughout the base of the excavation, however, a sub-slab drainage system is recommended to be provided throughout the basement level.

It is expected that foundations will be constructed in a blind-cast manner and against a wood-lagging and appropriately smooth and prepared surface (requirement to be confirmed by the manufacturer of the associated waterproofing membrane) to accept a waterproofing membrane. It is recommended that contractors provide approved manufacturer details for waterproofing at transition zones between lagging and soldier piles, at tieback locations and areas of pipe or conduit/utility penetrations.

For a blind-sided pour, a high-density polyethylene (HDPE) film and pressure sensitive adhesive face waterproofing membrane, such as *gcp PrePrufe 275* or *Henry Blueskin Preseal 320* (or equivalent other reviewed and approved by Paterson) should be placed against the foundation walls between underside of footing level and extending to finished grade.

The membrane is recommended to overlap below the overlying perimeter foundation footprint by a minimum of 600 mm 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, efforts will be required to be made to provide adequate bond conditions for the concrete foundation wall and proposed waterproofing membrane. Alternative membranes may be considered during the design phase, however, are not recommended to be coupled with a supplemental layer of composite foundation drainage board unless the drainage board layer may be permitted by the manufacturer to be non-drained (i.e., no connection to perimeter drainage system).

**Additional Waterproofing Recommendations and Considerations**

Where the membrane will be installed in contact with portions of the shoring system that would be removed in the future, it is recommended to install the membrane against a layer of pressure-treated timber sheathing, cement board or other flame-resistant material that will protect the membrane from being damaged by shoring removal efforts (anticipated to consist of the use of acetylene torches and excavation works). This should be completed for the entire height of the membrane that will be in contact with the pile shortening work.

Sufficient fasteners should be supplied to securely fasten the membrane to the substrate and promote a relatively taught application of the membrane to the substrate and minimize slack across the installed membrane. Sufficient manufacturer approved sealants and waterstops should be supplied to cover all protrusions, overlaps and fastener locations to provide an adequate seal from the intrusion of water onto and across the concrete foundation wall.

Where the membrane transitions from blind- to double-sided pours, it should not be folded onto the ground surface and restrained in place by the formwork since this will damage the integrity of the membrane and require remedial work/repairs.

If podium areas (i.e., foundation wall footprint does not align with overlying building footprint) will be considered in the design, additional details will be required to be prepared to mitigate the potential for water collected on the podium deck to migrate behind the waterproofing membrane. This should be verified by Paterson, the architect and associated design team members during the design stage.

A perimeter foundation drain may be implemented for the portion of the structure that would be backfilled and cast using a double-sided pour methodology. This would be beneficial for providing drainage to the upper backfill layers to minimize frost heave movement in frost-susceptible soils placed for backfill in the upper portion of the excavation. If considered, a perimeter drainage pipe consisting of a minimum 100 mm diameter corrugated perforated pipe outfit with a geosock placed no deeper than 2 m below finished grade and connected to a gravity outlet may be considered for this purpose.

### Elevator Shaft and Additional Sub-Floor Structure Waterproofing

Elevator shafts located below the underslab drainage system should be provided full-depth positive-side waterproofing and provided with a PVC waterstop at the shaft wall and footing interface. This may be accomplished by the use of the above-noted waterproofing membranes and detailed by Paterson during the design stage.

### Interior Perimeter and Underfloor Drainage

An interior perimeter and underfloor drainage system will be required to redirect water collected with the lowest level basement sub-slab fill layer 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 several runs of underfloor drainage pipes should consist of minimum 100 mm diameter corrugated perforated plastic pipe sleeved with a geosock placed in a north-south and east-west orientation.

The invert of the system may be equivalent to either the top of the raft or the top of the perimeter pile cap and remain flat across its footprint provided a gravity connection to the dedicated sump pit is provided. The spacing of the underfloor drainage should be confirmed by Paterson during the design stage and at the time of excavation when water infiltration can be better assessed.

### Foundation Backfill

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 to preserve the integrity of the waterproofing membrane during freeze-thaw cycles.

Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. Provisions should be carried to adequately protect the waterproofing membrane during backfilling of double-sided pour areas.

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).

**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, an additional waterproofing membrane is recommended to span the length of the cold-joint and extend a minimum of 1.2 m on both sides of the cold joint. This is recommended to be reviewed and coordinated with the construction team during the pre-construction phase once raft slab pour sequences are known.

**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.

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 Type 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 ($\gamma$), kN/m$^3$ | 20 |
| Submerged Unit Weight ($\gamma$'), 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.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The bedding layer thickness should be increased to a minimum thickness of 300 mm if the subgrade consists of grey silty clay. Clear stone is not recommended for use as bedding material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD. The bedding material should extent at a minimum to the spring line of the pipe.

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 consist of the soils exposed at the trench walls to minimize differential frost heaving. All trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD and is recommended to be reviewed and approved by Paterson field personnel at the time of construction.

**Clay Seals**

To reduce long-term lowering of the groundwater level at the subject site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The seals should consist of relatively dry and compactible brown or workable silty clay soil placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material’s SPMDD. The clay seals should be placed at the site boundaries in the service trenches.

## 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.

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

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

**Adverse Effects of Dewatering on Adjacent Properties**

Since the proposed development will be founded below the long-term groundwater level, a groundwater suppression system has been recommended to mitigate the potential for long-term dewatering of the surrounding clay subsoils. 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 and is not anticipated to impact neighboring structures due to the low hydraulic conductivity of the in-situ soils and short-term nature of the temporary dewatering efforts.

## 6.6 Winter Construction

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

Where excavations are completed in proximity to existing structures which may be adversely affected due to the freezing conditions. The subsurface conditions 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 particular, where a shoring system is constructed, 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 documents to protect the walls of the excavations from freezing, if and where applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and/or glycol lines and tarpaulins or other suitable means. 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 foundation is protected with sufficient soil cover to prevent freezing at founding level.

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

Under winter conditions, if snow and ice are present within imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

## 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.

## 6.8 Landscaping Considerations

### Tree Planting Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Given the founding depth of the proposed structures, trees may be planned assuming the pertinent portion of the subsoils consist of clay of low to medium potential for soil volume change.

The following tree planting setbacks are recommended for the low to medium sensitivity silty clay deposit and where trees are located near buildings founded on cohesive soils.

- Large trees (mature height over 14 m) can be planted within these areas provided that a tree to foundation setback equal to the full mature height of the tree can be provided.

- Tree planting setback limits may be reduced to 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m), provided that the conditions noted below are met.

- A small tree must be provided with a minimum of 25 $m^3$ of available soils volume while a medium tree must be provided with a minimum of 30 $m^3$ of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

- The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

- Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the Grading Plan.

It is well documented in the literature that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e., Manitoba Maples) and, as such, they should not be considered in the landscaping design.

# 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 waterproofing and underfloor drainage systems.
- ❑ Dynamic load testing of select test piles prior to production piling.
- ❑ 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 contractor's 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 Isatoria Limited Partnership or their agents is not authorized without review by Paterson.

**Paterson Group Inc.**

LICENSED PROFESSIONAL ENGINEER
December 10, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Drew Petahtegoose, P.Eng.

Faisal I. Abou-Seido, P.Eng.

**Report Distribution:**

- ❑ Isatoria Limited Partnership (Email Copy)
- ❑ Paterson Group (1 Copy)

# APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY TEST RESULTS

**patersongroup** Consulting Engineers

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 1-20

**BORINGS BY** Track-Mount Power Auger **DATE** October 6, 2020

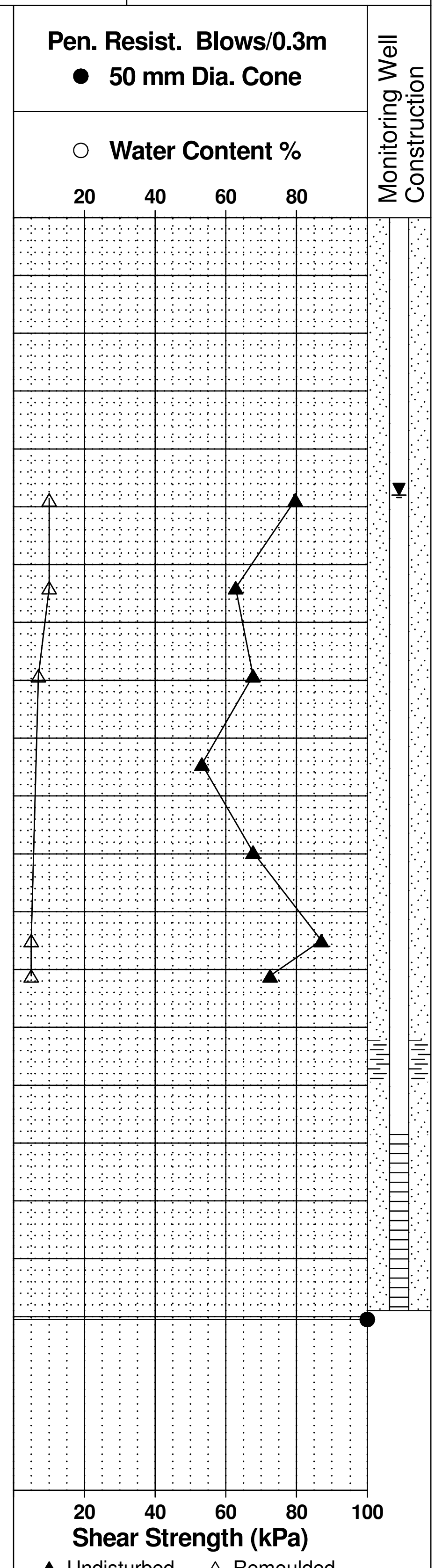
| SOIL DESCRIPTION | Bottom depth (m) | Sample TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|---|
| GROUND SURFACE | | | | | |
| **TOPSOIL** | 0.25 | AU | 1 | | |
| Stiff, brown **SILTY CLAY,** trace sand | 3.05 | SS | 2 | 100 | 7 |
| Stiff, grey **SILTY CLAY** | 13.72 | SS | 3 | 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 |



**patersongroup** Consulting Engineers

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

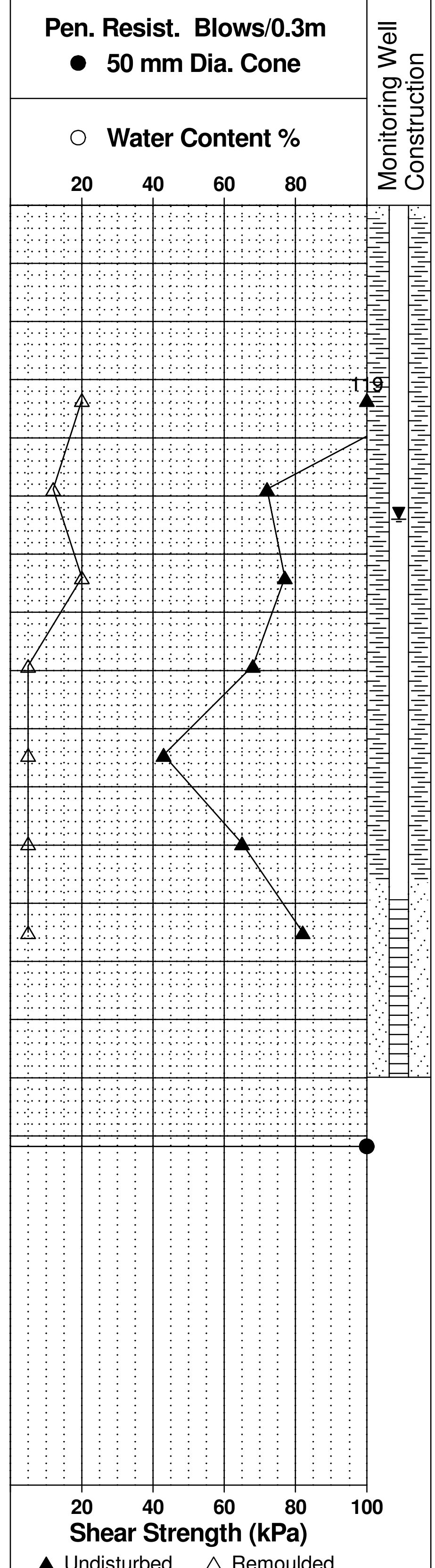
| SOIL DESCRIPTION | Depth to base (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|---|
| GROUND SURFACE | | | | | |
| Concrete pavement | 0.07 | AU | 1 | | |
| **FILL:** Brown silty sand with gravel | 0.60 | | | | |
| Stiff, brown **SILTY CLAY** | 3.05 | SS | 2 | 100 | 5 |
| Stiff to firm, grey **SILTY CLAY** | 13.36 | SS | 3 | 100 | 17 |
| **GLACIAL TILL:** Grey silty sand, trace clay, gravel, cobbles and boudlers | 15.18 | SS | 4 | 33 | 6 |
| | | SS | 5 | 100 | 3 |
| Weathered **BEDROCK** | 16.18 | SS | 6 | 42 | 23 |

End of Borehole

Practical DCPT refusal at 16.18m depth

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

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 68.14 |
| 1 | 67.14 |
| 2 | 66.14 |
| 3 | 65.14 |
| 4 | 64.14 |
| 5 | 63.14 |
| 6 | 62.14 |
| 7 | 61.14 |
| 8 | 60.14 |
| 9 | 59.14 |
| 10 | 58.14 |
| 11 | 57.14 |
| 12 | 56.14 |
| 13 | 55.14 |
| 14 | 54.14 |
| 15 | 53.14 |
| 16 | 52.14 |



# patersongroup Consulting Engineers

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

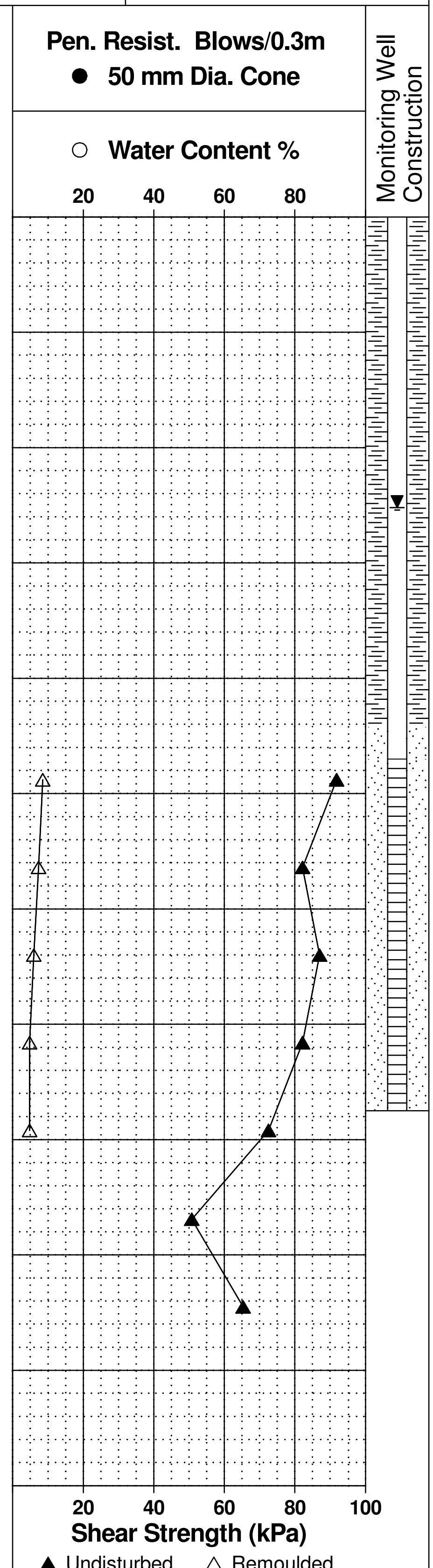
**FILE NO.** PG3944

**REMARKS**

**HOLE NO.** BH 1

**BORINGS BY** CME-55 Low Clearance Drill **DATE** October 7, 2016

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE 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 (0.60) | | SS | 1 | 83 | 10 | | |
| **FILL:** Brown silty clay with sand (1.37) | | SS | 2 | 67 | 4 | 1 | 66.45 |
| Stiff, brown **SILTY CLAY** | | SS | 3 | 79 | 3 | 2 | 65.45 |
| | | SS | 4 | 92 | 1 | | |
| | | SS | 5 | 100 | W | 3 | 64.45 |
| - grey by 3.7m depth | | SS | 6 | 100 | W | 4 | 63.45 |
| | | G | 1 | | | 5 | 62.45 |
| | | | | | | 6 | 61.45 |
| | | G | 2 | | | 7 | 60.45 |
| | | | | | | 8 | 59.45 |
| (9.45) | | G | 3 | | | 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**

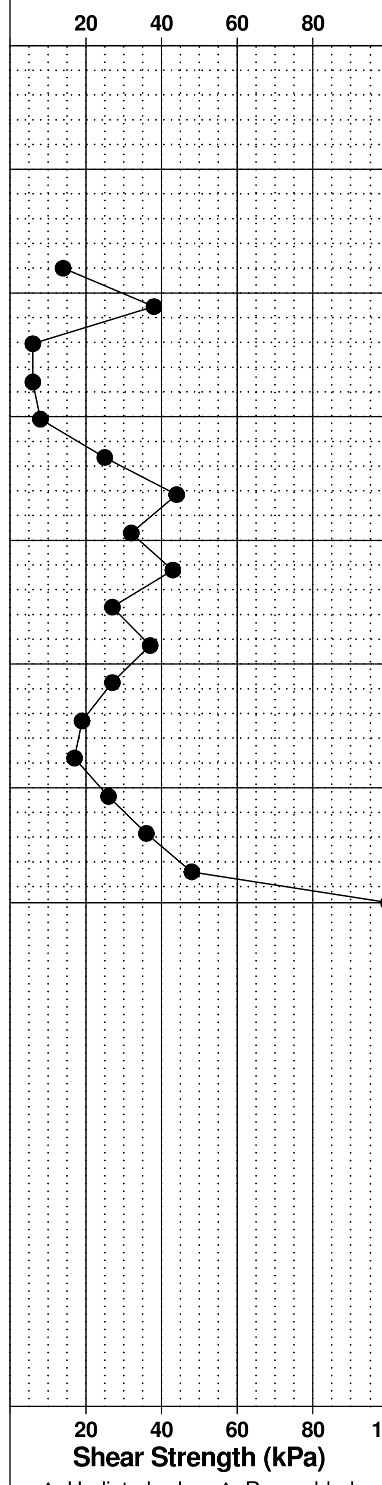
**BORINGS BY** CME-55 Low Clearance Drill

**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<br><br>Practical DCPT refusal at 17.93m depth<br><br>(GWL @ 2.52m-October 14, 2016) | | | | | | | | | |



**patersongroup** Consulting Engineers

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

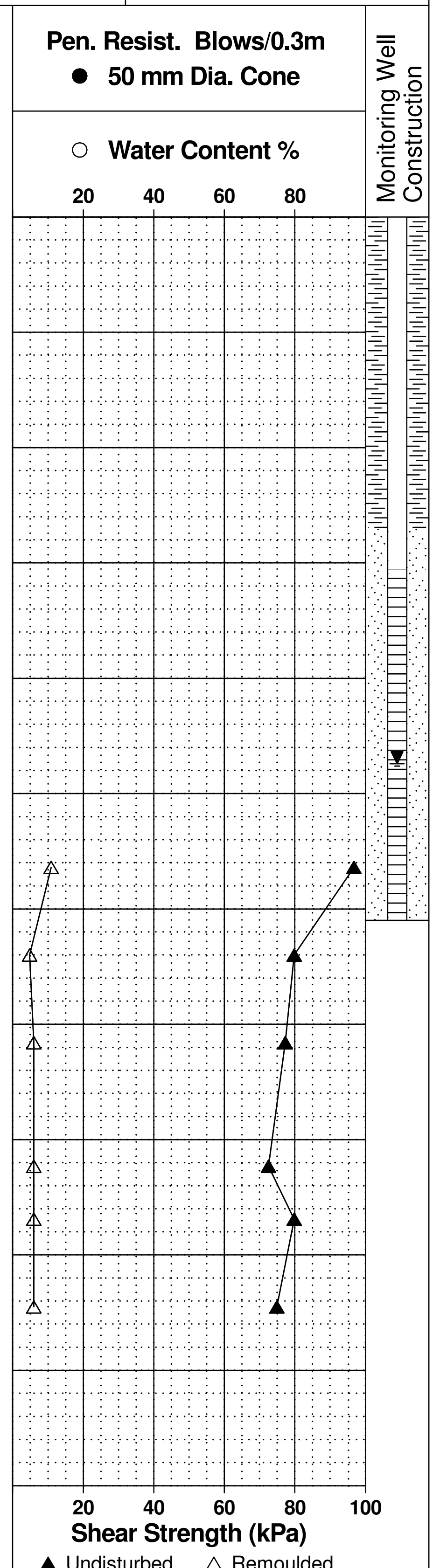
**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill **DATE** October 7, 2016

**FILE NO.** PG3944

**HOLE NO.** BH 2

| SOIL DESCRIPTION | SAMPLE TYPE | 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 | 3 | 64.90 |
| | SS | 5 | 100 | 2 | | |
| - 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 |



**patersongroup** Consulting Engineers

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**

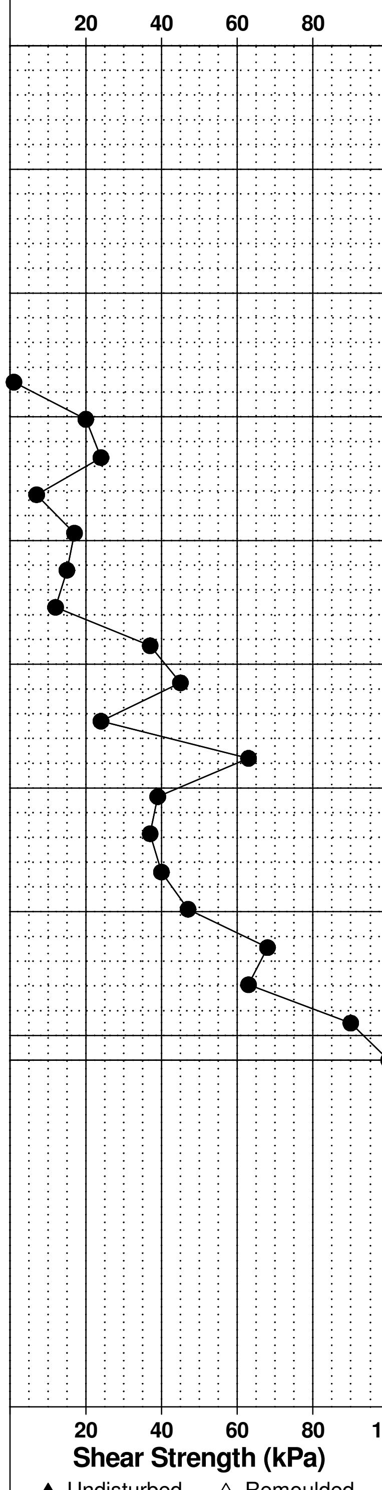
**BORINGS BY** CME-55 Low Clearance Drill

**DATE** October 7, 2016

**FILE NO.** PG3944

**HOLE NO.** BH 2

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | | |
| | | | | | | 11 | 56.90 |
| | | | | | | 12 | 55.90 |
| | | | | | | 13 | 54.90 |
| | | | | | | 14 | 53.90 |
| | | | | | | 15 | 52.90 |
| | | | | | | 16 | 51.90 |
| | | | | | | 17 | 50.90 |
| | | | | | | 18 | 49.90 |
| 19.20 | | | | | | 19 | 48.90 |
| End of Borehole<br><br>Practical DCPT refusal at 19.20m depth<br><br>(GWL @ 4.74m-October 14, 2016) | | | | | | | |




Monitoring Well Construction

**patersongroup** Consulting Engineers

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

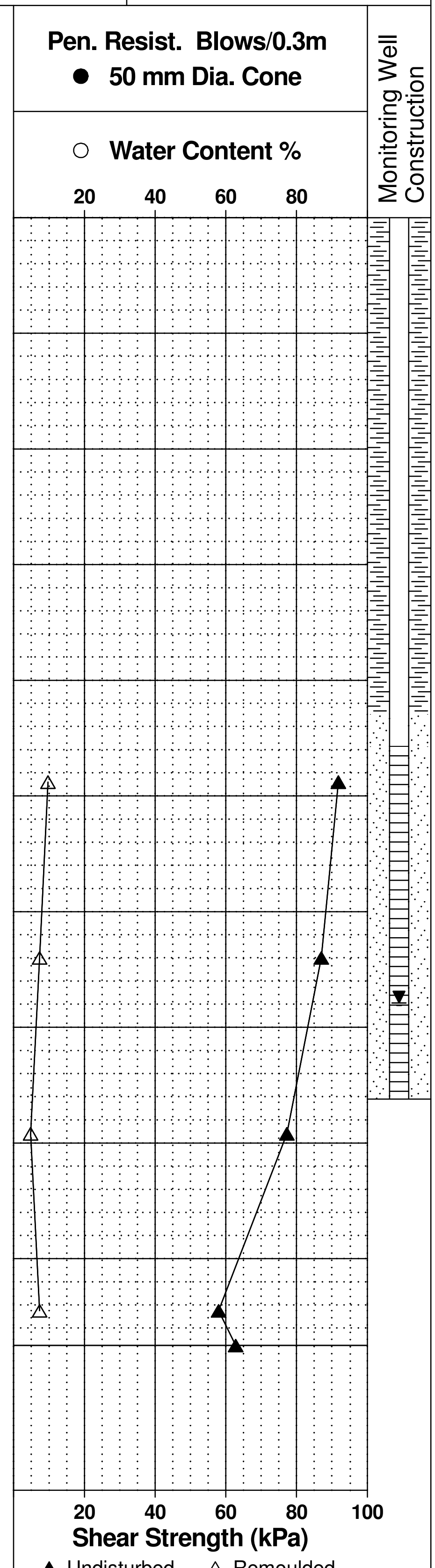
**FILE NO.** **PG3944**

**REMARKS**

**HOLE NO.** **BH 3**

**BORINGS BY** CME-55 Low Clearance Drill **DATE** October 7, 2016

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | 67.95 |
| | | AU | 1 | | | | |
| **FILL:** Brown silty clay with construction debris, some sand and gravel, trace organics | | SS | 2 | 4 | 2 | 1 | 66.95 |
| - styrofoam at 1.7m depth | | SS | 3 | 46 | 4 | 2 | 65.95 |
| 2.29 | | | | | | | |
| | | SS | 4 | 100 | 4 | 3 | 64.95 |
| | | SS | 5 | 100 | 2 | | |
| Stiff, brown **SILTY CLAY,** trace organics | | SS | 6 | 100 | 1 | 4 | 63.95 |
| - grey by 4.1m depth | | | | | | 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 |

# 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<br>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

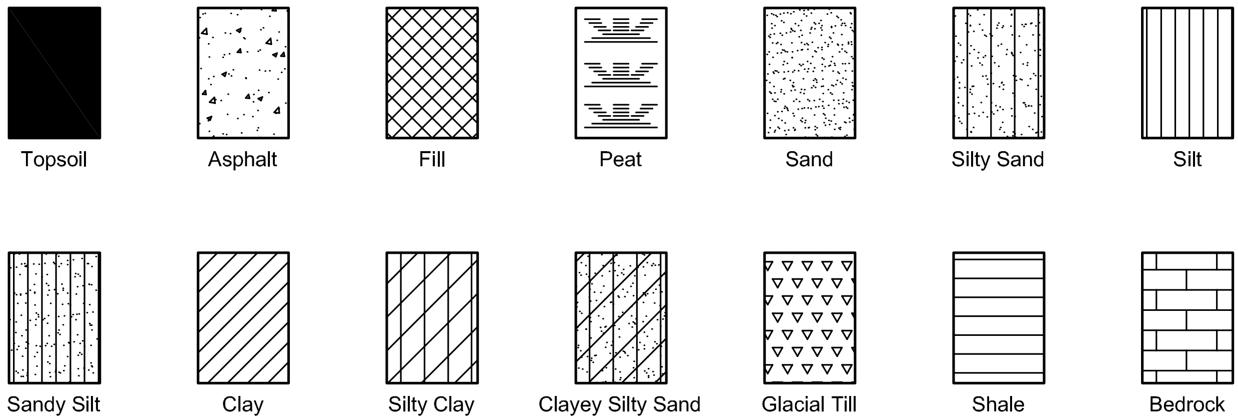
| | | |
|---|---|---|
| $p'_o$ | - | Present effective overburden pressure at sample depth |
| $p'_c$ | - | Preconsolidation pressure of (maximum past pressure on) sample |
| Ccr | - | Recompression index (in effect at pressures below $p'_c$) |
| Cc | - | Compression index (in effect at pressures above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

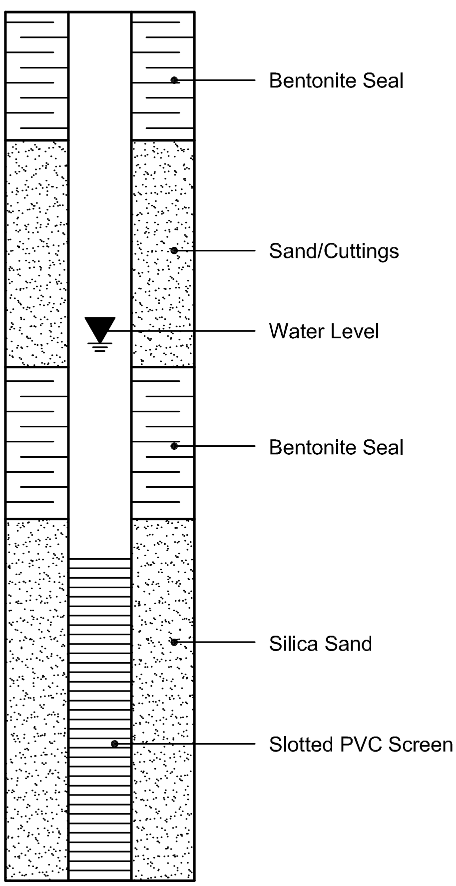
# SYMBOLS AND TERMS (continued)

## STRATA PLOT

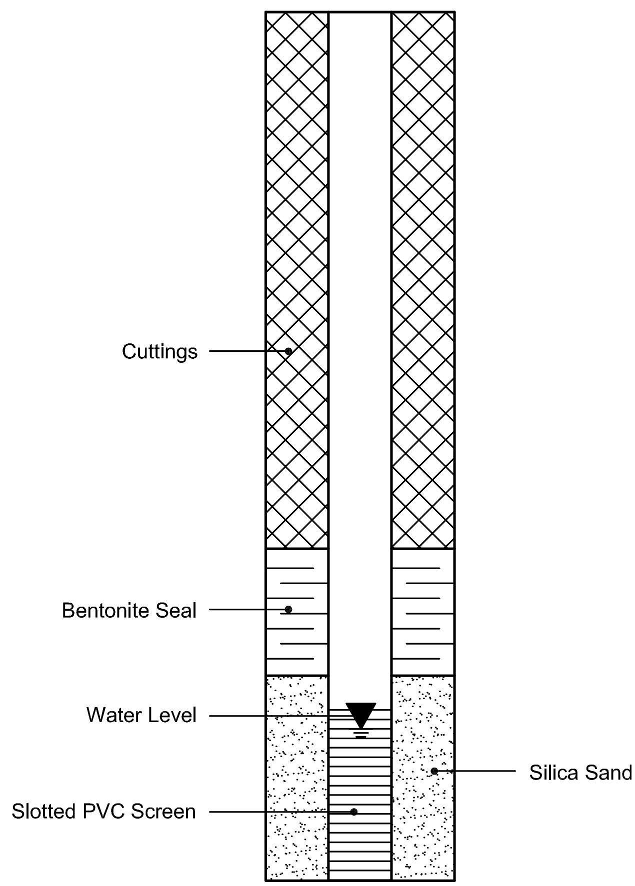



## MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION

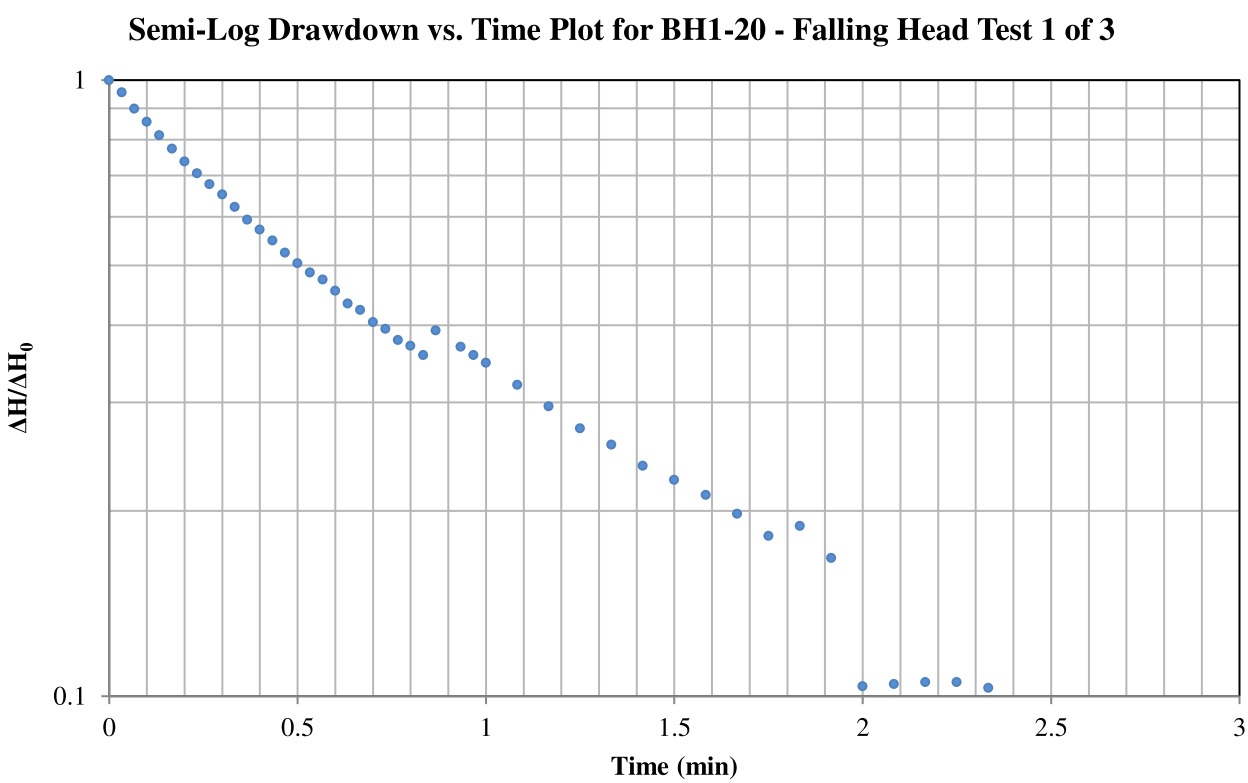



PIEZOMETER CONSTRUCTION



## 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 |

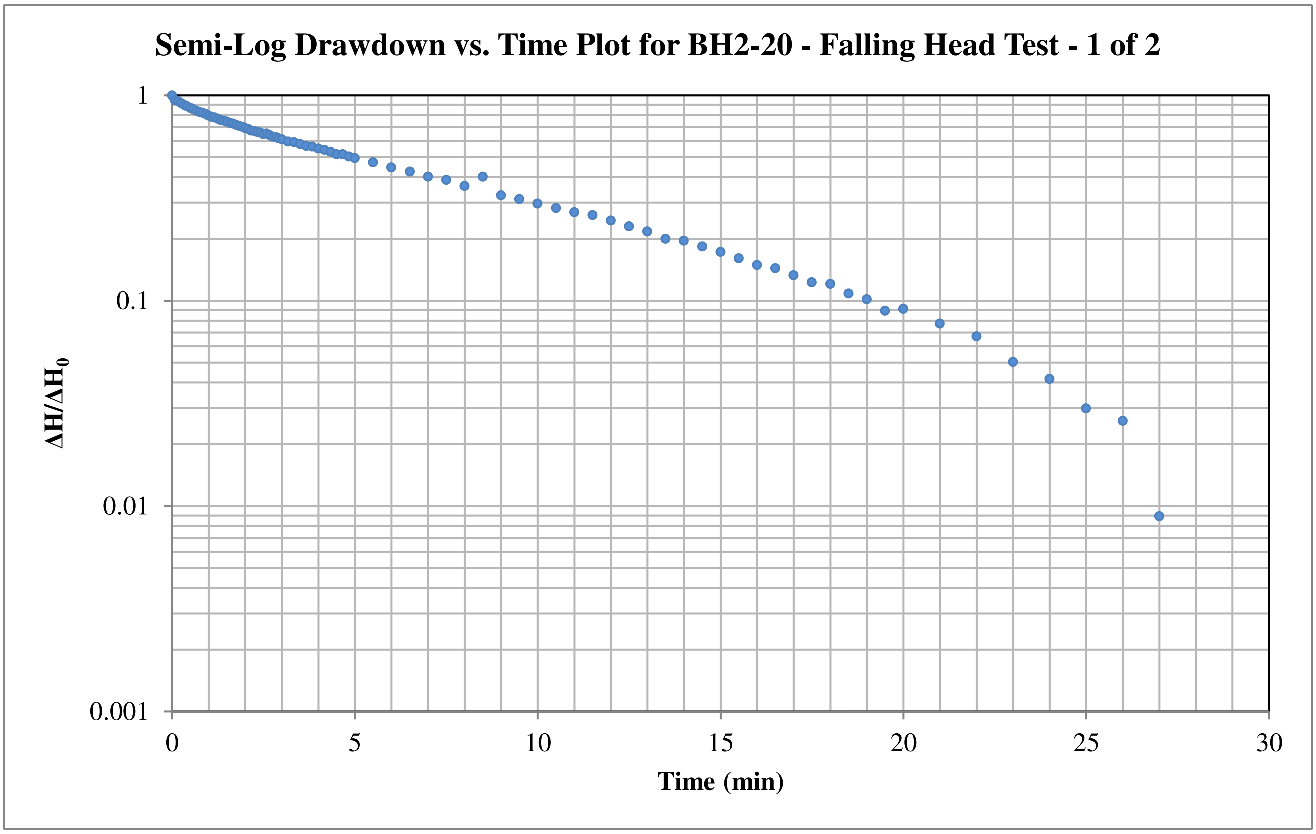
Data Points (from plot):

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**

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**

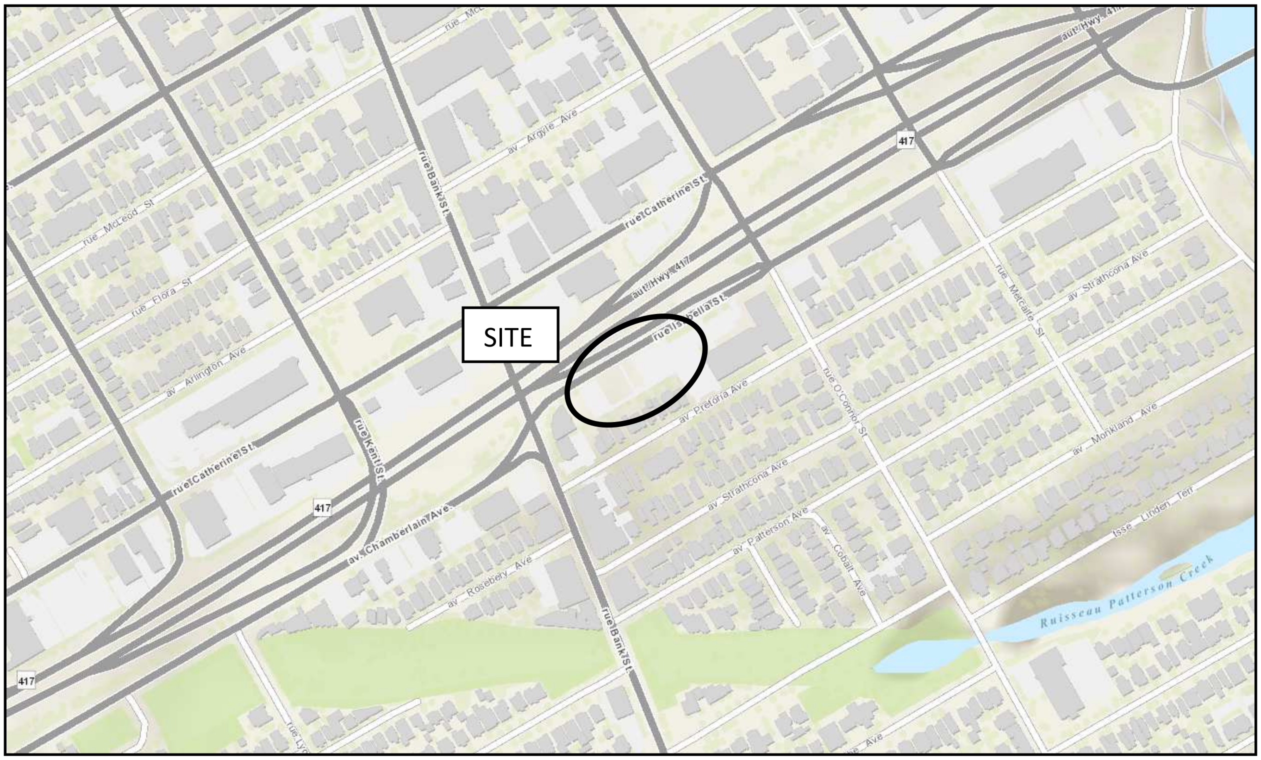
| | | | | | |
|---|---|---|---|---|---|
| | **Client ID:** | BH2-SS5 | - | - | - |
| | **Sample Date:** | 07-Oct-16 | - | - | - |
| | **Sample ID:** | 1642177-01 | - | - | - |
| | **MDL/Units** | Soil | - | - | - |
| **Physical Characteristics** | | | | | |
| % Solids | 0.1 % by Wt. | 59.2 | - | - | - |
| **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 | - | - | - |

# APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 & FIGURE 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5043-2 - TEST HOLE LOCATION PLAN




# FIGURE 1

## KEY PLAN

PATERSON GROUP

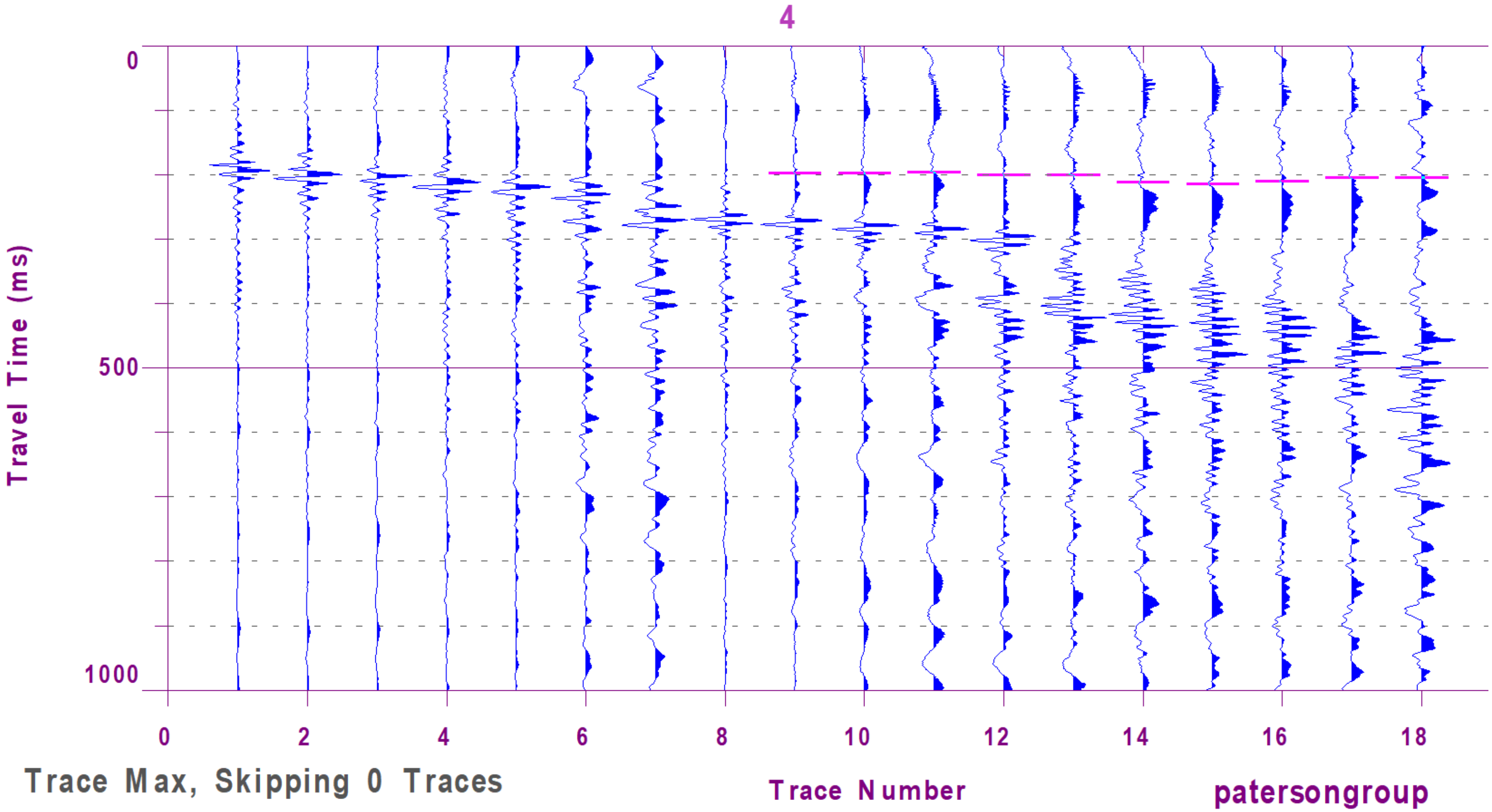



Figure 2 – Shear Wave Velocity Profile at Shot Location -20 m

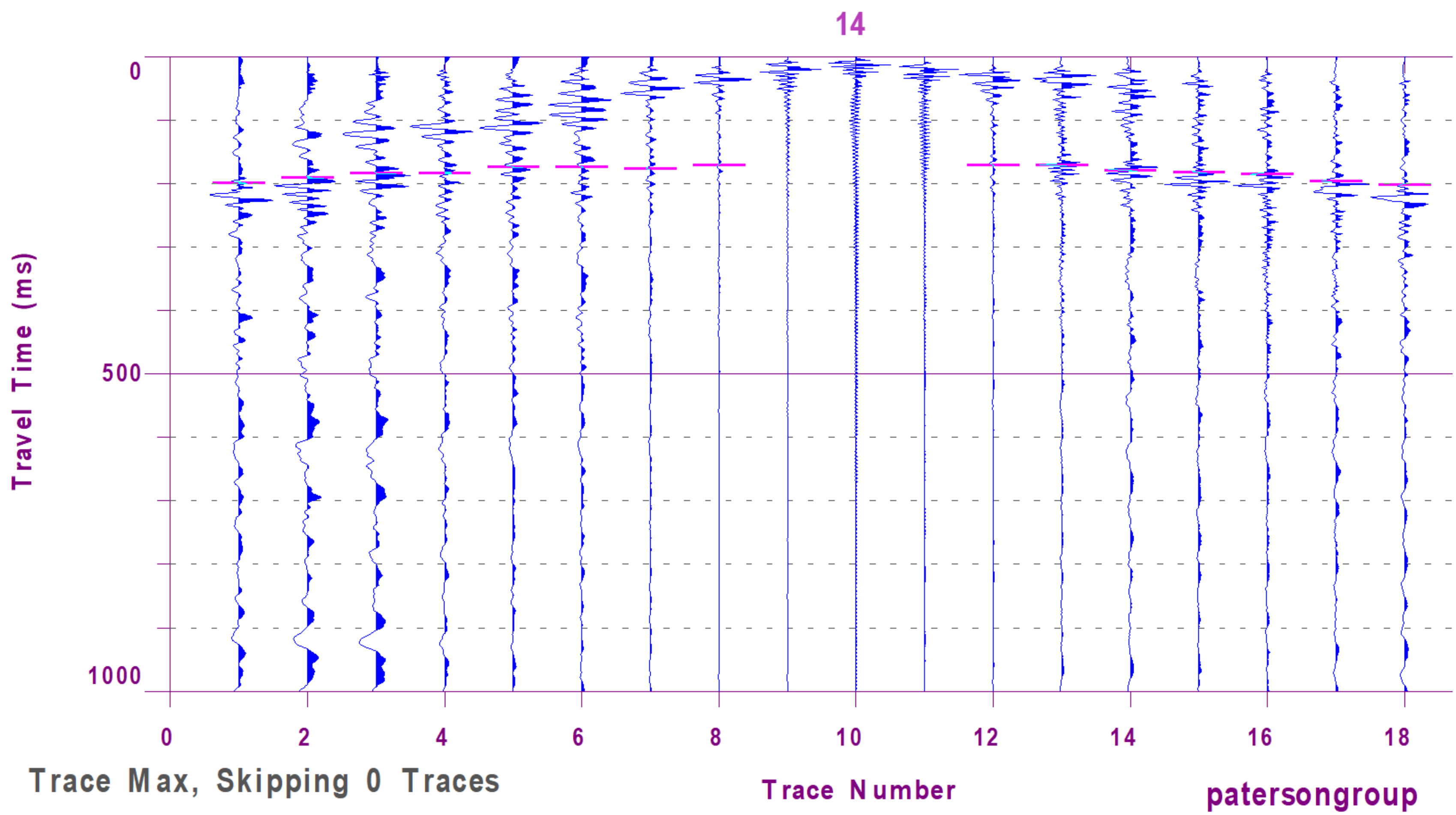
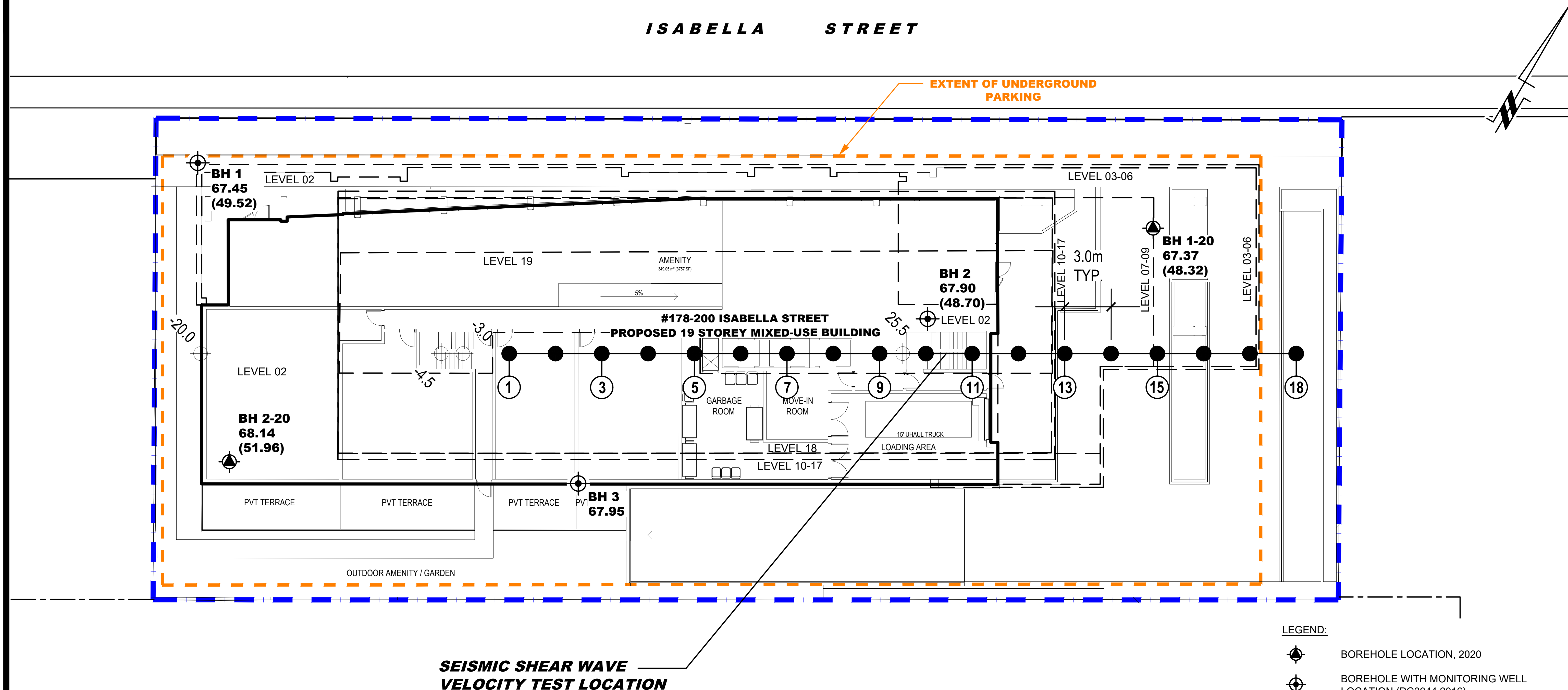



Figure 3 – Shear Wave Velocity Profile at Shot Location 25.5 m

PATERSON GROUP




LEGEND:

- GEOPHONE LOCATIONS
- 3 GEOPHONE NUMBER
- +10.0 SHOT LOCATION

LEGEND:

- BOREHOLE LOCATION, 2020
- BOREHOLE WITH MONITORING WELL LOCATION (PG3944,2016)
- 67.37 GROUND SURFACE ELEVATION (m)
- (48.32) PRACTICAL DCPT REFUSAL ELEVATION (m)

CONCEPTUAL PLAN PROVIDE BY PROJECT STIDIO 1.

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

SCALE: 1:250

0 1 2 3 4 5 10 15m

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

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 3 | AS PER CONCEPTUAL SITE PLAN | 28/11/2025 | FC |
| 2 | AS PER CONCEPTUAL SITE PLAN | 28/07/2023 | NP |
| 1 | SEISMIC SHEAR WAVE VELOCITY TEST ADDED TO PLAN | 02/01/2022 | FC |

ISATORIA PARTNERSHIP LIMITED

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.: 3 |

11x17

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