Geotechnical Investigation

Proposed Mixed-Use Development

265 Catherine Street
Ottawa, Ontario

Prepared for Brigil

Report PG5933-1 Revision 3 dated May 22, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Brigil to conduct a geotechnical investigation for the proposed mixed-use development, to be located at 265 Catherine Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ☐ Provide geotechnical recommendations pertaining to the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Based on the current concept drawings, it is understood that the proposed development will consist of two hi-rise mixed-use buildings, with shared underground levels, connected at and below the podium deck surface. It is understood that the high-rise buildings will be provided with two levels of underground parking whose footprints are anticipated to occupy the majority of the subject site. It is understood that a townhouse will be constructed over the podium deck slab.

Associated at grade access lanes, pedestrian pathways and landscaped areas are also anticipated. It is further anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Paterson previously conducted a series of field program for the subject site. The most recent investigation was carried out on August 19, 2020. At that time, 3 boreholes (BH 1 through BH 5) were advanced to a maximum depth of 14.7 m below the existing ground surface. Paterson had previously carried out an investigation on August 24 and 25, 2010. At that time, six (6) boreholes were advanced to a maximum sampling depth of 6 m and five (5) of the boreholes were extended to inferred bedrock based on practical refusal to augering. A previous investigation was carried out by others in August of 1971. At that time, five (5) boreholes were advanced to a maximum depth fo 12.5 m.

The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the test holes are shown on Drawing PG5933-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 2-20. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

Groundwater monitoring wells were installed in BH1-20, BH 2-20 and BH 3-20 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. A flexible polyethylene standpipe was installed within boreholes from the previous investigation to measure the stabilized groundwater levels subsequent to completion of the sampling program.

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The locations of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG5933-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for 1 month after this report is completed. They will then be discarded unless we are otherwise directed.

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is occupied by a bus terminal building with associated asphalt covered parking areas and access lanes. The subject site is approximately at grade with surrounding streets.

The site is bordered by Catherine Street to the south, Lyon Street to the west, Arlington Avenue and further by residential dwellings to the north and Kent Street to the east. The existing ground surface across the subject site is relatively flat and at grade with adjacent properties.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of concrete or asphaltic concrete underlain by fill extending to an approximate depth of 0.6 to 2.3 m below the existing ground surface. The fill was generally observed to consist of a compact brown silty sand with crushed stone and occasional brick, metal, and plastic fragments.

A native silty sand layer and/or silty clay deposit was encountered underlying the fill. The silty clay deposit was observed to consist of a very stiff to stiff, brown silty clay, becoming a stiff grey silty clay below an approximate depth ranging between 3.0 to 7.6 m below the existing ground surface.

Underlying the silty clay deposit below approximate depths ranging between 4.4 to 9.7 m, a glacial till layer was encountered. The glacial till deposit was observed to consist of a grey sandy silt, clayey silt or silty clay with gravel, cobbles and boulders.

Practical refusal to augering or the DCPT was encountered at depths ranging from 7.4 to 11.7 m below the existing ground surface.

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

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and shale of the Verulam Formation and shale of the Billings Formation at depths ranging from 10 to 15 m.

4.3 Groundwater

The observed groundwater levels are summarized in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1-20*	68.62	4.60	64.02	September 1, 2020
BH 2-20*	68.46	Dry	-	September 1, 2020
BH 3-20*	68.11	4.26	63.85	September 1, 2020
BH 1	-	3.48	-	September 16, 2010
BH 1	-	5.32	-	September 16, 2010
BH 3	-	5.30	-	September 16, 2010
BH 4	-	N/A	-	September 16, 2010
BH 5	-	4.59	-	September 16, 2010
BH 6	-	2.18	-	September 16, 2010

Note: Ground surface elevations at borehole locations were surveyed by Paterson and are referenced to a geodetic datum.

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Based on our field observations, experience with the local area and the colouring of the recovered samples, it is expected that the long-term groundwater level is between 4 to 5 m below the existing ground surface within the silty clay layer. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. Based on the subsurface conditions encountered in the test holes and the anticipated building depth and loads, it is recommended that the building foundation be comprised of conventional footings placed over an approved bedrock bearing surface or a raft foundation placed over an undisturbed glacial till or approved bedrock bearing surface.

Alternately, to avoid excavating the entire building footprint to the bedrock level, footings could be placed over lean concrete infilled trenches. Near vertical, zero entry trench extending at least 300 mm beyond the footing face should be excavated to a clean bedrock surface approved by the geotechnical consultant. The trenches should be infilled by a minimum of 15 MPa lean concrete to the underside of the footing.

Due to the permeable water bearing glacial till layer at depth, special considerations should be taken for construction excavation and dewatering to avoid excess groundwater pumping and affecting neighbouring properties.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas and other settlement sensitive structures. Existing construction debris should be entirely removed from within the perimeter of all buildings.

Protection of Subgrade (Raft 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 glacial till subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of 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 or glacial till to potential disturbance due to drying.

Bedrock Removal

Where the bedrock is weathered and/or only small quantities of bedrock need to be removed, hoe ramming is an option of bedrock removal. Where large quantities of bedrock need to be removed, line drilling and controlled blasting may be required.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

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

Vibration Considerations

Construction operations could be 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 a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause or the source of detrimental vibrations at the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the permissible vibrations, 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). These guidelines are for current construction standards.

These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the proposed building footprint, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified 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.

Site excavated soils are not suitable for use as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

Bearing Surface Preparation

The excavation is expected to be completed below the groundwater table. Where the bearing surface will consist of glacial till, measures to protect against heaving and ground disturbance should be put in place. Accordingly, it is recommended that the entirety of each building footprint be excavated to the underside of footing elevation, and then covered with a 150 mm thick mud slab to protect the glacial till from disturbance.

Furthermore, groundwater pumping using dry wells with sump pumps which are located centrally within the excavation will be required to control the influx of water during construction. Details can be provided once the groundwater influx is better assessed during the excavation process.

Lean Concrete In-Filled Trenches

Where footings are designed to be supported on bedrock, and the bedrock is not encountered at the design underside of footing elevation, consideration should be given to excavating zero-entry vertical trenches to expose the underlying bedrock surface and then backfilling with lean concrete (15 MPa 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation.

The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. The excavation bottom should be relatively clean using the hydraulic shovel only (no worker entry). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2000 kPa**. This is discussed further below.

5.3 Foundation Design

Spread Footing Foundations

Conventional spread and pad footings placed on the upper levels of the fractured bedrock a clean, surface sounded sandstone bedrock bearing surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa**, incorporating a geotechnical resistance factor of 0.5. Alternately, footings can be placed over concrete in-filled (minimum 15 MPa) zero entry, near vertical trenches extended to a surface sounded bedrock bearing surface using the same bearing resistance values. The concrete in-filled trenches should extend a minimum 300 mm beyond the footing faces in all directions.

A factored bearing resistance value at ULS of **7,000 kPa,** incorporating a geotechnical resistance factor of 0.5, can be used for footings founded on clean, surface sounded bedrock and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footing footprint(s). One drill hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.
Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Raft Foundation

Alternately, a raft foundation can be constructed to support the high-rise portion of the proposed building consist of a raft foundation bearing on undisturbed glacial till or bedrock.

For 3 levels of underground parking, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between 10 to 11 m. The contact pressure provided considers the stress relief associated with the soil removal required for 3 levels of underground parking.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

For 2 levels of underground parking at a founding elevation of approximately 60.0 m, a bearing resistance value at SLS (contact pressure) of **430 kPa** will be considered acceptable for a raft supported on the undisturbed glacial till or sound bedrock. It should be noted that the weight of the raft slab and everything above must be included when designing with this value. The factored bearing resistance (contact pressure) at ULS can be taken as **650 kPa**. For this case, the modulus of subgrade reaction was calculated to be **17.0 MPa/m** for a contact pressure of **430 kPa**.

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

Settlement

The total and differential settlement will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 to 20 mm, respectively.

Footings placed completely over an acceptable bedrock bearing surface will be subjected to negligible post construction total and differential settlements.

Bedrock/Soil Transition

If the raft slab is constructed in the areas underlain by bedrock, it is recommended that a minimum 500 mm thick layer (native soil and or crushed stone layer) be present between the raft slab and the bedrock surface to reduce the risks of bending stresses developing in the concrete slab. The rock should be broken down a minimum of 500 mm and backfill using Granular B Type II crushed stone compacted to 98% of the material’s SPMDD. The bending stress could lead to cracking of the concrete slab. This requirement could be waived in areas where the bedrock surface is relatively flat within the footprint of the building. This recommendation does not refer to potential concrete shrinkage cracking which should be controlled in the usual manner.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Adequate lateral support is provided to a silty clay or glacial till bearing medium 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 of the same or higher capacity as the soil.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

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

Field Program

The seismic array testing location was placed as presented on drawing PG5933-1-Test Hole Location Plan, attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

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

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 15.0, 3.0 and 2.0 m away from the first and last geophone, and at the centre of the 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 buildings. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records 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 **124 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 10 m.

Site Class for Buildings Founded Directly or Indirectly on Bedrock

If the building foundation consists of footings founded on the bedrock surface or lean concrete filled trenches extended to the underlying bedrock surface, the Vs_{30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

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

$$V_{s30=}\ 2{,}045\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed buildings is **2,045 m/s** provided the totality of the footings are placed directly on the bedrock surface or zero entry lean concrete filled trenches are extended to the bedrock surface.

Therefore, for the previously mentioned foundation, a **Site Class A** is applicable for design as per Table 4.1.8.4.A of the OBC 2012. However due to bedrock variation on site and currently proposed elevations it is expected that the foundation will be approximately 3.0 m above bedrock. See below for seismic site classification for 2 levels of underground parking levels.

The soils underlying the subject site are not susceptible to liquefaction.

Site Class for Buildings Founded on Overburden and Within 3 m of Bedrock

If the building foundation consists of conventional footings founded on soil, or a raft foundation located on soil, and bedrock is anticipated to be located within a maximum depth of 3 m of the founding depth or at an approximate elevation of 60.0 m, the Vs_{30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{3\ m}{124\ m/s} + \frac{27\ m}{2{,}045\ m/s}\right)}$$

$$V_{s30=}\ 802\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed buildings with foundation located within 3 m of the bedrock surface is **802 m/s**.

Therefore, a **Site Class B** is applicable for design of buildings if footings will be founded upon a soil bearing surface within 3 m of the bedrock, or a raft foundation located within 3 m of the bedrock surface, and as per Table 4.1.8.4.A of the OBC 2012.

The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

Multi-Storey Buildings

The basement areas for the proposed project will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. The upper 200 mm sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade construction.

All backfill material within the footprint of the proposed building(s) should be placed in a maximum of 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building(s) should be placed in a maximum of 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions encountered at the time of the current and previous fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor (discussed in Subsection 6.1).

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

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

Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters 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 material(0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).
The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:
a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for Carleton Place is 0.23 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$
The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Design

Rigid Pavement Structures

For design purposes, it is recommended that the rigid pavement structure for the lower underground parking level of the proposed building consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 2 below. The flexible pavement structure presented in Table 3 and Table 4 should be used for driveways and car only parking areas and at grade access lanes and heavy loading parking areas.

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 2 – Recommended Rigid Pavement Structure – Lower Parking Level

Thickness (mm)	Material Description
125	**Exposure Class C2 – 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** – OPSS Granular A Crushed Stone
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

Table 3 - Recommended Pavement Structure - Car Only Parking Areas

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

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

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

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

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

Consideration should also be given to installing subdrains during the pavement construction as per City of Ottawa standards. These drains should extend in four orthogonal directions or longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines, or the pipe should be wrapped with suitable filter cloth. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines. Discharge of the subdrains should be directed by gravity to storm sewers or deeper drainage ditches.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

For the proposed underground parking levels of the high-rise buildings, it is expected that the building's foundation walls will be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation walls be blind poured against a drainage system and waterproofing system fastened to a near watertight shoring system.

Waterproofing of the foundation walls is recommended, and the waterproofing membrane is to be installed from 300 mm above the proposed P1 level to the bottom of foundation.

It is also recommended that a composite drainage system, such as Delta Drain 6000 or equivalent, be installed over the waterproofing membrane and extend from the exterior finished grade to the founding elevation (underside of footing or raft slab). The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the perimeter footing or raft slab interface to allow the infiltration of water to flow to an interior perimeter underfloor drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

If a permeable shoring system is considered a tanked raft foundation should be considered for final design.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration below the lowest level floor slabs. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximately 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Subfloor Water Infiltration

Due to the variability in the limestone, it is expected that water might infiltrated through seems and cracks. Paterson should review the water infiltration. It is recommended to carry a minimum 75 mm mubslab and horizontal membrane to act has hydraulic barrier on top of the bedrock.

Elevator Pit Waterproofing

The horizontally applied Colphene BSW H waterproofing membrane (or approved other) should be placed on an adequately prepared mud slab and extend vertically within the inside of the temporary forms of the elevator raft slab. Once the concrete raft slab and elevator shaft sidewalls are poured in place, it is recommended that a waterproofing membrane, such as Colphene Torch'N Stick (or approved other) should be applied to the exterior of the elevator pit sidewalls. The Colphene Torch'N Stick waterproofing membrane should extend over the vertical portion of the previously applied Colphene BSW H waterproofing membrane installed on the concrete raft slab in accordance with the manufacturer's specifications. As a secondary defense, a continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the concrete raft slab below the elevator pit sidewalls.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the elevator pit and bedrock excavation face should be in-filled with lean concrete, OPSS Granular B Type 2 or Granular A crushed stone. Refer to Figure 3 – Waterproofing System for Elevator, for specific details of the elevator waterproofing in Appendix 2.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials, such as clean sand or OPSS Granular A granular material. 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.

Adverse Effects from Dewatering on Adjacent Structures

Since the excavation is expected to extend in a water bearing clayey till, construction dewatering is not recommended at depths greater than 5 to 6 m. The excavation should consider the use of a nearly waterproofed shoring system. It is estimated that groundwater lowering will affect the residential neighborhood to the north if more than 400,000 L/day is pumped during the excavation process. The use of a secant or diaphragm wall socketed a minimum of 1.5 m in bedrock will lower the groundwater infiltration into the excavation to controllable and acceptable levels.

The temporary dewatering of the bedrock during the excavation and construction stage will not be susceptible to significant consolidation.

Implementation of dual use shoring system recommended above is expected to limit the drawdown of the local groundwater table over the long term and in a limited area. Therefore, in our opinion, no adverse effects to nearby structures and infrastructure are expected over the long term if a watertight shoring is used for construction.

6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as isolated 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 an equivalent combination of soil cover and foundation insulation.

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

6.3 Excavation Side Slopes

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

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.
Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring will 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 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 shoring system is recommended to consist of secant pile walls or pile and sheet pile system such as a combi-wall which could be cantilevered. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

It is recommended to use a watertight shoring system to reduce water infiltration into the excavation and building and prevent dewatering of the surrounding areas. A waterproof shoring system will also ensure the stability of the soil at the back of the wall and prevent washouts caused by high water infiltration.

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 – Soils Parameter for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_O)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included in the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated to 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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, should be placed from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

Generally, it should be possible to re-use the moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay material will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material’s SPMDD.

All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

Clay Seals

Where silty clay is encountered, to reduce long-term lowering of the groundwater level at this 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, sub-bedding and cover material.

The barriers should consist of relatively dry and compactable brown silty clay 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 and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR).

A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Persons 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

Long-term groundwater control will be required for the subject site to prevent dewatering of the surrounding areas. Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1.

Any groundwater which breaches the building's perimeter groundwater infiltration control system will be directed to the 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 long-term groundwater flow will be very low to negligible (ie.- less than 30,000 L/day).

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means.

In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of n aggressive to very aggressive corrosive environment.

6.8 Hydraulic Conductivity and Groundwater Infiltration

Hydraulic conductivity testing was completed at all boreholes outfitted with monitoring wells screened within the overburden material and below the bedrock surface. Falling head tests (“slug testing”) were completed in accordance with ASTM Standard Test Method D4404 - Field Procedure for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers.

Following the completion of the slug testing, the test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter.

The assumption regarding aquifer storage is considered to be appropriate for groundwater flow through the overburden aquifer. The assumption regarding screen length and well diameter is considered to be met based on the screen lengths of 1.5 m and well diameter of 0.0508 m.

While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site.

The Hvorslev analysis is based on the line of best fit through the field data (hydraulic head recovery vs. time), plotted on a semi-logarithmic scale. In cases where the initial hydraulic head displacement is known with relative certainty, such as in this case where a physical slug has been introduced/removed, the line of best fit is considered to pass through the origin.

Results

Based on testing at the subject site, the hydraulic conductivity values for the glacial till varies from $2.02\text{x}10^{-6}$ to $2.32\text{x}10^{-6}$ m/s. Based on testing at the subject site, the hydraulic conductivity values for the bedrock varies from $3.68\text{x}10^{-7}$ to $2.01\text{x}10^{-5}$ m/s. The results from the hydraulic conductivity testing have been included in Appendix 1. An estimate on water infiltration can be made once more detail drawings are available.

Estimated Groundwater Infiltration during Construction

The dewatering and infiltration quantity estimated below are based on the current information for the proposed development. Based on available plans at the time of writing it is expected that the towers will be constructed over 2 levels of underground parking.

Based on the hydraulic conductivity testing results of the overburden and bedrock material, a conservative unfactored steady state volume of groundwater is anticipated to be approximately between 700,000 L/day to >1,000,000 L/day if the entire proposed excavation does not extend below a depth of 12 m below the existing ground surface.

Note that excavation in bedrock can lead to highly variable conditions in the case an open fracture is encountered. The contractor should be ready to seal open factures to limit the influx of water.

It should be noted that the calculated infiltration rates do not account for the initial groundwater inflow into the excavation resulting from perched water conditions. The estimate is provided for a fully open excavation. A factor of safety should be applied to the calculated infiltration rates to account for perched conditions, variability in the overburden material and the quality of bedrock, levels of hydrostatic pressure in the bedrock, and any unforeseen circumstances that may arise during construction activities.

It should also be noted that an additional 150,000 L/day of surface water infiltration can be expected during a 5yr-1hr duration precipitation event based on the proposed building footprint.

7.0 Recommendations

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

- Review of the as built grading plan, from a geotechnical perspective.
- Review of the contractor's design of the temporary shoring system.
- Review and inspection of the foundation waterproofing system and all foundation drainage systems.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by Paterson.

8.0 Statement of Limitations

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

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Brigil, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 22, 2024
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Nicolas Seguin

Nicolas Seguin, EIT, CPI

Joey R. Villeneuve, M.A.Sc, P.Eng.

Report Distribution:

☐ Brigil (email copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY TESTING

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
265 Catherine Street
Ottawa, Ontario

EASTING: 367830.466 **NORTHING:** 5030202.891 **ELEVATION:** 68.37

DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** February 28, 2024

FILE NO. PG5933

HOLE NO. BH 1-24

SAMPLE DESCRIPTION	Depth to base (m)
Ground Surface	
ASPHALT with brown silty sand	0.08
FILL: Granular crushed stone	0.30
FILL: Brown silty sand, trace gravel	1.52
FILL: Brown silty sand with gravel	1.98
Stiff, grey **SILTY CLAY**	8.31
GLACIAL TILL: Loose to compact, grey silty sand with gravel, trace clay - with cobbles and boulders by 10.7m depth	11.10

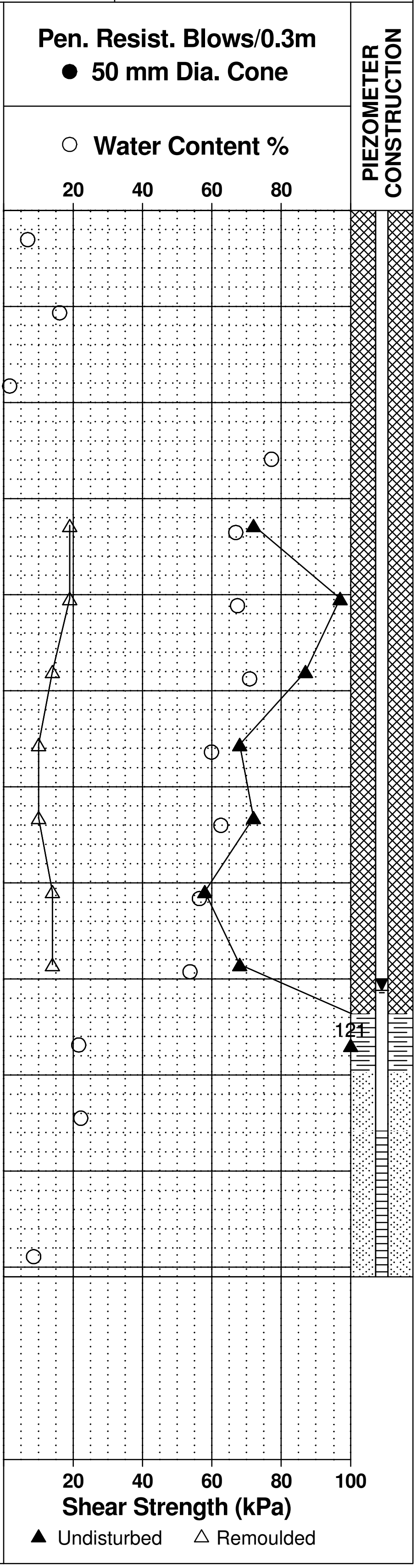
SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	12
SS	3	17	30
SS	4	100	<1
SS	5	100	P
SS	6	92	P
SS	7	75	P
SS	8	100	P
SS	9	100	P
SS	10	100	P
SS	11	100	P
SS	12	83	5
SS	13	75	2
SS	14	0	13
SS	15		+50

DEPTH (m)	ELEV. (m)
0	68.37
1	67.37
2	66.37
3	65.37
4	64.37
5	63.37
6	62.37
7	61.37
8	60.37
9	59.37
10	58.37
11	57.37

End of Borehole

Practical refusal to augering at 11.10m depth

(GWL @ 8.12m - Mar. 14, 2024)



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
265 Catherine Street
Ottawa, Ontario

EASTING: 367780.982 **NORTHING:** 5030179.343 **ELEVATION:** 68.27
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** February 28, 2024

FILE NO. PG5933
HOLE NO. BH 2-24

SAMPLE DESCRIPTION	Depth (m)
Ground Surface	
ASPHALT	0.05
FILL: Granular crushed stone	0.30
FILL: Brown silty sand with gravel - trace brick and ash from 0.8m to 1.4m depth	2.59
Stiff, grey **SILTY CLAY**	6.63
Loose to compact, grey **SILTY SAND** to **SANDY SILT**, trace clay - trace gravel by 7.6m depth	8.31
GLACIAL TILL: Compact, grey silty sand with gravel, trace to some clay, occasional cobbles - boulders and shale fragments by 9.1m depth - very dense by 9.4m depth	9.60

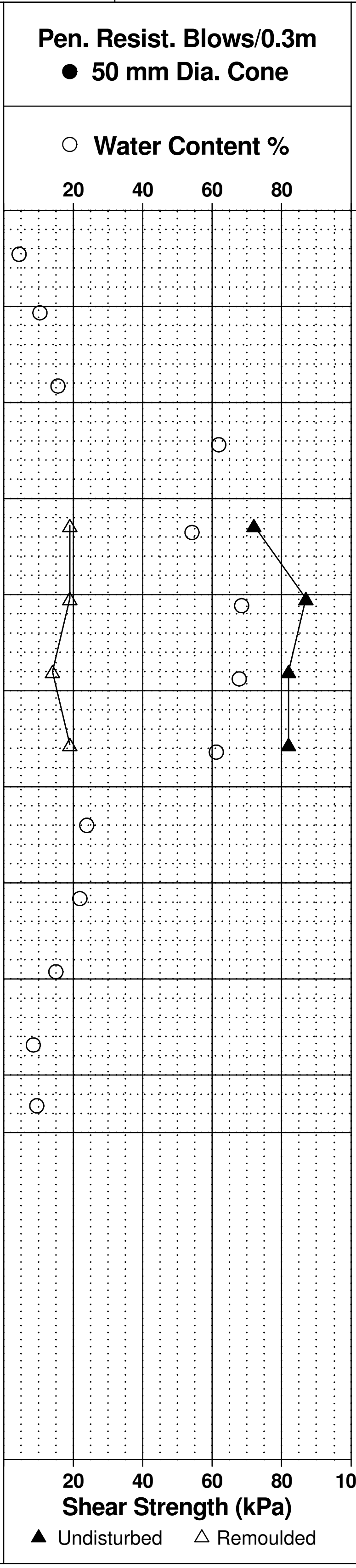
End of Borehole

Practical refusal to augering at 9.60m depth

(Borehole Blocked - Mar. 14, 2024)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	10
SS	3	50	27
SS	4	83	8
SS	5	75	P
SS	6	83	P
SS	7	83	P
SS	8	58	P
SS	9	17	P
SS	10	92	9
SS	11	13	6
SS	12	67	19
SS	13	100	+50

DEPTH (m)	ELEV. (m)
0	68.27
1	67.27
2	66.27
3	65.27
4	64.27
5	63.27
6	62.27
7	61.27
8	60.27
9	59.27



PIEZOMETER CONSTRUCTION

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
265 Catherine Street
Ottawa, Ontario

EASTING: 367779.458 **NORTHING:** 5030141.587 **ELEVATION:** 68.32

DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** February 28, 2024

FILE NO. PG5933

HOLE NO. BH 3-24

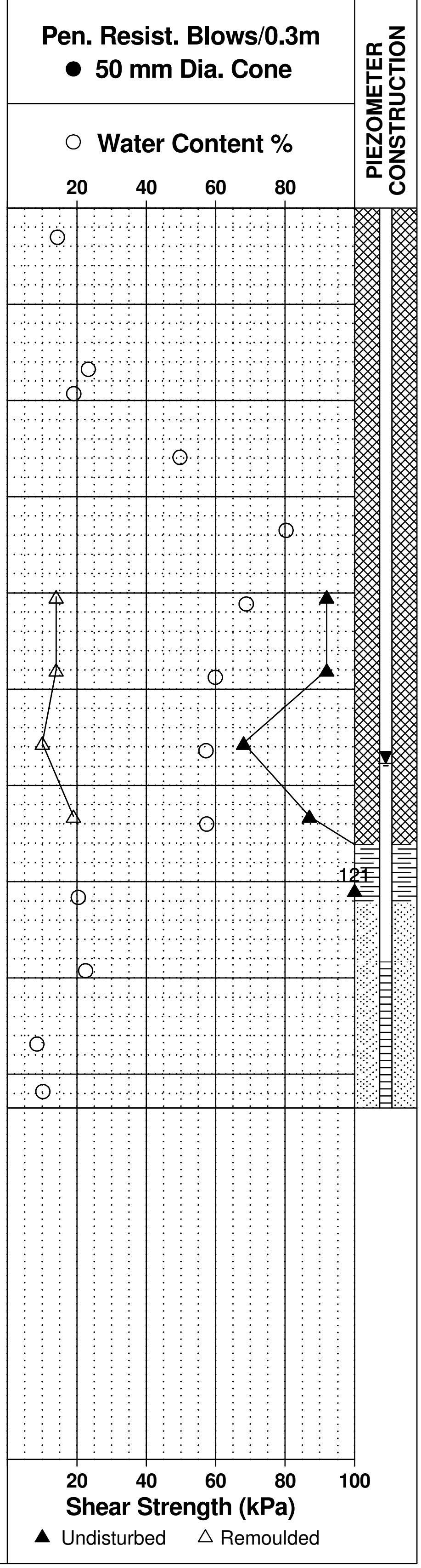
SAMPLE DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
Ground Surface					
ASPHALT	0.05				
FILL: Brown silty sand with gravel		AU	1		
- trace brick and wood by 0.8m depth		SS	2	62	+50
	1.98	SS	3	83	11
- organics by 2.0m depth Loose, grey **SILTY SAND**		SS	4	75	5
	2.97				
Stiff, grey **SILTY CLAY**		SS	5	100	2
		SS	6	92	P
		SS	7	100	P
		SS	8	92	P
		SS	9	100	P
	6.78				
GLACIAL TILL: Loose to compact, grey silty sand with gravel, trace clay, occasional cobbles		SS	10	83	10
		SS	11	83	2
- shale fragments by 8.4m depth		SS	12	42	18
	9.35	SS	13	100	+50

End of Borehole

Practical refusal to augering at 9.35m depth

(GWL @ 5.77m - Mar. 14, 2024)

DEPTH (m)	ELEV. (m)
0	68.32
1	67.32
2	66.32
3	65.32
4	64.32
5	63.32
6	62.32
7	61.32
8	60.32
9	59.32



patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
265 Catherine Street
Ottawa, Ontario

EASTING: 367838.673 **NORTHING:** 5030168.358 **ELEVATION:** 68.29
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** February 29, 2024

FILE NO. **PG5933**
HOLE NO. **BH 4-24**

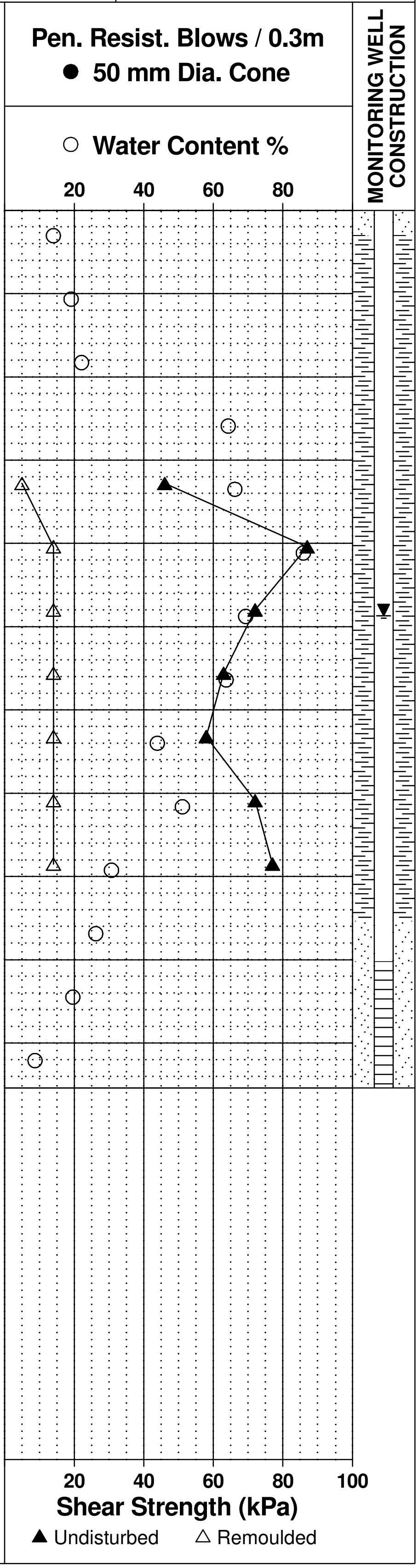
SAMPLE DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with gravel 0.53	AU	1		
FILL: Brown silty sand, trace gravel 1.45	SS	2	67	8
Compact, brown **SAND**, trace silt 2.21	SS	3	67	14
Firm to stiff, grey **SILTY CLAY**	SS	4	100	3
	SS	5	100	P
	SS	6	100	P
	SS	7	100	P
	SS	8	83	P
	SS	9		
	SS	10	100	P
- some sand by 7.6m depth 8.31	SS	11	50	P
Loose, grey **SILTY SAND** to **SANDY SILT**, trace clay 9.07	SS	12	58	3
GLACIAL TILL: Compact, grey silty sand with gravel and clay, occasional cobbles - some shale fragments by 9.9m depth 10.54	SS	13	75	17
	SS	14	58	37

End of Borehole

Practical refusal to augering at 10.54m depth

(GWL @ 4.87m - Mar. 14, 2024)

DEPTH (m)	ELEV. (m)
0	68.29
1	67.29
2	66.29
3	65.29
4	64.29
5	63.29
6	62.29
7	61.29
8	60.29
9	59.29
10	58.29



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
265 Catherine Street
Ottawa, Ontario

EASTING: 367879.574 **NORTHING:** 5030192.335 **ELEVATION:** 68.54
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** February 29, 2024

FILE NO. **PG5933**
HOLE NO. **BH 5-24**

SAMPLE DESCRIPTION

Ground Surface

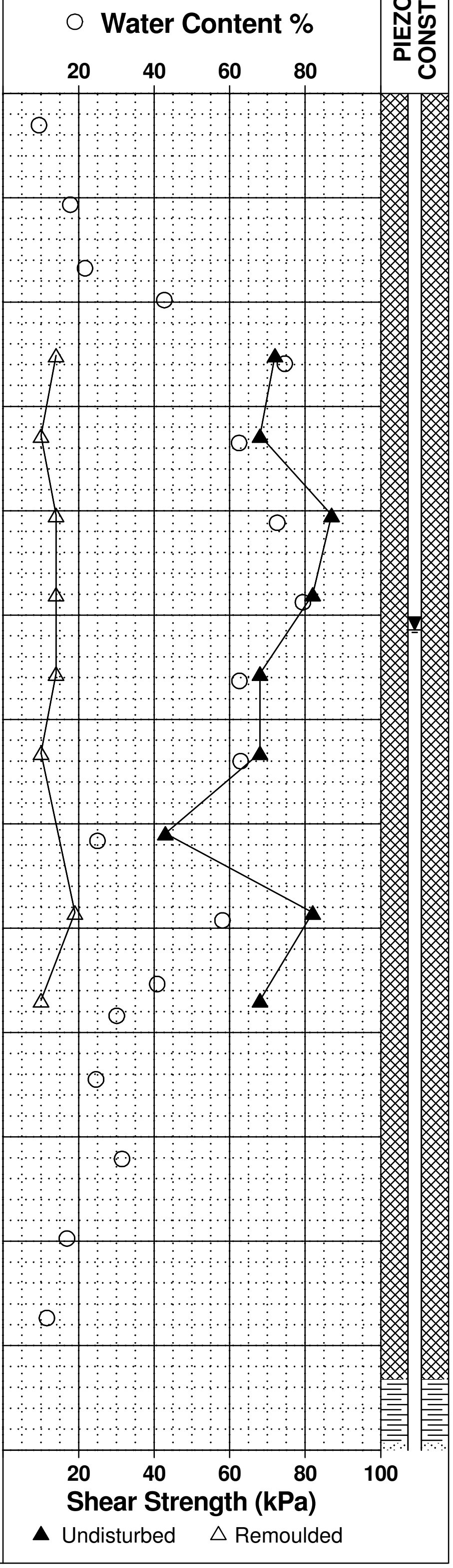
SAMPLE DESCRIPTION	Depth to base (m)
FILL: Brown silty sand with gravel	1.45
Loose, brown **SILTY SAND**	1.83
Stiff, brown **SILTY CLAY**	2.21
Stiff to firm, grey **SILTY CLAY**	8.69
Loose to compact, grey **SILTY SAND** to **SANDY SILT**, trace clay	10.59
GLACIAL TILL: Dense to very dense, grey silty sand with gravel, occasional cobbles and boulders - shale fragments by 11.4m depth	11.84
BEDROCK: Good to excellent quality, grey limestone interbedded with black shale	

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	6
SS	3	50	4
SS	4	83	P
SS	5	100	P
SS	6	75	P
SS	7	83	P
SS	8	100	P
SS	9	83	P
SS	10	83	P
SS	11	100	P
SS	12	58	P
SS	13	67	6
SS	14	67	6
SS	15	50	29
SS	16	86	+50
RC	1	100	78
RC	2	100	85

DEPTH (m)	ELEV. (m)
0	68.54
1	67.54
2	66.54
3	65.54
4	64.54
5	63.54
6	62.54
7	61.54
8	60.54
9	59.54
10	58.54
11	57.54
12	56.54
13	55.54

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



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
265 Catherine Street
Ottawa, Ontario

EASTING: 367879.574 **NORTHING:** 5030192.335 **ELEVATION:** 68.54

DATUM: Geodetic

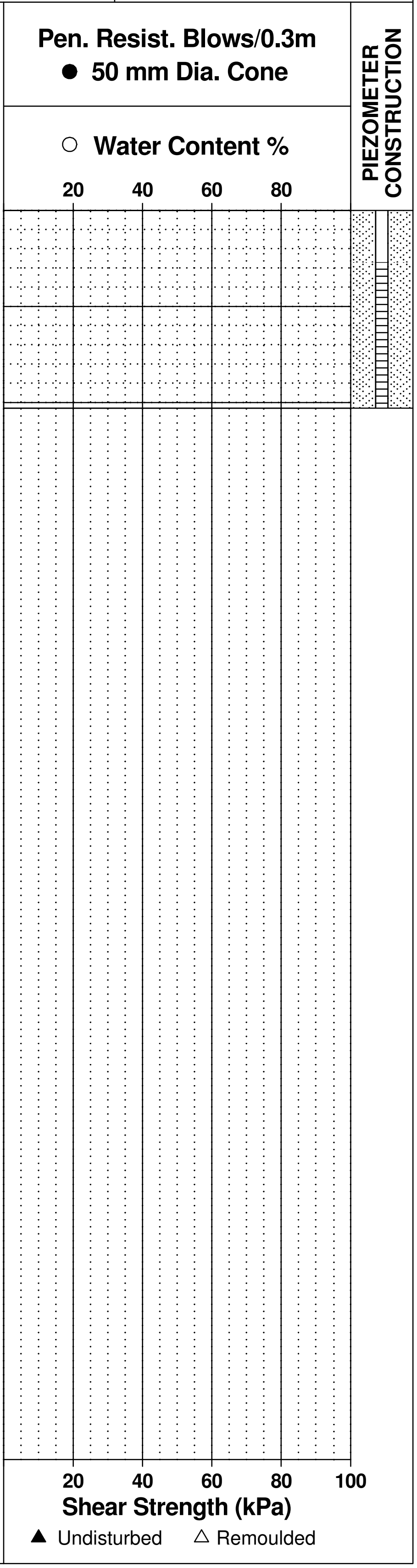
REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** February 29, 2024

FILE NO. PG5933

HOLE NO. BH 5-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
Ground Surface							
BEDROCK: Good to excellent quality, grey limestone interbedded with black shale						13	55.54
		RC	3	100	100	14	54.54
15.06						15	53.54
End of Borehole (GWL @ 5.14m - Mar. 14, 2024)							



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
265 Catherine Street
Ottawa, Ontario

EASTING: 367748.417 **NORTHING:** 5030139.276 **ELEVATION:** 68.12

DATUM: Geodetic

REMARKS:

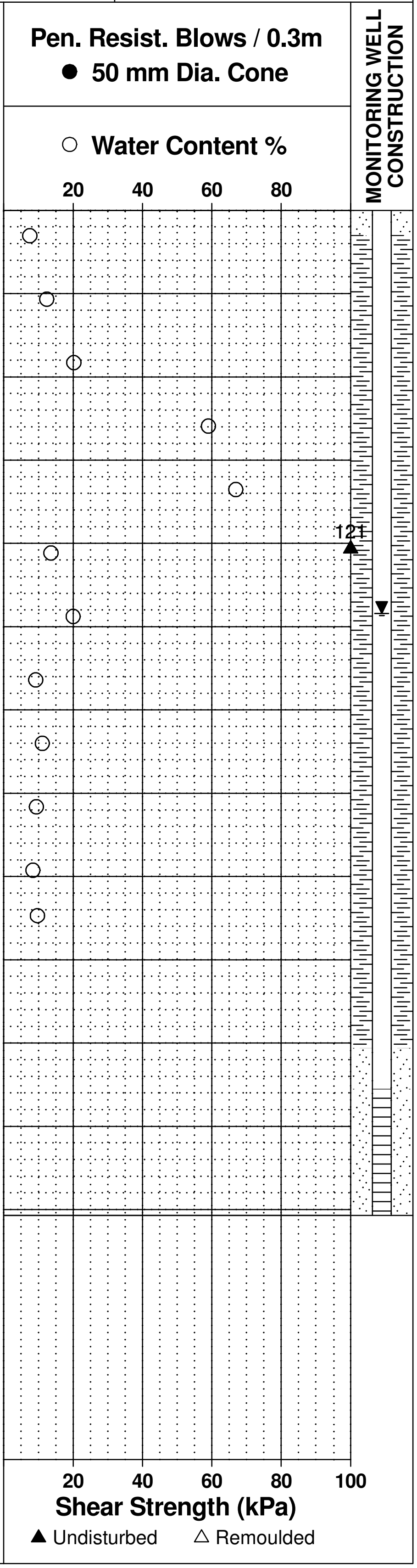
BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 1, 2024

FILE NO. PG5933

HOLE NO. BH 6-24

SAMPLE DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
ASPHALT	0.05				
FILL: Brown silty sand with gravel	0.56	AU	1		
FILL: Brown silty sand, trace gravel		SS	2	75	11
- trace wood by 1.5m depth	2.21	SS	3	67	6
Loose to compact, grey **SILTY SAND**	2.97	SS	4	50	6
Firm, grey **SILTY CLAY**	3.73	SS	5	100	2
GLACIAL TILL: Compact, grey silty sand with gravel, trace clay		SS	6	50	6
		SS	7	33	7
		SS	8	42	10
		SS	9	42	9
		SS	10	75	7
		SS	11	67	19
- very dense by 8.4m depth	8.61	SS	12	57	+50
BEDROCK: Excellent quality, grey limestone interbedded with black shale		RC	1	100	100
		RC	2	100	97
	12.07	RC	3	100	100
End of Borehole					
(GWL @ 4.84m - Mar. 14, 2024)					

DEPTH (m)	ELEV. (m)
0	68.12
1	67.12
2	66.12
3	65.12
4	64.12
5	63.12
6	62.12
7	61.12
8	60.12
9	59.12
10	58.12
11	57.12
12	56.12



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
265 Catherine Street
Ottawa, Ontario

EASTING: 367801.934 **NORTHING:** 5030156.324 **ELEVATION:** 68.31
DATUM: Geodetic
REMARKS:
BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 4, 2024

FILE NO. PG5933
HOLE NO. BH 7-24

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty sand with gravel 0.76

FILL: Brown sand, trace gravel 1.52

FILL: Brown silty sand with gravel, trace peat 2.21

FILL: Brown to grey silty sand 2.97

Stiff, grey **SILTY CLAY** 6.78

Grey **SILTY SAND** to **SANDY SILT** 7.16

Grey **SILTY SAND** with gravel, trace clay

- trace to some shale fragments by 8.3m depth 9.55

BEDROCK: Fair to good quality, grey limestone interbedded with black shale

- excellent quality by 10.5m depth

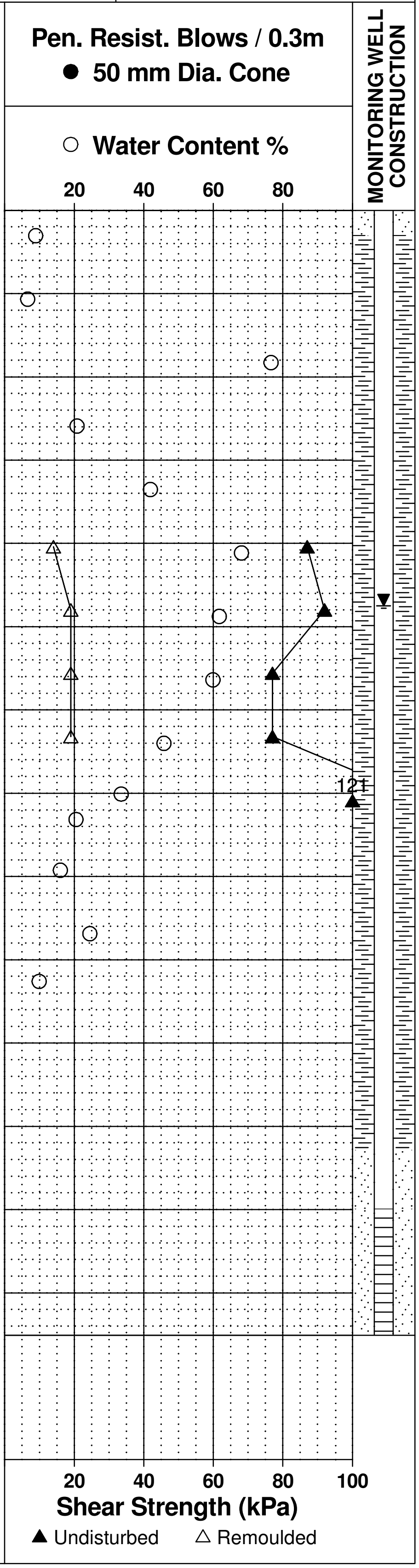
- shale content decreases with depth 13.51

End of Borehole

(GWL @ 4.75m - Mar. 14, 2024)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	18
SS	3	75	4
SS	4	17	15
SS	5	100	3
SS	6	75	P
SS	7	100	P
SS	8	100	P
SS	9	100	P
SS	10	100	2
SS	11	83	4
SS	12	31	+50
SS	13	40	+50
RC	1	100	65
RC	2	100	93
RC	3	100	100

DEPTH (m)	ELEV. (m)
0	68.31
1	67.31
2	66.31
3	65.31
4	64.31
5	63.31
6	62.31
7	61.31
8	60.31
9	59.31
10	58.31
11	57.31
12	56.31
13	55.31



SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. High-Rise Building - 265 Catherine Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

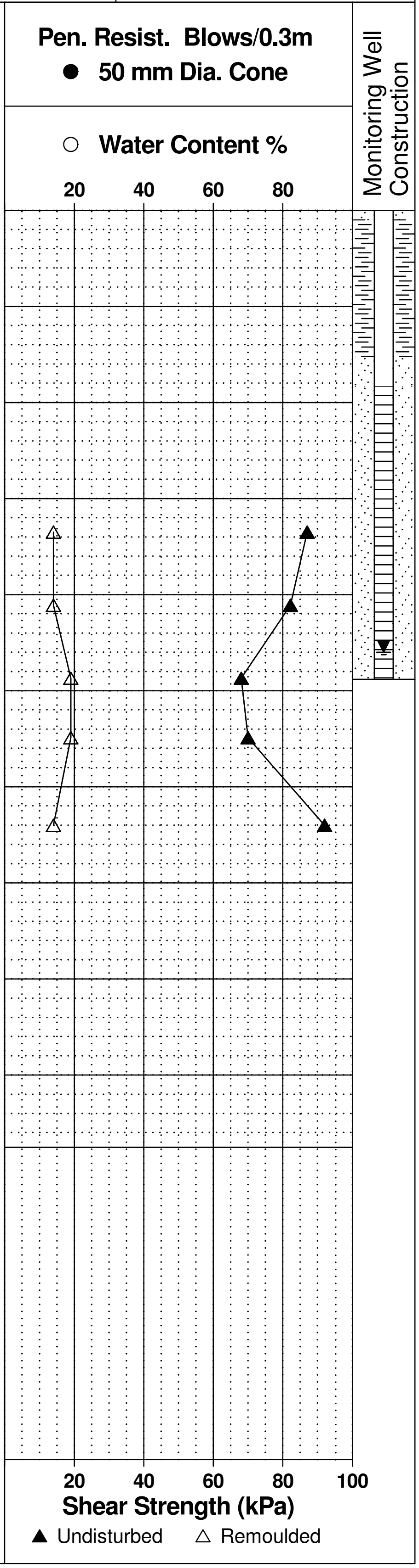
DATE August 19, 2020

FILE NO. PG5498

HOLE NO. BH 1-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete 0.10					
FILL: Brown silty sand 0.63		AU	1		
FILL: Brown silty sand with gravel, cobbles and debris (wood, bricks)		SS	2	75	50+
2.29		SS	3	58	18
Compact, brown **SILTY SAND** 3.05		SS	4	75	2
Stiff, grey **SILTY CLAY,** some fine sand seams		SS	5	100	P
		SS	6	100	P
7.62		SS	7	38	P
Grey **SILTY CLAY,** trace silty sand		SS	8	100	2
9.75		SS	9	100	2
End of Borehole (GWL @ 4.60m - Sept. 1, 2020)					

DEPTH (m)	ELEV. (m)
0	68.62
1	67.62
2	66.62
3	65.62
4	64.62
5	63.62
6	62.62
7	61.62
8	60.62
9	59.62



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. High-Rise Building - 265 Catherine Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 19, 2020

FILE NO. PG5498

HOLE NO. BH 2-20

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with crushed stone	0.60
FILL: Brown silty sand with gravel, trace wood and brick	2.13
Brown **SILTY CLAY,** trace brown silty sand	9.14
GLACIAL TILL: Grey clayey silty sand with gravel, cobbles and boulders	9.75
Dynamic Cone Penetration Test commenced at 9.75m depth. Inferred **GLACIAL TILL**	10.84

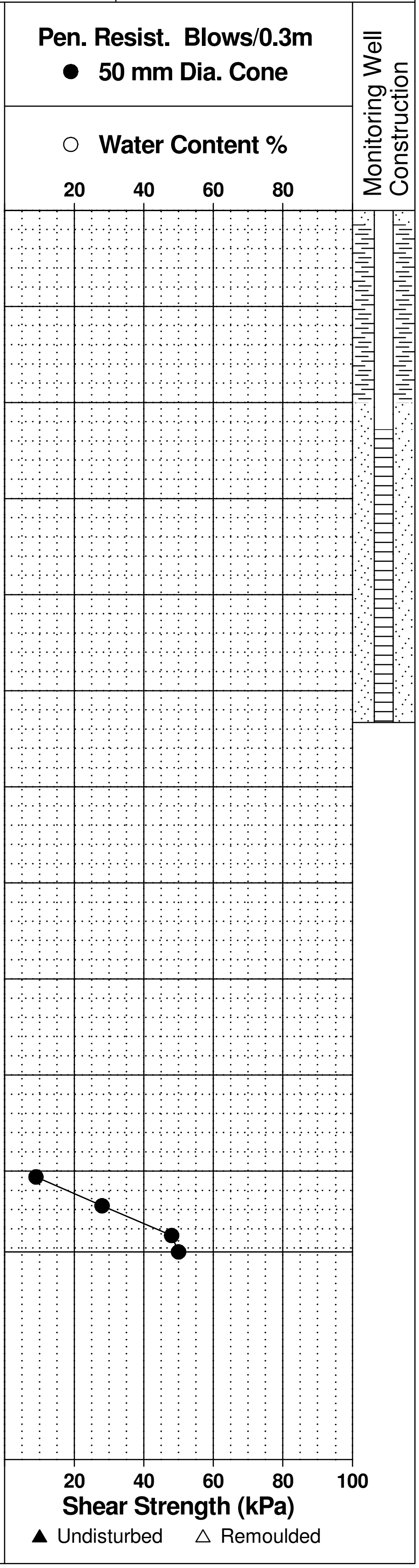
End of Borehole

Practical DCPT refusal at 10.84m depth.

(BH dry - Sept. 1, 2020)

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	16
SS	3	18	9
SS	4	100	4
SS	5	100	2
SS	6	100	4
SS	7	100	2
SS	8	100	3
SS	9	100	4
SS	10	100	2
SS	11	58	3

DEPTH (m)	ELEV. (m)
0	68.46
1	67.46
2	66.46
3	65.46
4	64.46
5	63.46
6	62.46
7	61.46
8	60.46
9	59.46
10	58.46



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. High-Rise Building - 265 Catherine Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 19, 2020

FILE NO. PG5498

HOLE NO. BH 3-20

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with silty clay and crushed stone	0.60
Loose to compact, brown **SILTY SAND,** some organics	2.13
Stiff, grey **SILTY CLAY** with sandy silt	4.42
GLACIAL TILL: Compact, grey sandy silt with some clay, gravel and cobbles	5.33
GLACIAL TILL: Grey clayey silty sand with gravel, cobbles and boulders	7.49

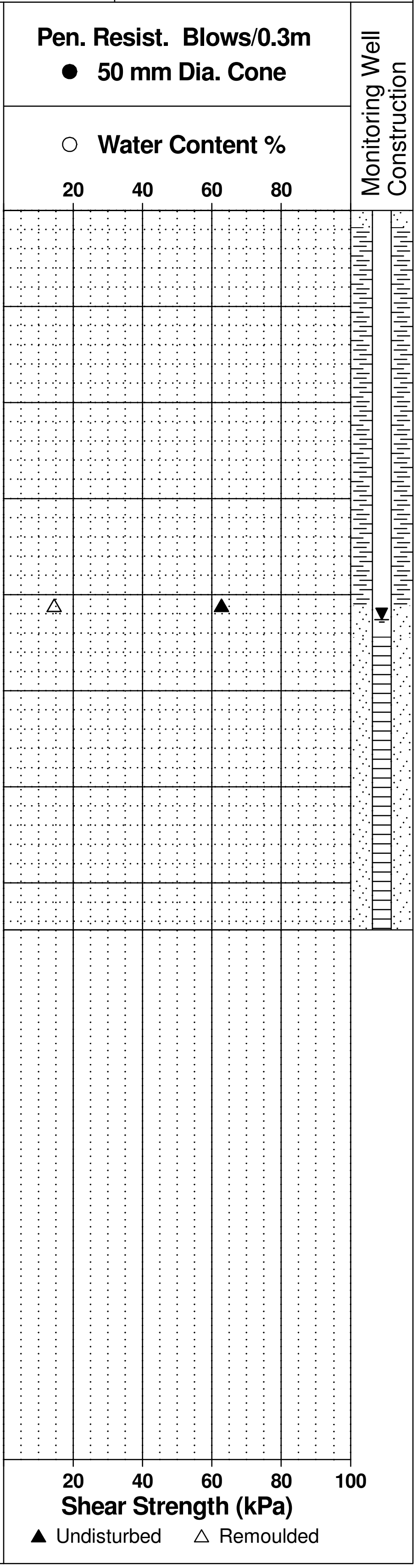
End of Borehole

Practical refusal to augering at 7.49m depth.

(GWL @ 4.26m - August 28, 2020)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	38	9
SS	3	67	13
SS	4	100	2
SS	5	100	2
SS	6	100	P
SS	7	42	11
SS	8	62	4
SS	8	46	7

DEPTH (m)	ELEV. (m)
0	68.11
1	67.11
2	66.11
3	65.11
4	64.11
5	63.11
6	62.11
7	61.11



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnica Investigation
265 Catherine Street
Ottawa, Ontario

DATUM TBM - Finished floor level of existing building at gate 2. Assumed elevation = 100.00m.

FILE NO. PG2174

REMARKS

HOLE NO. BH 1

BORINGS BY CME 45 Power Auger

DATE 24 August 2010

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Crushed stone	0.20
FILL: Dark brown silty sand with gravel and cobbles	1.83
Black **PEAT**	2.10
Loose, grey **SILTY SAND**	2.97
Very stiff to stiff, grey **SILTY CLAY**	6.00
	9.75

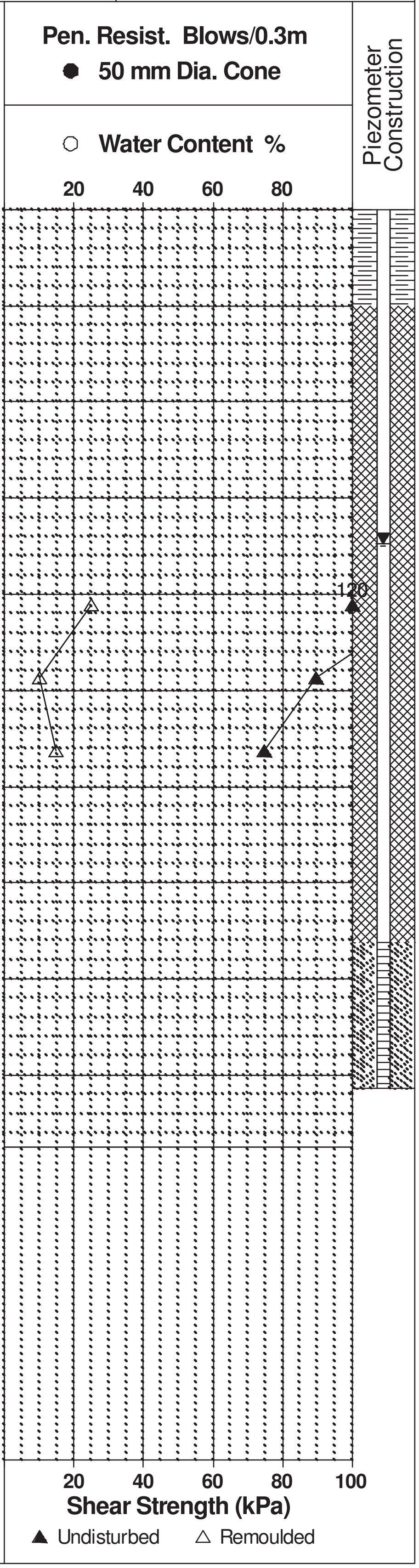
End of Borehole

Practical refusal to augering @ 9.75m depth

(GWL @ 3.48m-Nov. 22/10)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	29	50+
SS	3	62	3
SS	4	54	10
SS	5	92	2
SS	6		
SS	7	88	
SS	8	92	

DEPTH (m)	ELEV. (m)
0	99.45
1	98.45
2	97.45
3	96.45
4	95.45
5	94.45
6	93.45
7	92.45
8	91.45
9	90.45



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnica Investigation
265 Catherine Street
Ottawa, Ontario

DATUM TBM - Finished floor level of existing building at gate 2. Assumed elevation = 100.00m.

FILE NO. PG2174

REMARKS

HOLE NO. BH 2

BORINGS BY CME 45 Power Auger

DATE 24 August 2010

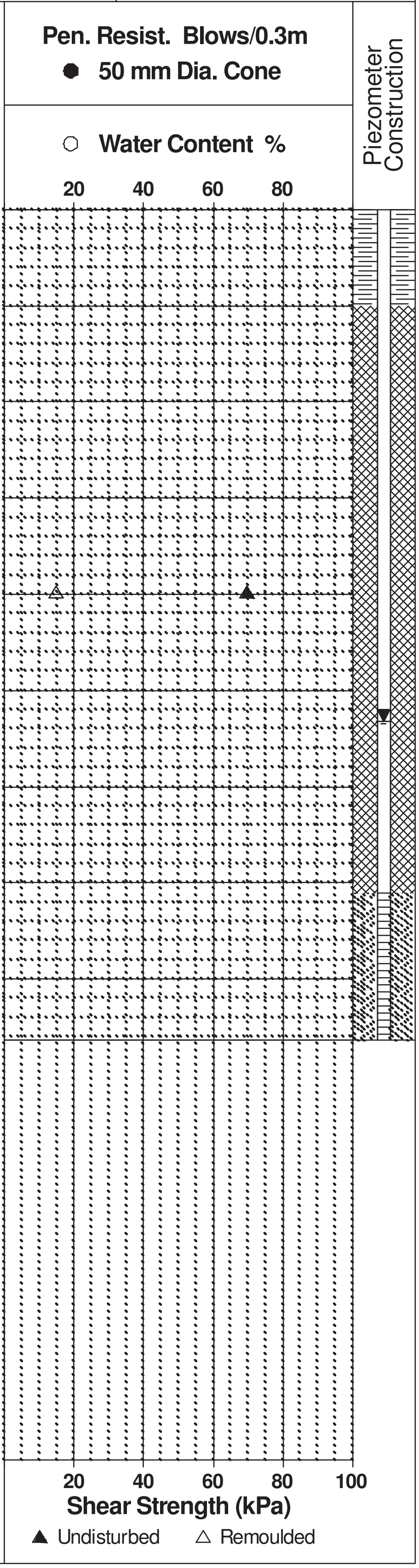
SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with gravel and cobbles	0.60
FILL: Brown silty sand with gravel, wood and brick	1.45
Loose, grey **SILTY SAND**	2.97
Stiff, grey-brown **SILTY CLAY**	4.85
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	8.63
End of Borehole	

Practical refusal to augering @ 8.63m depth

(GWL @ 5.32m-Sept. 16/10)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	7
SS	3	0	11
SS	4	33	8
SS	5	92	1
SS	6	100	
SS	7	67	26
SS	8	17	11
SS	9	42	4

DEPTH (m)	ELEV. (m)
0	99.38
1	98.38
2	97.38
3	96.38
4	95.38
5	94.38
6	93.38
7	92.38
8	91.38



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnica Investigation
265 Catherine Street
Ottawa, Ontario

DATUM TBM - Finished floor level of existing building at gate 2. Assumed elevation = 100.00m.

FILE NO. PG2174

REMARKS

HOLE NO. BH 3

BORINGS BY CME 45 Power Auger

DATE 24 August 2010

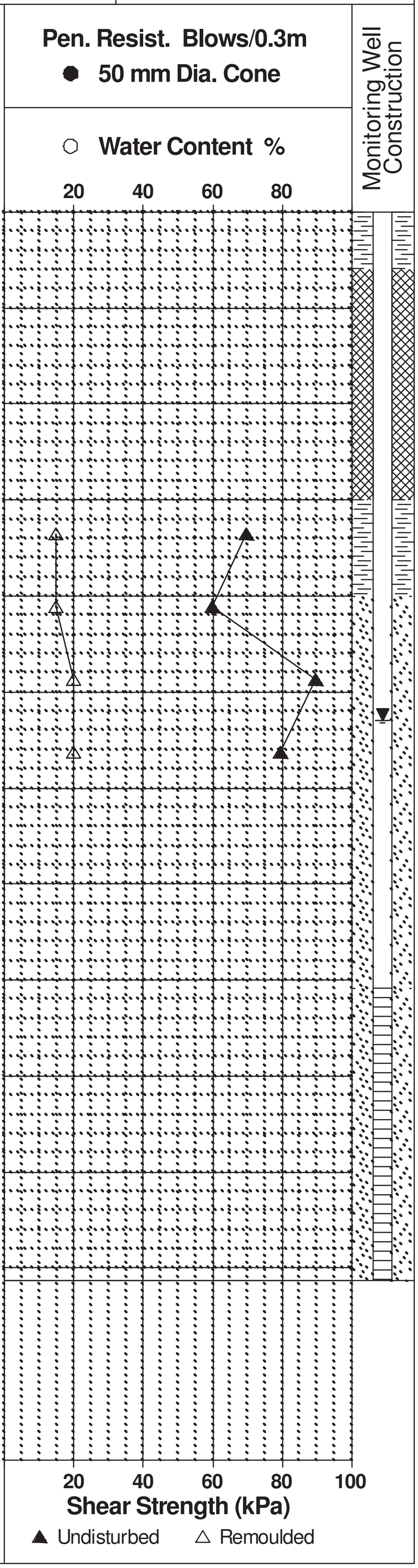
SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Concrete 0.45	AU	1		
FILL: Grey-brown sand 1.45	SS	2	50	10
FILL: Brown silty sand with gravel, cobbles and boulders 2.21	SS	3	58	23
Stiff, grey-brown **SILTY CLAY**	SS	4	100	2
	SS	5	92	
	SS	6	92	
	SS	7	92	
	SS	8	92	
6.70	SS	9	92	
11.13				

End of Borehole

Practical refusal to augering @ 11.13m depth

(GWL @ 5.30m-Sept. 16/10)

DEPTH (m)	ELEV. (m)
0	99.82
1	98.82
2	97.82
3	96.82
4	95.82
5	94.82
6	93.82
7	92.82
8	91.82
9	90.82
10	89.82
11	88.82



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnica Investigation
265 Catherine Street
Ottawa, Ontario

DATUM TBM - Finished floor level of existing building at gate 2. Assumed elevation = 100.00m.

FILE NO. PG2174

REMARKS

HOLE NO. BH 4

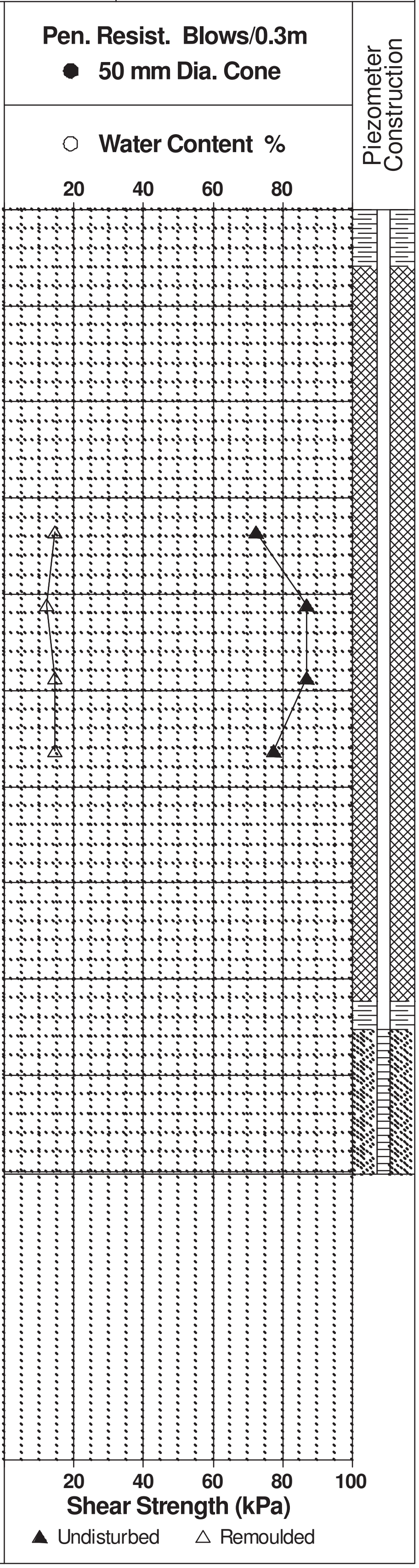
BORINGS BY CME 45 Power Auger

DATE 25 August 2010

SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.09
FILL: Brown silty sand with gravel and cobbles	0.60
	0.76
FILL: Grey silty clay with gravel and asphalt	
FILL: Brown silty sand, trace gravel	2.21
Stiff, grey-brown **SILTY CLAY**	8.53
Inferred **GLACIAL TILL**	10.03
End of Borehole Practical refusal to augering @ 10.03m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	83	25
SS	3	38	14
SS	4	92	4
SS	5	0	
SS	6	92	
SS	7	92	
SS	8		

DEPTH (m)	ELEV. (m)
0	99.67
1	98.67
2	97.67
3	96.67
4	95.67
5	94.67
6	93.67
7	92.67
8	91.67
9	90.67
10	89.67



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnica Investigation
265 Catherine Street
Ottawa, Ontario

DATUM TBM - Finished floor level of existing building at gate 2. Assumed elevation = 100.00m.

FILE NO. **PG2174**

REMARKS

HOLE NO. **BH 5**

BORINGS BY CME 45 Power Auger

DATE 25 August 2010

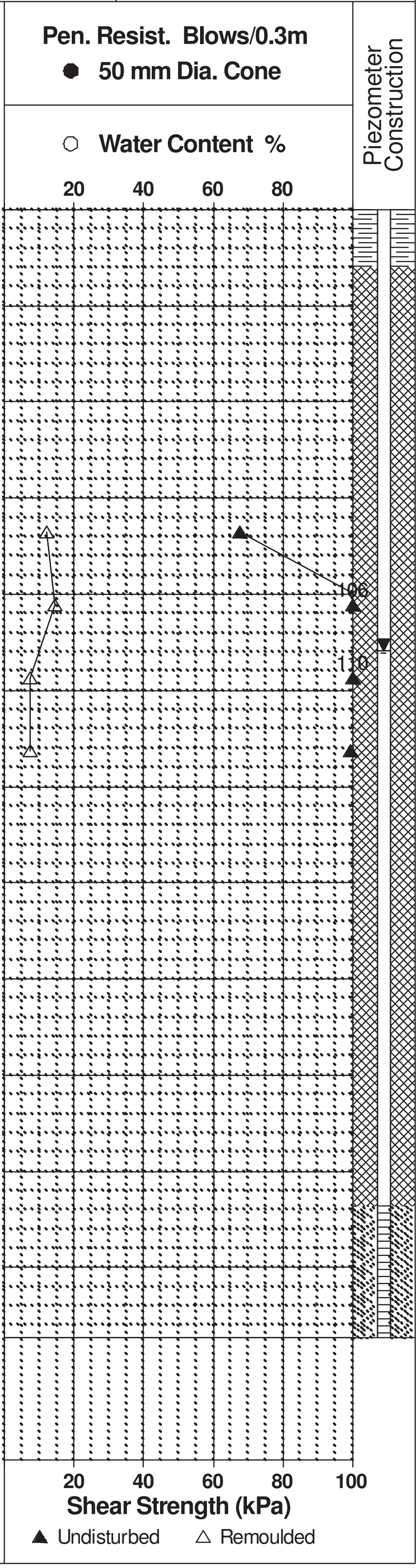
SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.13
FILL: Brown silty sand with gravel and cobbles	2.06
Stiff to very stiff, grey-brown **SILTY CLAY**	5.94
Inferred **SILTY CLAY**	9.75
Inferred **GLACIAL TILL**	11.73

End of Borehole
Practical refusal to augering @ 11.73m depth

(GWL @ 4.59m-Sept. 16/10)

SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
AU	1		
SS	2	58	10
SS	3	58	8
SS	4	88	2
SS	5	96	
SS	6	100	
SS	7	100	
SS	8	100	

DEPTH (m)	ELEV. (m)
0	99.75
1	98.75
2	97.75
3	96.75
4	95.75
5	94.75
6	93.75
7	92.75
8	91.75
9	90.75
10	89.75
11	88.75



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnica Investigation
265 Catherine Street
Ottawa, Ontario

DATUM TBM - Finished floor level of existing building at gate 2. Assumed elevation = 100.00m.

FILE NO. **PG2174**

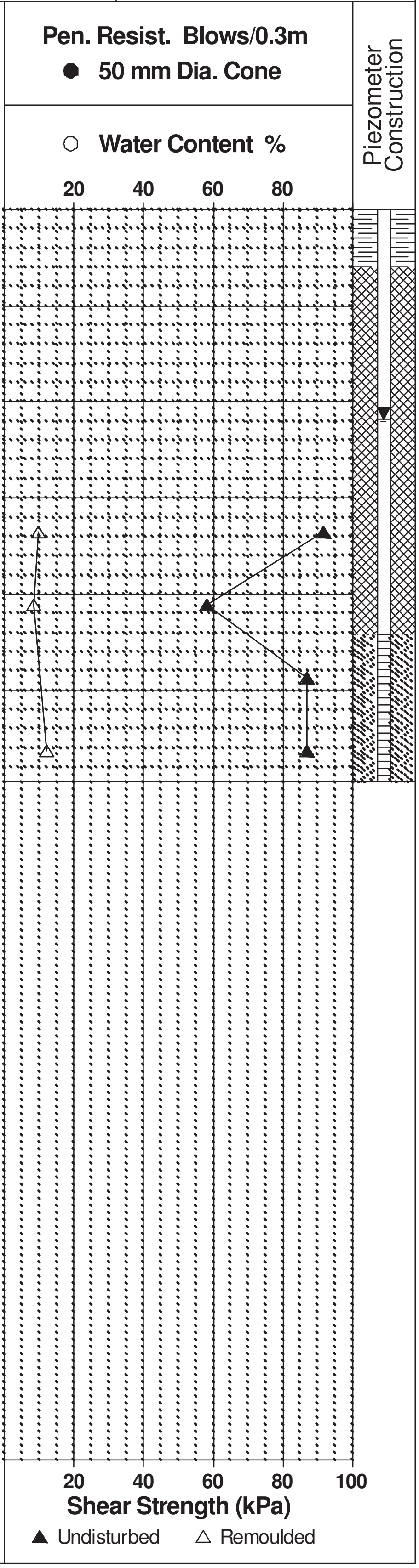
REMARKS

HOLE NO. **BH 6**

BORINGS BY CME 45 Power Auger

DATE 25 August 2010

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.94
Asphaltic concrete 0.10		AU	1				
FILL: Brown silty sand with gravel, cobbles and boulders		SS	2		4	1	98.94
		SS	3	50	8	2	97.94
2.36							
Stiff, grey-brown **SILTY CLAY**		SS	4	92	1	3	96.94
		SS	5	96			
		SS	6	92		4	95.94
		SS	7	58		5	94.94
		SS	8	92			
5.94							
End of Borehole (GWL @ 2.18m-Sept. 16/10)							



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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water 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 at 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

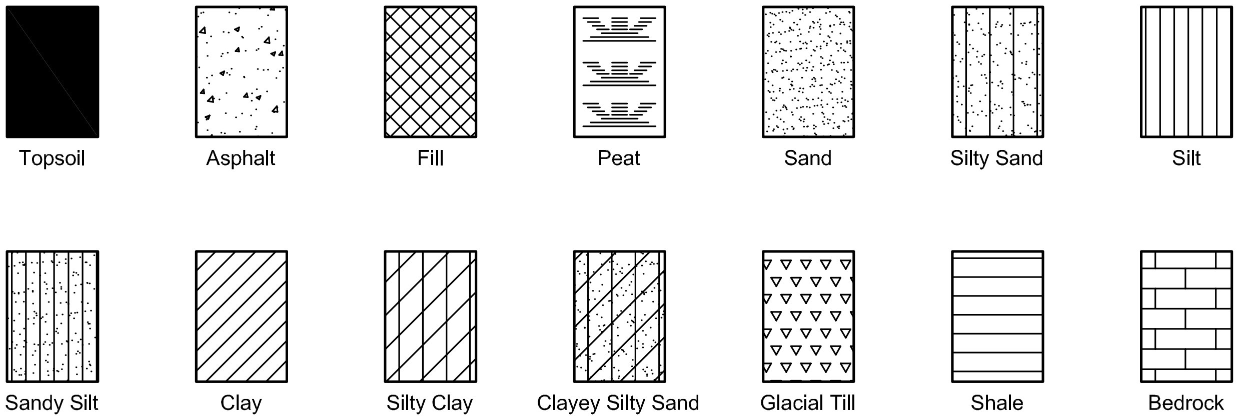
Wo - Initial water content (at start of consolidation test)

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.

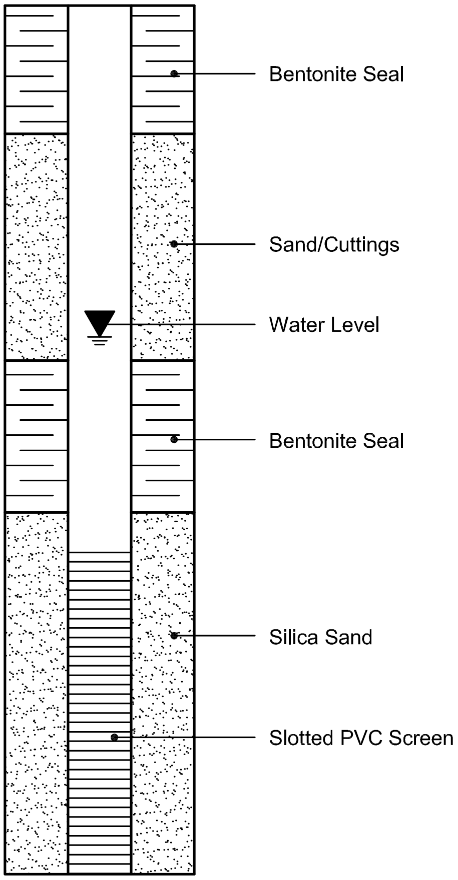
SYMBOLS AND TERMS (continued)

STRATA PLOT

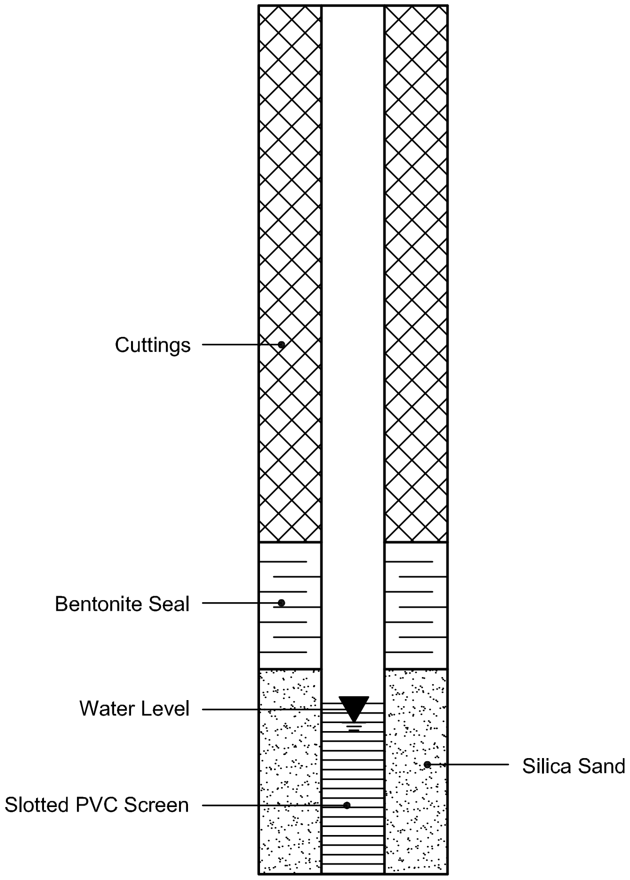


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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SOIL PROFILE & TEST SUMMARIES
PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

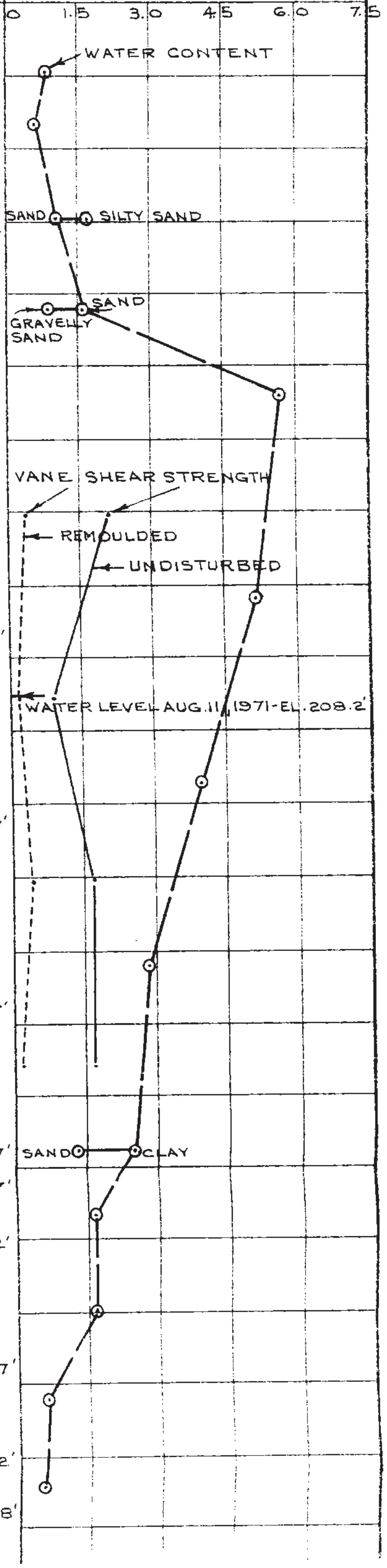
KENT & LYON

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 227.2′ DATE AUG. 6, 1971
NIVEAU DU SOL (PROFONDEUR ZÉRO)
NOTES B.M. (ELEV. 231.49′) GEODETIC. SOUTHEAST CORNER OF KENT & McLEOD

HOLE No.
FORAGE
1

Compressive Strength K.S.F. Résistance à la Compression K/Pd.²	Small Scale Penetrometer K.S.F. Petit Pénétromètre K/Pd.²	Essai-Standard Penetration Blows/ft-Coups/pd	Sample No. Échantillon	DESCRIPTION OF SOIL DU SOL Ground Surface – Niveau du Sol	Depth in Feet Profondeur – Pied	Elevation Niveau
				FILL- SAND, GRAVEL & CRUSHED STONE	0′	227.2′
		11	1-1	MEDIUM DENSE FINE & VERY FINE SAND WITH A ½″ SILTY VERY FINE SAND LAYER	1.0′	226.2′
		12 10 FOR 6″	1-2			
		21	1-3			
		31 32 FOR 6″	1-4	DENSE FINE & VERY FINE SAND WITH SOME SILT & A 2″ GRAVELLY SAND LAYER	6.5′	220.7′
	1.5, 1.6, 1.4	5	1-5	MEDIUM SOFT GRAY CLAY WITH SOME SILT	10.0′	217.2′
	2.4, 3.2, 2.2 2.2, 2.5, 2.6 R-0.2		1-6	STIFF GRAY CLAY WITH SOME SILT WITH A FEW FISSURES	11.5′	215.7′
	3.2, 2.6, 3.0 2.8, 3.4, 3.2 2.2, 2.2, 2.0 2.4, 2.0, 2.2 R-0.2		1-7	STIFF SILTY GRAY CLAY WITH A FEW FISSURES	17.5′	209.7′
	1.2, 1.4, 1.4 1.4, 1.4, 1.2 R-0.0		1-8	MEDIUM SOFT SILTY GRAY CLAY WITH A TRACE OF VERY FINE SAND	22.5′	204.7′
	2.5, 2.4, 2.6 2.2, 2.5, 2.2 2.0, 2.2, 2.4 2.0, 2.2, 1.6 R-0.0		1-9	STIFF SILTY GRAY CLAY WITH A FEW FISSURES	27.5′	199.7′
				STIFF SILTY GRAY CLAY WITH SOME ¼″ TO ½″ MEDIUM & FINE SAND LAYERS	31.5′	195.7′
		22	1-10	MEDIUM DENSE CLAYEY SILT WITH SOME VERY FINE SAND & A TRACE OF GRAVEL	32.5′	194.7′
		21	1-11	MEDIUM DENSE VERY FINE SAND WITH A LITTLE SILT & A TRACE OF GRAVEL	34.0′	193.2′
		26	1-12	MEDIUM DENSE TILL	37.5′	189.7′
		100	1-13	VERY DENSE SILTY TILL	40.0′	187.2′
				LIMESTONE	41.4′	185.8′

~~PROBING OR~~ VANE TEST — ~~SONDAGE OU~~ ESSAI AU MOULINET
MARTEAU---HAMMER / CHUTE LIBRE---DROP
NO CASING / SANS TUBAGE / BARRE----DIA. ROD
~~BLOWS/FOOT OR~~ SHEAR STRENGTH K.S.F.
~~COUPS/PIED OU~~ RÉSISTANCE AU, K/PD.² CISAILLEMENT



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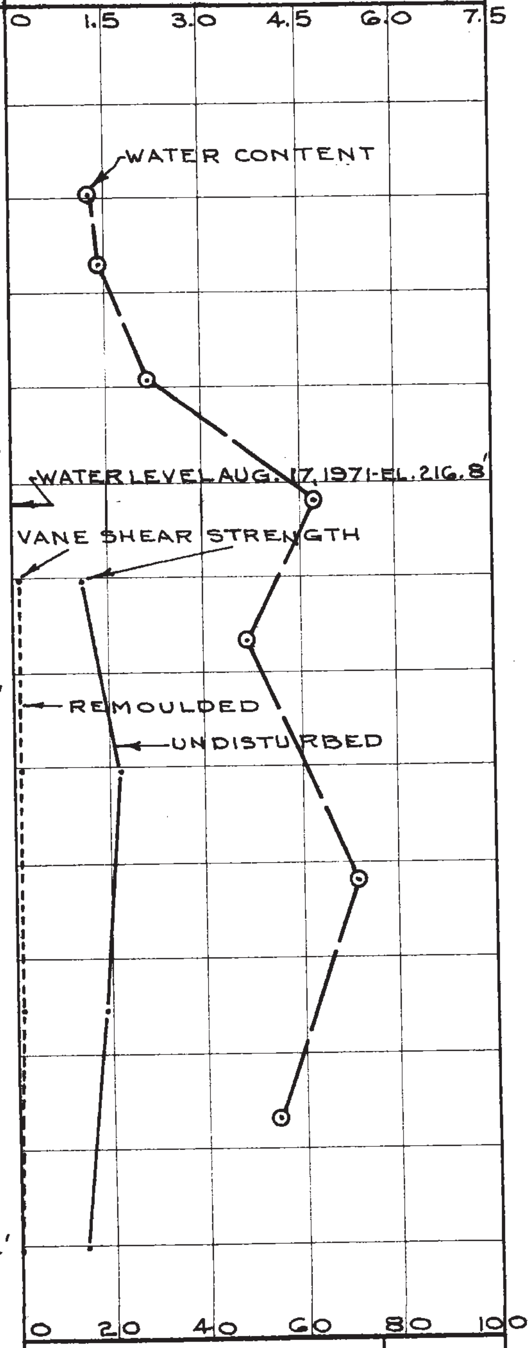
KENT & LYON

ELEVATION OF GROUND SURFACE (ZERO DEPTH) NIVEAU DU SOL (PROFONDEUR ZERO) 225.2'
DATE AUG. 9, 1971
HOLE FORAGE No. 2
NOTES SEE PLATE No. 2

Compressive Strength K.S.F. Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. Petit Pénétromètre K/Pd.2	Essai-Standard Penetration Blows/ft.-Coups/pd	Sample No. Echantillon No.	DESCRIPTION OF SOIL DU SOL Ground Surface – Niveau du Sol	Depth in Feet Profondeur - Pied	Elevation Niveau
					0'	225.2'
		13	2-1	FILL- SAND WITH SOME CLAY & SILT & WITH A TRACE OF GRAVEL, CINDERS & ORGANIC MATERIAL		
		12	2-2			
		11	2-3			
					7.5'	217.7'
	1.2, 1.4, 1.4	8	2-4	MEDIUM SOFT TO STIFF BROWNISH GRAY CLAY WITH SOME SILT & A FEW FISSURES		
	2.6, 2.4, 2.4 2.6, 2.5, 2.8 3.0, 3.0, 3.2 3.4, 3.2, 3.6 1.8, 1.8, 1.6 R-0.0		2-5			
					12.5'	212.7'
	2.4, 2.0, 2.6 3.2, 3.4, 3.2 3.6, 3.6, 3.6 2.5, 2.4, 2.6 2.8, 2.6, 2.5 R-0.0		2-6	STIFF GRAY CLAY WITH SOME SILT		
	2.8, 2.6, 2.5 3.0, 2.6, 2.8 2.4, 2.6, 2.4 2.6, 3.0, 3.4 2.6, 2.4, 2.4 R-0.0		2-7			
					24.0'	201.2'
				BOTTOM OF HOLE		

~~PROBING OR~~ VANE TEST
~~SONDAGE OU~~ ESSAI AU MOULINET
MARTEAU----HAMMER
CHUTE LIBRE---DROP
NO CASING
SANS TUBAGE
BARRE-----DIA. ROD
~~BLOWS/FOOT OR~~ SHEAR STRENGTH K.S.F.
~~COUPS/PIED OU~~ RÉSISTANCE AU CISAILLEMENT K/PD.2



WATER CONTENT
% TENEUR EN EAU
NATURAL NATURELLE ⊙
LIQUID LIMIT LIMITE DE LIQUIDITÉ □
PLASTIC LIMIT LIMITE DE PLASTICITÉ △

PLATE PLAQUE No. 3

R = REMOULDED-REMANIE
CR = CORE RECOVERY CAROTTE RECUPEREE

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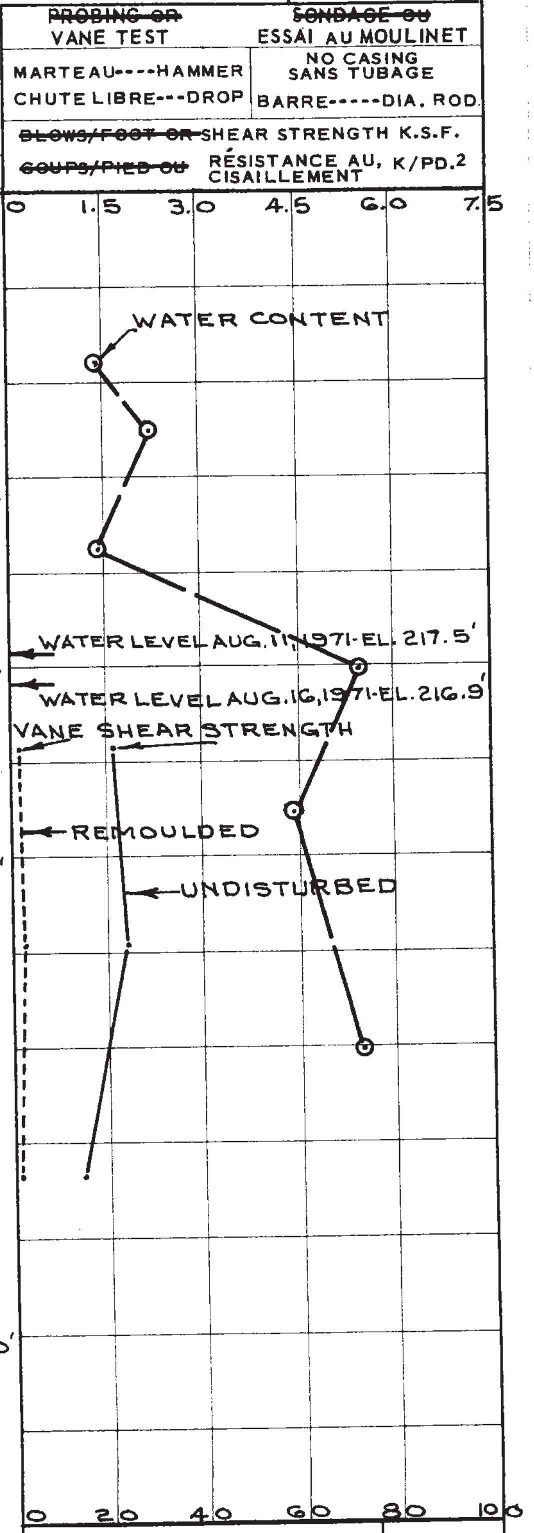
KENT & LYON

ELEVATION OF GROUND SURFACE (ZERO DEPTH) / NIVEAU DU SOL (PROFONDEUR ZERO) 225.5'
DATE AUG. 9, 1971
HOLE / FORAGE No. 3
NOTES SEE PLATE No. 2

Compressive Strength K.S.F. Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. Petit Pénétromètre K/Pd.2	Essai-Standard Penetration Blows/ft.-Coups/pd.	Sample No. Echantillon	DESCRIPTION OF SOIL / DU SOL	Depth in Feet Profondeur-Pied	Elevation Niveau
				Ground Surface – Niveau du Sol	0'	225.5'
		7	3-1	FILL- SAND WITH SOME CLAY & A LITTLE BRICK & CINDERS & WITH A TRACE OF GLASS & METAL		
		4	3-2		5.0'	220.5'
		9	3-3	LOOSE SILTY FINE & VERY FINE SAND		
		4	3-4		8.7'	216.8'
	2.6, 2.8, 2.8 2.8, 2.6, 2.8 2.2, 2.4, 2.2 2.4, 2.0, 2.2 1.0, 1.2, 1.2 R-0.0		3-5	MEDIUM SOFT TO STIFF BROWNISH GRAY CLAY WITH SOME SILT & A FEW FISSURES	12.5'	213.0'
	3.2, 3.4, 3.2 2.5, 2.4, 2.6 2.8, 2.8, 2.8 4.2, 4.2, 4.2 3.0, 3.0, 2.8 R-0.0		3-6	STIFF SILTY GRAY CLAY		
	3.5, 3.4, 3.2 3.6, 3.6, 3.8 3.6, 3.8, 3.8 3.4, 3.6, 3.8 2.6, 2.5, 2.4 R-0.0		3-7		22.5'	203.0'
				BOTTOM OF HOLE		



WATER CONTENT %TENEUR EN EAU
NATURAL / NATURELLE ⊙
LIQUID LIMIT / LIMITE DE LIQUIDITÉ ☐
PLASTIC LIMIT / LIMITE DE PLASTICITÉ △

PLATE / PLAQUE No. 4

R = REMOULDED-REMANIE
CR = CORE RECOVERY / CAROTTE RECUPEREE

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PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

KENT & LYON

ELEVATION OF GROUND SURFACE (ZERO DEPTH) NIVEAU DU SOL (PROFONDEUR ZERO) 225.1' DATE AUG. 9, 1971

HOLE FORAGE No. 4

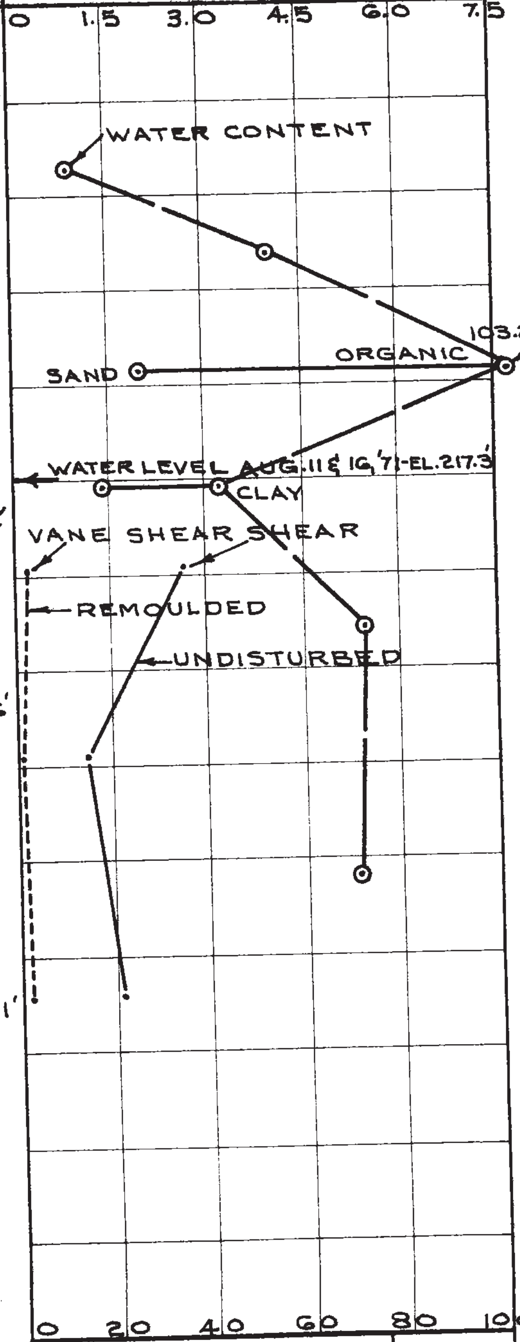
NOTES SEE PLATE No. 2

Compressive Strength K.S.F. Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. Petit Pénétromètre K/Pd.2	Essai-Standard Penetration Blows/ft.-Coups/pd.	Sample No. Echantillon	DESCRIPTION OF SOIL DU SOL	Depth in Feet Profondeur-Pied	Elevation Niveau
				Ground Surface – Niveau du Sol	0'	225.1'
		51 15 5 6FOR6	4-1 4-2 4-3	FILL- SAND & ORGANIC MATERIAL WITH SOME BROKEN ROCK & A TRACE OF WOOD & GRAVEL	6.0'	219.1'
	5.5, 5.4, 5.5	65	4-4	FILL- SILTY VERY FINE SAND & BROWNISH GRAY CLAY WITH SOME SAND & GRAVEL	8.7'	216.4'
	2.2, 1.6, 1.8 2.5, 2.5, 2.2 2.4, 3.2, 2.0 2.6, 2.6, 3.0 2.2, 1.8, 2.0 R-0.4		4-5	MEDIUM SOFT TO STIFF SILTY BROWNISH GRAY CLAY WITH SOME FISSURES	12.5'	212.6'
	4.4, 4.2, 4.2 3.5, 3.6, 3.5 3.8, 3.8, 3.8 3.4, 2.8, 2.8 3.6, 3.5, 3.5 R-0.2		4-6	STIFF TO VERY STIFF SILTY GRAY CLAY WITH SOME FISSURES	19.0'	206.1'
				BOTTOM OF HOLE		

~~PROBING OR~~ VANE TEST – ~~SONDAGE OU~~ ESSAI AU MOULINET

MARTEAU----HAMMER
CHUTE LIBRE---DROP
NO CASING SANS TUBAGE
BARRE-----DIA. ROD

~~BLOWS/FOOT OR~~ SHEAR STRENGTH K.S.F.
~~COUPS/PIED OU~~ RÉSISTANCE AU, K/PD.2 CISAILLEMENT



WATER CONTENT
%TENEUR EN EAU

NATURAL NATURELLE ⊙
LIQUID LIMIT LIMITE DE LIQUIDITÉ □
PLASTIC LIMIT LIMITE DE PLASTICITÉ △

PLATE PLAQUE No. 5

R = REMOULDED-REMANIE
CR = CORE RECOVERY CAROTTE RECUPEREE

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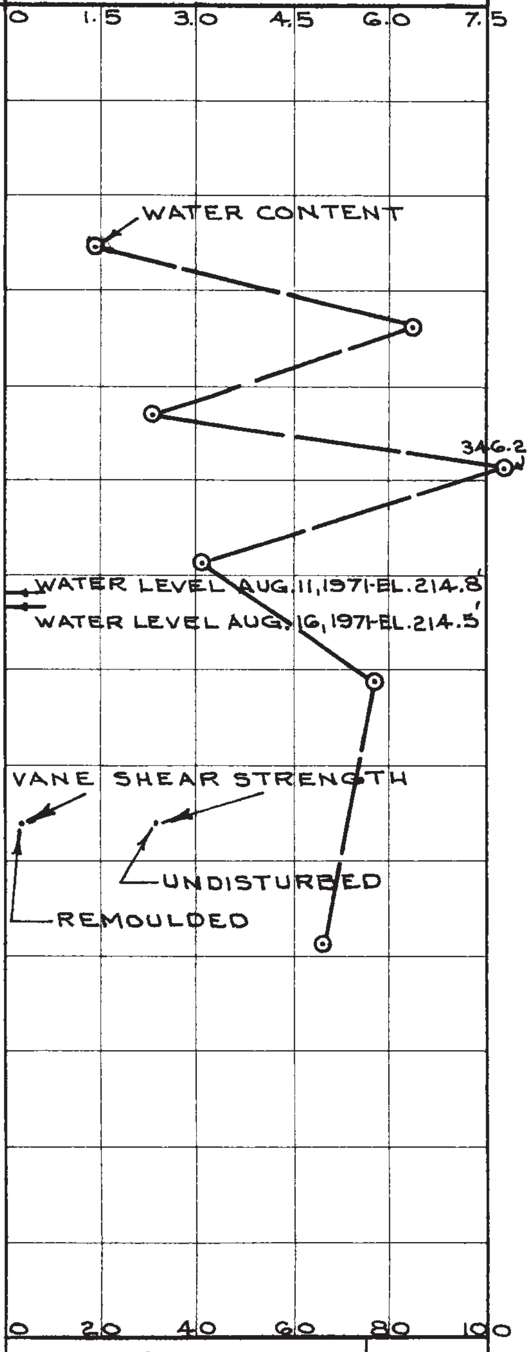
KENT & LYON

ELEVATION OF GROUND SURFACE (ZERO DEPTH) / NIVEAU DU SOL (PROFONDEUR ZERO): 223.7'
DATE: AUG. 10, 1971
HOLE / FORAGE No.: 5
NOTES: SEE PLATE No. 2

Compressive Strength K.S.F. Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. Petit Pénétromètre K/Pd.2	Essai-Standard Penetration Blows/ft.-Coups/pd	Sample No. Echantillon	DESCRIPTION OF SOIL DU SOL	Depth in Feet Profondeur-Pied	Elevation Niveau
				Ground Surface – Niveau du Sol	0'	223.7'
		5	5-1	FILL- SAND & ORGANIC MATERIAL WITH SOME GRAVEL, A LITTLE CLAY, ASHES & WOOD & A TRACE OF GLASS		
		4	5-2			
		4	5-3		6.0'	217.7'
			5-4	FILL- SILTY VERY FINE SAND & FIBROUS ORGANIC MATERIAL & A 1" STONE		
		3	5-5		10.7'	213.0'
	0.6, 0.4, 0.8	2 FOR 18"	5-6	SOFT SILTY GRAY CLAY	11.5'	212.2'
	2.6, 2.4, 2.5 2.2, 3.6, 2.4 3.4, 3.2, 3.4 4.4, 3.2, 3.5 3.0, 2.8, 2.6 R-0.5		5-7	STIFF SILTY GRAY CLAY WITH A FEW FISSURES & A 1/64" VERTICAL FINE SAND LAYER & A FEW 1/8" TO 1/4" CLAYEY FINE SAND LAYERS	18.0'	205.7'
				BOTTOM OF HOLE		

~~PROBING OR~~ VANE TEST – ~~SONDAGE OU~~ ESSAI AU MOULINET
MARTEAU----HAMMER
CHUTE LIBRE---DROP
NO CASING / SANS TUBAGE
BARRE-----DIA. ROD
~~BLOWS/FOOT OR~~ SHEAR STRENGTH K.S.F.
~~COUPS/PIED OU~~ RÉSISTANCE AU CISAILLEMENT, K/PD.2



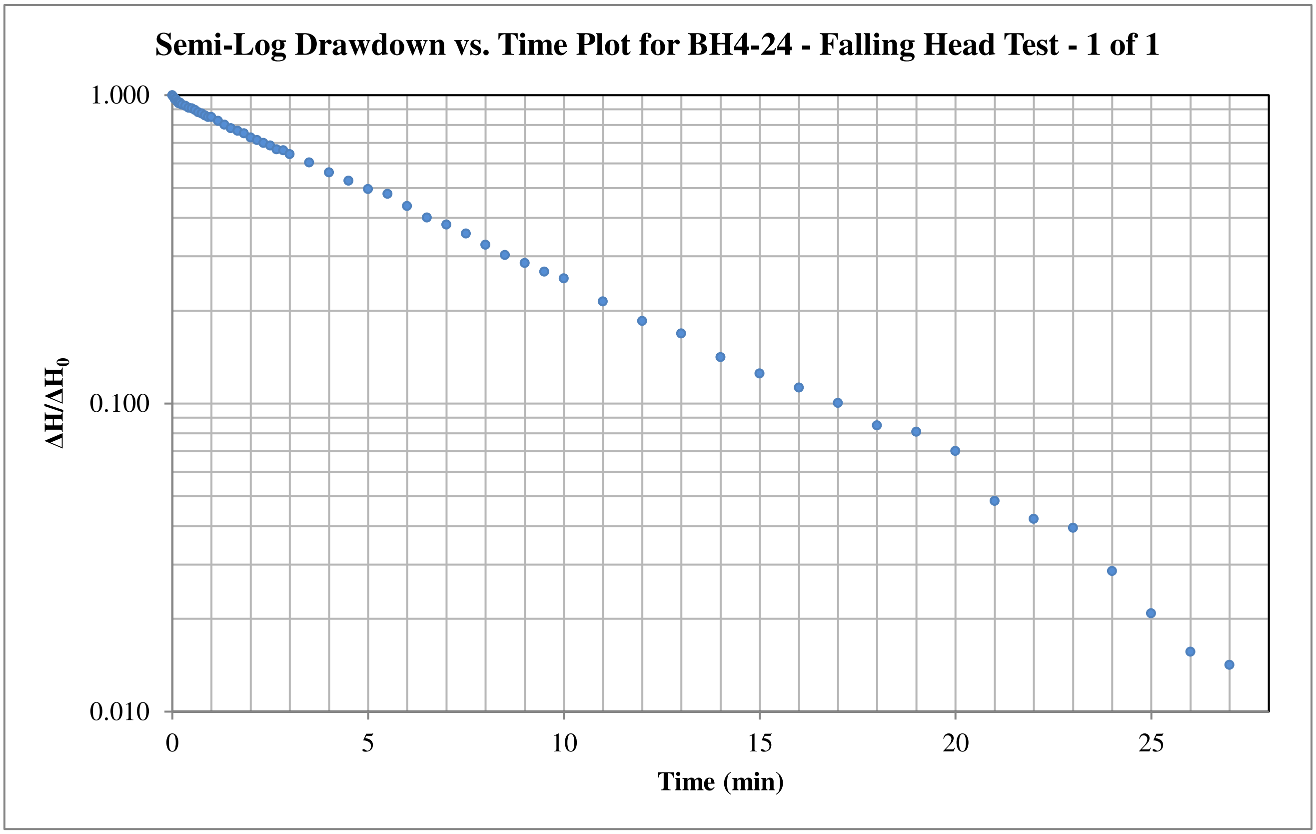
WATER CONTENT % TENEUR EN EAU
NATURAL / NATURELLE ⊙
LIQUID LIMIT / LIMITE DE LIQUIDITÉ □
PLASTIC LIMIT / LIMITE DE PLASTICITÉ △

PLATE / PLAQUE No. 6

R = REMOULDED-REMANIE
CR = CORE RECOVERY / CAROTTE RECUPEREE

Hvorslev Hydraulic Conductivity Analysis

Project: Brigil - 265 Catherine Street
Test Location: BH4-24
Test: Falling Head - 1 of 1
Date: March 14, 2024



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: 2.31086

Well Parameters:

L	1.5 m	Saturated length of screen or open hole
D	0.0508 m	Diameter of well
r_c	0.0254 m	Radius of well

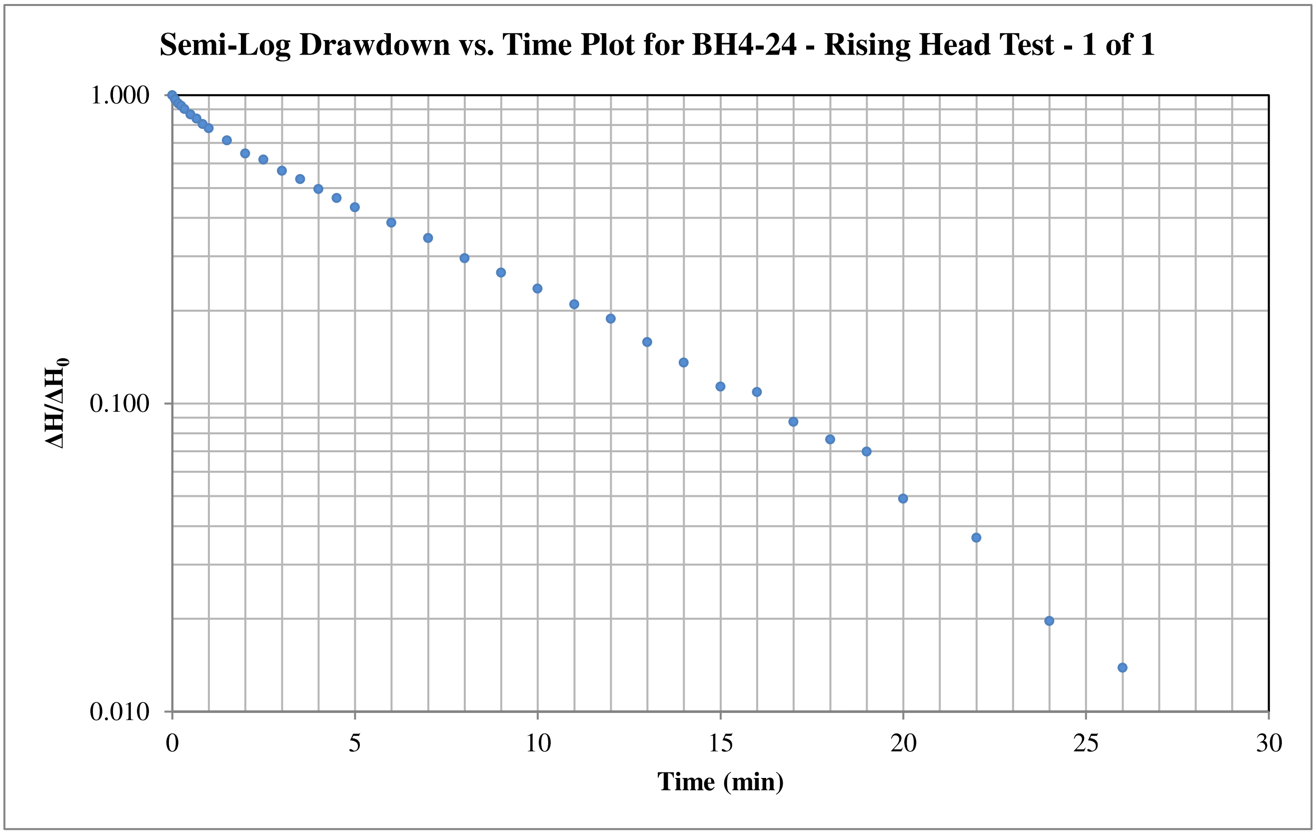
Data Points (from plot):

t*: 7.190 minutes ΔH*/ΔH_0: 0.37

Horizontal Hydraulic Conductivity
K = 2.02E-06 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Brigil - 265 Catherine Street
Test Location: BH4-24
Test: Rising Head - 1 of 1
Date: March 14, 2024



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: 2.31086

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

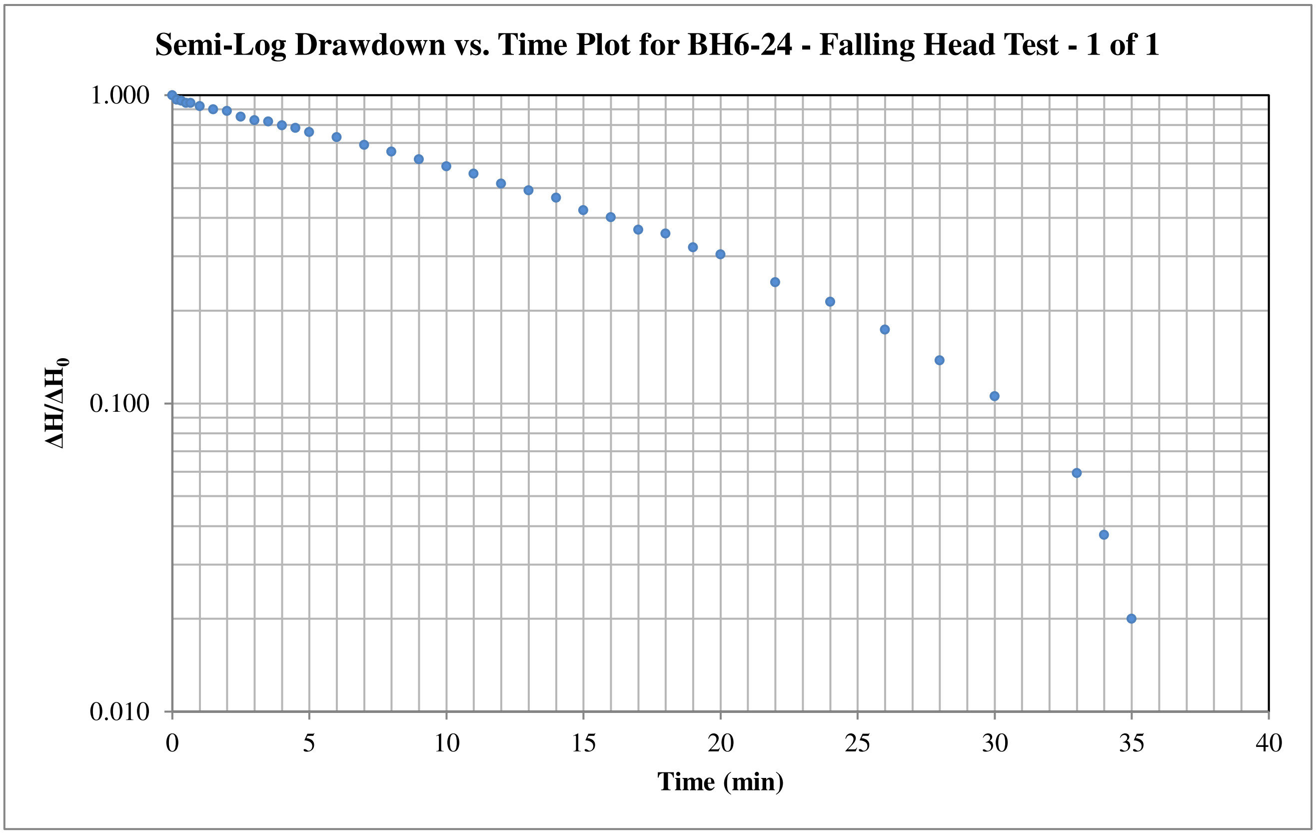
Data Points (from plot):

t^*: 6.257 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.32E-06 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Brigil - 265 Catherine Street
Test Location: BH6-24
Test: Falling Head - 1 of 1
Date: March 14, 2024



Hvorslev Horizontal Hydraulic Conductivity

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

Hvorslev Shape Factor

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

Valid for L>>D

Hvorslev Shape Factor F: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

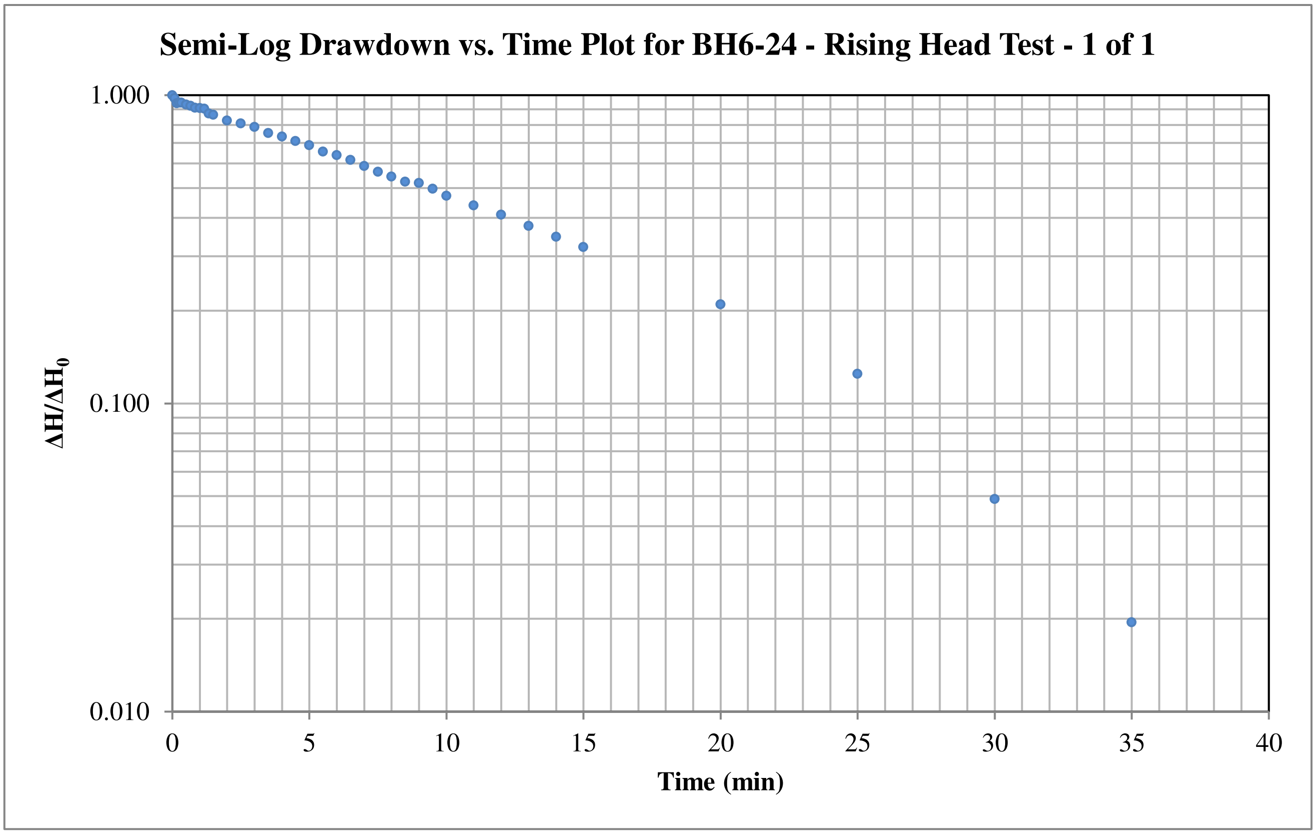
Data Points (from plot):

t*: 17.214 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.68E-07 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Brigil - 265 Catherine Street
Test Location: BH6-24
Test: Rising Head - 1 of 1
Date: March 14, 2024



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: 2.07207

Well Parameters:

L	1.5 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

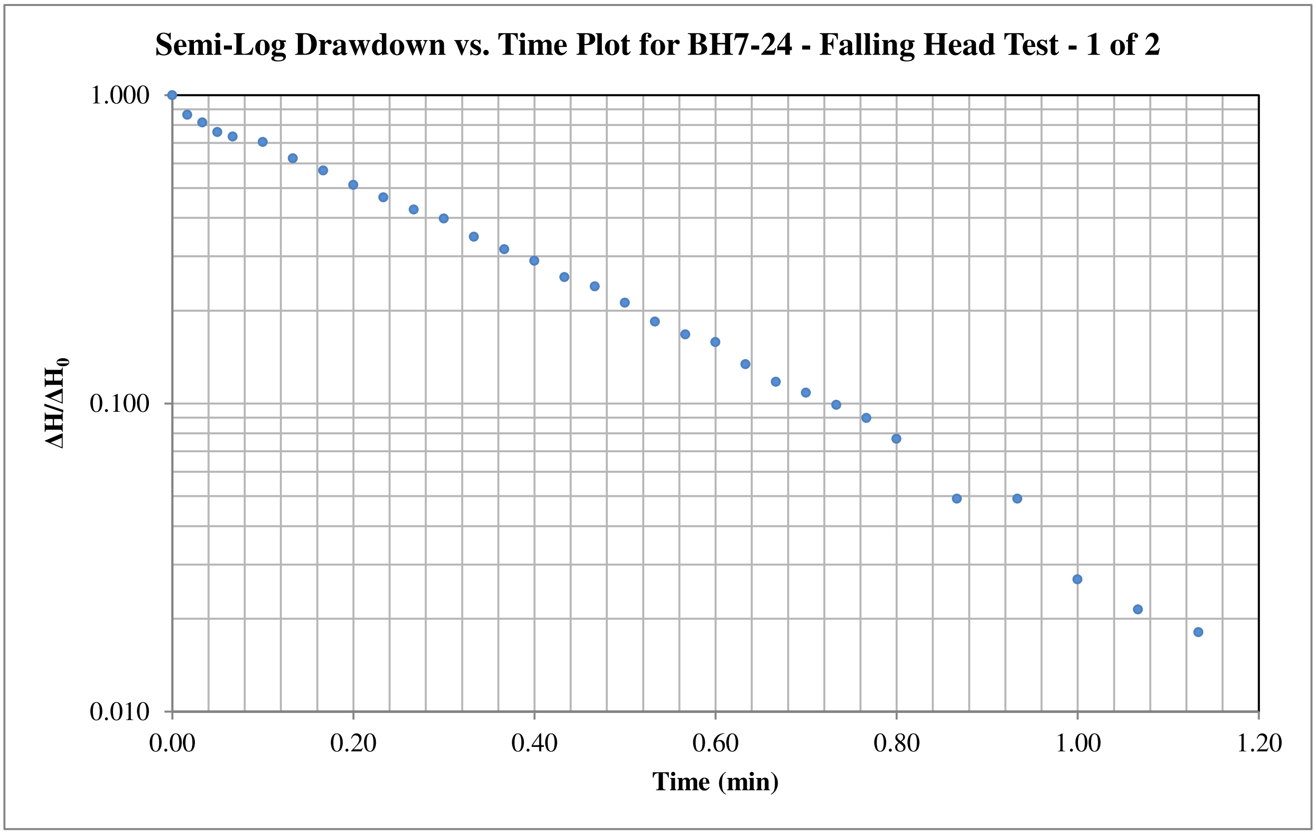
Data Points (from plot):

t*: 13.128 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.82E-07 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Brigil - 265 Catherine Street
Test Location: BH7-24
Test: Falling Head - 1 of 2
Date: March 14, 2024



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: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

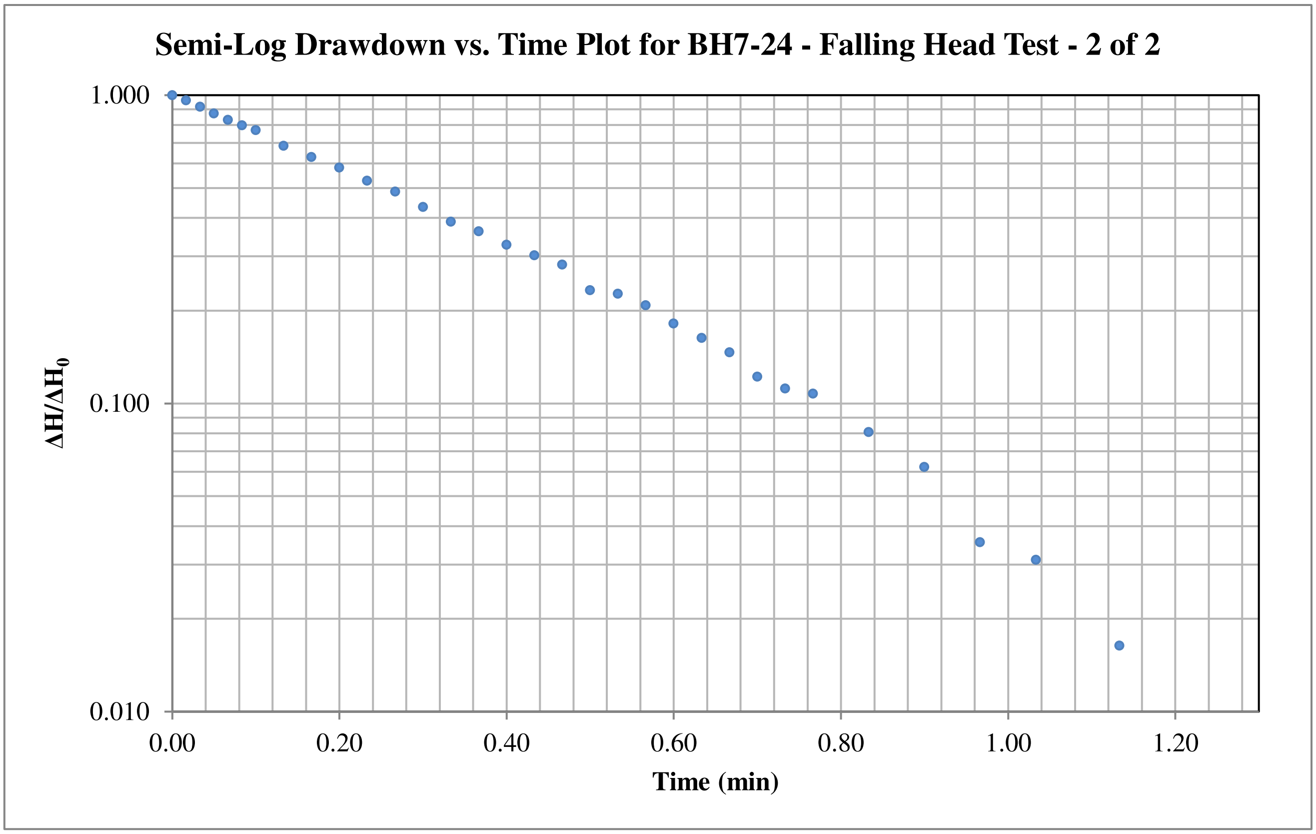
Data Points (from plot):

t*: 0.315 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.01E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Brigil - 265 Catherine Street
Test Location: BH7-24
Test: Falling Head - 2 of 2
Date: March 14, 2024



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: 2.07207

Well Parameters:

L	1.5 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

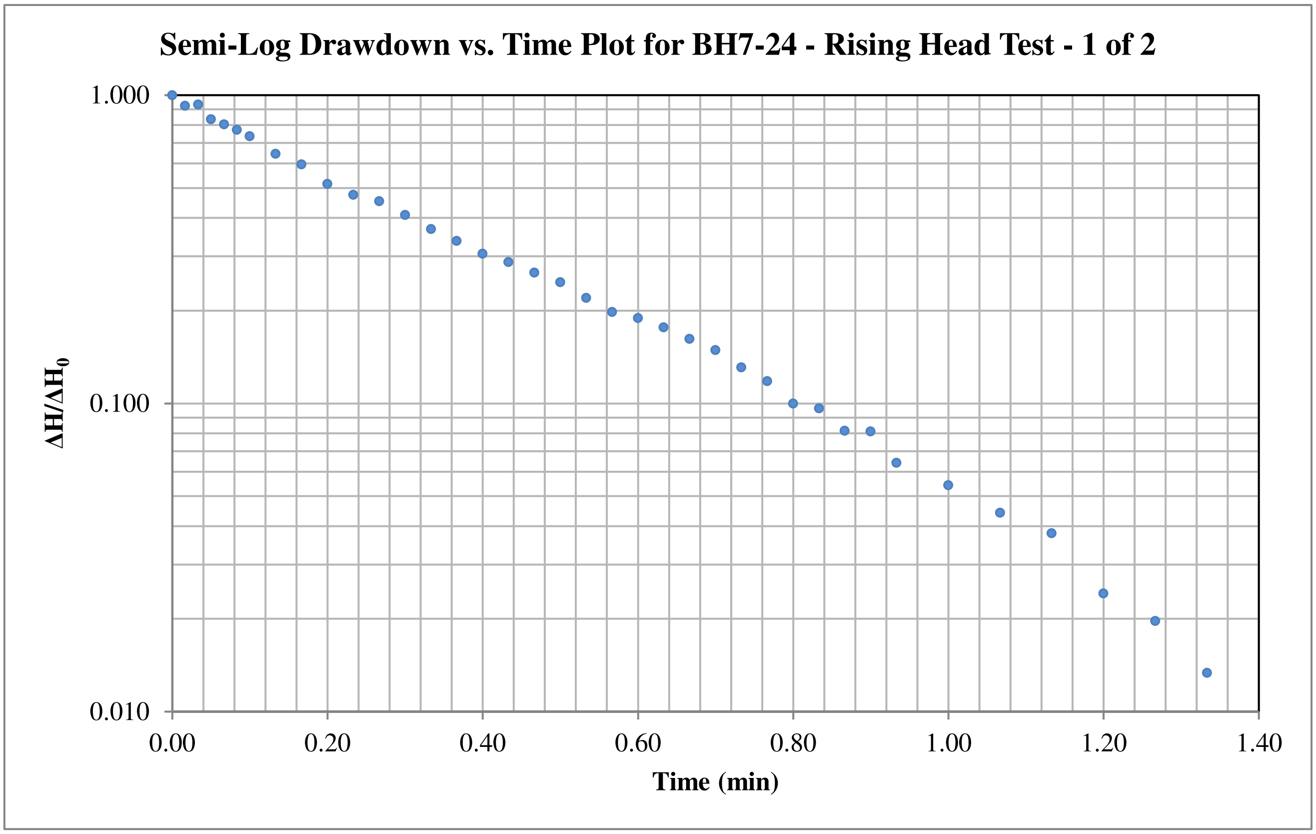
Data Points (from plot):

t*: 0.357 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.77E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Brigil - 265 Catherine Street
Test Location: BH7-24
Test: Rising Head - 1 of 2
Date: March 14, 2024



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: 2.07207

Well Parameters:

L	1.5 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

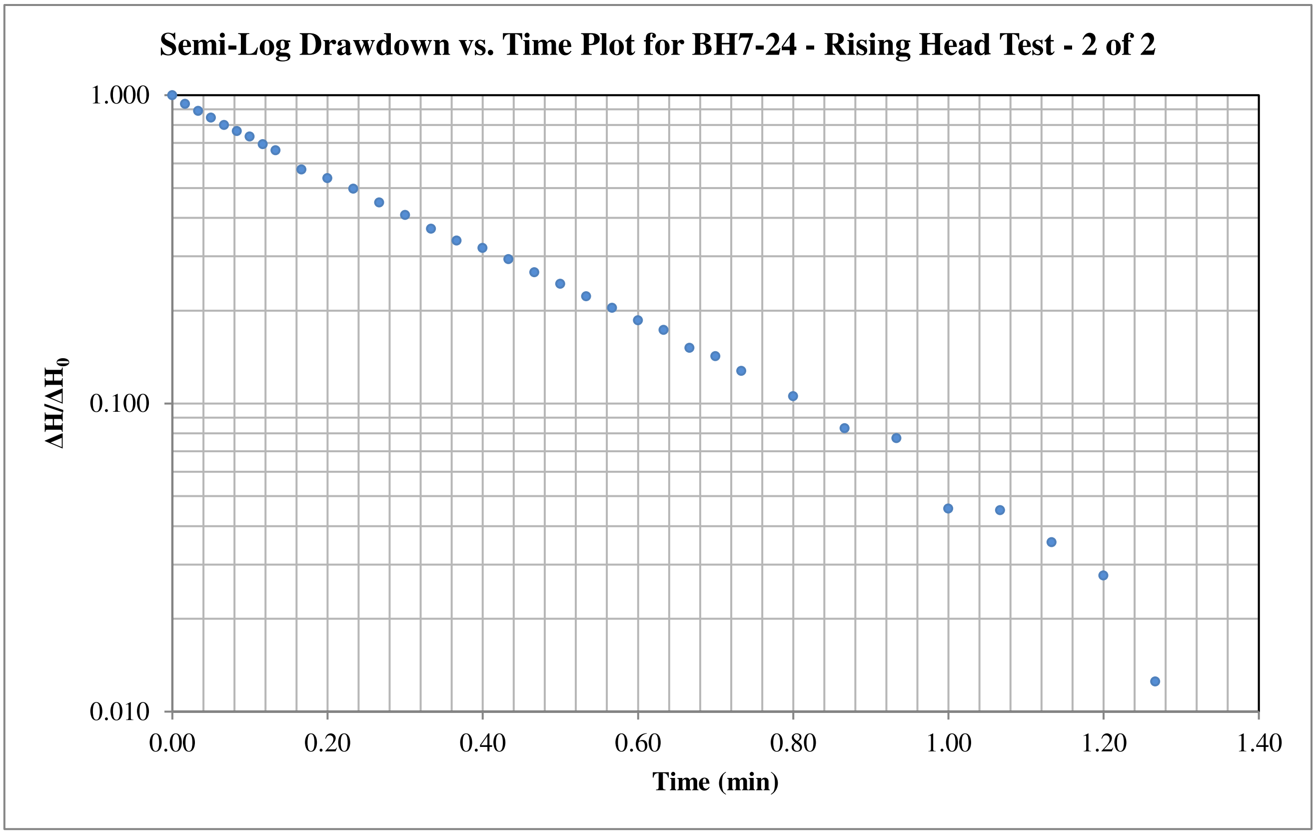
Data Points (from plot):

t*: 0.332 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.91E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Brigil - 265 Catherine Street
Test Location: BH7-24
Test: Rising Head - 2 of 2
Date: March 14, 2024



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: 2.07207

Well Parameters:

L	1.5	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

Data Points (from plot):

t*: 0.333 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.90E-05 m/sec

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 30690

Report Date: 26-Aug-2020

Order Date: 20-Aug-2020

Project Description: PE2703

	Client ID:	BH3-20 SS4	-	-	-
	Sample Date:	19-Aug-20 09:00	-	-	-
	Sample ID:	2034480-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	59.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.40	-	-	-
Resistivity	0.10 Ohm.m	3.33	-	-	-
Anions					
Chloride	5 ug/g dry	1780	-	-	-
Sulphate	5 ug/g dry	398	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5933-1 - TEST HOLE LOCATION PLAN

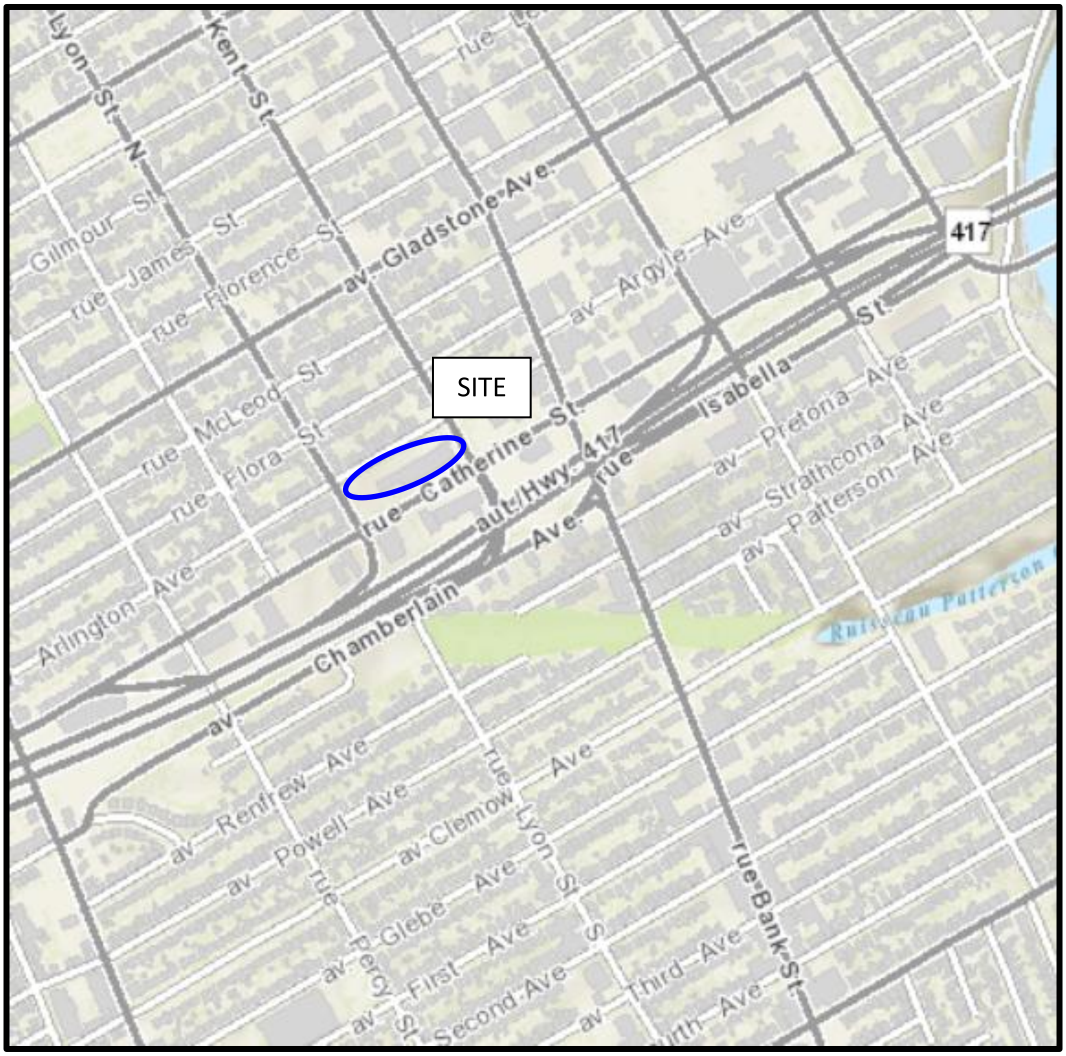


FIGURE 1

KEY PLAN

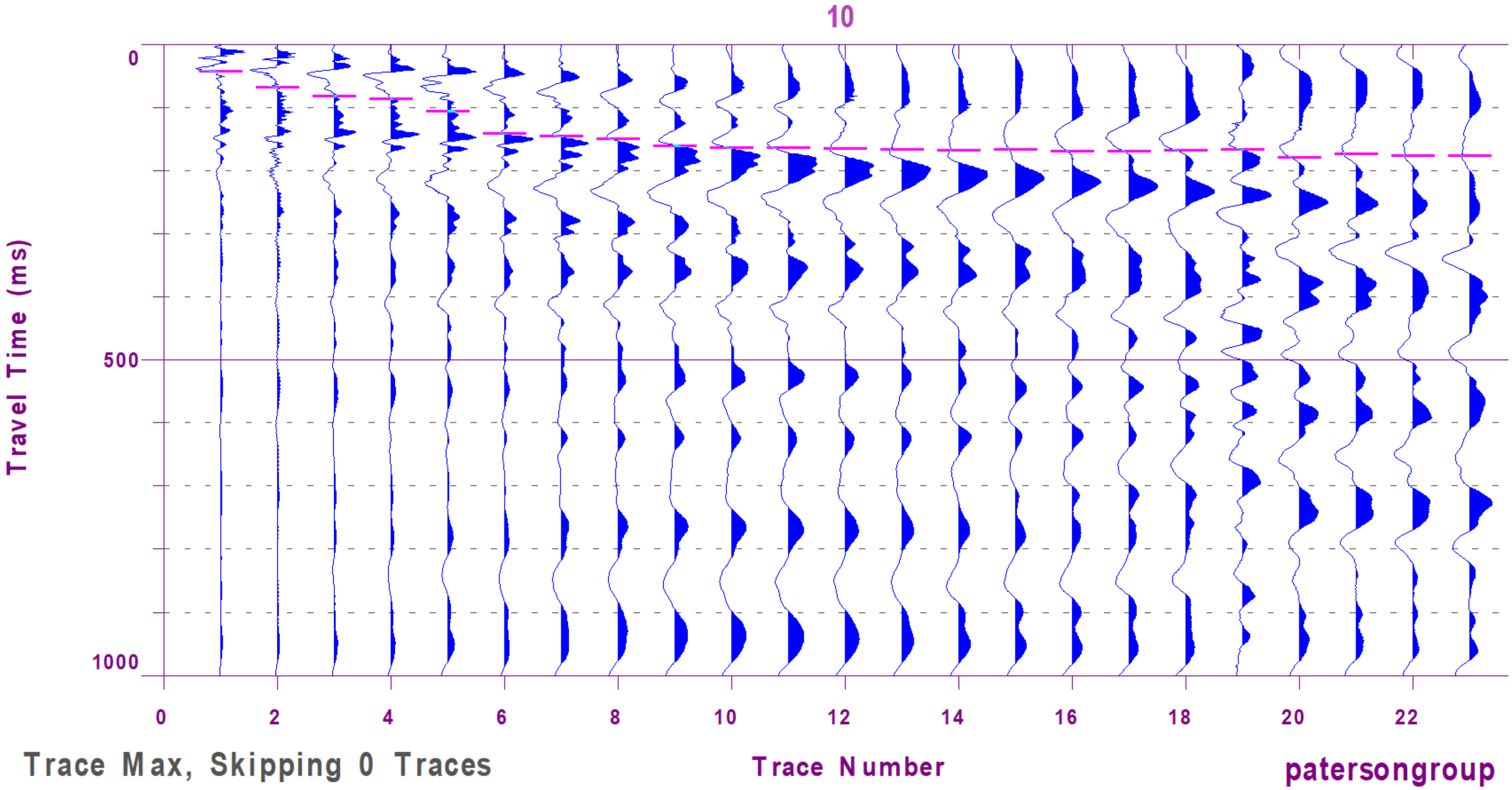


Figure 2 – Shear Wave Velocity Profile at Shot Location -2 m

PATERSON GROUP

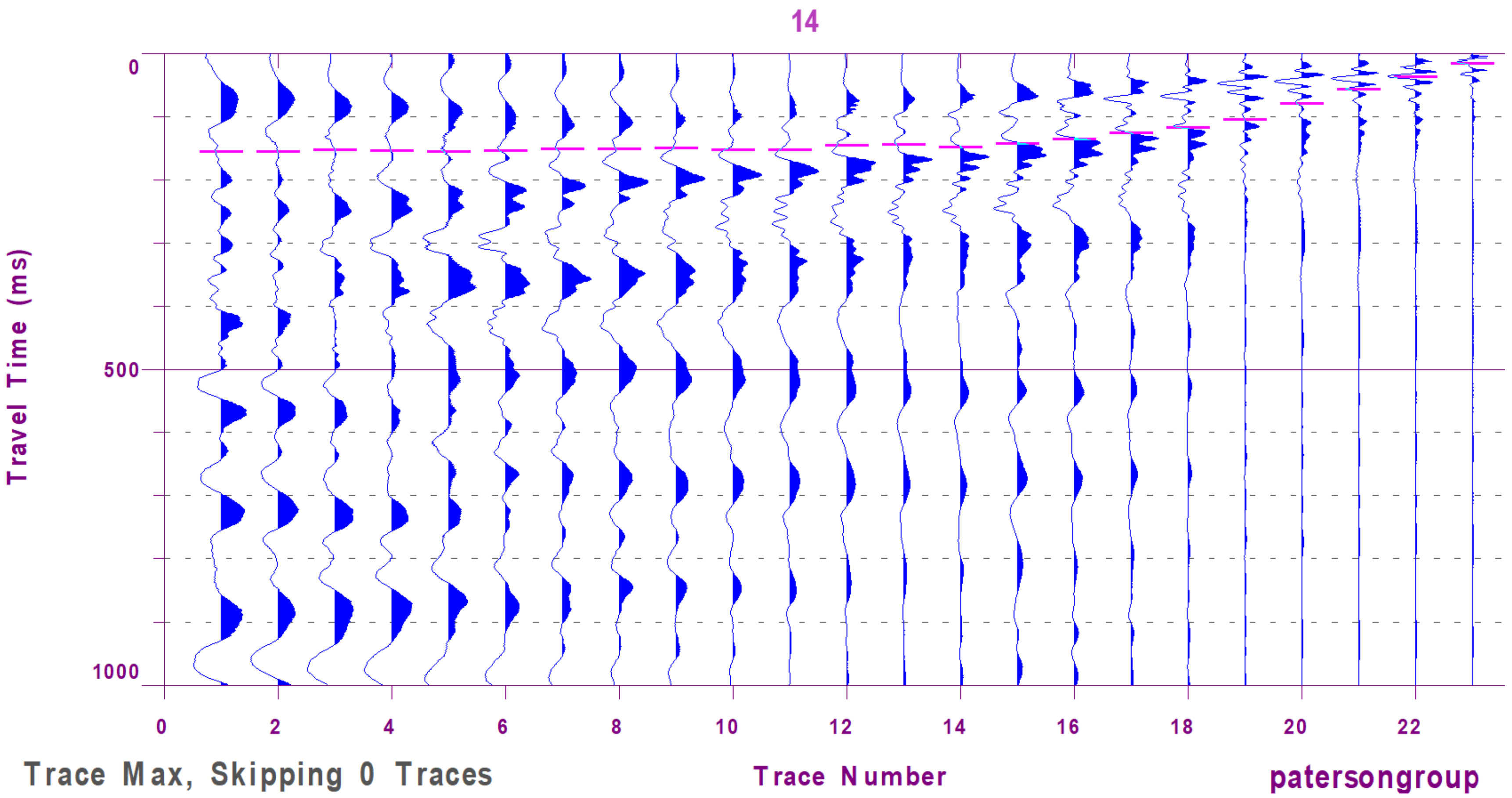
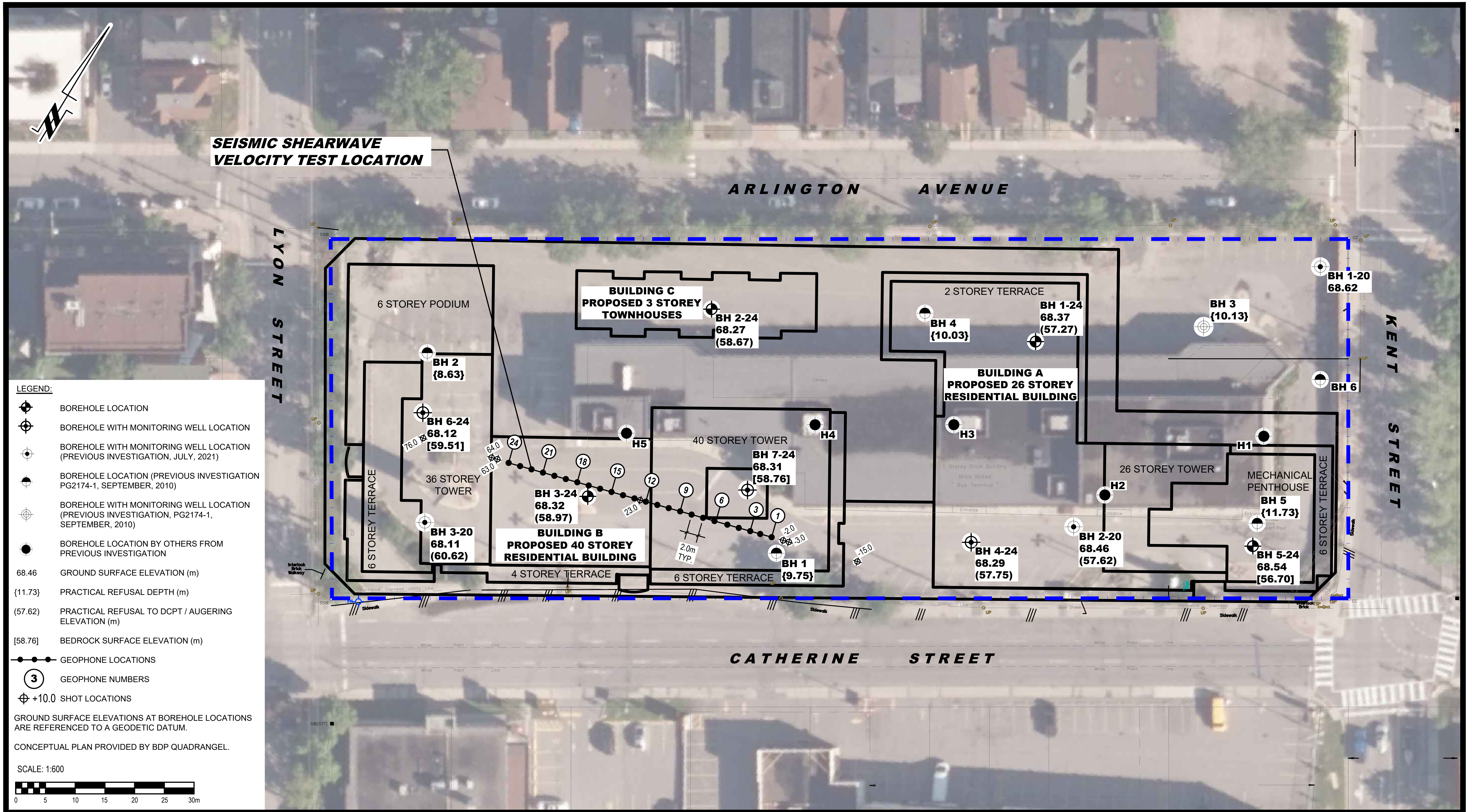


Figure 3 – Shear Wave Velocity Profile at Shot Location 48 m

PATERSON GROUP



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION (PREVIOUS INVESTIGATION, JULY, 2021)
◒	BOREHOLE LOCATION (PREVIOUS INVESTIGATION PG2174-1, SEPTEMBER, 2010)
⌖	BOREHOLE WITH MONITORING WELL LOCATION (PREVIOUS INVESTIGATION, PG2174-1, SEPTEMBER, 2010)
●	BOREHOLE LOCATION BY OTHERS FROM PREVIOUS INVESTIGATION
68.46	GROUND SURFACE ELEVATION (m)
{11.73}	PRACTICAL REFUSAL DEPTH (m)
(57.62)	PRACTICAL REFUSAL TO DCPT / AUGERING ELEVATION (m)
[58.76]	BEDROCK SURFACE ELEVATION (m)
—●—●—●—	GEOPHONE LOCATIONS
③	GEOPHONE NUMBERS
⌖ +10.0	SHOT LOCATIONS

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

CONCEPTUAL PLAN PROVIDED BY BDP QUADRANGEL.

SCALE: 1:600

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

NO.	REVISIONS	DATE	INITIAL
1	BH 1-24 TO BH 7-24 ADDED, UPDATED CONCEPTUAL PLAN, ADDED SEISMIC SHEAR WAVE VELOCITY TEST LOCATION	27/03/2024	JV

BRIGIL (11034936 CANADA INC.)
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED USE DEVELOPMENT
265 CATHERINE STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600
Date: 07/2021
Drawn by: RCG
Report No.: PG5933-1
Checked by: BN
Dwg. No.: PG5933-1
Approved by: JV
Revision No.: 1

11x17

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