patersongroup

consulting engineers

memorandum

APPROVED
By Allison Hamlin at 1:08 pm, May 05, 2022

re: **Geotechnical Response to City Comments - Third Submission**
Proposed Commercial Development
8800 Campeau Drive - Ottawa

to: Maritime Ontario c/o Realinc-SCS - **Mr. David Plumb** - dplumb@r

date: November 18, 2021

file: PG5618-MEMO.05

AHamlin

ALLISON HAMLIN
MANAGER (A), DEVELOPMENT REVIEW WEST
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DEPARTMENT, CITY OF OTTAWA

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide geotechnical responses to the third submission city comments as well as comments resulting from the October 15, 2021 meeting with the City of Ottawa regarding the proposed commercial building at the aforementioned site. This memorandum should be read in conjunction with Paterson Geotechnical Report PG5618-1 Revision 4 dated November 24, 2021 and Paterson Memorandums PG5618-MEMO.01 Revision 1 dated May 11, 2021 and PG5618-MEMO.04 dated July 27, 2021.

Comments from October 15, 2021 Meeting with City of Ottawa

Comment 1: *As discussed at the meeting on Friday October 15th, the design package will be revised to provide more details to justify the long-term groundwater elevation and whether the interim groundwater* ***elevation will result in groundwater conveyed to the downstream storm sewer system via the proposed LID practice. Please read through the following and ensure that the signed and stamped report from the geotechnical engineer addresses the concerns raised below:***

Paterson Response: Upon review of the civil drawings, it is understood that the base of the proposed infiltration gallery will be located at an approximate geodetic elevation of 103.6 m and that the outlet pipe (STM MH 304), is to have an invert elevation of 104.2 m which connects to STM MH 200 before entering into the City storm sewer. It should be noted that it is expected that the bedding layer below this storm pipe alignment will provide a sufficient outlet for any water currently perched at a higher elevation in the area of the infiltration gallery. The invert of the storm pipe at STM MH 200 is 102.0 m elev., which is the expected long-term elevation of the groundwater level in this area upon completion of the development.

It should be further noted that any pre-development groundwater, which has established above design invert level will be pumped out of the open excavation during construction to enable proper compaction of the bedding layer for the service pipes and placement of the service pipe and infiltration gallery. The dewatering work during construction is expected to remove the perched water within the subject site.

It is acknowledged that the elevation of the post-development long-term water table may not be accurately predicted in areas outside of the radius of influence of permanent structures and site servicing. However, based on the proximity of the proposed infiltration gallery to the storm sewer and the hydraulic conductivity of the silty sand to sandy silt encountered across the subject site, it is anticipated that the long-term groundwater table across the subject site will be lowered to the approximate invert elevation of the storm sewer which ranges from approximately 101.2 to 100.9 m along the adjacent section of Upper Canada Street.

Based on the above, long-term and seasonal high groundwater level under post-development conditions is not expected to be conveyed to the downstream storm sewer system and SWMF. The current revision of our report has been updated to include the abovenoted discussion in Subsection 6.5.

Comment 1 Continued:

- *It is not clear whether the MWs were constructed properly given the following response from the Geotech Engineer:*

  "section 4.3 Groundwater of Paterson Group Report PG5618 Revision 3, dated May 11, 2021… Groundwater Levels were measured periodically throughout the spring months at the monitoring well constructed at borehole BH 1-21 as well as within the flexible piezometers installed in the remaining boreholes, seasonal high groundwater was generally encountered at geodetic elevations ranging from 105.2 to 106.0. However, it is anticipated that the elevation groundwater levels observed within boreholes BH 1-21 through BH 3-21 have been influenced by surface water collected within the adjacent drainage ditch which runs along the western limit of the subject site. Therefore, groundwater levels are expected to be lowered within the immediate are of the drainage ditch if a culvert is installed to allow flow to continue toward the adjacent ditch along the access road".

Paterson Response: The monitoring well at borehole BH 1-21 was constructed as per the typical detail described below:

- Slotted 51 mm diameter PVC screen at the base of the borehole.
- 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No. 3 silica sand backfill within annular space around screen.
- Bentonite hole plug directly above PVC slotted screen.
- Clean Backfill from top of bentonite plug to the ground surface.

The groundwater observation detailed in the excerpt above is noting the influence of poor surface drainage that is present due to the blocked outlet of the adjacent drainage ditch. The well construction is as per industry standard and acceptable. However, the pre-development groundwater level noted in this area is being influenced by this poor drainage and it is expected that development of the subject block will have a great reduction on the surface water that is being collected within this existing drainage ditch, so Paterson has noted this impact since it is relevant to the discussion seasonally high groundwater levels.

- ❑ *It appears that there is only one MW (BH 1-21). The rest of the data was collected from the flexible pipes in BHs, which is not an acceptable method for determining the water table. Properly constructed, screened and sealed MWs should always be used.*

Paterson Response: Groundwater levels were measured periodically throughout the spring at the monitoring well installed at borehole BH 1-21 as well as the piezometers installed in the remaining boreholes. As per the Geotechnical Investigation and Reporting Guidelines for Development Applications in the City of Ottawa, piezometers consisting of HDPE or PVC tubing with slotted opening can be installed in boreholes to enable monitoring of the groundwater levels. In addition to recorded groundwater levels, the long-term groundwater level can also be inferred from the colour and water content of the samples.

- ❑ *It is not possible to predict the post-construction water table. The installation of sewers and the reduction in recharge will likely lower the water table but we do not know by how much. Furthermore, depending on the hydraulic conductivity, the lowering of the water table due to sewers could be localized around the sewers, and may not be throughout the development.*

Paterson Response: It is acknowledged that the elevation of the post-development water table will be difficult to accurately determine. However, the post-development water table in the vicinity of the proposed structures and utilities will be governed by their invert elevations at a given location based on past experience with developments in similar soil conditions. Therefore, it is anticipated that the post-development water table at the proposed LID practice location will be governed the invert level of the nearby existing services located along Upper Canada Street.

- ❑ *Consultants should be following the City's LID Technical Guidance Report, prepared in discussions with the consulting industry, CAs and MECP. Note that Section 2.3.3 of this reports that **pre-development** water table elevations should be used in design. The report also describes the MWs and how to take water levels.*

Paterson Response: This comment has been acknowledged.

Best Regards,

Paterson Group Inc.

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Geotechnical Investigation

Proposed Commercial Development
Kanata West Business Park - Block 4
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Prepared For

Maritime Ontario

November 24, 2021

Report: PG5618-1
Revision 4

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Maritime Ontario to conduct a geotechnical investigation for the proposed warehouse/office to be located at 8800 Campeau Drive in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2). The objective of the investigation was to:

- ❑ determine the subsoil and groundwater conditions at this site by means of a borehole program, supplemental to the original geotechnical investigation completed at the subject site.

- ❑ provide geotechnical recommendations for 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. The report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the proposed development as understood at the time of this report.

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed commercial development will consist of a 2-storey office building with one underground level which is connected to an approximate 5,600 m^2, 1-storey warehouse of slab-on-grade construction. It is further understood that an infiltration gallery is to be located at the eastern limit of the subject site and that the majority of the parking areas and access lanes within the development will consist of a roller compacted concrete structure. The site is also anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

A previous investigation was completed at the subject site by this firm in January 2014. At that time, a total of 5 boreholes were advanced at the subject site to a maximum depth of 6.7 m. A supplemental geotechnical investigation was conducted by Paterson on March 15, 2021. At that time, a total of 3 boreholes were advanced at the subject site to a maximum depth of 6.1 m.

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

The borehole locations were distributed in a manner to provide general coverage of the subject site taking into consideration existing site features and underground utilities. The approximate locations of the boreholes are shown on Drawing PG5618-1 - Test Hole Location Plan included in Appendix 2.

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 and subsequently placed in sealed plastic bags. All samples were 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 in Appendix 1.

A Standard Penetration Test (SPT) was conducted at each borehole 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.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at borehole BH 1 and BH 2-21. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

As part of the supplemental investigation, a 51 mm diameter groundwater monitoring well was installed in one borehole to monitor the groundwater level subsequent to the completion of the sampling program. The remaining two boreholes conducted during the supplemental investigation were fitted with flexible piezometers to allow groundwater level monitoring. As part of the previous geotechnical investigation, the observed groundwater levels were recorded in the field at the time of drilling. Ground observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All samples from the supplemental investigation will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless otherwise directed.

3.2 Field Survey

The test hole locations for the previous geotechnical investigation were selected by Paterson and laid out in the field by Stantec Geomatics. The test hole locations for the supplemental investigation were selected by Paterson and surveyed in the field by Paterson personnel.

The ground surface elevation at each test hole location are referenced to a geodetic datum. The location of the test holes and ground surface elevations at each test hole location are presented on Drawing PG5618-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the field investigations were visually examined in our laboratory to review the results of the field logging. A total of two (2) grain size distribution analyses (sieve analysis) were completed on selected soil samples. The results of the testing are presented in Subsection 4.2 and on Grain Size Distribution Testing sheets presented in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against the subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant, grass covered and has been stripped of trees. Ponding water was noted at several locations across the site during the current field investigation. A gravel finished access road formerly ran across the central portion of the site, running southwest to northeast. Fill piles were previously observed in the northeast corner of the property. A gravel covered access roadway was observed running along the west property boundary. A drainage ditch running north-south was noted to contain approximately 0.3 m of water and abutted the access roadway, however, no culvert was observed, which would connect the drainage ditch within the site to the east side of the access roadway.

The subject site is bordered to the north by agricultural lands to the east by future Upper Canada Street and vacant properties, to the south by Campeau Drive and to the west by commercial aggregate quarrying operations. The existing ground surface across the subject site is relatively flat with an approximate geodetic elevation of 105 to 106 m.

4.2 Subsurface Profile

Generally, the soil profile encountered at the test hole locations consists of a topsoil layer underlain by fill material composed of brown silty clay with sand, gravel and trace topsoil at boreholes BH 1-21, BH 2-21 and BH -21. A compact to very loose, silty sand to sandy silt deposit was encountered underlying the topsoil or fill at all boreholes. A glacial till deposit, consisting of grey silty sand, with gravel, cobbles and boulders, was encountered underlying all boreholes with the exception of BH 18 and BH 3-21.

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

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of interbedded limestone and shale of the Verulam formation as well as interbedded limestone and dolomite of the Gull River formation with an overburden thickness of 5 to 15 m.

Grain Size Distribution Sieve Analysis

Grain size distribution (sieve analysis) was also completed on two (2) selected soil samples. The results of the grain size analysis are summarized in Table 2 and presented on the Grain-Size Distribution Testing Results sheets in Appendix 1.

Table 1 - Summary of Grain Size Distribution Analysis				
Test Hole	**Sample**	**Gravel (%)**	**Sand (%)**	**Silt and Clay (%)**
BH 1-21	SS5	0.0	12.4	87.6
BH 3-21	SS6	1.0	38.4	60.6

4.3 Groundwater

Groundwater level readings were recorded on March 31, April 12 and May 7, 2021. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1 and summarized in Table 2. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

Based on available design plans, the proposed finished floor elevation in the office is 108.2 m and the underground level underside of footing elevation is 3.7 m below the finished floor elevation, corresponding to a geodetic elevation of 104.5 m.

Table 2 - Summary of Groundwater Levels				
Borehole Number	**Ground Elev. (m)**	**Groundwater Levels (m)**		**Recording Date**
		Depth	**Elevation**	
BH 1-21	105.80	0.30	**105.50**	Mar. 31, 2021
		0.65	**105.15**	April 12, 2021
		0.57	**105.23**	May 7, 2021
BH 2-21	106.18	0.68	**105.50**	Mar. 31, 2021
		0.73	**105.45**	April 12, 2021
		0.68	**105.50**	May 7, 2021

Table 2 - Summary of Groundwater Levels cont.

BH 3-21	106.59	0.62	**105.97**	Mar. 31, 2021
		0.93	**105.66**	April 12, 2021
		0.86	**105.73**	May 7, 2021
BH 1	105.99	1.5	104.49	Jan. 15, 2014
BH 2	104.99	1.8	103.19	Jan. 15, 2014
BH 17	105.68	2.0	103.68	Jan. 15, 2014
BH 18	105.53	1.5	104.03	Jan. 15, 2014
BH 19	105.08	2.0	103.08	Jan. 15, 2014

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is expected that the proposed warehouse and office building will be founded on conventional shallow footings bearing on an undisturbed silty sand to sandy silt and/or glacial till bearing surface.

A waterproofing and drainage system are recommended for the office building with the basement level. Details are provided in Subsection 6.1. It is understood that the basement level will be located below the storm sewer invert level, therefore, a sump pump system will be required to provide an outlet for the discharge water. Based on the current groundwater level and hydraulic conductivity of the underlying soil, a sump pit chamber with main pump and back-up system should be installed at the opposite ends of the basement area to ensure that water levels can be maintained during spring melt and after significant precipitation events. It is recommended that the sump pumps be connected to a natural-gas powered generator. It is further recommended that the primary pumps are connected to individual, dedicated breakers to mitigate the risk of tripping.

The 2 sump pit chambers with main pump and backup systems should be sized such that the groundwater level beneath the basement footprint is maintained at a maximum elevation of 20 cm below the underside of footing. Further, the 2 sump pump systems should be located in opposite ends of the basement area and located such that dewatering across the basement area is achieved uniformly beneath the basement area footprint. A minimum flow rate of 100,000 L/day, or 50,000 L/day per pump, should be used in sizing of the sump pump and backup systems. As the proposed basement level is to be founded below the groundwater table, dewatering of the excavation footprint will be required to maintain a suitable bearing surface during construction.

Where the footing subgrade consists of silty sand which is observed to be in a loose state of compactness, the material should be proof compacted using suitable vibratory equipment making several passes under dry conditions and above freezing temperatures and which is approved by Paterson at the time of construction.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. It is anticipated that existing fill within the proposed building footprint, free of deleterious material and significant amounts of organics, and approved by the geotechnical consultant at the time of construction can be left in place below the proposed building footprint outside of lateral support zones for the footings. However, it is recommended that the existing fill layer be proof-rolled by a vibratory roller making several passes under dry and above freezing conditions and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

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. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified fill along with 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 at least 95% of their respective SPMDD.

Non-specified fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Conventional Spread Footings

Footings placed directly on an undisturbed, compact silty sand to sandy silt bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Footings placed on an undisturbed glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **350 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings placed on a soil bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

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

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**. If a higher seismic site class is required (Class C), a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building addition, as presented in Table 4.1.8.4.A of the Ontario Building Code 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the existing fill, silty sand to sandy silt, or glacial till subgrade approved by the geotechnical consultant at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade or basement slab construction. Where the subgrade consists of existing fill or silty sand to sandy silt, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as Granular B Type II.

For slab-on-grade areas, it is recommended that the upper 200 mm of sub-slab fill consist OPSS Granular A crushed stone. For basement slabs, it is recommended that the upper 200 mm of sub-floor fill consist of 19 mm clear crushed stone

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should also be provided under the basement slab areas. This is discussed further 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 18 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.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g 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 Structure

Car only parking areas, access lanes and heavy truck parking/loading areas are anticipated at this site. It is understood that a roller compacted concrete (RCC) pavement structure is planned for the proposed development. Our recommended pavement structure for a roller compacted concrete is presented in Table 3.

Table 3 - Recommended Roller Compacted Concrete Pavement Structure Heavy Truck Access Lanes, Parking and Loading Areas	
Thickness (mm)	**Material Description**
200	**Zero slump, no air entrainment -32 MPa at 28 day strength - 12 to 13% cement content**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

The compacted concrete must be compacted to a minimum 100% of its SPMDD and frequently tested by Paterson personnel. A well graded aggregate blend with a maximum particle size of 19 mm is further recommended, as well as, a water-cement ratio of 0.25 to 0.35. Moisture control during curing is critical for the RCC pavement structure. To control cracking, it is recommended that strategically located saw cuts be used to create control joints at a 8 to 10 m spacing. The joints should be cut between 25 and 30% of the thickness of the concrete slab.

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 should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure - Flexible Pavement Structure

Recommended flexible pavement structures are shown in Tables 4 and 5. Minimum Performance Graded (PG) 58-34 asphalt cement should be used where a flexible pavement structure is to be used for this project.

Table 4 - Recommended Flexible Pavement Structure - Car Only Parking Areas	
Thickness (mm)	**Material Description**
50	**Wear Course** - HL 3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 5 - Recommended Flexible Pavement Structure - Access Lanes and Heavy Truck Parking/loading 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	

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 100% of the material's SPMDD using suitable vibratory equipment.

5.8 Hydraulic Conductivity Testing

Hydraulic conductivity testing was completed at borehole BH 1-21 which was outfitted with a monitoring well and screened within the silty sand to sandy silt and glacial till layers. Rising head and falling head testing ("slug testing") was completed within the soil strata 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 length of 3 m and the well diameter of 0.051 m at borehole BH 1-21.

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.

Based on the above test methods, the monitoring wells screened in the overburden soils displayed a hydraulic conductivity value ranging from **4.6 x 10^{-6} to 1.34 x 10^{-5} m/sec**. The values measured within the monitoring well are generally consistent with similar material Paterson has encountered on other sites and typical published values for silty sand and sandy silt. These values typically range from 1 x 10^{-4} to 1 x 10^{-6} m/sec. The results from the hydraulic conductivity testing are attached to the current report.

5.9 Percolation Rates

Infiltration galleries are anticipated to be located beneath the roller compacted concrete pavement structure within the subject site. Paterson completed hydraulic conductivity testing at borehole BH 1-21 in order to establish hydraulic conductivity and percolation time of in-situ materials.

It is anticipated that a silty sand to sandy silt will be encountered at the base of the infiltration galleries during the installation and will affect the rate of stormwater infiltration into the underlying material. The results of the hydraulic conductivity testing were used to determine the estimated percolation rates of the in-situ soils using the approximate relationship between infiltration rate and hydraulic conductivity, as described in the Draft LID Guidance Document, dated December 2019. Based on this relationship, the estimated percolation rate (T-Time) was estimated to be within the ranges in Table 6.

Table 6 - Estimated Percolation Rates

Material	Hydraulic Conductivity - k (m/sec)	Percolation (T-time) - (mins/cm)
Silty Fine Sand to Sandy Silt	**1.3×10^{-5} to 4.6×10^{-6}**	**6 to 10**

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The outlet pipe should have a positive outlet to one of the sump pit basins, which will discharge to the storm sewer.

It is also recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter sub-slab drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the basement area.

Where a basement level is in place, waterproofing of the foundation walls is recommended and the membrane is to be installed from 1.5 m below finished grade down the foundation walls to the underside of the footing. It is expected that sufficient room will be available for the greater part of the basement excavation to be undertaken by open-cut methods.

Sub-slab Drainage

Sub-slab drainage is also recommended in order to control groundwater infiltration for basement area. For preliminary design purposes, we recommend that 150 mm diameter perforated pipes be placed at approximate 3 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.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m thick soil cover should be provided for adequate frost protection of heated structured, or an equivalent combination of soil cover and foundation insulation.

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

6.3 Excavation Side Slopes

The side slopes of the excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly 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.

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's SPMDD.

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

Where hard surface ares are considered above the trench backfill, the trench backfill material within the frost zone, (about 1.5 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 lifts and compacted to 95% of the materials SPMDD.

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 bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

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

Long-Term Groundwater Lowering

Lowering of the perched groundwater level noted under pre-development conditions at BH 1-21, BH 2-21 and BH 3-21 is anticipated during development of the subject site. The percolation rates presented in Table 6 support the concept that this groundwater lowering process will take place during the initial installation of the proposed servicing alignments. In review of the current civil drawings, it was noted that the base of the proposed infiltration gallery will be located at a geodetic elevation of 103.86 m and that the outlet pipe (STM MH 304), is to have an invert elevation of 104.2 m, which connects to STM MH 200 at the southeast corner of the site before entering into the City storm sewer. It is expected that the bedding layer below this storm pipe alignment will provide a sufficient outlet for any water currently perched at a higher elevation in the area of the infiltration gallery. The invert of the storm pipe at STM MH 200 is 102. m elev., which is the expected long-term elevation of the groundwater level in this area upon completion of the development.

It is acknowledged that the elevation of the post-development long-term water table may not be accurately predicted in areas outside of the radius of influence of permanent structures and site servicing. However, based on the proximity of the proposed infiltration gallery to the storm sewer and the hydraulic conductivity of the silty sand to sandy silt encountered across the subject site, it is anticipated that the long-term groundwater table across the subject site will be lowered to the approximate invert elevation of the storm sewer which ranges from approximately 101.2 to 100.9 m along the adjacent section of Upper Canada Street.

Based on the above, long-term and seasonal high groundwater level under post-development conditions is not expected to be conveyed to the downstream storm sewer system and SWMF.

6.6 Winter Construction

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

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

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

One (1) sample was submitted for testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.1%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the resistivity indicate the presence of a moderately aggressive to aggressive environment for exposed ferrous metals at this site. It is anticipated that standard measures for corrosion protection are sufficient for services placed within the subject site.

7.0 Recommendations

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

- ❑ A review of the site grading plan(s) from a geotechnical perspective, once available.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ 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 placing backfilling materials.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

The client should be aware that any information pertaining to soils and the test hole logs are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

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 to be notified immediately in order to permit reassessment of the recommendations.

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 Maritime Ontario or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
December 13, 2021
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Kevin A. Pickard, EIT

David J. Gilbert, P.Eng.

Report Distribution:

❑ Maritime Ontario
❑ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION SIEVE ANALYSIS RESULTS

ANALYTICAL TEST RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development
8800 Campeau Drive - Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 March 15

FILE NO. PG5618

HOLE NO. BH 1-21

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.80
TOPSOIL	0.08						
FILL: Brown silty clay trace sand and gravel	0.46	AU	1				
Compact brown **SILTY SAND** to **SANDY SILT**		SS	2	58	23	1	104.80
	1.83	SS	3	33	13		
Compact to loose grey **SILTY SAND** to **SANDY SILT**						2	103.80
		SS	4	50	17		
						3	102.80
		SS	5	42	9		
						4	101.80
	4.52	SS	6	33	5		
GLACIAL TILL: Grey silty sand to sandy silt some gravel, trace clay, cobbles and boulders		SS	7	50	1	5	100.80
	5.94	SS	8	50	33		
End of Borehole (GWL @ 0.30 m depth - March 31, 2021)							

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development
8800 Campeau Drive - Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

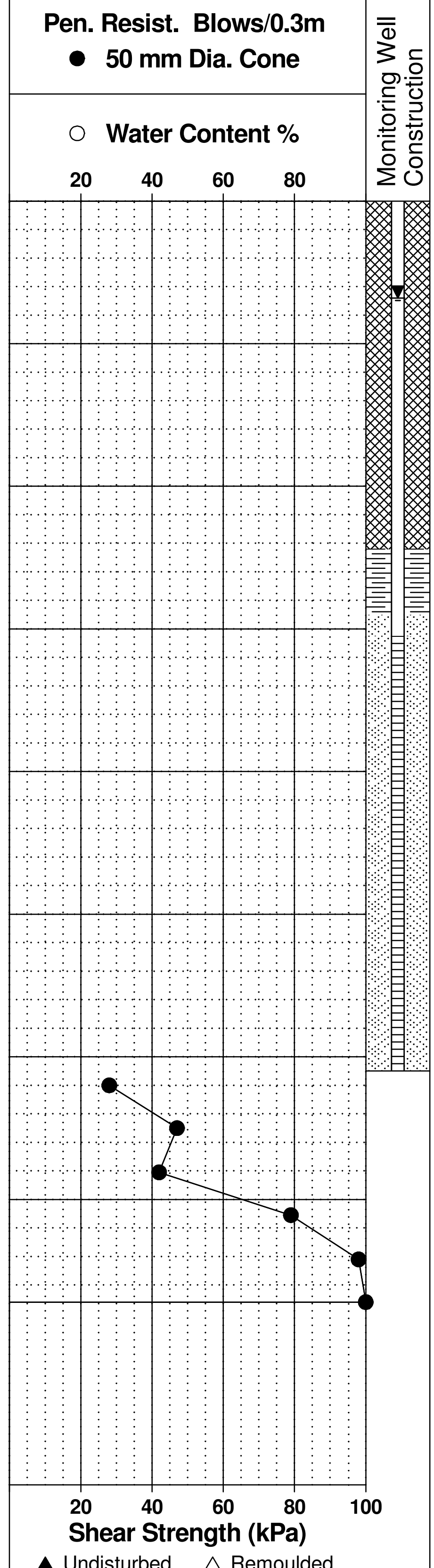
DATE 2021 March 15

FILE NO. PG5618

HOLE NO. BH 2-21

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.10	AU	1		
FILL: Brown silty clay with sand and gravel	0.36				
Loose to compact brown **SILTY SAND** to **SANDY SILT**		SS	2	25	6
	2.13	SS	3	42	18
Loose to firm grey **SILTY SAND** to **SANDY SILT**		SS	4	42	9
		SS	5	50	9
		SS	6	50	6
		SS	7	42	11
	5.64				
GLACIAL TILL: Compact grey silty sand to sandy silt some gravel, trace clay, cobbles and boulders	6.10	SS	8	58	20
Dynamic Cone Penetration Test commenced at 6.10 m depth.	7.72				
End of Borehole					
Practical refusal to DCPT at 7.72 m depth					
(GWL @ 0.68 m depth - March 31, 2021)					

DEPTH (m)	ELEV. (m)
0	106.18
1	105.18
2	104.18
3	103.18
4	102.18
5	101.18
6	100.18
7	99.18



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development
8800 Campeau Drive - Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 March 15

FILE NO. PG5618

HOLE NO. BH 3-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.08	AU	1		
FILL: Brown silty clay with sand, trace gravel and topsoil	0.30				
Compact brown **SILTY SAND** to **SANDY SILT**		SS	2	25	3
- Dense by 1.7 m depth	2.13	SS	3	67	20
Compact to loose grey **SILTY SAND** to **SANDY SILT**		SS	4	50	12
		SS	5	50	11
		SS	6	50	5
		SS	7	33	8
	5.94	SS	8	67	W
End of Borehole					
(GWL @ 0.62 m depth - March 31, 2021)					

DEPTH (m)	ELEV. (m)
0	106.59
1	105.59
2	104.59
3	103.59
4	102.59
5	101.59

STRATA PLOT

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

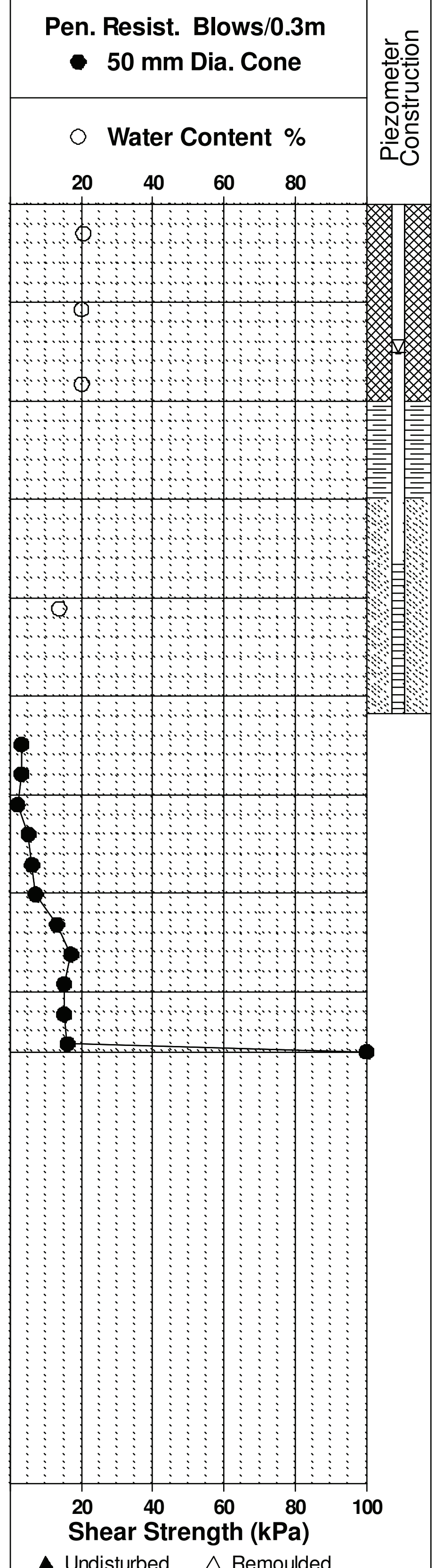
BORINGS BY CME 55 Power Auger

DATE January 15, 2014

FILE NO. PG3115

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.99
TOPSOIL	0.28	AU	1				
Compact, brown **SILTY FINE SAND**		SS	2	71	11	1	104.99
- grey by 1.5m depth		SS	3	54	11	2	103.99
	2.97	SS	4	25	13		
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders		SS	5	58	3	3	102.99
		SS	6	67	17	4	101.99
	5.18	SS	7	50	3	5	100.99
Dynamic Cone Penetration Test commenced at 5.18m depth.						6	99.99
Inferred **GLACIAL TILL**						7	98.99
	8.61					8	97.99
End of Borehole							
Practical DCPT refusal at 8.61m depth							
(GWL @ 1.5m depth based on field observations)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

FILE NO. PG3115

REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger **DATE** January 15, 2014

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	104.99
TOPSOIL 0.36		AU	1				
Brown **SILTY CLAY,** trace sand 0.69		AU	2				
		SS	3	100	14	1	103.99
		SS	4	83	12	2	102.99
Compact, brown **SILTY FINE SAND**		SS	5	71	8	3	101.99
- grey by 3.0m depth		SS	6	58	9		
		SS	7	54	6	4	100.99
		SS	8	58	10	5	99.99
6.02		SS	9	58	12	6	98.99
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders, trace clay 6.70		SS	10	67	24		
End of Borehole							
(GWL @ 1.8m depth based on field observations)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

FILE NO. PG3115

REMARKS

HOLE NO. BH17

BORINGS BY CME 55 Power Auger

DATE January 15, 2014

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.68
TOPSOIL 0.30							
		SS	1	83	10	1	104.68
		SS	2	54	8	2	103.68
Loose, grey **SILTY FINE SAND**		SS	3	67	2		
		SS	4	62	3	3	102.68
		SS	5	67	8	4	101.68
		SS	6	67	5	5	100.68
6.00		SS	7	54	2	6	99.68
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles and boulders 6.70		SS	8	50	3		
End of Borehole							
(GWL @ 2.0m depth based on field observations)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE January 15, 2014

FILE NO. PG3115

HOLE NO. BH18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.53
TOPSOIL 0.20		AU	1				
Compact to loose, brown **SILTY FINE SAND** - grey by 3.0m depth		SS	2	83	11	1	104.53
		SS	3	62	13	2	103.53
		SS	4	67	9		
		SS	5	58	7	3	102.53
		SS	6	58	10	4	101.53
		SS	7	71	7	5	100.53
		SS	8	67	6	6	99.53
6.70		SS	9	50	3		
End of Borehole (GWL @ 1.5m depth based on field observations)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

FILE NO. PG3115

REMARKS

HOLE NO. BH19

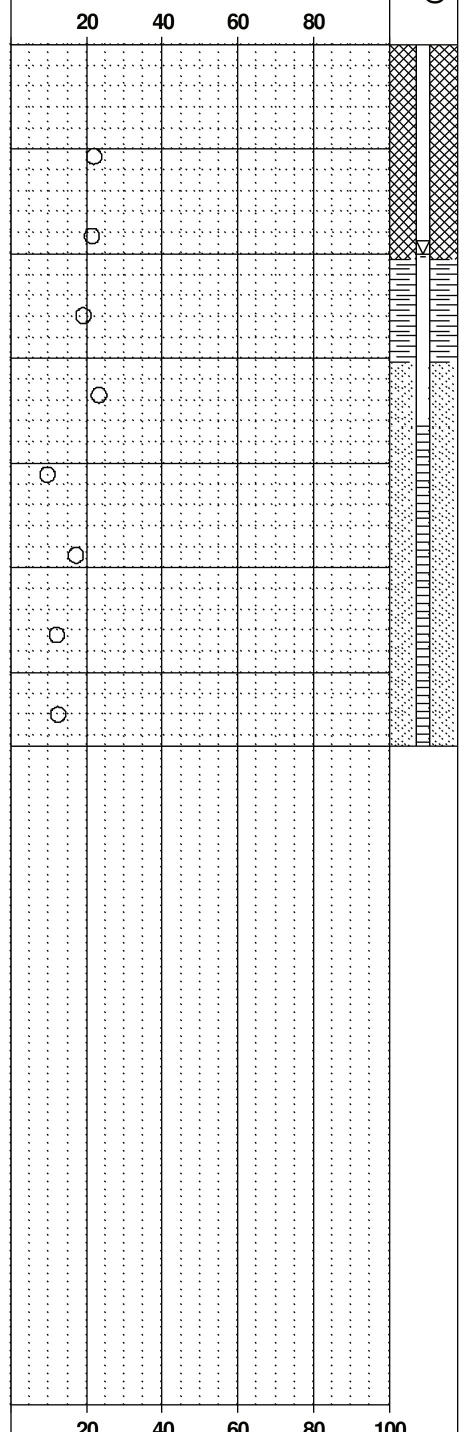
BORINGS BY CME 55 Power Auger **DATE** January 15, 2014

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	105.08
TOPSOIL 0.23						
Compact o very loose, brown **SILTY FINE SAND**	SS	1	62	12	1	104.08
- grey-brown by 2.2m depth	SS	2	71	1	2	103.08
- trace gravel below 3.0m depth	SS	3	62	1	3	102.08
- grey by 3.7m depth	SS	4	67	8	4	101.08
	SS	5	83	1	5	100.08
5.26	SS	6	25	3	6	99.08
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders	SS	7	100	14		
6.70	SS	8	100	32		

End of Borehole

(GWL @ 2.0m depth based on field observations)

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



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the 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 = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = $D60 / D10$

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

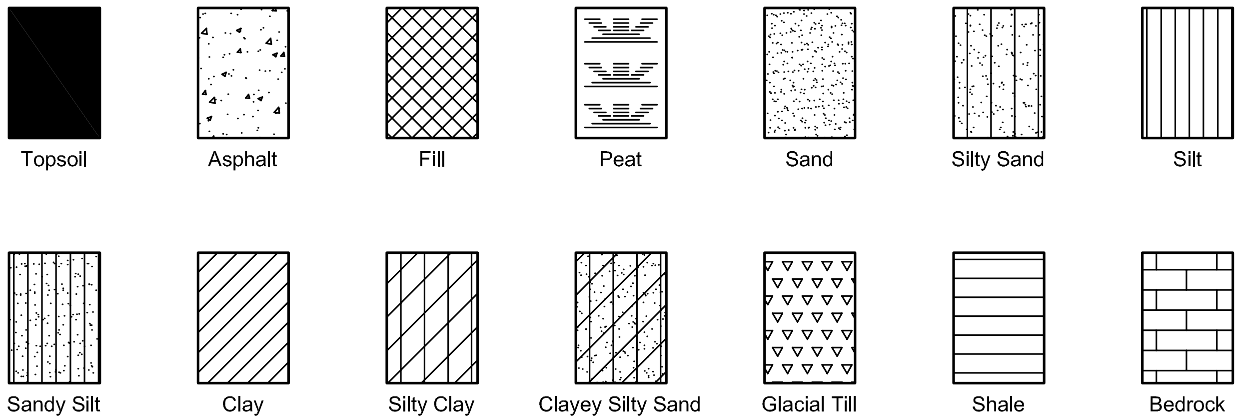
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

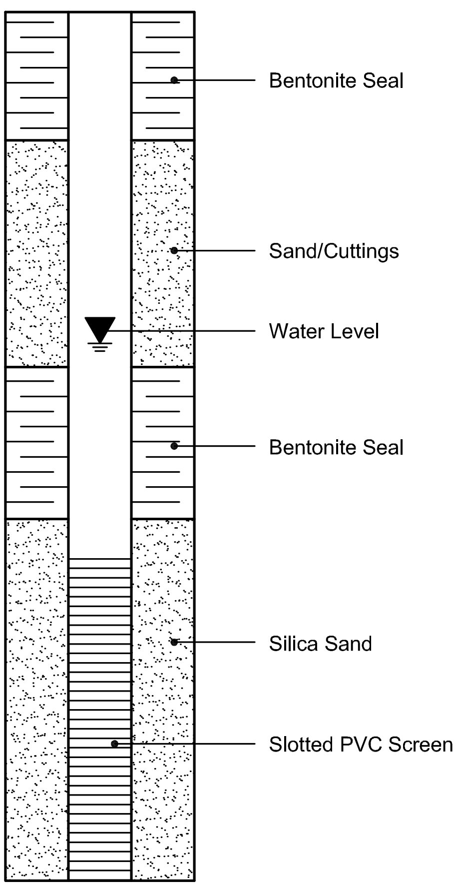
SYMBOLS AND TERMS (continued)

STRATA PLOT

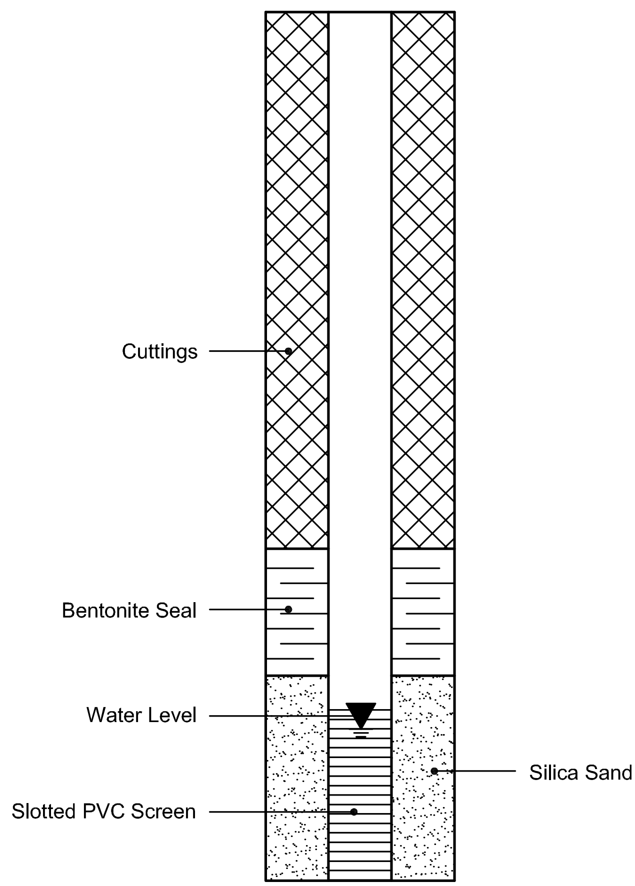


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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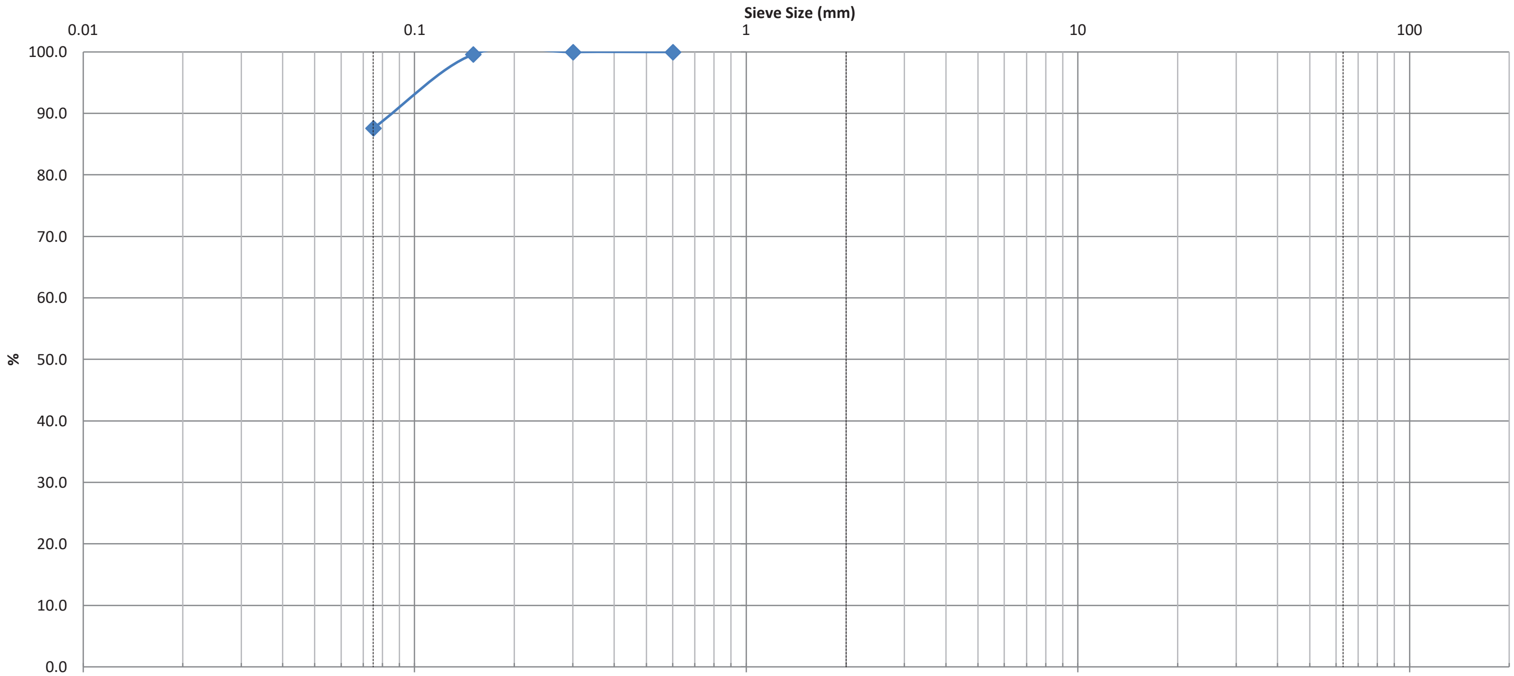
SIEVE ANALYSIS
ASTM C136

CLIENT:	Maritime Ontario c/o Realine	DESCRIPTION:	Soil	FILE NO:	PG5618
CONTRACT NO.:	-	SPECIFICATION:	**Silty Sand**	LAB NO:	23677
PROJECT:	8800 Campeau Drive	INTENDED USE:	-	DATE RECEIVED:	17-Mar-21
		PIT OR QUARRY:	In-Situ	DATE TESTED:	19-Mar-21
DATE SAMPLED:	15-Mar-21	SOURCE LOCATION:	BH1 SS5	DATE REPORTED:	30-Mar-21
SAMPLED BY:	A.C	SAMPLE LOCATION:	3.05 - 3.66 m	TESTED BY:	DK



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									1.13	2.0
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	0.18	0.02	0.015	0.01	0.0	12.4	87.6			

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Fosyth, P. Eng.**

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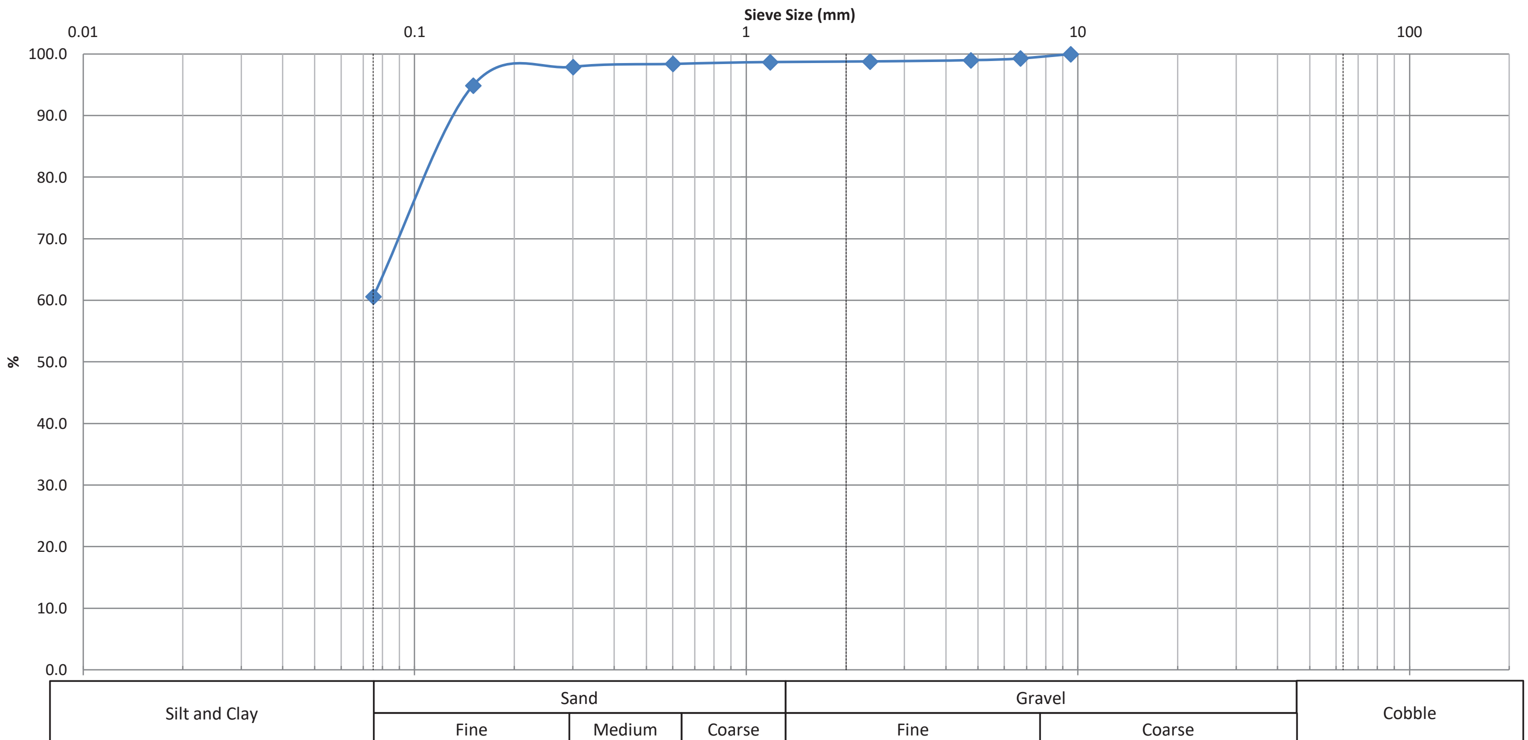
SIEVE ANALYSIS
ASTM C136

CLIENT:	Maritime Ontario c/o Realine	DESCRIPTION:	Soil	FILE NO:	PG5618
CONTRACT NO.:	-	SPECIFICATION:	**Silty Sand**	LAB NO:	23676
PROJECT:	8800 Campeau Drive	INTENDED USE:	-	DATE RECEIVED:	17-Mar-21
		PIT OR QUARRY:	In-Situ	DATE TESTED:	19-Mar-21
DATE SAMPLED:	15-Mar-21	SOURCE LOCATION:	BH3 SS4	DATE REPORTED:	30-Mar-21
SAMPLED BY:	A.C	SAMPLE LOCATION:	2.29 - 2.89 m	TESTED BY:	DK



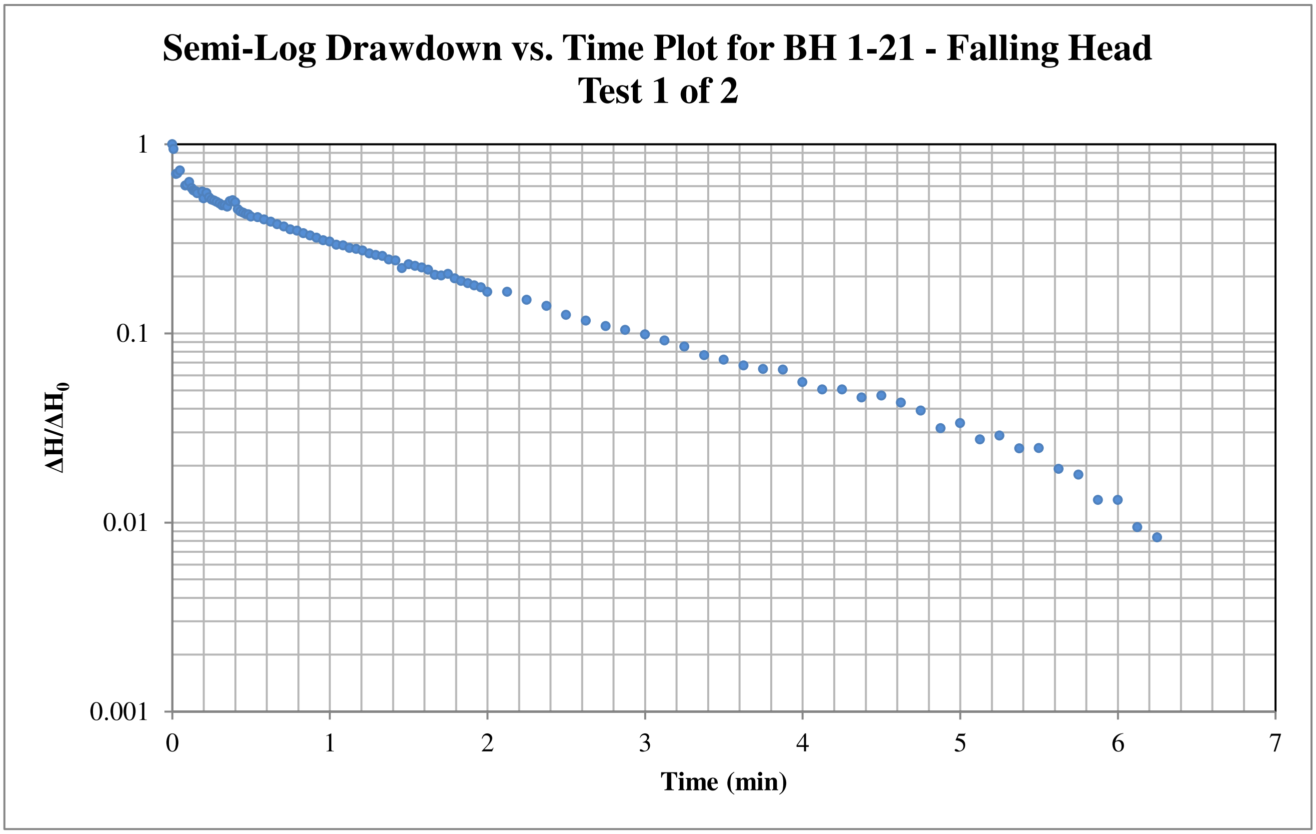
Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									0.65	3.0
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	6.7	0.075	0.035	0.025	1.0	38.4	60.6			

Comments:

REVIEWED BY:	Curtis Beadow	Joe Fosyth, P. Eng.

Hvorslev Hydraulic Conductivity Analysis

Project: Maritime Ontario c/o Realinc-SCS - 8800 Campeau Drive
Test Location: BH 1-21
Test: 1 of 2 Falling Head
Date: May 7, 2021



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: 4.00025

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

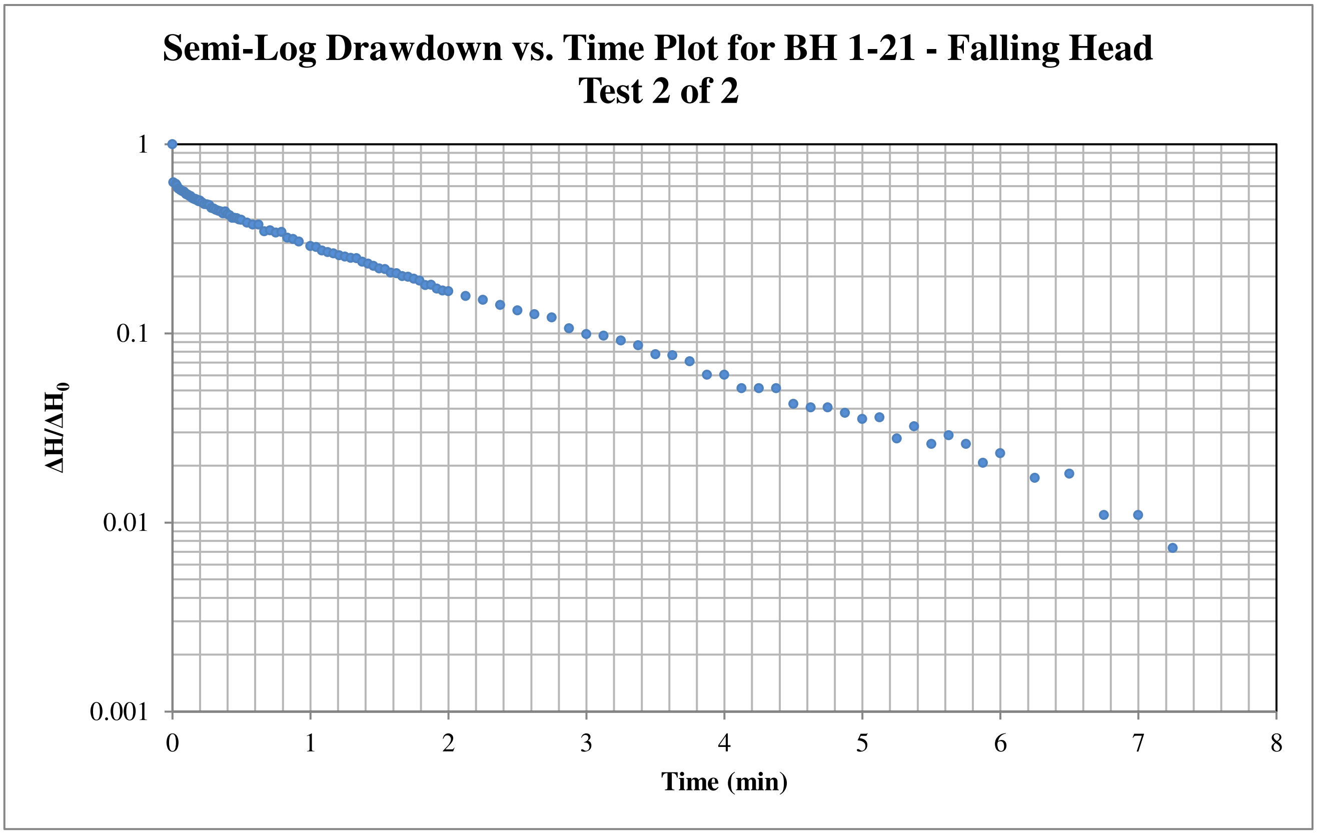
Data Points (from plot):

t^*: 0.679 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.24E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Maritime Ontario c/o Realinc-SCS - 8800 Campeau Drive
Test Location: BH 1-21
Test: 2 of 2 Falling Head
Date: May 7, 2021



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: 4.00025

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

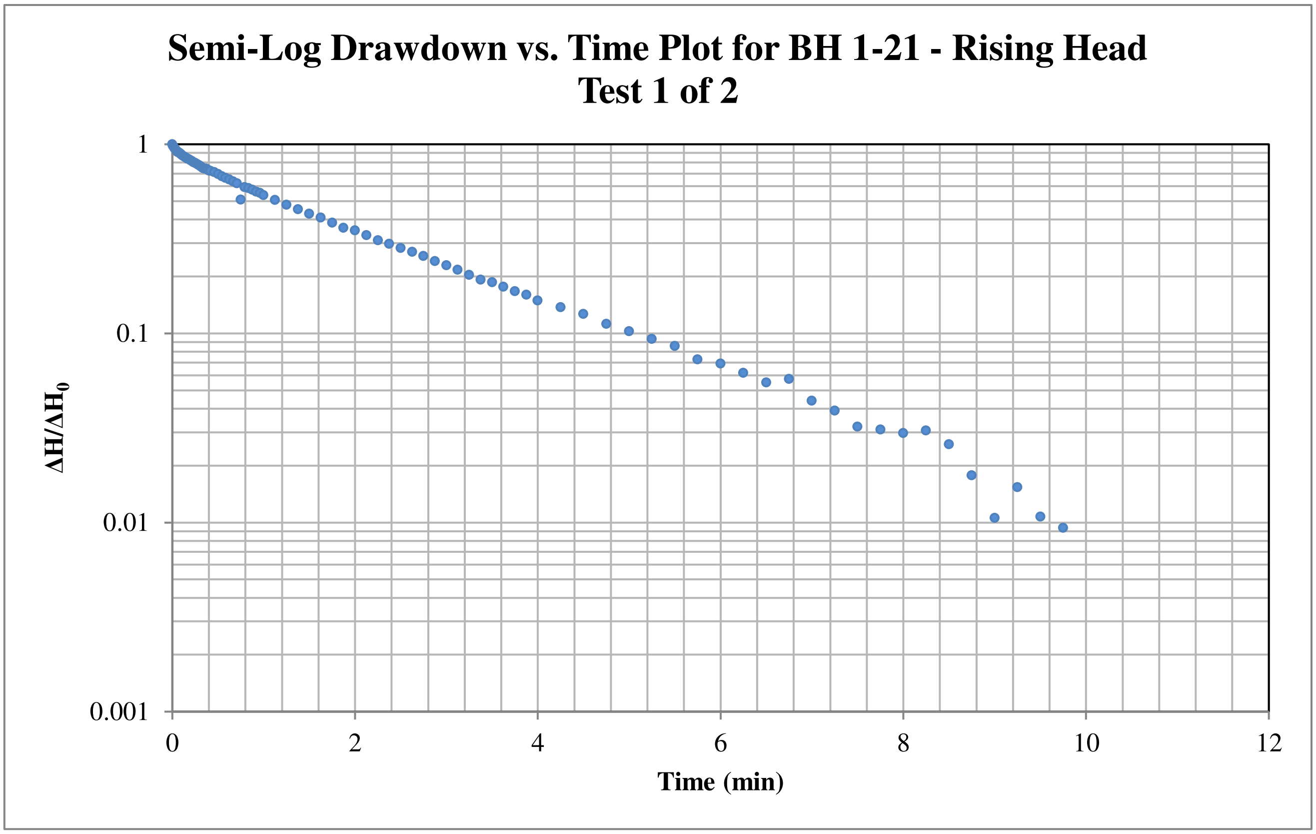
Data Points (from plot):

t^*: 0.628 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.34E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Maritime Ontario c/o Realinc-SCS - 8800 Campeau Drive
Test Location: BH 2-21
Test: 1 of 2 Rising Head
Date: May 7, 2021



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: 4.00025

Well Parameters:

L	3.048	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):

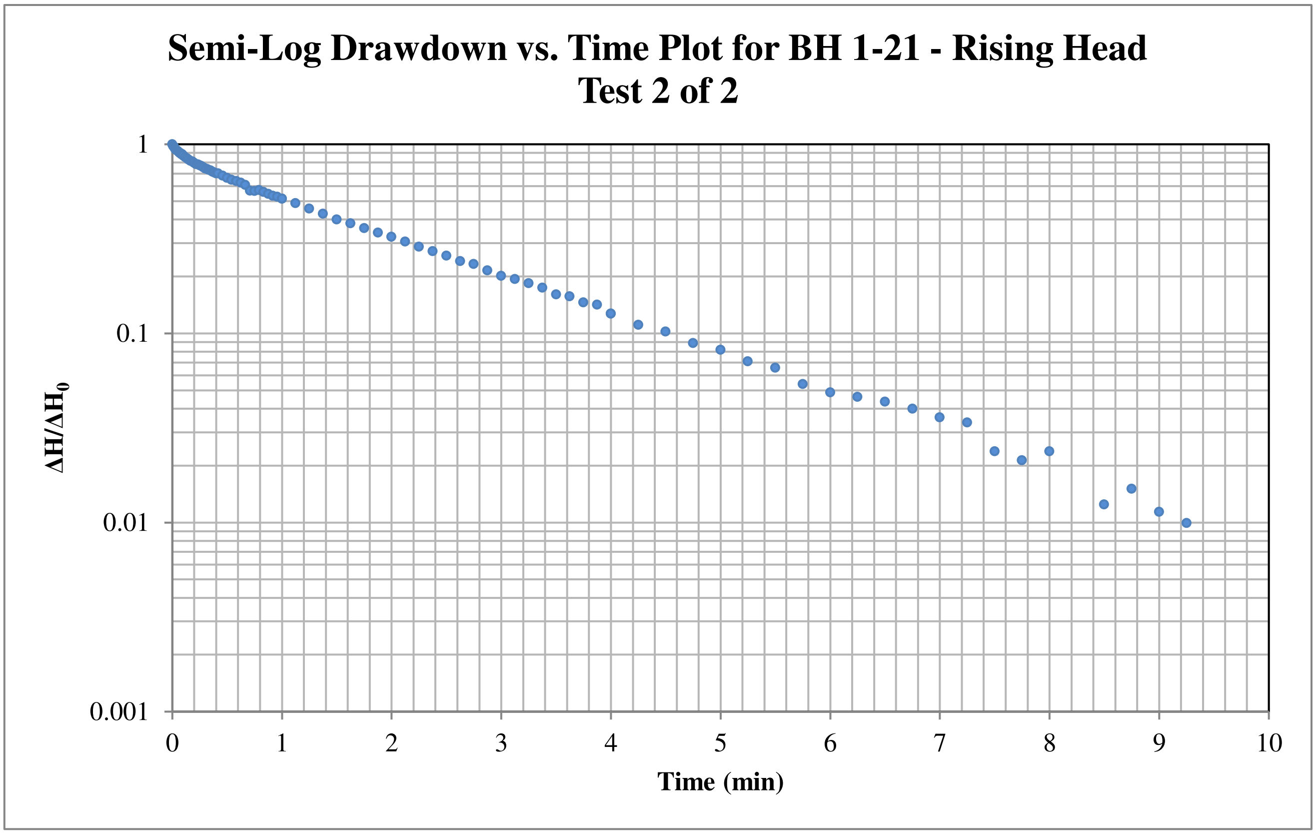
t*: 1.841 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.56E-06 m/sec

Report: PG5673

Hvorslev Hydraulic Conductivity Analysis

Project: Maritime Ontario c/o Realinc-SCS - 8800 Campeau Drive
Test Location: BH 2-21
Test: 2 of 2 Rising Head
Date: May 7, 2021



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: 4.00025

Well Parameters:

L	3.048	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^*: 1.705 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.93E-06 m/sec

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: **32789**

Report Date: 22-Mar-2021

Order Date: 17-Mar-2021

Project Description: PG5618

	Client ID:	BH2-21 SS3	-	-	-
	Sample Date:	15-Mar-21 09:00	-	-	-
	Sample ID:	2112400-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.3	-	-	-
General Inorganics					
pH	0.05 pH Units	7.79	-	-	-
Resistivity	0.10 Ohm.m	38.1	-	-	-
Anions					
Chloride	5 ug/g dry	44	-	-	-
Sulphate	5 ug/g dry	136	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5618-1 - TEST HOLE LOCATION PLAN

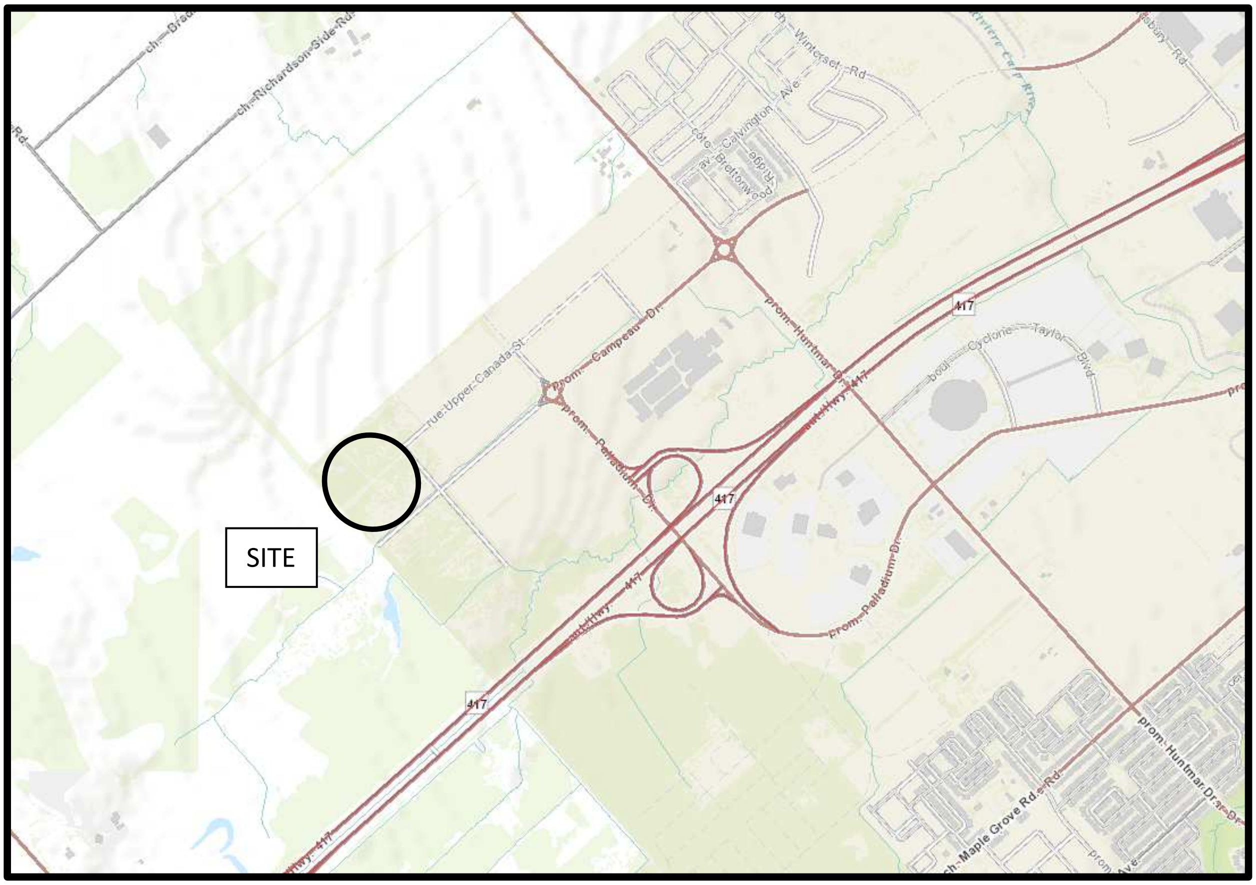


FIGURE 1

KEY PLAN