Geotechnical Engineering

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Materials Testing

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Geotechnical Investigation

Greystone Village Development
Proposed Multi-Storey Building - Tower 1C
360 Deschatelets Avenue
Ottawa, Ontario

Prepared For

eQ Homes

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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Report PG5249-1

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by eQ Homes to conduct a geotechnical investigation for the proposed multi-storey building to be located at 360 Deschatelets Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ☐ 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. 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 available drawings, it is understood that the proposed building will consist of a multi-storey structure with 2.5 to 3 levels of underground parking which will extend to approximate geodetic elevation 56 m. It is also anticipated that the proposed building will be surrounded by asphalt-paved access lanes and parking areas with landscaped margins.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was conducted on February 10 and 11, 2020, and consisted of 2 boreholes advanced to a maximum depth of 12.8 m below the existing ground surface. A previous geotechnical investigation by others also included 2 boreholes (BH 14-210 and BH 15-8) advanced at, or in the vicinity of, the subject site to a maximum depth of 33.6 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG5249-1 - Test Hole Location Plan included in Appendix 2.

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 at the selected locations, and sampling and testing the overburden.

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 presented 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.

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 the current borehole locations. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the 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

Nested groundwater monitoring wells were installed in the current boreholes to permit monitoring of the groundwater levels and to perform hydraulic conductivity testing subsequent to the completion of the sampling program.

Hydraulic Conductivity Testing

Hydraulic conductivity testing was completed at the two nested wells on February 13, 2020. Falling head tests (slug tests) were completed by first measuring the static water level in the well, then inducing a near-instantaneous change of head in the nested wells and monitoring the water level recovery with an electronic water level tape and a Mini Diver water level logger. An aluminum slug, 1 m in length and 0.03 m in diameter, was used to raise the groundwater level within the well. The decrease in water level over time was then recorded until it had stabilized. Results from the testing are discussed further in Section 4.3.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5249-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Two soil samples were submitted for grain size distribution analysis. The results of the grain size distribution analyses are discussed in section Section 4.2 and are provided in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and the results are discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an asphalt-paved turning circle and bus shelter, with the exception of the eastern end of the site which is currently being used as a construction access road. The site is bordered by Hazel Street to the northwest, Deschatelets Avenue to the northeast, an active construction site to the east, and an asphalt-paved parking lot to the south and west. The ground surface across the site is relatively level at approximate geodetic elevation 65 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of an approximate 1.4 to 1.8 m thickness of fill from the existing ground surface or underlying the asphalt surface. The fill was generally observed to consist of a loose to compact, brown silty sand to silty clay with occasional gravel and organics.

A silty clay deposit was generally encountered underlying the fill, and was observed to consist of a very stiff to firm, brown to grey silty clay.

Underlying the silty clay, a sand deposit was encountered at approximate depths of 9.1 to 11.4 m below the existing ground surface. The sand deposit was generally observed to consist of a dense, brown to grey sand with some silt.

Within BH 15-8, by others, glacial till was encountered underlying the sand deposit at an approximate depth of 26.8 m. The glacial till was observed to consist of a silty sand to clayey silt with gravel, cobbles, and boulders.

Bedrock

At BH 15-8, a black to dark grey shale bedrock was cored from an approximate depth of 31.7 to 33.4 m.

Based on available geological mapping, the bedrock at the subject site consists of shale of the Billings formation with a drift thickness of 25 to 50 m.

Laboratory Testing

Grain size distribution analyses were completed on 2 selected soil samples from the sand deposit. The results of the analyses are summarized in Table 1 below and are presented on the Grain Size Distribution Results sheets in Appendix 1.

Table 1 - Summary of Grain Size Distribution Analysis

Test Hole	Sample	Gravel (%)	Sand (%)	Silt & Clay (%)
BH1	SS5	0.0	84.7	15.3
BH2	SS5	0.0	88.9	11.1

From the grain size analyses, the samples collected from boreholes BH1 and BH2 are classified as sand with some silt.

4.3 Groundwater

Groundwater levels were measured on February 13, 2020 in the monitoring wells installed in the completed boreholes. The observed groundwater levels are summarized in Table 2.

Table 2 - Summary of Groundwater Level Readings

Test Hole	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1	65.95	Dry	-	February 13, 2020
BH1A	65.95	9.10	56.85	February 13, 2020
BH 2	65.16	7.35	57.81	February 13, 2020
BH2A	65.16	8.04	57.12	February 13, 2020

Note: - The ground surface elevations are referenced to a geodetic datum.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 7 to 8 m below ground surface. The recorded groundwater levels are provided on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should also be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Hydraulic Conductivity Testing

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 a screen length of 3 m and a diameter of 0.03 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.

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, the line of best fit is considered to pass through the origin.

Based on the above test methods, the monitoring wells screened in the sand displayed hydraulic conductivity values ranging from **8.94×10^{-5} to 6.02×10^{-6} m/sec**. The values measured within the monitoring wells are consistent with similar material Paterson has encountered on other sites and typical published values for sand. These values typically range from 1×10^{-4} to 1×10^{-6} m/sec for sand. The range in hydraulic conductivity values is due to the variability of the sand encountered. The results of the hydraulic conductivity testing are presented in Appendix 1.

It should be noted that testing could not be completed in the monitoring wells screened in the silty clay given the absence of groundwater following the recent construction of the wells and insufficient time to recharge prior to testing. However, based on our experience and available published values, the hydraulic conductivity for the silty clay is considered to be approximately 1×10^{-7} m/s.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed multi-storey building. It is expected that the proposed multi-storey building will be founded on a raft foundation bearing on the undisturbed, stiff silty clay or undisturbed, compact to dense sand.

The lower parking levels should be tanked in order to avoid long term dewatering of adjacent areas. Further, as the excavation is expected to extend below the groundwater level and to the sand deposit, it is recommended that a steel sheet pile cofferdam be advanced to sufficient depth below the bottom of excavation in order to act as temporary shoring and to achieve a groundwater cut-off. This is discussed further in Section 6.0.

Due to the presence of a silty clay layer, the proposed development will be subjected to grade raise restrictions. Our permissible grade raise recommendations are discussed in Subsection 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any building, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

Protection of Subgrade (Raft Foundation)

Since the subgrade material will consist of a silty clay or sand deposit, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed subgrade shortly after the completion of the excavation. The main purpose of the mudslab 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 sand to potential disturbance.

Pressure Relief Chamber

To prevent the long term dewatering of adjacent structures surrounding the site including mature trees, a pressure relief chamber is recommended to be installed along with collection pipes within the silty clay or sand deposit. The collection pipe trenching should extend along the proposed building perimeter and lead to the pressure relief chamber. It is suggested that the pressure relief chamber be incorporated into the lowest section of the lowest level of underground parking. Figure 2 - Pressure Relief Chamber in Appendix 2 provides an example of the required pressure relief chamber. Once the pressure relief chamber and associated piping is installed, the proposed raft slab can be constructed. The purpose of the pressure relief chamber will be as follows:

- ❏ Manage any water infiltration along the founding surface during the excavation program.
- ❏ Manage the water infiltration during the pouring of the raft slab to prevent water flow in the fresh concrete.
- ❏ Manage water infiltration below the raft slab until sufficient load is applied to resist any potential hydrostatic uplift.

- ☐ Regulate the discharge valve to control water infiltration once the raft slab is in place and over the long term to manage the hydrostatic pressure to permit any repairs associated with any water infiltration.
- ☐ Once sufficient load is applied to the raft slab, the pressure relief valve will be fully closed to prevent any further dewatering.

Hydrostatic Pressure

With the fully closed valve within the pressure relief chamber and a perfectly watertight foundation, it is expected that a maximum hydrostatic pressure of **30 kPa** will be developed over the long term and should be incorporated in the design of the raft foundation and the foundation walls. Achieving a fully watertight foundation is not always possible due to minor water infiltration and, therefore, a realistic long term hydrostatic pressure will be closer to 15 to 20 kPa.

5.3 Foundation Design

Raft Foundation

Based on the expected loads from the proposed multi-storey building, a raft foundation is recommended for foundation support of the proposed building. For 2.5 to 3 levels of underground parking, it is expected that the excavation will extend approximately 7 to 10 m below existing ground surface.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **200 kPa** will be considered acceptable. 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. The contact pressure provided considers the stress relief associated with the soil removal required for proposed building. The factored bearing resistance (contact pressure) at ULS can be taken as **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **8 MPa/m** for a contact pressure of **200 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

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

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **1 m** is recommended for grading at the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**. 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 Floor Slab

The lowest level slab should be placed over a granular layer of OPSS Granular A which will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

A sub-slab drainage system is required between the finished floor slab and the underlying raft slab to direct water infiltration to the building sump pit

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed 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.

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 and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 3 and 4.

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 Ramp

Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either 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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

For the proposed underground parking levels, it is understood that the building 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 the shoring system.

Waterproofing of the foundation walls is recommended and the membrane is to be installed from 4 m below finished grade down the foundation walls to the bottom of foundation.

It is also recommended that a composite drainage system, such as Delta Drain 6000 or equivalent, be installed between the waterproofing membrane and the foundation wall, and extend from the exterior finished grade to the founding elevation (underside of raft). 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 raft slab 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 lower basement area.

It is also recommended that a perimeter foundation drainage system be provided at a depth of 2 m for the proposed structure to control any surficial groundwater. The perimeter drainage pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Raft Slab Construction Joints

It is expected that the raft slab will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the raft slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Sub-slab Drainage

Sub-slab drainage will be required to control water which infiltrates through the raft foundation. For design purposes, we recommend that 150 mm diameter perforated pipes be placed along the interior perimeter of the foundation walls and within the building at approximate 6 m spacing. The spacing of the sub-slab drainage system should be confirmed at the time of backfilling the floor completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where sufficient space is available, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. A waterproofing system should be provided for any elevator pits (pit bottom and walls).

Pressure Relief Chamber

The pressure relief chamber will be used to control the groundwater infiltration and hydrostatic pressure created by tanking the lower levels of underground parking. To avoid uplift on the raft foundation slab prior to having sufficient loading to resist uplift, it is recommended that the water infiltration be pumped via the pressure relief chamber during construction.

The valve of the pressure relief chamber can be gradually closed during construction as the loading is applied to resist hydrostatic pressure. Once sufficient load is available to resist the full hydrostatic pressure, the valve of the pressure relief chamber can be adjusted and closed to minimize water infiltration volumes.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

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 structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, unheated structures such as the access ramp may require insulation against the deleterious effect of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the 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. Based on the depth of the proposed structure and the proximity to property lines, it is anticipated that a temporary shoring system will be required to support the excavation.

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 a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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 is anticipated to be required to support the overburden soils during the proposed building excavation. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system is recommended to consist of a steel sheet pile cofferdam advanced to sufficient depth below the bottom of excavation in order to achieve a groundwater cut-off.

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.

Table 5 - Soil Parameters

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	21
Submerged Unit Weight (γ), kN/m^3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

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

Excavation Base Stability

The base of supported excavations can fail by three (3) general modes:

❑ Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,
❑ Piping from water seepage through granular soils, and
❑ Heave of layered soils due to water pressures confined by intervening low permeability soils.

The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems. The factor of safety with respect to basal heave, FS_b, is:

$$FS_b = N_b s_u / \sigma_z$$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.
s_u - undrained shear strength of the soil below the base level
σ_z - total overburden and surcharge pressures at the bottom of the excavation

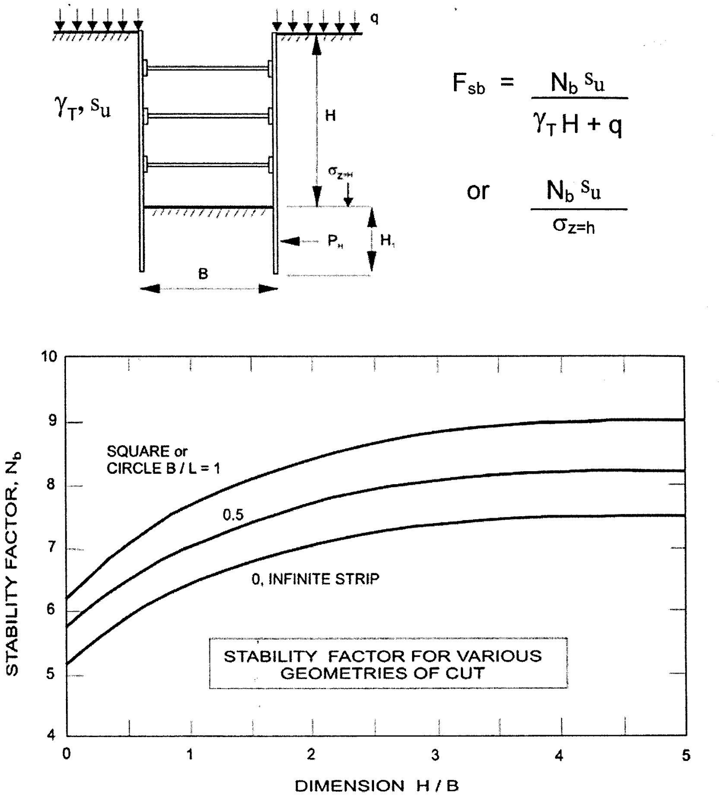


Figure 1 - Stability Factor for Various Geometries of Cut

In the case of stiff clays or compact to dense sands, a factor of safety of 2 is recommended for base stability.

6.4 Pipe Bedding and Backfill

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 98% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.5 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 loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

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, subbedding 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

As noted above, it is recommended that the excavation for the underground parking levels be supported using a steel sheet pile cofferdam to manage the volumes of water to be pumped and to prevent dewatering of adjacent areas.

Further, drilled relief wells should be utilized during the excavation to depressurize the sand layer sufficiently to protect against basal heave. It is recommended that the design of the dewatering system be provided by a specialty dewatering contractor and be submitted to the project design team for review.

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.

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 Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long-term Groundwater Control

Our recommendations for the proposed building’s long-term groundwater control are presented in Subsection 6.1. Any groundwater which breaches the building’s perimeter groundwater infiltration control system will be directed to the proposed building’s sump pit. Provided the proposed groundwater infiltration control system and the tanked system are properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be very low to negligible (less than 2,000 L/day). A more accurate estimate can be provided at the time of construction, once the pressure relief chamber valve is closed and full hydrostatic pressure is applied to the structure.

Impacts on Neighbouring Properties

As the proposed multi-storey building will be founded below the long term groundwater level, a groundwater infiltration control system has been recommended to mitigate the effects of groundwater infiltration. Any long term dewatering of the site will be minimal and should have no adverse effects to the surrounding buildings or structures. The short term dewatering during the excavation program will be managed by the excavation contractor, as discussed above.

6.6 Winter Construction

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

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters, tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- ❑ Review of the grading plan from a geotechnical perspective.
- ❑ Review of the Contractor’s design of the temporary shoring system
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Inspection and approval of the installation of the pressure relief chamber.
- ❑ Inspection of the foundation waterproofing and all foundation drainage systems.
- ❑ 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 backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request 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 its 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 eQ Homes or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Scott Dennis

Scott S. Dennis, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Feb. 14, 2020
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Report Distribution

☐ eQ Homes. (e-mail copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION ANALYSIS RESULTS

HYDRAULIC CONDUCTIVITY TESTING RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
360 Deschatelets Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2020 February 10

FILE NO. PG5249

HOLE NO. BH 1

SOIL DESCRIPTION	
GROUND SURFACE	
ASPHALTIC CONCRETE	0.08
FILL: Brown silty sand with gravel	0.36
FILL: Loose Brown silty sand	1.37
Compact, brown **SAND**, trace silt	2.29
Stiff to firm, grey **SILTY CLAY**	11.43
Dense, brown to grey **SAND**, some silt	12.80
Dynamic Cone Penetration Test commenced at 12.8m depth.	18.52

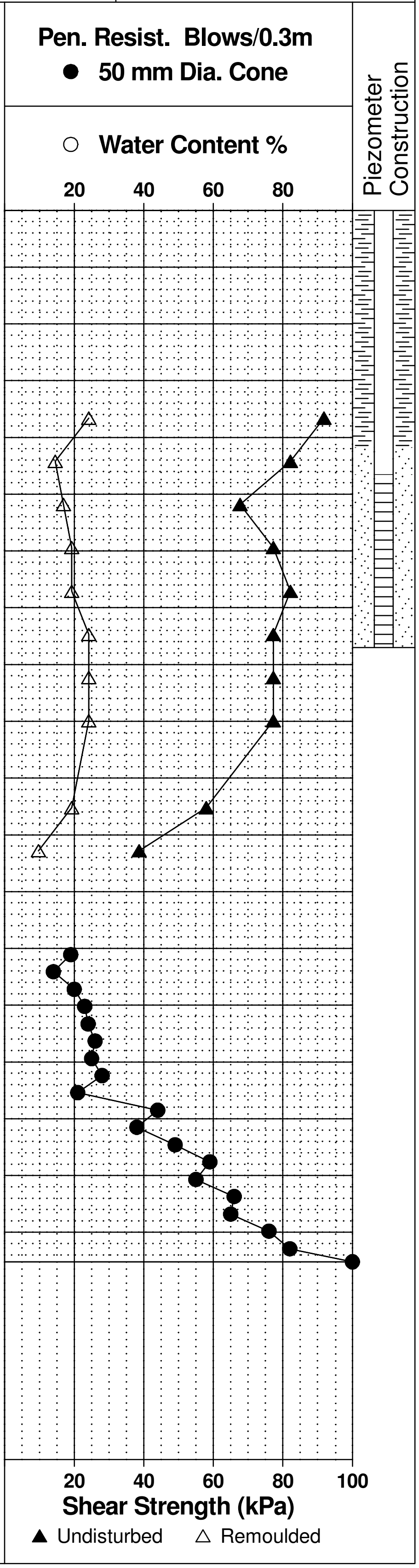
End of Borehole

Practical refusal to DCPT @ 18.52m depth

(BH dry - Feb. 3/2020)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	46	7
SS	3	50	10
SS	4	83	2
SS	5	100	4
SS	6	38	33

DEPTH (m)	ELEV. (m)
0	65.95
1	64.95
2	63.95
3	62.95
4	61.95
5	60.95
6	59.95
7	58.95
8	57.95
9	56.95
10	55.95
11	54.95
12	53.95
13	52.95
14	51.95
15	50.95
16	49.95
17	48.95
18	47.95



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
360 Deschatelets Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2020 February 10

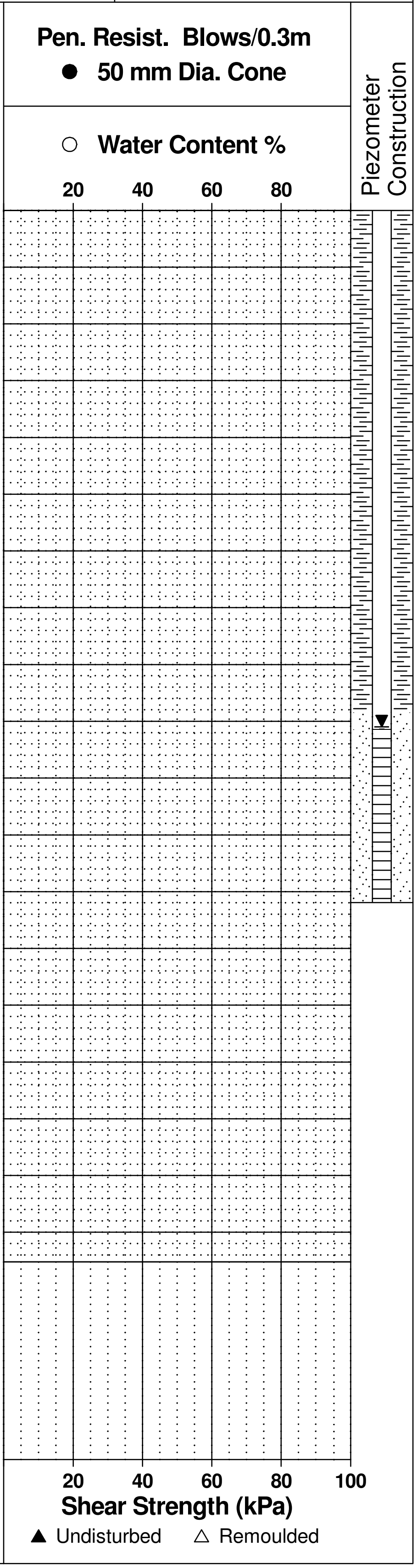
FILE NO. PG5249

HOLE NO. BH 1A

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
ASPHALTIC CONCRETE	0.08
FILL: Brown silty sand with gravel	0.36
FILL: Loose Brown silty sand	1.37
Compact, brown **SAND**, trace silt	2.29
Stiff to firm, grey **SILTY CLAY**	11.43
Dense, brown to grey **SAND**, some silt	12.80
Dynamic Cone Penetration Test commenced at 12.8m depth.	18.52
End of Borehole Practical refusal to DCPT @ 18.52m depth (GWL @ 9.10m - Feb. 13/2020)	

DEPTH (m)	ELEV. (m)
0	65.95
1	64.95
2	63.95
3	62.95
4	61.95
5	60.95
6	59.95
7	58.95
8	57.95
9	56.95
10	55.95
11	54.95
12	53.95
13	52.95
14	51.95
15	50.95
16	49.95
17	48.95
18	47.95

STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
360 Deschatelets Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

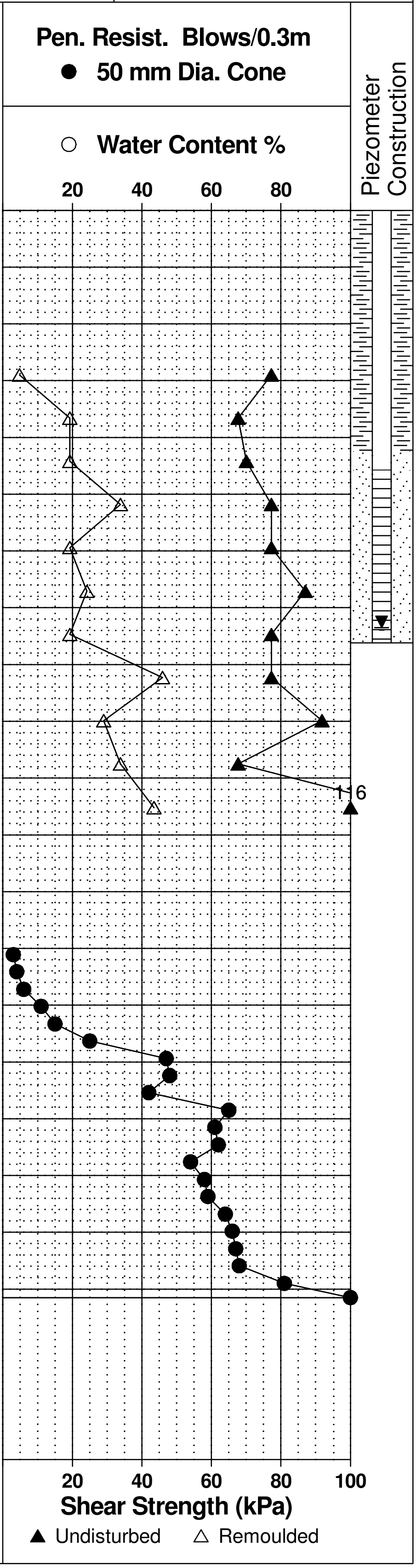
DATE 2020 February 10

FILE NO. PG5249

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with gravel	0.53	AU	1		
FILL: Brown/grey silty clay, some sand, trace organics	1.52	SS	2	25	8
Stiff, brown **SILTY CLAY** - stiff to very stiff and grey by 3 m depth	10.67	SS	3	83	3
Dense grey **SAND**, some silt	12.80	SS SS	4 5	17 50	31 35
Dynamic Cone Penetration Test commenced at 12.8m depth.	19.15				
End of Borehole (GWL @ 7.35m - Feb. 13/2020)					

DEPTH (m)	ELEV. (m)
0	65.16
1	64.16
2	63.16
3	62.16
4	61.16
5	60.16
6	59.16
7	58.16
8	57.16
9	56.16
10	55.16
11	54.16
12	53.16
13	52.16
14	51.16
15	50.16
16	49.16
17	48.16
18	47.16
19	46.16



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
360 Deschatelets Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2020 February 10

FILE NO. PG5249

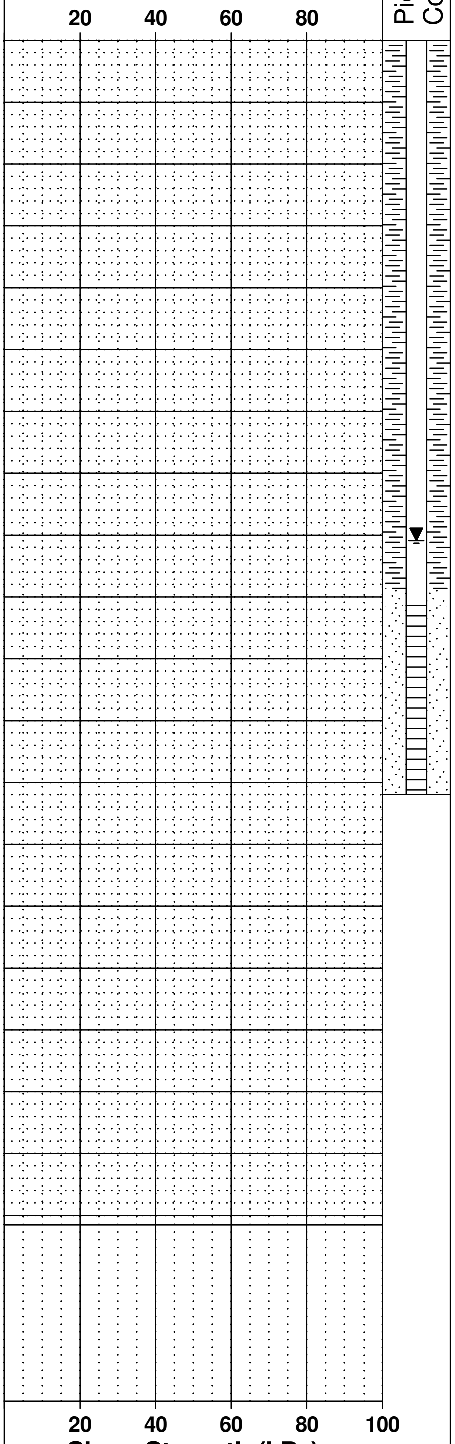
HOLE NO. BH 2A

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE						
FILL: Brown silty sand with gravel	0.53					
FILL: Brown/grey silty clay, some sand, trace organics	1.52					
Stiff, brown **SILTY CLAY** - stiff to very stiff and grey by 3 m depth	10.67					
Dense grey **SAND**, some silt	12.80					
Dynamic Cone Penetration Test commenced at 12.8m depth.	19.15					
End of Borehole (GWL @ 8.09m - Feb. 13/2020)						

DEPTH (m)	ELEV. (m)
0	65.16
1	64.16
2	63.16
3	62.16
4	61.16
5	60.16
6	59.16
7	58.16
8	57.16
9	56.16
10	55.16
11	54.16
12	53.16
13	52.16
14	51.16
15	50.16
16	49.16
17	48.16
18	47.16
19	46.16

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 strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 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)

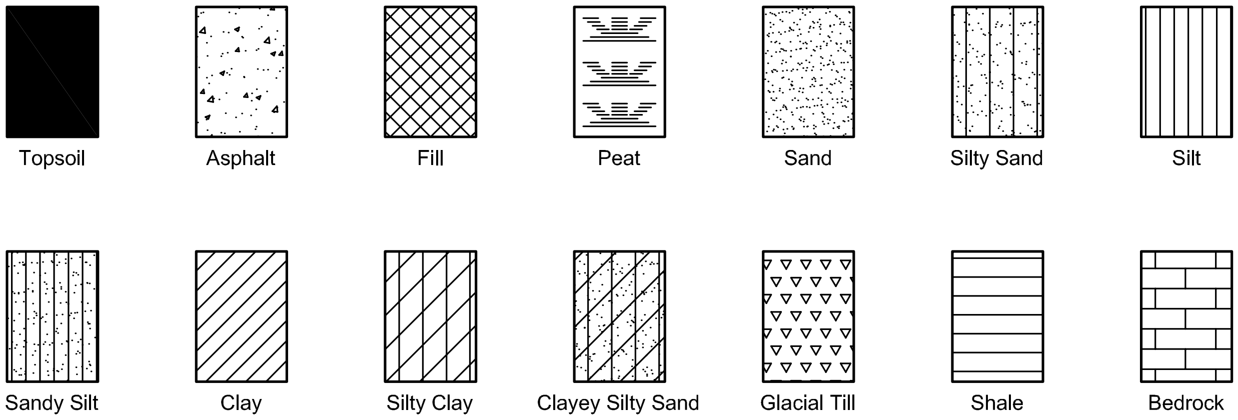
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

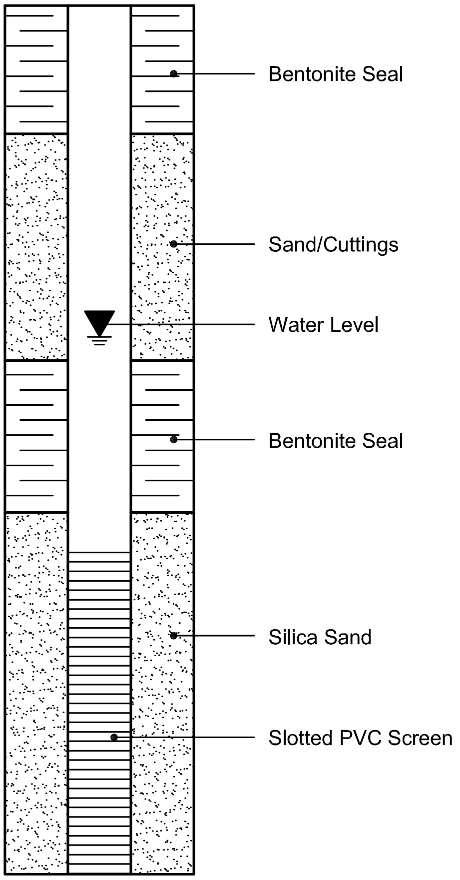
SYMBOLS AND TERMS (continued)

STRATA PLOT

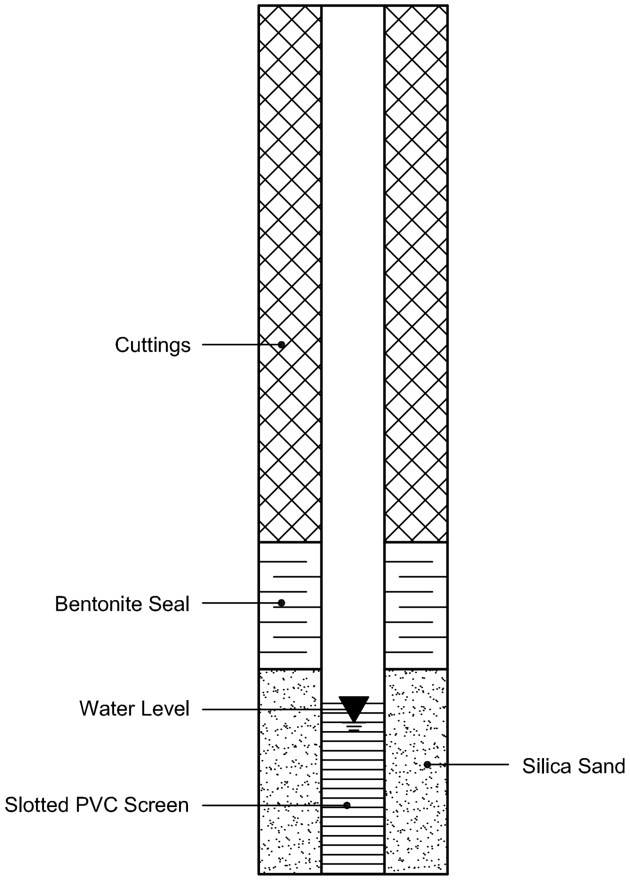


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PROJECT: 14-1122-0005-5100

RECORD OF BOREHOLE: 14-210

SHEET 1 OF 2

LOCATION: See Site Plan

BORING DATE: August 8-11, 2014

DATUM: Geodetic

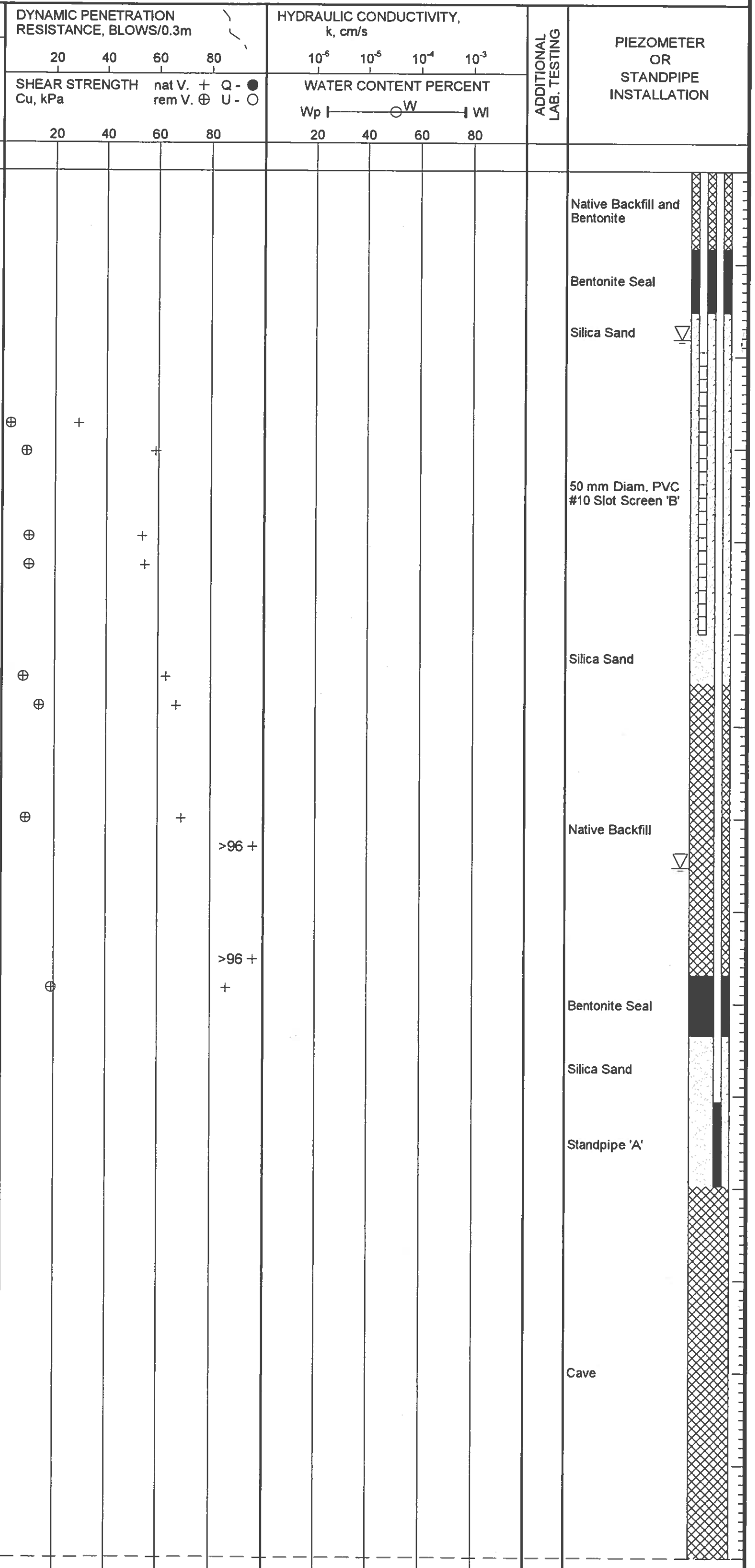
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	65.76
FILL/TOPSOIL - (ML) sandy SILT; brown; moist	0.00
FILL - (SM) SILTY SAND; light brown; non-cohesive, moist, loose	65.61 0.15
FILL - (SM-SP) SILTY SAND to SAND, fine to medium, some silt; light brown to brown; non-cohesive, moist to wet, compact	65.15 0.61
(ML) CLAYEY SILT, trace sand; grey; cohesive, w>PL, firm to stiff	63.93 1.83
(CI/CH) SILTY CLAY to CLAY, trace sand; grey, with black streaks; cohesive, w>PL, stiff	62.71 3.05
(CI/CH) SILTY CLAY to CLAY; grey; cohesive, w>PL, stiff to very stiff	59.66 6.10
(SM) SILTY SAND, fine; grey; non-cohesive, wet, compact	56.62 9.14
(SM) SILTY SAND, fine; grey, with silt layers; non-cohesive, wet, compact	55.10 10.66
(SM) SILTY SAND, fine; grey; non-cohesive, wet, compact	52.05 13.71

SAMPLES NUMBER	TYPE	BLOWS/0.30m
1	SS	9
2	SS	15
3	SS	11
4	SS	2
5	SS	1
6	TP	PH
7	SS	PH
8	SS	PH
9	TP	PH
10	SS	25
11	SS	27
12	SS	23
13	SS	11



CONTINUED NEXT PAGE

DEPTH SCALE
: 75

LOGGED: DWM
CHECKED: CK

PROJECT: 14-1122-0005-5100

RECORD OF BOREHOLE: 14-210

SHEET 2 OF 2

LOCATION: See Site Plan

BORING DATE: August 8-11, 2014

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80

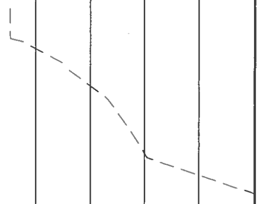
SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80

ADDITIONAL LAB. TESTING

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---						
15 16	Power Auger 200 mm Diam. (Hollow Stem)	(SM) SILTY SAND, fine; grey; non-cohesive, wet, compact		14	SS	13		
17 18		(SM) SILTY SAND, fine; grey, with silt seams; non-cohesive, wet, loose	49.00 16.76	15	SS	6		Cave
		(ML) sandy SILT; grey; non-cohesive, wet, compact	47.32 18.44	16	SS	17		
19	DCPT	Possible sandy Silt	46.86 18.90					WL in Standpipe 'A' at Elev. 58.23 m on Sept. 9, 2014 WL in Screen 'B' at Elev. 63.94 m on Sept. 9, 2014
20 21 22 23 24 25		Possible Glacial Till	46.26 19.50				128 147 141 136 125 135 130 130 128 136 185 171 130 170 206	
26 27 28 29 30		End of Borehole Dynamic Cone Penetration Test Refusal	40.46 25.30					



DEPTH SCALE

1 : 75

Golder Associates

LOGGED: DWM

CHECKED: CK

PROJECT: 15243371524337-2000

RECORD OF BOREHOLE: 15-8

SHEET 1 OF 4

LOCATION: See Site Plan

BORING DATE: September 8-9, 2015

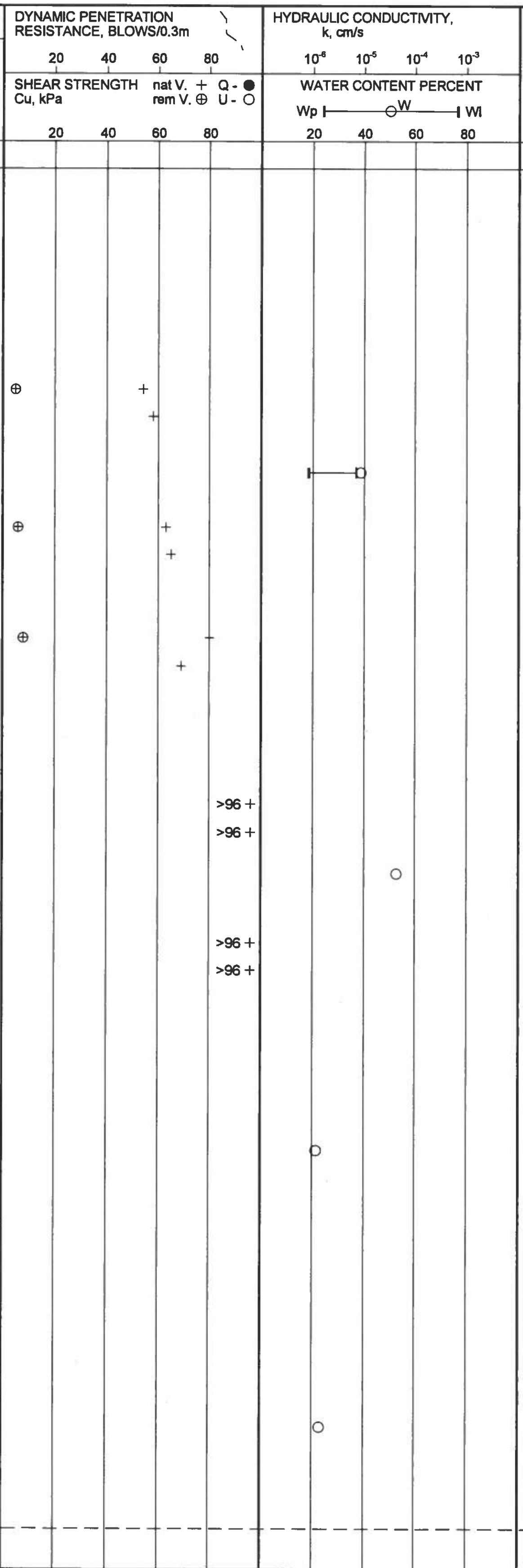
DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)
		GROUND SURFACE		65.14
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL - (SM) SILTY SAND; dark brown, contains brick, organic matter and rootlets; non-cohesive, moist	▨	0.00
		FILL - (SM) SILTY SAND; brown; non-cohesive, moist	▨	64.55 0.59
1		FILL - (SM) SILTY SAND; brown to grey brown; non-cohesive, moist to wet, loose	▨	64.25 0.89
		(CI) SILTY CLAY, trace to some sand; grey, with black mottling; cohesive, w>PL, stiff to very stiff	▨	63.54 1.60
3	Wash Boring HW Casing			
9		(CL/ML) SILTY CLAY to CLAYEY SILT; grey; cohesive, w>PL	▨	56.15 8.99
		(SM/ML) SILTY SAND to sandy SILT; grey; non-cohesive, wet, compact to dense	░	9.15
15		*CONTINUED NEXT PAGE*		

SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	SS	8	
2	SS	WH	
3	SS	WH	
4	SS	WH	
5	TP	PH	
6	SS	2	
7	SS	26	
8	SS	11	MH
9	SS	36	
10	SS	15	M



PIEZOMETER OR STANDPIPE INSTALLATION

Cement Seal

64 mm Diam. PVC Casing

Bentonite-Cement Grout

∇

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: RI

CHECKED: SAT

PROJECT: 15243371524337-2000

RECORD OF BOREHOLE: 15-8

SHEET 2 OF 4

LOCATION: See Site Plan

BORING DATE: September 8-9, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 20 40 60 80

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
15		— *CONTINUED FROM PREVIOUS PAGE* —							
	Wash Boring HW Casing	(SM/ML) SILTY SAND to sandy SILT; grey; non-cohesive, wet, compact to dense		11	SS	23			
16									
17				12	SS	27			
18				13	SS	30	○	M	
19									
20				14	SS	35			
21				15	SS	45			
22									64 mm Diam. PVC Casing Bentonite-Cement Grout
23				16	SS	39			
24				17	SS	28			
25									
26				18	SS	12			
27		(SM/ML) gravelly SILTY SAND to gravelly sandy SILT; brown (GLACIAL TILL); non-cohesive, wet, compact	38.31 26.83	19	SS	24			
28									
29				20	SS	17			
30		*CONTINUED NEXT PAGE*							

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: RI
CHECKED: SAT

PROJECT: 15243371524337-2000

RECORD OF BOREHOLE: 15-8

SHEET 3 OF 4

LOCATION: See Site Plan

BORING DATE: September 8-9, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		— CONTINUED FROM PREVIOUS PAGE —									
30	Wash Boring HW Casing										
		(ML) gravelly sandy CLAYEY SILT; grey (GLACIAL TILL); cohesive, wet, dense		34.81 30.33	21	SS	34				
31											
		Borehole continued on RECORD OF DRILLHOLE 15-8		33.61 31.53							
32											
33											
34											
35											
36											
37											
38											
39											
40											
41											
42											
43											
44											
45											

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: RI

CHECKED: SAT

PROJECT: 15243371524337-2000

RECORD OF DRILLHOLE: 15-8

SHEET 4 OF 4

LOCATION: See Site Plan

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: September 8-9, 2015
DRILL RIG: CME 850
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

JN - Joint	BD- Bedding	PL - Planar	PO- Polished	BR - Broken Rock
FLT - Fault	FO- Foliation	CU- Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR- Shear	CO- Contact	UN- Undulating	SM- Smooth	
VN - Vein	OR- Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB- Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		BEDROCK SURFACE		33.61																
	Rotary Drill HQ Core	GLACIAL TILL		31.53	1	100							BD,PL,SM							
32 33		Fresh, thinly laminated, black to dark grey, fine grained, porous SHALE		31.76 31.73	2	100														64 mm Diam. PVC Casing Bentonite-Cement Grout
34 35 36 37 38 39 40 41 42 43 44 45 46		End of Drillhole		33.41																WL in open borehole at 9.73 m depth below ground surface upon completion of drilling

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: RI
CHECKED: SAT

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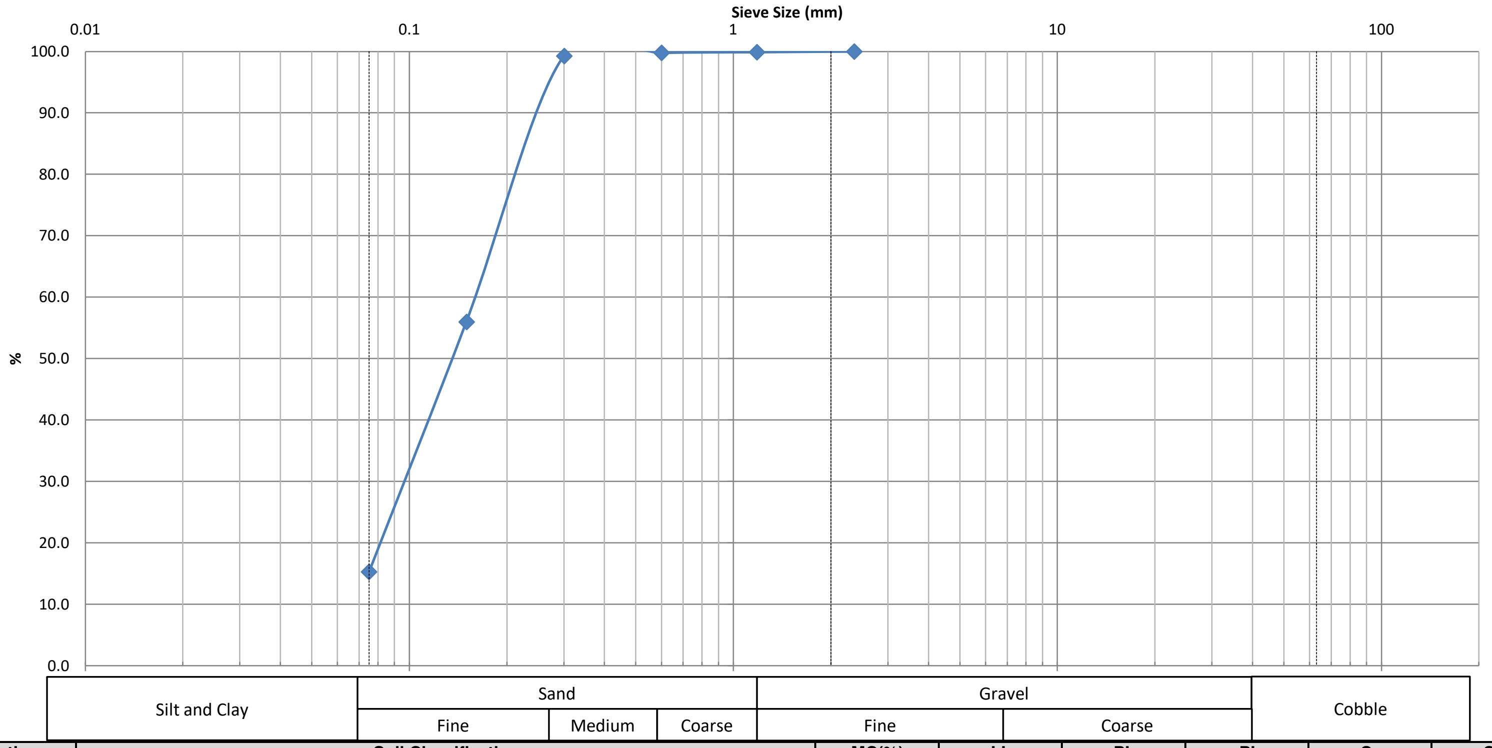
SIEVE ANALYSIS
ASTM C136

CLIENT:	EQ Homes	DESCRIPTION:	Silty Sand	FILE NO:	PG5249
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO:	14886
PROJECT:	Deschatelets Ave	INTENDED USE:	-	DATE RECEIVED:	10-Feb-20
		BOREHOLE:	BH1	DATE TESTED:	11-Feb-20
DATE SAMPLED:	10-Feb-20	SOURCE LOCATION:	SS6	DATE REPORTED:	12-Feb-20
SAMPLED BY:	A. Cooney	SAMPLE LOCATION:	40 - 42'	TESTED BY:	DB



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
						0.83	2.5

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
2.36	0.17	0.098	0.068	0.0	84.7	15.3	

Comments:

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

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SIEVE ANALYSIS
ASTM C136

CLIENT:	EQ Homes	DESCRIPTION:	Silty Sand	FILE NO.:	PG5249
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO.:	14886
PROJECT:	Deschatelets Ave	INTENDED USE:	-	DATE REC'D:	10-Feb-20
		BOREHOLE:	BH1	DATE TESTED:	11-Feb-20
DATE SAMPLED:	10-Feb-20	SOURCE LOCATION:	SS6	DATE REP'D:	12-Feb-20
SAMPLED BY:	A. Cooney	SAMPLE LOCATION:	40 - 42'	TESTED BY:	DB

WEIGHT BEFORE WASH	322.7
WEIGHT AFTER WASH	295.4

SIEVE SIZE (mm)	WEIGHT RETAINED	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
150						
106						
75						
63						
53						
37.5						
26.5						
19						
16						
13.2						
9.5						
6.7						
4.75						
2.36	**0.0**	0.0	**100.0**			
1.18	**0.4**	0.1	**99.9**			
0.6	**0.6**	0.2	**99.8**			
0.3	**2.1**	0.7	**99.3**			
0.15	**142.0**	44.0	**56.0**			
0.075	**273.2**	84.7	**15.3**			
PAN	**295.4**					

SIEVE CHECK FINE	0.00	0.3% max.	**REFERENCE MATERIAL**

OTHER TESTS	RESULT	LAB NO.	RESULT

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

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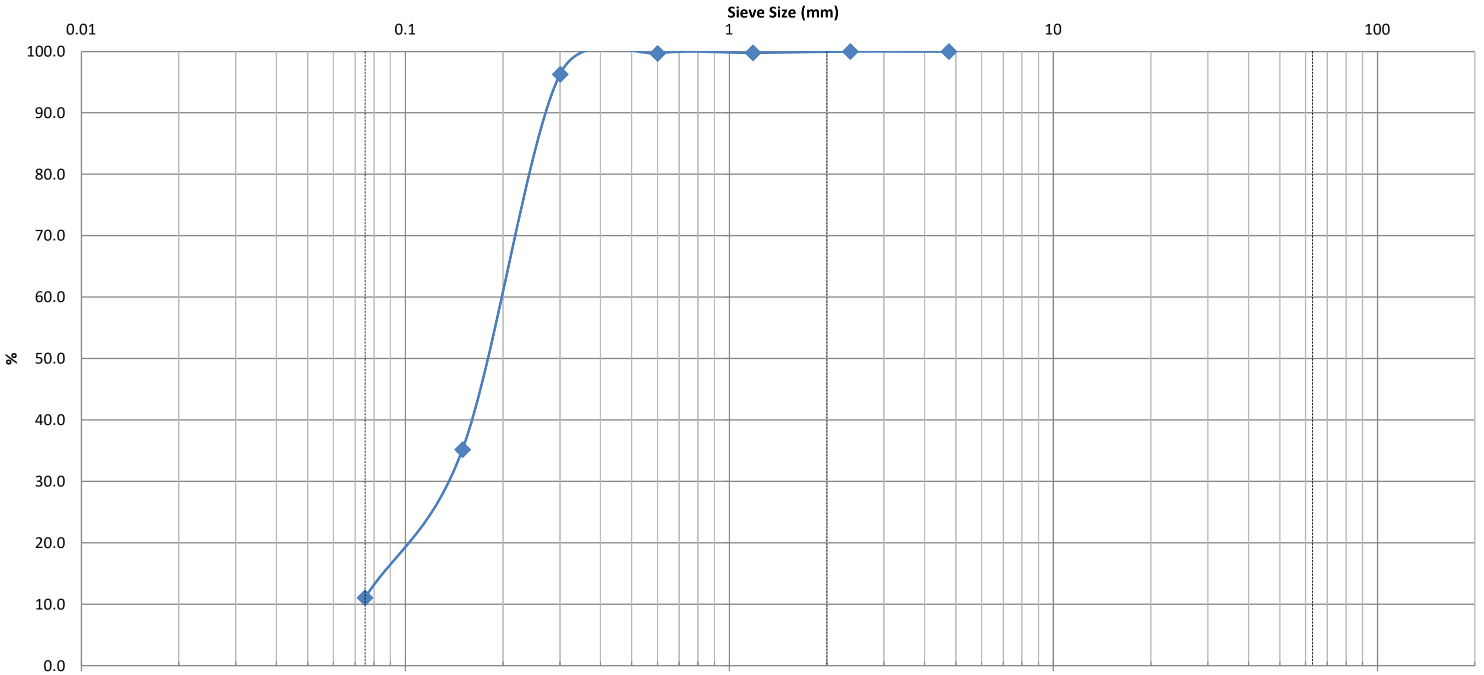
SIEVE ANALYSIS
ASTM C136

CLIENT:	EQ Homes	DESCRIPTION:	Silty Sand	FILE NO:	PG5249
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO:	14896
PROJECT:	Deschatelets Ave	INTENDED USE:	-	DATE RECEIVED:	11-Feb-20
		BOREHOLE:	BH2	DATE TESTED:	12-Feb-20
DATE SAMPLED:	11-Feb-20	SOURCE LOCATION:	SS5	DATE REPORTED:	12-Feb-20
SAMPLED BY:	A. Cooney	SAMPLE LOCATION:	40 - 42'	TESTED BY:	DB



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
						1.48	2.9

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
4.75	0.195	0.14	0.068	0.0	88.9	11.1	

Comments:

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

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SIEVE ANALYSIS
ASTM C136

CLIENT:	EQ Homes	DESCRIPTION:	Silty Sand	FILE NO.:	PG5249
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO.:	14896
PROJECT:	Deschatelets Ave	INTENDED USE:	-	DATE REC'D:	11-Feb-20
		BOREHOLE:	BH2	DATE TESTED:	12-Feb-20
DATE SAMPLED:	11-Feb-20	SOURCE LOCATION:	SS5	DATE REP'D:	12-Feb-20
SAMPLED BY:	A. Cooney	SAMPLE LOCATION:	40 - 42'	TESTED BY:	DB

WEIGHT BEFORE WASH	601.8
WEIGHT AFTER WASH	545.9

SIEVE SIZE (mm)	WEIGHT RETAINED	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
150						
106						
75						
63						
53						
37.5						
26.5						
19						
16						
13.2						
9.5						
6.7						
4.75	**0.0**	0.0	**100.0**			
2.36	**0.2**	0.0	**100.0**			
1.18	**1.2**	0.2	**99.8**			
0.6	**2.1**	0.3	**99.7**			
0.3	**22.4**	3.7	**96.3**			
0.15	**390.1**	64.8	**35.2**			
0.075	**534.9**	88.9	**11.1**			
PAN	**545.7**					

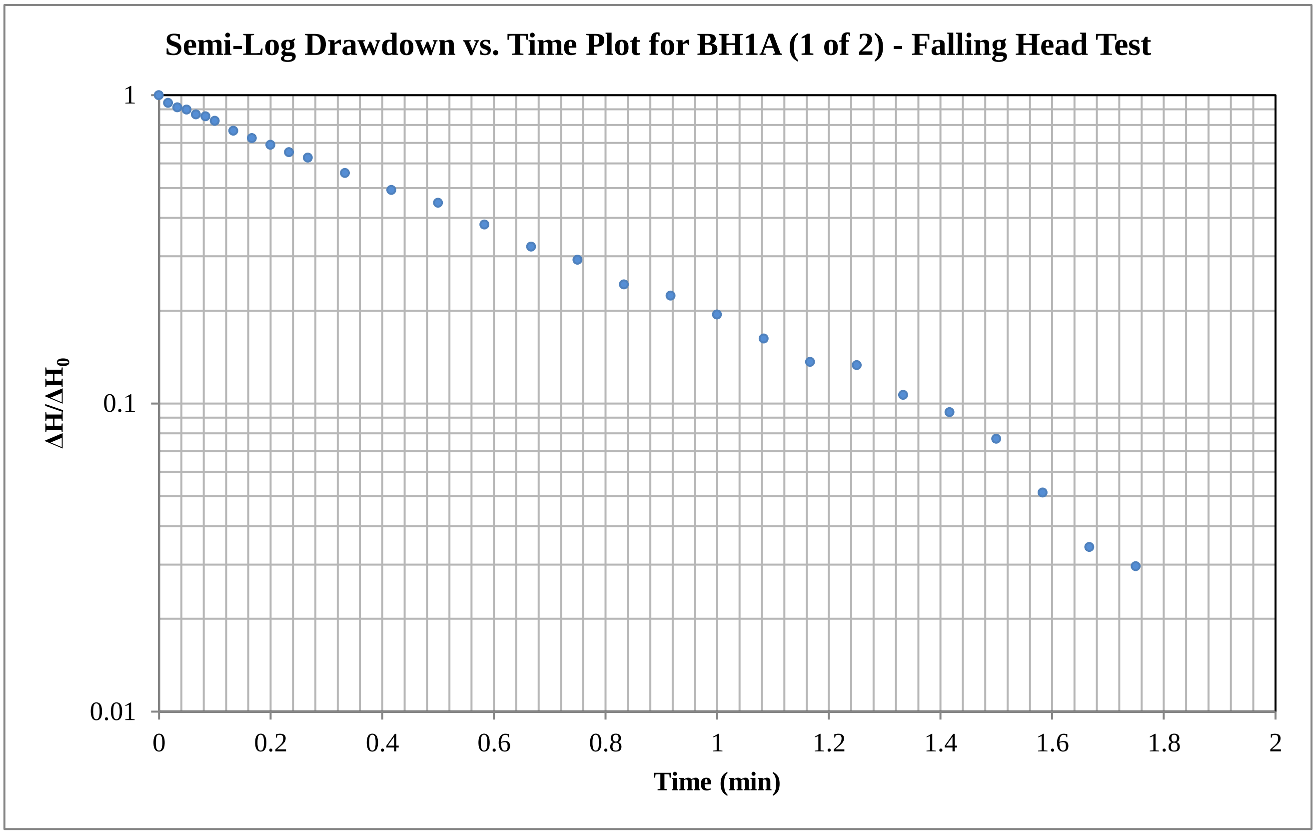
SIEVE CHECK FINE	0.04	0.3% max.	**REFERENCE MATERIAL**

OTHER TESTS	**RESULT**	**LAB NO.**	**RESULT**

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

Hvorslev Hydraulic Conductivity Analysis

Project: PG5249 - EQ Homes
Test Location: BH1A
Test: Falling Head
Date: February 13, 2020



Hvorslev Horizontal Hydraulic Conductivity

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

Hvorslev Shape Factor

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

Valid for L>>D

Hvorslev Shape Factor F: 3.64373

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.0318	m	Diameter of well
r_c	0.0159	m	Radius of well

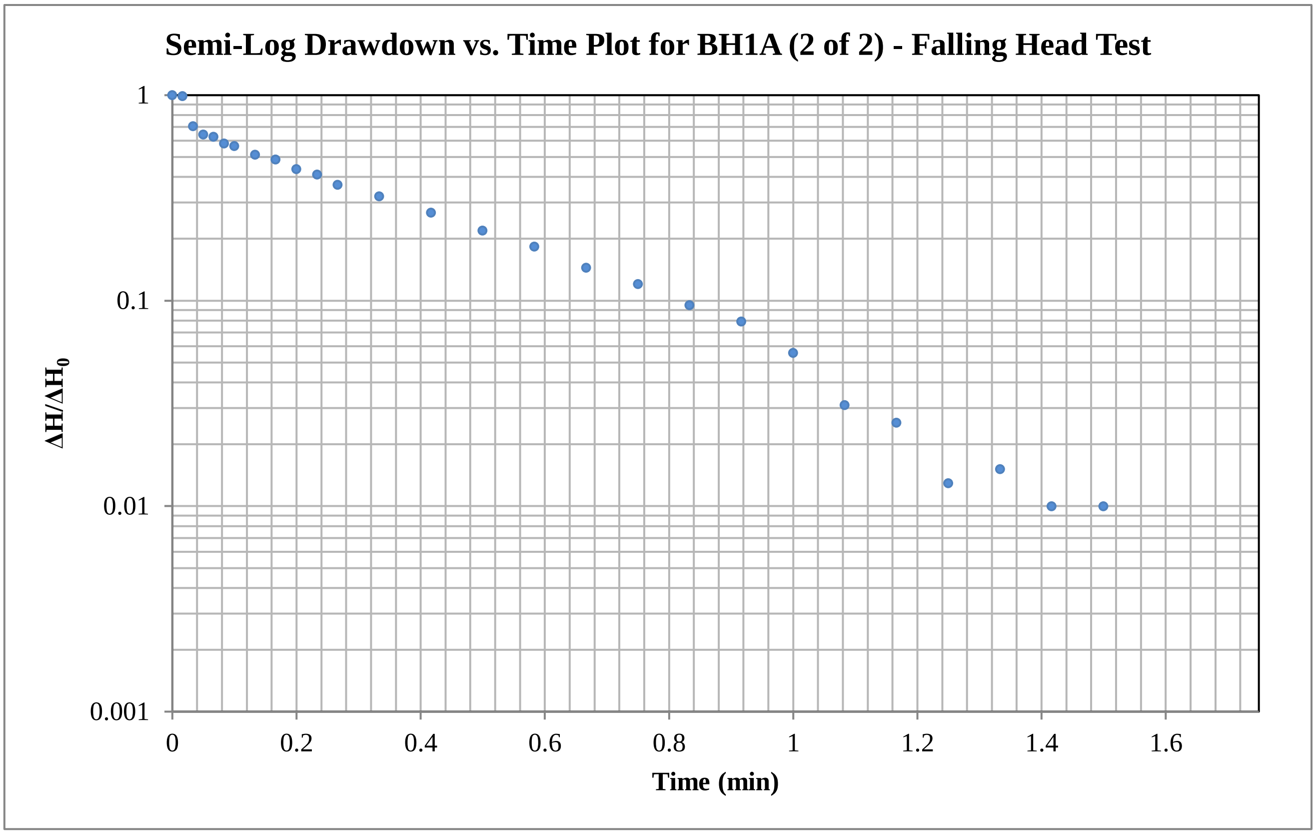
Data Points (from plot):

t*: 0.600 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 6.02E-06 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: PG5249 - EQ Homes
Test Location: BH1A
Test: Falling Head
Date: February 13, 2020



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: 3.64373

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.0318	m	Diameter of well
r_c	0.0159	m	Radius of well

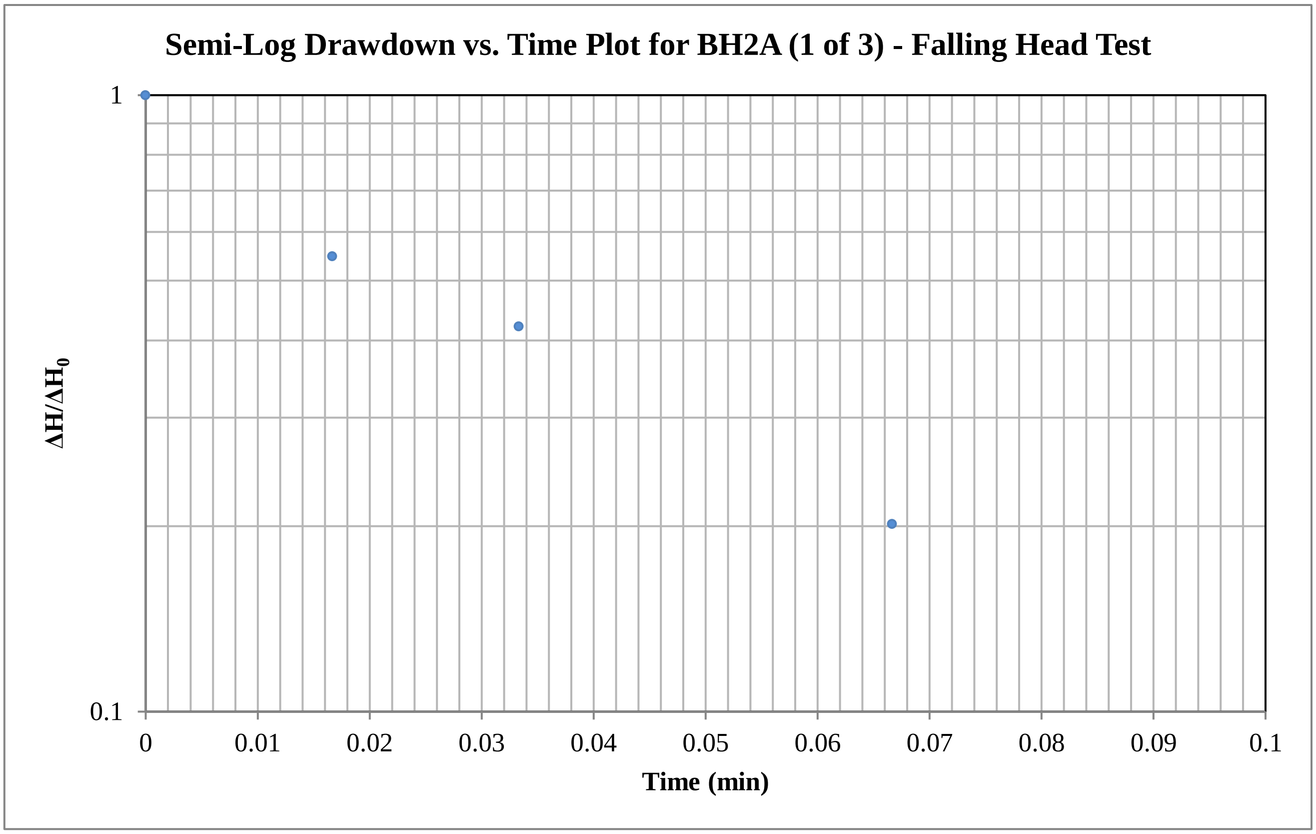
Data Points (from plot):

t*: 0.262 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.38E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: PG5249 - EQ Homes
Test Location: BH2A
Test: Falling Head
Date: February 13, 2020



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: 3.64373

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.0318	m	Diameter of well
r_c	0.0159	m	Radius of well

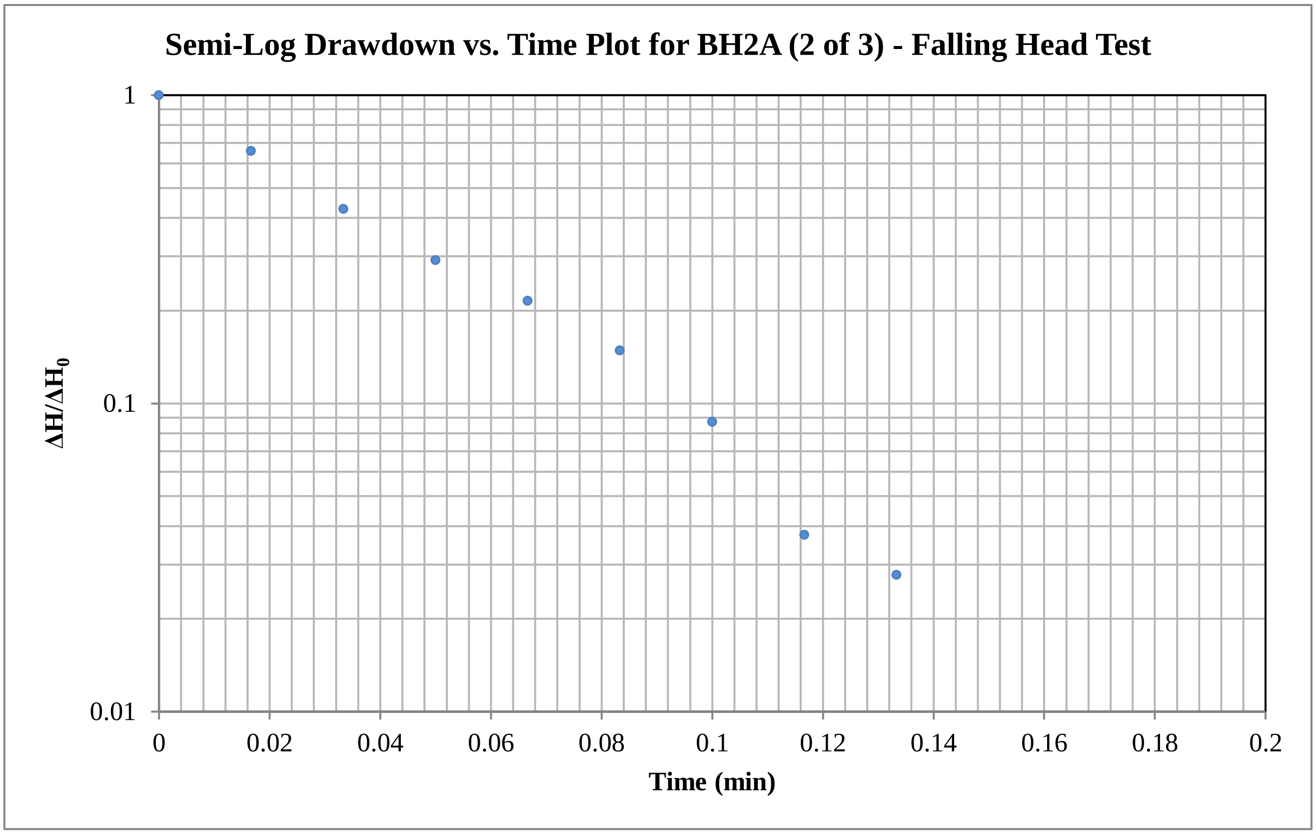
Data Points (from plot):

t*: 0.041 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 8.78E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: PG5249 - EQ Homes
Test Location: BH2A
Test: Falling Head
Date: February 13, 2020



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: 3.64373

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.0318	m	Diameter of well
r_c	0.0159	m	Radius of well

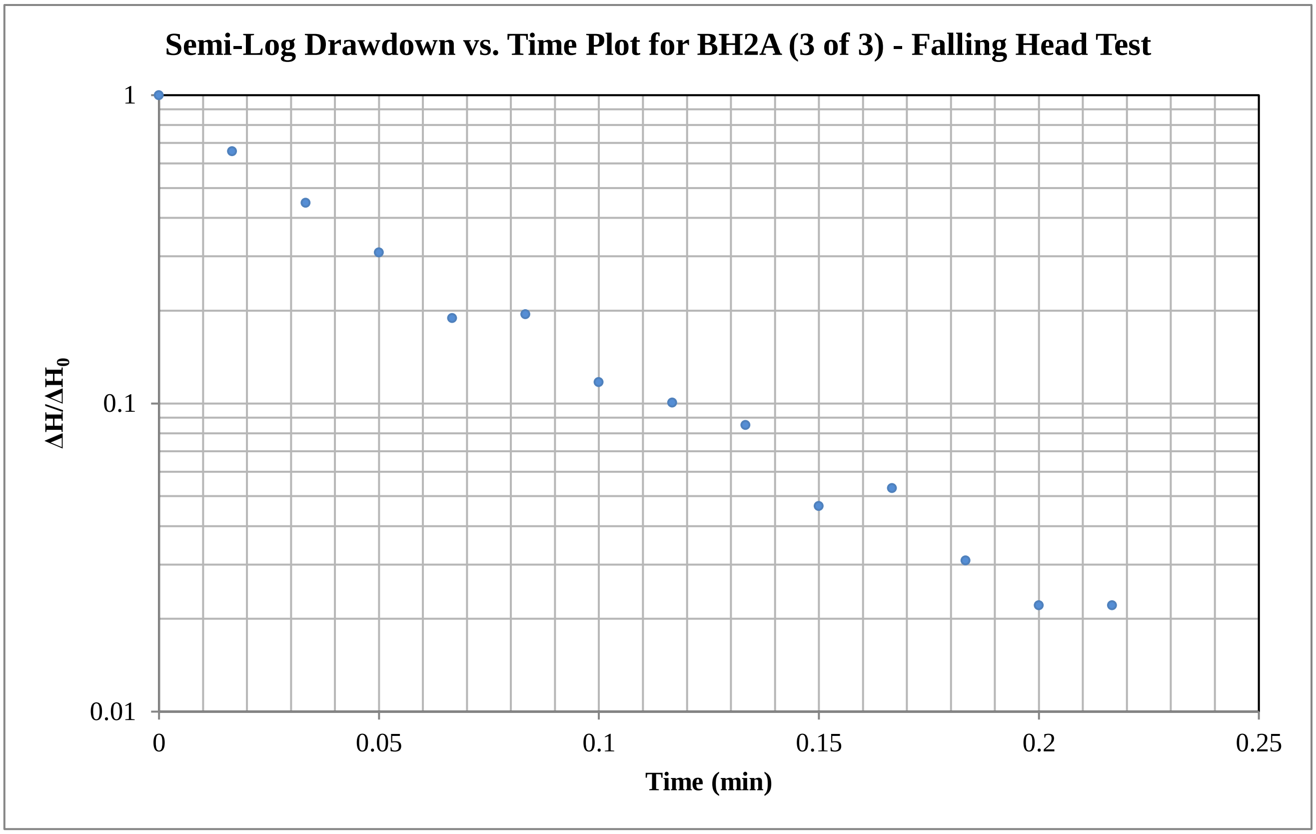
Data Points (from plot):

t*: 0.040 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 8.94E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: PG5249 - EQ Homes
Test Location: BH2A
Test: Falling Head
Date: February 13, 2020



Hvorslev Horizontal Hydraulic Conductivity

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

Hvorslev Shape Factor

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

Valid for L>>D

Hvorslev Shape Factor F: 3.64373

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.0318	m	Diameter of well
r_c	0.0159	m	Radius of well

Data Points (from plot):

t*: 0.043 minutes $\Delta H*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 8.47E-05 m/sec

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29303

Report Date: 14-Feb-2020

Order Date: 12-Feb-2020

Project Description: PG5249

	Client ID:	BH 1 30'-32'	-	-	-
	Sample Date:	10-Feb-20 16:00	-	-	-
	Sample ID:	2007297-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	66.5	-	-	-
General Inorganics					
pH	0.05 pH Units	8.33	-	-	-
Resistivity	0.10 Ohm.m	41.0	-	-	-
Anions					
Chloride	5 ug/g dry	14	-	-	-
Sulphate	5 ug/g dry	42	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - PRESSURE RELIEF CHAMBER DETAIL

DRAWING PG5249-1 - TEST HOLE LOCATION PLAN

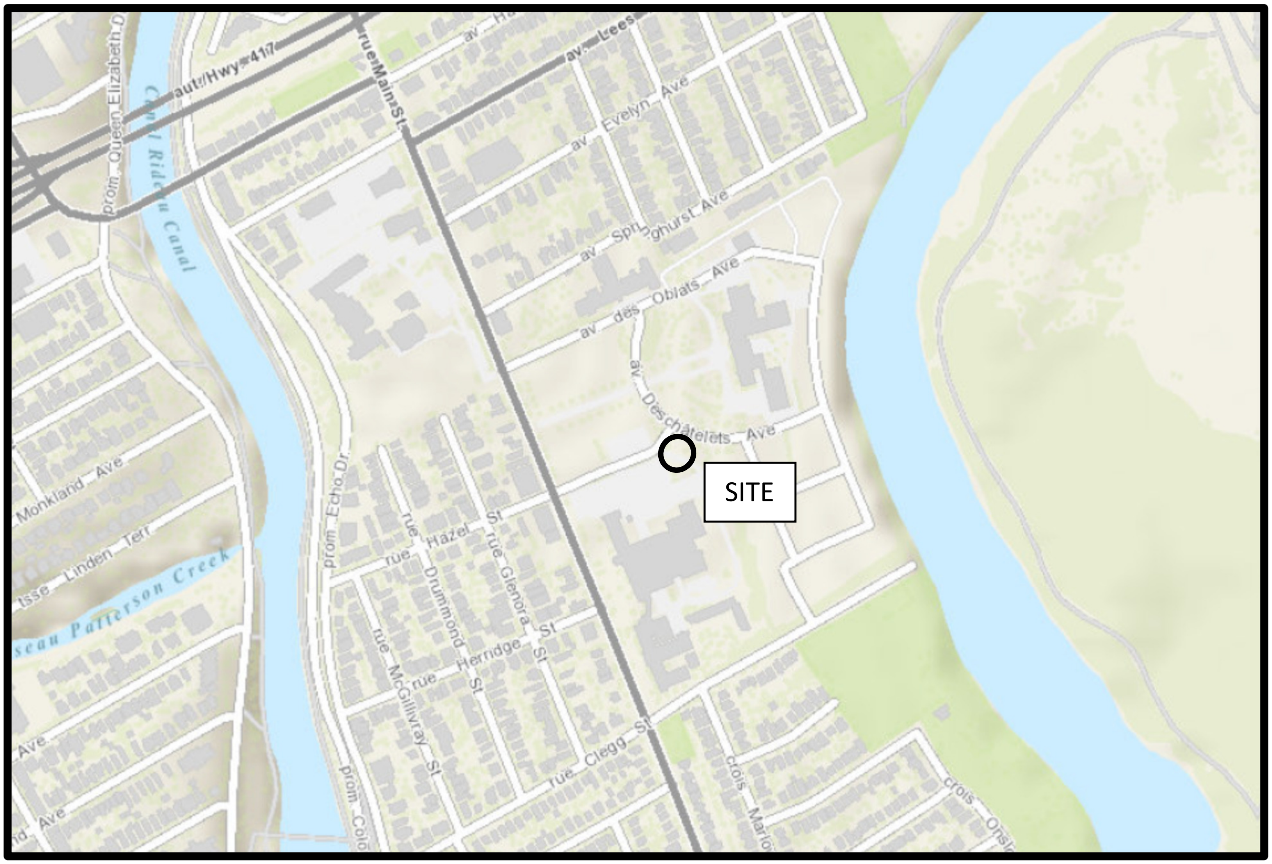


FIGURE 1

KEY PLAN

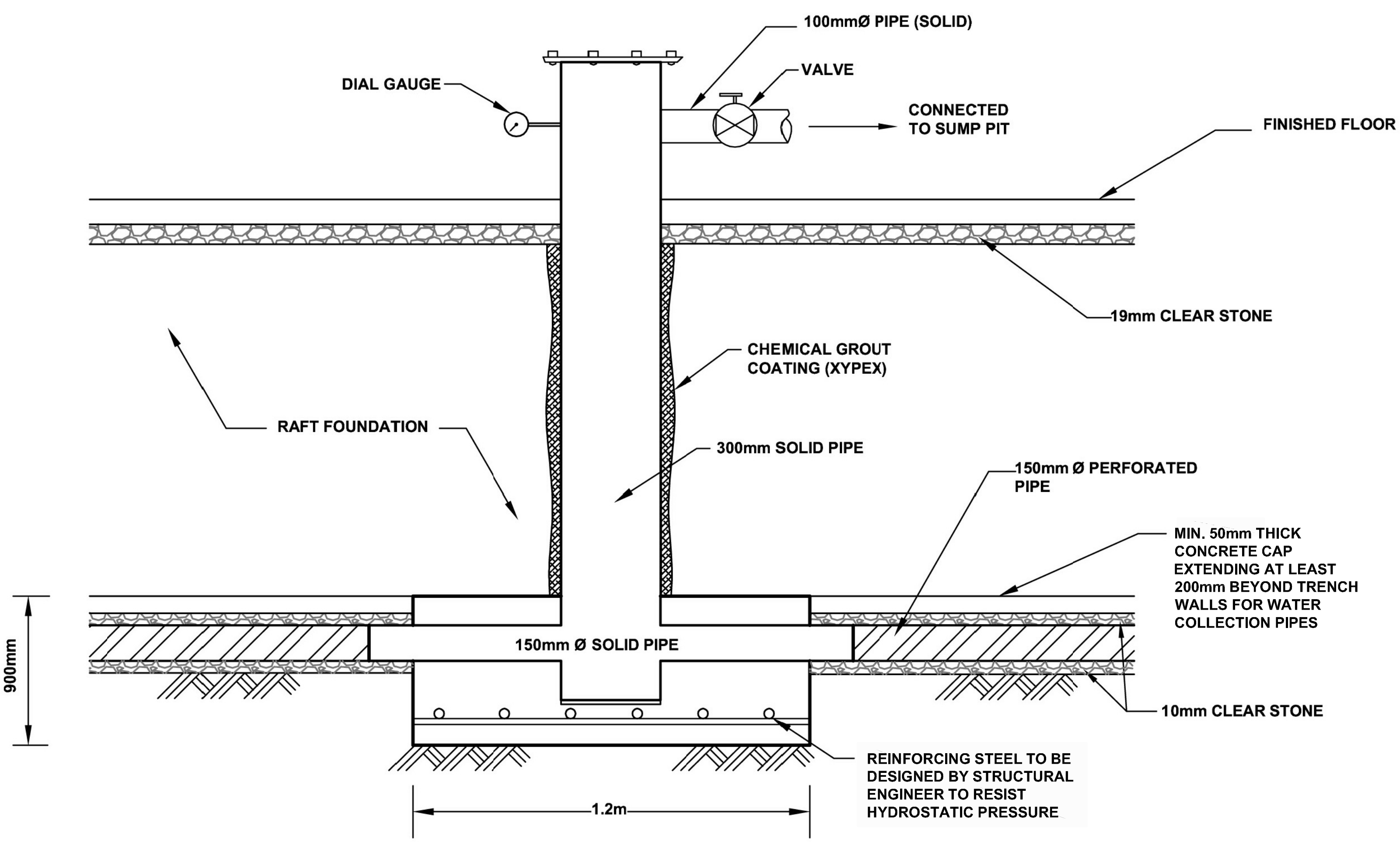
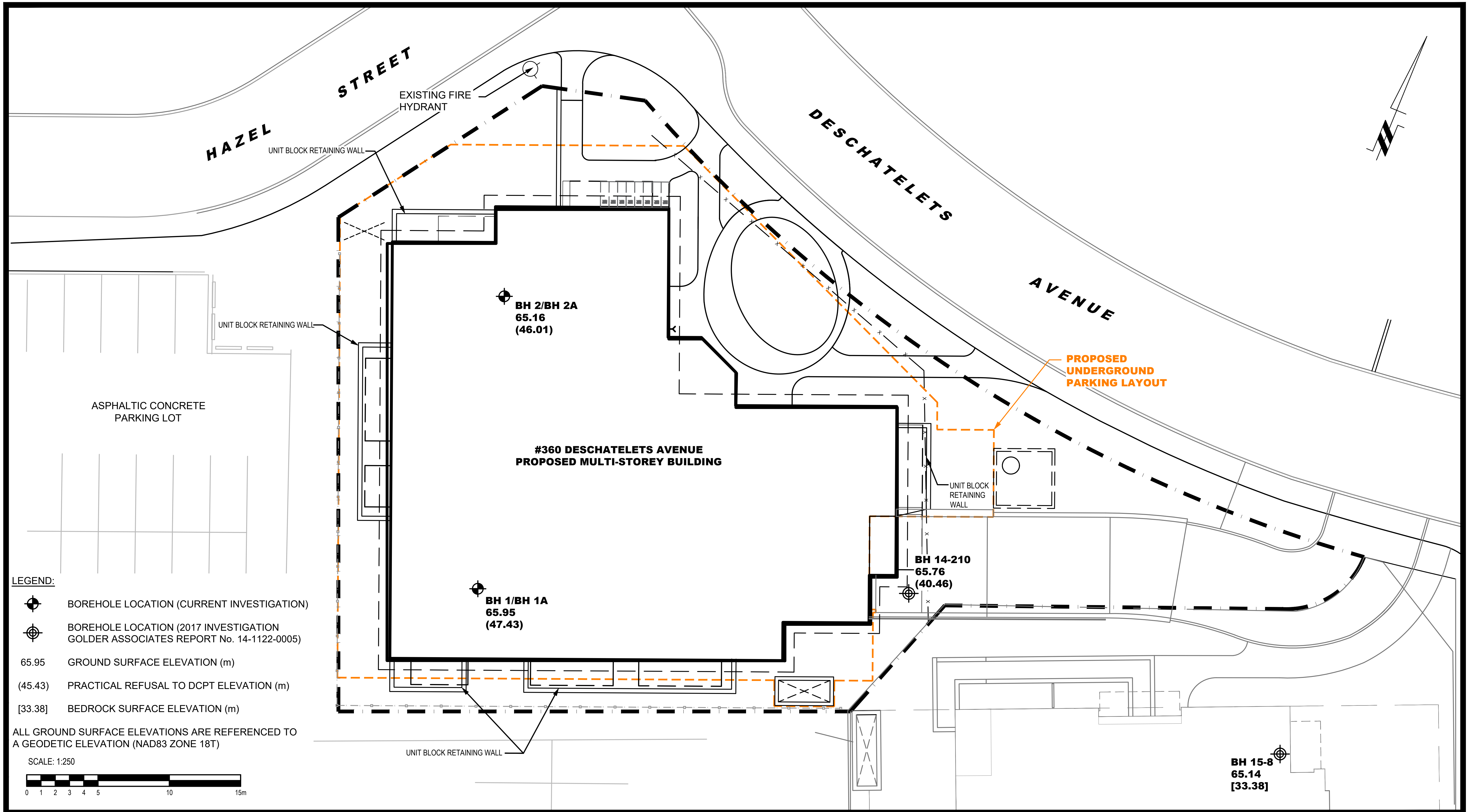


FIGURE 2 - PRESSURE RELIEF CHAMBER



LEGEND:

◉	BOREHOLE LOCATION (CURRENT INVESTIGATION)
⊕	BOREHOLE LOCATION (2017 INVESTIGATION GOLDER ASSOCIATES REPORT No. 14-1122-0005)
65.95	GROUND SURFACE ELEVATION (m)
(45.43)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)
[33.38]	BEDROCK SURFACE ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC ELEVATION (NAD83 ZONE 18T)

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

EQ HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING-360 DESCHATELETS AVENUE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:250
Drawn by: NFRV
Checked by: SD
Approved by: SD
Date: 02/2020
Report No.: PG5249-1
Dwg. No.: PG5249-1
Revision No.:

p:\autocad drawings\geotechnical\pg52xx\pg5249\pg5249-1-test hole location plan.dwg