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Geotechnical Engineering

Environmental Engineering

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Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation

Proposed Residential Development
8555 Campeau Drive - Ottawa

Prepared For

Reid's Heritage Homes

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Reid's Heritage Homes to conduct a geotechnical investigation for the proposed development to be located at 319 Huntmar 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 subsurface soil and groundwater conditions by means of boreholes.

❑ provide geotechnical recommendations for the design of the proposed development based on the results of the test holes and other soil information available.

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.

2.0 Proposed Development

Based on available drawings, the proposed development at the subject site is understood to consist of 4 multi-storey residential buildings (Building A - D) as well as a one-storey Amenity Building which will connect Buildings C and D. It is further understood that Buildings A and B will each have one level of underground parking, and that a single, shared level of underground parking will be located underlying the proposed Amenity Building, Building C and Building D. Associated access roads, parking areas and landscaped areas are also anticipated within the subject site. It is also expected that the subject site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was carried out from March 12 to 17, 2020. At that time, 15 boreholes were advanced to a maximum depth of 9.8 m below existing ground surface. Previous investigations were also completed at the subject site in October 2012, December 2010, and October 2006, consisting of a total of 2 boreholes, drilled to a maximum depth of 9.8 m below original ground surface, and 1 test pit, excavated to approximately 4.1 m below original ground surface. The test hole locations are presented on Drawing PG5287-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track-mounted auger drill rig operated by a two person crew and the test pit was excavated using a hydraulic shovel. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the auger flights or a 50 mm diameter split-spoon sampler. The split-spoon samples were placed in sealed plastic bags. All the samples were transported to our laboratory. 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 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 was carried out in cohesive soils using a field vane apparatus.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 5-20 and BH 15-20. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations

Groundwater

Flexible piezometers were installed in all the boreholes to monitor the groundwater level subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground services. The location and ground surface elevation at each borehole location was surveyed by Paterson personnel with respect to a geodetic datum. Borehole locations and ground surface elevations at the borehole locations are presented on Drawing PG5287-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples recovered from our field investigation were visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after report completion, unless otherwise directed.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample 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 6.8.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped with the exception of a gravel-surfaced access road, which runs northwest to southeast through the center of the site, and a gravel finished parking area located in the northeast corner of the site. However, based on available aerial photos, a single family residential dwelling and multiple agricultural buildings were located along the northwestern portion of the property as recently as 1991, and were no longer present in 1999. Reference should be made to the aerial photographs in Figure 2 - Aerial Photograph - 1991 and Figure 3 - Aerial Photograph - 2017 which illustrate the former and present site conditions, respectively.

The subject site is bordered to the northeast by Huntmar drive, to the south and southeast by the Trans-Canada Highway 417, and to the west and northwest by treed and grassed areas as well as an asphalt-surfaced access road leading to an existing commercial development. The existing ground surface across the subject site generally slopes down from the southwest to the northeast from approximate geodetic elevation 102 to 100 m.

4.2 Subsurface Profile

Generally, the subsurface profile at the test hole locations consists of an approximate 0.1 to 0.2 m thick topsoil layer overlying either a hard to very stiff brown silty clay crust or a fill layer up to 2.3 m in thickness, which is underlain by the silty clay crust. Underlying the silty clay crust, a deep deposit of stiff to firm grey silty clay was encountered at an approximate depth of 4.6 m below the existing ground surface.

Practical refusal to the DCPT was encountered at a depth of 19.4 m in borehole BH 5 and 13.8 m in BH 15.

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 with an overburden thickness of 10 to 25 m.

4.3 Groundwater

Groundwater level readings were measured at the piezometer locations on March 20, 2020. The observed groundwater levels are summarized in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Levels (m)	Groundwater Elevation (m)	Recording Date
BH1	100.98	Peizometer Blocked	-	March 20, 2020
BH2	99.47	Peizometer Blocked	-	March 20, 2020
BH3	100.37	1.39	98.98	March 20, 2020
BH4	100.26	0.92	99.34	March 20, 2020
BH5	100.66	Peizometer Blocked	-	March 20, 2020
BH6	101.13	2.05	99.08	March 20, 2020
BH7	101.18	Peizometer Blocked	-	March 20, 2020
BH8	101.47	Peizometer Blocked	-	March 20, 2020
BH9	101.60	2.09	99.51	March 20, 2020
BH10	101.74	1.83	99.91	March 20, 2020
BH11	101.98	1.87	100.11	March 20, 2020
BH12	101.44	Peizometer Blocked	-	March 20, 2020
BH13	100.73	1.65	99.08	March 20, 2020
BH14	100.33	1.09	99.24	March 20, 2020
BH15	99.97	0.76	99.21	March 20, 2020

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

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Long-term groundwater levels can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is expected between 3 to 5 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that each proposed multi-storey building be founded on one of the following:

☐ a raft foundation bearing on an undisturbed, silty clay bearing surface.

☐ a deep foundation, consisting of end-bearing piles, which extend to the bedrock surface.

Further, it is expected that the single-storey Amenity Building, and any portions of the underground parking levels which extend beyond the overlying building footprint, could be supported on conventional shallow footings bearing on an undisturbed, stiff silty clay bearing surface.

Due to the presence of a deep silty clay deposit, a permissible grade raise restriction is required for the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings and other settlement sensitive structures.

Should they be encountered, existing foundation walls and other construction debris should be entirely removed from within each building perimeter and within the lateral support zones of the foundations. Existing foundation walls and other construction debris are not considered suitable for reuse at the site. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Granular material should be tested and approved prior to delivery to the site. The fill should be placed in loose lifts of 300 mm thickness or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD).

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

Protection of Subgrade (Raft Foundation)

Where a raft foundation is to be constructed, as the subgrade material will most likely consist of a silty clay deposit, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the 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 to potential disturbance due to drying.

Compacted Granular Fill Working Platform (Pile Foundation)

For proposed buildings to be supported on a driven pile foundation, which will require the use of heavy equipment (i.e. pile driving crane), it is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance.

A typical working platform could consist of 600 mm of OPSS Granular B Type II material, placed and compacted to a minimum of 98% of its SPMDD in lifts not exceeding 300 mm in thickness.

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

5.3 Foundation Design

Conventional Spread Footings

Pad footings, up to 6 m wide, and strip footings up to 3 m wide, placed on an undisturbed, stiff silty clay 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 **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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.

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

Raft Foundation

As noted above, it is expected that a raft foundation will be required to support the proposed multi-storey buildings. For our design calculations, one level of underground parking was assumed which would extend approximately 4 m below existing ground surface (corresponding to approximate geodetic elevation of 96.0 - 98.0 m). The maximum SLS contact pressure is **195 kPa** for a raft foundation bearing on the undisturbed, stiff silty clay bearing surface. It should be noted that the weight of the raft slab and everything above has to be included when designing with this value. 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 factored bearing resistance (contact pressure) at ULS can be taken as **275 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 **195 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, silty clay is not elastic and limits have to be placed on the stress ranges of a particular modulus.

The proposed building can be designed using the above parameters with total and differential settlements of 25 and 15 mm, respectively.

End Bearing Pile Foundation

Where the proposed building loads exceed the recommended contact pressures for a raft foundation, a deep foundation system driven to refusal in the bedrock is recommended for foundation support of the proposed multi-storey buildings. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance values at ultimate limit states (ULS) are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance Factored at ULS (kN)
245	9	1460
245	11	1650
245	13	1760

The minimum centre-to-centre pile spacing is 2.5 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

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

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **2 m** is recommended within 5 m of the proposed buildings. A permissible grade raise restriction of **3 m** is recommended in the parking areas and access lanes. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations.

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

A seismic site response **Class D** should be used for design of the proposed buildings at the subject site according to the OBC 2012. The soils underlying the site are not susceptible to liquefaction.

5.5 Basement Slab

If a raft slab is considered, a granular layer of OPSS Granular A crushed stone will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

For a building founded on footings or piles, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab. The spacing of the sub-slab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, if any. 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 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 = $(1.45\text{-}a_{max}/g)a_{max}$
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

$P_o = .5\ K_o\ \gamma\ H^2$, 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

For design purposes, the pavement structures presented in the following tables could be used for the design of car only parking areas and access lanes.

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 - Acces Lanes

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
400	**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 98% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

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

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

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 each proposed building. The system should consist of a 100 to 150 mm diameter, geotextile-wrapped, perforated, corrugated plastic pipe surrounded on all sides by 150 mm of 10 mm clear crushed stone which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Sub-slab Drainage

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

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 materials will be frost susceptible and, as such, are not recommended for placement as backfill against the foundation walls unless used in conjunction with a composite drainage system, such as Delta Drain 6000. Imported granular materials, such as clean sand or OPSS Granular B Type I granular materials, should be placed for this purpose.

Foundation Raft Slab Construction Joints

Where utilized, 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.

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 of soil cover should be provided for adequate frost protection of heated structured.

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

The proposed underground parking levels may require protection against frost action depending on the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

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.

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 flatter. The flatter 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.

Temporary Shoring

Temporary shoring may be required to support the overburden soils. 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.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural designer prior to implementation.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure.

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.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A crushed stone 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 SPMDD.

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

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.

To reduce long-term lowering of the groundwater level at the subject site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of a relatively dy and compatible 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

Due to the relatively impervious nature of the silty clay materials, 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.

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.

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.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials into the trenches. Pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

6.7 Limit of Hazard Lands

A slope stability analysis was previously conducted at the subject site by Paterson in 2012. From the analysis, a geotechnical limit of hazard lands setback line was provided along the top of slope for the valley corridor walls of the Feedmill Creek valley corridor.

The subject section of the Feedmill Creek is located within a 40 to 50 m wide valley corridor with a 1 to 3 m high valley wall. The valley corridor is less defined within the west portion of the site, where the valley walls are close to 1 m or less. The majority of the slope face was noted at the time to be grass covered with minor surficial erosional activities noted. Some sloughing and minor undercutting along the valley wall face was noted where the watercourse has meandered in close proximity to the slope.

The limit of hazard lands includes a 6 m erosion access allowance taken from the top of slope. It should be noted that based on the analysis results, the majority of the slope was considered stable.

The toe erosion allowance for the valley corridor wall slopes was based on the cohesive nature of the soils, the erosional activities observed at the time of investigation and the width of the watercourse. Signs of erosion were noted along the existing watercourse during the 2012 investigation, especially where the watercourse meandered in close proximity to the toe of the corridor wall. It was considered that a toe erosion allowance of 5 m was appropriate for the corridor walls confining the existing watercourse. The toe erosion allowance should be applied from the top of stable slope, where the watercourse has meandered to within 10 m of the slope toe. The toe erosion allowance should be taken from the bank full water's edge in areas, where the watercourse is greater than 10 m from the toe of the existing slope.

A site visit was conducted on March 16, 2020 to investigate current site conditions. The slope along the subject section of the Feedmill Creek was noted to be treed and grass covered. No signs of sloughing or undercutting was observed.

As no signs of erosion were observed during the current investigation and the slope face was noted to be treed and grassed, the original limit of hazard lands setback limits are still applicable.

6.8 Corrosion Potential and Sulphate

One (1) sample was submitted for testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.01%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a sightly to moderately 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 the grading plan 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.

☐ Observation of clay seal placement at specified locations.

☐ Field density tests to ensure 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 the grading plan once available. Paterson's recommendations should also be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole log 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 Reid's Heritage Homes or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Kevin A. Pickard, EIT

LICENSED PROFESSIONAL ENGINEER
May 5, 2021
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng

Report Distribution:

☐ Reid's Heritage Homes
☐ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
8555 Campeau Drive
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5287

REMARKS

HOLE NO. BH 1-20

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 13

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	100.98		
TOPSOIL 0.08		AU	1						
FILL: Brown silty clay, trace sand		SS	2	67	16	1	99.98		
2.13		SS	3	12	8	2	98.98		
Very stiff, brown **SILTY CLAY**		SS	4	100	12	3	97.98		
3.66		SS	5	100	17				
End of Borehole									

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
8555 Campeau Drive
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5287**

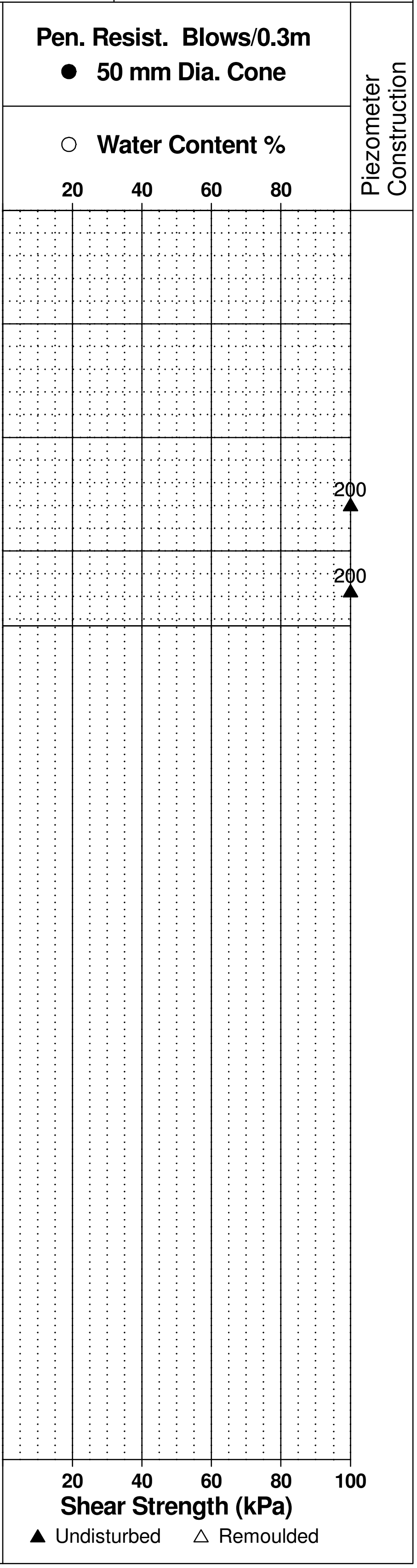
REMARKS

HOLE NO. **BH 2-20**

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 13

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.47
TOPSOIL 0.20		AU	1				
FILL: Brown silty clay, trace gravel 1.37		SS	2	12	2	1	98.47
Hard, brown **SILTY CLAY**		SS	3	75	6	2	97.47
3.66						3	96.47
End of Borehole							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM Geodetic

REMARKS

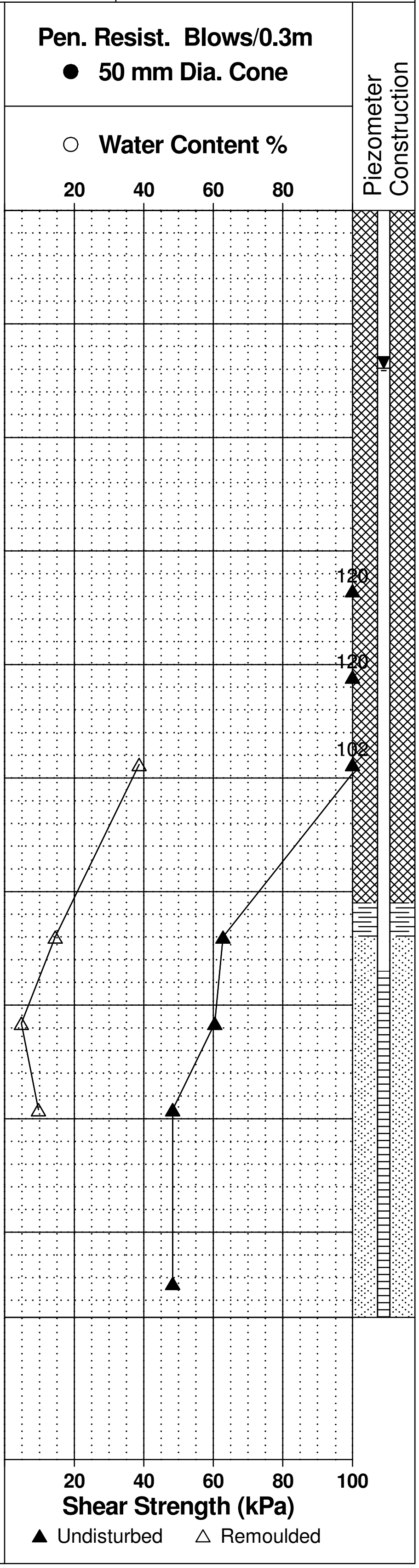
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 12

FILE NO. PG5287

HOLE NO. BH 3-20

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	100.37
FILL: Brown silty sand with crushed stone	AU	1				
0.53						
FILL: Brown silty clay with gravel, some sand	SS	2	46	4	1	99.37
	SS	3	79	6	2	98.37
2.30						
Hard to very stiff, brown **SILTY CLAY**	SS	4	100	6	3	97.37
	SS	5	100	P	4	96.37
- stiff to firm and grey by 5.3m depth	SS	6	100	2	5	95.37
					6	94.37
					7	93.37
					8	92.37
					9	91.37
9.75						
End of Borehole						
(GWL @ 1.39m - March 20, 2020)						



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DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 13

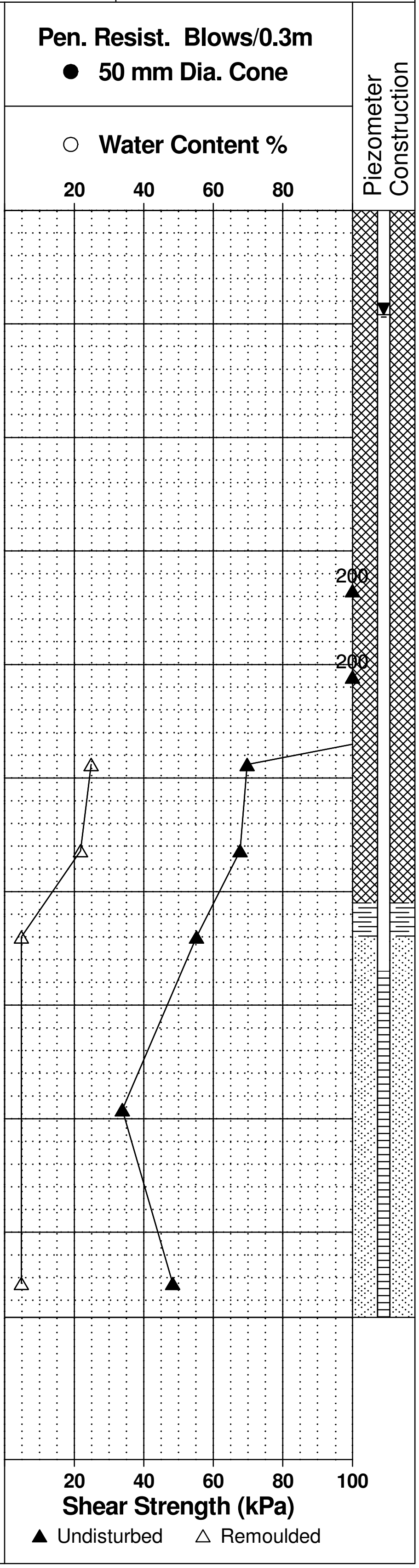
FILE NO. PG5287

HOLE NO. BH 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
TOPSOIL 0.18		AU	1			0	100.26
FILL: Brown silty clay, trace sand and gravel		SS	2	0	6	1	99.26
2.13		SS	3	38	4	2	98.26
		SS	4	100	8		
						3	97.26
Hard to very stiff, brown **SILTY CLAY**						4	96.26
- stiff to firm and grey by 4.6m depth						5	95.26
						6	94.26
						7	93.26
						8	92.26
						9	91.26
9.75							

End of Borehole

(GWL @ 0.92m - March 20, 2020)



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Ottawa, Ontario

DATUM Geodetic

REMARKS

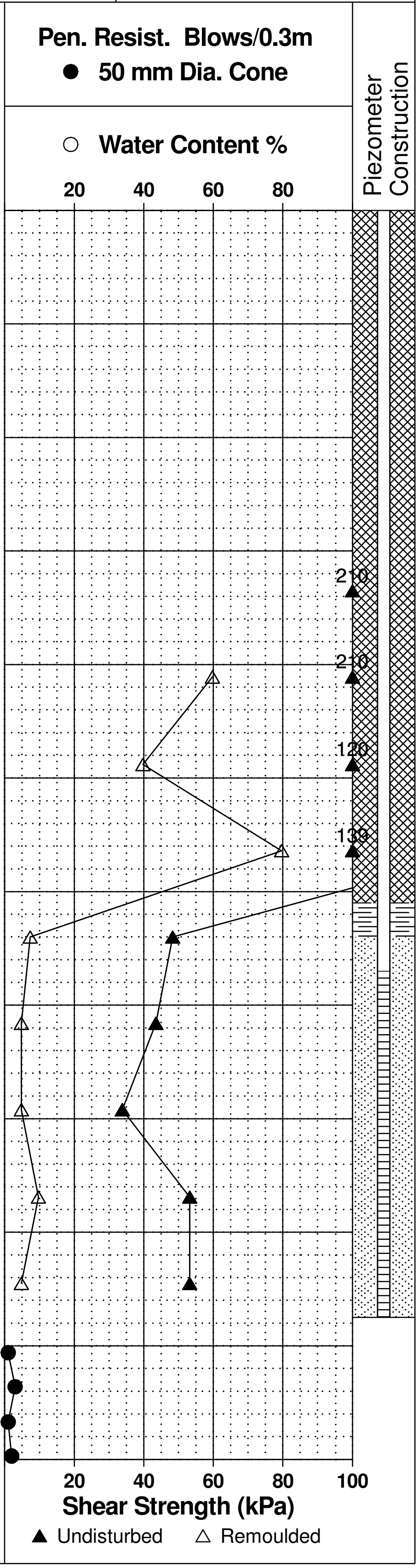
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 13

FILE NO. PG5287

HOLE NO. BH 5-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL 0.10		AU	1			0	100.66
		SS	2	29	5	1	99.66
		SS	3	88	6	2	98.66
		SS	4	100	7		
						3	97.66
Hard to very stiff, brown **SILTY CLAY**						4	96.66
						5	95.66
- stiff to firm and grey by 6.1m depth						6	94.66
						7	93.66
						8	92.66
9.75						9	91.66
Dynamic Cone Penetration Test commenced at 9.75m depth.						10	90.66
						11	89.66



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
8555 Campeau Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

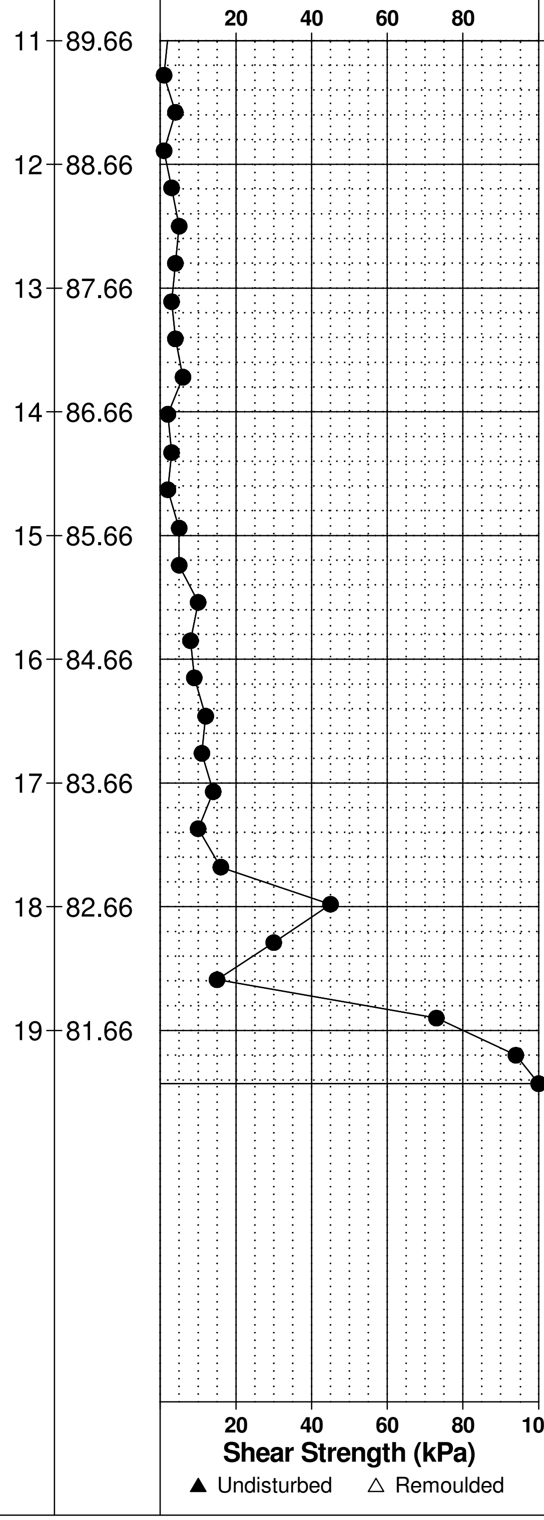
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 13

FILE NO. PG5287

HOLE NO. BH 5-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE									
19.43									
End of Borehole Practical DCPT refusal at 19.43m depth. (BH dry - March 20, 2020)									



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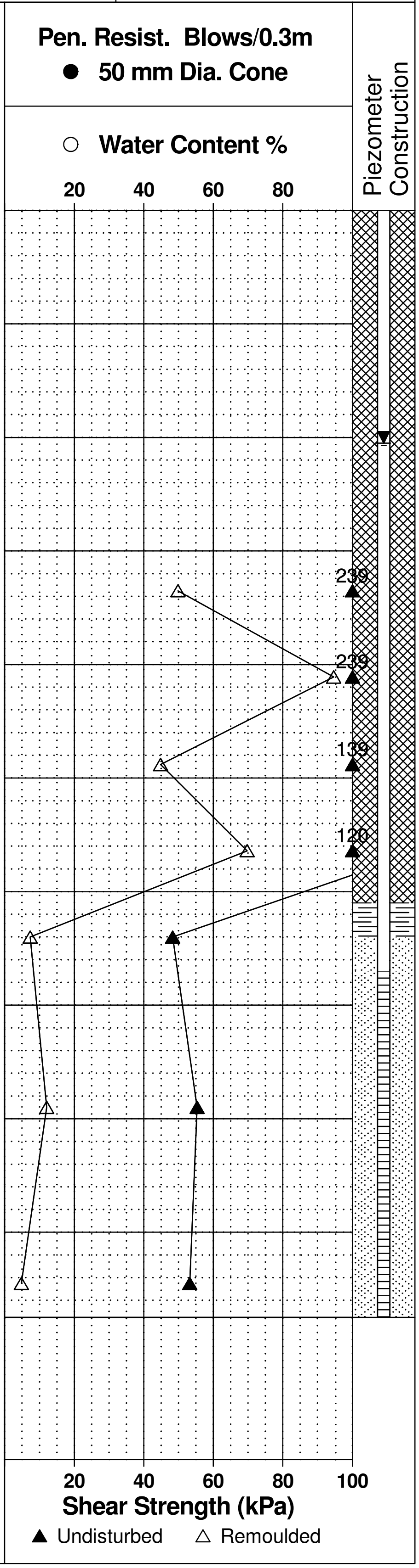
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** 2020 March 16

FILE NO. PG5287

HOLE NO. BH 6-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	101.13
TOPSOIL 0.13	AU	1				
	SS	2	29	8	1	100.13
	SS	3	4	9	2	99.13
	SS	4	100	7		
					3	98.13
Hard to very stiff, brown **SILTY CLAY**					4	97.13
					5	96.13
- firm to stiff and grey by 6.10m depth					6	95.13
					7	94.13
					8	93.13
					9	92.13
9.75						
End of Borehole						
(GWL @ 2.05m - March 20, 2020)						



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 13

FILE NO. PG5287

HOLE NO. BH 7-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	101.18		
TOPSOIL 0.20		AU	1						
FILL: Brown silty clay, trace crushed stone 1.37		SS	2	8	6	1	100.18		
Very stiff, brown **SILTY CLAY**		SS	3	62	7	2	99.18		
		SS	4	92	7	3	98.18		
3.66		SS	5	100	9				
End of Borehole									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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REMARKS

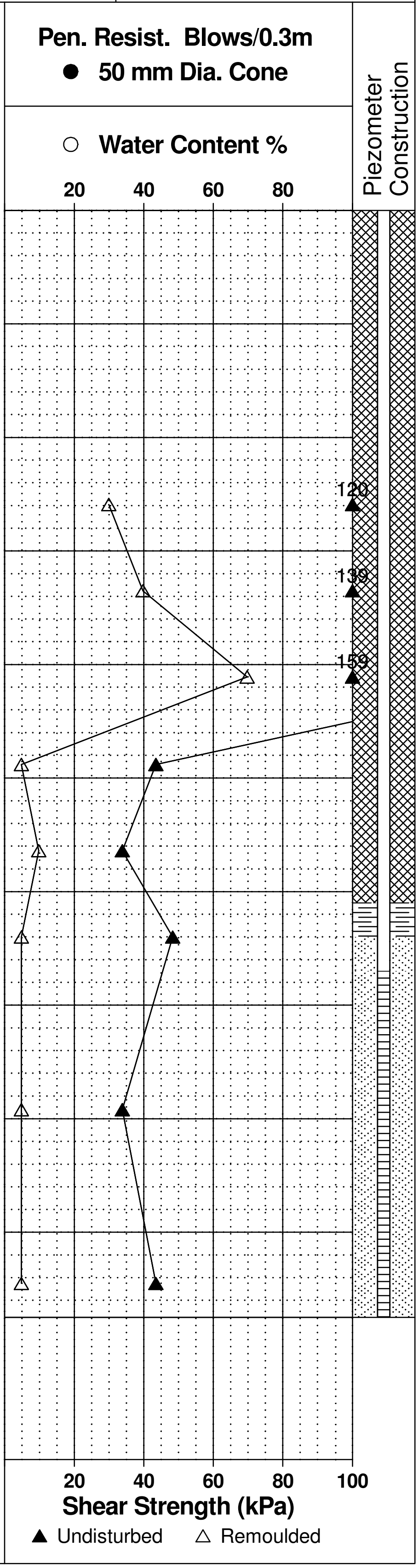
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 16

FILE NO. PG5287

HOLE NO. BH 8-20

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	101.47
TOPSOIL 0.18	AU	1				
	SS	2	75	7	1	100.47
	SS	3	100	3	2	99.47
Very stiff, brown **SILTY CLAY**					3	98.47
					4	97.47
- stiff to firm and grey by 4.6m depth					5	96.47
					6	95.47
					7	94.47
					8	93.47
9.75					9	92.47
End of Borehole						
(BH dry - March 20, 2020)						



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5287**

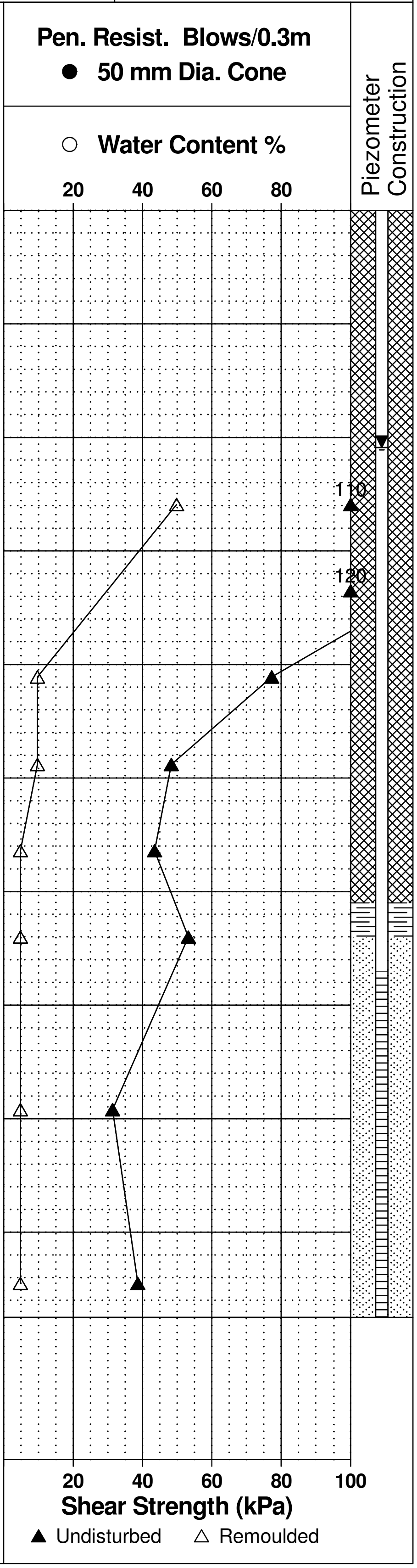
REMARKS

HOLE NO. **BH 9-20**

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	101.60
TOPSOIL 0.20		AU	1				
		SS	2	75	4	1	100.60
		SS	3	67	4	2	99.60
Hard to stiff, brown **SILTY CLAY**						3	98.60
						4	97.60
- stiff to firm and grey by 4.6m depth						5	96.60
						6	95.60
						7	94.60
						8	93.60
						9	92.60
9.75							
End of Borehole							
(GWL @ 2.09m - March 20, 2020)							



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FILE NO. PG5287

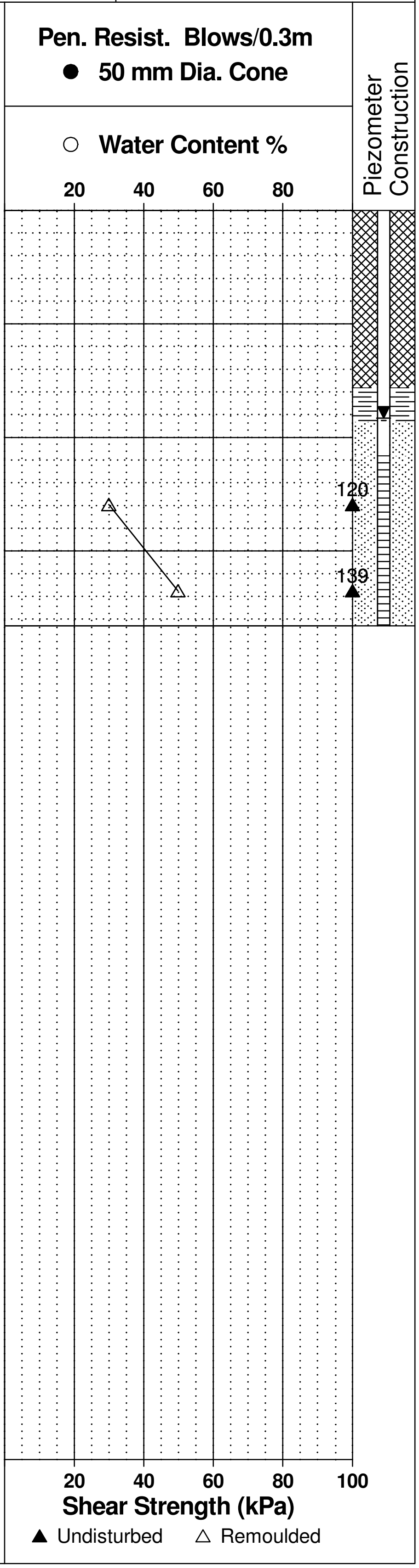
REMARKS

HOLE NO. BH10-20

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	101.74
TOPSOIL 0.15		AU	1				
Very stiff, brown **SILTY CLAY**		SS	2	79	4	1	100.74
		SS	3	88	3	2	99.74
3.66						3	98.74
End of Borehole (GWL @ 1.83m - March 20, 2020)							



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DATUM Geodetic

REMARKS

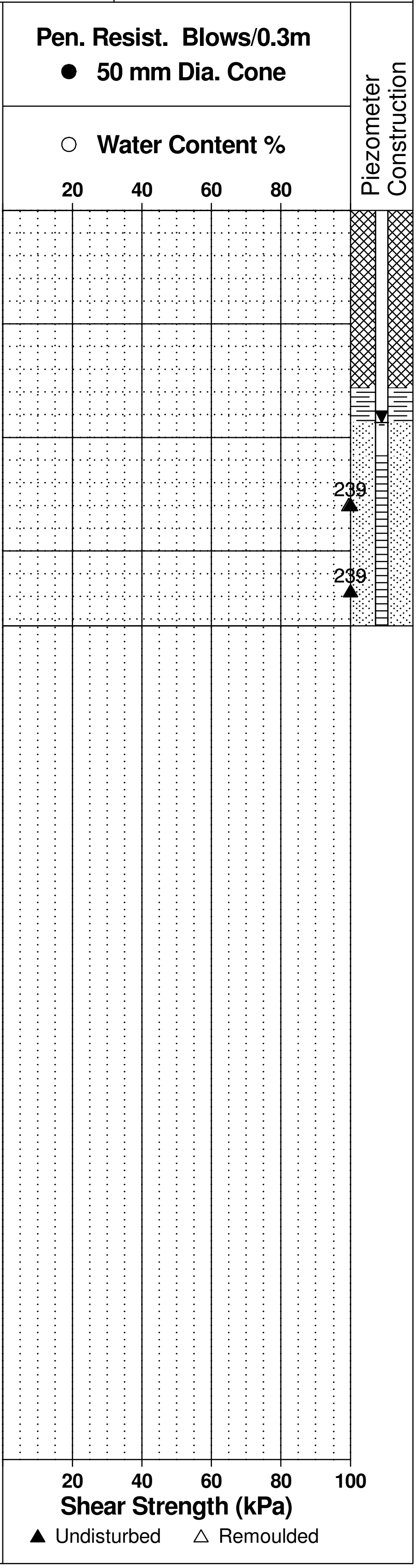
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 17

FILE NO. **PG5287**

HOLE NO. **BH11-20**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	101.98
TOPSOIL	0.20						
FILL: Brown silty clay with gravel	0.76	AU	1				
Hard, brown **SILTY CLAY**		SS	2	50	4	1	100.98
		SS	3	88	4	2	99.98
	3.66					3	98.98
End of Borehole (GWL @ 1.87m - March 20, 2020)							



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 17

FILE NO. PG5287

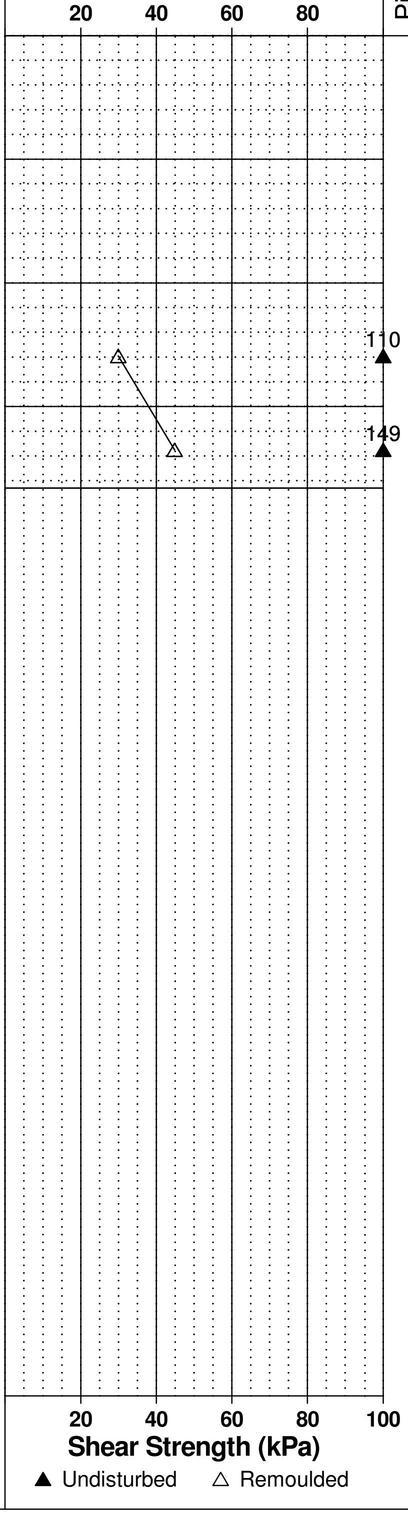
HOLE NO. BH12-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	101.44
TOPSOIL 0.20		AU	1				
Very stiff, brown **SILTY CLAY**		SS	2	83	3	1	100.44
		SS	3	96	3	2	99.44
						3	98.44
3.66 End of Borehole							

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

○ **Water Content %**

Piezometer Construction



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
8555 Campeau Drive
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5287**

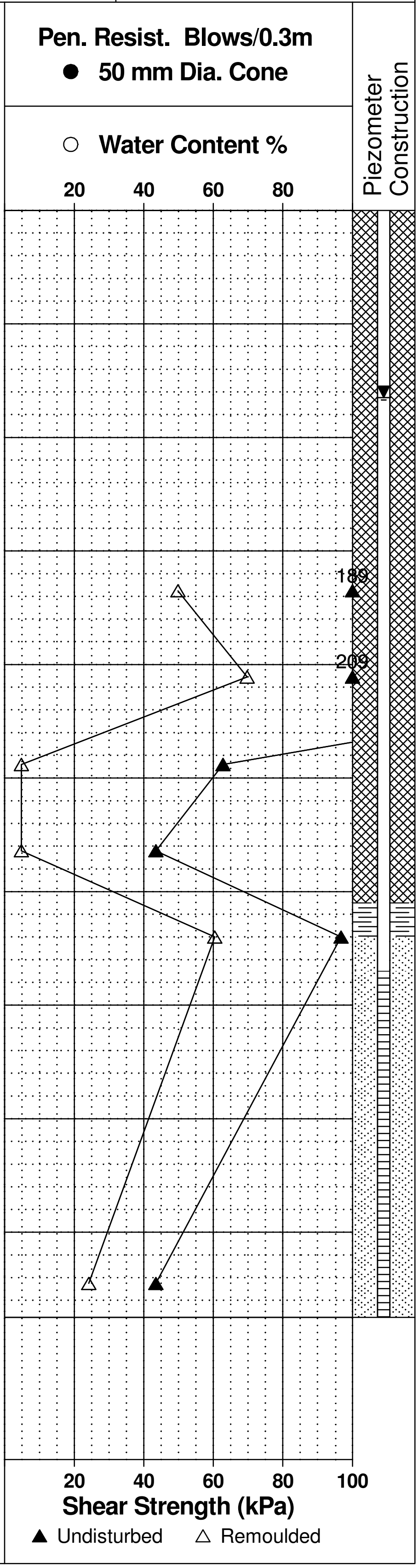
REMARKS

HOLE NO. **BH13-20**

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 16

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	100.73
TOPSOIL 0.15	AU	1				
	SS	2	54	5	1	99.73
	SS	3	100	7	2	98.73
	SS	4	100	4		
Hard to stiff, brown **SILTY CLAY**					3	97.73
					4	96.73
- stiff to firm and grey by 4.6m depth					5	95.73
					6	94.73
					7	93.73
	SS	5	100	W	8	92.73
					9	91.73
9.75						
End of Borehole						
(GWL @ 1.65m - March 20, 2020)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 12

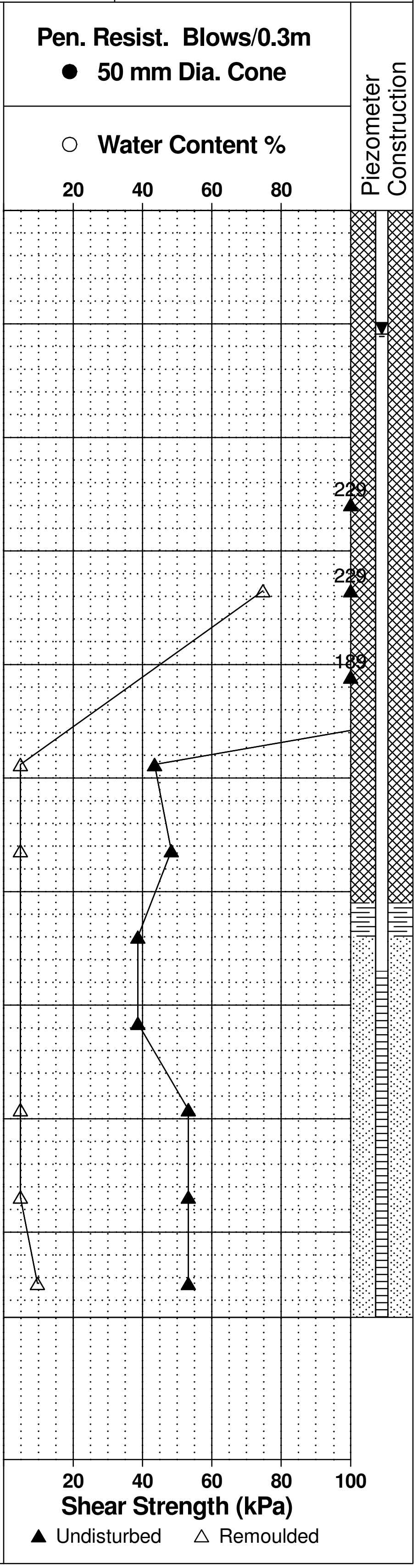
FILE NO. PG5287

HOLE NO. BH14-20

SOIL DESCRIPTION		STRATA PLOT
GROUND SURFACE		
TOPSOIL	0.08	
Hard to very stiff, brown **SILTY CLAY** - firm to stiff and grey by 4.6m depth	9.75	
End of Borehole (GWL @ 1.09m - March 20, 2020)		

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	7
SS	3	92	8

DEPTH (m)	ELEV. (m)
0	100.33
1	99.33
2	98.33
3	97.33
4	96.33
5	95.33
6	94.33
7	93.33
8	92.33
9	91.33



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DATUM Geodetic

REMARKS

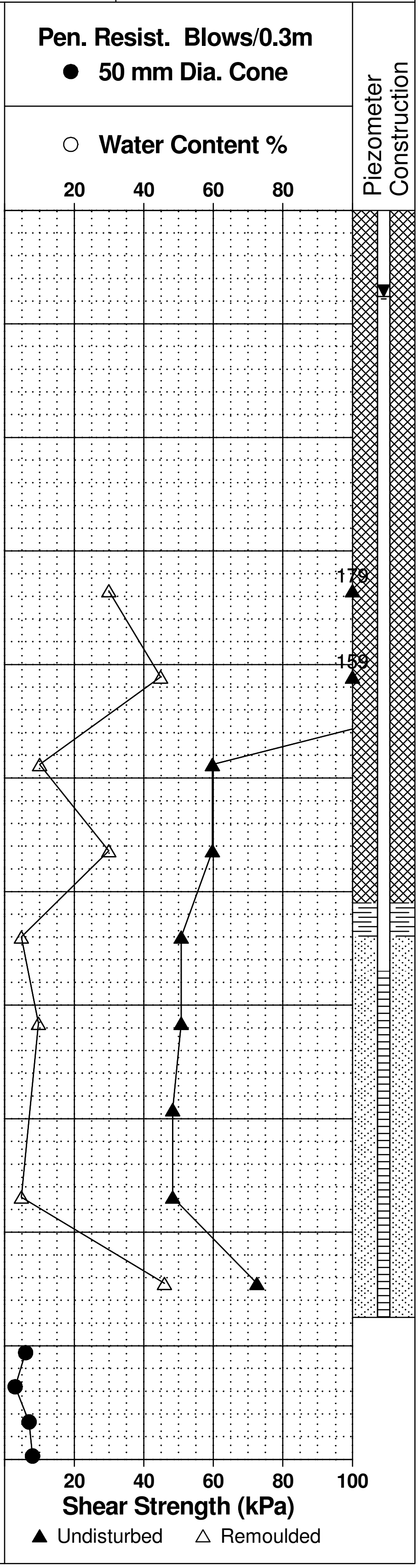
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 March 12

FILE NO. PG5287

HOLE NO. BH15-20

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.97
TOPSOIL	0.08	AU	1				
		SS	2	92	7	1	98.97
		SS	3	100	8	2	97.97
		SS	4	100	5		
Very stiff to stiff, brown **SILTY CLAY**						3	96.97
						4	95.97
- grey by 4.6m depth						5	94.97
						6	93.97
						7	92.97
						8	91.97
	9.75					9	90.97
Dynamic Cone Penetration Test commenced at 9.75m depth.						10	89.97
						11	88.97



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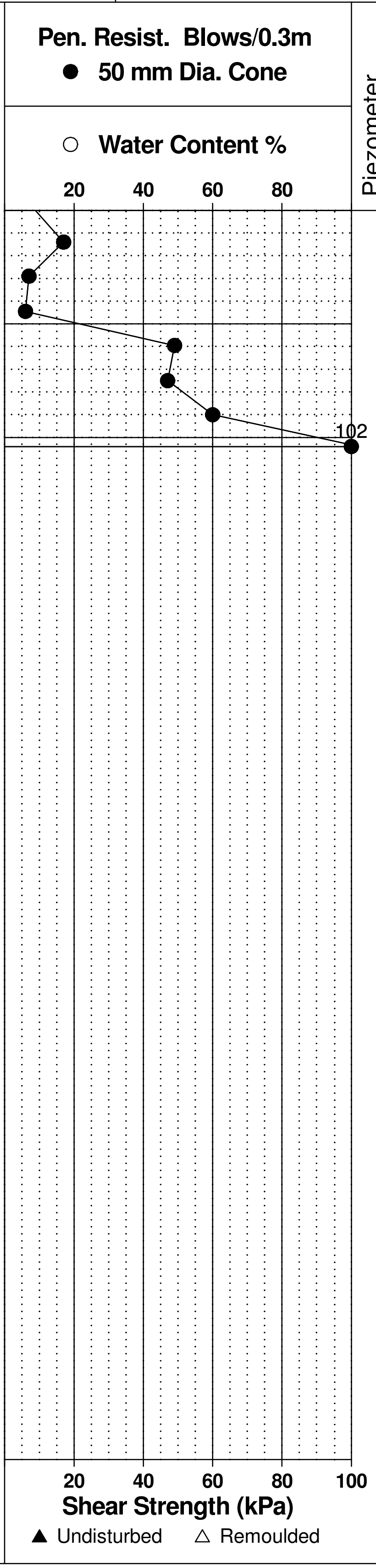
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** 2020 March 12

FILE NO. PG5287

HOLE NO. BH15-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						11	88.97
						12	87.97
13.08						13	86.97
End of Borehole Practical DCPT refusal at 13.08m depth. (GWL @ 0.76m - March 20, 2020)							



Piezometer Construction

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Campeau Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

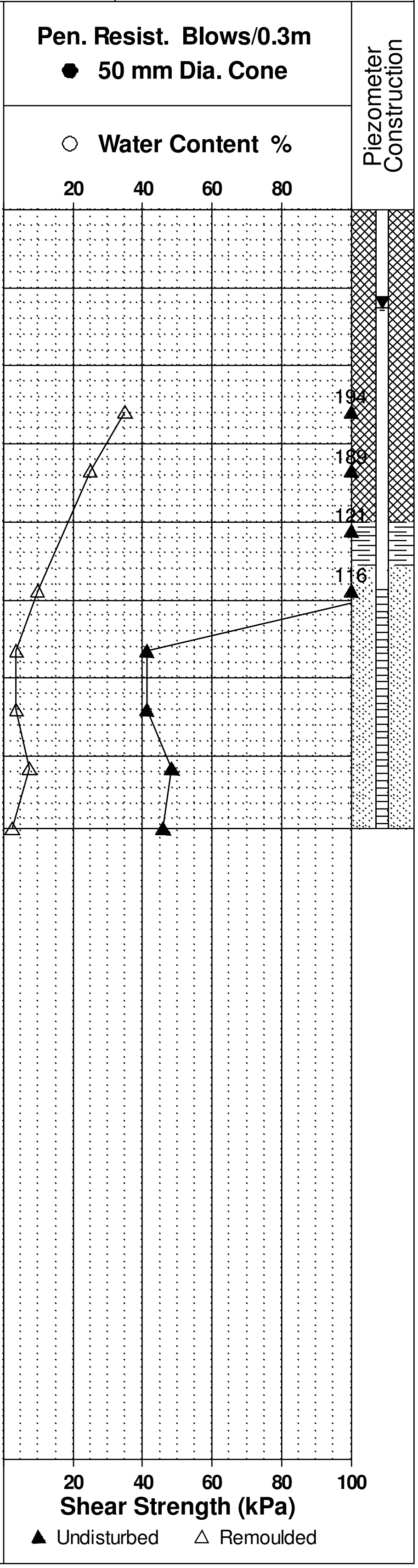
BORINGS BY CME 850X Power Auger

DATE October 24, 2012

FILE NO. PG2767

HOLE NO. BH16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL 0.25		AU	1			0	100.48
		SS	2	100	16	1	99.48
		SS	3	100	12	2	98.48
						3	97.48
Very stiff, brown **SILTY CLAY**						4	96.48
- firm by 5.1m depth						5	95.48
- grey by 5.9m depth						6	94.48
7.92						7	93.48
End of Borehole							
(GWL @ 1.26m-Nov. 22, 2012)							



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Kanata West Business Park - Huntmar Road
Ottawa, Ontario

DATUM Ground suface elevations provided by Stantec Geomatics

REMARKS

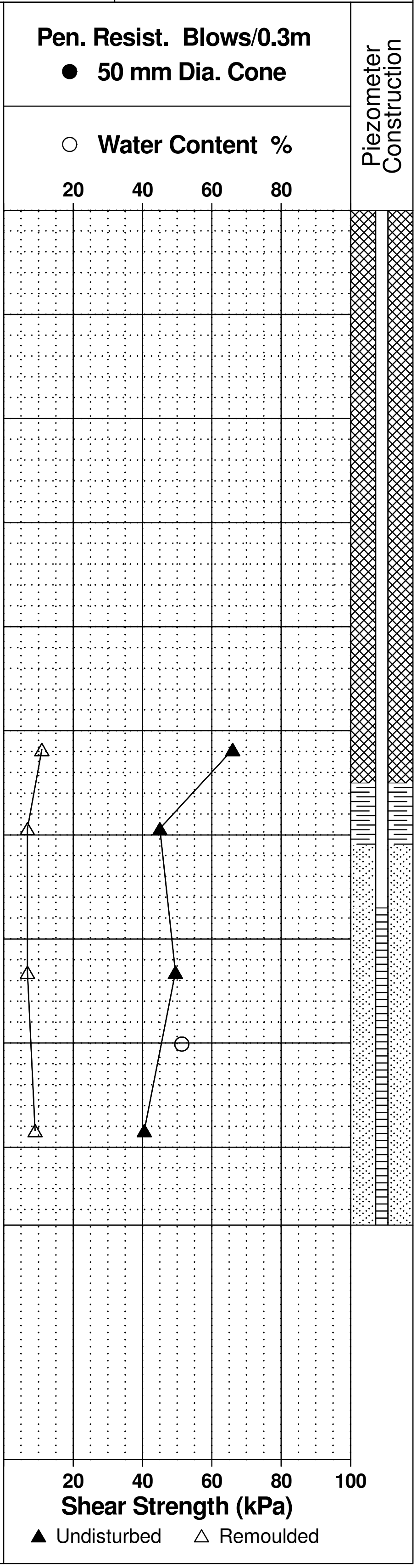
BORINGS BY CME 55 Power Auger

DATE 3 December 2010

FILE NO. PG0912

HOLE NO. BH10-10

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	100.62
TOPSOIL 0.10	AU	1				
FILL: Brown silty sand 0.76	AU	2				
	SS	3		10	1	99.62
	SS	4		11		
					2	98.62
	SS	5	92			
					3	97.62
	SS	6	100	4		
					4	96.62
Very stiff to stiff, brown **SILTY CLAY**	SS	7	100			
- firm and grey by 5.2m depth					5	95.62
					6	94.62
	TW	8				
					7	93.62
	TW	9				
					8	92.62
					9	91.62
9.75	TW	10				
End of Borehole						



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Terrace Lands - Highway 417 at Huntmar Road
Ottawa, Ontario

DATUM

REMARKS

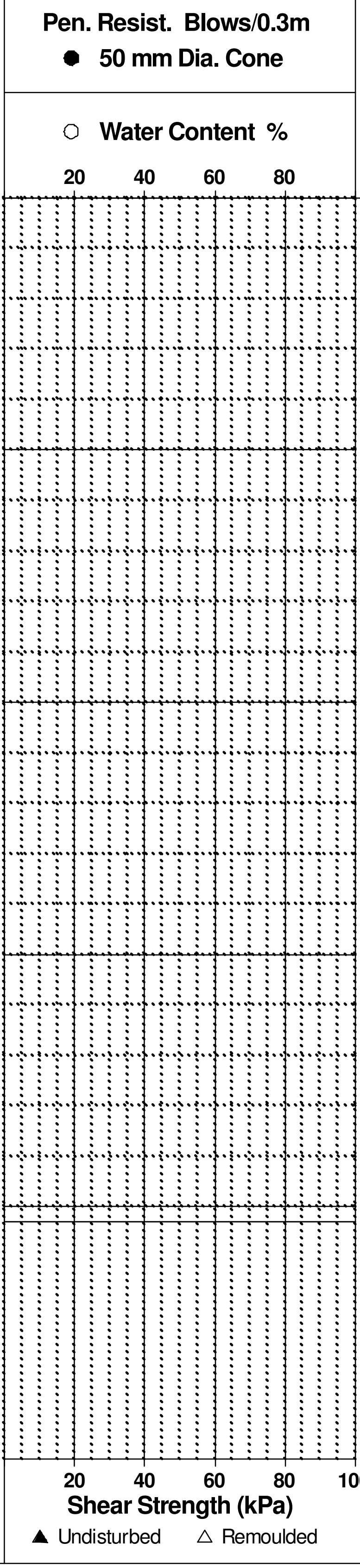
BORINGS BY Backhoe

DATE 19 Oct 06

FILE NO. PG0912

HOLE NO. TP 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0		
FILL: Topsoil, clay, gravel, asphalt, filter cloth and piece of wire	▩	G	1					∇
						1		
1.80								
FILL: Brown silty sand-gravel with clay	▩	G	2					
2.00						2		
Very stiff, brown **SILTY CLAY**	▨	G	3					
						3		
	▨	G	4					
						4		
4.06								
End of Test Pit Open hole GWL @ 1.8m depth								



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)

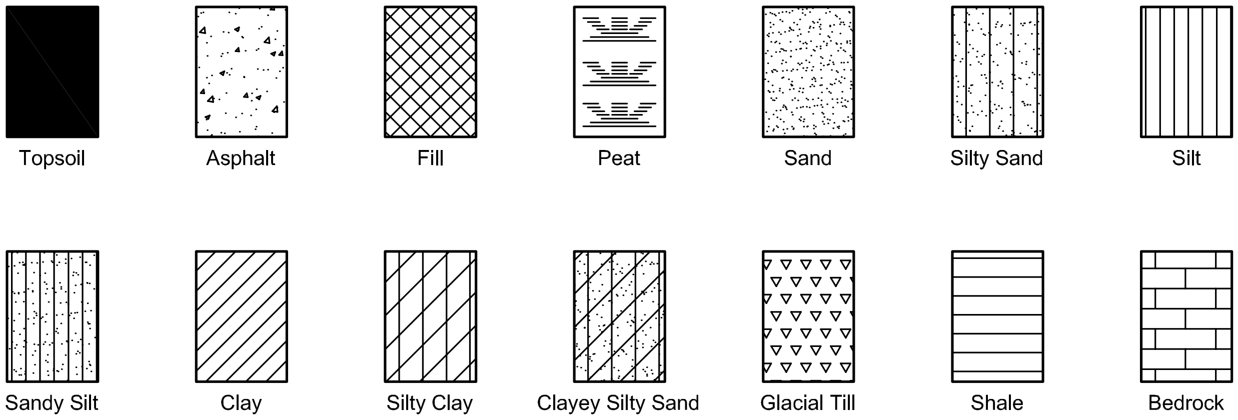
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

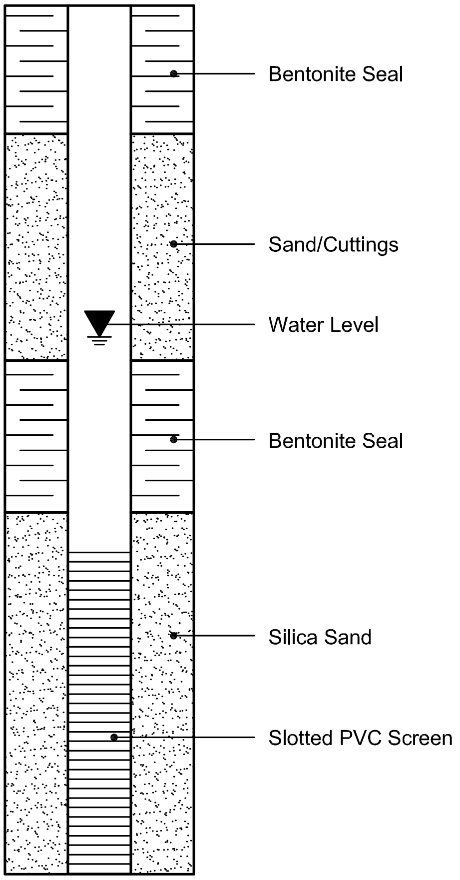
SYMBOLS AND TERMS (continued)

STRATA PLOT

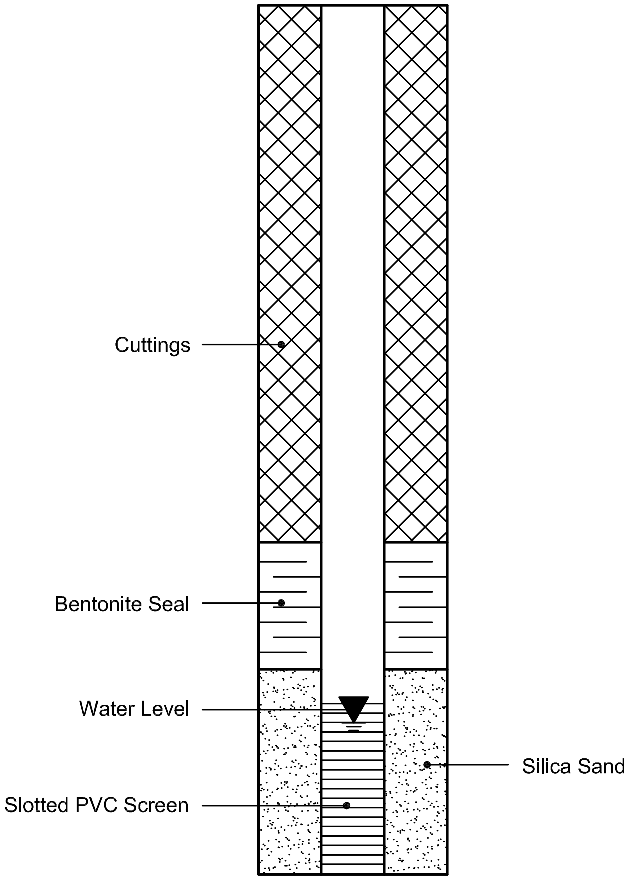


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 24703

Report Date: 20-Mar-2020

Order Date: 17-Mar-2020

Project Description: PG5287

	Client ID:	BH14-SS3	-	-	-
	Sample Date:	12-Mar-20 09:00	-	-	-
	Sample ID:	2012222-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	72.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.59	-	-	-
Resistivity	0.10 Ohm.m	5.54	-	-	-
Anions					
Chloride	5 ug/g dry	1100	-	-	-
Sulphate	5 ug/g dry	88	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - AERIAL PHOTOGRAPH - 1991

FIGURE 3 - AERIAL PHTOGRAPH - 2017

DRAWING PG5287-1 - TEST HOLE LOCATION PLAN

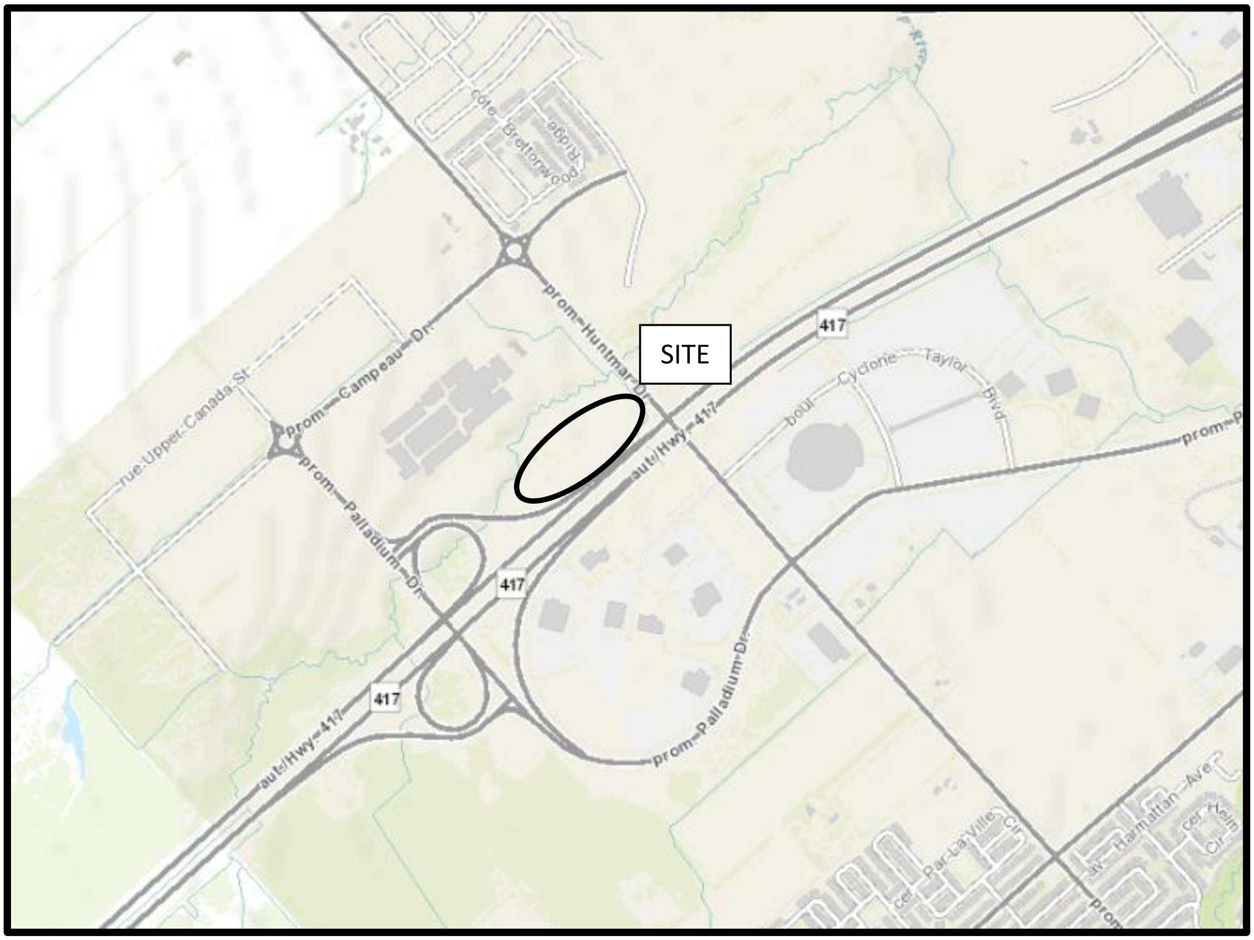


FIGURE 1

KEY PLAN

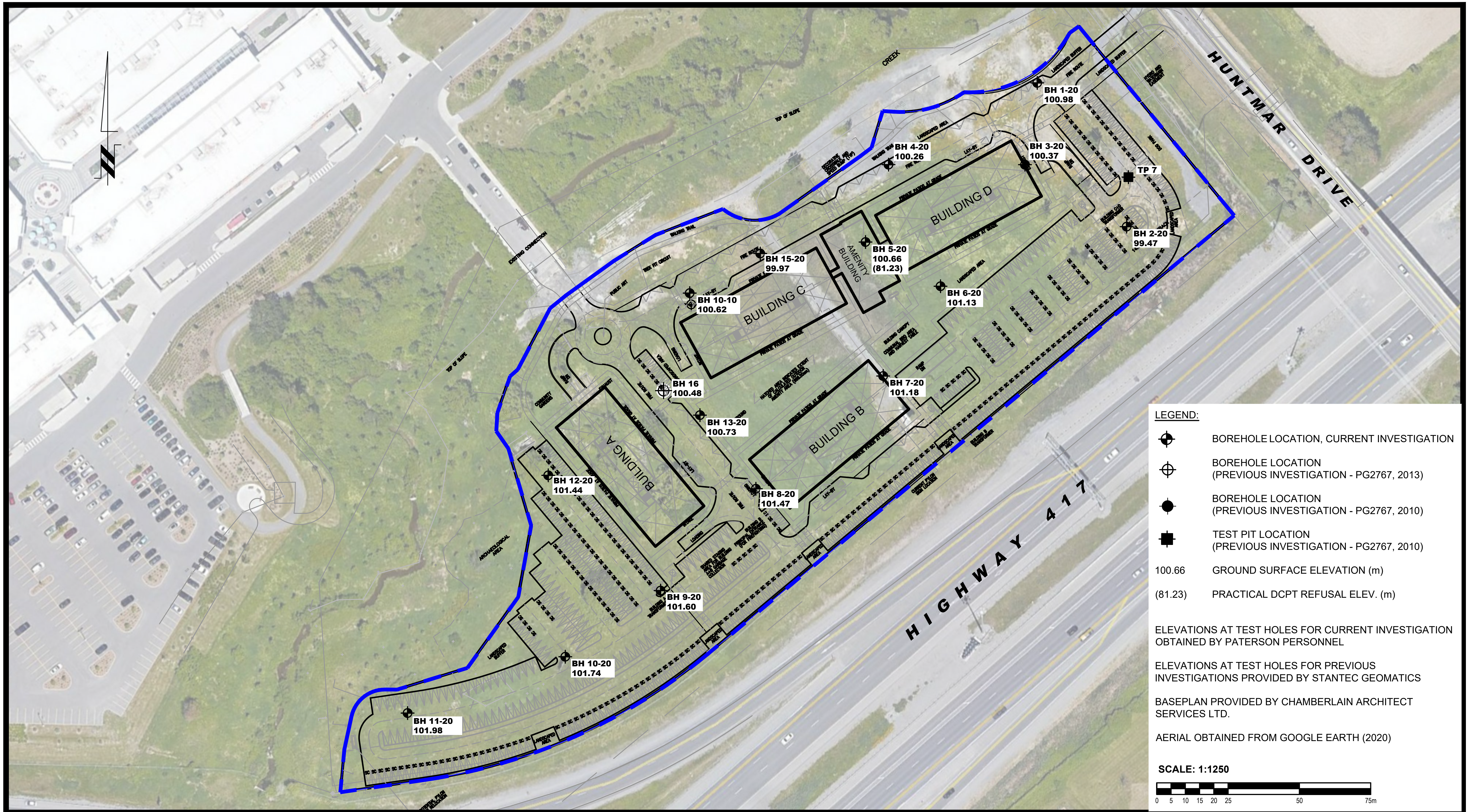
patersongroup

FIGURE 2

Aerial Photograph - 1991

FIGURE 3

Aerial Photograph - 2017



LEGEND:

⌖	BOREHOLE LOCATION, CURRENT INVESTIGATION
⌖	BOREHOLE LOCATION (PREVIOUS INVESTIGATION - PG2767, 2013)
●	BOREHOLE LOCATION (PREVIOUS INVESTIGATION - PG2767, 2010)
■	TEST PIT LOCATION (PREVIOUS INVESTIGATION - PG2767, 2010)
100.66	GROUND SURFACE ELEVATION (m)
(81.23)	PRACTICAL DCPT REFUSAL ELEV. (m)

ELEVATIONS AT TEST HOLES FOR CURRENT INVESTIGATION OBTAINED BY PATERSON PERSONNEL

ELEVATIONS AT TEST HOLES FOR PREVIOUS INVESTIGATIONS PROVIDED BY STANTEC GEOMATICS

BASEPLAN PROVIDED BY CHAMBERLAIN ARCHITECT SERVICES LTD.

AERIAL OBTAINED FROM GOOGLE EARTH (2020)

SCALE: 1:1250

0 5 10 15 20 25 50 75m

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consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
2	UPDATED BASED ON LATEST CONCEPTUAL PLAN	04/05/2021	KP
1	UPDATED BASED ON LATEST CONCEPTUAL PLAN	08/01/2020	KP

REID'S HERITAGE HOME

GEOTECHNICAL INVESTIGATION - PROPOSED COMMERCIAL DEVELOPMENT
319 HUNTMAR DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1250
Date: 03/2020
Drawn by: RCG
Report No.: PG5287-1
Checked by: KP
Dwg. No.: PG5287-1
Approved by: DJG
Revision No.: 2

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