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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation

Proposed Commercial Development
910 March Road
Ottawa, Ontario

Prepared For

Wexcom Developments (March Rd.) Ltd.

November 13, 2019

Report: PG5119-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Wexcom Developments (March Rd.) Ltd. to conduct a geotechnical investigation for the proposed commercial development located at 910 March Road, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.

☐ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its 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.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. A Phase I - Environmental Site Assessment (ESA) was conducted by Paterson for the subject site. The results and recommendations of the Phase I - ESA are presented under separate cover.

2.0 Proposed Development

Based on the preliminary conceptual drawings, it is our understanding that proposed commercial development consists of several commercial slab-on-grade structures including a six (6) storey hotel. Asphaltic covered car parking, access lanes and landscaping areas will occupy the remainder of the subject site.

It is expected that the subject site will be serviced by municipal water and sewer.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the geotechnical investigation was carried out on October 15, 2019. At that time, a total of nine (9) boreholes were placed across the subject site to provide general coverage taking into consideration site features and underground utilities. The locations of the test holes are shown on Drawing PG5119-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

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

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

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

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

Monitoring wells were installed in BH 5, BH 6 and BH 7 while flexible polyethylene standpipes were installed in all other boreholes to permit the monitoring of groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ 1.5 to 3 m long slotted 51 mm diameter PVC screen sealed at strategic depths.
- ☐ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ☐ No.3 silica sand backfill within annular space around screen.
- ☐ Bentonite hole plug directly above PVC slotted screen to approximately 300 mm from the ground surface.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well

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 location and ground surface elevation at each test hole location was surveyed by Paterson personnel. The ground surface elevations at the test hole locations were reference to a temporary benchmark (TBM), consisting of the catch basin manhole lid located near the southwest corner of the subject site. A geodetic elevation of 78.58 m was provided to the TBM on the plan prepared by Stantec Geomatics Ltd.. The location of the TBM, test holes and the ground surface elevation at the each test hole locations are presented on Drawing PG5119-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One (1) representative 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 samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is located on the east side of March Road, north of the intersection of Maxwell Bridge Road and March Road in the City of Ottawa, Ontario. The site is bordered to the north by vacant agricultural land, to the east by Shirley's Brook followed by newly constructed single family residential dwellings and to the south by a tributary (ditch) to Shirley's Brook followed by a commercial property. The north and north-east boundaries of the site are occupied by a shallow tributary to Shirley's Brook which bisects the northeast corner of the site before traveling south along the east property boundary.

The site is generally flat with a slight downward slope toward the north, east and south property boundaries toward Shirley's Brook and its tributaries. The west portion of the site was observed to be approximately at grade with March Road.

The site is currently occupied by a single family residential dwelling constructed with a stone and mortar foundation with a basement level and an attached garage. Several outbuildings and sheds of either slab-on-grade or wood pier construction occupy the central portion of the site. Several storage containers, trailers, sheet metal and farm equipment occupy the remainder of the site.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consists of a 130 to 360 mm thickness of topsoil underlain by a hard to stiff brown silty clay. A glacial till deposit was generally observed underlying the silty clay at approximate depths of 2.3 to 2.7 m below the existing ground surface. Practical auger refusal was encountered at all test hole locations at depths varying between 1.9 and 4.7 m below existing ground surface on inferred bedrock. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for details of the soil profiles encountered at the test hole locations.

Based on available geological mapping, the bedrock in this area consists of interbedded quartz sandstone and sandy dolostone of the March Formation with an overburden drift thickness of 2 to 10 m depth.

4.3 Groundwater

The groundwater levels recorded on September 18, 2019 within the monitoring wells and piezometers installed within the boreholes are are presented in Table 1 below and further noted on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 - Measured Groundwater Levels

Test Hole Location	Ground Surface Elevation (m)	Groundwater Level		Date
		Depth (m)	Elevation (m)	
BH 1	78.38	Blocked at 0.83	< 77.55	September 18, 2019
BH 2	77.63	3.25	74.38	September 18, 2019
BH 3	77.04	2.98	74.06	September 18, 2019
BH 4	76.82	Blocked at 1.80	< 75.02	September 18, 2019
* BH 5	77.27	1.57	75.70	September 18, 2019
* BH 6	77.83	2.22	75.61	September 18, 2019
* BH 7	78.80	2.05	76.75	September 18, 2019
BH 8	77.99	Block at 0.78	< 77.21	September 18, 2019
BH 9	77.55	Destroyed	77.55	September 18, 2019

Note: - The ground surface elevations at the test hole locations were reference to a temporary benchmark (TBM), consisting of the catch basin manhole lid located near the southwest corner of the subject site. A geodetic elevation of 78.58 m was provided for the TBM on the plan prepared by Stantec Geomatics Ltd..
* Denotes boreholes instrumented with monitoring wells.

It is important to note that groundwater readings at piezometers can be influenced by surface water perched within the borehole backfill material. Long-term groundwater conditions can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, it is estimated that long-term groundwater level can be expected between **2** to **3 m** depth. However, groundwater levels are subject to seasonal fluctuations and therefore could vary during time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed commercial development. It is expected that the proposed buildings will be constructed with conventional shallow foundations placed on undisturbed, hard to stiff silty clay, glacial till and/or clean, surface sounded bedrock.

Due to the presence of a silty clay deposit underlying the subject site, a permissible grade raise restriction will be required where footings are founded over a silty clay bearing surface.

The slopes bordering Shirley’s Brook and its tributaries located along the north, south and east boundaries of the site are considered stable from a geotechncial perspective. This is discussed further in Section 6.9.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Fill Placement

Fill used for grading beneath the proposed structures should consist, unless otherwise specified, 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 areas should be compacted to at least 98% of its 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 their respective 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.

Bedrock Removal

If bedrock removal is required, consideration should be given to line-drilling in conjunction with hoe-ramming or controlled blasting. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming only.

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

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

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

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system (if required).

Vibration Considerations

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

The following construction equipments could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of these equipments. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, hard to stiff silty clay bearing surface can be designed using the 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**.

Strip and pad footings founded on an undisturbed, compact to dense glacial till and/or engineered fill bearing surface can be designed using the bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

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

Footings designed using the bearing resistance value at SLS provided herein will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Footings placed on clean, surface sounded bedrock can be designed using a factored bearing resistance value at ULS of **1,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance at ULS.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the subexcavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

Permissible Grade Raise Recommendations

Where silty clay is present below the underside of footing, a permissible grade raise restriction of **2.5 m** is recommended.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a stiff silty clay above the groundwater table 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 of the same or higher capacity as the bearing medium soil.

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

5.4 Design for Earthquakes

Based on the subsoil profile encountered across the subject site, foundation design at the subject site can be designed using a seismic site classification **Class C** according to Table 4.1.8.4.A of the Ontario Building Code 2012. A higher site class, such as Class A or B, may be available for foundations placed on or near the bedrock surface. However, the higher site class would have to be confirmed by site specific seismic shear wave velocity testing. The soils underlying the site are not susceptible to liquefaction.

5.5 Slab on Grade Construction

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed buildings, the native soil surface, free of deleterious and organic materials, will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

The upper 200 mm of sub-slab fill should consist of an OPSS Granular A material for slab-on-grade construction. All backfill material within the proposed building footprint should be placed in maximum 300 mm lifts and compacted to a minimum of 98% of the SPMDD.

Any soft subgrade areas should be removed and backfilled with appropriate backfill material, such as OPSS Granular B Type II, with a maximum particle size of 50 mm.

5.6 Pavement Structure

For design purposes, the pavement structures presented in the following tables are recommended for the design of car parking areas and access lanes.

Table 2 - 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
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	

Table 3 - Recommended Pavement Structure - Truck Parking and Access 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
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	

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 I or II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material’s SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

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

Due to the impervious nature 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.

Hard Surfaces adjacent to Structures

Since the underlying soils at the subject site are considered frost susceptible, it is recommended that the following precautionary measures be taken to reduce the severity of the differential frost heaving of the overlying hard surfaces, such as concrete sidewalks and pavement structures, adjacent to the proposed structures

In areas where concrete sidewalk, pavement structures or other hard surfaces abut the proposed structures, differential frost heaving could occur between the granular fill material immediately adjacent to these structures and the more frost susceptible material beyond the backfill material.

To reduce the severity of this differential heaving, it is recommended that the foundation be designed with a foundation drainage system, detailed in Section 6.1, and the foundation backfill adjacent to the foundation should be placed to form a frost taper. The frost taper should be brought up to the pavement subgrade level from 1.5 m below the finished exterior grade level at 5H:1V, or flatter, away from the foundation wall.

Alternatively, consideration can be taken to increasing the thickness of the structure below the hard surface to a minimum thickness of 600 mm in conjunction with frost protection. For preliminary design purposes, the frost protection should consist of a minimum of 100 mm of SM rigid insulation in pedestrian traffic areas, HI-40 for light duty car traffic areas and HI-60 for heavy truck traffic areas. It is further recommended that the frost protection be extended to a minimum of 2.4 m from the vertical face of the exterior foundation wall and match the existing pavement structure with a 5H:1V frost taper from the transition of the frost protection.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. 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.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system is provided.

6.2 Protection Against Frost Action

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

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

6.3 Excavation Side Slopes

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

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

6.4 Pipe Bedding and Backfill

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. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 95% of the material’s SPMDD.

It should generally be possible to re-use the organic free, moist (not wet) overburden material above the cover material. Wet materials will be difficult to re-use as their high water contents make compacting these materials impractical without an extensive drying period. All stones greater than 300 mm in their longest dimension and other deleterious materials should be removed prior to reusing these materials.

Well fractured bedrock should be acceptable as backfill provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones 300 mm or larger in their longest dimension are removed. Where blast rock is used, a blinding layer (OPSS Granular A crushed stone) or a geotextile may be required above the blast rock to reduce the loss of fine particles within the voids of the rockfill.

Based on the soil profile encountered, the subgrade for the services will be placed in both bedrock and in overburden soils. It is recommended that the subgrade medium be inspected in the field to determine how steeply the bedrock surface, where encountered, drops off. A transition treatment should be provided where the bedrock slopes at more than 3H:1V. At these locations, the bedrock should be excavated and extra bedding be placed to provide a 3H:1V (or flatter) transition from the bedrock subgrade towards the soil subgrade. This treatment reduces the propensity for bending stress to occur in the service pipes.

Trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

To reduce long-term lowering of the groundwater level at this site, where services are completed within the silty clay deposit, 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

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

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 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.

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.

6.6 Winter Construction

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

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

The trench excavations should be carried out in a manner that will avoid the introduction of frozen materials into the trenches. As well, 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. In addition, 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. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Restrictions

Due to the absence of atterberg limit test results for the undisturbed silty clay deposit, it is difficult to determine the sensitivity of the silty clay within the subject site. Therefore, conservatively, the underlying clay will be treated as a high sensitivity clay soil for tree planting restrictions.

A very stiff to stiff silty clay deposit was encountered between anticipated underside of footing elevations and 3.5 m below anticipated finished grade as per City Guidelines. Therefore, the following tree planting setbacks are recommended for the assumed high sensitivity area.

Large trees (mature height over 14 m) can be planted within this area provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits are 7.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the following conditions are met:

☐ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.

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

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

☐ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

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

6.9 Slope Stability Recommendations

Based on our site reconnaissance on October 15, 2019 and review of the available grading plan prepared by Stantec Geomatics Ltd., Shirley's Brook and its associated tributaries bordering the north, south and east boundaries of the site generally extend a maximum depth of 2 m below the table lands of the subject site.

The 0.5 m deep tributary to Shirley's Brook bordering the north property boundary was observed to slope at 5H:1V or flatter. The tributary (ditch) to Shirley's Brook located along the south property boundary is confined within a narrow channel extending to a maximum depth of 2.5 m below the table lands. The majority of the slopes were generally observed to be at 3H:1V or flatter with the exception of an isolated area where suspected fill was placed in close proximity to the ditch. Some minor erosion was observed along the bank face where the suspected fill was historically placed in close proximity to the confined ditch. The rear portion of the site slopes gradually down to Shirley's Brook at slopes of 5H:1V or flatter to an approximately 1.5 to 3 m wide watercourse. Minor signs of toe erosion were observed along the shallow banks of the existing watercourse. All slopes observed along Shirley's Brook and its tributaries were heavily vegetated with grass and sparsely occupied by trees along the south ditch.

Following our site reconnaissance and review, the slopes bordering Shirley's Brook and its tributaries located along the north, south and east boundaries of the site are considered stable from a geotechnical perspective.

7.0 Recommendations

It is recommended that the following be carried out once the master plan and site development are determined:

☐ Review detailed grading plan(s) from a geotechnical perspective.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to placing backfilling materials.

☐ Field density tests to ensure that the specified level of compaction has been 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 material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request permission to review the grading plan once available. Also, our recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and 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, we request that we be notified immediately in order to permit reassessment of our 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 Wexcom Developments (March Rd.) Ltd. 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.

Richard Groniger, C. Tech.

LICENSED PROFESSIONAL ENGINEER
Nov. 13, 2019
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Scott Dennis

Scott S. Dennis, P.Eng.

Report Distribution:

☐ Wexcom Developments (March Rd.) Ltd. (3 copies)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 910 March Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northwest corner of subject site. Geodetic elevation = 78.58m, as per survey plan prepared by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 15

FILE NO. PG5119

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	78.38
TOPSOIL with organics and silty clay, some sand 0.20	AU	1				
Hard to very stiff, brown **SILTY CLAY**	SS	2	100	14	1	77.38
1.93	SS	3	100	50+		
End of Borehole Practical refusal to augering at 1.93m depth (Piezometer blocked at 0.83m depth - Oct. 18, 2019)						

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

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

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 910 March Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northwest corner of subject site. Geodetic elevation = 78.58m, as per survey plan prepared by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 15

FILE NO. PG5119

HOLE NO. BH 2

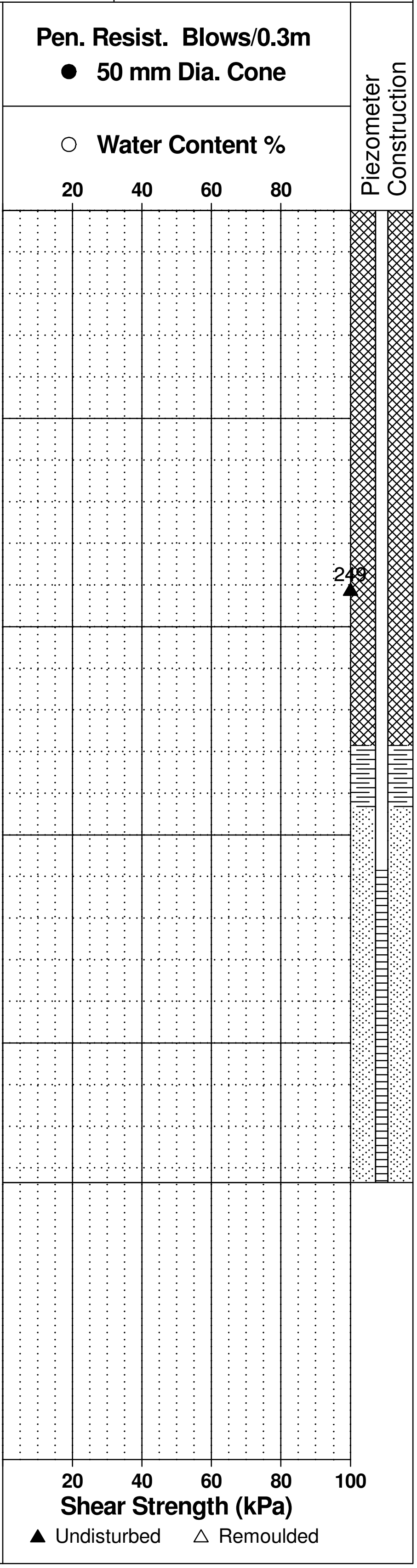
SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL with organics and silty clay, some sand	0.13	AU	1		
Hard to very stiff, brown **SILTY CLAY**	2.62	SS SS	2 3	100 100	9 9
Very stiff, brown **SILTY CLAY** with sand, gravel, cobbles, occasional boulders	3.50	SS SS	4 5	100 71	5 16
Stiff, grey **SILTY CLAY,** trace sand - increasing sand and gravel with depth	4.67	SS SS	6 7	54 75	6 50+

End of Borehole

Practical refusal to augering at 4.67m depth

(Piezometer blocked at 3.25m depth - Oct. 18, 2019)

DEPTH (m)	ELEV. (m)
0	77.63
1	76.63
2	75.63
3	74.63
4	73.63



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 910 March Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northwest corner of subject site. Geodetic elevation = 78.58m, as per survey plan prepared by Stantec Geomatics Ltd.
REMARKS

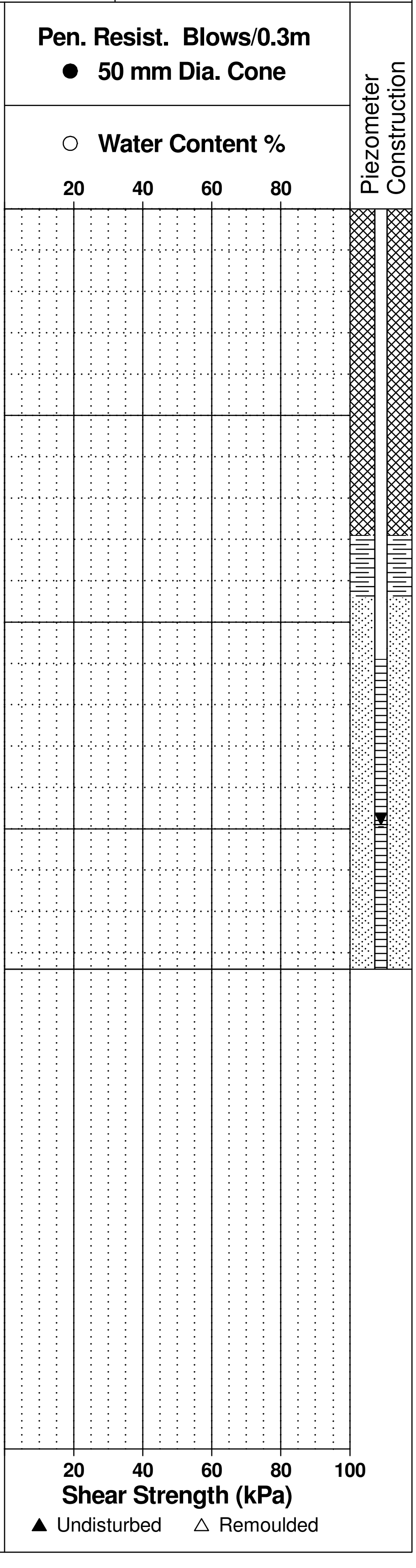
BORINGS BY CME 55 Power Auger

DATE 2019 October 15

FILE NO. PG5119

HOLE NO. BH 3

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	77.04
TOPSOIL with organics, silty sand and gravel 0.28	AU	1				
	SS	2	88	12	1	76.04
Hard to stiff, brown **SILTY CLAY**	SS	3	92	8		
					2	75.04
	SS	4	96	4		
- trace sand and gravel by 2.7m depth					3	74.04
- increasing gravel with depth, occasional cobbles and boulders 3.68	SS	5	100	6		
End of Borehole Practical refusal to augering at 3.68m depth (GWL @ 2.98m - Oct. 18, 2019)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 910 March Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northwest corner of subject site. Geodetic elevation = 78.58m, as per survey plan prepared by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 15

FILE NO. PG5119

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.82
TOPSOIL with organics, some sand and gravel 0.36		AU	1				
Hard to very stiff, brown **SILTY CLAY**		SS	2	100	17	1	75.82
		SS	3	67	9	2	74.82
2.29							
GLACIAL TILL: Very stiff, brown clayey silt to silty clay with sand and gravel, occasional cobbles and boulders		SS	4	62	11	3	73.82
		SS	5	64	50+		
3.81							
End of Borehole Practical refusal to augering at 3.81m depth (Piezometer blocked at 1.80m depth - Oct. 18, 2019)							

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

○ **Water Content %**
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 910 March Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northwest corner of subject site. Geodetic elevation = 78.58m, as per survey plan prepared by Stantec Geomatics Ltd.

REMARKS

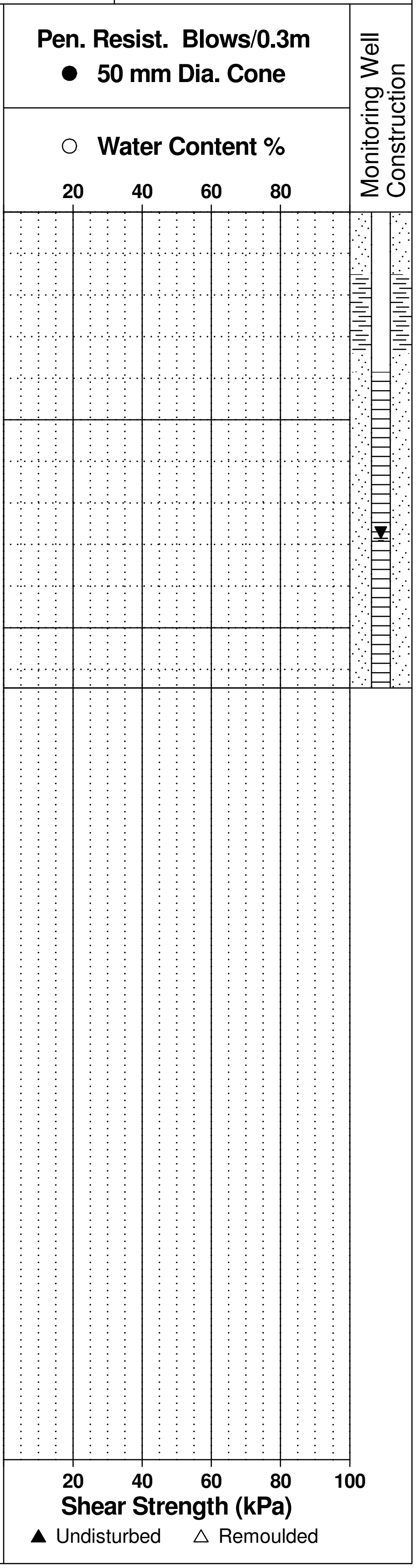
BORINGS BY CME 55 Power Auger

DATE 2019 October 15

FILE NO. PG5119

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.27
TOPSOIL with organics, some silty clay 0.30		AU	1				
Hard to stiff, brown **SILTY CLAY** - some sand, trace gravel by 0.9m depth - increasing sand and gravel with depth		SS	2	83	6	1	76.27
		SS	3	75	9	2	75.27
2.29							
End of Borehole Practical refusal to augering at 2.29m depth (GWL @ 1.57m - Oct. 18, 2019)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 910 March Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northwest corner of subject site. Geodetic elevation = 78.58m, as per survey plan prepared by Stantec Geomatics Ltd.

REMARKS

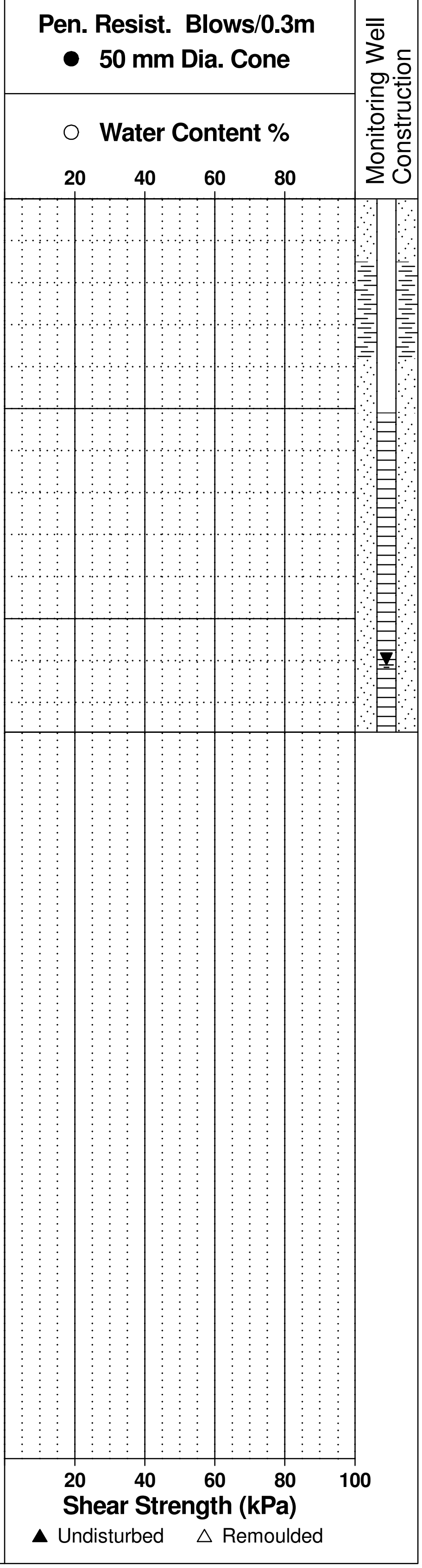
BORINGS BY CME 55 Power Auger

DATE 2019 October 15

FILE NO. PG5119

HOLE NO. BH 6

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	77.83
TOPSOIL 0.18						
FILL: Crushed stone with sand, topsoil and organics 0.46	AU	1				
Hard to very stiff, brown **SILTY CLAY**	SS	2	79	18	1	76.83
	SS	3	100	15	2	75.83
2.54	SS	4	78	50+		
End of Borehole Practical refusal to augering at 2.54m depth (GWL @ 2.22 - Oct. 18, 2019)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 910 March Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northwest corner of subject site. Geodetic elevation = 78.58m, as per survey plan prepared by Stantec Geomatics Ltd.

REMARKS

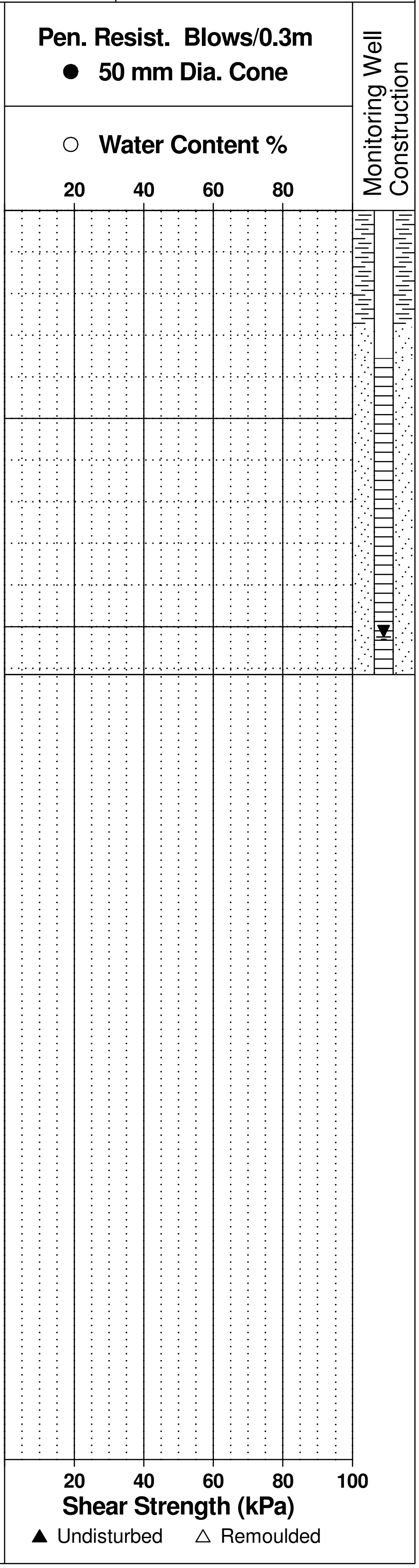
BORINGS BY CME 55 Power Auger

DATE 2019 October 15

FILE NO. PG5119

HOLE NO. BH 7

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	78.80
TOPSOIL with organics, some sand and gravel 0.15	AU	1				
Hard to very stiff, brown **SILTY CLAY**	SS	2	54	16	1	77.80
- trace sand and gravel by 1.7m depth 2.23	SS	3		12	2	76.80
End of Borehole Practical refusal to augering at 2.23m depth (GWL @ 2.05m - Oct. 18, 2019)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 910 March Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northwest corner of subject site. Geodetic elevation = 78.58m, as per survey plan prepared by Stantec

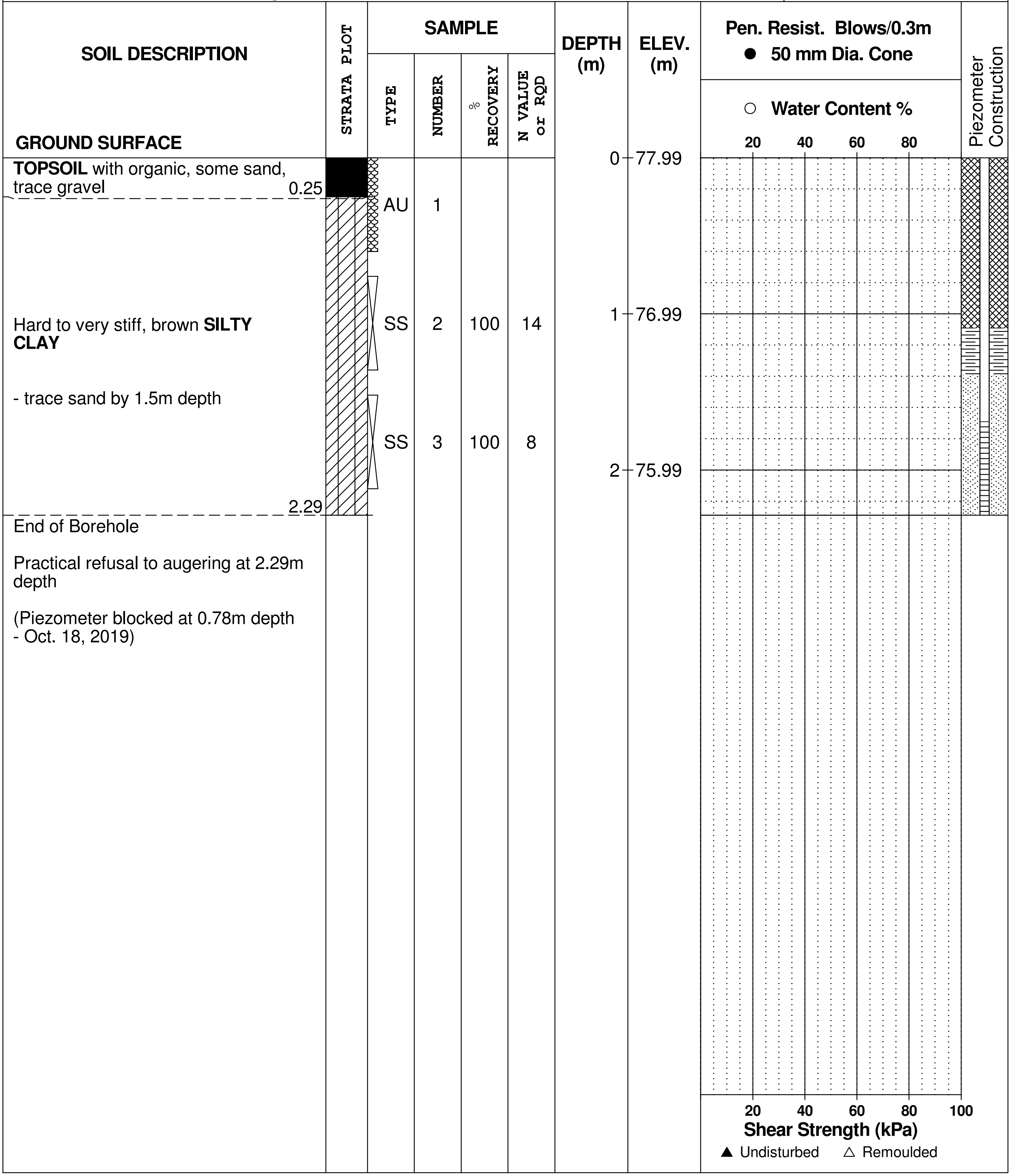
REMARKS Geomatics Ltd.

BORINGS BY CME 55 Power Auger

DATE 2019 October 15

FILE NO. PG5119

HOLE NO. BH 8



SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	77.99
TOPSOIL with organic, some sand, trace gravel 0.25	AU	1				
Hard to very stiff, brown **SILTY CLAY**	SS	2	100	14	1	76.99
- trace sand by 1.5m depth	SS	3	100	8	2	75.99
2.29						
End of Borehole Practical refusal to augering at 2.29m depth (Piezometer blocked at 0.78m depth - Oct. 18, 2019)						

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - 910 March Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located near the northwest corner of subject site. Geodetic elevation = 78.58m, as per survey plan prepared by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 October 15

FILE NO. PG5119

HOLE NO. BH 9

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE					0	77.55	
FILL: Crushed stone — 0.46	AU	1					
FILL: Brown sand with gravel — 0.84							
Hard to very stiff, brown **SILTY CLAY**	SS	2	83	9	1	76.55	
	SS	3	100	14	2	75.55	
— 2.69	SS	4	100	11			▽
GLACIAL TILL: Hard to very stiff, brown silty clay with sand, gravel, occasional cobbles and boulders — 3.81	SS	5	42	29	3	74.55	

End of Borehole

Practical refusal to augering at 3.81m depth

(Piezometer observed to be destroyed on Oct. 18, 2019)

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

○ Water Content %
20 40 60 80

20 40 60 80 100
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 = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

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

CONSOLIDATION TEST

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

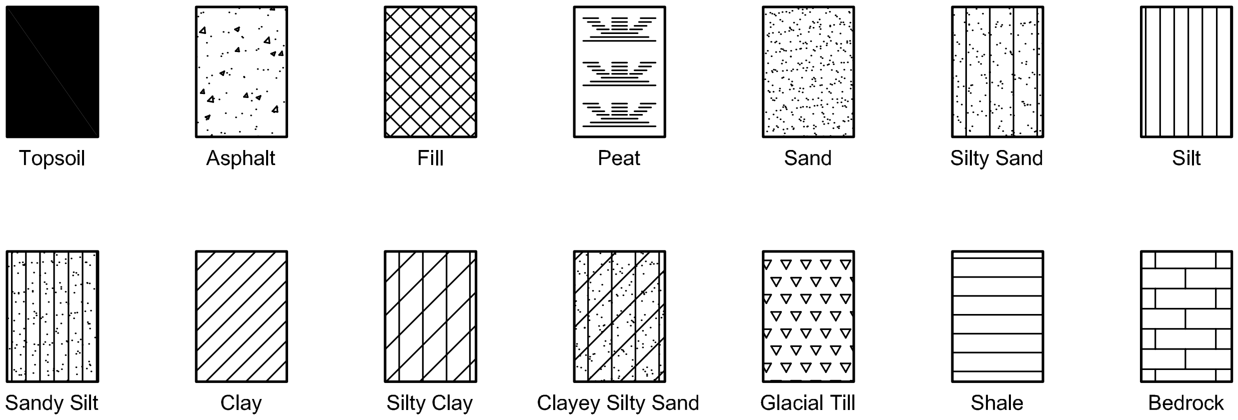
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

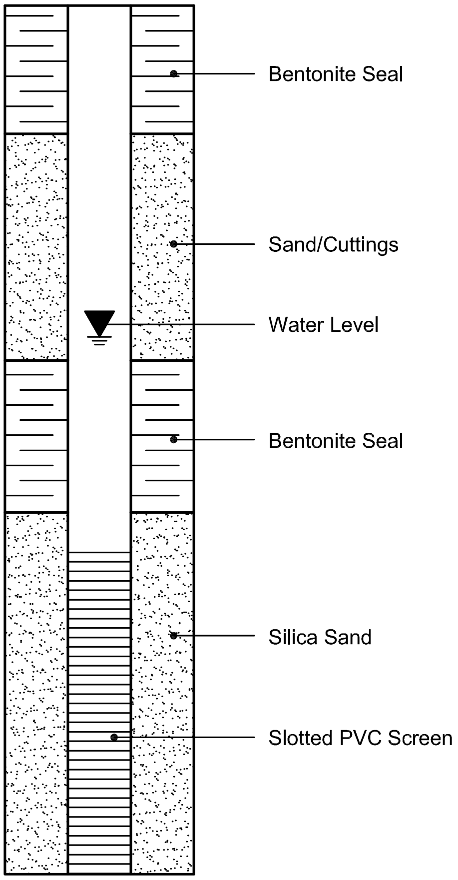
SYMBOLS AND TERMS (continued)

STRATA PLOT

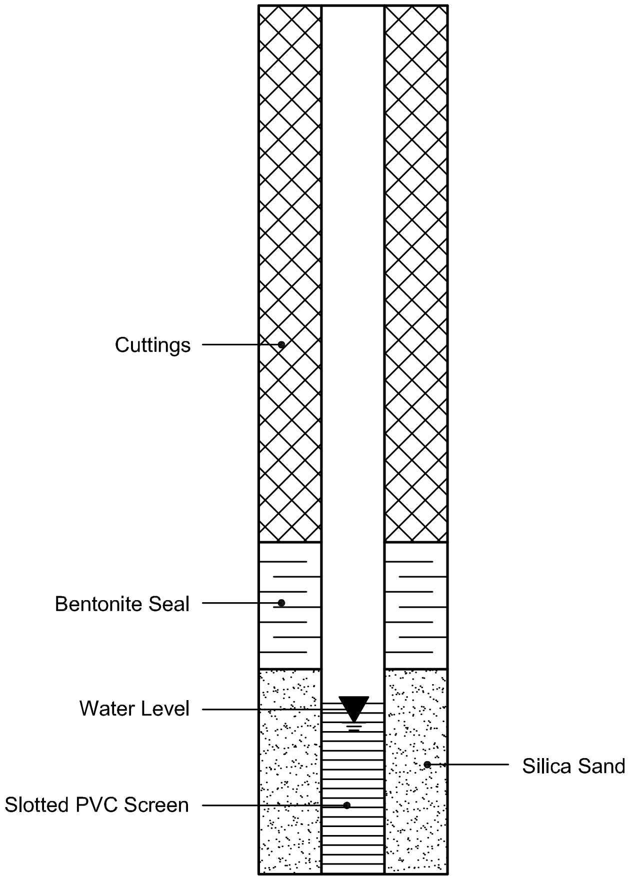


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 28423

Report Date: 23-Oct-2019

Order Date: 17-Oct-2019

Project Description: PG5119

Client ID:		BH6-SS3	-	-	-
Sample Date:		15-Oct-19 09:00	-	-	-
Sample ID:		1942383-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	76.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.02	-	-	-
Resistivity	0.10 Ohm.m	39.8	-	-	-
Anions					
Chloride	5 ug/g dry	43	-	-	-
Sulphate	5 ug/g dry	60	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5119-1 - TEST HOLE LOCATION PLAN

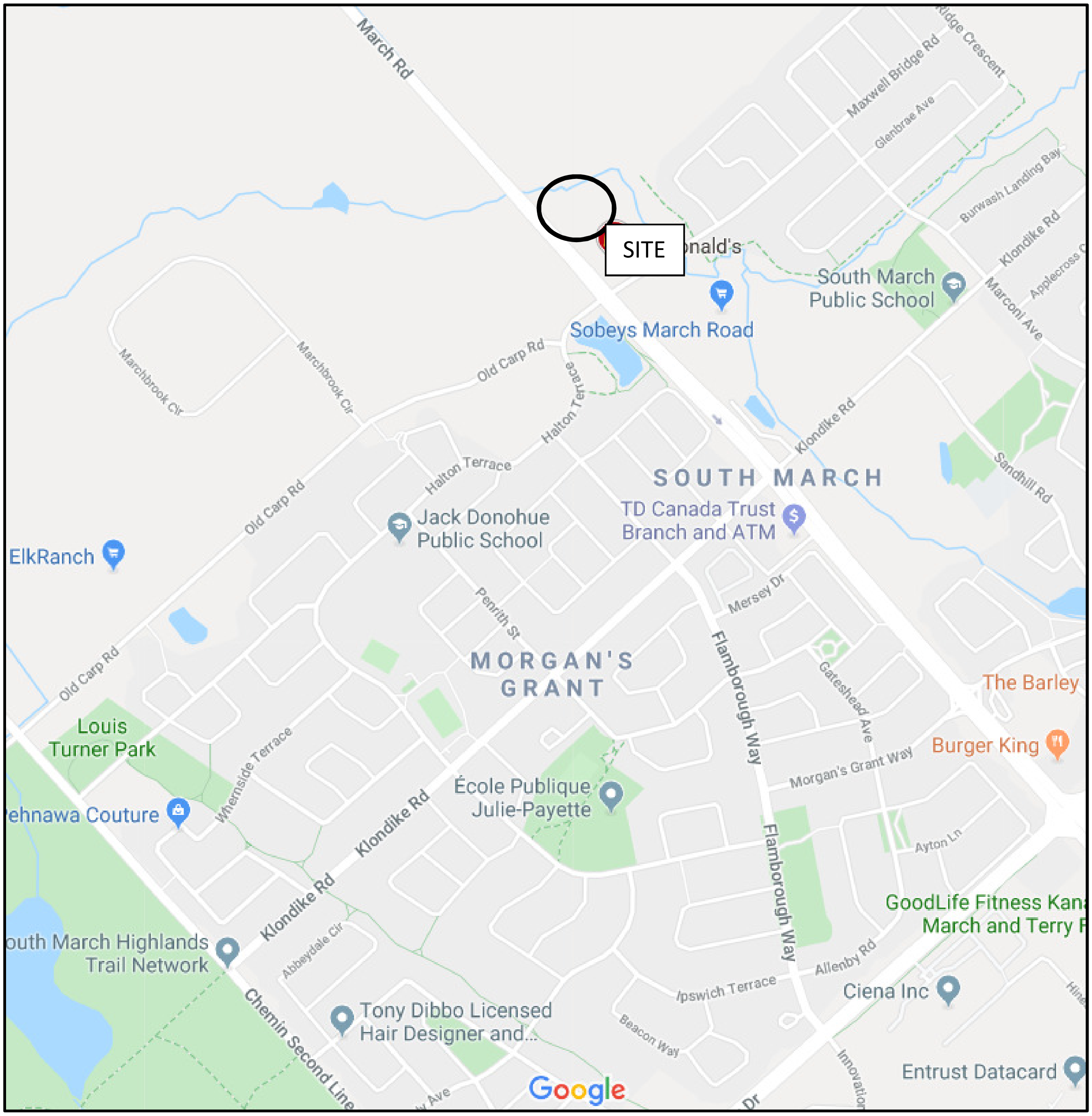


FIGURE 1

KEY PLAN

LEGEND:

●	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
77.63	GROUND SURFACE ELEVATION (m)
(72.96)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

BASE PLAN PROVIDED BY STANTEC GEOMATICS LTD.
CONCEPTUAL PLAN PROVIDED BY GREYSTONE GROUP

TBM- TOP OF GRATE OF CATCH BASIN LOCATED NEAR THE NORTHWEST CORNER OF SUBJECT SITE. GEODETIC ELEVATION= 78.58m AS PER SURVEY PLAN PREPARED BY STANTEC GEOMATICS LTD.

SCALE: 1:1000

0 10 20 30 40 50m 75m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

WEXFORD DEVELOPMENTS (MARCH ROAD) LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT- 910 MARCH ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000
Date: 10/2019
Drawn by: RCG
Report No.: PG5119-1
Checked by: RG
Approved by: DJG

PG5119-1

Revision No.:

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