PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

s

927 March Road - Ottawa, Ontario

Prepared for Brigil

Report PG5330 -1- Revision 1 dated December 14, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Brigil to conduct a geotechnical investigation for the proposed residential development to be located at 927 March Road in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was to:

☐ determine the subsurface soil and groundwater conditions by means of boreholes and monitoring well program.

☐ provide preliminary geotechnical recommendations for the foundation design of the proposed buildings and provide geotechnical construction precautions which may affect the design.

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

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

2.0 Proposed Development

Based on the latest conceptual site plan, it is understood that the proposed development will consist of multi-storey buildings having two to four underground parking levels, and townhouses. Local roadways and residential driveways are also anticipated for the proposed development. Park areas are currently proposed on the northern, central west, and southwest portions of the site, and a school area is proposed to be constructed on the south portion of the site. Furthermore, a storm water pond is proposed to be located on the central north portion of the site, north of the existing watercourse intercepting the site from southeast to northwest. It is further anticipated that the site will be serviced by future municipal services.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was carried out from April 23 to 27, 2020. At that time, 11 boreholes were completed to a maximum depth of 7.1 m below existing ground surface. A previous investigation was carried out by Paterson on February 25, 2008. At that time 5 test pits were excavated using a rubber tire backhoe to a maximum depth of 3.1 m. The test hole locations were placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations for the current investigation are presented on Drawing PG5330-1 - 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. 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 and using 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 for review. 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 Appendix1.

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

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

Diamond drilling was carried out at 7 borehole locations to assess the bedrock quality. The depth at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the quality of the bedrock.

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

Two boreholes (BH 2 and BH 3) where equipped with groundwater monitoring wells while flexible piezometers were installed in all the other 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 test hole locations were located and surveyed by Paterson personnel in the field. The elevations are provided in reference to a geodetic datum. The ground surface elevations and locations of the boreholes are presented on Drawing PG5330-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from our field investigation were examined in our laboratory to collaborate the field findings. Grain size distribution analysis was carried out on 5 selected soil samples.

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 6.6.

4.0 Observations

4.1 Surface Conditions

The subject site consists of an undeveloped, agricultural land, mainly covered by harvest crop and grassed areas. A 1 to 2 m deep ditch was noted crossing the site from west to east with a 40 m wide pond area. The site is approximately "U" shaped around residential and agricultural properties. The ground surface at the subject site is relatively flat with a slight slope down from west to east towards March Road. The ground surface is slightly below grade of March Road.

The subject site is bordered to the east by March Road, to the south by undeveloped land, to the west by a residential development and to the north by agricultural lands. A tree line was noted bordering the south property line.

4.2 Subsurface Profile

Overburden

Generally, the soil profile encountered at the test hole locations consists of an agriculturally disturbed clayey organic layer overlying a very stiff to hard brown silty clay crust. Glacial till, composed of gravel, cobbles and boulders within a silty clay soil matrix was encountered underlying the brown silty clay crust at several of the borehole locations.

Practical refusal to augering was encountered at all borehole locations on an inferred bedrock surface. Bedrock was confirmed by coring at BH2, BH3, BH7, BH 8, BH 9 and BH 10.

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 in the area is part of the March formation, which consists of sandstone and dolostone, with an average overburden thickness ranging from 1 to 5 m.

4.3 Groundwater

Groundwater level readings were recorded on May 5, 2020 at the piezometer and monitoring well locations. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1, and in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH1	80.90	Artesian Pressure higher than piezometer		May 1, 2020
BH2	81.49	Artesian Pressure higher than well		May 1, 2020
BH3	82.62	-0.11	82.73	May 1, 2020
BH4	85.90	0.15	85.75	May 1, 2020
BH5	81.40	0.09	81.31	May 1, 2020
BH6	81.98	0.30	81.68	May 1, 2020
BH7	83.72	0.42	83.30	May 1, 2020
BH8	82.84	0.68	82.16	May 1, 2020
BH9	83.44	1.11	82.33	May 1, 2020
BH10	83.78	1.20	82.58	May 1, 2020
BH11	80.03	0.19	79.84	May 1, 2020

Note: Ground surface elevations at test hole locations are referred to a geodetic datum.

Due to the impermeable nature of the overburden material, it should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. It is important to note that based on observations of the soil samples recovered from the borehole locations, such as coloring, moisture levels and consistency, the long-term groundwater level is not expected within the overburden soils.

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

The groundwater level readings within the monitoring wells indicate that an artesian pressure is present below the bedrock surface within the north portion of the site.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed residential development. However, due to the presence of the silty clay layer, a proposed development will be subjected to grade raise restrictions.

Bedrock removal may be required to complete some basement/parking levels and for the SWMP excavation based on the current project details. Moderate to high groundwater infiltration through the excavated bedrock is expected during construction. It is also anticipated that artesian groundwater pressure issues will be encountered during excavation and construction. Therefore, groundwater control measures should be implemented, such as waterproofing or a clay liner above the bedrock surface for the pond construction.

A review of the existing ditch side slopes by Paterson confirms that the slopes are considered to be stable, and no active erosion is occurring along the watercourse. Therefore, no setback from top of slope is required from a geotechnical perspective. It is further expected that the watercourse alignment will be adjusted into an engineered corridor as part of the site re-development work. Additional review can be completed by Paterson once design details are available.

The above and other considerations are 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.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock for the underground parking levels. 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**.**

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 superv ision 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.

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 as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment 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 equipment. Vibrations, whether caused by blasting operations or by construction operations, could be 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.

Fill Placement

The 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 thick 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 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.

Infilling of Existing Ditches

In-filling the existing ditches, where required, should be completed in a stepped fashion within the lateral support of the proposed buildings. The fill should consist of clean imported granular fill such as OPSS Granular A or Granular B Type II. The steps should have a minimum horizontal length of 1.0 m and maximum vertical height of 0.5 m. The material should be placed in maximum 300 mm thick loose lifts and compacted using suitable compaction equipment to a minimum 98% of the material's SPMDD.

The engineered fill should extend to the underside of footing of the proposed residential units, underside of pavement structure or to the spring line of the service pipe. Furthermore, the engineered fill should be placed within the zone of influence of the footing extending 1.5H:1V from the bottom outside edge of the footing. Outside settlement sensitive structures, backfilling of existing ditches can be done using dry, workable brown silty clay, placed in maximum 300 mm thick lifts and compacted using sheepsfoot layer making several passes.

The placement of the silty clay backfill should be completed in dry conditions and above freezing temperatures, reviewed and approved by Paterson at the time of placement.

Decommissioning of Existing Drainage pipes

Where encountered within the footprint of the proposed buildings, the drainage pipes should be cut back a minimum 1 m from edge of the nearest footing. The end of the drainage pipe should be capped, and a clay seal should be placed between the pipe ends and adjacent footing/structure. The clay seal should consist of a minimum 1.0 m wide layer of clay extending from excavation walls to walls and a minimum of 300 mm above the pipe.

The drainage outlet should be cut back a minimum of 15 m from the ditch or drainage creek and capped as indicated above. Any area within the lateral support zone of the structures should be backfill using clean imported granular material as per our geotechnical recommendations.

5.3 Foundation Design

Bearing Resistance Values

Pad footings, up to 6 m wide, and strip footings, up to 3 m wide, placed on an undisturbed, very stiff silty clay bearing surface can be designed using a bearing resistance value at SLS of **200 kPa** and a factored bearing resistance value at ULS of **300 kPa**.

Footings placed on an undisturbed compact glacial till bearing surface can be designed using a bearing resistance of **200 kPa** at SLS and **300 kPa** at ULS.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in-situ or not, have been removed, prior to the placement of concrete for footings.

Footings placed on a clean surface sounded bedrock surface at the proposed founding elevation can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value 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.

A factored bearing resistance value at ULS of **5,000 kPa**, incorporating a geotechnical resistance factor of 0.5, could be used if founded on limestone bedrock and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footprint(s) of the footing(s). At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant.

Lean Concrete Filled Trenches

Where the proposed footings are to be founded on bedrock which is located below the underside of footing elevation, zero-entry vertical trenches should be excavated to the clean, surface sounded bedrock, and backfilled with lean concrete to the founding elevation (minimum **17 MPa** 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

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 engineered fill or native soil above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1H: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).

Settlement

The total and differential settlements will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed.

The potential post construction total and differential settlements are dependent on the position of the long-term groundwater level when buildings are situated over deposits of compressible silty clay. Efforts can be made to reduce the impacts of the proposed development on the long-term groundwater level by placing clay dykes in the service trenches, reducing the sizes of paved areas, leaving green spaces to allow for groundwater recharge or limiting planting of trees to areas away from the buildings. However, it is not economically possible to control the groundwater level.

Permissible Grade Raise Recommendations

Based on the undrained shear strength testing results and experience with the local silty clay deposit, a permissible grade raise recommendation has been designed for the subject site. **It is recommended that a permissible grade raise restriction of 3.0 m be implemented for the subject site.**

5.4 Design for Earthquakes

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

The soils underlying the proposed shallow foundations are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab/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 or approved engineered fill pad will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material. A clear crushed stone fill is recommended for backfilling below the floor slab for limited span slab-on-grade areas, such as front porch or garage footprints. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone below basement floor slabs.

5.6 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of driveways, local residential streets and roadways with bus traffic. It should be noted that for residential driveways and car only parking areas, an Ontario Traffic Category A is applicable. For local roadways and roadways with bus traffic, an Ontario Traffic Category B and Category D should be used for design purposes, respectively.

Table 2 - Recommended Pavement Structure - Driveways

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 soils or OPSS Granular B Type I or II material placed over in situ soil, or fill	

Table 3 - Recommended Pavement Structure - Local Residential Roadways

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 soils or OPSS Granular B Type I or II material placed over in situ soil, or fill	

Table 4 - Recommended Pavement Structure - Roadways with Bus Traffic

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. 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.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for driveways and local roadways and PG 64-34 asphalt cement should be used for roadways with bus traffic. 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 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

A perimeter foundation drainage system is recommended for proposed structures founded on or above the existing bedrock elevation. The system should consist of a 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, 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 or sump pump.

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 behind foundation walls unless a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system is provided.

Underground Parking Garage

Based on the preliminary information provided, it is expected that a portion of the proposed multi-storey building foundation walls be located below the bedrock surface and could encounter the local groundwater table. To limit long-term groundwater lowering, it is recommended that a groundwater infiltration control system be designed for the proposed buildings. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater which breaches the primary ground infiltration control system.

The groundwater infiltration control system should extend to the ground level and the following is suggested for preliminary design purposes.

- ☐ Place a composite drainage layer, such as Delta Drain 6000 or equivalent, against the foundation wall (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
- ☐ Place a suitable waterproofing membrane, such as a bentomat liner system or equivalent, over a composite drainage system. The membrane liner should extend down to footing level. The membrane liner should also extend horizontally a minimum 600 mm below the footing at underside of footing level.

It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3-6 m centers be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area. It is important to note that the building's sump pit and elevator pit be considered for waterproofing in a similar fashion. A detail can be provided by Paterson once the design drawings are available.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas throughout the remainder of the subject site should be provided with a minimum 300 mm thick layer of OPSS Granular A or OPSS Granular B Type II. The subgrade material should be shaped to promote positive drainage towards the buildings perimeters drainage system.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 m centers. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

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.

The parking garage is expected to not require protection against frost action due to the founding depth. Unheated structures such as the access ramp may be required to be insulated against the deleterious effect of frost action.

6.3 Excavation Side Slopes

The excavations for the proposed development will be mostly through a stiff silty clay. Where excavation is above the groundwater level to a depth of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Flatter slopes could be required for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be used.

The subsoil at this site is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

The slope cross-sections recommended above are for temporary slopes. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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.

It is expected that deep service trenches in excess of 3 m will be completed using a temporary shoring system designed by a structural engineer, such as stacked trench boxes in conjunction with steel plates. The trench boxes should be installed to ensure that the excavation sidewalls are tight to the outside of the trench boxes and that the steel plates are extended below the base of the excavation to prevent basal heave (if required).

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.

Temporary Shoring

If a temporary shoring system is considered, the design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner’s representative prior to implementation.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced.

The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 5 - Soil Parameters for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (Ka)	0.33
Passive Earth Pressure Coefficient (Kp)	3
At-Rest Earth Pressure Coefficient (Ko)	0.5
Unit Weight (ã), kN/m^3	20
Submerged Unit Weight (ã), kN/m^3	13

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

6.4 Groundwater Control

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into shallow 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. However, due to the presence of artesian pressure within the bedrock, it is expected that the water influx following rock removal will be high.

Permit to Take Water

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

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

A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

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.

6.5 Winter Construction

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

6.6 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 results of the resistivity indicate the presence of a low to moderately aggressive environment for exposed ferrous metals at this site, which is typical of silty clay samples submitted for the subject area. It is anticipated that standard measures for corrosion protection are sufficient for services placed within the silty clay deposit.

6.8 Landscaping Considerations

Tree Planting Considerations

Paterson reviewed the conceptual grading plans prepared by Stantec for the proposed development at the subject site, and the soil conditions encountered during the current geotechnical investigation, to determine the tree planting setback requirements for the proposed development, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines). Based on our review of the project plans, the proposed buildings will have two to four underground parking structures. Based on the proposed founding depth for all the buildings, no tree planting setback will be required. On the other hand, for the proposed townhouses to be located along the west and northwest portions of the site, it is understood that a minimum tree planting setback of 7.5m for small to medium size trees will be adopted in the landscaping design.

Paterson should review the landscape plans once available to confirm that the tree planting setbacks are in accordance with the requirements of the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines).

7.0 Recommendations

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

☐ Review detailed grading plan(s) and landscaping plans 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.

☐ Observation of placement of perimeter and underfloor drainage and waterproofing system

☐ Observation of clay seal placement at specified locations.

☐ 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.

All excess soils generated by construction activities should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

A report confirming that these works have been conducted in general accordance with 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 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 Brigil 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.

LICENSED PROFESSIONAL ENGINEER
December 14, 2023
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Maha Saleh, M.A.Sc., P.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ☐ Brigil (email copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
972 March Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

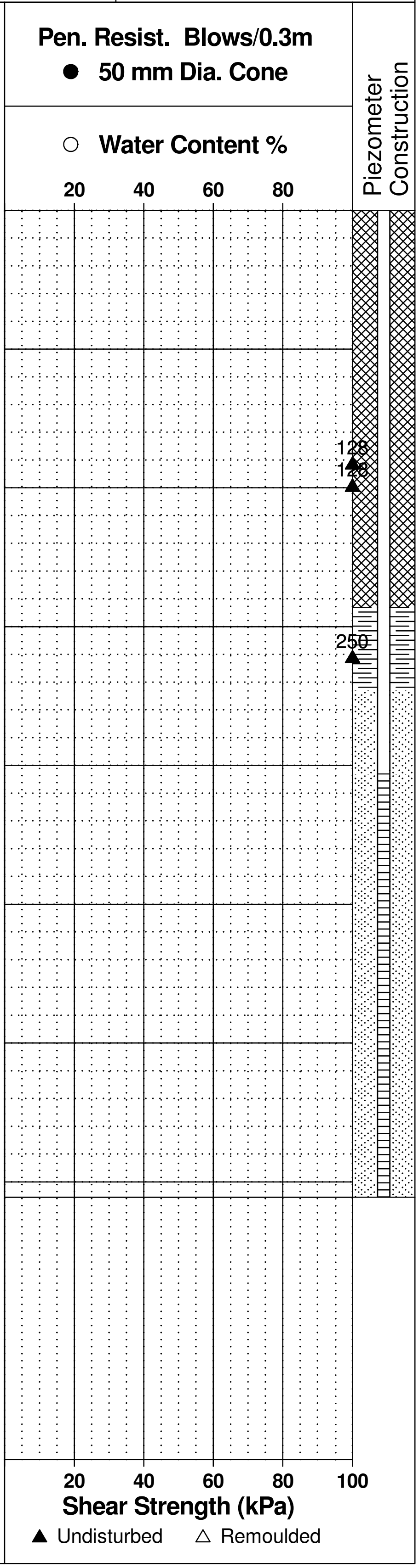
BORINGS BY Track-Mount Power Auger

DATE April 23, 2020

FILE NO. PG5330

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	80.90
TOPSOIL 0.30	AU	1				
Very stiff to hard, brown **SILTY CLAY**	SS	2	96	8	1	79.90
	SS	3		P	2	78.90
3.12	SS	4		3	3	77.90
GLACIAL TILL: Brown silty clay with sand, trace cobbles and boulders 3.66	SS	5	100	50+		
BEDROCK: Good quality, grey limestone	RC	1	79		4	76.90
	RC	2	100	83	5	75.90
	RC	3	100	83	6	74.90
7.11					7	73.90
End of Borehole						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
972 March Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

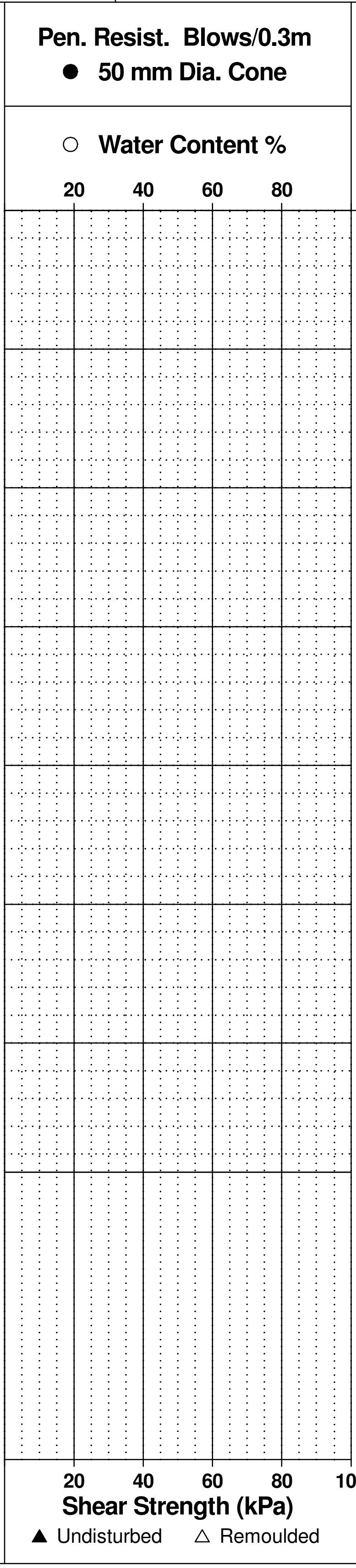
DATE April 24, 2020

FILE NO. PG5330

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL	AU	1		
0.60				
Very stiff, brown **SILTY CLAY**	SS	2	83	11
	SS	3	75	8
2.34				
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders	SS	4	83	9
3.30	SS	5	90	50+
BEDROCK: Excellent quality, grey limestone	RC	1	100	95
	RC	2	100	90
	RC	3	100	98
6.93				
End of Borehole				

DEPTH (m)	ELEV. (m)
0	81.49
1	80.49
2	79.49
3	78.49
4	77.49
5	76.49
6	75.49



Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
972 March Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

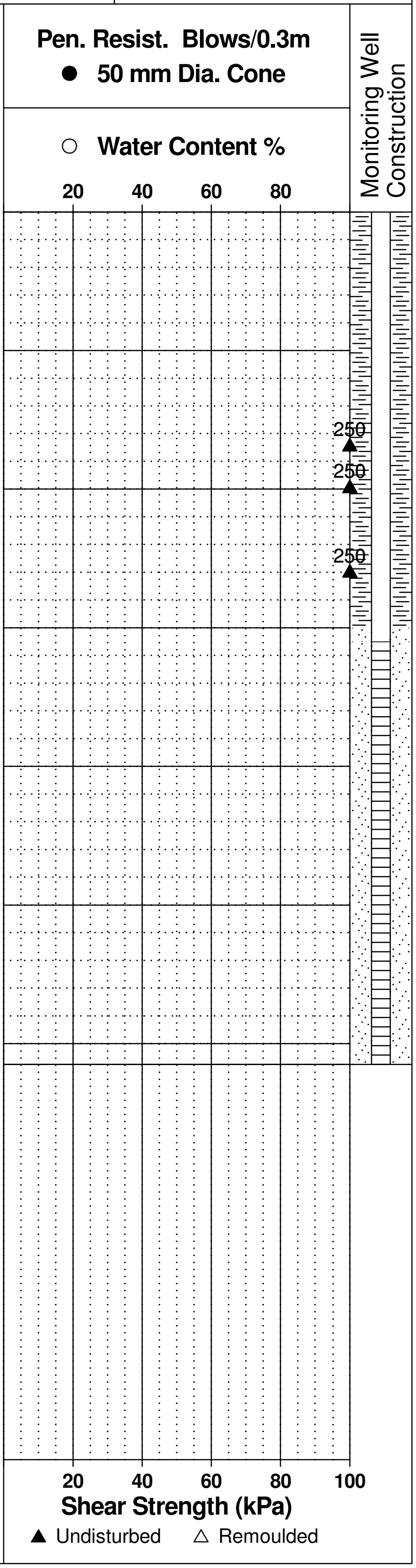
BORINGS BY Track-Mount Power Auger

DATE April 23, 2020

FILE NO. **PG5330**

HOLE NO. **BH 3**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	82.62
TOPSOIL — 0.60	AU	1				
Hard brown **SILTY CLAY**	SS	2	92	13	1	81.62
2.13					2	80.62
GLACIAL TILL: Brown silty clay with gravel, cobbles and boulders — 2.95	SS	3	100	P		
					3	79.62
BEDROCK: Fair to good quality, grey limestone	RC	1	100	70	4	78.62
	RC	2	100	83	5	77.62
6.15	RC	3	90	78	6	76.62
End of Borehole						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
972 March Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5330**

REMARKS

HOLE NO. **BH 4**

BORINGS BY Track-Mount Power Auger

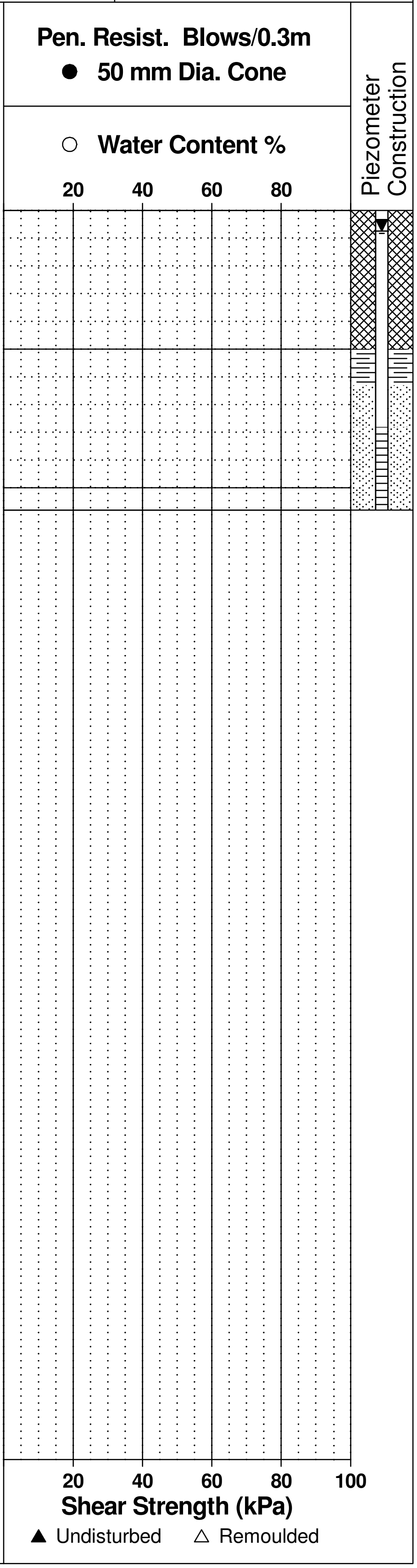
DATE April 23, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	85.90
TOPSOIL 0.60		AU	1				
Brown **SILTY CLAY,** trace sand 1.70		SS	2	43	50+	1	84.90
GLACIAL TILL: Brown silty clay with sand, trace cobbles and boulders 2.16		SS	3	60	50+	2	83.90

End of Borehole

Practical refusal to augering at 2.16m depth

(GWL @ 0.15m - May 1, 2020)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
972 March Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 24, 2020

FILE NO. PG5330

HOLE NO. BH 5

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	83.66		
TOPSOIL 0.30	■	AU	1						▼
Very stiff, brown **SILTY CLAY** 0.81	▨								
End of Borehole Practical refusal to augering at 0.81m depth (GWL @ 0.09m - May 1, 2020)									

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
972 March Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 24, 2020

FILE NO. PG5330

HOLE NO. BH 5A

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									
TOPSOIL 0.30						0	83.66		
Very stiff, brown **SILTY CLAY** 0.79									
End of Borehole Practical refusal to augering at 0.79m depth									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
972 March Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5330

REMARKS

HOLE NO. BH 6

BORINGS BY Track-Mount Power Auger

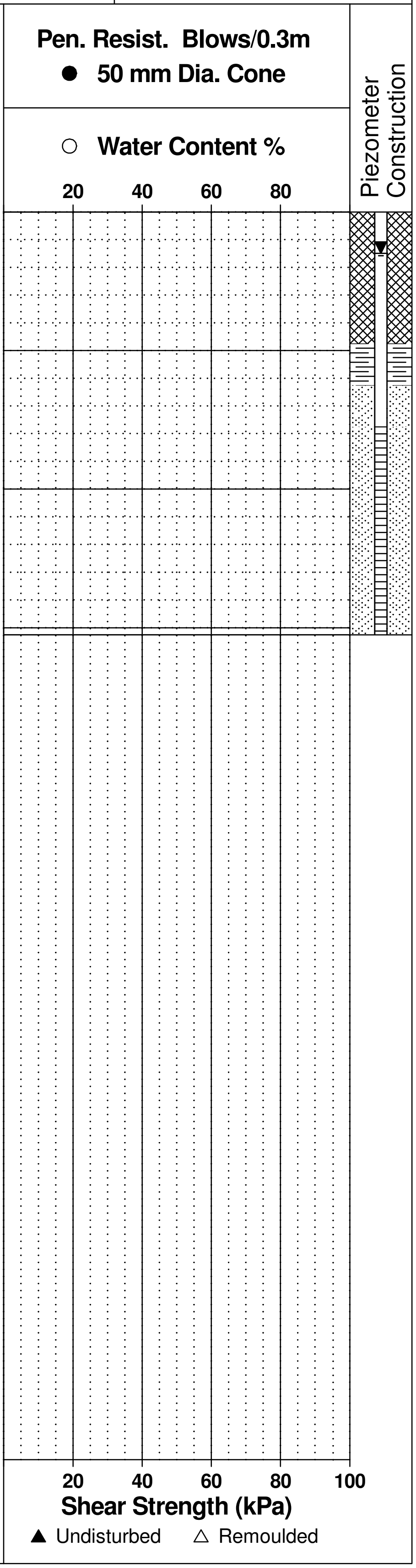
DATE April 24, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	83.73
TOPSOIL 0.30		AU	1				
FILL: Brown silty clay 0.76							
Very stiff, brown **SILTY CLAY** 1.37		SS	2	62	6	1	82.73
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders		SS	3	4	10	2	81.73
3.05		SS	4	50	11	3	80.73

End of Borehole

Practical refusal to augering at 3.05m depth

(GWL @ 0.30m - May 1, 12020)



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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5330

REMARKS

HOLE NO. BH 7

BORINGS BY Track-Mount Power Auger

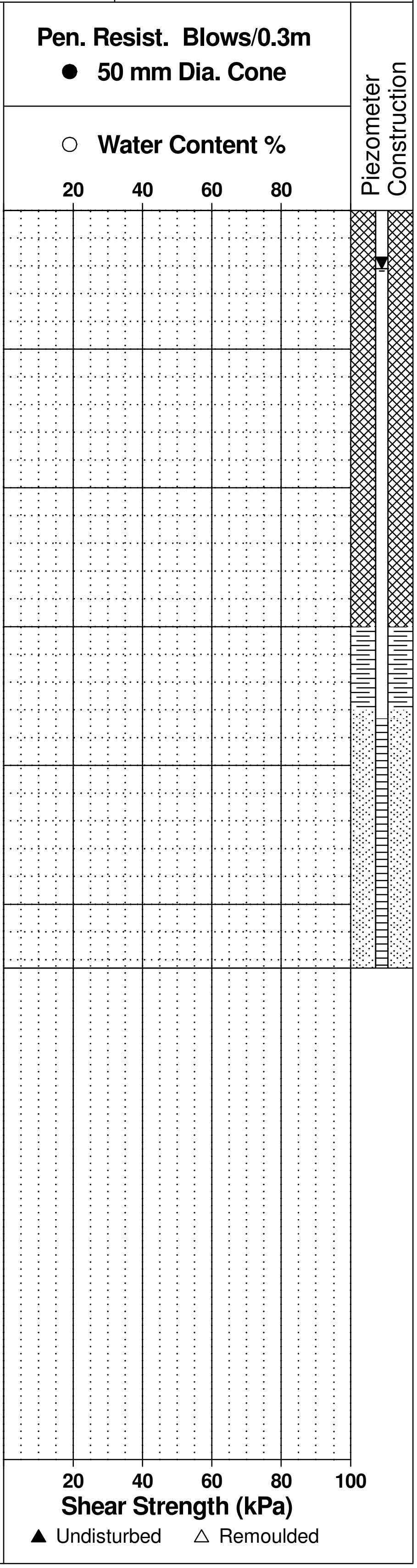
DATE April 24, 2020

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.60	AU	1		
FILL: Brown silty sand, some clay and gravel	1.52	SS	2	83	6
Very stiff, grey **SILTY CLAY**	1.90	SS	3	83	11
GLACIAL TILL: Brown silty clay, some sand, gravel, cobbles and boulders	2.39	SS	4	25	50+
BEDROCK: Excellent quality, grey limestone interbedded with shale		RC	1	100	92
	5.46	RC	2	100	98

End of Borehole

(GWL @ 0.42m - May 1, 12020)

DEPTH (m)	ELEV. (m)
0	82.84
1	81.84
2	80.84
3	79.84
4	78.84
5	77.84



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
972 March Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5330

REMARKS

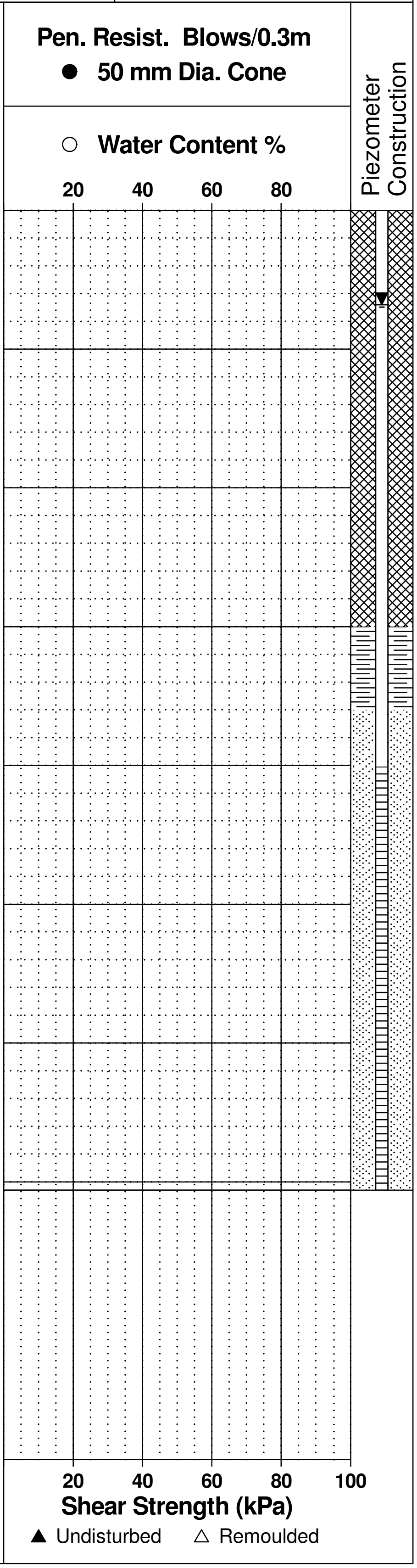
HOLE NO. BH 8

BORINGS BY Track-Mount Power Auger

DATE April 27, 2020

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	81.98
TOPSOIL	0.30	AU	1				
Reworked brown silty clay	1.27	SS	2	75	8	1	80.98
Loose, brown **SILTY SAND**	1.52						
Veyr stiff, brown **SILTY CLAY**		SS	3	75	6	2	79.98
	2.90	SS	4	83	3		
GLACIAL TILL: Brown silty clay, some sand, gravel, cobbles and boulders		SS	5	67	5	3	78.98
	4.01	SS	6	100	50+	4	77.98
BEDROCK: Good to excellent quality, grey limestone		RC	1	100	88	5	76.98
		RC	2	100	92	6	75.98
	7.06	RC	3	98	97	7	74.98
End of Borehole							

(GWL @ 0.68m - May 1, 12020)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM Geodetic

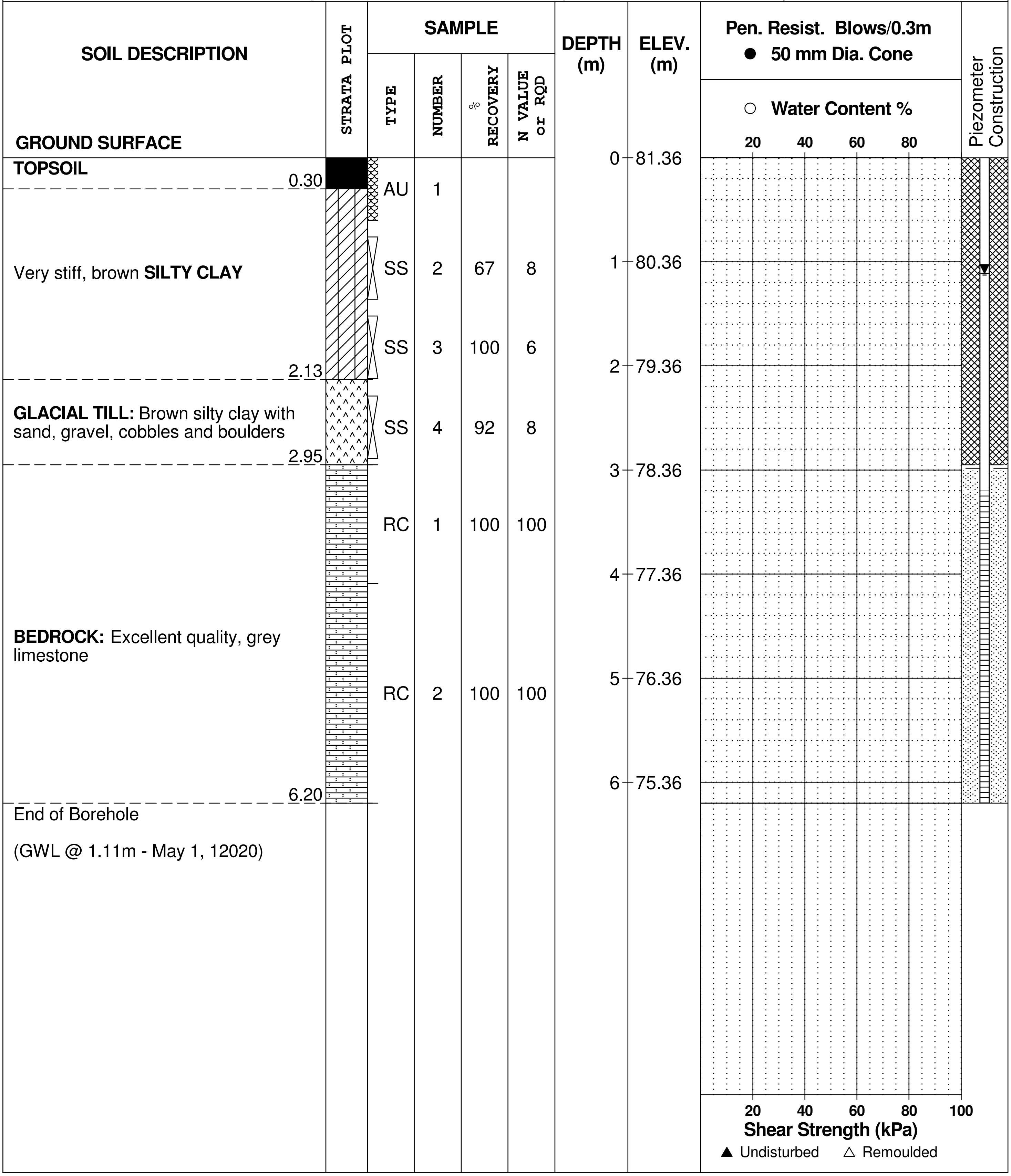
REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 27, 2020

FILE NO. PG5330

HOLE NO. BH 9



SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30	AU	1		
Very stiff, brown **SILTY CLAY**		SS	2	67	8
	2.13	SS	3	100	6
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders	2.95	SS	4	92	8
BEDROCK: Excellent quality, grey limestone		RC	1	100	100
	6.20	RC	2	100	100

End of Borehole

(GWL @ 1.11m - May 1, 12020)

DEPTH (m)	ELEV. (m)
0	81.36
1	80.36
2	79.36
3	78.36
4	77.36
5	76.36
6	75.36

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
972 March Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 27, 2020

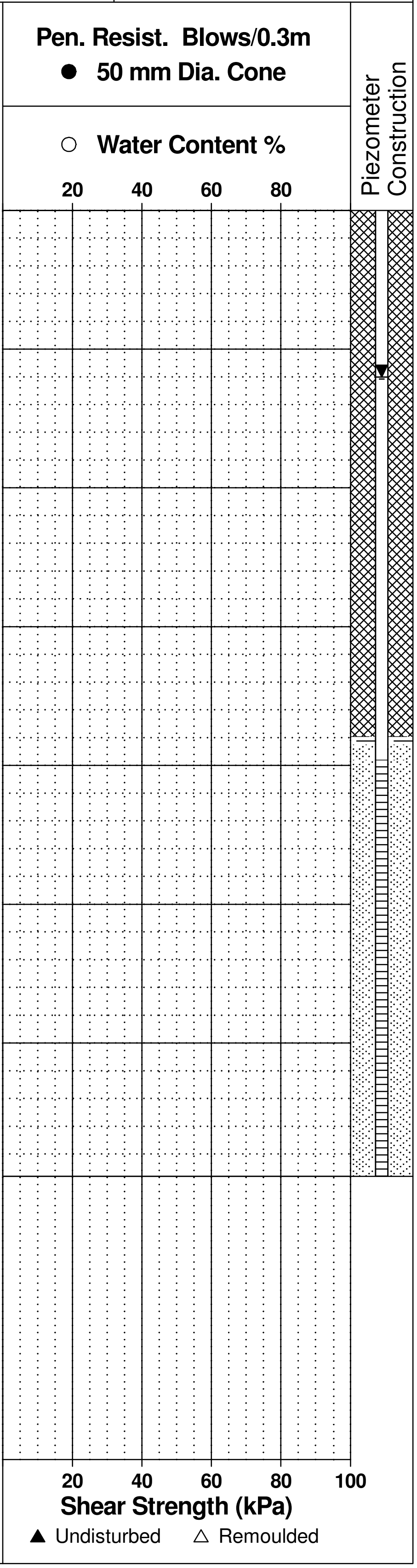
FILE NO. PG5330

HOLE NO. BH10

SOIL DESCRIPTION	Layer bottom (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.46	AU	1		
Very stiff, brown **SILTY CLAY**	2.84	SS	2	75	11
		SS	3	100	7
		SS	4	100	4
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders	3.79	SS	5	96	12
BEDROCK: Fair to excellent quality, grey limestone	6.96	RC	1	93	60
		RC	2	100	66
		RC	3	100	100
End of Borehole					

(GWL @ 1.20m - May 1, 12020)

DEPTH (m)	ELEV. (m)
0	80.03
1	79.03
2	78.03
3	77.03
4	76.03
5	75.03
6	74.03



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
972 March Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE April 24, 2020

FILE NO. PG5330

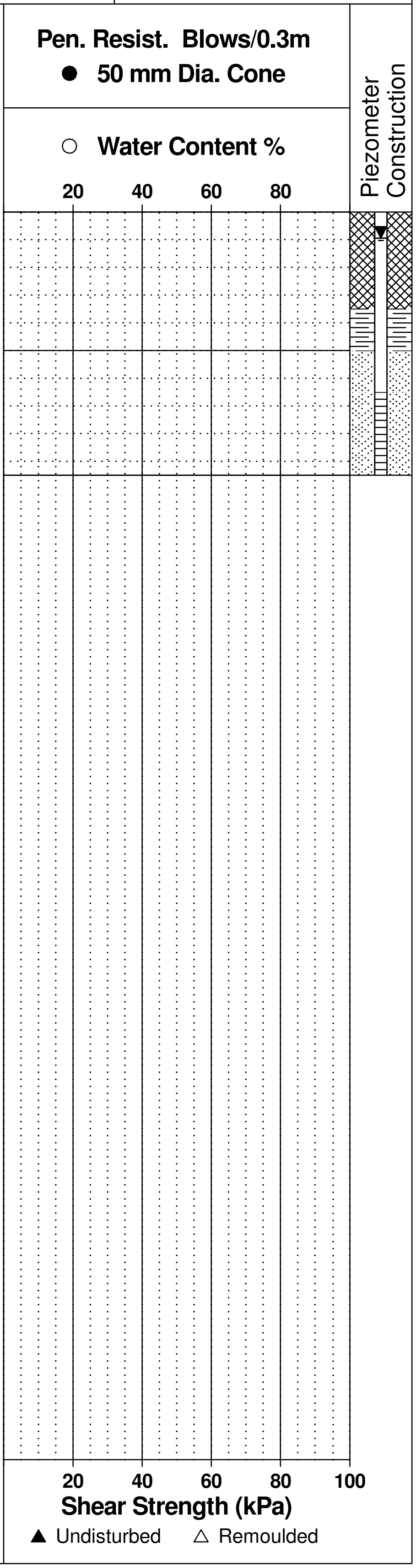
HOLE NO. BH11

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	83.78
TOPSOIL 0.60	AU	1				
Very stiff, brown **SILTY CLAY** 1.37	SS	2	79	7	1	82.78
GLACIAL TILL: Brown silty clay, some gravel, trace sand 1.90	SS	3	18	50+		

End of Borehole

Practical refusal to augering at 1.90m depth

(GWL @ 0.19m - May 1, 12020)



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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
927 March Road at Old Carp Rd.
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. **PG1626**

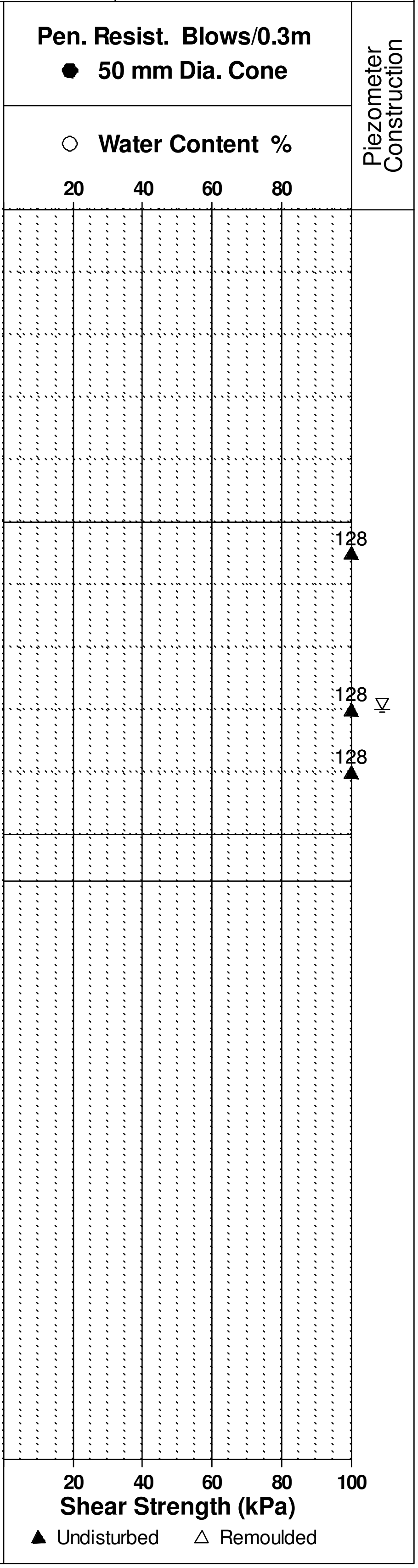
REMARKS

HOLE NO. **TP 1**

BORINGS BY Hydraulic Shovel

DATE Feb 25, 08

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	81.47
TOPSOIL 0.30	■						
Very stiff, brown **SILTY CLAY**	▨	G	1			1	80.47
2.15	▨					2	79.47
End of Test Pit TP terminated on bedrock surface @ 2.15m depth (Groundwater infiltration @ 1.6m depth)							



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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
927 March Road at Old Carp Rd.
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG1626

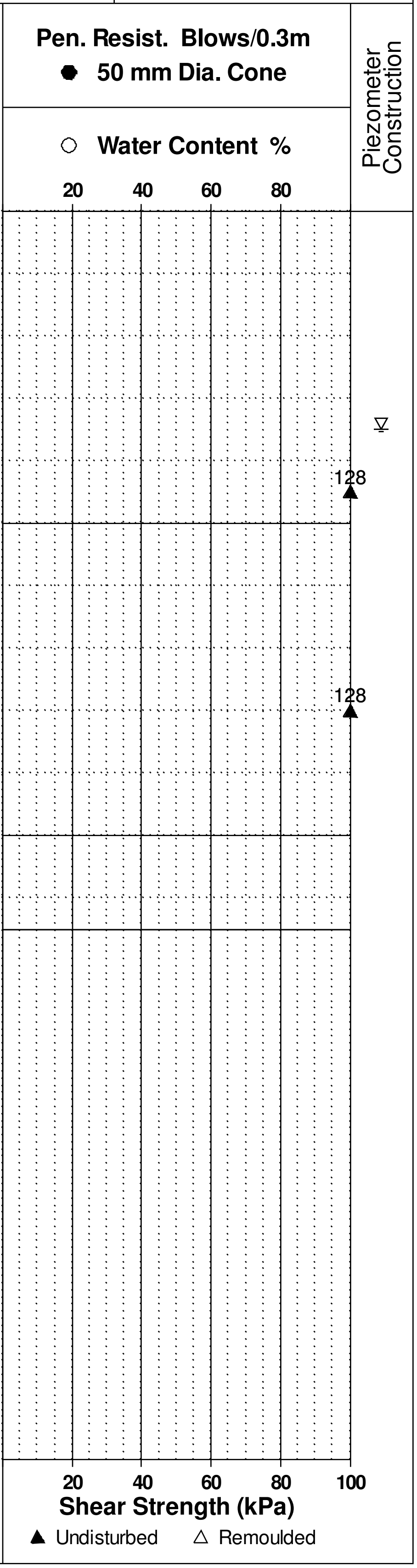
REMARKS

HOLE NO. TP 2

BORINGS BY Hydraulic Shovel

DATE Feb 25, 08

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	83.13
TOPSOIL 0.25							
Very stiff, brown **SILTY CLAY**		G	1			1	82.13
1.80							
GLACIAL TILL: Brown silty sand with gravel, cobbles, boulders, trace clay		G	2			2	81.13
2.30							
End of Test Pit TP terminated on bedrock surface @ 2.30m depth (Groundwater infiltration @ 0.70m depth)							



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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
927 March Road at Old Carp Rd.
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

REMARKS

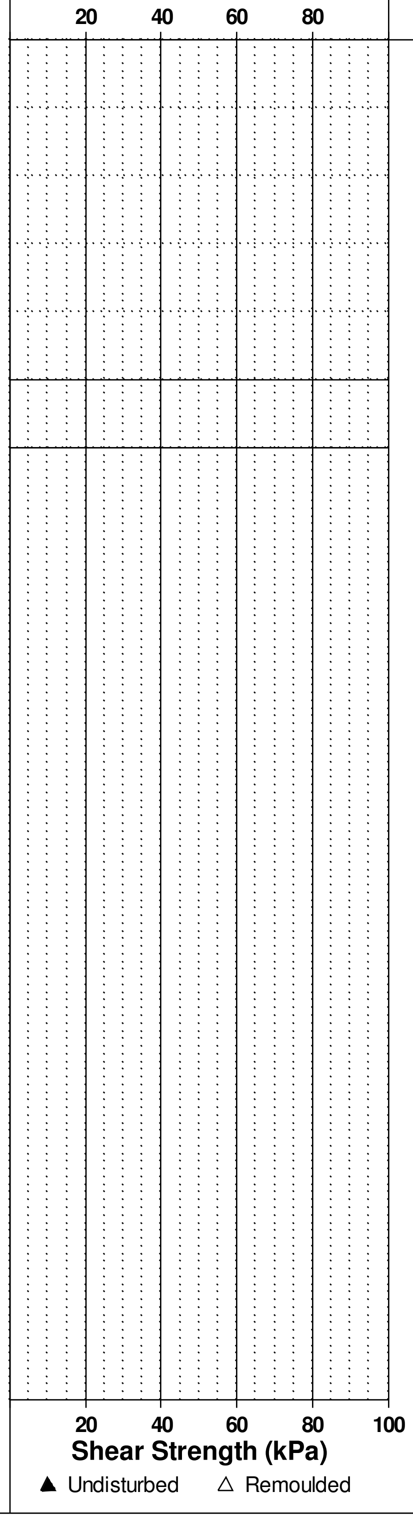
BORINGS BY Hydraulic Shovel

DATE Feb 25, 08

FILE NO. PG1626

HOLE NO. TP 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
GROUND SURFACE						0	83.59		
TOPSOIL 0.30	■								
Very stiff, brown **SILTY CLAY** 0.70	▨								
BEDROCK 1.20	▤					1	82.59		
End of Test Pit (TP dry upon completion)									



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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
927 March Road at Old Carp Rd.
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

REMARKS

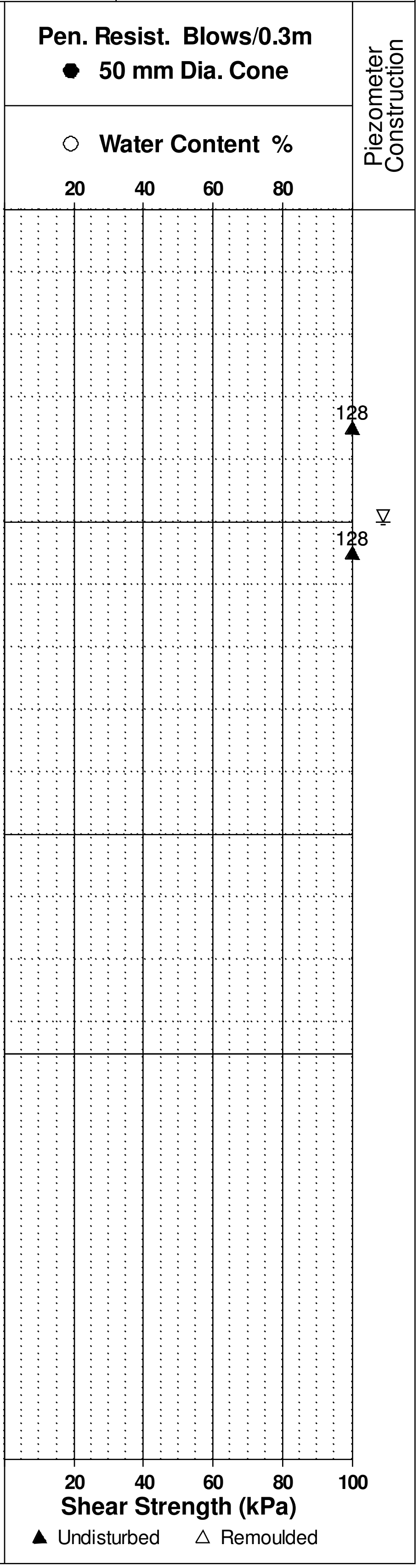
BORINGS BY Hydraulic Shovel

DATE Feb 25, 08

FILE NO. PG1626

HOLE NO. TP 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	83.80
TOPSOIL 0.30							
Very stiff, brown **SILTY CLAY**		G	1			1	82.80
1.65							
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders		G	2			2	81.80
2.70		G	3				
End of Test Pit TP terminated on bedrock surface @ 2.70m depth (Groundwater infiltration @ 1.0m depth)							



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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
927 March Road at Old Carp Rd.
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG1626

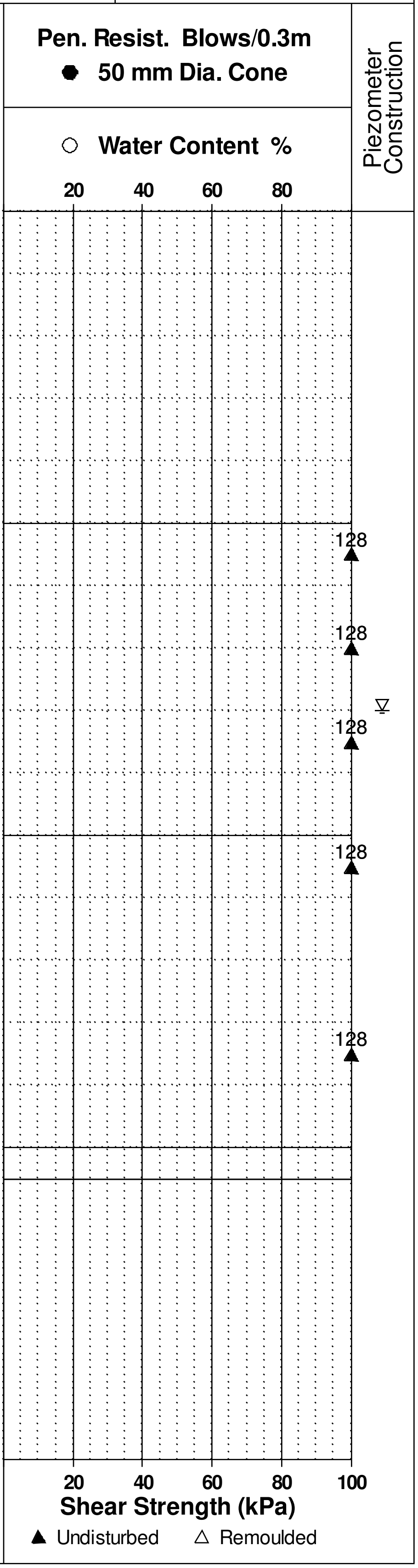
REMARKS

HOLE NO. TP 5

BORINGS BY Hydraulic Shovel

DATE Feb 25, 08

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	81.55
TOPSOIL 0.25						
	G	1				
					1	80.55
Very stiff, brown **SILTY CLAY**						
					2	79.55
	G	2				
2.80						
GLACIAL TILL: Brown silty sand with clay, gravel, cobbles and boulders	G	3				
					3	78.55
3.10						
End of Test Pit TP terminated on bedrock surface @ 3.10m depth (Groundwater infiltration @ 1.6m 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

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)

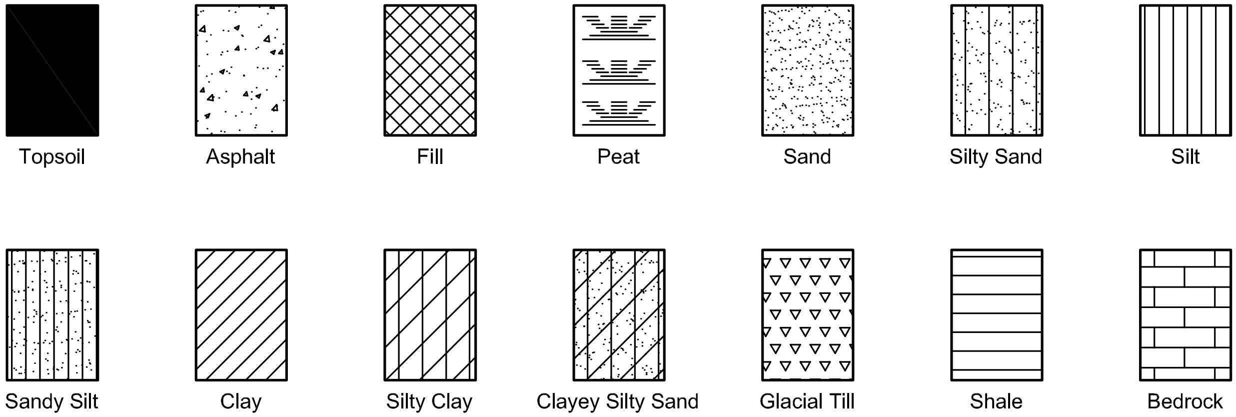
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

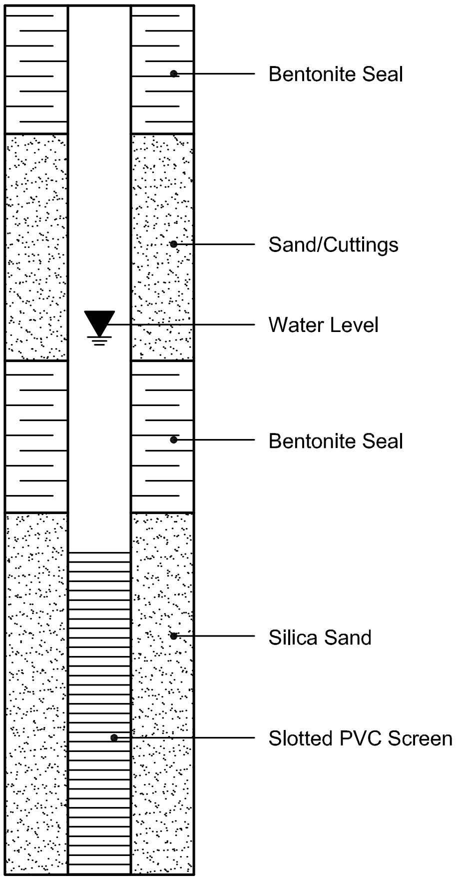
SYMBOLS AND TERMS (continued)

STRATA PLOT

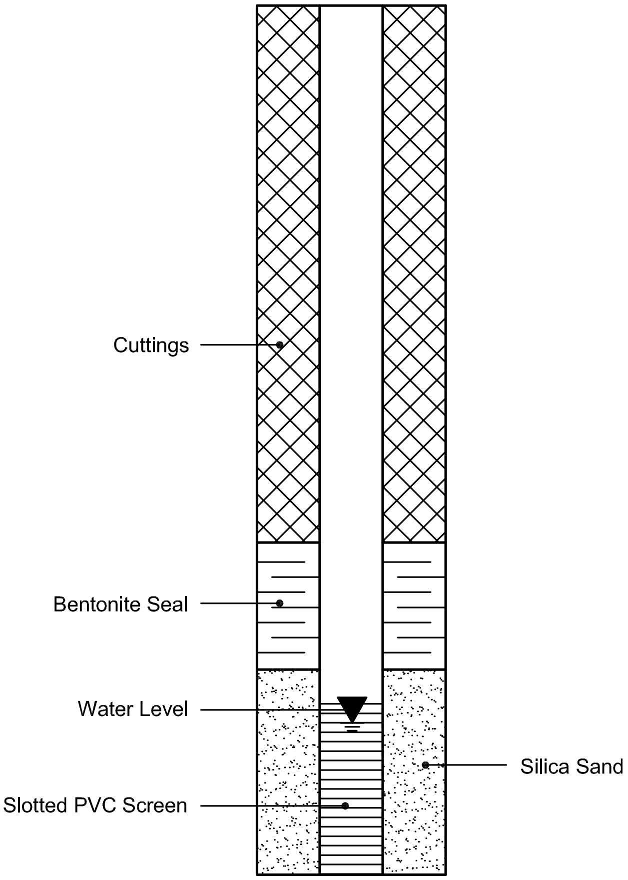


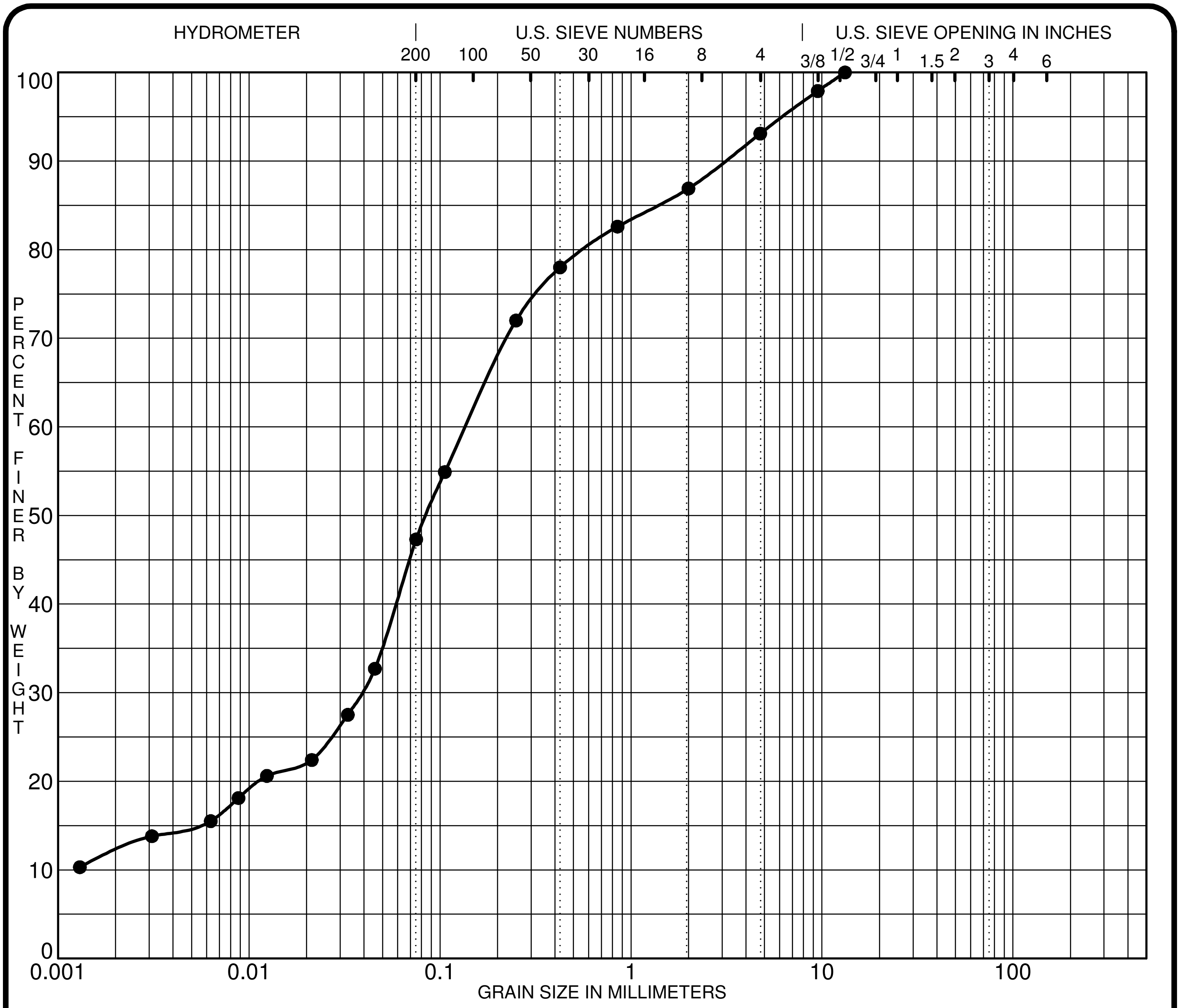
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

Specimen Identification			Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 2**	**SS2**							
☒									
▲									
★									

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 2**	**SS2**	**13.20**	**0.14**	**0.038**		**6.9**	**45.8**	**47.3**	
☒										
▲										
★										

CLIENT **Brigil Construction**

FILE NO. **PG5330**

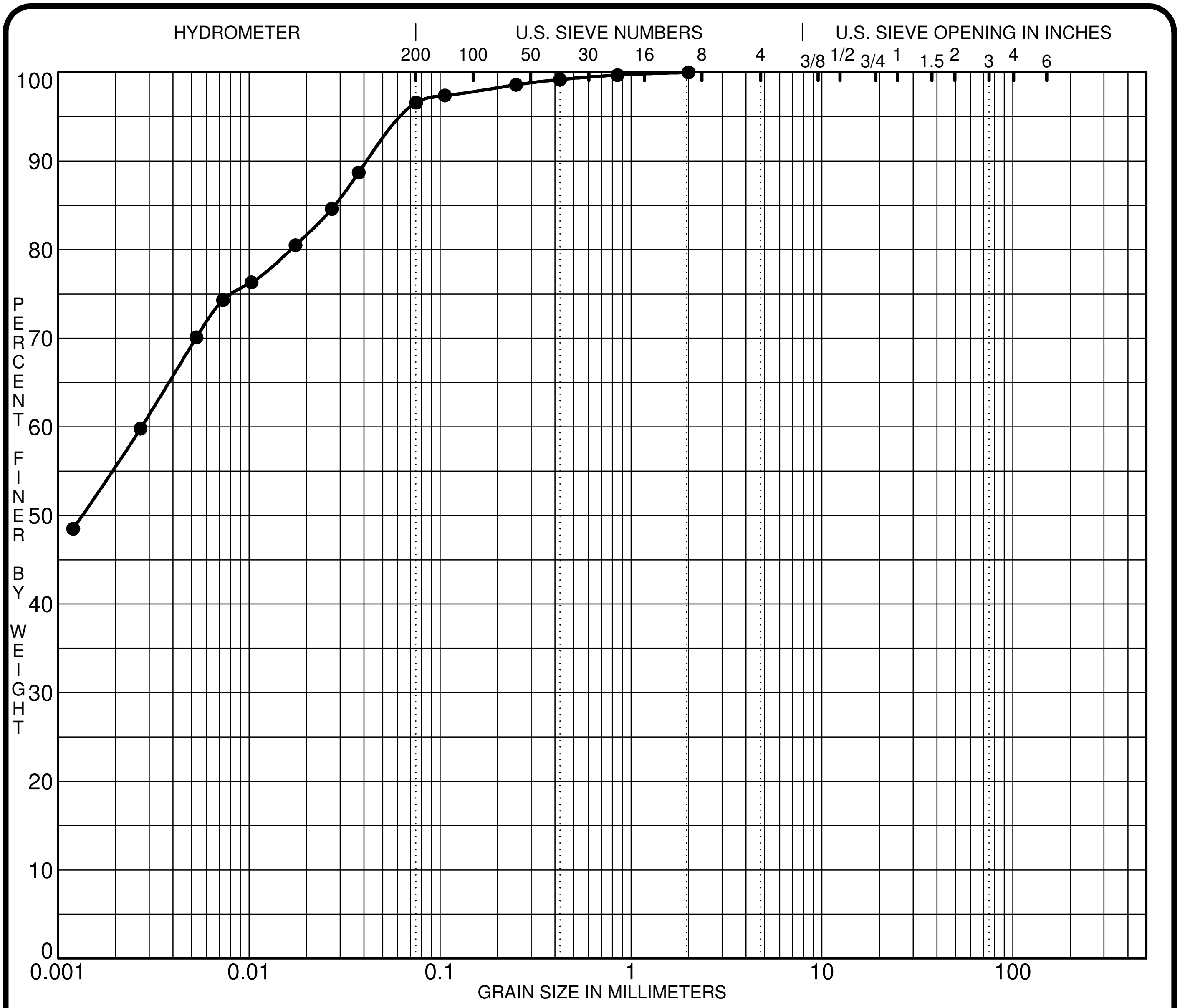
PROJECT **Geotechnical Investigation - 927 March Road**

DATE **24 Apr 20**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 2**	**SS3**							
⌧									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 2**	**SS3**	**2.00**	**0.00**			**0.0**	**3.4**	**96.6**	
⌧										
▲										
★										

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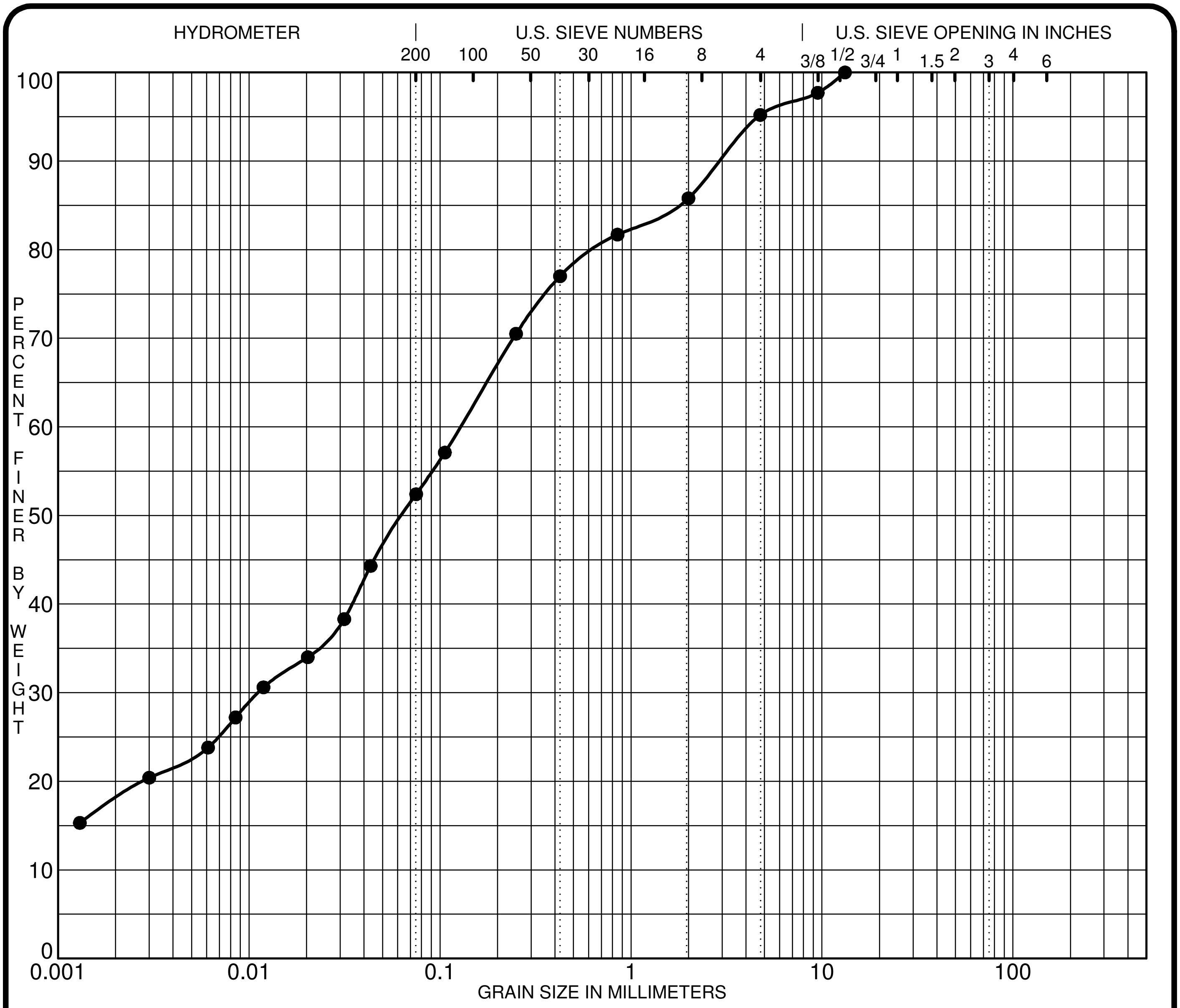
GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 3**	**SS4**							
☒									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 3**	**SS4**	**13.20**	**0.13**	**0.011**		**4.8**	**42.8**	**52.4**	
☒										
▲										
★										

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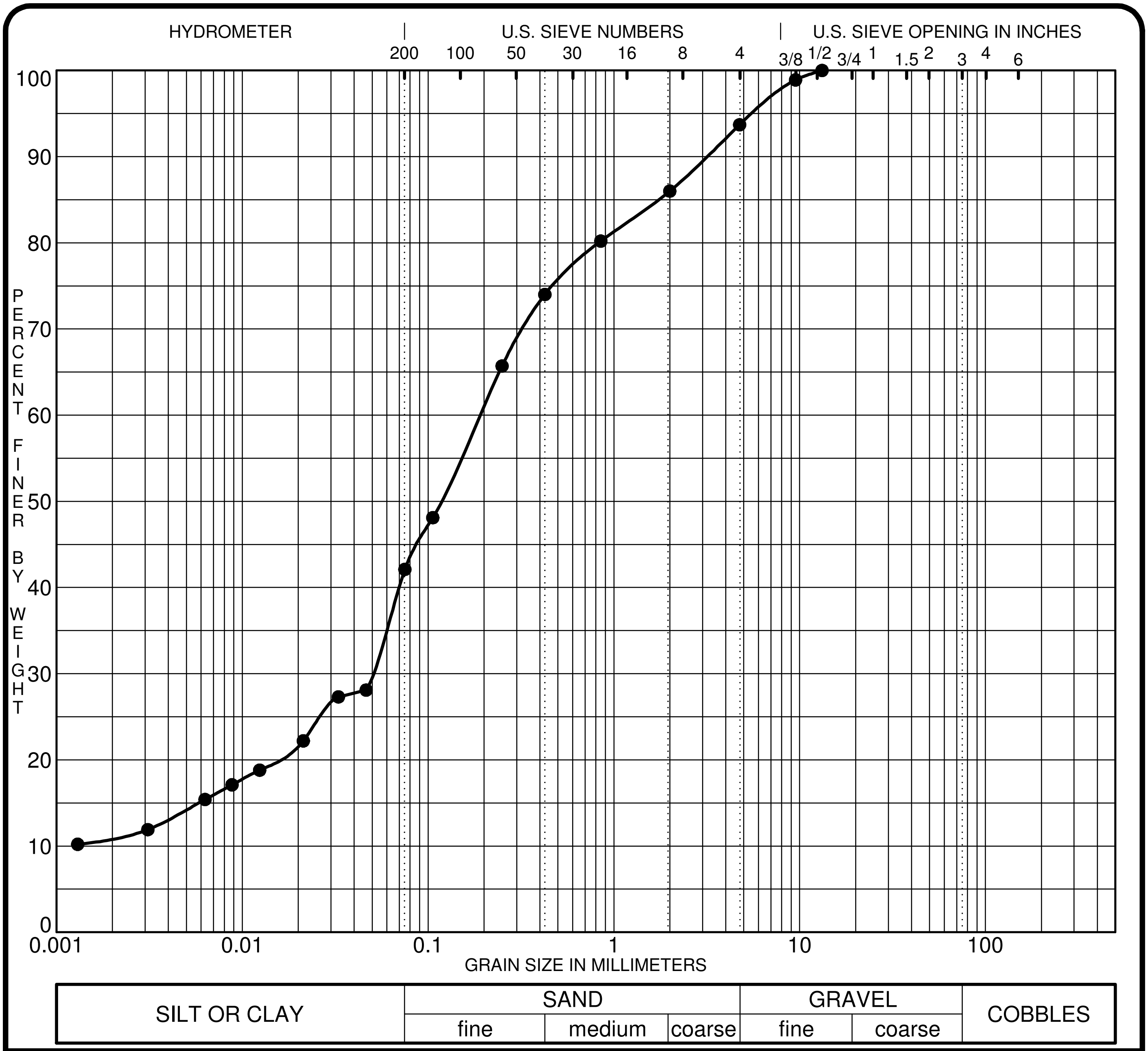
GRAIN SIZE DISTRIBUTION



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 6**	**SS4**							
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 6**	**SS4**	**13.20**	**0.19**	**0.050**		**6.3**	**51.6**	**42.1**	
⊠										
▲										
★										

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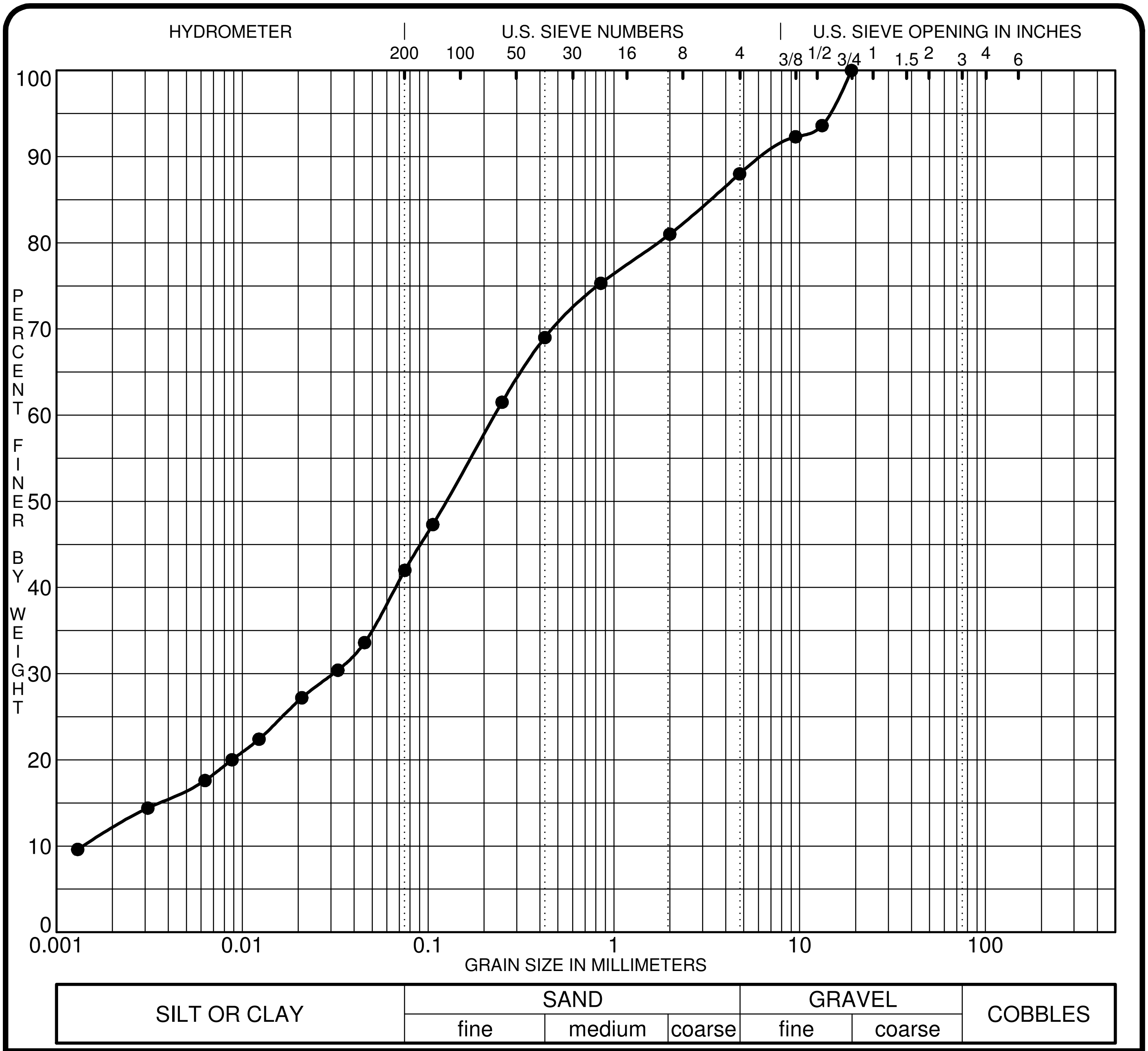
GRAIN SIZE DISTRIBUTION



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH10 SS5**						**3.00**	**163.4**
☒								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH10 SS5**	**19.00**	**0.23**	**0.031**	**0.0014**	**12.0**	**46.0**	**42.0**	
☒									
▲									
★									

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PROJECT **Geotechnical Investigation - 927 March Road**

DATE **27 Apr 20**

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5330-1 – TEST HOLE LOCATION PLAN

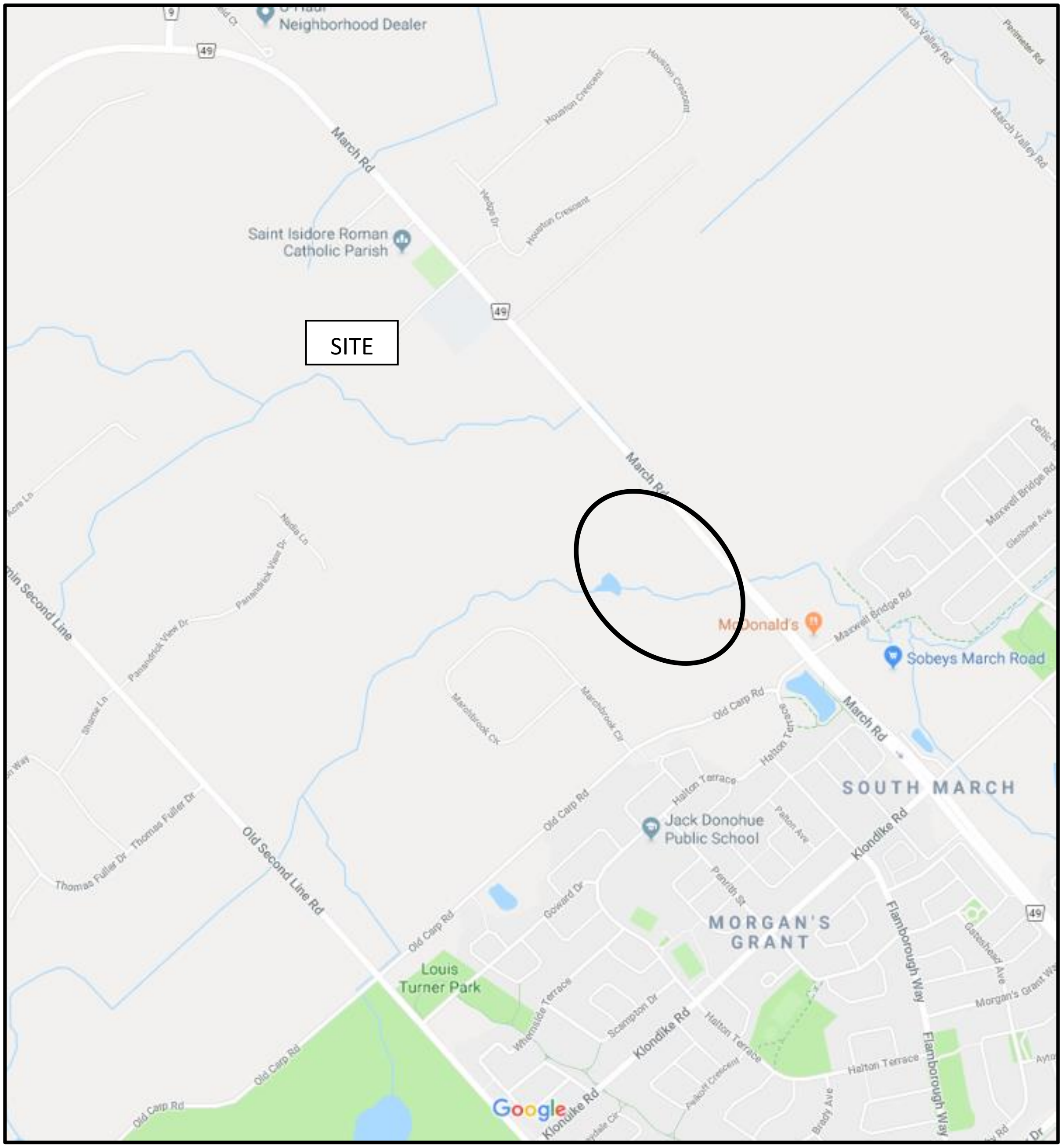
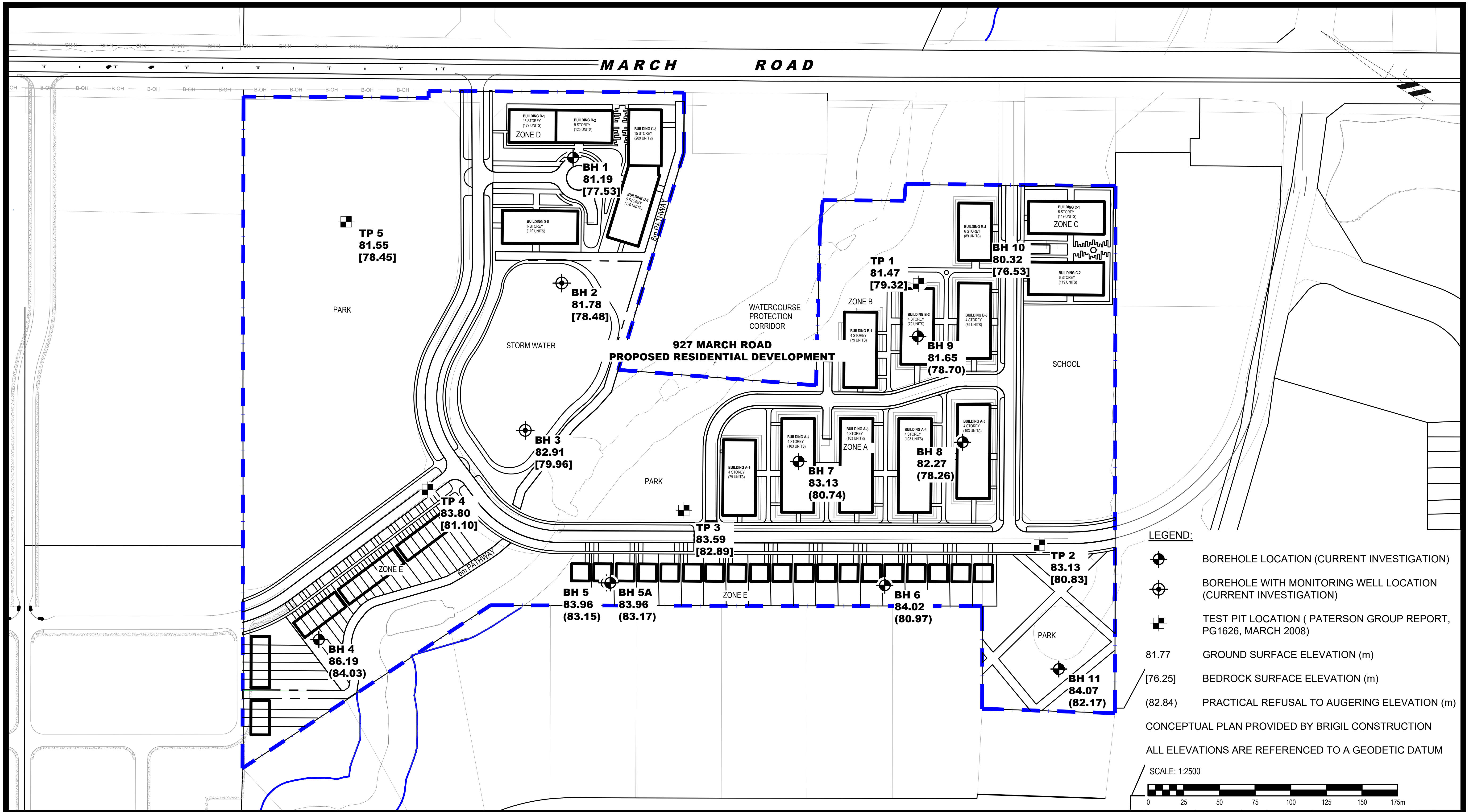


FIGURE 1

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION (CURRENT INVESTIGATION)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (CURRENT INVESTIGATION)
▚	TEST PIT LOCATION (PATERSON GROUP REPORT, PG1626, MARCH 2008)
81.77	GROUND SURFACE ELEVATION (m)
[76.25]	BEDROCK SURFACE ELEVATION (m)
(82.84)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY BRIGIL CONSTRUCTION

ALL ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED CONCEPTUAL PLAN	04/12/2023	GA

BRIGIL CONSTRUCTION

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT- 927 MARCH ROAD

OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:2500
Date: 05/2020
Drawn by: YA
Report No.: PG5330-1
Checked by: JV
Dwg. No.: PG5330-1
Approved by: DJG
Revision No.: 1

p:\autocad drawings\geotechnical\pg53xx\pg5330\pg5330-1-test hole location plan (rev.01).dwg