PATERSON GROUP

Geotechnical Investigation

Proposed Commercial Development

60 Denzil Doyle Court
Ottawa, Ontario

Prepared for Access Property Development Inc. and Huntington Properties

Report PG3798-2 dated November 23, 2022

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Access Property Development Inc. and Huntington Properties to conduct a geotechnical investigation for the proposed development to be located at 60 Denzil Doyle Court in the City of Ottawa, Ontario (reference should be made to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of test holes.
- Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of several one-storey storage unit buildings (Building B to Building F) and a two-storey office building (Building A). It is understood the storage and office buildings will consist of slab-on-grade construction supported by a thickened-edge slab and conventional spread footing foundation, respectively.

It is also understood that associated access lanes and parking areas will be provided as part of the proposed development. It is anticipated that the development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was completed at the subject site by Paterson on April 14, 2016. At that time, a total of six (6) boreholes were advanced to a maximum depth of 6.4 m below ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG3798-2 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples collected from the boreholes were either recovered directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All soil samples were visually inspected and initially classified on site. The soil samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification.

The depths at which the auger and split spoon samples were recovered from the test holes are shown as AU and SS, 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 at regular intervals of depth in cohesive soils using a field vane apparatus.

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

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

Groundwater

All boreholes were fitted with flexible standpipe piezometers to allow for groundwater level monitoring. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson and referenced to the top of a manhole cover with an assumed geodetic elevation of 101.63 m, as provided by David Schaeffer Engineering Limited (DSEL).

The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG3798-2 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and generally vacant. The site is bordered by Denzil Doyle Court to the west, the intersection of Terence Matthews Court and Michael Cowpland Drive to the south, commercial and residential properties followed by Eagleson Road to the east, and commercial buildings to the north. The existing ground surface slopes gently to the south and is at grade with the surrounding properties.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consists of a thin layer of topsoil and/or fill material underlain by a deposit of silty clay.

The silty clay deposit generally consisted of a hard to stiff brown weathered crust to depths ranging between 2.3 and 3.8 m below ground surface. The brown silty clay was observed to be underlain by a stiff to soft grey silty clay.

Practical refusal to DCPT was encountered at an approximate depth of 15.8 m below existing ground surface at the location of boreholes BH 2.

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 subject area consists of Paleozoic interbedded limestone and shale of the Verulam formation, with an overburden drift thickness of 5 to 10 m depth.

4.3 Groundwater

Groundwater levels were measured on April 21, 2016 within the installed piezometers. The measured groundwater levels are presented in Table 1 below. However, it is important to note that groundwater readings can be influenced by surface water perched within the borehole backfill material.

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1	102.62	1.44	101.18	April 21, 2016
BH 2	102.04	1.17	100.87	
BH 3	101.47	0.76	100.71	
BH 4	101.57	1.12	100.45	
BH 5	101.90	0.64	101.26	
BH 6	101.57	1.04	100.53	

Note: The ground surface elevation at each test hole location was surveyed using a temporary benchmark (TBM) consisting of the top of a manhole cover with a geodetic elevation of 101.63 m as provided by DSEL Ltd.

It should be noted that surface water can become trapped within a backfilled borehole, which can lead to higher than typical groundwater level observations. Similarly, it is our experience that surface water generated by snowmelt and rainfall events may sheet drain into the borehole column given the relatively impermeable nature of the clay soil surface.

The long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 2 to 3 m below ground surface. However, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is understood the following buildings will be founded by the following foundation systems:

- **Building A (Two-Storey Office Building)**: conventional spread footings bearing on an undisturbed very stiff silty clay bearing surface.
- **Remaining Buildings (One-Storey Storage Units)**: a thickened-edge slab foundation structure placed on an appropriately prepared engineered fill pad underlain by an undisturbed very stiff silty clay bearing surface.

Due to the presence of the silty clay layer, the subject site will have a permissible grade raise restriction.

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids.

If site-excavated brown silty clay, free of organics and deleterious materials, is to be used to build up the subgrade level for areas to be paved, it is recommended that the material be placed under dry conditions and above freezing temperatures. The silty clay should be compacted in thin lifts (i.e., up to between 300 to 500 mm thick) and compacted using a suitably sized vibratory sheepsfoot roller.

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

5.3 Foundation Design

Bearing Resistance Values – Conventional Spread Footings – Building A

Strip footings up to 3 m wide, and pad footings up to 5 m wide, placed over an undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at Serviceability Limit States (SLS) of **120 kPa,** and a factored bearing resistance value at Ultimate Limit States (ULS) of **180 kPa**. A geotechnical factor of 0.5 was incorporated to the bearing resistance value at ULS.

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

The above-noted bearing resistance values at SLS for undisturbed, stiff silty clay bearing surfaces will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Bearing Resistance Values – Thickened-Edge Slab– Building B to Building F

A thickened-edge concrete slab placed upon an undisturbed, very stiff brown silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **120 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **180 kPa**. The bearing medium must be reviewed and approved by Paterson personnel at the time of construction and prior to placement of concrete or engineered fill.

Thickened-edge slab strip footings designed using the bearing resistance value at SLS provided above will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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, in the dry, prior to the placement of concrete for footings.

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 the in-situ soils 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.

Permissible Grade Raise Restrictions

Based on the undrained shear strength testing results, a permissible grade raise of **0.8 m** is recommended for the subject site and within 6 m of building footprints. A permissible grade raise restriction of up to **1.1 m** is recommended for areas located a minimum of 6 m beyond building footprints.

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

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the proposed shallow foundations bearing on the undisturbed, very stiff silty clay deposit.

The soils underlying the subject site 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 Thickened Edge / Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprint of the proposed building, the existing fill or stiff silty clay, approved by Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD. It is recommended that the upper 300 mm of sub-slab fill should consist of a Granular A crushed stone for either slab-on-grade or thickened-edge concrete slab construction. Unheated thickened-edge footing slab-on-grade structures should are recommended to be provided with a minimum 450 mm thick layer of OPSS Granular A crushed stone below their footprint.

A layer of rigid insulation should be placed directly below and beyond the engineered fill pad for thickened-edge slab structures as detailed in Section 6.2 of this report.

Any soft or disturbed areas should be removed and backfilled with appropriate backfill material prior to placing fill. OPSS Granular B Type I or II, with a maximum particle size of 50 mm, are recommended for in-filling soft spots and for building up the subgrade below the slab-on-grade.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

5.6 Pavement Design

For design purposes, the pavement structure presented in the following tables could be used for the design of asphalt-paved 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
300	**SUBBASE –** OPSS Granular B Type II
SUBGRADE – Either fill, in-situ soil, or OPSS Granular B Type I or II material over in-situ soil or fill.	

Table 3 – Recommended Pavement Structure – Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course –** HL-8 or Superpave 19 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 over fill or in-situ soil.	

Minimum Performance Grade (PG) 58-34 asphalt cement should be used for this project.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

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.

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. These drains should extend in four orthogonal directions or longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines, or the pipe, should be wrapped with suitable filter cloth. 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. Discharge of the subdrains should be directed by gravity to storm sewers or deeper drainage ditches.

6.0 Design and Construction Precautions

6.1 Foundation Backfill

Foundation Drainage

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 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 structures. The clear stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls unless placed in conjunction with a composite drainage system, such as Delta Drain 6000, Miradrain G100N or equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose.

Concrete Sidewalks Adjacent to Buildings

To avoid differential settlements within the proposed walkways adjacent to the proposed building, it is recommended that the upper 600 mm of backfill placed below the walkways adjacent to the building footprint to consist of free draining, non-frost susceptible material such as, Granular A or Granular B Type II.

The granular material should be placed in maximum 300 mm loose lifts and compacted to 98% of the material's SPMDD using suitable compaction equipment. Consideration could be given to placing a rigid insulation layer below the granular fill layer to prevent frost heave issues at the building entrances.

6.2 Protection of Footings Against Frost Action

Perimeter footings of structures are required to be insulated against the deleterious effect of frost action. A minimum 1.5 m thick layer of soil cover (or equivalent as provided by the geotechnical consultant) should be provided for heated structures in this regard. A minimum 2.1 m thick layer of soil cover (or equivalent as provided by the geotechnical consultant) should be provided for heated structures in this regard.

Frost Protection for Thickened Edge Slab Structures

In order to adequately protect the founding soils from the migration of frost throughout the footprint of the proposed thickened-edge slab structures, the following recommendations are provided with regards to the use of rigid insulation.

Heated Storage Structures

Perimeter footings and the underlying founding soils of heated structures are recommended to be provided a minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation. To achieve this, it is recommended that the 300 mm thick sub-slab fill layer be underlain by a 50 mm thick layer of HI-40 XPS (extruded polystyrene) rigid insulation.

The 50 mm thick layer of rigid insulation should extend a minimum of 1.2 m horizontally beyond the outside exterior perimeter of the proposed thickened edge slab structures. Further, the layer of rigid insulation should extend below the entire footprint of the thickened-edge footing and a minimum of 600 mm below the building footprint and beyond the interior face of the thickened edge footing. The insulation layer should be placed upon an undisturbed soil bearing medium and below the sub-slab fill layer when located within the building footprint.

Unheated Storage Structures

Perimeter footings and the underlying founding soils of unheated structures are recommended to be provided a minimum of 2.1 m of soil cover alone, or a combination of soil cover and foundation insulation.

To achieve this, it is recommended that a minimum 450 mm thick layer of sub-slab fill layer be underlain by a 100 mm thick layer of HI-40 XPS (extruded polystyrene) rigid insulation or an approved alternative below the entire footprint of the unheated storage structures.

The 100 mm thick layer of rigid insulation should extend a minimum of 1.8 m horizontally beyond the outside exterior perimeter of the proposed thickened edge slab structures. The insulation layer should be placed upon an undisturbed soil bearing medium and below the sub-slab fill layer when located within the building footprint.

Frost Taper Recommendations

A frost taper is recommended in areas where hard surfaces (asphaltic parking and access lanes) are placed adjacent to the concrete pad structure. It is recommended to sub-excavate at least 300 mm below the subgrade level of the pavement structure along the outside edge of the rigid insulation to provide a suitable frost taper.

The sub-excavated area should extend at least 600 mm horizontally and beyond the exterior face of the rigid insulation layer. A minimum 3H:1V slope profile can be used to raise the sub-excavated area back to subgrade level. The frost taper area should be backfilled with a free draining, non-frost susceptible engineered fill, such as OPSS Granular A or OPSS Granular B Type II compacted to a minimum of 98% of the materials SPMDD.

Additional Considerations and Remarks

It is recommended that the rigid insulation layer be placed below, and not above, the sub-slab fill layer throughout the footprint of the proposed storage structures. Placing the insulation layer directly below the concrete slab structure will diminish the ability for the underlying layer of soil to retain subsurface heat. Provided the above-noted insulation product is considered for this project, the insulation layer is not anticipated to be crushed by the compaction of the overlying fill layer using conventional compaction equipment (plate compactors and mid-sized rollers).

The insulation layer should be installed upon a relatively smooth and level subgrade surface. The contractor may consider the use of a maximum 50 mm thick layer of bedding sand or stone dust to provide a level surface to place the insulation layers upon.

Where multiple layers of insulation boards are required, the orientation of the longer insulation board dimension should be alternated between each lift to provide additional rigidity to the overall layer of insulation. Placement of the above-noted insulation layer is recommended to be reviewed at the time of construction by the geotechnical consultant personnel.

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 anticipated 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

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

The pipe bedding for water and pipes placed on a relatively dry, undisturbed subgrade surface should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located upon silty clay the thickness of the bedding material should be increased to a minimum of 300 mm of OPSS Granular A. The bedding layer should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A or Granular B Type II. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 99% 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. Saturated soil fill, such as grey silty clay, will be difficult to re-use as their high water contents make compacting these materials impractical without an extensive drying period.

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, 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, sub-bedding and cover material.

The barriers should consist of relatively dry and compactable brown silty clay placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and 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 and controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 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, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

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

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 is expected to occur thereafter.

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.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

- Review detailed grading plan(s) from a geotechnical perspective.
- Review of insulation details from a geotechnical perspective.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling materials.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

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 immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Access Property Development Inc., Huntington Properties, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
December 13, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

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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
Commercial Development - 60 Denzil Doyle Cres.
Ottawa, Ontario

DATUM TBM - Top of Hydro manhole cover. Geodetic elevation = 101.63m, as provided by DSEL.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** April 14, 2016

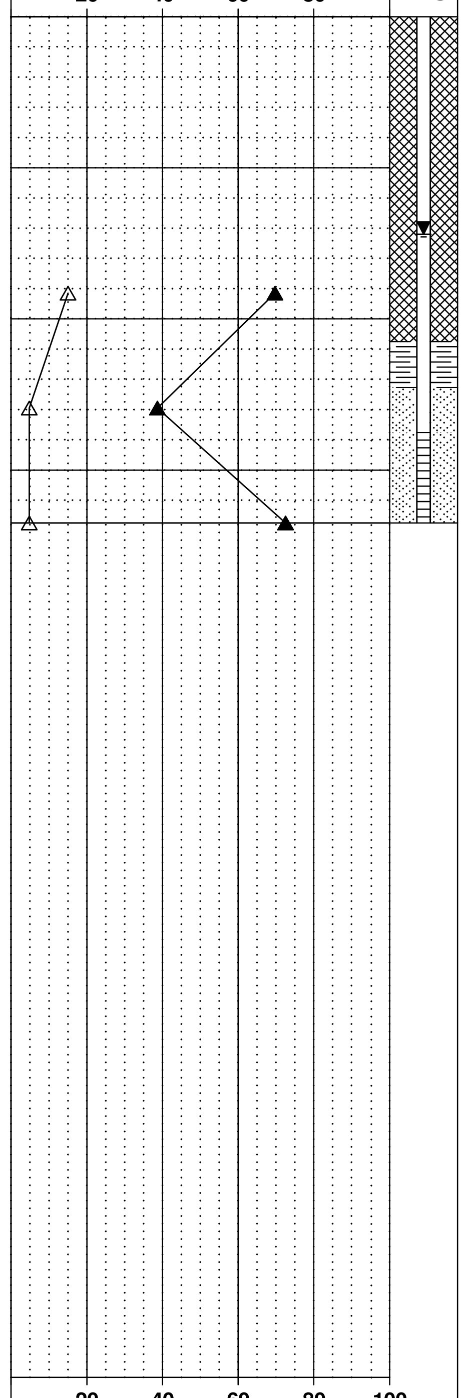
FILE NO. PG3798

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
FILL: Brown silty sand with blast rock 0.25		AU	1			0	102.62
TOPSOIL 0.41							
Stiff, brown **SILTY CLAY,** trace sand		SS	2	100	5	1	101.62
- firm to stiff and grey by 2.3m depth						2	100.62
3.35						3	99.62
End of Borehole							
(GWL @ 1.44m-April 21, 2016)							

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

Piezometer Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 60 Denzil Doyle Cres.
Ottawa, Ontario

DATUM TBM - Top of Hydro manhole cover. Geodetic elevation = 101.63m, as provided by DSEL.

REMARKS

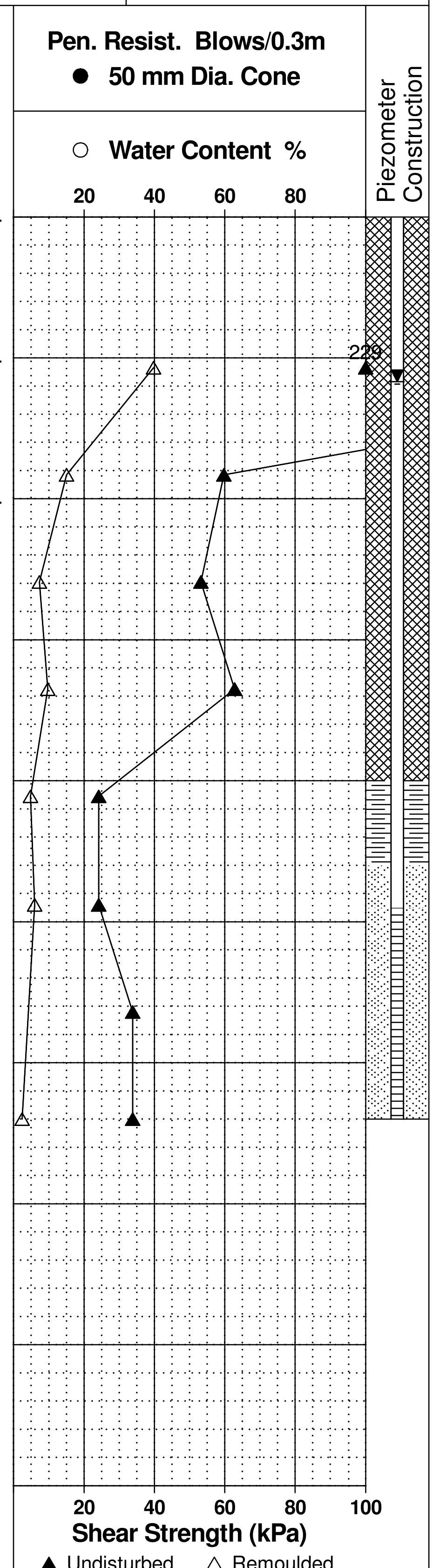
BORINGS BY CME 55 Power Auger

DATE April 14, 2016

FILE NO. PG3798

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL 0.30		AU	1			0	102.04
Hard to very stiff, brown **SILTY CLAY**						1	101.04
- stiff to firm by 1.5m depth						2	100.04
						3	99.04
- soft to firm and grey by 3.8m depth						4	98.04
						5	97.04
6.40						6	96.04
Dynamic Cone Penetration Test commenced at 6.40m depth. Cone pushed to 13.4m depth.						7	95.04
						8	94.04
						9	93.04



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 60 Denzil Doyle Cres.
Ottawa, Ontario

DATUM TBM - Top of Hydro manhole cover. Geodetic elevation = 101.63m, as provided by DSEL.

REMARKS

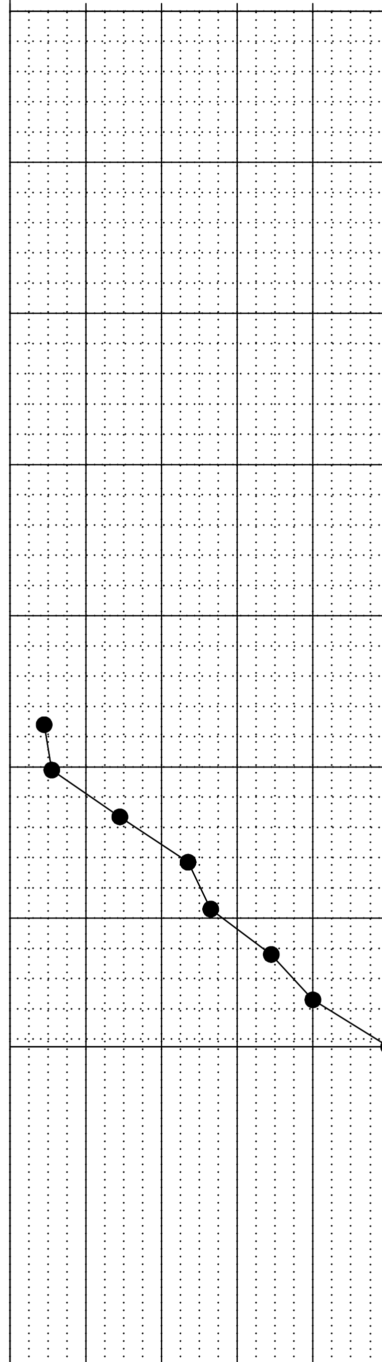
BORINGS BY CME 55 Power Auger

DATE April 14, 2016

FILE NO. PG3798

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE								20 40 60 80	
						9	93.04		
						10	92.04		
						11	91.04		
						12	90.04		
						13	89.04		
						14	88.04		
						15	87.04		
15.85									
End of Borehole Practical DCPT refusal at 15.85m depth (GWL @ 1.17m-April 21, 2016)									



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 60 Denzil Doyle Cres.
Ottawa, Ontario

DATUM TBM - Top of Hydro manhole cover. Geodetic elevation = 101.63m, as provided by DSEL.

REMARKS

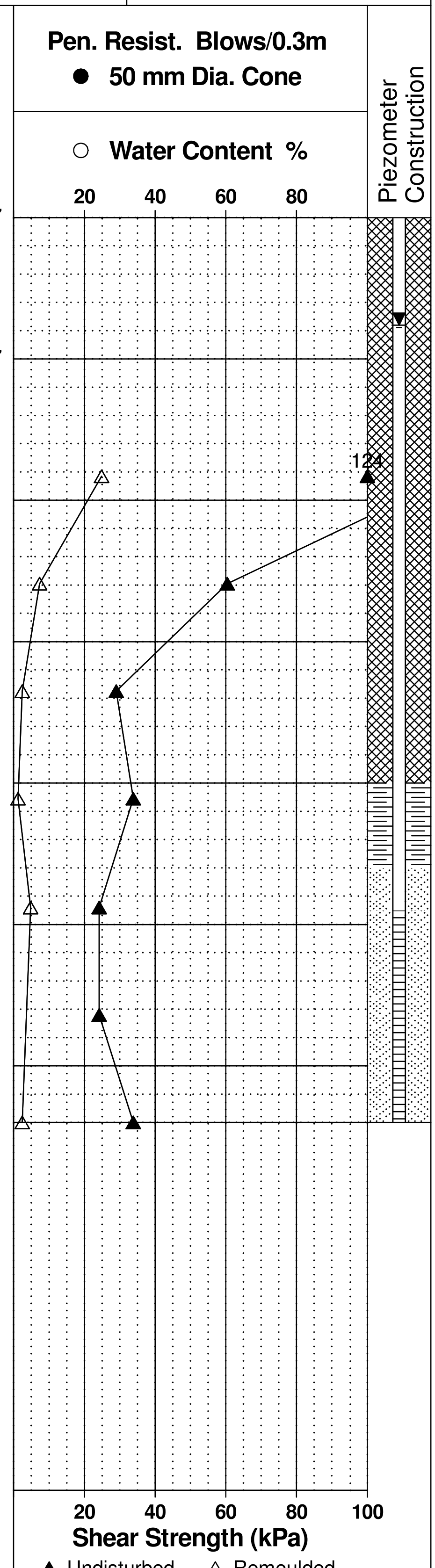
BORINGS BY CME 55 Power Auger

DATE April 14, 2016

FILE NO. PG3798

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	101.47
TOPSOIL 0.28		AU	1				
Very stiff to stiff, brown **SILTY CLAY**		SS	2	100	7	1	100.47
- firm and grey by 3.0m depth						2	99.47
						3	98.47
						4	97.47
						5	96.47
6.40						6	95.47
End of Borehole							
(GWL @ 0.76m-April 21, 2016)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 60 Denzil Doyle Cres.
Ottawa, Ontario

DATUM TBM - Top of Hydro manhole cover. Geodetic elevation = 101.63m, as provided by DSEL.

REMARKS

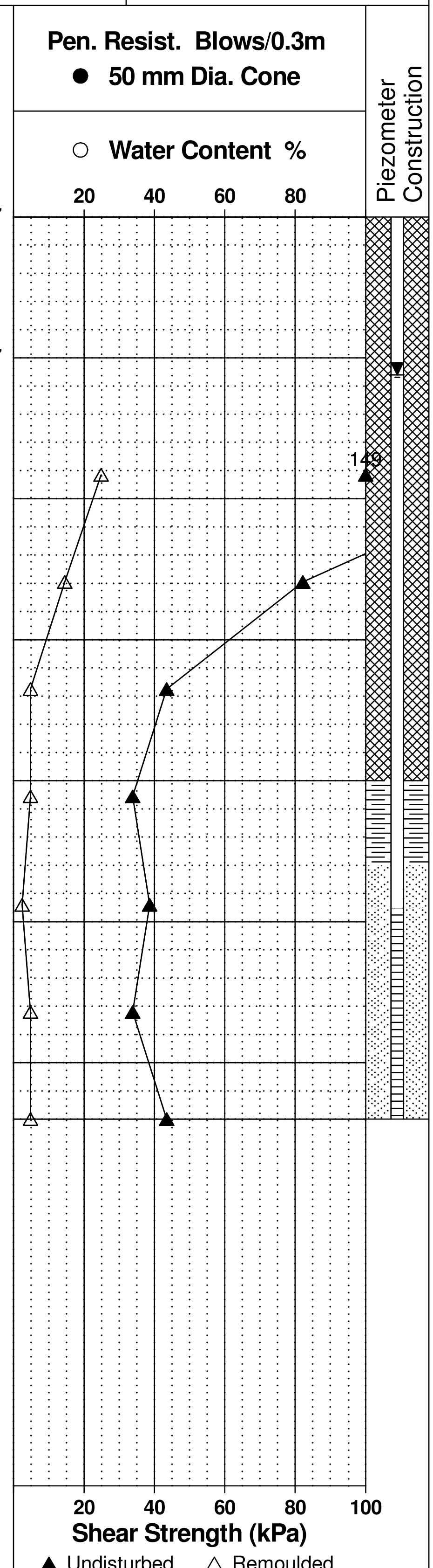
BORINGS BY CME 55 Power Auger

DATE April 14, 2016

FILE NO. PG3798

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	101.57
TOPSOIL 0.25		AU	1				
		SS	2	92	11	1	100.57
Very stiff to stiff, brown **SILTY CLAY**						2	99.57
- firm and grey by 3.0m depth						3	98.57
						4	97.57
						5	96.57
6.40						6	95.57
End of Borehole							
(GWL @ 1.12m-April 21, 2016)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 60 Denzil Doyle Cres.
Ottawa, Ontario

DATUM TBM - Top of Hydro manhole cover. Geodetic elevation = 101.63m, as provided by DSEL.

REMARKS

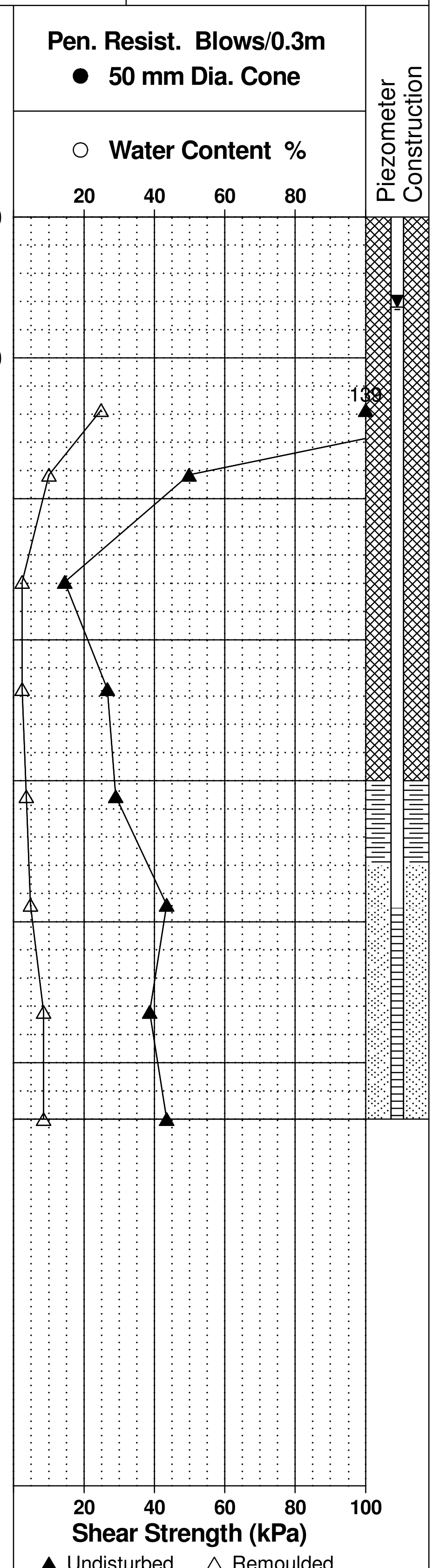
BORINGS BY CME 55 Power Auger

DATE April 14, 2016

FILE NO. PG3798

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL — 0.23						0	101.90
Brown **SILTY CLAY,** some clay — 0.76		AU	1				
Very stiff to stiff, brown **SILTY CLAY,** some sand						1	100.90
						2	99.90
- soft to firm and grey 2.3m depth							
						3	98.90
						4	97.90
						5	96.90
— 6.40						6	95.90
End of Borehole							
(GWL @ 0.64m-April 21, 2016)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - 60 Denzil Doyle Cres.
Ottawa, Ontario

DATUM TBM - Top of Hydro manhole cover. Geodetic elevation = 101.63m, as provided by DSEL.

REMARKS

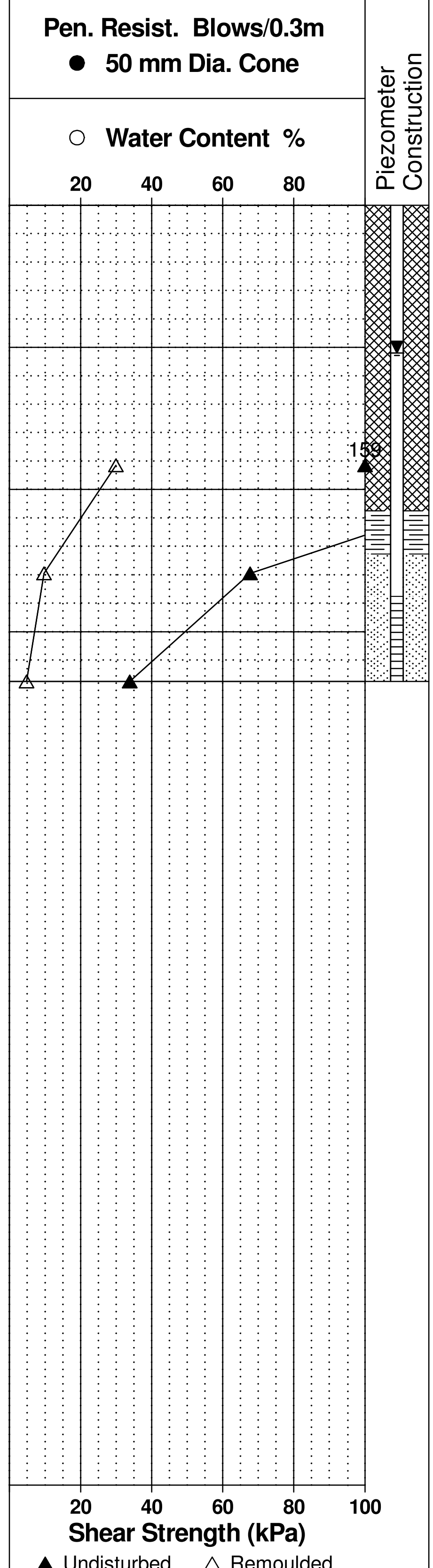
BORINGS BY CME 55 Power Auger

DATE April 14, 2016

FILE NO. PG3798

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL 0.30		AU	1			0	101.57
Very stiff to stiff, brown **SILTY CLAY,** some sand		SS	2	83	6	1	100.57
						2	99.57
- firm and grey by 3.0m depth 3.35						3	98.57
End of Borehole							
(GWL @ 1.04m-April 21, 2016)							



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

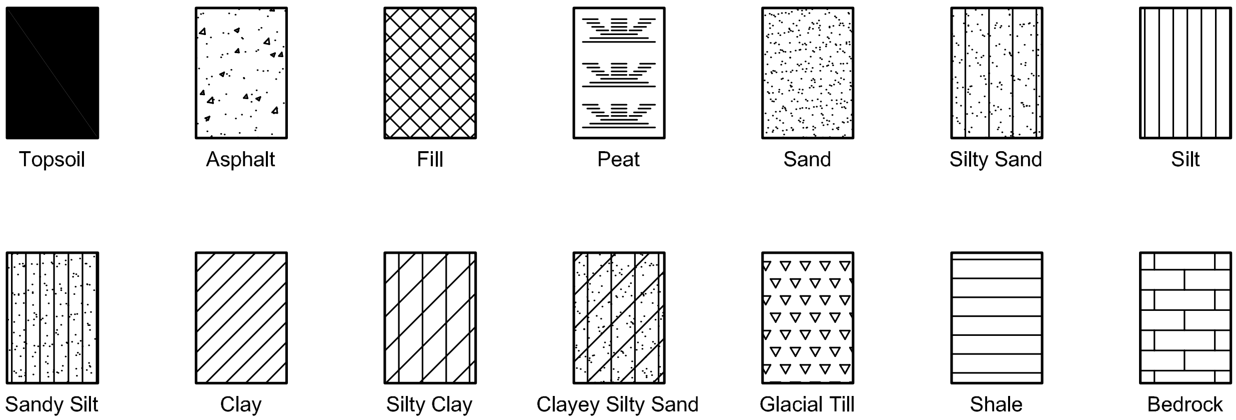
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

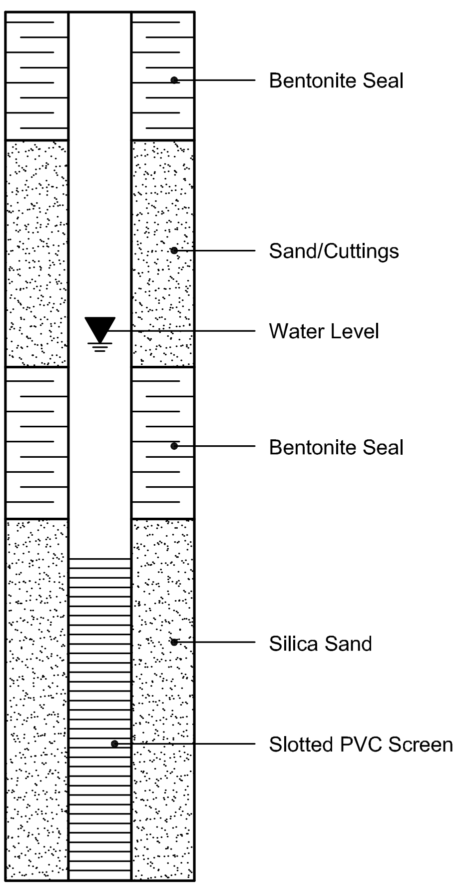
SYMBOLS AND TERMS (continued)

STRATA PLOT

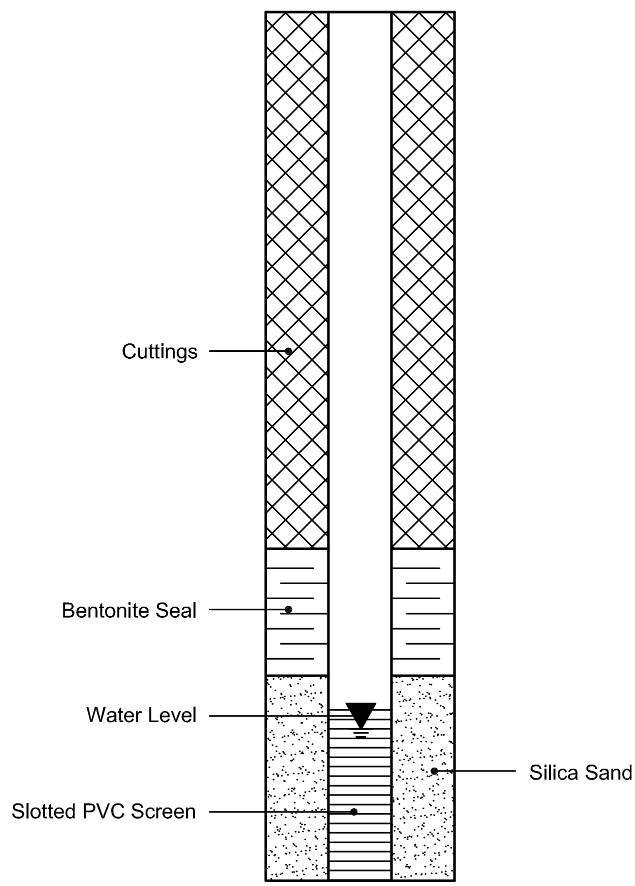


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 19243

Report Date: 20-Apr-2016

Order Date: 15-Apr-2016

Project Description: PG3798

	Client ID:	BH6 SS2	-	-	-
	Sample Date:	14-Apr-16	-	-	-
	Sample ID:	1616468-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.33	-	-	-
Resistivity	0.10 Ohm.m	49.0	-	-	-
Anions					
Chloride	5 ug/g dry	53	-	-	-
Sulphate	5 ug/g dry	22	-	-	-

APPENDIX 2

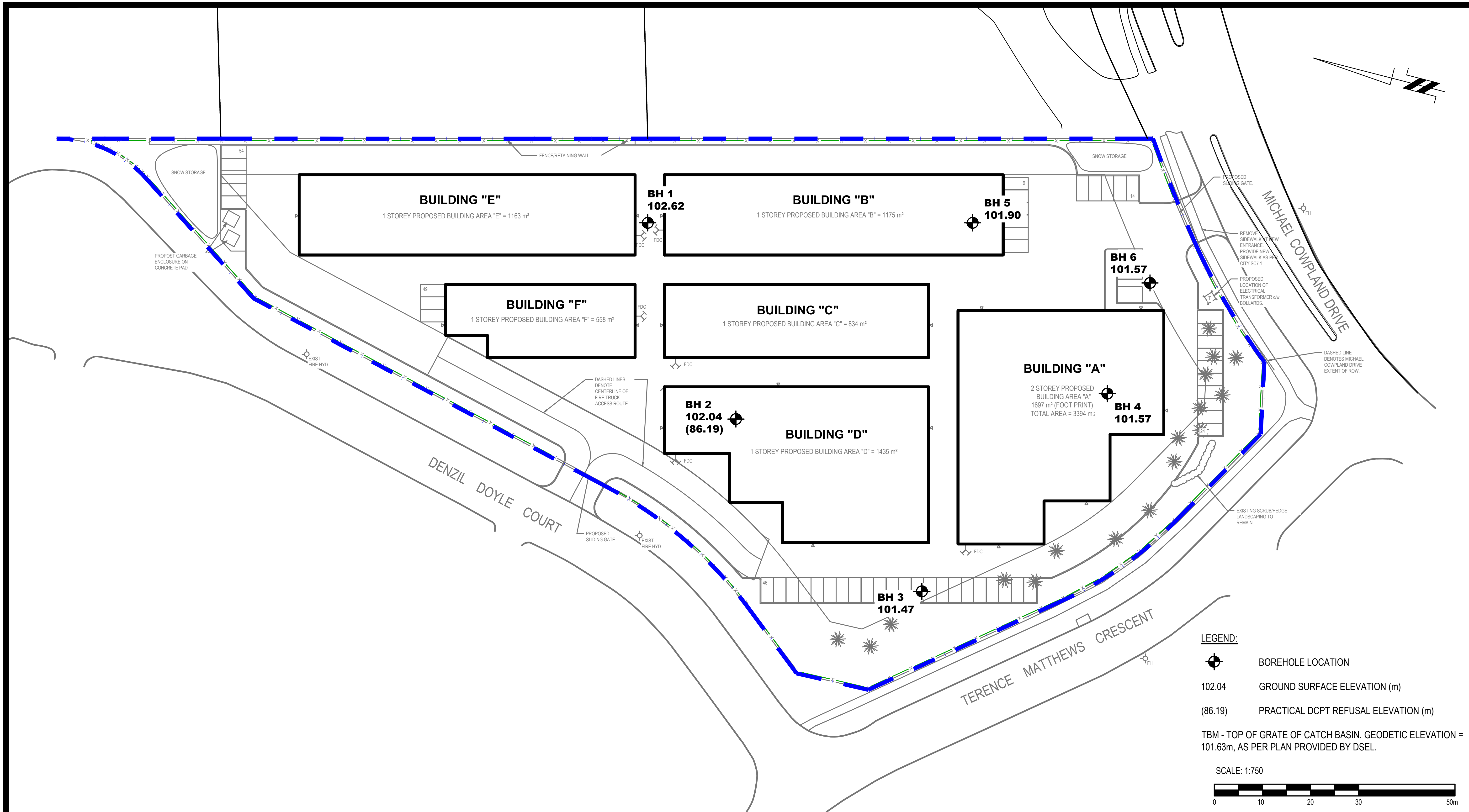
FIGURE 1 – KEY PLAN

DRAWING PG3798-2 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



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

NO.	REVISIONS	DATE	INITIAL
4	AS PER REVISED CLIENT	23/11/2022	DP
3	AS PER REVISED CONCEPTUAL SITE PLAN	23/11/2022	DP
2	AS PER REVISED CONCEPTUAL SITE PLAN	18/11/2022	DP
1	AS PER REVISED CONCEPTUAL SITE PLAN	15/07/2021	FC

HUNTINGTON CONSTRUCTION & DEVELOPMENT & ACCESS PROPERTY DEVELOPMENT INC.

GEOTECHNICAL INVESTIGATION
PROP. COMMERCIAL DEVELOPMENT - 60 DENZIL DOYLE CRES.

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 07/2021
Drawn by: NFRV	Report No.: PG3798-2
Checked by: FC	Dwg. No.: PG3798-1
Approved by: DG	Revision No.: 4

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