

Geotechnical
Engineering

Environmental
Engineering

Hydrogeology

Geological
Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation

Proposed Commercial Development
6150 Hazeldean Road
Ottawa, Ontario

Prepared For

Capital Commercial

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Report: PG4549-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Capital Commercial to conduct a geotechnical investigation for the proposed commercial development to be located at 6150 Hazeldean Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were:

- ☐ to determine the subsurface soil and groundwater conditions by means of boreholes,
- ☐ to 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.

2.0 Proposed Development

It is understood that the proposed development will consist of 2 commercial buildings:

- ☐ a single-storey restaurant located in the northwest corner of the site
- ☐ a 2-storey medical/office building located in the northeast corner of the site

Both buildings will have footprints of approximately 460 m² and will consist of slab-on-grade construction. Asphalt-paved drive aisles and parking areas with landscaped margins will surround the proposed buildings.

A future Phase 2 development is also expected on the southern half of the site, however, specific details regarding this phase are not known at this time.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

A previous geotechnical investigation included 1 borehole (BH5-15) completed at the subject site on December 14, 2015, which was advanced to a depth of 7.3 m. The location of the test hole is shown on Drawing PG4549-1 - Test Hole Location Plan included in Appendix 2.

The borehole was advanced using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the auger flights and a 50 mm diameter split-spoon sampler. The soil from the auger flights and split-spoon samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the auger flight and split-spoon samples were recovered from the boreholes are depicted as AU and SS, respectively, on the Soil Profile and Test Data sheets 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.

Rock samples were recovered from borehole BH5-15 using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the borehole are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value was calculated for each drilled section (core run) of bedrock and are shown on the borehole log. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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 in Appendix 1.

Groundwater

A flexible PVC standpipe was installed in the borehole to permit monitoring of the groundwater level subsequent to the completion of the sampling program.

3.2 Field Survey

The ground surface elevation at the borehole location was referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located on the north side of Neil Avenue. A geodetic elevation of 114.69 m was assigned to the TBM. The location of the borehole and the ground surface elevation at the borehole location are presented on Drawing PG4549-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One soil sample was obtained during the above-noted geotechnical investigation and submitted for analytical purposes. This sample was obtained from a borehole located approximately 100 m to the east of the subject site, and 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 also submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the soil. The analytical test results are presented in Appendix 1 and discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant and is bordered by Hazeldean Road to the north, vacant asphalt-paved lots to the east and west, and Neil Avenue to the south. The ground surface across the site is generally asphalt-paved and is relatively level at approximate geodetic elevation 118 to 119 m.

4.2 Subsurface Profile

Overburden

The subsurface profile encountered at the borehole location consisted of an approximate 1.8 m thickness of fill underlying the ground surface which consisted of a compact, brown silty sand with gravel and boulders. Although not observed within borehole BH5-15 completed at the subject site, an organic deposit was encountered underlying the fill at certain test hole locations at the adjacent 6130 Hazeldean Road site to the east.

Underlying the fill, a glacial till was encountered which consisted of boulders with some silt, sand, and gravel. The glacial till deposit was observed to extend to the bedrock surface at an approximate depth of 3 m.

Bedrock

A poor to fair quality grey limestone with interbedded shale partings was cored at borehole BH5-15 to a depth of 7.3 m.

Based on available geological mapping, the bedrock in this area consists of limestone of the Bobcaygeon formation with an overburden drift thickness of 1 to 5 m.

4.3 Groundwater

The groundwater level was measured in the standpipe at borehole BH5-15 at a depth of 4.5 m on December 18, 2015, which corresponds to geodetic elevation 114.6 m. It should be noted that surface water can become trapped within the backfilled borehole, which can lead to higher than normal groundwater level readings. The long term groundwater level can also be estimated based on the recovered soil sample's moisture level and consistency. Based on these observations, the long term groundwater table is anticipated to be at a geodetic elevation of 113 to 114.5 m. It should be further noted that the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed development. It is expected that the proposed commercial buildings will be founded by conventional shallow footings placed on the undisturbed glacial till.

Where fill is encountered at the underside of footing, the fill should be sub-excavated to the surface of the undisturbed glacial till and replaced with engineered fill or lean concrete to the proposed founding elevation. The lateral limits of the engineered fill placement should be in accordance with our lateral support recommendations provided in Subsection 5.3. The excavation for the lean concrete trenches should be near vertical and extending at least 150 mm beyond the outside face of the footing.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. It is anticipated that the existing granular fill within the future building footprints, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprints outside of lateral support zones for the footings. However, it is recommended that the existing fill layer be proof-rolled several times and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of 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, the material should be compacted in thin lifts to a minimum density of 95% of the respective SPMDD.

Backfill against foundation walls should consist of free-draining, non frost susceptible granular materials. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against the foundation walls, unless used in conjunction with a composite drainage blanket connected to the perimeter foundation drainage system.

Lean Concrete Filled Trenches

As an alternative to placing engineered fill, where required, consideration should be given to excavating vertical trenches to the undisturbed glacial till and backfilling 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 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 glacial till. 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. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed glacial till bearing surface, or on engineered fill or lean concrete placed directly over the undisturbed glacial till 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**. A geotechnical factor of 0.5 was incorporated to the bearing resistance value at ULS.

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

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

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 undisturbed glacial till 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.

5.4 Design for Earthquakes

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

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Slab on Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the existing suitable fill subgrade approved by the geotechnical consultant at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade construction. A vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as Granular B Type II.

It is recommended that the upper 200 mm of sub-floor fill consist of OPSS Granular A crushed stone. All backfill materials required to raise grade within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Pavement Structure

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

It is anticipated that the proposed pavement structures will be placed over either the existing fill, or engineered fill placed over the existing fill.

Table 1 - Recommended Pavement Structure - Car Only Parking Areas	
Thickness (mm)	Material Description
50	Wear Course - Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
400	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil	

Table 2 - Recommended Pavement Structure Heavy Truck Parking Areas and Access Lanes	
Thickness (mm)	Material Description
40	Wear Course - Superpave 12.5 Asphaltic Concrete
50	Binder Course - Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structure as an outlet for perched water below the sidewalks anticipated to surround the proposed buildings. Perched water below the sidewalks can lead to heaved sidewalks due to freeze/thaw cycles. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 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.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. 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 used in conjunction with a composite drainage blanket, such as Miradrain G100N or Delta Drain 6000.

6.2 Protection of Footings Against Frost Action

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

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for exterior unheated footings, or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

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

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

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

Based on the shallow nature of the proposed excavation, it's expected that groundwater infiltration volumes should be less than 50,000 L/day. However, if deeper excavations are being proposed, 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 MOECC review of the PTTW application.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters 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.

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and granular fill materials used.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test hole locations, we request immediate notification to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Capital Commercial 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.



Scott S. Dennis, P.E.



Carlos P. Da Silva, P.Eng., ing., QP_{ESA}



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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - 6176 Hazeldean Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the north side of Neil Avenue, near the southeast corner of subject property. Geodetic elevation = 114.69m.

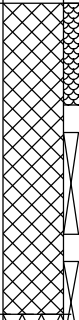


REMARKS

FILE NO.
PG3712

HOLE NO.
BH 5-15

BORINGS BY CME 55 Power Auger

DATE December 14, 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %					
								20	40	60	80		
GROUND SURFACE						0	119.14						
FILL: Brown silty sand with gravel, trace boulders		AU	1										
		SS	2	21	18	1	118.14						
		SS	3	53	14	2	117.14						
GLACIAL TILL: Boulders with some silt, sand and gravel		RC	1	65									
						3	116.14						
BEDROCK: Grey limestone with shale partings and seams		RC	2	87	35	4	115.14						
						5	114.14						
		RC	3	100	44	6	113.14						
						7	112.14						
		RC	4	98	52								
End of Borehole													
(GWL @ 4.51m-Dec. 18, 2015)													
								20	40	60	80	100	
								Shear Strength (kPa)					
								▲ Undisturbed △ Remoulded					

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = D_{60} / D_{10}

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		Overconsolidation 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.
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SYMBOLS AND TERMS (continued)

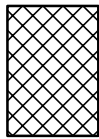
STRATA PLOT



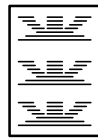
Topsoil



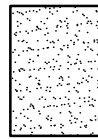
Asphalt



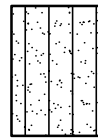
Fill



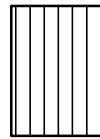
Peat



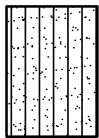
Sand



Silty Sand



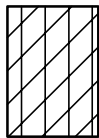
Silt



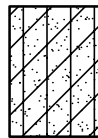
Sandy Silt



Clay



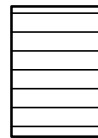
Silty Clay



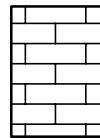
Clayey Silty Sand



Glacial Till



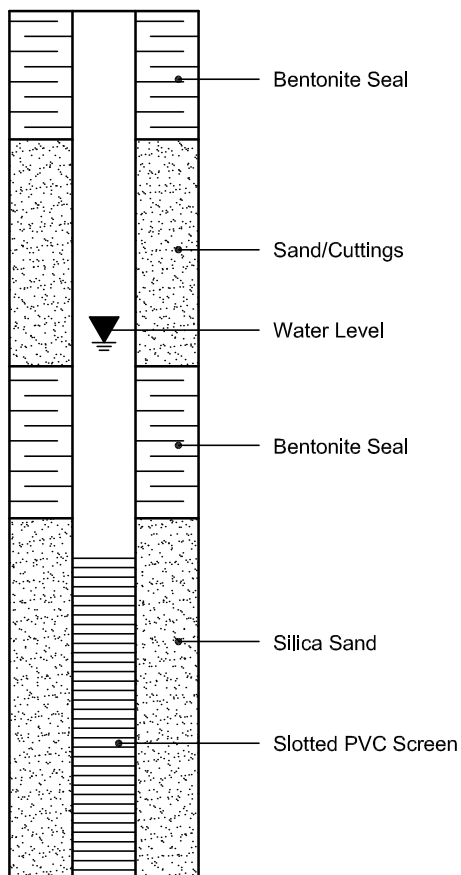
Shale



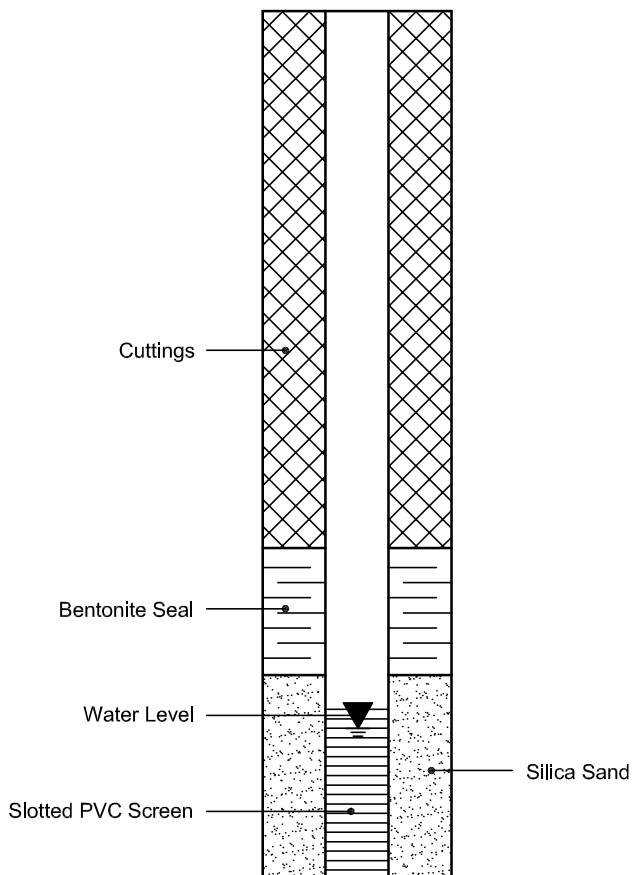
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: **18945**

Report Date: 21-Dec-2015

Order Date: 15-Dec-2015

Project Description: PG3712

Client ID:	BH2 SS4	-	-	-
Sample Date:	14-Dec-15	-	-	-
Sample ID:	1551144-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	89.9	-	-	-
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General Inorganics

pH	0.05 pH Units	7.63	-	-	-
Resistivity	0.10 Ohm.m	35.4	-	-	-

Anions

Chloride	5 ug/g dry	12	-	-	-
Sulphate	5 ug/g dry	8	-	-	-

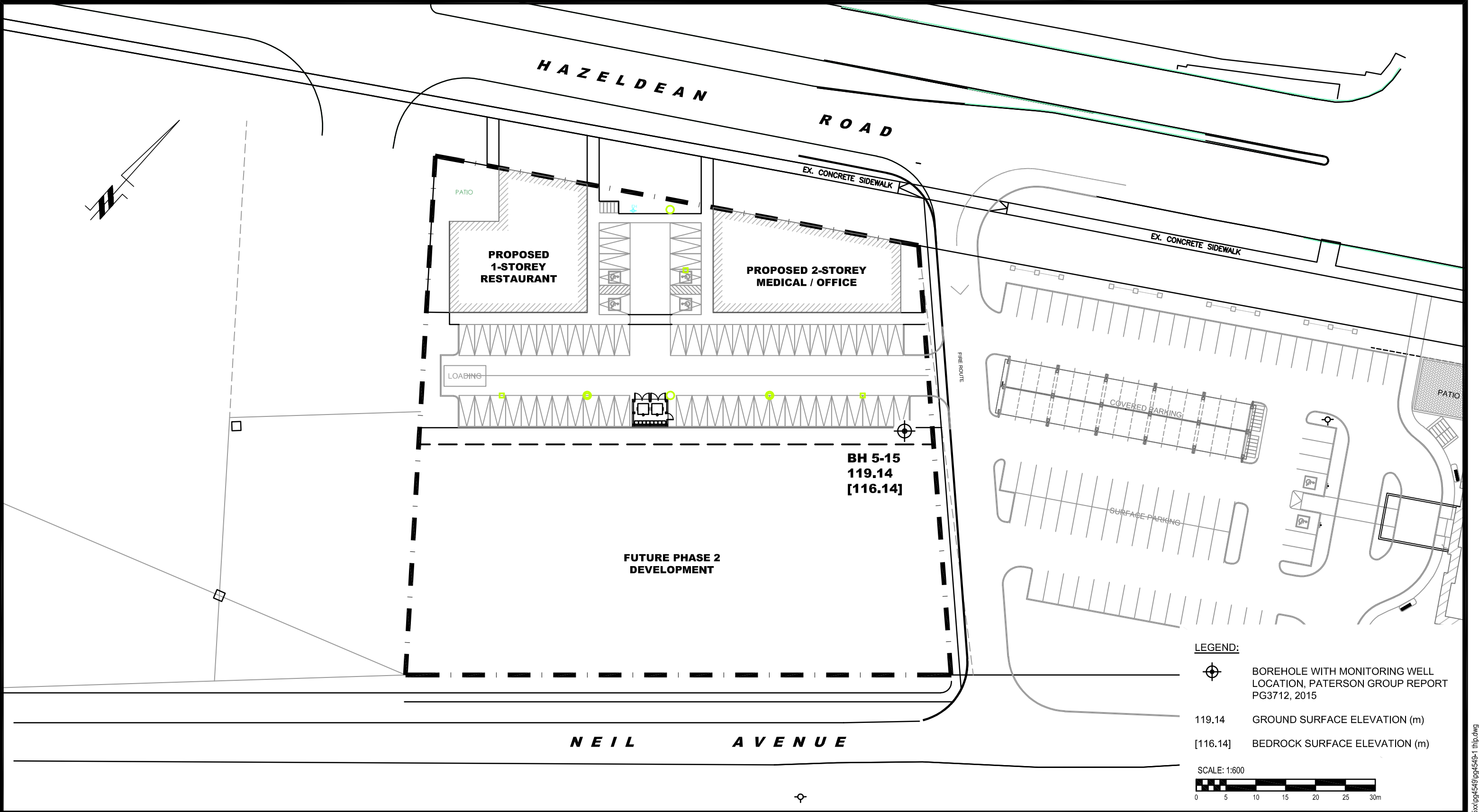
APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4549-1 - TEST HOLE LOCATION PLAN



FIGURE 1
KEY PLAN



patersongroup
consulting engineers

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

0			
NO.	REVISIONS	DATE	INITIAL

CAPITAL COMMERCIAL
GEOTECHNICAL INVESTIGATION
PROP. COMMERCIAL DEVELOPMENT - 6150 HAZELDEAN ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale:	1:600	Date:	05/2018
Drawn by:	MPG	Report No.:	PG4549-1
Checked by:	DS	Dwg. No.:	PG4549-1
Approved by:	DS	Revision No.:	0