patersongroup

Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Geotechnical Investigation

Proposed Multi-Storey Building
288 Booth Street
Ottawa, Ontario

Prepared For

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Report: PG2403-1

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APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Mr. Peter Evans to conduct a geotechnical investigation for a proposed multi-storey building to be located at 288 Booth Street, 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.
- ☐ Provide geotechnical recommendations for the 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. The report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

A Phase I - Environmental Site Assessment (ESA) was conducted by Paterson for the subject site. The results and recommendations of the Phase I - ESA are presented under separate cover.

2.0 PROPOSED PROJECT

It is understood that the proposed project consists of a six (6) storey building (commercial/residential), along with associated at-grade parking areas and access lanes.

The subject site is located at the northwest corner of Somerset Street and Booth Street.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The field program for the investigation was conducted on July 8, 2011. At that time, four (4) boreholes were advanced to a maximum depth of 6.9 m The borehole locations were distributed in a manner to provide general coverage of the proposed development with consideration to site features and underground services. The approximate locations of the boreholes are shown on Drawing PG2403-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down 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 using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, 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.

Diamond drilling was completed at one (1) location to confirm the depth to bedrock and quality of bedrock. 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 of the length of the bedrock sample recovered over the length of the drilled section, in percentage. The RQD value is the ratio of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

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

Groundwater

One (1) 19 mm PVC groundwater monitoring well was installed in BH 3 and three (3) flexible PVC standpipes were installed in the remaining boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations were determined by Paterson personnel with consideration of underground and aboveground services. The location and ground surface elevation at each borehole location was surveyed by Paterson personnel. The boreholes were surveyed with respect to a temporary benchmark (TBM), consisting of the top of a catch basin located along Booth Street, approximately 15 m North of Somerset Street. An geodetic elevation of 73.44 m was provided for the TBM. Borehole locations and ground surface elevations, as well as the TBM location are presented on Drawing PG2403-1 - 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 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. The sample was 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

Currently, the subject site consists of a gravel parking lot surrounded by low rise commercial and residential properties along the north and west property boundaries. The parking lot is relatively flat, at grade with nearby roadways and it is mostly gravel covered with some asphalt covered areas.

4.2 Subsurface Profile

The subsurface profile at the borehole locations consists of a either a pavement structure or crushed stone granular fill material overlying silty sand fill with some gravel and/or glacial till, consisting of a brown silty sand matrix with gravel, cobbles and boulders. Practical refusal to augering was encountered at all the borehole locations. Limestone bedrock was cored at BH 3. Based on RQD values, the bedrock quality consists of fair to good quality. Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and shale from the Veralum Formation. The overburden drift thickness ranges between ground surface to 10 m depth.

4.3 Groundwater

The groundwater levels were measured in the monitoring well and standpipes on July 13, 2011 and August 11, 2011, respectively. The results are presented in Table 1. The groundwater level fluctuates seasonally and could vary at the time of construction.

Table 1 - Measured Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Water Levels	
		Depth (m)	Elevation (m)
BH 1	74.05	2.94	71.11
BH 2	73.32	2.32	71.00
BH 3	73.46	2.51	70.95
BH 4	73.59	2.23	71.36

5.0 DISCUSSION

5.1 Geotechnical Assessment

The subject site is considered adequate from a geotechnical perspective for the proposed building.

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

5.2 Site Grading and Preparation

Stripping Depth

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

The existing fill could be assessed at the time of construction by the geotechnical consultant, once the footing excavations are completed and a large area of the fill is exposed. If found satisfactory, the fill located outside the zones of influence of the footings could be left in place.

Existing foundation walls, and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. 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).

5.3 Foundation Design

Bearing Resistance Values

Footings placed over an undisturbed, compact glacial till or engineered fill can be designed using a bearing resistance value at Serviceability Limit States (SLS) of **150 kPa** and a factored bearing resistance value at Ultimate Limit States (ULS) of **225 kPa**, incorporating a geotechnical resistance factor of 0.5. Engineered fill should consist of materials, which are suitable for use below the proposed building footprint as described in Subsection 5.2.

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

The bearing resistance value given for footings at SLS will be subjected to potential post construction total and differential settlements of 25 and 15 mm, respectively.

Footings placed over a clean surface sound limestone bedrock surface can be designed using a factored bearing resistance value at ULS of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance at SLS of **500 kPa**.

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 geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

A factored bearing resistance value at ULS of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance at SLS of **1,500 kPa** 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 conducted by the geotechnical consultant.

Footings bearing on surface sounded bedrock and designed using the above mentioned bearing pressures will be subjected to negligible post-construction total and differential settlements.

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. A higher site class, such as Class A or B, may be applicable. However, the higher site class has to be confirmed by site specific shear wave velocity testing. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil, and fill, containing organic matter within the footprint of the proposed buildings, the native soil surface or engineered fill will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction. Provision should be made for proof-rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any fill. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone.

In consideration of the groundwater conditions encountered at the time of the construction, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, may need to be provided in the clear stone under the basement floor.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the proposed retaining walls and basement walls. However, provided free-draining granular backfill is used, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m^3. The soils against the foundation are anticipated to be drained. An interface friction angle of 17 degrees between the wall and the backfill material is applicable for the abovenoted parameters. For undrained conditions, the effective unit weight of soil (10 kN/m^3) should be used to calculate the earth pressure component below the groundwater table, and hydrostatic pressure should be added within this portion to calculate the total static earth pressure.

The earth pressures acting on earth retaining structures are dependent on the characteristics of the structure, particularly with respect to whether the earth retaining structure is a “yielding” or an “unyielding” structure. A basement wall, which is restrained laterally by the floors of the structure, is generally considered to be an unyielding structure. The at-rest earth pressure case is recommended to be used for basement walls under static conditions.

During an earthquake event, a basement wall is considered to be a “yielding” earth retaining structure, due to the magnitude of wall rotation. Therefore, an active earth pressure should be calculated for seismic design considerations.

Two (2) distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two (2) conditions are presented below.

Static Earth Pressures

Under static conditions, the retaining walls and basement walls may be designed using a triangular earth pressure distribution with a maximum stress value at the base of the wall equal to $K_o \gamma H$ where:

K_o - At-rest earth pressure coefficient = 0.5
γ - unit weight of the fill = 20 kN/m^3
H - height of the retained fill against the wall, m

An additional pressure having a magnitude equal to $K_o q$ and acting on the entire height of the wall must be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to stay at least 0.3 m away from the walls with the compaction equipment.

Seismic Earth Pressures

Seismic loading conditions influence the earth pressures that will act on earth retaining structures during seismic events. In Ottawa, the peak ground acceleration (PGA) is 0.42 for the OBC 2006.

The magnitude of seismic earth pressures acting on a structure is dependent upon the relative flexibility of the structure. Isolated free-standing retaining walls are generally flexible enough to be considered as “yielding” earth retaining structures.

The total active earth force acting on a wall under seismic conditions can be estimated using a pseudo-static approach based on the Mononobe-Okabe (M-O) Method. The seismic intensity is represented by the horizontal seismic coefficient, k_h. For yielding structures, the value of k_h can be taken to be one half of PGA. Note that the vertical seismic coefficient is taken to be zero.

The M-O Method is used to calculate the total active earth pressure (P_{AE}). The resulting force is then split into the static (active) (P_A) and seismic component (ΔP_{AE}). The total active earth pressure (P_{AE}) can be calculated using $0.5K_{AE}\,\gamma H^2$ where:

K_{AE} - Dynamic active earth pressure coefficient. For the conditions previously stated, K_{AE} is 0.21.
γ - unit weight of the fill of the applicable retained soil (kN/m^3)
H - height of the wall (m)

The static component (P_A) can be calculated using $K_A\,\gamma\,H$ where:

K_A = dynamic active earth pressure coefficient, 0.33
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

The dynamic seismic component (ΔP_{AE}) can be calculated by $\Delta P_{AE} = P_{AE} - P_A$.

The static component (P_A) is a conventional triangular shaped pressure distribution with the resultant located H/3 up from the wall base. The seismic component (ΔP_{AE}) is acting approximately 0.6H up from the wall base.

On this basis, the total active pressure (P_{AE}) will act from a height:

$$h = \left\{P_A(H/3)+\Delta P_{AE}(0.6H)\right\} / P_{AE}$$

The earth pressures calculated are unfactored. For the ULS case, the earth pressure loads must be factored as live loads, as per OBC 2006.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada) or Williams Form Engineering, have qualified personnel on staff to recommend appropriate rock anchor size and materials.

The centre to centre spacing between bond lengths should be at least four (4) times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. Anchors in close proximity to each other is recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout fluid does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. A post-tensioned anchor will take the uplift load with less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole is to be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between about 50 and 120 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. A **Rock Mass Rating (RMR) of 44** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.128 and 0.00009**, respectively. For design purposes, we assumed that all rock anchors will be placed at least 1.2 m apart to reduce group anchor effects.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2.

Table 2 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.2 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Fair quality Limestone - Hoek and Brown parameters	44 m=0.128 and s=0.00009
Unconfined compressive strength - Limestone	50 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter holes are provided in Table 3. The factored tensile resistance values given in Table 2 are based on a single anchor with no group influence effects.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.0	1.7	3.7	250
	3.0	2.3	5.3	500
	4.0	2.6	6.6	750
125	1.7	1.8	3.5	250
	2.3	2.6	4.9	500
	3.0	3.1	6.1	750

Other considerations

The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of grout tube to place grout from the bottom up in the anchor holes is recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request.

Due to the intended use of the rock anchors and nature of the passive rock anchor design, proof testing is not required provided the grout installation is adequately completed to the satisfaction of the geotechnical consultant. The compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Design

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill

Table 5 - Recommended Pavement Structure - Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill

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

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

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

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

A perimeter drainage system is recommended for the proposed structure. The system should consist of a 100 to 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. 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 system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for backfill material.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

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

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. Sufficient room should 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.

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

Where neighboring buildings are in close proximity to the proposed building foundation, it is recommended that the geotechnical consultant review the excavation area to determine if underpinning is required.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. 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 (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the SPMDD.

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) should match the soils exposed at the trench walls to reduce the 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 SPMDD.

6.5 Groundwater Control

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.

The rate of flow of groundwater into the excavation through the overburden and bedrock should be low to moderate for the expected subsurface conditions. Pumping from open sumps is anticipated to provide sufficient control for the groundwater influx through the sides of the excavations.

6.6 Winter Construction

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. 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 conducted in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. Precaution must be taken where excavations are in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

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

7.0 RECOMMENDATIONS

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

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

☐ Review the bedrock stabilization and excavation requirements.

☐ Sampling and testing of the concrete and fill materials used.

☐ 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 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 soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request 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 Mr. Peter Evans 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.

Joe Forsyth, EIT.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
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Report Distribution:

- ☐ Mr. Peter Evans (3 copies)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 288 Booth Street
Ottawa, Ontario

DATUM TBM - Top of catch basin in front of subject site, on Booth Street. Geodetic elevation = 73.44m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 8 July 2011

FILE NO. PG2403

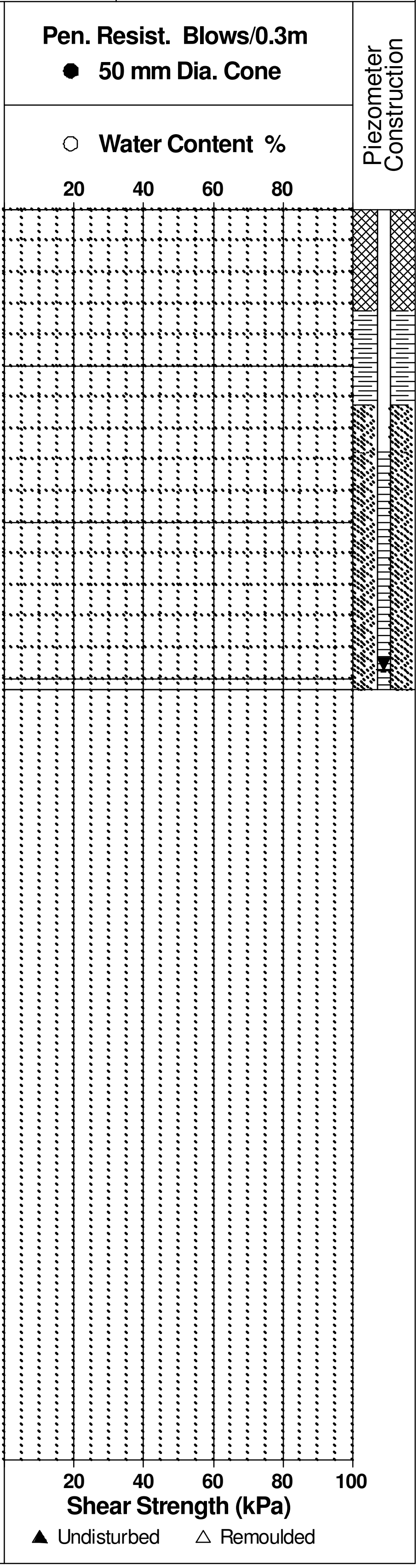
HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	74.05
25mm Asphaltic concrete over crushed stone 0.08	AU	1				
FILL: Brown silty sand with gravel, asphalt and brick 0.69	AU	2				
FILL: Brown silty sand, trace gravel 1.45	SS	3	25	7	1	73.05
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	SS	4	46	50+		
					2	72.05
	SS	5	42	50+		
3.07	SS	6	50	50+	3	71.05

End of Borehole

Practical refusal to augering @ 3.07m depth

(GWL @ 2.94m-Aug. 11/11)



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 288 Booth Street
Ottawa, Ontario

DATUM TBM - Top of catch basin in front of subject site, on Booth Street. Geodetic elevation = 73.44m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 8 July 2011

FILE NO. PG2403

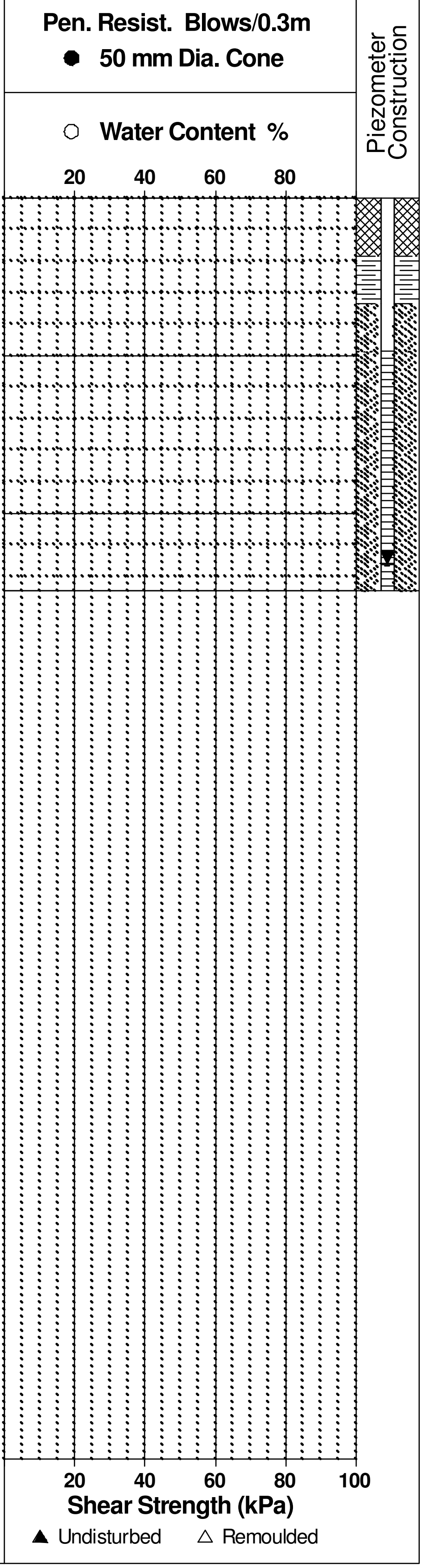
HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	73.32
FILL: Crushed stone 0.38		AU	1				
FILL: Brown silty sand with gravel 0.69		AU	2				
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders		SS	3	38	26	1	72.32
		SS	4	42	11	2	71.32
2.49		SS	5	83	50+		

End of Borehole

Practical refusal to augering @ 2.49m depth

(GWL @ 2.32m-Aug. 11/11)



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 288 Booth Street
Ottawa, Ontario

DATUM TBM - Top of catch basin in front of subject site, on Booth Street. Geodetic elevation = 73.44m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 8 July 2011

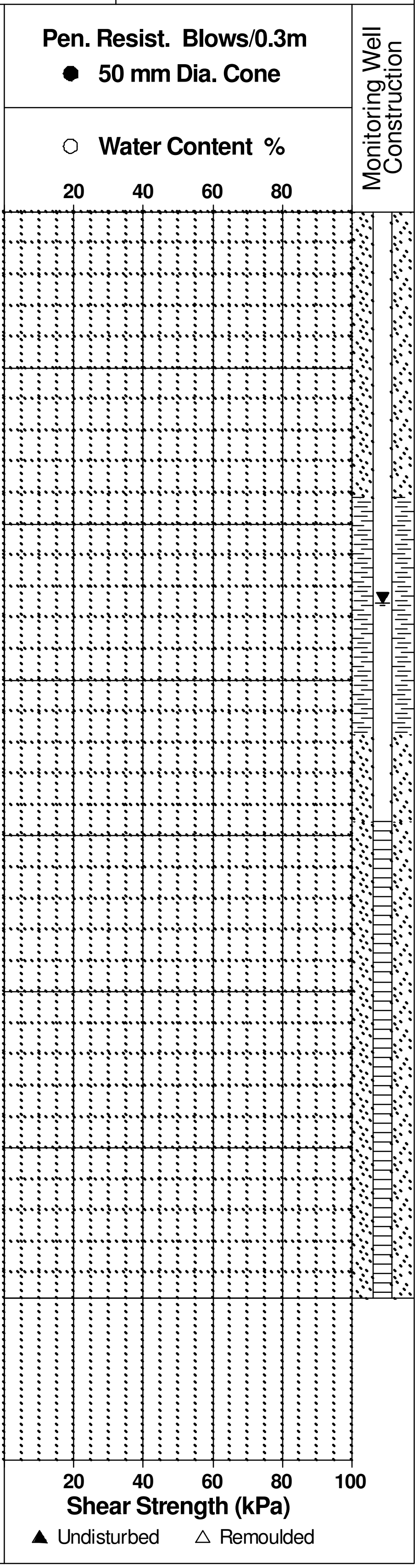
FILE NO. PG2403

HOLE NO. BH 3

SOIL DESCRIPTION	Depth of base (m)
GROUND SURFACE	
FILL: Crushed stone	0.36
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders	2.97
BEDROCK: Grey limestone	6.96
End of Borehole (GWL @ 2.51m-July 13/11)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
AU	2		
SS	3	40	50+
SS	4	50	9
SS	5	42	7
RC	1	95	63
RC	2	100	72
RC	3	100	85

DEPTH (m)	ELEV. (m)
0	73.46
1	72.46
2	71.46
3	70.46
4	69.46
5	68.46
6	67.46



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 288 Booth Street
Ottawa, Ontario

DATUM TBM - Top of catch basin in front of subject site, on Booth Street. Geodetic elevation = 73.44m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 8 July 2011

FILE NO. PG2403

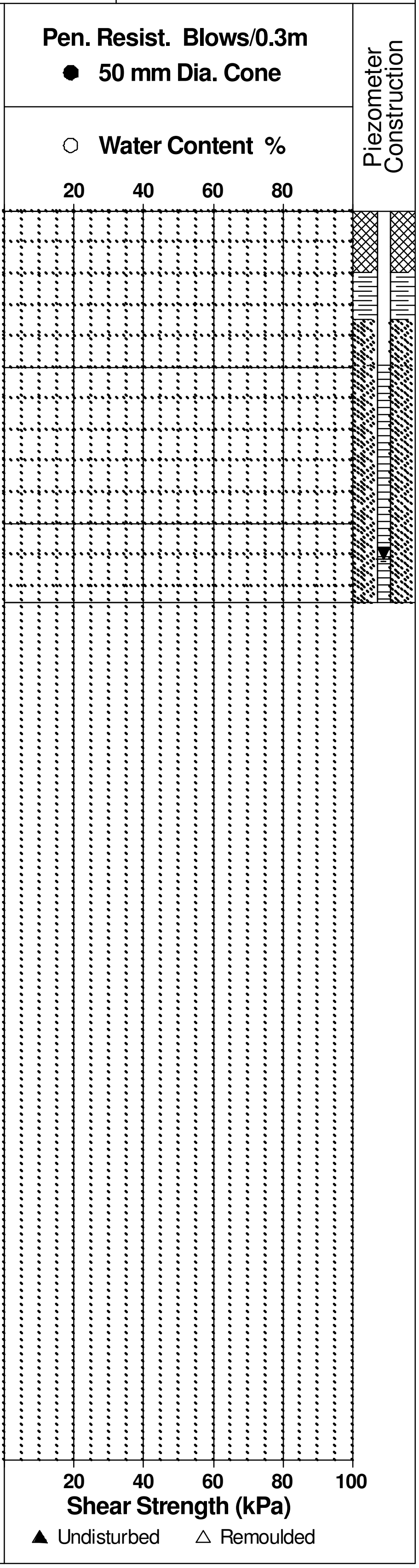
HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	73.59
FILL: Crushed stone 0.43	AU	1				
FILL: Brown silty sand with gravel 0.69	AU	2				
FILL: Brown silty sand 1.45	SS	3	25	10	1	72.59
GLACIAL TILL: Brown silty sand with gravel and cobbles	SS	4	33	6		
					2	71.59
2.51	SS	5	29	50+		

End of Borehole

Practcial refusal to augering @ 2.51m depth

(GWL @ 2.23m-Aug. 11/11)



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

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

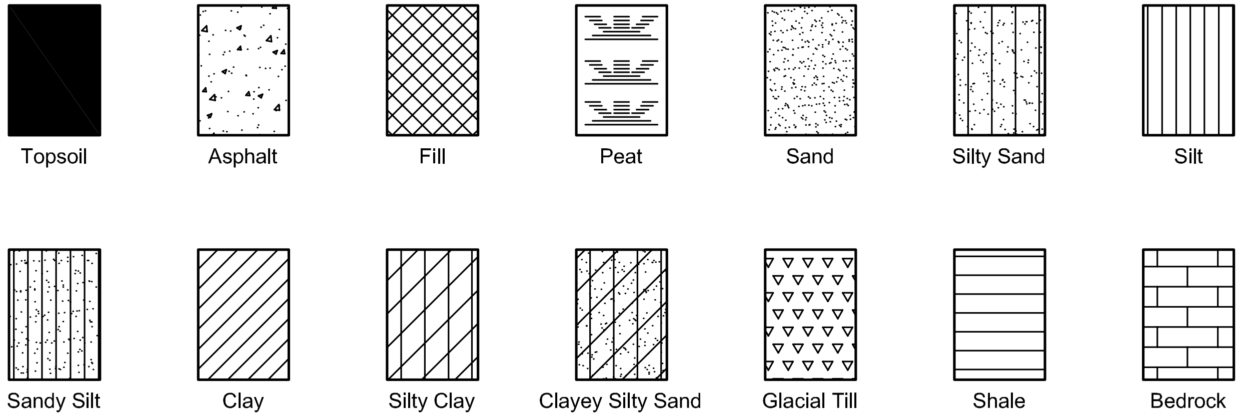
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

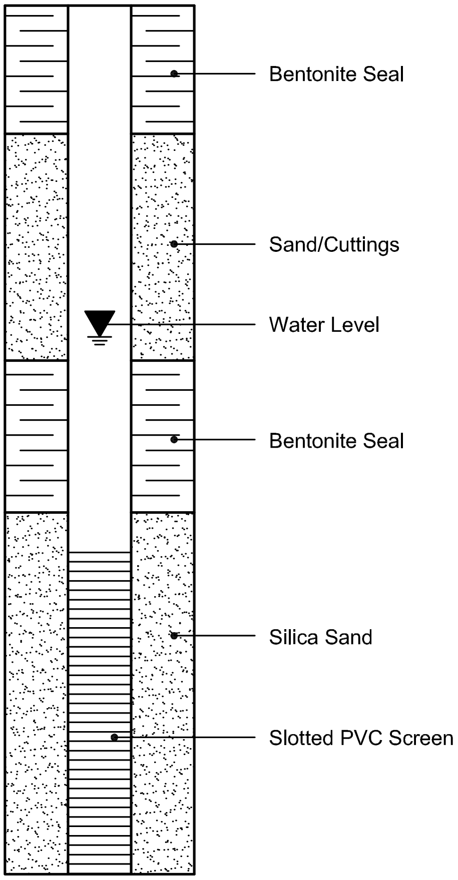
SYMBOLS AND TERMS (continued)

STRATA PLOT

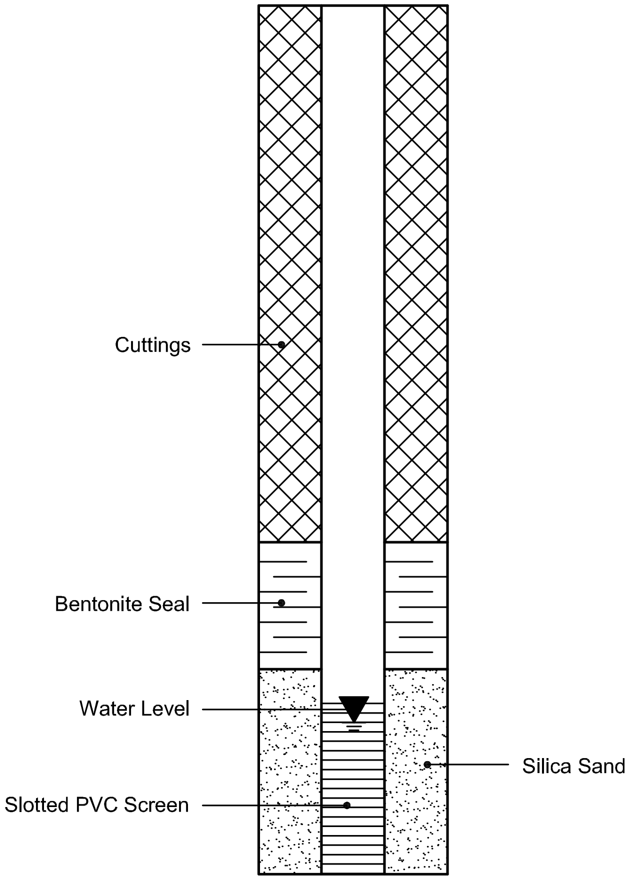


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 14-Jul-2011

Client: **Paterson Group Consulting Engineers**

Order Date:8-Jul-2011

Client PO: 10220

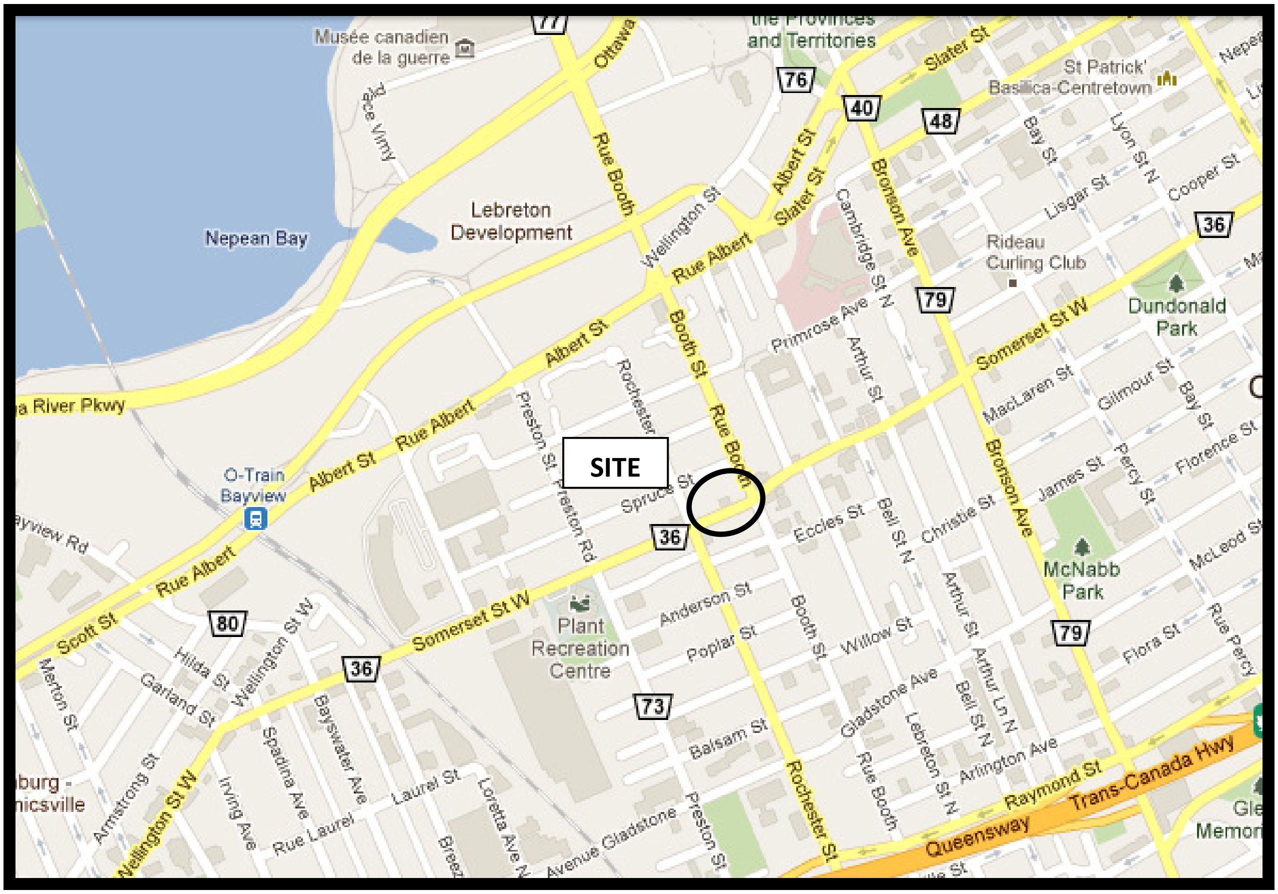
Project Description: PG2403

	Client ID:	BH3-SS4	-	-	-
	Sample Date:	08-Jul-11	-	-	-
	Sample ID:	1128303-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	93.3	-	-	-
General Inorganics					
pH	0.1 pH Units	7.8	-	-	-
Resistivity	0.10 Ohm.m	39.2	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	137	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

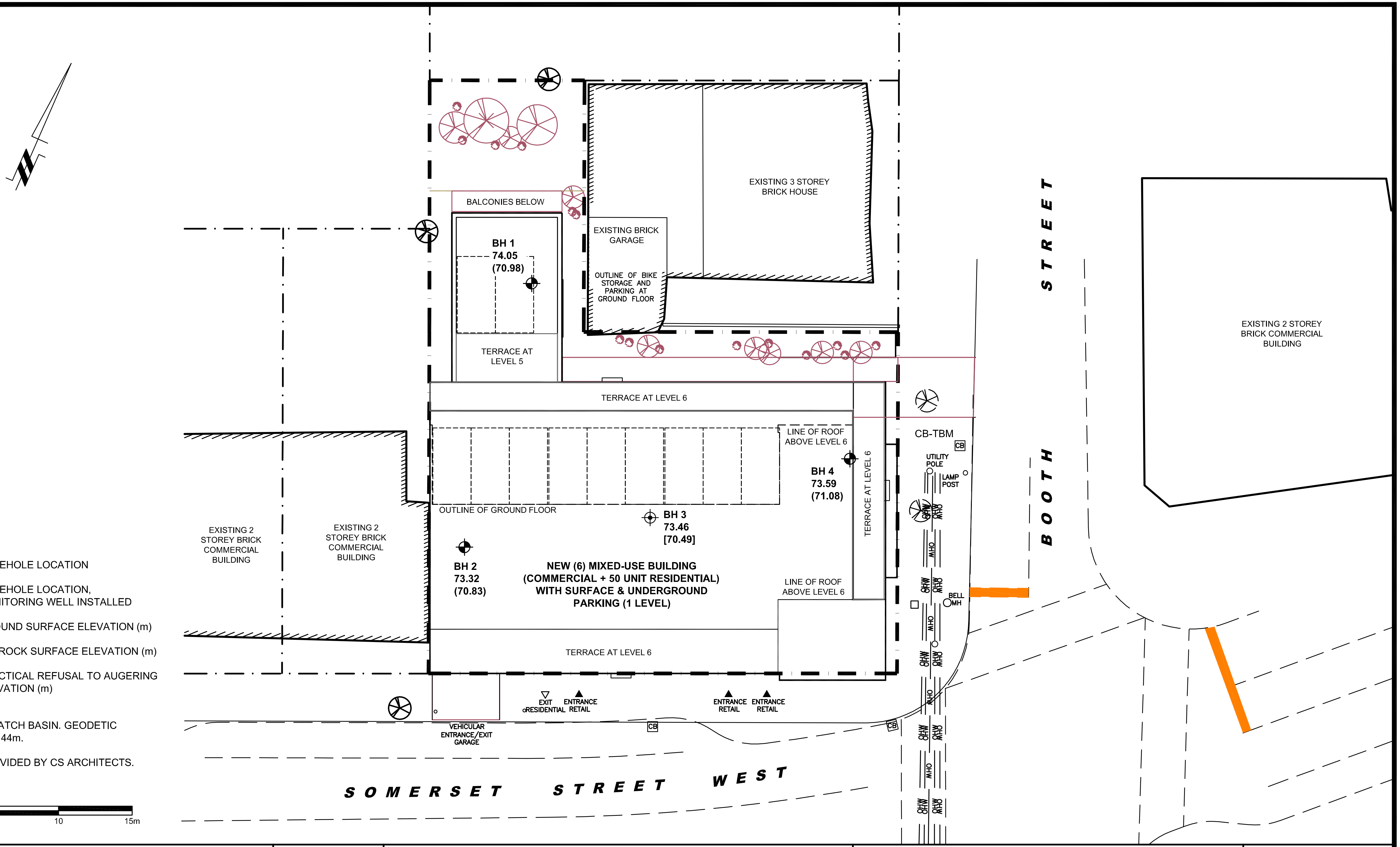
DRAWING PG2403-1 - TEST HOLE LOCATION PLAN



Source: Google Maps

FIGURE 1

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION
⌖	BOREHOLE LOCATION, MONITORING WELL INSTALLED
73.32	GROUND SURFACE ELEVATION (m)
[70.49]	BEDROCK SURFACE ELEVATION (m)
(70.83)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - TOP OF CATCH BASIN. GEODETIC ELEVATION = 73.44m.

BASE PLAN PROVIDED BY CS ARCHITECTS.

SCALE - 1:250

0 2 4 10 15m

patersongroup
consulting engineers
28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

Scale: 1:250
Des.: SB
Dwn: MPG
Chkd: DG

MR. PETER EVANS

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING-288 BOOTH STREET
OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No. PG2403-1
Report No.: PG2403-1
Date: 07/2011