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Geotechnical Investigation

Proposed Multi-Storey Development
590 to 594 Rideau Street
Ottawa, Ontario

Prepared For

Richcraft Group of Companies

April 13, 2016

Report: PG2730-1 Revision 1

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Richcraft Group of Companies to conduct a geotechnical investigation for the proposed development to be located at 590 - 594 Rideau Street, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

☐ Obtain subsurface soil and groundwater information by means of boreholes

☐ Provide geotechnical recommendations for 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. This 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 Development

The available drawings indicate that the proposed structure will consist of a seven storey building with a mechanical room on the rooftop with two levels of underground parking. The building footprint will occupy the majority of the subject site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was conducted on July 18, 2012. A total of four (4) boreholes (BH) were located throughout the subject site for general coverage. The boreholes were advanced to a maximum of 15 m depth. The locations of the test holes are shown on Drawing PG2730-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights or a 50 mm diameter split-spoon sampler. All soil samples were visually inspected and classified on site. The auger and split spoon 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 completed in cohesive soils using a field vane apparatus.

Overburden thickness was evaluated during the course of the investigation by dynamic cone penetration test (DCPT) at BH 1 and BH 3. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip. The steel drill rod is struck by 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 boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets presented in Appendix 1.

Groundwater

A 32 mm PVC groundwater monitoring well was installed in BH 2 and three (3) flexible standpipes were installed in the remaining boreholes to monitor the groundwater levels subsequent to the completion of the sampling program. The groundwater results are discussed in Subsection 4.3.

Sample Storage

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

3.2 Field Survey

The location and ground surface elevation at each borehole was surveyed by Paterson personnel. The boreholes were surveyed with respect to a temporary benchmark (TBM), consisting of a fire hydrant top spindle located on the north side of Rideau Street between Charlotte Street and Wurtemburg Street. An geodetic elevation of 72.2 m was provided by Annis, O'Sullivan Vollebek Ltd. for the TBM. Borehole locations, ground surface elevations, and the TBM location are presented on Drawing PG2730-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

All soil samples were recovered from the subject site and visually examined in our laboratory to review the soil investigation results.

3.4 Analytical Testing

A 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 analysed to determine the concentrations of sulphate and chloride, the resistivity and the pH of the sample. The analytical test results are presented in Appendix 1 and discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The site is relatively flat and at grade with surrounding streets and properties. The majority of the subject site is an asphalt surface with some landscaping around the perimeter. A maximum difference in ground surface elevation of 0.4 m was measured among the borehole locations.

The site is located in the south east corner of the intersection of Rideau Street and Charlotte Street, bordered south by low rise residential buildings and east by commercial buildings. The site is currently occupied with commercial/residential structures.

4.2 Subsurface Profile

Generally, the soil profile encountered at the borehole locations consists of an asphalt surface followed by fill material overlying a native silty sand and silty clay layer. The fill material consists of a silty sand matrix with gravel, cobbles, boulders and various construction debris. Practical refusal to augering was encountered in all boreholes. Practical refusal to DCPT was encountered at a 21 m and 22 m depth at BH 1 and BH 3, respectively.

Based on available geological mapping, interbedded limestone and shale bedrock of the Verulam Formation is present in this area with an overburden thickness ranging between 15 to 25 m.

Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

4.3 Groundwater

The monitoring well and standpipes were inspected and found dry on August 1, 2011. Groundwater can be interpolated from drilling behavior and soil colour. Based on this information local groundwater should be approximately 3 to 4 m below ground surface. Groundwater levels are subject to seasonal fluctuations and therefore, groundwater levels could be higher at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. Conventional footing foundations can be considered provided the bearing resistance values are compliant with the anticipated building loads. Where design loads exceed the given bearing resistance, consideration may be given to a foundation founded on end bearing piles, caissons or a raft foundation.

Deleterious fill should be removed from within the proposed building footprint.

Since the proposed building will be lower than the surrounding neighbouring buildings and will be founded on conventional footing foundations and/or raft foundations, a waterproofing system will have to be considered to prevent significant dewatering of adjacent structures.

The subject site is currently at grade with Rideau Street. Therefore, it is assumed that no significant grade raise will be required.

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

5.2 Site Grading and Preparation

Stripping Depth

Since the site excavation will occupy the entire site to a minimum depth of 10 to 11 m below the existing grade, all topsoil and fill materials will be removed from within the perimeter of the proposed building and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the proposed building, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill 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 area 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 where settlement of the ground surface is of minor concern. 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. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Protective Mud Slab

It is our understanding that the excavation will extend to a depth of approximately 10 to 11 m to accommodate a raft slab or conventional spread footing foundations. In both cases, the excavation bottom will be on silty clay which will require protection from disturbance due to worker traffic. A 50 to 75 mm thick lean concrete mud slab is recommended to be poured onto the undisturbed clay surface once exposed. The lean concrete should consist of 17 MPa compressive strength concrete.

5.3 Foundation Design

Spread Footing Foundation

Consideration may be given to placing footings for an eight storey building design at a higher elevation would utilize the stiff silty clay material. Shallow pad footings, up to 6 m wide, founded on an undisturbed stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

A permissible grade raise is not recommended as it is assumed the building will be constructed at or close to the current grade.

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.

The bearing resistance value given for footings at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Raft Foundation

Consideration can be given to a raft foundation if the building loads are acceptable. The following parameters may be used for raft design.

For design purposes, it was assumed that the base of the raft foundation will be located at a 7 to 9 m depth to accommodate for two levels of underground parking. The factored bearing resistance (contact pressure) at ULS can be taken as **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **170 kPa** will be considered acceptable. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

The modulus of subgrade reaction was calculated to be **2.2 MPa/m** for a contact pressure of **170 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters. A total and differential settlement of 25 and 15 mm, respectively. The basement slab is expected to be located at or below 6 m depth and the long term groundwater level will be at or below 3 m depth. Therefore, the raft slab should incorporate a waterproofing membrane system along with the perimeter foundation walls.

Deep Foundation

As an alternative to the above founding suggestions, for support of the proposed multi-storey building consideration could be given to using concrete filled steel pipe piles driven to refusal on the bedrock surface.

For deep foundations, concrete-filled steel pipe piles are commonly utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 1. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula. The piles load bearing capacity should be confirmed during pile installation with a program of dynamic monitoring. The dynamic monitoring of two to four piles is recommended. The pipe piles should be equipped with a minimum 20 mm thick base plate minimize damage to the pile tip during installation. Re-striking of all piles at least once will be required after at a minimum of 48 hours have elapsed since initial installation.

Table 1 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	910	1090	10	28.5
245	11	1050	1260	10	34.2
245	13	1185	1420	10	40.7

5.4 Design for Earthquakes

A shear wave velocity test was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity test was completed by Paterson. Two shear wave velocity profiles are presented in Appendix 2.

Field Program

The shear wave test location is presented in Drawing PG2730-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel installed 24 horizontal geophones in a straight line oriented roughly east-west across the southside border of the subject site. The 4.5 Hz. horizontal geophones were mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 3 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was connected to a computer and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a signal to the seismograph to commence recording. The hammer strikes an I-Beam seated into the ground surface, which produces a polarized shear wave. The hammer shots are repeated between four to eight times at each shot location to provide an accurate signal and reduce noise. The shot locations are completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were distributed at the centre of the geophone array and 3, 4.5 and 6 m away from the first and last geophone.

The testing method completed by Paterson are guided by the standard test procedures outlined by expert seismologists at Carleton University and Geological Survey of Canada (GSC).

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson. The shear wave velocity measurement was calculated using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, immediately below the proposed building's foundation of the upper 30 m profile. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. As bedrock quality increases, the bedrock shear wave velocity increases.

The proposed buildings are expected to be founded on native soil. Based on our analysis, the seismic shear wave velocity was calculated to be 276 m/s. The V_{s30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012, as presented below.

Based on the seismic test results, the average shear wave velocity, V_{s30}, for shallow foundations located at the subject site is 276 m/s. Therefore, a **Site Class D** is applicable for design of the proposed buildings at the subject site, as per Table 4.1.8.4.A of the OBC 2012.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{276m/s}\right)}$$

$$V_{s30} = 276m/s$$

5.5 Rock Anchor Design

In the event that rock anchors are required for lateral shear wave for seismic considerations, the following geotechnical design parameters are provided.

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 a 60 to 90 degree cone pullout 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 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.

Centre to centre spacing between bond lengths should be a minimum of four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. Rock anchors in close proximity are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or “post-tensioned”, 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. However, a post-tensioned anchor will sustain the uplift load force with less deflection than a passive anchor.

Regardless of whether an anchor is passive or post tensioned, it is recommended that the anchor be provided with a bonded length(fixed anchor length) at the base of the anchor and an unbonded length(free anchor length) between the rock surface and the start of the bonded length. The bonded length will provide the anchor capacity. Shear failure cone tend to develop midway along the bonded length, therefore a fully bonded anchor would tend to have a much shallower cone and less resistance force 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, this requires that the entire drill hole be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

The unconfined compressive strength of limestone/dolomite ranges between about 60 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.2 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. Based on local subsurface information, 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, all rock anchors are assumed to be installed 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/Shale Hoek and Brown parameters	44 m=0.128 and s=0.00009
Unconfined compressive strength - Limestone/Shale bedrock	60 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 75 and 125 mm diameter holes are provided in Table 3. The factored tensile resistance values given in Table 3 are based on a single anchor with no group influence effects. One test hole should be completed prior to rock anchor installation to determine accurate bedrock properties.

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	0.5	2.0	2.5	500
	1.0	2.7	3.7	750
	1.5	3.3	4.8	1,250
125	0.5	2.50	2.9	500
	0.5	3.0	3.5	750
	1.0	3.5	4.5	1,250

Other considerations

The anchor drill holes should be inspected by the geotechnical consultant and should be flushed clean prior to grouting. A tremie pipe method is recommended to place grout from the bottom to the top of the rock anchor holes. The anchor drill hole diameter should be within 1.5 to 2 times the rock anchor tendon diameter.

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. 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.

5.6 Basement Slab

The basement area is expected to be mainly parking and a concrete slab topping with a subfloor granular layer will be incorporated in the design to accommodate services and a rigid pavement structure noted in Subsection 5.8 will be applicable. The thickness of the granular subfloor layer will be dependent on what services are incorporated in the design. It is also expected that a sump pit will be incorporated to drain any water which enters the granular layer.

A concrete mud slab should be poured to protect the native soil from construction equipment and traffic if spread footings are considered. A granular working mat up to 600 mm thick may be required for the support of the pile driving crane.

5.7 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, in our opinion, 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 bulk (drained) unit weight of 20 kN/m^3.

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_A) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

The static horizontal earth pressure (P_A) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.5
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

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

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

$a_c = (1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, $9.81\ m/s^2$

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The total earth pressure (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{Pa \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth pressures calculated are unfactored. For the ULS case, the earth pressure loads should be factored as live loads, as per OBC 2012.

5.8 Pavement Structure

The proposed lower basement slab will be considered a rigid pavement structure. The following rigid pavement structure is suggested to support car parking only.

Table 4 - Recommended Rigid Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
125	**Wear Course -** Concrete slab
150	**BASE** - 20 mm clear stone
	SUBGRADE - Mud slab and/or native soils

Asphalt pavement is not anticipated to be required at the subject site. However, should a flexible pavement be considered for the project, the recommended flexible pavement structures shown in Tables 5 and 6 would be applicable.

Table 5 - Recommended Flexible 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 6 - Recommended Flexible Pavement Structure - Access Ramp

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 building. 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. 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, such as Miradrain G100N or Delta Drain 6000. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for backfill material.

For the basement levels extending below the low groundwater level, consideration should be given to waterproofing the lower levels to prevent dewatering of adjacent sites which may cause long term settlements of neighbouring structures. Details can be provided for the waterproofing system once the building design is confirmed.

6.2 Protection of Footings, Pile Caps and Grade Beams Against Frost Action

Footings, pile caps and grade beams 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

Excavation Side Slopes

Excavation side slopes constructed in fill materials should either be cut back at acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. Sufficient room is assumed to be available for the greater part of the excavation to be constructed 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. A flatter slope is required for excavations below groundwater level. The subsurface soils are considered to be 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 protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Underpinning

Founding conditions of adjacent structures bordering the site should be assessed and underpinning requirements should be evaluated.

6.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of the SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and 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 minimize 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 silty clay should be low due to the relative impervious nature of these materials. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

6.6 Winter Construction

Precautions should be considered if construction occurs during the winter. The subsurface soil conditions 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. The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until 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 difficult activities to complete during winter without introducing frost in the excavation subgrade base or walls. Precautions should be considered if such activities are to be completed during sub-zero temperatures.

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 non-aggressive to slightly aggressive corrosive environment.

7.0 Recommendations

The following material testing and observation program should be performed by a geotechnical consultant and is required for the foundation design data provided herein to be applicable:

☐ Review of the proposed structure(s) and adjacent structures from a geotechnical perspective.

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

☐ Sampling and testing of the concrete and 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 and follow-up 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.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. Also, our recommendations should be reviewed when the project drawings and specifications are complete.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Richcraft Group of Companies or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
April 15, 2016
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Carlos P. Da Silva. P.Eng.

Report Distribution:

❑ Richcraft Group of Companies (3 copies)
❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 590-594 Rideau St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, north side of Rideau Street, across from subject site. Geodetic elevation = 72.20m.

REMARKS

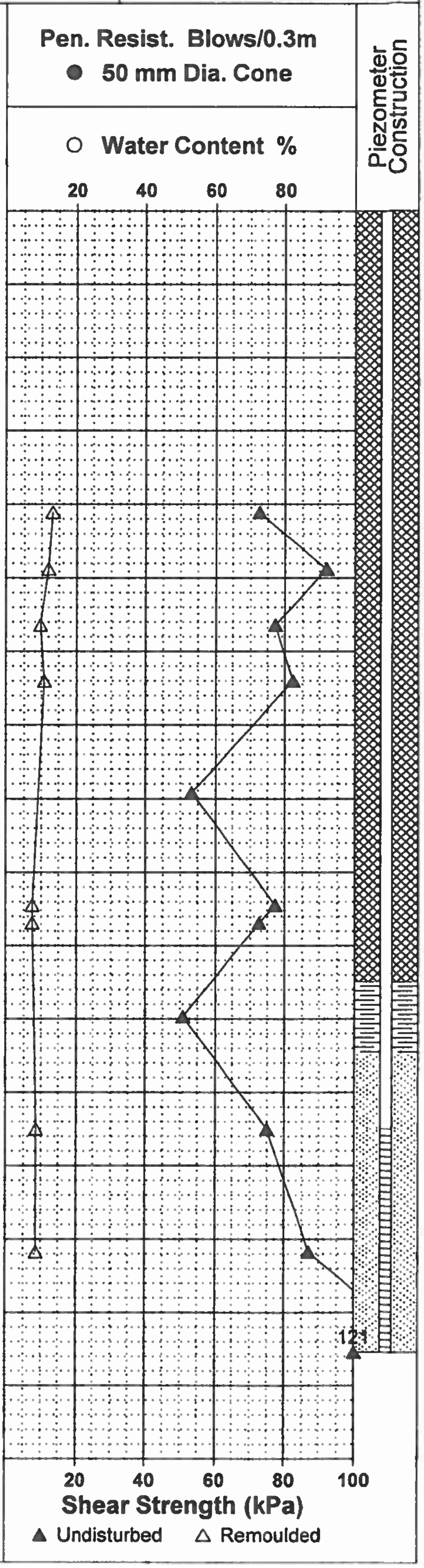
BORINGS BY CME 55 Power Auger

DATE July 18, 2012

FILE NO. PG2730

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	71.14
FILL: Sand and gravel	AU	1				
	AU	2				
0.69						
Compact, brown **SILTY SAND**	SS	3	58	10	1	70.14
	SS	4	67	15	2	69.14
	SS	5	62	15		
2.97					3	68.14
Stiff to very stiff, grey **SILTY CLAY**	SS	6	100	2		
					4	67.14
					5	66.14
					6	65.14
					7	64.14
					8	63.14
					9	62.14
					10	61.14
					11	60.14
					12	59.14
					13	58.14
					14	57.14
					15	56.14
15.54						
Dynamic Cone Penetration Test commenced at 15.54m depth. Cone pushed to 17.1m depth.					16	55.14
					17	54.14



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 590-594 Rideau St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, north side of Rideau Street, across from subject site. Geodetic elevation = 72.20m.

REMARKS

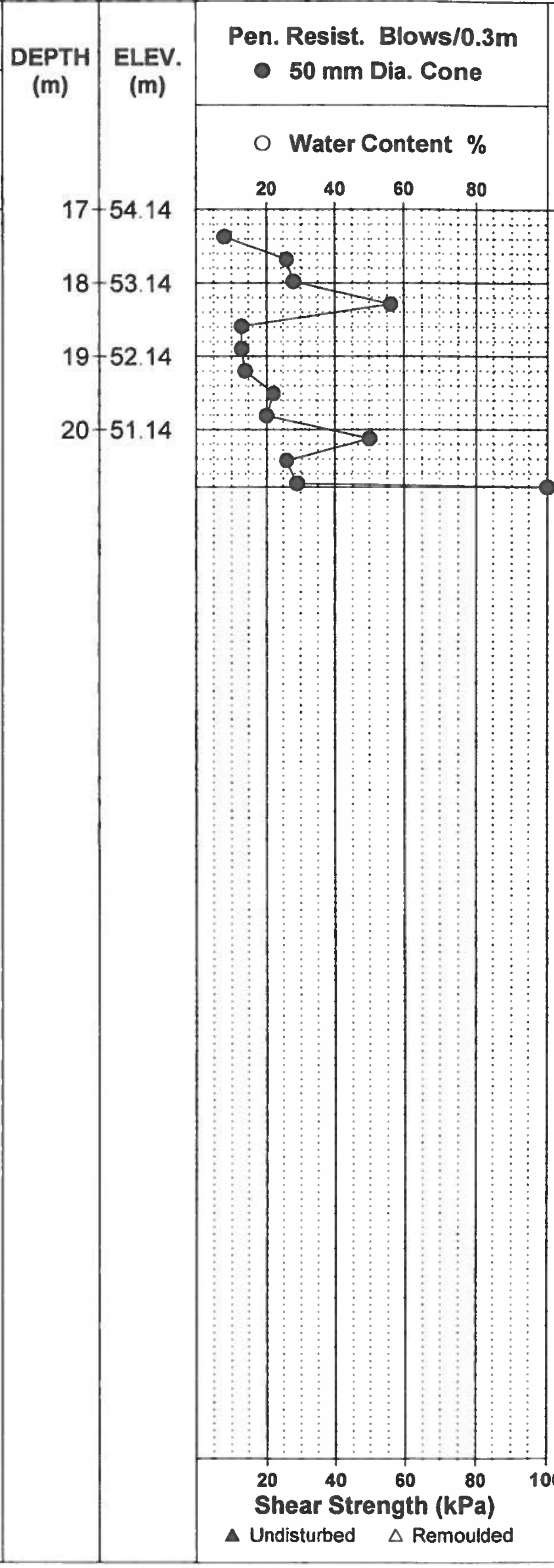
BORINGS BY CME 55 Power Auger

DATE July 18, 2012

FILE NO. PG2730

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
20.78					
End of Borehole Practical DCPT refusal at 20.78m depth.					



Piezometer Construction

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 590-594 Rideau St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, north side of Rideau Street, across from subject site. Geodetic elevation = 72.20m.

REMARKS

BORINGS BY CME 55 Power Auger

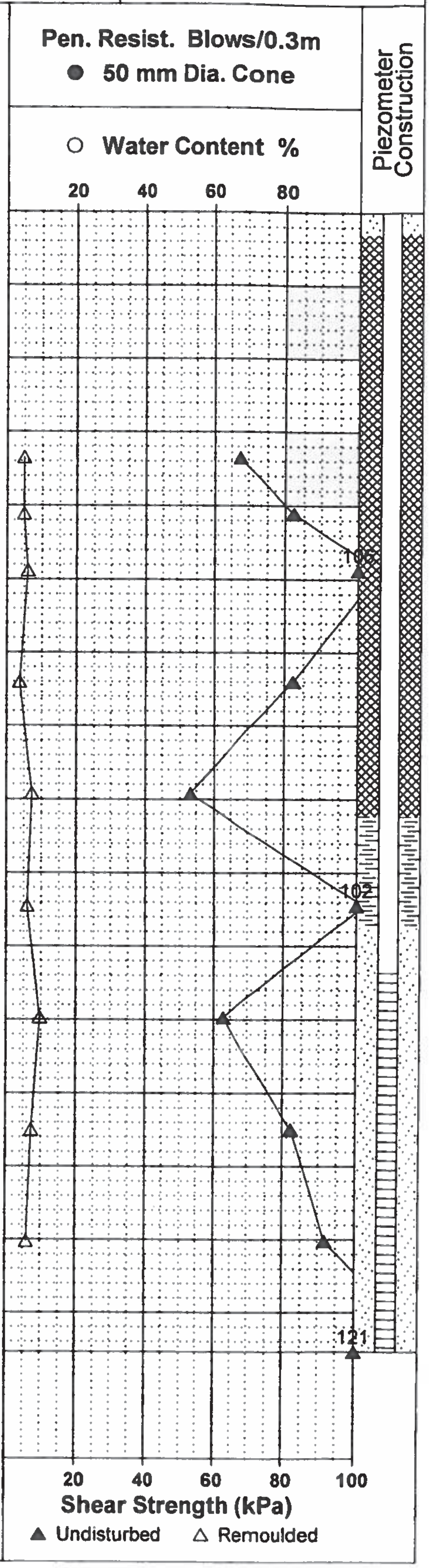
DATE July 19, 2012

FILE NO. PG2730

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
25mm Asphaltic concrete over silty sand with gravel - trace brick and asphalt by 0.3m depth 1.45	AU AU SS	1 2 3	 0	 50+
Compact, brown **SILTY SAND** 2.97	SS SS	4 5	71 75	11 14
Stiff to very stiff, grey **SILTY CLAY** 15.54				
End of Borehole				

DEPTH (m)	ELEV. (m)
0	70.93
1	69.93
2	68.93
3	67.93
4	66.93
5	65.93
6	64.93
7	63.93
8	62.93
9	61.93
10	60.93
11	59.93
12	58.93
13	57.93
14	56.93
15	55.93



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 590-594 Rideau St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, north side of Rideau Street, across from subject site. Geodetic elevation = 72.20m.

REMARKS

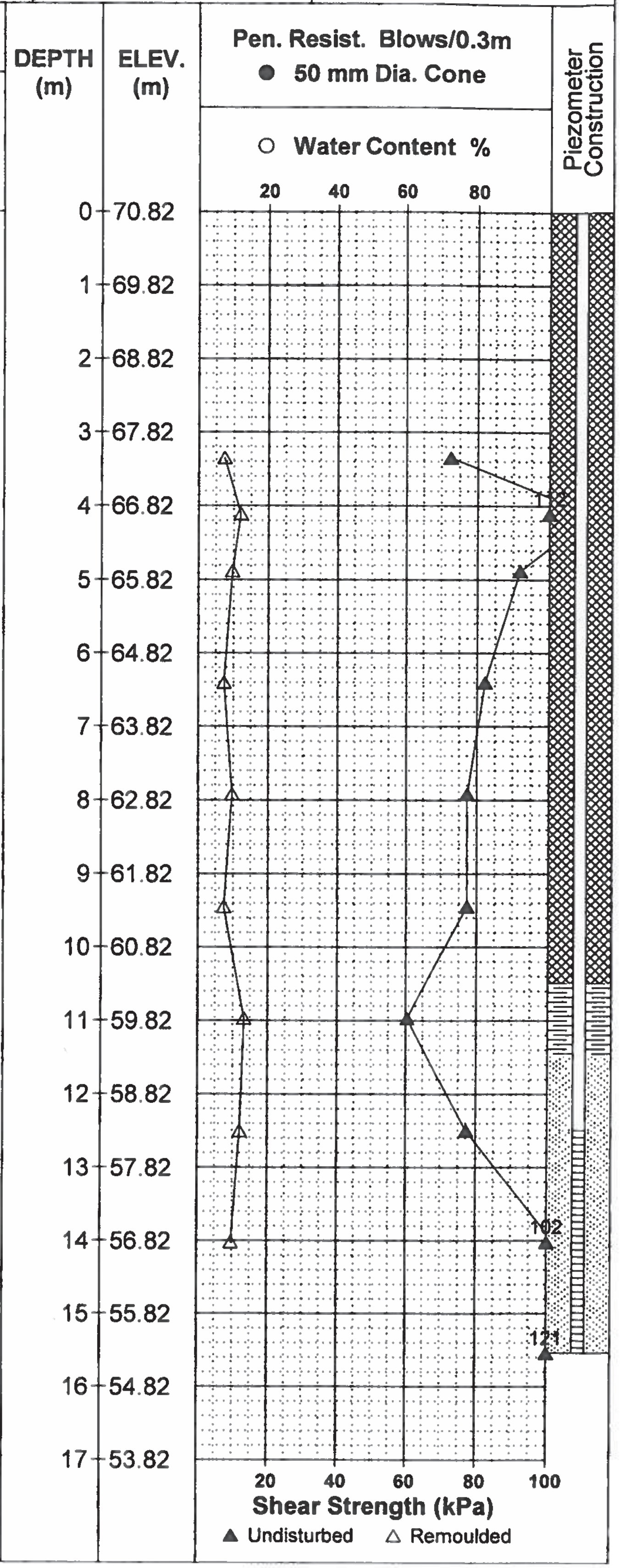
BORINGS BY CME 55 Power Auger

DATE July 18, 2012

FILE NO. PG2730

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
25mm Asphaltic concrete over crushed stone 0.30	AU	1		
FILL: Brown silty sand with gravel, trace clay 1.45	AU	2		
	SS	3	12	5
Compact, brown **SILTY SAND** 2.97	SS	4	83	14
	SS	5	50	10
Stiff to very stiff, grey **SILTY CLAY** 15.54				
Dynamic Cone Penetration Test commenced at 15.54m depth. Cone pushed to 16.8m depth.				



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 590-594 Rideau St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, north side of Rideau Street, across from subject site. Geodetic elevation = 72.20m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 18, 2012

FILE NO. PG2730

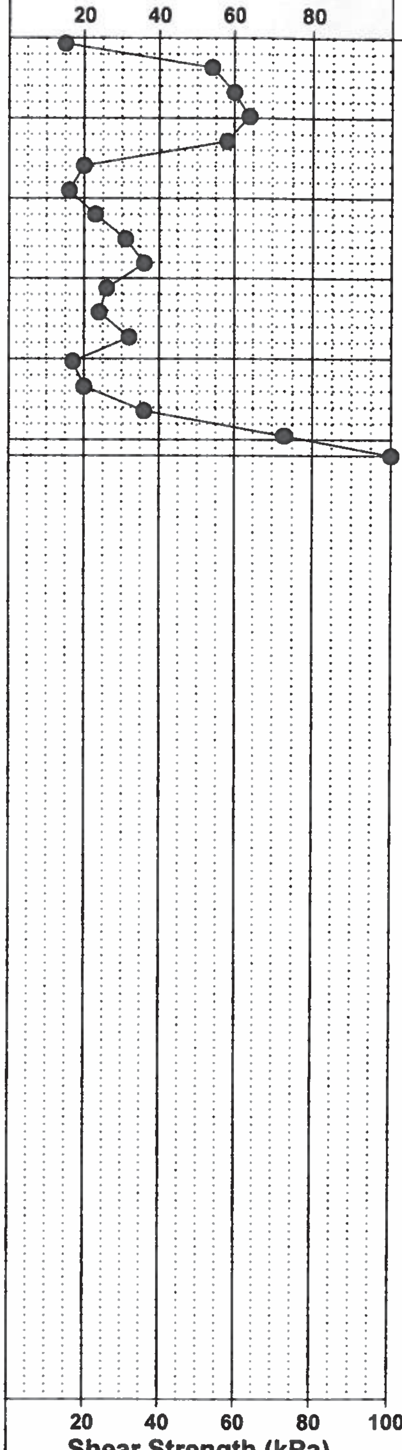
HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						17	53.82
						18	52.82
						19	51.82
						20	50.82
						21	49.82
22.20						22	48.82
End of Borehole Practical DCPT refusal at 22.20m depth							

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - 590-594 Rideau St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, north side of Rideau Street, across from subject site. Geodetic elevation = 72.20m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 19, 2012

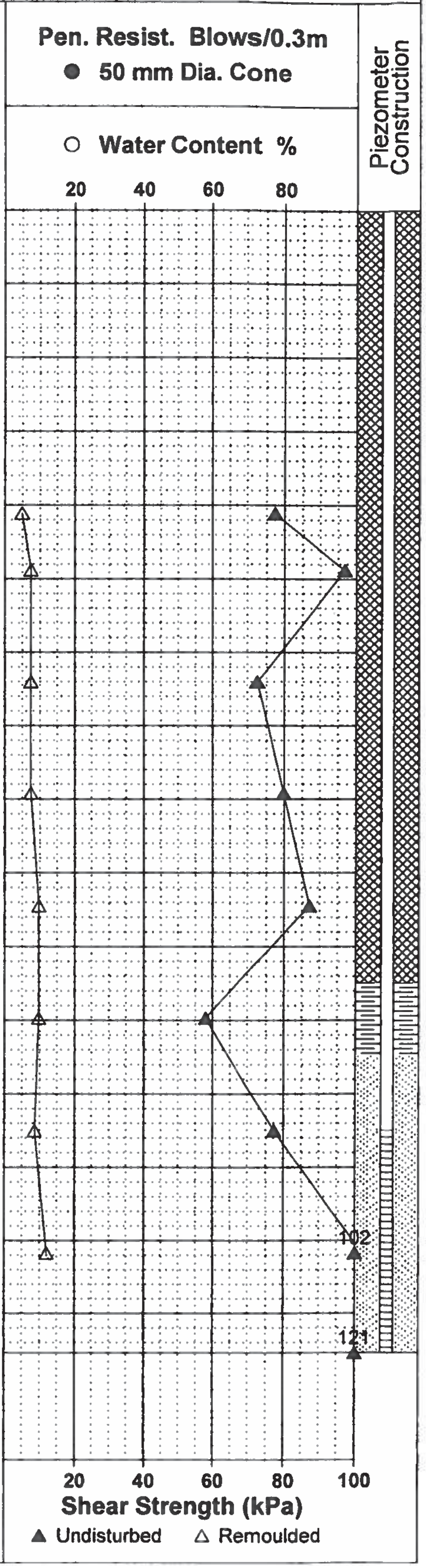
FILE NO. PG2730

HOLE NO. BH 4

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
FILL: Brown silty sand with gravel, cobbles, brick, concrete	1.83
Compact, brown **SILTY SAND**	3.73
Stiff to very stiff, grey **SILTY CLAY**	15.54
End of Borehole	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	54	
SS	2	33	12
SS	3	62	14
SS	4		14

DEPTH (m)	ELEV. (m)
0	71.37
1	70.37
2	69.37
3	68.37
4	67.37
5	66.37
6	65.37
7	64.37
8	63.37
9	62.37
10	61.37
11	60.37
12	59.37
13	58.37
14	57.37
15	56.37



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)

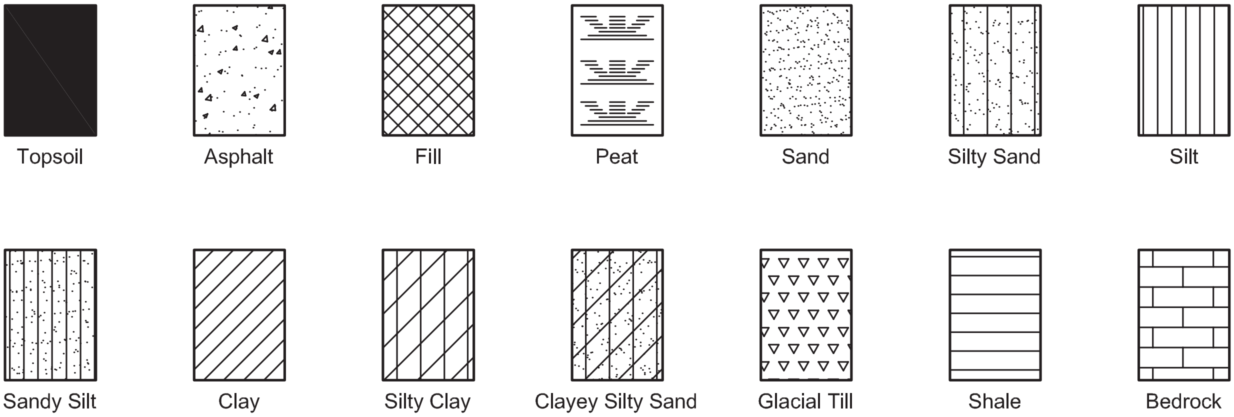
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

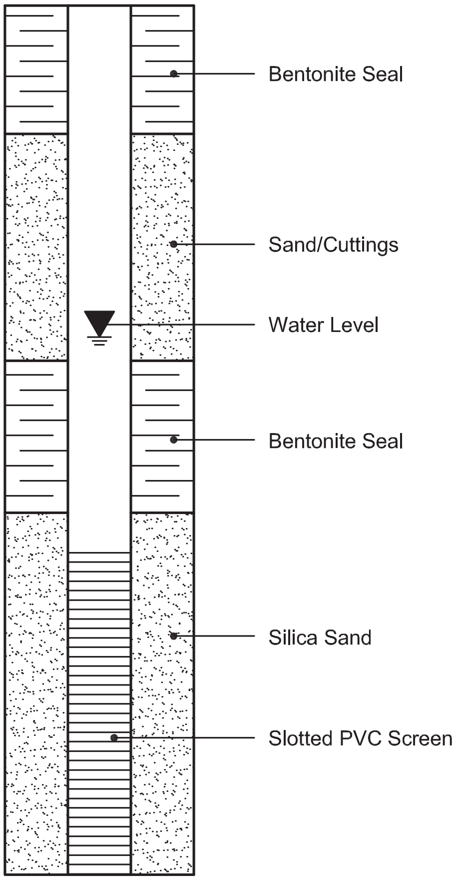
SYMBOLS AND TERMS (continued)

STRATA PLOT

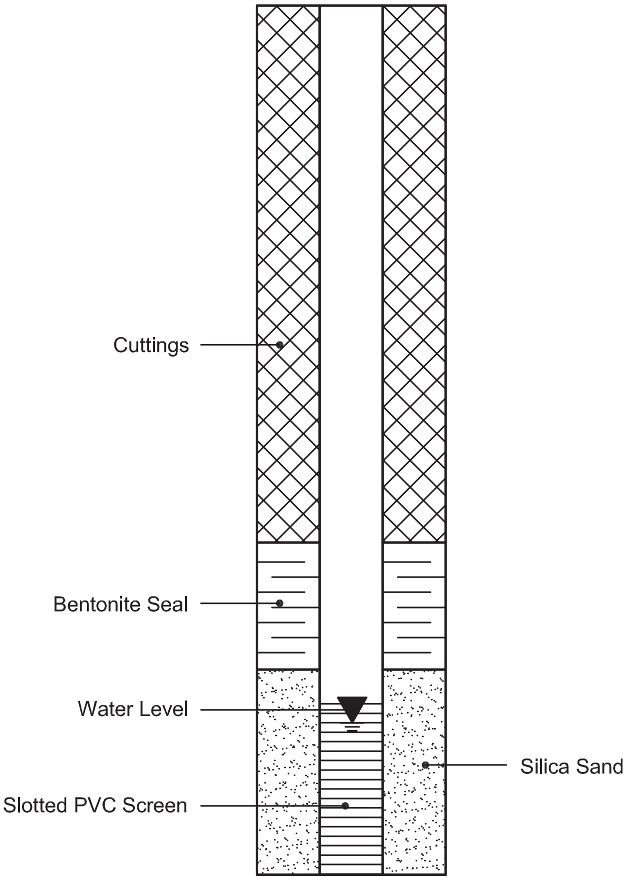


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Order #: 1231046

Certificate of Analysis

Report Date: 03-Aug-2012

Order Date:30-Jul-2012

Client: **Paterson Group Consulting Engineers**

Client PO: 13231

Project Description: PG2730

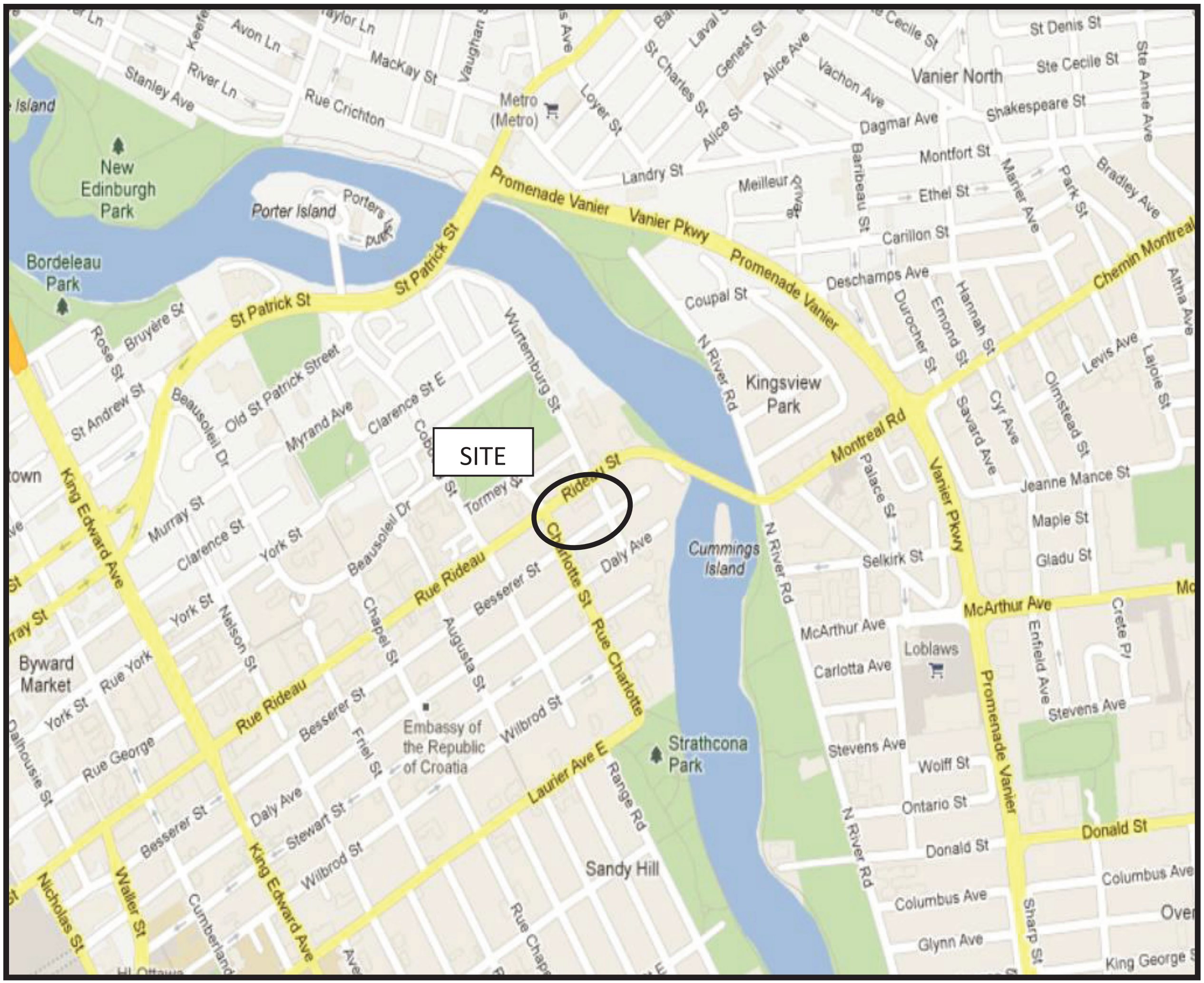
	Client ID:	BH2-SS4	-	-	-
	Sample Date:	19-Jul-12	-	-	-
	Sample ID:	1231046-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.52	-	-	-
Resistivity	0.10 Ohm.m	45.2	-	-	-
Anions					
Chloride	5 ug/g dry	25	-	-	-
Sulphate	5 ug/g dry	58	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG2730-1 - TEST HOLE LOCATION PLAN



Source: Google Maps

FIGURE 1

KEY PLAN

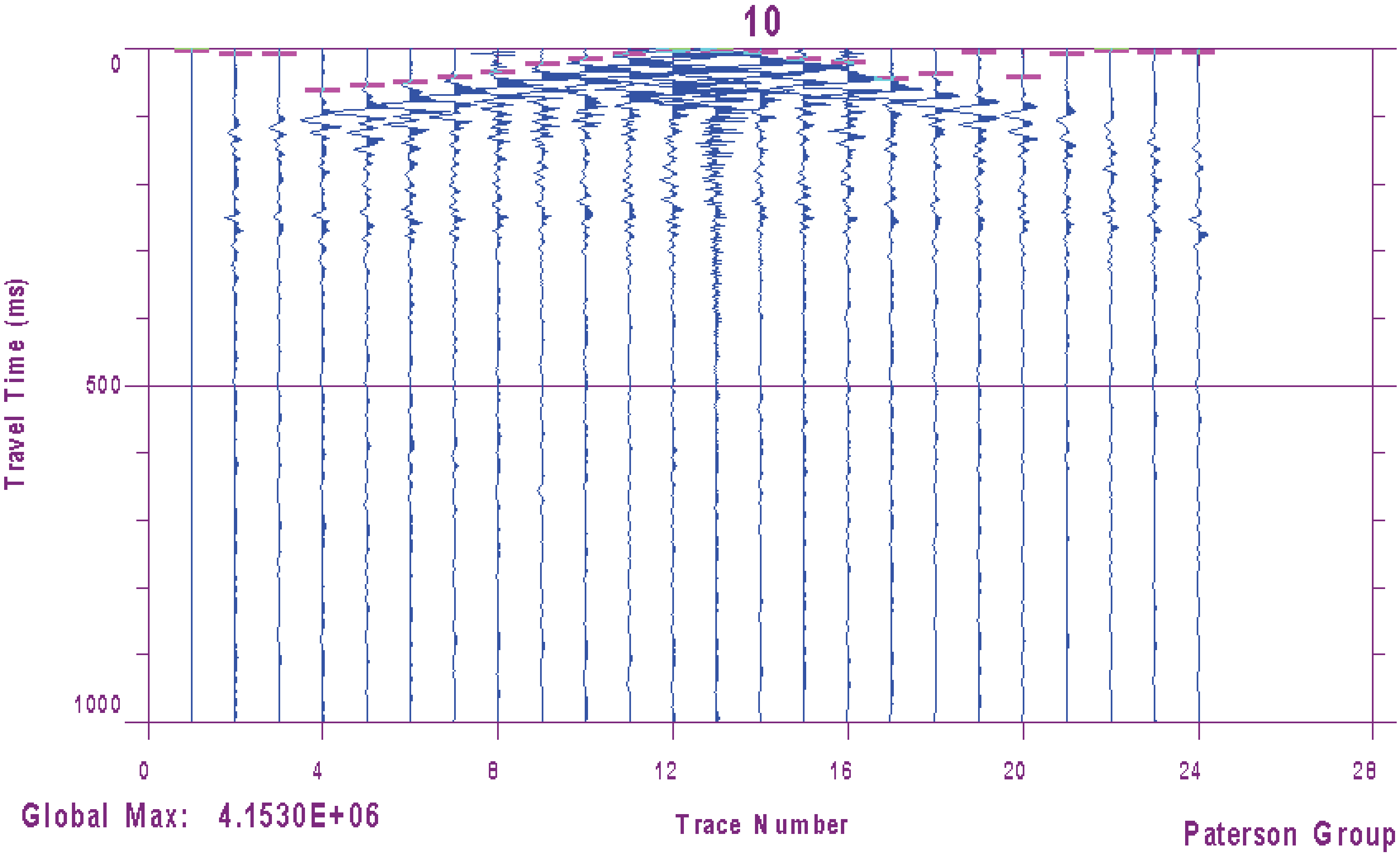


Figure 2 – Shear Wave Velocity Profile at Shot Location 23 m

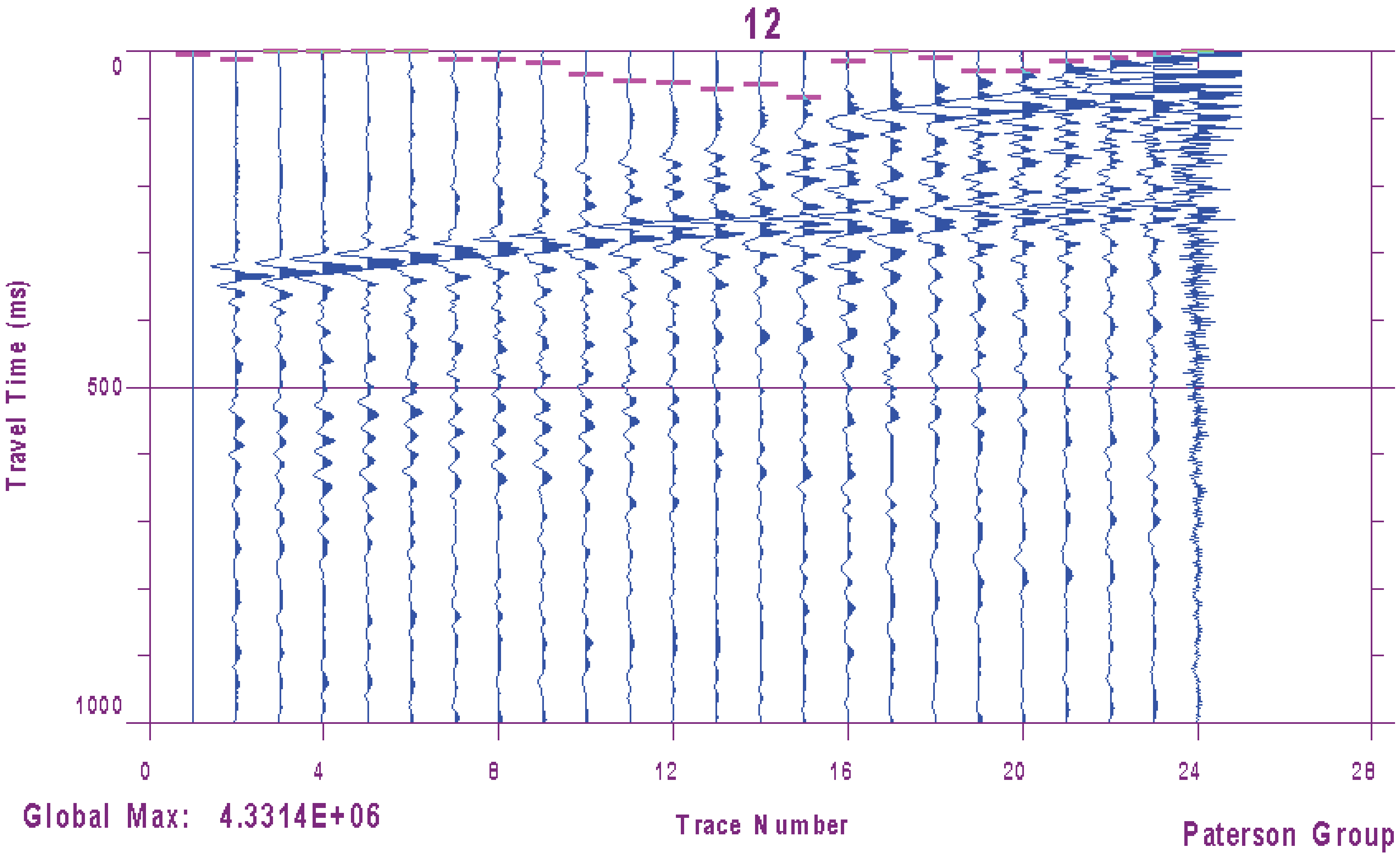
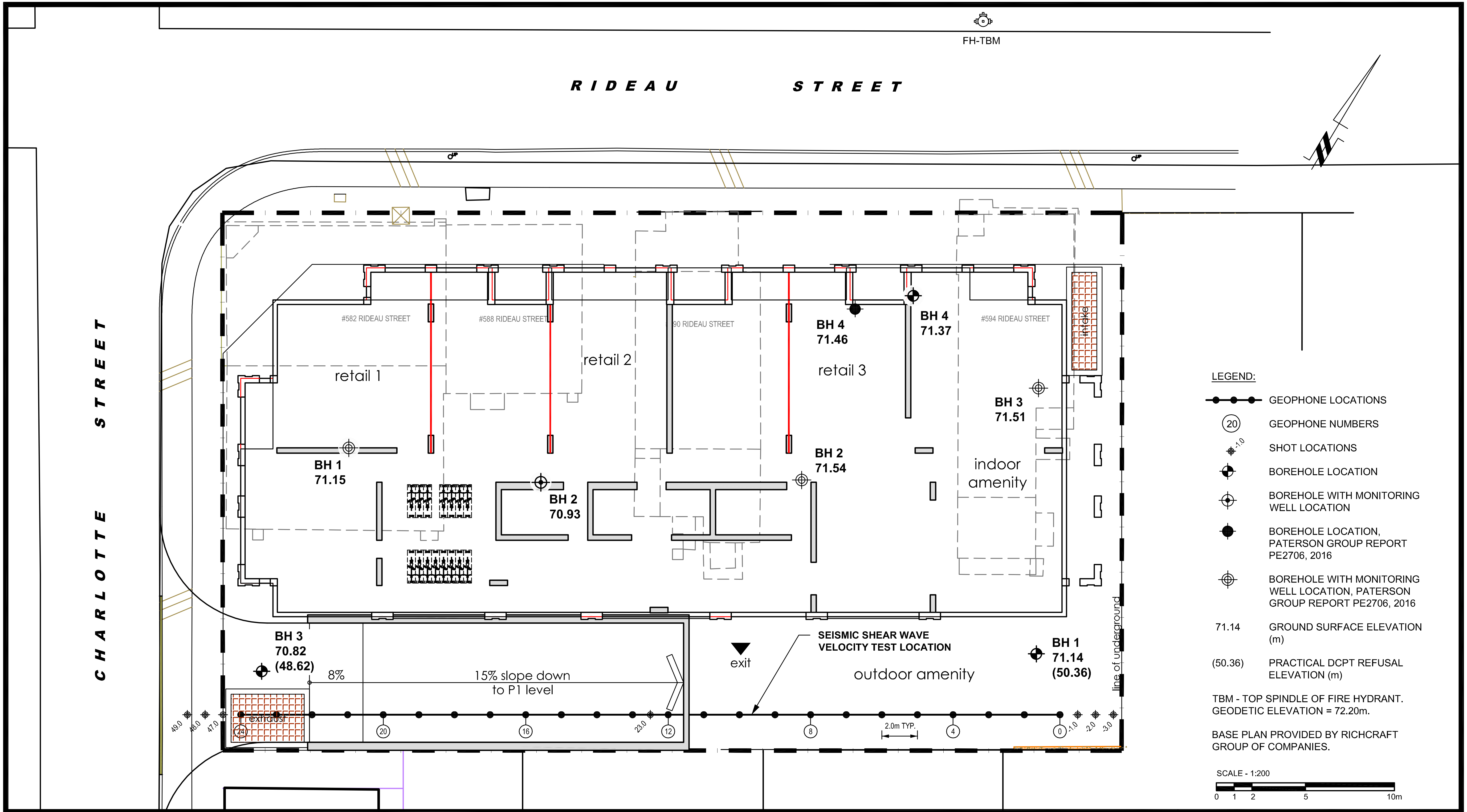


Figure 3 – Shear Wave Velocity Profile at Shot Location 49 m



patersongroup
consulting engineers

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

0	BASE PLAN UPDATED	14/04/2016	DJG
NO.	REVISIONS	DATE	INITIAL

RICHCRAFT GROUP OF COMPANIES

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 590-594 RIDEAU STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:200
Drawn by: MPG
Checked by: JAF
Approved by: DJG
Date: 08/2012
Report No.: PG2730-1
Dwg. No.: PG2730-1
Revision No.: 1

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