Geotechnical Engineering

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Materials Testing

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Geotechnical Investigation

Proposed Multi-Storey Building
113 and 115 Echo Drive
Ottawa, Ontario

Prepared For

Uniform Developments

January 15, 2014

Report PG3142-1

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1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Uniform Developments to conduct a geotechnical investigation for a proposed multi-storey building to be located at 113 and 115 Echo Drive, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The investigation objectives 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. This report contains the findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation.

2.0 PROPOSED PROJECT

It is understood that specific details of the proposed project are not currently available. However, the preliminary design details consist of an eight storey building with two levels of underground parking levels occupying the majority of the subject site.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The field program for the investigation was completed on December 4 and 5, 2013. Four (4) boreholes were complete to a maximum 12 m depth. The test hole locations were placed in a manner to provide general coverage of the subject site while considering site constraints. The test hole locations are presented on Drawing PG3142-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed with a truck-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The testing procedure consisted of augering to the required depths and at select locations sampling the overburden.

Sampling and In Situ Testing

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

The Standard Penetration Test (SPT) was conducted and 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 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Overburden thickness was evaluated by a dynamic cone penetration test (DCPT) at BH 1. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded every 300 mm increment.

Undrained shear strength tests were completed in cohesive soils with a shear vane apparatus. The tests were completed in general accordance with ASTM D2573-08 - Standard Test Method for Field Vane Shear Test in Cohesive Soil.

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

Groundwater

A 32 mm diameter groundwater monitoring well was installed at BH 1, BH 2 and BH 4 to monitor the groundwater level subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were determined in the field by Paterson personnel with consideration of underground and aboveground services. The test holes were located in the field by Paterson and were referenced to a temporary benchmark (TBM). The TBM consisted of the top spindle of the fire hydrant located along the west side of the subject site along Echo Drive. An assumed elevation of 100 m was assigned to the TBM. The location of the boreholes and ground surface elevations at the borehole locations are presented on Drawing PG3142-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the field investigation were examined in our laboratory by a geotechnical engineer. The subsurface soils were classified in general accordance with ASTM D2488-09a, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).

Sample Storage

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

4.0 OBSERVATIONS

4.1 Surface Conditions

Currently, the subject site is occupied by two buildings within the west portion of the site. The remainder of the subject site consists of paved access lanes, parking areas and grass covered areas. The ground surface across the site is relatively flat and at grade with the adjacent roadways and neighbouring properties.

4.2 Subsurface Profile

Generally, the subsurface profile at the borehole locations consists of a pavement structure overlying a silty sand fill or a native silty sand deposit and a very stiff to stiff grey silty clay deposit. A glacial till layer, consisting of a silty sand with gravel, cobbles and boulders was encountered below the silty clay. Practical refusal to DCPT was encountered at approximately 25.5 m depth in BH 1.

4.3 Groundwater

Groundwater level measurements were attempted on January 14, 2014. However, due to an ice layer or snow pile over the monitoring well locations, the wells were not accessible. The long-term groundwater table can also be estimated based on consistency, moisture levels and colour of the recovered soil samples. Based on these observations, the long-term groundwater level is expected at a depth ranging from 3.5 to 4.5 m depth. It should be noted that the groundwater is subject to seasonal fluctuations and therefore, groundwater could vary at the time of construction.

5.0 DISCUSSION

5.1 Geotechnical Assessment

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

The above and other considerations are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

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

Fill Placement

Fill placed 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. The granular material should be placed in maximum 300 mm thick loose lifts and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building should be compacted to a minimum of 98% of the Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed under the subgrade level for areas to be paved, the material should be compacted in thin lifts to a minimum density of 95% of the SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Conventional Shallow Footings - Bearing Resistance Values

Strip footings, up to 5 m wide, and pad footings, up to 10 m wide, placed on an undisturbed, stiff silty clay bearing surface 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**.

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

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

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to an engineered fill, silty clay bearing medium when a plane extending a minimum of 1.5H:1V, from the footing perimeter to the founding soil/engineered fill.

Due to the high building loads anticipated and the undrained shear strength testing values noted within the silty clay deposit encountered at the test hole locations, a permissible grade raise restriction of **1 m** is recommended for grading in close proximity of the proposed building.

Raft Foundation

Alternatively, consideration can be given to a raft foundation if the building loads exceed the bearing resistance values provided for a conventional shallow footing foundation. 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 1.5 m depth with no anticipated underground levels.

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 **180 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 contact pressure provided considers the stress relief associated with the soil removal required for proposed building. The factored bearing resistance (contact pressure) at ULS can be taken as **240 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **5 MPa/m** for a contact pressure of **180 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 with a total and differential settlement of 25 and 15 mm, respectively.

Deep Foundation - End Bearing Piles

Alternatively, a deep foundation method, such as end bearing piles, can be considered for the proposed structure if the design building loads exceed the bearing resistance values provided for a conventional shallow footing foundation. Concrete filled steel pipe piles driven to refusal on a bedrock surface are a typical deep foundation option in Ottawa.

Applicable pile resistance at SLS values and factored pile resistance at ULS values are provided in Table 1. Additional resistance values can be provided if available pile sizes vary from those detailed 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 calculating the Hiley dynamic formula. The piles should be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of four piles is recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles will also be required after at least 48 hours have elapsed since initial driving.

Table 1 - End Bearing Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 25 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	10	975	1460	10	35.9
245	12	1100	1650	10	42
245	13	1175	1760	10	45.4

5.4 Design for Earthquakes

The site class for seismic site response is a **Class D** for the proposed building. The soils underlying the subject site are not susceptible to liquefaction. Refer to the most recent 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 deleterious fill material, the native soil will be considered to be an acceptable subgrade surface on which to commence backfilling for the basement slab. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-floor fill consist of OPSS Granular A crushed stone. All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of the SPMDD.

A concrete mud slab should be placed to protect the native soil from worker traffic and equipment before pouring the raft slab.

5.6 Basement Walls

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, 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. 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.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) 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 total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ 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 earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Structure

Car only parking areas and access lanes are anticipated at this site. The proposed pavement structures are shown in Tables 2 and 3.

Table 2 - 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 - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

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

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

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

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

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should be installed at each catch basin, be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

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

The groundwater infiltration control system should extend at least 1 m above the long-term groundwater level and the following is suggested for preliminary design purposes:

- ❏ Place a suitable membrane against the temporary shoring surface, such as a bentomat liner system or equivalent. The membrane liner should extend down to footing level. The membrane liner should also extend horizontally a minimum 600 mm below the footing at underside of footing level.
- ❏ Place a composite drainage layer, such as Miradrain G100N or equivalent, over the membrane (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
- ❏ Pour foundation wall against the composite drainage system.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

It is recommended that 100 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of any water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Backfill

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

6.2 Protection of Footings Against Frost Action

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

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

6.3 Excavation Side Slopes

Unsupported Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. Insufficient room is expected for majority 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 excavated at 1H:1V or shallower. The shallower slope is required for excavation 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 should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring may be required to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

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

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

Table 4 - Soil Parameters for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

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

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.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. 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.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further 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. 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.

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

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 crushed stone, 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.

Generally,the brown silty clay should be possible to place above the cover material if the excavation and backfilling operations are completed in dry weather conditions. Wet silty clay materials will be difficult for placement, as the high water content are impractical for the desired compaction without an extensive drying period.

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.

To reduce long-term lowering of the groundwater level, clay seals should be provided in the service trenches. The seals should be a minimum of 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. The seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry impervious material placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at a maximum of 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

Due to the relatively impervious nature of the silty clay and existing groundwater level, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

A temporary MOE permit to take water (PTTW) will be required for this project if more than 50,000 L/day are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MOE.

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.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which breaches the building's perimeter groundwater infiltration control system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e.- less than 10,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

It is understood that preliminary concepts indicate that a two (2) levels of underground parking is planned for the proposed building. Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsurface soil conditions mainly 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 installation 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 constructed in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

Precaution must be taken where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is installed, 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.

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.

❏ Full time inspection of pile driving operation, if required.

❏ Inspection of all foundation drainage systems.

❏ Sampling and testing of the concrete and fill materials placed.

❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

❏ Observation of all subgrades prior to backfilling.

❏ Field density tests to determine the level of compaction achieved.

❏ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the above items have been completed in general accordance with our recommendations could be issued following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 STATEMENT OF LIMITATIONS

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

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. Should any conditions at the site be encountered which differ from those at the test locations, we request notification 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 Uniform Developments or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

David J. Gilbert, P.Eng.

Carlos P. Da Silva, P.Eng.

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 113 and 115 Echo Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located along Echo Drive, in front of subject site. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

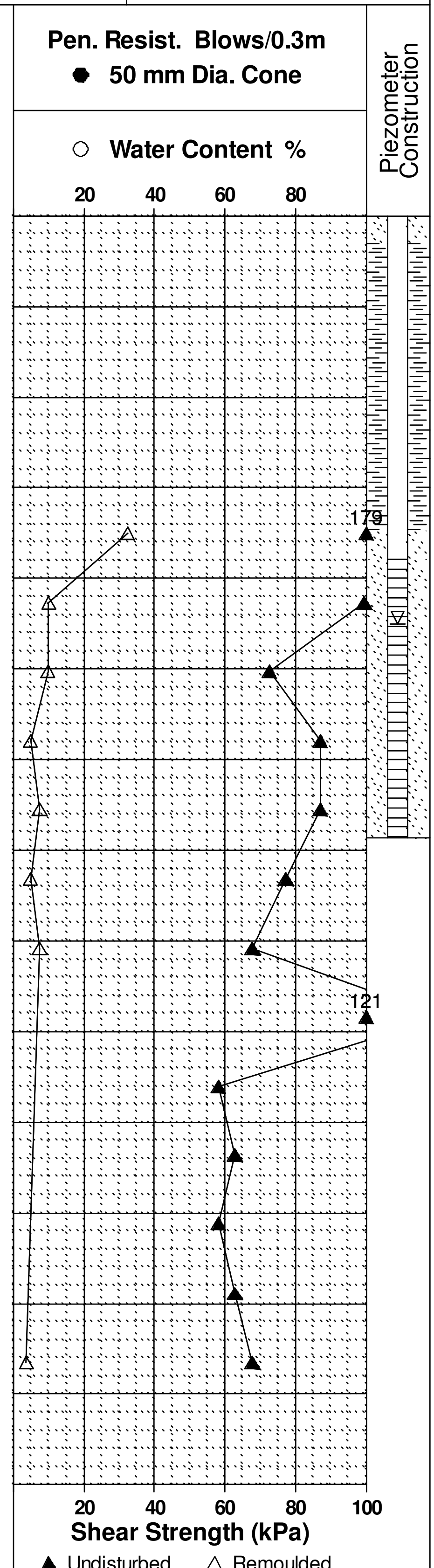
BORINGS BY CME 75 Power Auger

DATE December 4, 2013

FILE NO. PG3142

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.32
50mm Asphaltic concrete over brown silty sand with gravel — 0.69		AU	1				
Very loose, brown **SILTY SAND,** trace gravel — 1.45		SS	2	33	2	1	98.32
Compact, brown **SILTY SAND,** trace clay — 2.21		SS	3	83	15	2	97.32
		SS	4	100	9		
						3	96.32
						4	95.32
Very stiff to stiff, brown **SILTY CLAY**						5	94.32
						6	93.32
- grey-brown by 6.7m depth						7	92.32
						8	91.32
- grey by 9.0m depth						9	90.32
						10	89.32
						11	88.32
— 12.65						12	87.32
Dynamic Cone Penetration Test commenced at 12.65m depth. Cone pushed to 16.5m depth.						13	86.32
						14	85.32



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 113 and 115 Echo Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located along Echo Drive, in front of subject site. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 75 Power Auger

DATE December 4, 2013

FILE NO. PG3142

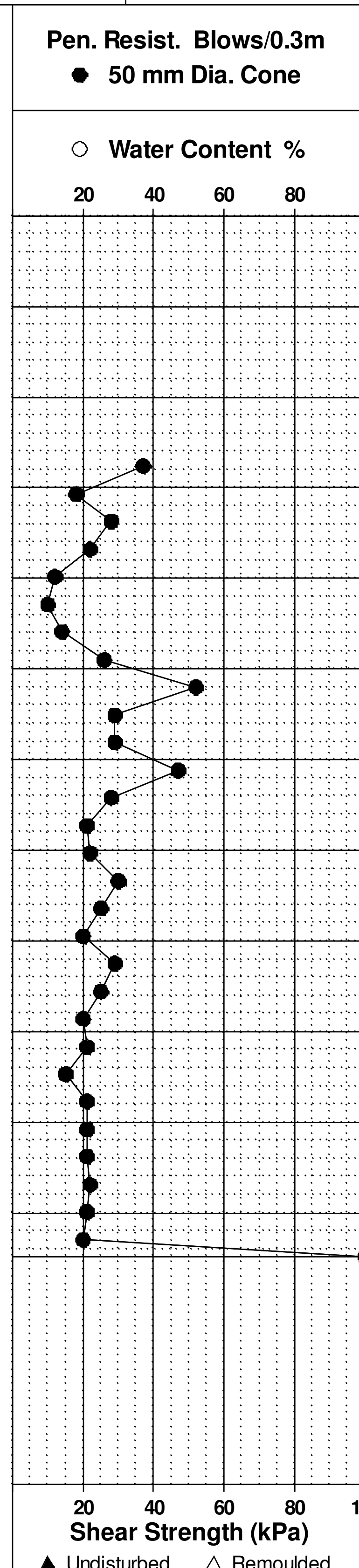
HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						14	85.32
Inferred **SILTY CLAY**						15	84.32
16.50						16	83.32
Inferred **GLACIAL TILL**						17	82.32
						18	81.32
						19	80.32
						20	79.32
						21	78.32
						22	77.32
						23	76.32
						24	75.32
25.48						25	74.32

End of Borehole

Practical DCPT refusal at 25.48m depth.

(GWL @ 4.5m depth based on field observations)



Piezometer Construction

patersongroup Consulting Engineers 154 Colonnade Road South, Ottawa, Ontario K2E 7J5	**SOIL PROFILE AND TEST DATA** **Geotechnical Investigation** **Prop. Multi-Storey Building - 113 and 115 Echo Drive** **Ottawa, Ontario**

DATUM TBM - Top spindle of fire hydrant located along Echo Drive, in front of subject site. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 75 Power Auger **DATE** December 5, 2013

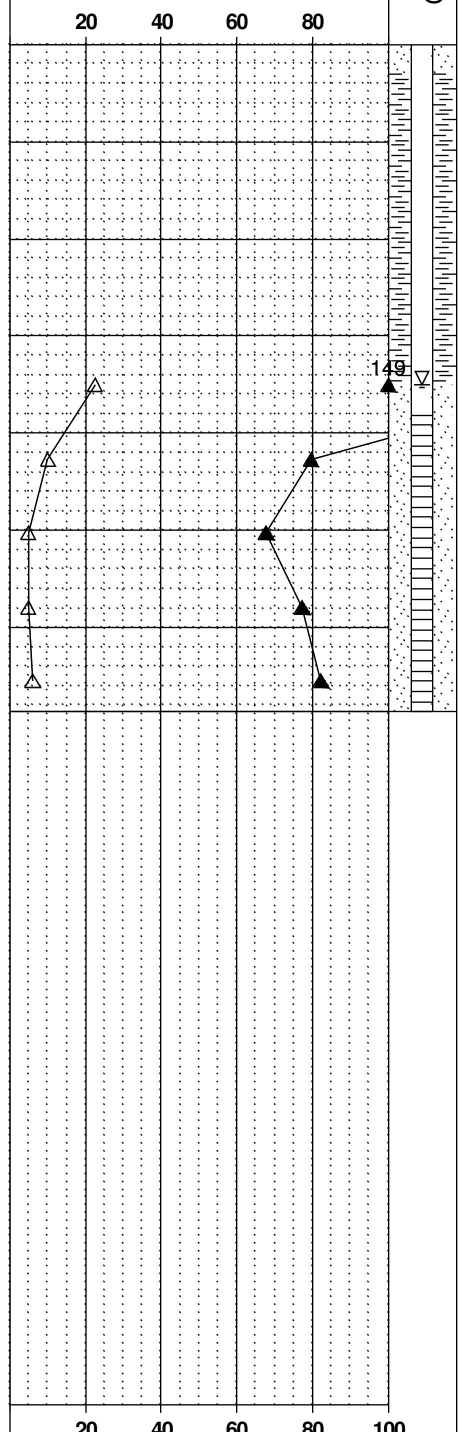
FILE NO. PG3142

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	99.35
50mm Asphaltic concrete over brown silty sand and gravel (to 0.60)	AU	1				
FILL Brown silty sand, trace gravel and brick (to 1.45)	SS	2	42	5	1	98.35
Very stiff to stiff, brown **SILTY CLAY** with sand to 2.2m depth	SS	3	21	6	2	97.35
	SS	4	100	7	3	96.35
					4	95.35
- grey by 5.2m depth					5	94.35
(to 6.86)					6	93.35
End of Borehole						
(GWL @ 3.5m depth based on field observations)						

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located along Echo Drive, in front of subject site. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 75 Power Auger

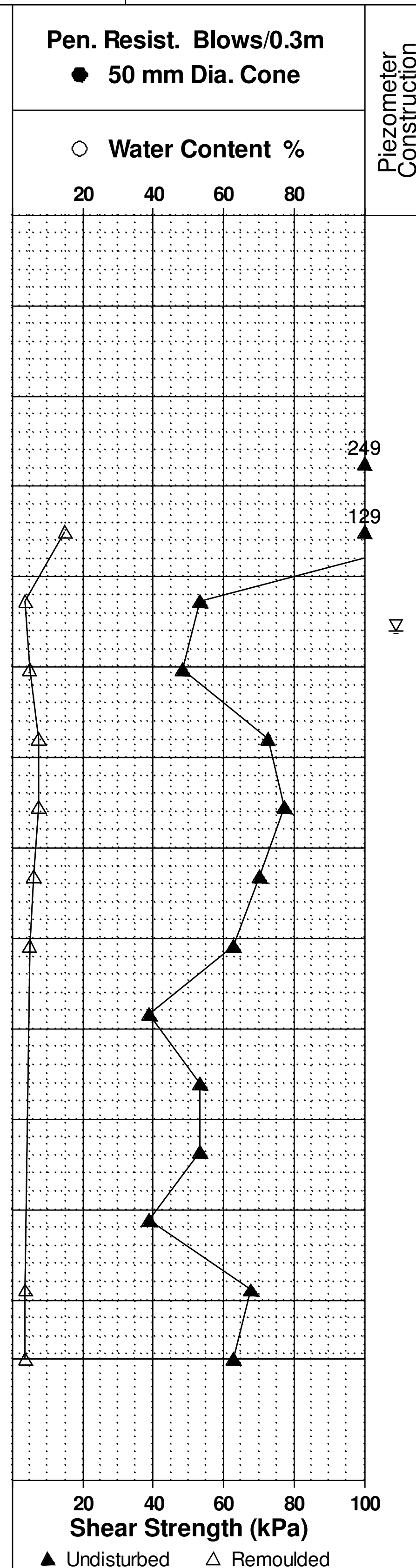
DATE December 5, 2013

FILE NO. PG3142

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
50mm Asphaltic concrete over brown silty sand with gravel (0.30)		AU	1		
FILL: Brown silty sand, trace clay, gravel and wood (1.45)		AU	2		
		SS	3	67	4
Hard to stiff, brown **SILTY CLAY**		SS	4	100	8
- grey-brown by 5.9m depth					
(12.65)					
End of Borehole					
(GWL @ 4.6m depth based on field observations)					

DEPTH (m)	ELEV. (m)
0	99.26
1	98.26
2	97.26
3	96.26
4	95.26
5	94.26
6	93.26
7	92.26
8	91.26
9	90.26
10	89.26
11	88.26
12	87.26



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 113 and 115 Echo Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located along Echo Drive, in front of subject site. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

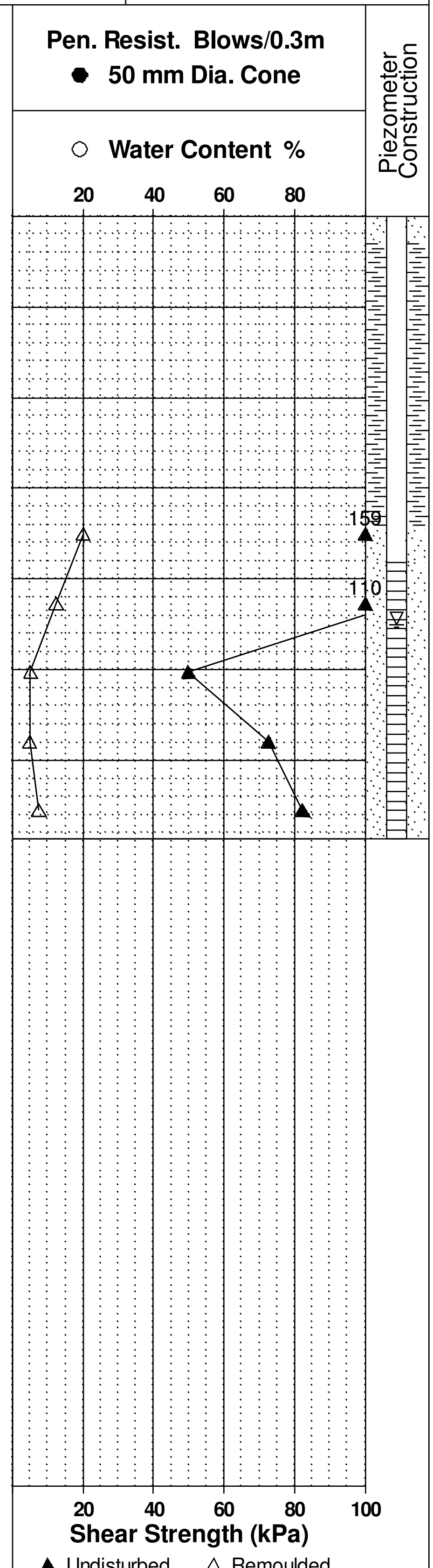
BORINGS BY CME 75 Power Auger

DATE December 5, 2013

FILE NO. PG3142

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.15
50mm Asphaltic concrete over brown silty sand with gravel	0.30	AU	1				
FILL: Brown silty sand with gravel and crushed stone	0.69	AU	2				
FILL: Brown silty sand, some clay trace gravel and cobbles	1.45	SS	3	67	5	1	98.15
Very stiff to stiff, brown SILTY CLAY	6.86	SS	4	100	6	2	97.15
		SS	5	100	4	3	96.15
						4	95.15
						5	94.15
						6	93.15
End of Borehole							
(GWL @ 4.5m depth based on field observations)							



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 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

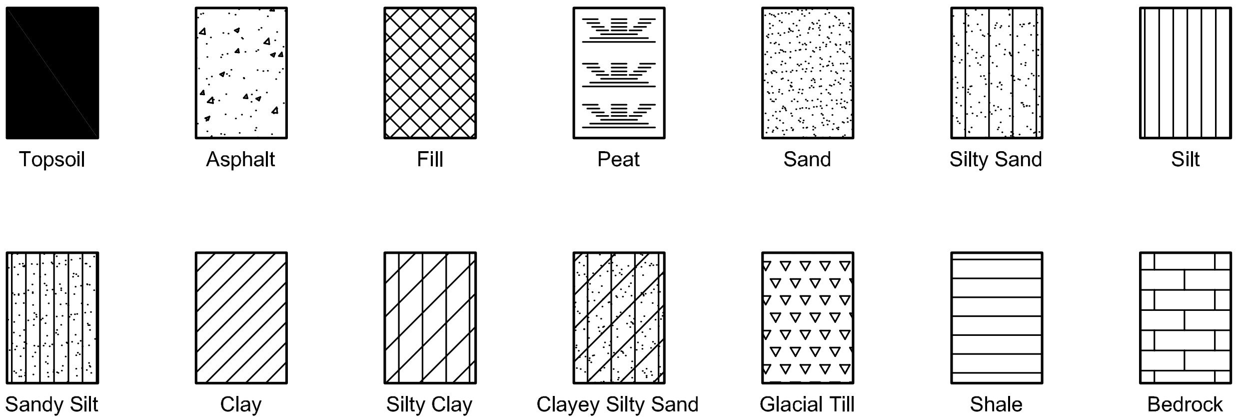
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

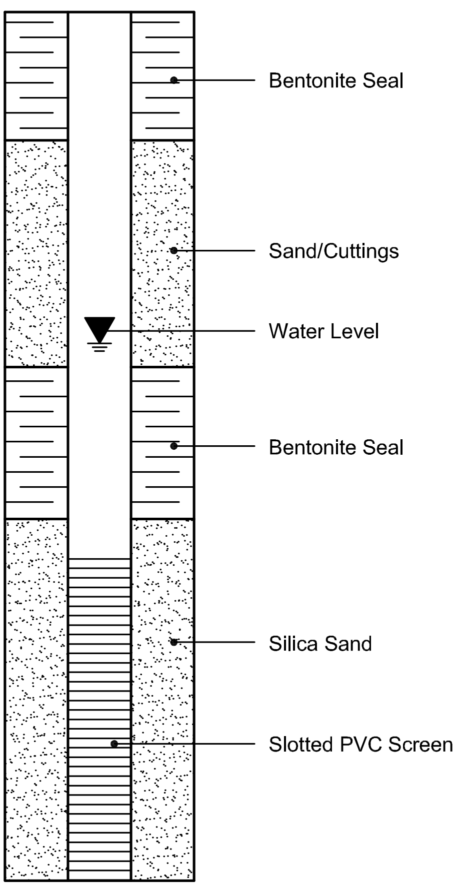
SYMBOLS AND TERMS (continued)

STRATA PLOT

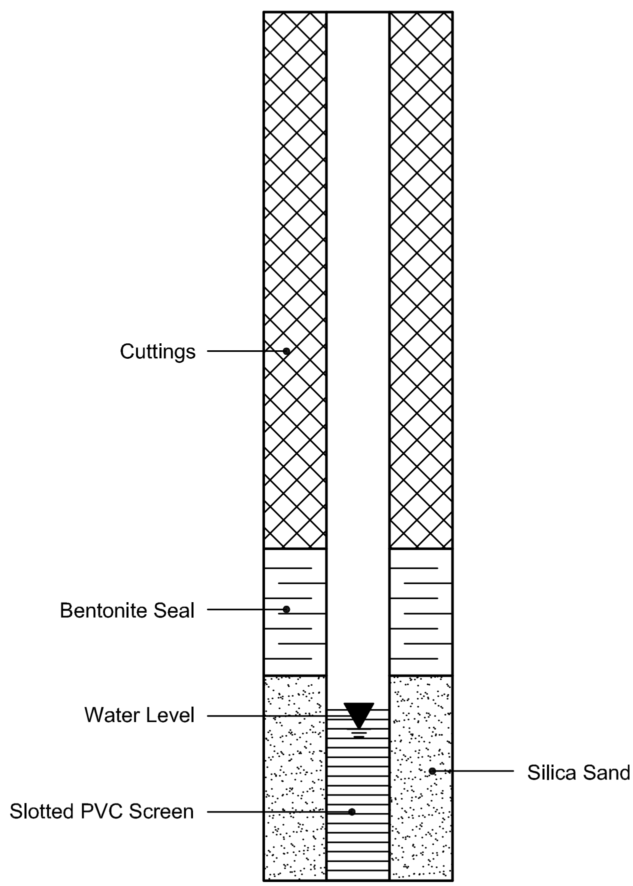


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG3142-1 - TEST HOLE LOCATION PLAN

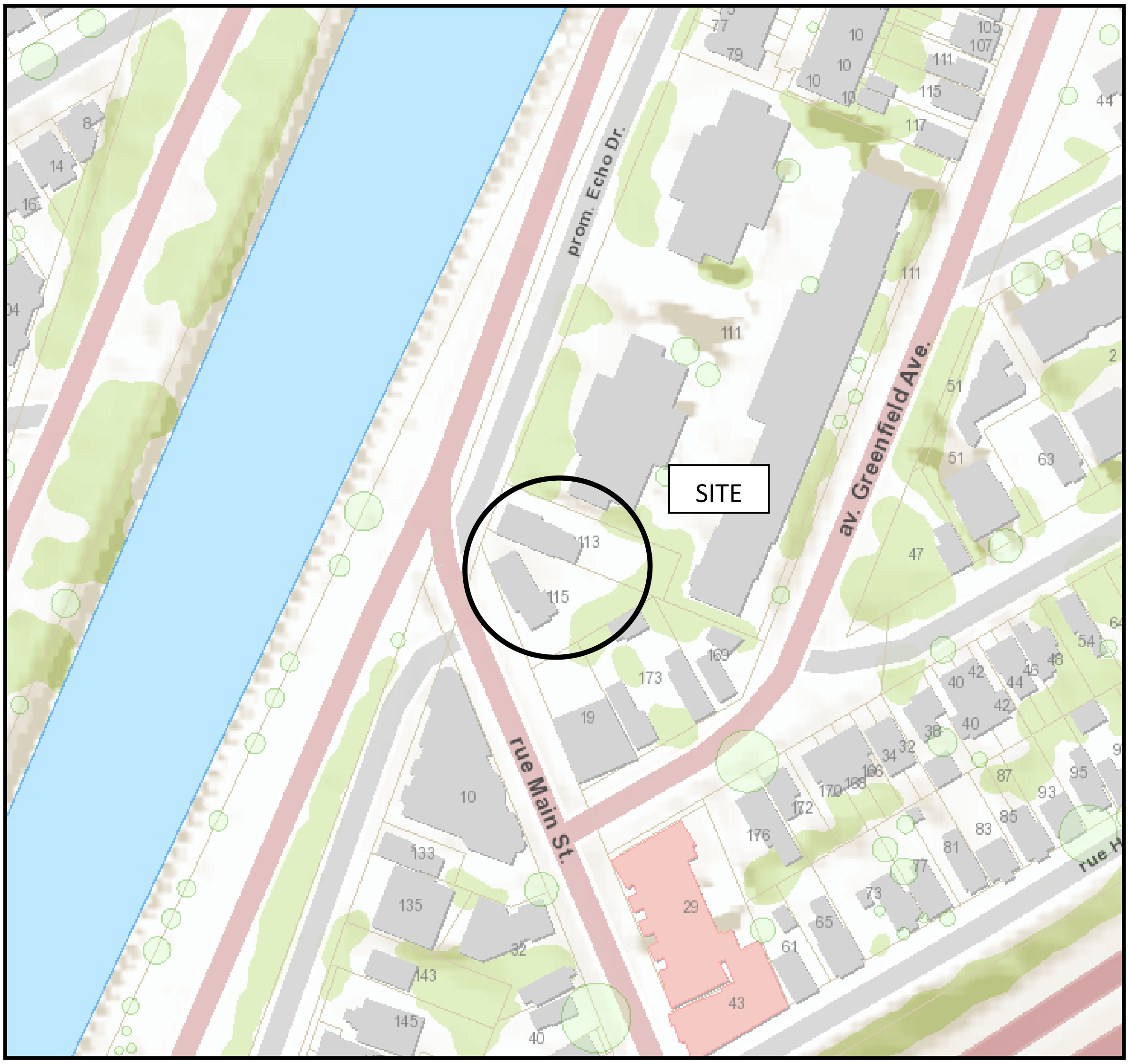
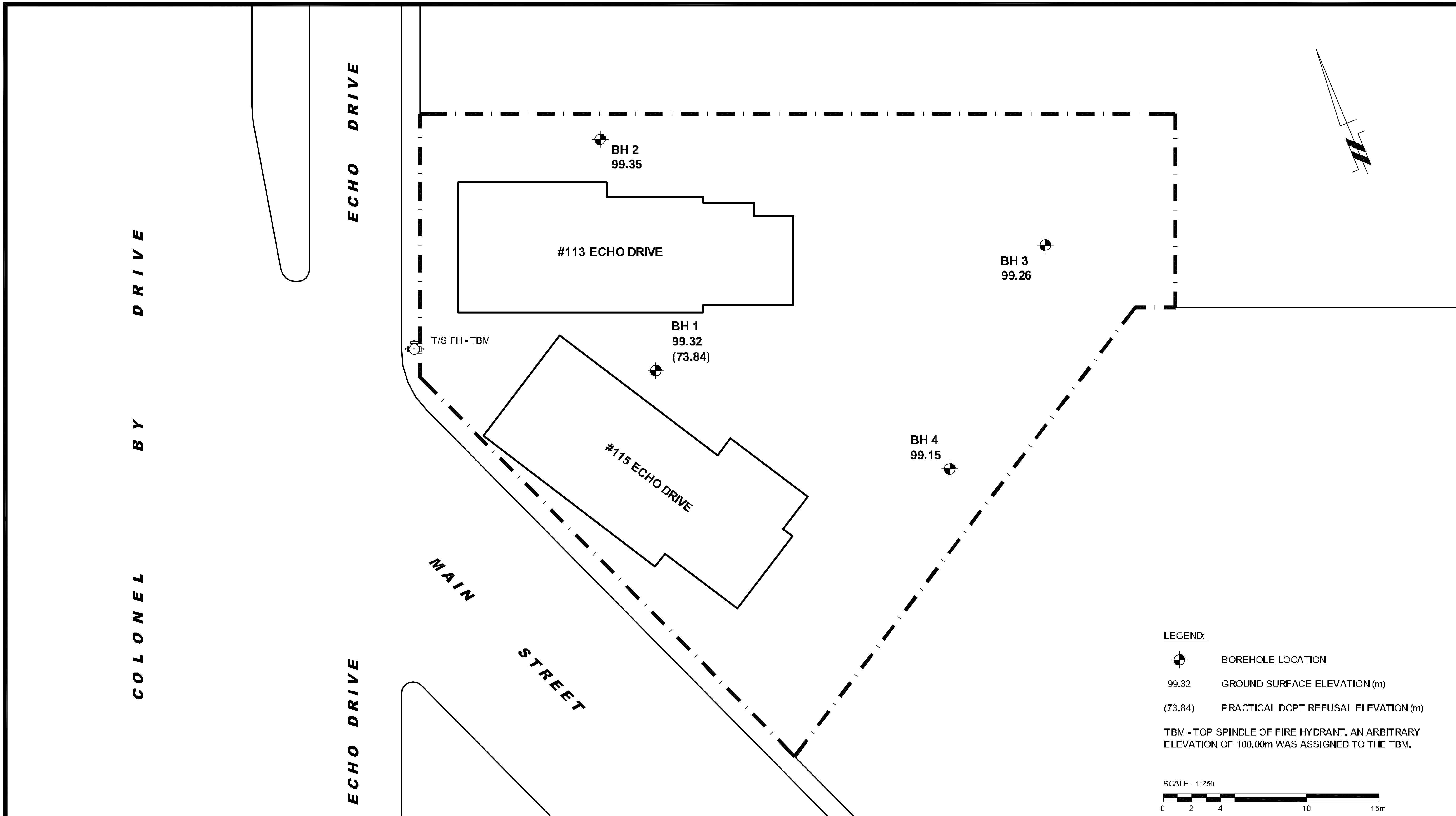


FIGURE 1

KEY PLAN



paterson group consulting engineers 154 Colonnade Road South, Ottawa, Ontario K2E 7J5	Scale: 1:250 Des.: DG Dwn: MPG Chkd: DG	UNIFORM DEVELOPMENTS GEOTECHNICAL INVESTIGATION 113 AND 115 ECHO DRIVE OTTAWA, ONTARIO	**TEST HOLE LOCATION PLAN**	Dwg. No. **PG3142-1** Report No.: PG3142-1 Date: 01/2014