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Supplemental Geotechnical Investigation

Proposed Multi-Storey Building(s)
1034 McGarry Terrace and
1117 Longfields Drive
Ottawa, Ontario

Prepared For

1897365 Ontario Inc.

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Report: PG2846-2

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Table of Contents

Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Figures 2 and 3 - Seismic Shear Wave Velocity Profiles Drawing PG2846-3 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by 1897365 Ontario Inc to undertake a supplemental geotechnical investigation for the proposed multi-storey building(s) to be located at 1034 McGarry Terrace and 1117 Longfields Drive, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

❑ determine the subsoil and groundwater conditions at this site by means of a subsurface investigation consisting of boreholes.

❑ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Development

It is understood that the proposed development will consist of residential multi-storey buildings with four levels of underground parking. It is further understood that car parking and access lanes, as well as, landscaped areas are also anticipated.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for our geotechnical investigation was carried out on February 28 to March 2, 2018. At that time, 4 boreholes were advanced to a maximum depth of 17.7 m below existing grade. Previous investigations were conducted in November 2012, April 2013 and February to March 2015 and consisted of drilling a total of 9 boreholes to a maximum depth of 19.1 m below existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the proposed development taking into consideration underground utilities and site features. The locations of the boreholes are illustrated on Drawing PG2846-3 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights or using a 50 mm diameter split-spoon sampler. All soil samples were classified on site, placed in sealed plastic bags and transported to our laboratory for further review. 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.

Diamond drilling using NQ size coring equipment was carried out at BH 3 (November 2012) after practical auger refusal was encountered at a depth of 6.5 m. The diamond drilling was extended through a glacial till consisting of a dense brown silty sand with gravel, cobbles and boulders and terminated at a depth of 9.4 m in a dense glacial till.

Rock samples were recovered from all the current boreholes (BH1-18 to BH4-18) using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

Subsurface conditions observed at the boreholes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

Flexible polyethylene standpipes were installed in all the previous boreholes to permit the monitoring of groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson personnel in a manner to provide general coverage of the proposed development taking into consideration underground utilities and site features. The ground surface elevations at the borehole locations were reference to a TBM consisting of the top of flange of a fire hydrant located south of McGarry Terrace. A geodetic elevation of 103.10 m was provided to the TBM based on the drawing prepared by Annis, O'Sullivan, Vollebekk Limited. The location of the TBM, boreholes and the ground surface elevations at the borehole locations are presented on Drawing PG2846-3 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

Two (2) soil samples were submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the soil. The analytical test results are presented in Appendix 1 and discussed in Subsection 6.7 of this report.

4.0 Observations

4.1 Surface Conditions

The west portion of the site (1034 McGarry Terrace) is currently occupied by an abandoned one storey building and exposed slab-on-grade building pads, with a relatively flat and level ground surface consisting of granular material and landscaped areas. The east portion of the site (1117 Longfields Drive) is currently vacant, grass covered and slopes gradually down to the east. The east and west portions of the site are separated by a grass covered and forested area, and a slope approximately 4 m high sloping down from west to east.

The subject site is bordered to the west by Longfields Drive, to the south by Marketplace Avenue, to the east by an existing commercial building and to the north by McGarry Terrace, vacant land and an existing multi-storey residential complex with associated outbuildings. It should be noted that an armour stone retaining wall approximately 0.5 to 1.5 m high lies along the southern border of the subject site between the site and Marketplace Avenue. With the exception of the retaining wall, the subject site is generally at grade with the neighbouring properties. It should also be noted that the aforementioned commercial building is located in close proximity to the subject site.

4.2 Subsurface Profile

Generally, the soil conditions encountered at the test hole locations consist of a thin layer of topsoil or fill consisting of asphaltic concrete and granular crushed stone. The topsoil or fill layer in turn overlies a very dense to dense glacial till consisting of brown silty sand with gravel, cobbles and boulders. Practical auger refusal was encountered at several of the borehole locations on inferred boulders. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole.

Bedrock

The upper 3 m of the bedrock was cored at BH1-18 to BH4-18. Weathered sandstone and dolomite bedrock was encountered at depths ranging from 12.6 to 14 m below the existing ground surface. Based on the RQDs of the recovered rock core, the bedrock can be classified as very poor to fair quality in the upper 2 m and good to excellent quality at depth.

Based on available geological mapping, the bedrock in this area consists of interbedded sandstone and dolomite of the March formation with an overburden drift thickness of 10 to 25 m depth.

4.3 Groundwater

The groundwater levels measured in the boreholes are presented in Table 1. Groundwater conditions can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, it is estimated that groundwater can be expected at a 4 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 1	103.00	Damaged	-	December 10, 2012
BH 2	103.81	dry	-	December 10, 2012
BH 3	104.20	2.80	101.40	December 10, 2012
BH 4	98.86	3.52	95.34	April 11, 2013
BH 5	97.90	5.33	92.57	April 11, 2013
BH 6	97.16	Damaged	-	April 11, 2013

Note: The ground surface elevations at the borehole locations were reference to a TBM consisting of the top of spindle of the fire hydrant located south of the McGarry Terrace. A geodetic elevation of 103.75 m was provided to the TBM based on the drawing prepared by Annis, O'Sullivan, Vollebekk Limited.

5.0 Discussion

5.1 Geotechnical Assessment

Based on the results of the geotechnical investigation, the subject site is satisfactory for the proposed multi-storey buildings. The proposed multi-storey building(s) are anticipated to be founded on conventional spread footings placed on the sound bedrock bearing surface.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A, Granular B Type II or select subgrade material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings should be compacted to at least 98% of its 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. It is anticipated that site-excavated material will include large cobbles and boulders and should be well graded to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils, are not suitable as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Footings placed on an undisturbed, dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **250 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **400 kPa**. The 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 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.

Footings placed on a clean, surface sounded sandstone or dolomite bedrock surface can be designed using a factored bearing resistance value at ULS of **1,500 kPa**. Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance value provided herein will be subjected to negligible post-construction total and differential settlements.

A clean, surface sounded bedrock bearing surface should be free of loose material, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

A factored bearing resistance value at ULS of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5, could be designed if founded on sandstone or dolomite bedrock and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footing footprint(s). One drill hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed buildings in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the seismic shear wave interpretation are presented in Appendix 2.

Field Program

The shear wave velocity testing array was placed across the eastern portion of the site, oriented approximately northwest-southeast as presented on Drawing PG2846-3 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface parallel to the geophone array, which creates a polarized shear wave. The hammer shots are repeated between 4 to 8 times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e. striking both sides of the I-Beam). The shot locations are located at 3, 4.5 and 20 m away from the first and last geophones, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the proposed building foundations. 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 bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on the test results, the average glacial till overburden shear wave velocity is **363 m/s** and the bedrock shear wave velocity is **2,467 m/s**. It is understood that the currently proposed design includes four levels of underground parking. The western building (1034 McGarry Terrace) is therefore understood to be founded on glacial till overburden and the eastern building (1117 Longfields Drive) will be founded directly on bedrock. An analysis was completed for both scenarios and the details are presented below.

1034 McGarry Terrace

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the OBC 2012. If the building is founded on glacial till, approximately 6 m above the bedrock surface, the following equation applies:

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

$$V_{s30} = \frac{30m}{\left(\frac{5.5m}{363m/s} + \frac{24.5m}{2{,}467m/s}\right)}$$

$$V_{s30} = 1{,}195m/s$$

It should be noted that if the footings are extended to bedrock or within 3 m of the bedrock surface, the following equation applies:

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

$$V_{s30} = \frac{30m}{\left(\frac{3.0m}{363m/s} + \frac{27.0m}{2{,}467m/s}\right)}$$

$$V_{s30} = 1{,}561m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30} for foundations placed on glacial till and 6 m above the bedrock surface, is 1,195 m/s. Therefore, a **Site Class C** is applicable for design of the proposed building, as per Table 4.1.8.1.A of the OBC 2012. Alternatively, if footings are extended to bedrock or within 3 m of the bedrock surface, the Vs_{30} would be 1,561 m/s, and a **Site Class A** would apply.

1117 Longfields Drive

It is understood that the proposed building will be founded directly on bedrock. Therefore, the following equation applies:

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

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2{,}467m/s}\right)}$$

$$V_{s30} = 2{,}467m/s$$

Based on the results of the sear wave velocity testing, the average shear wave velocity, Vs_{30} for foundations placed directly on bedrock is 2,467 m/s. Therefore, a **Site Class A** is applicable for design of the proposed building.

The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

It is expected that the basement area will be mostly parking and that a rigid concrete pavement structure with a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor. 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.

5.6 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.

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 presented in Tables 2 and 3

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 3 - Recommended Pavement Structure - Access Lanes

Thickness mm	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil, select subgrade material 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 material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is understood that the building foundation walls will be placed in close proximity to all the property boundaries. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the following system is recommended:

☐ A waterproofing membrane will be required to lessen the effect of water infiltration for the basement levels starting at 4 m below finished grade. The waterproofing membrane can be placed and fastened to the shoring system (expected to be a soldier pile and timber lagging) and should extend to the bottom of the excavation at the founding level of proposed footings.

☐ A composite drainage layer will be placed from the ground surface elevation to the founding level.

It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves placed at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 to 9 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where required, 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, such as Delta Drain 6000, 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.

Adverse Effects from Dewatering on Adjacent Structures

Based on the review of the general founding conditions existing structures surrounding the subject site and based on the proposed foundation drainage program being suggested above, no adverse effects from temporary dewatering are expected for the subject site and adjacent structures.

6.2 Protection of Footings Against Frost Action

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

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. The recommended minimum thickness of soil cover is 2.1 m (or equivalent).

6.3 Excavation Side Slopes and Temporary Shoring

At this site, temporary shoring may be required to complete the required excavations. However, it is recommended that where sufficient room is available, open cut excavation in combination with temporary shoring can be used.

Excavation Side Slopes

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

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

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

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

Temporary Shoring

The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. However, due to the bouldery glacial till layer, the site may not be suitable for 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. Earth pressures acting on the shoring system may be calculated using the parameters provided in Table 4.

Table 4 - Soil Parameters

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

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 $0.65 \cdot K \cdot \gamma \cdot H$ for strutted or anchored shoring, or a triangular earth pressure distribution with a maximum value of $K \cdot \gamma \cdot H$ 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

Bedding and backfill materials should be in accordance with City of Ottawa standards and specifications.

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 its SPMDD. The bedding material should extent 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 its 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 material’s 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.

It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

Groundwater Control for Building Construction

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MOECC.

Impacts on Neighbouring Properties

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 adverse effects to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur. In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Precautions must be taken where excavations are carried out in proximity to existing structures which may be adversely affected due to freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract documents to protect the walls of the excavation from freezing, if applicable.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

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

It is recommended that the following be carried out once the site development plans are finalized and during site development:

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

❑ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.

❑ 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.

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

❑ Inspection of below grade waterproofing installation.

❑ 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. We request permission to review our recommendations when the 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. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well as the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than 1897365 Ontario Inc. 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.

Nathan F. S. Christie, P.Eng.

LICENSED PROFESSIONAL ENGINEER
C. P. DA SILVA
PROVINCE OF ONTARIO

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

Report Distribution

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 1034 McGarry Terrace
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE November 19, 2012

FILE NO. PG2846

HOLE NO. BH 1

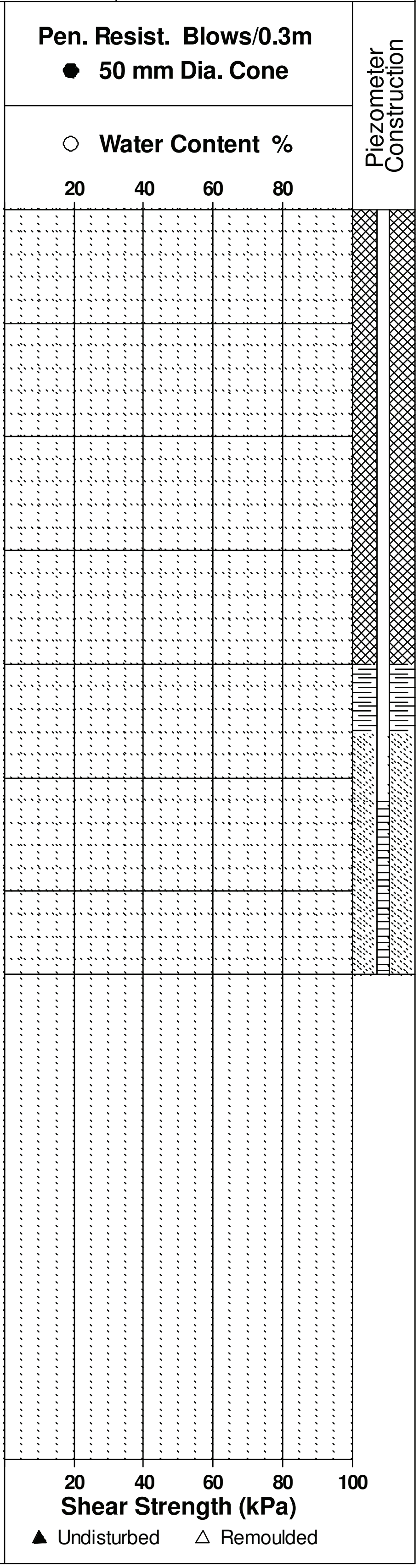
SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
50mm Asphaltic concrete over crushed stone	0.15	AU	1		
FILL: Brown silty sand with gravel	0.69	AU	2		
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders, some to trace clay		SS	3	75	9
		SS	4	83	13
		SS	5	87	44
		SS	6	60	50+
		SS	7	50	60
		SS	8	83	35
		SS	9	40	50+
	6.73	SS	10	75	75

DEPTH (m)	ELEV. (m)
0	103.00
1	102.00
2	101.00
3	100.00
4	99.00
5	98.00
6	97.00

End of Borehole

Practical refusal to augering at 6.73m depth

(Piezometer blocked at 5.03m depth - Dec. 10, 2012)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 1034 McGarry Terrace
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE November 19, 2012

FILE NO. PG2846

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	103.81		
75mm Asphaltic concrete over crushed stone 0.18									
FILL: Brown silty sand with gravel 0.60		AU	1						
		SS	3	100	34	1	102.81		
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders, some to trace clay		SS	4	100	50+				
						2	101.81		
		SS	5	55	50+				
						3	100.81		
3.28		SS	6	100	50+				
End of Borehole Practical refusal to augering at 3.28m depth (BH dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 1034 McGarry Terrace
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

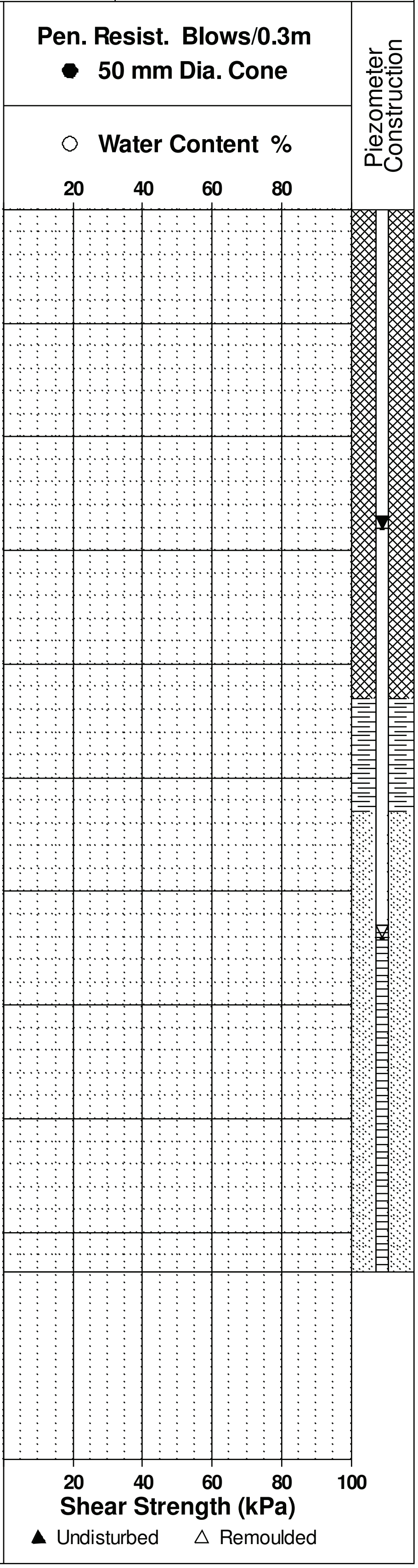
BORINGS BY CME 55 Power Auger

DATE November 19, 2012

FILE NO. PG2846

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
75mm Asphaltic concrete over crushed stone 0.20	AU	1			0	104.20
FILL: Brown silty sand with gravel 0.69	AU	2				
	SS	3	83	9	1	103.20
	SS	4	100	36	2	102.20
	SS	5	100	67		
					3	101.20
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders, some to trace clay	SS	6	71	50+		
	SS	7	100	50+	4	100.20
	SS	8	100	80	5	99.20
	SS	9	100	54	6	98.20
	SS	10		50+		
	RC	1			7	97.20
					8	96.20
Practical auger refusal at 6.53m depth. Extended diamond drill coring equipment to a depth of 9.35m depth through very dense glacial till.	RC	2				
					9	95.20
9.35						
End of Borehole						
(GWL @ 6.4m depth based on field observations)						
(GWL @ 2.80m-Dec. 10, 2012)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 1034 McGarry Terrace
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS GPS: 18T 0442057 5013442

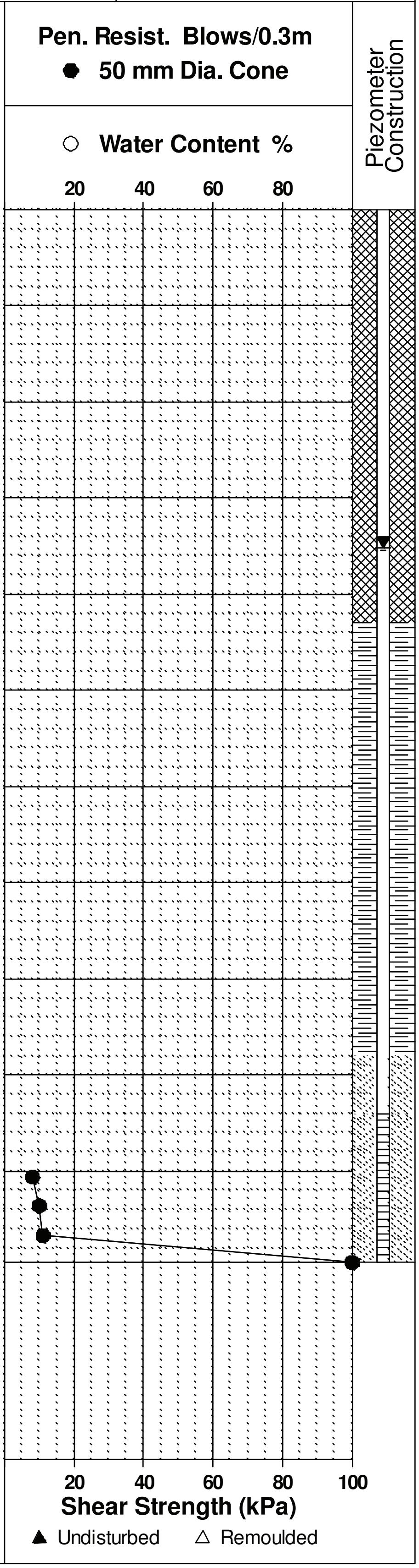
BORINGS BY CME 55 Power Auger

DATE April 3, 2013

FILE NO. PG2846

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL 0.30		AU	1			0	98.86
SILTY CLAY with sand and gravel 0.69		AU	2				
		SS	3	59	78	1	97.86
		SS	4	100	50+		
						2	96.86
						3	95.86
		SS	5	71	70		
						4	94.86
GLACIAL TILL: Very dense grey silty sand with gravel, cobbles and boulders		SS	6	11	73		
						5	93.86
						6	92.86
		SS	7	42	33		
						7	91.86
		SS	8	75	51		
						8	90.86
						9	89.86
9.75		SS	9	100	11		
Dynamic Cone Penetration Test commenced at 9.75m depth.						10	88.86
10.95							
End of Borehole Practical DCPT refusal at 10.95m depth (GWL @ 3.52m-April 11, 2013)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 1034 McGarry Terrace
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS GPS: 18T 0442111 5013454

BORINGS BY CME 55 Power Auger

DATE April 3, 2013

FILE NO. PG2846

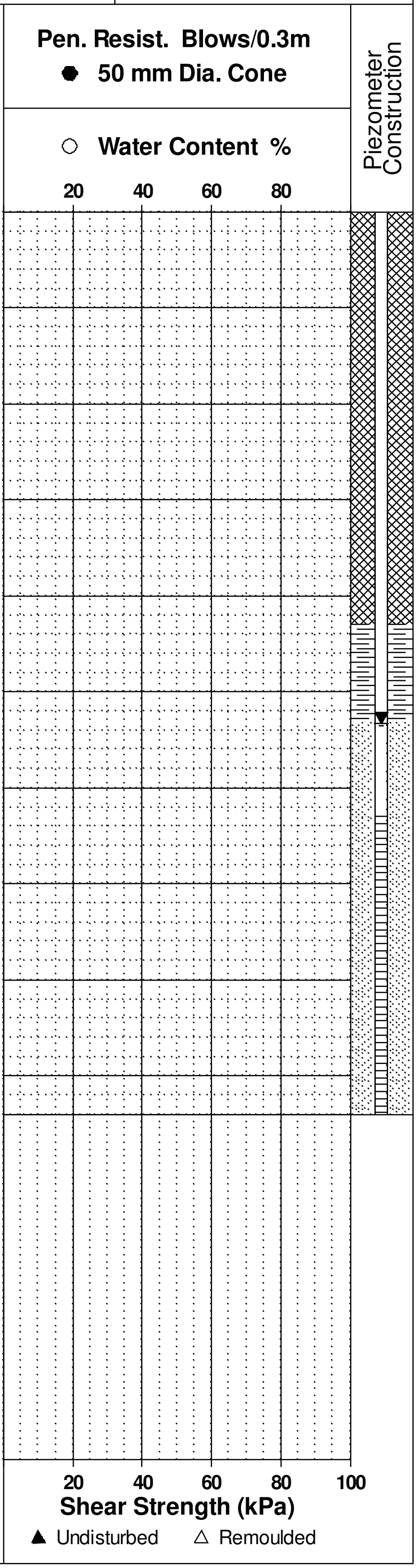
HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	97.90
TOPSOIL 0.30						
FILL: Brown silty sand with trace clay and gravel	AU	1				
	SS	2	50	16	1	96.90
2.21	SS	3	42	14	2	95.90
Stiff to very stiff, brown **SILTY CLAY**	SS	4	100	8	3	94.90
3.66	SS	5	100	7		
GLACIAL TILL: Compact to very dense, grey silty clay with sand, gravel, cobbles and boulders	SS	6	0	17	4	93.90
	SS	7	83	7	5	92.90
					6	91.90
	SS	8	54	17	7	90.90
	SS	9	74	40	8	89.90
					9	88.90
9.40	SS	10		50+		

End of Borehole

Practical refusal to augering @ 9.40m depth

(GWL @ 5.33m-April 11, 2013)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 1034 McGarry Terrace
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS GPS: 18T 0442101 5013427

BORINGS BY CME 55 Power Auger

DATE April 3, 2013

FILE NO. PG2846

HOLE NO. BH 6

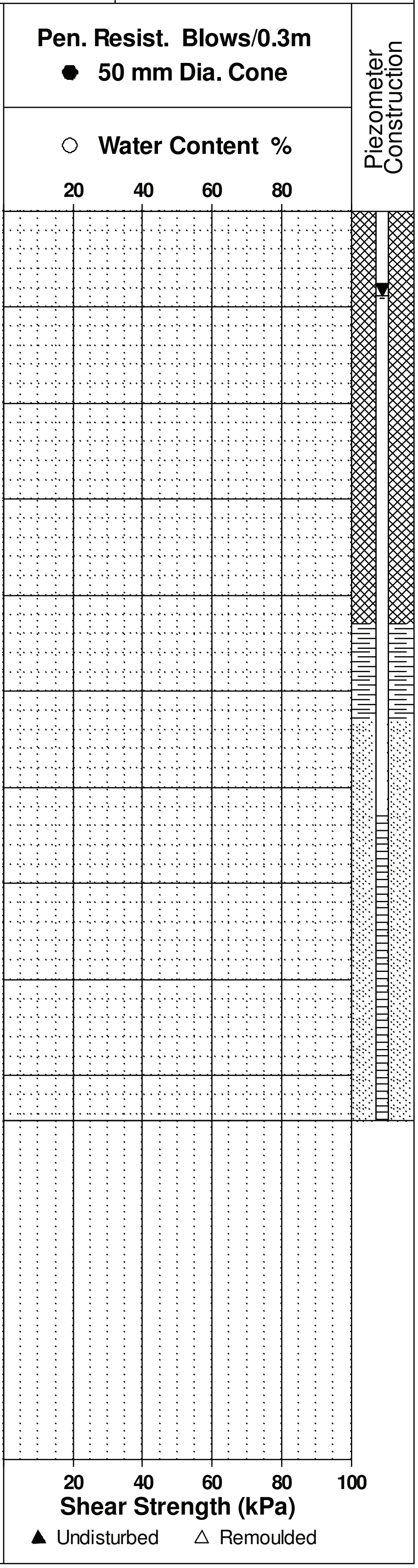
SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30	AU	1		
Very stiff, brown **SILTY CLAY**	1.45	SS	2	67	20
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders, trace clay	9.47	SS	3	83	34
		SS	4	0	50+
		SS	5	0	50+
		SS	6	42	10
		SS	7	50	6
		SS	8		50+

DEPTH (m)	ELEV. (m)
0	98.16
1	97.16
2	96.16
3	95.16
4	94.16
5	93.16
6	92.16
7	91.16
8	90.16
9	89.16

End of Borehole

Practical refusal to augering @ 9.49m depth

(GWL @ 0.88m-April 11, 2013)



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 18, 2015

FILE NO. PG2846

HOLE NO. BH 7

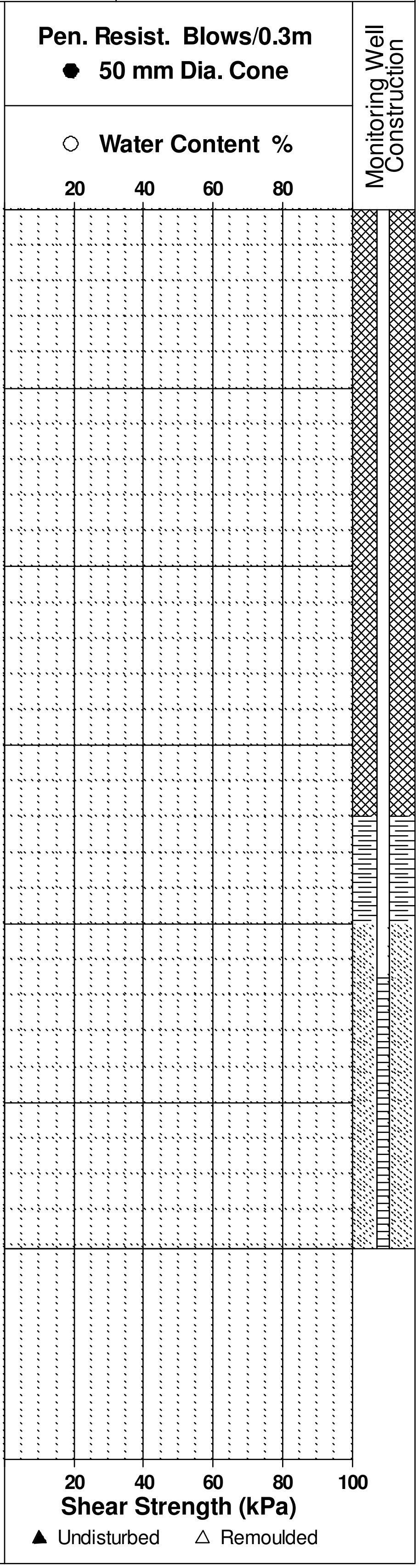
SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Brown silty fine sand with crushed stone, gravel, trace asphalt	0.60	AU	1		
FILL: Brown silty fine sand with gravel, trace topsoil	1.37	SS	2	33	13
FILL: Brown silty clay with sand, trace gravel and wood	1.93	SS	3	75	50+
GLACIAL TILL: Very dense to compact, brown silty fine sand with gravel, cobbles and boulders, trace clay	5.82	SS	4	60	50+
		SS	5	94	50+
		SS	6	88	62
		SS	7	96	48
		SS	8	95	23

End of Borehole

Practical refusal to augering at 5.82m depth

(Borehole dry to 5.9m depth on March 4, 2015)

DEPTH (m)	ELEV. (m)
0	103.55
1	102.55
2	101.55
3	100.55
4	99.55
5	98.55



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 18, 2015

FILE NO. PG2846

HOLE NO. BH 8A

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Monitoring Well Construction
GROUND SURFACE								20 40 60 80	
						0	103.68		
Asphaltic concrete 0.10									
FILL: Brown silty fine sand with crushed stone 0.46		AU	1						
FILL: Black silty fine sand with topsoil and gravel, trace cobbles 0.97		AU SS	2 3		 50+				
						1	102.68		
FILL: Coarse gravel with cobbles, trace boulders 1.65									
End of Borehole Practical refusal to augering at 1.65m depth									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 19, 2015

FILE NO. PG2846

HOLE NO. BH 8B

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	103.68		
Asphaltic concrete 0.10									
FILL: Brown silty fine sand with crushed stone 0.46									
FILL: Black silty fine sand with topsoil and gravel, trace cobbles 0.97						1	102.68		
FILL: Coarse gravel with cobbles, trace boulders 1.62		SS	1	100	50+				
End of Borehole Practical refusal to augering at 1.62m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 19, 2015

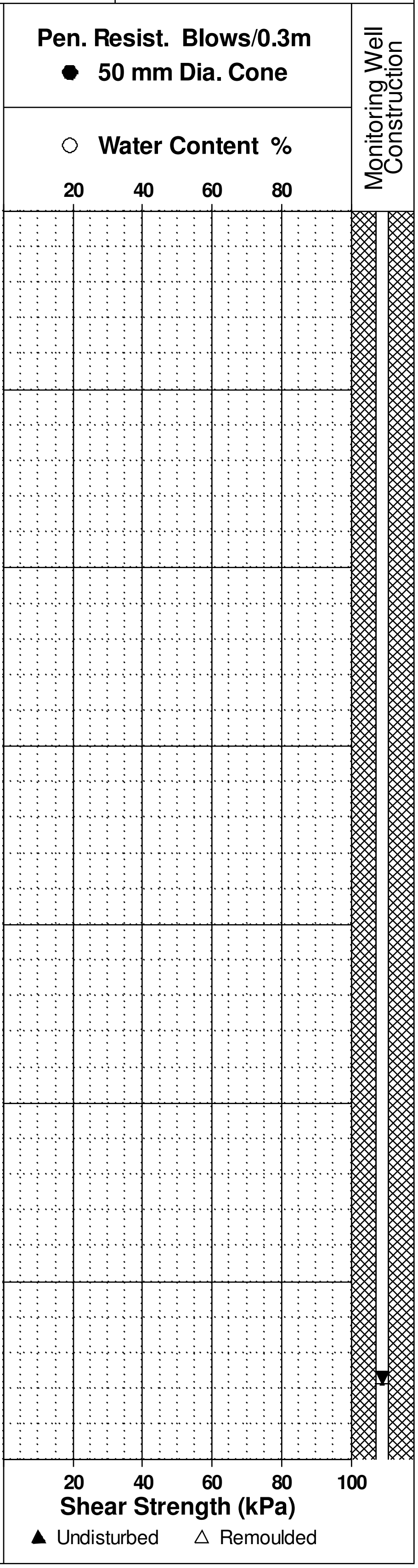
FILE NO. PG2846

HOLE NO. BH 8C

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty fine sand with crushed stone	0.46
FILL: Black silty fine sand with topsoil and gravel, trace cobbles	0.97
FILL: Coarse gravel with cobbles, trace boulders	1.68
GLACIAL TILL: Very dense to compact, brown silty fine sand with gravel, cobbles and boulders, trace clay	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	92	61
SS	3	100	91
SS	4	96	76
SS	5	100	76
SS	6	75	58

DEPTH (m)	ELEV. (m)
0	103.68
1	102.68
2	101.68
3	100.68
4	99.68
5	98.68
6	97.68
7	96.68



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 19, 2015

FILE NO. PG2846

HOLE NO. BH 8C

SOIL DESCRIPTION

GROUND SURFACE

GLACIAL TILL: Very dense to compact, brown silty fine sand with gravel, cobbles and boulders, trace clay

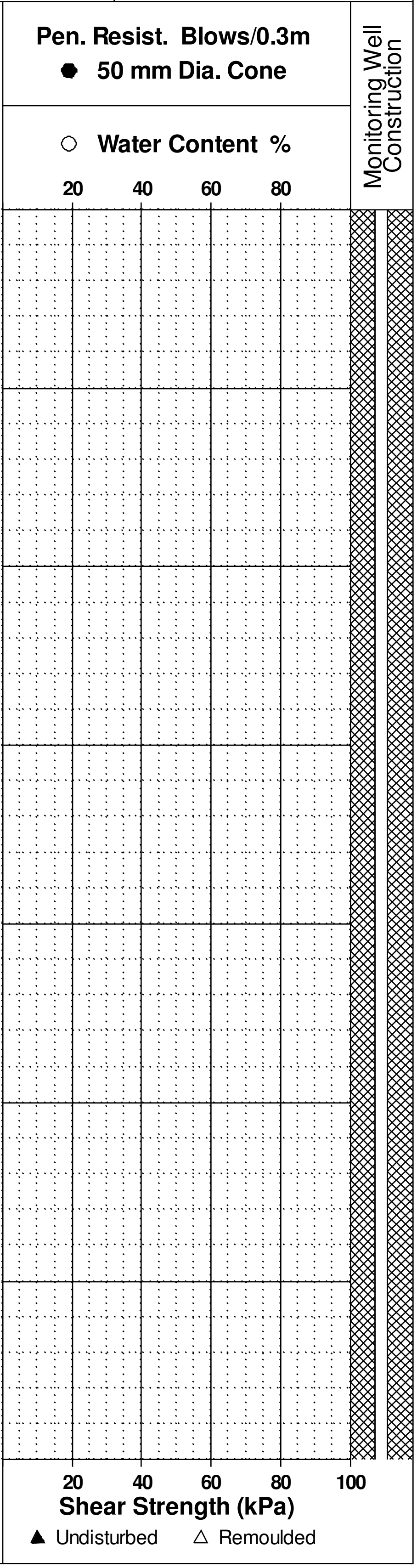
- grey by 11.6m depth

13.40

GLACIAL TILL: Dense to compact, grey silty sand with gravel

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	7	58	33
SS	8	67	27
SS	9	50	54
SS	10	62	41

DEPTH (m)	ELEV. (m)
7	96.68
8	95.68
9	94.68
10	93.68
11	92.68
12	91.68
13	90.68
14	89.68



SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

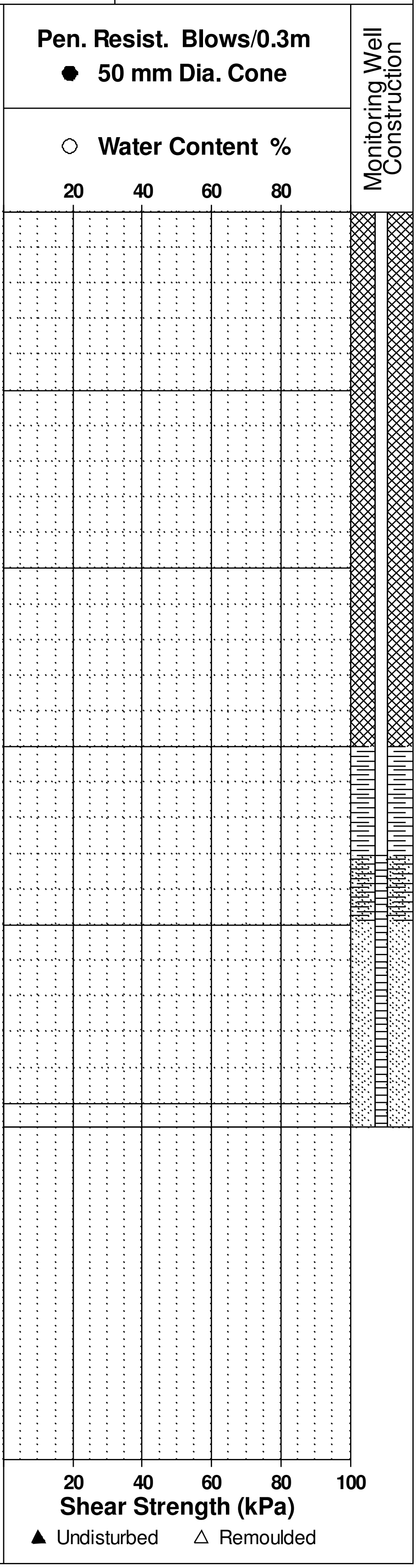
BORINGS BY CME 55 Power Auger

DATE February 19, 2015

FILE NO. PG2846

HOLE NO. BH 8C

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
- some running sand by 13.7m depth		SS	11	25	44	14	89.68
						15	88.68
GLACIAL TILL: Dense to compact, grey silty sand with gravel		SS	12	75	11		
						16	87.68
						17	86.68
17.98							
						18	85.68
BEDROCK: Sandstone interbedded with dolomite		RC	1	91	56		
		RC	2			19	84.68
19.13							
End of Borehole (GWL @ 6.57m-March 4, 2015)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

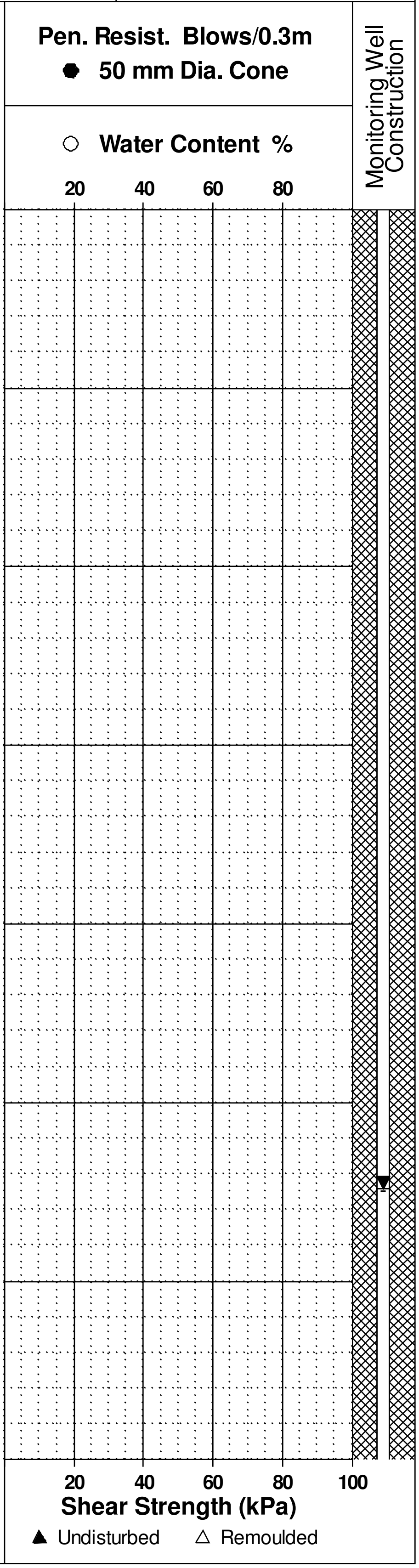
BORINGS BY CME 55 Power Auger

DATE March 2, 2015

FILE NO. PG2846

HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.02
						1	99.02
						2	98.02
OVERBURDEN						3	97.02
						4	96.02
						5	95.02
						6	94.02
						7	93.02



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 2, 2015

FILE NO. PG2846

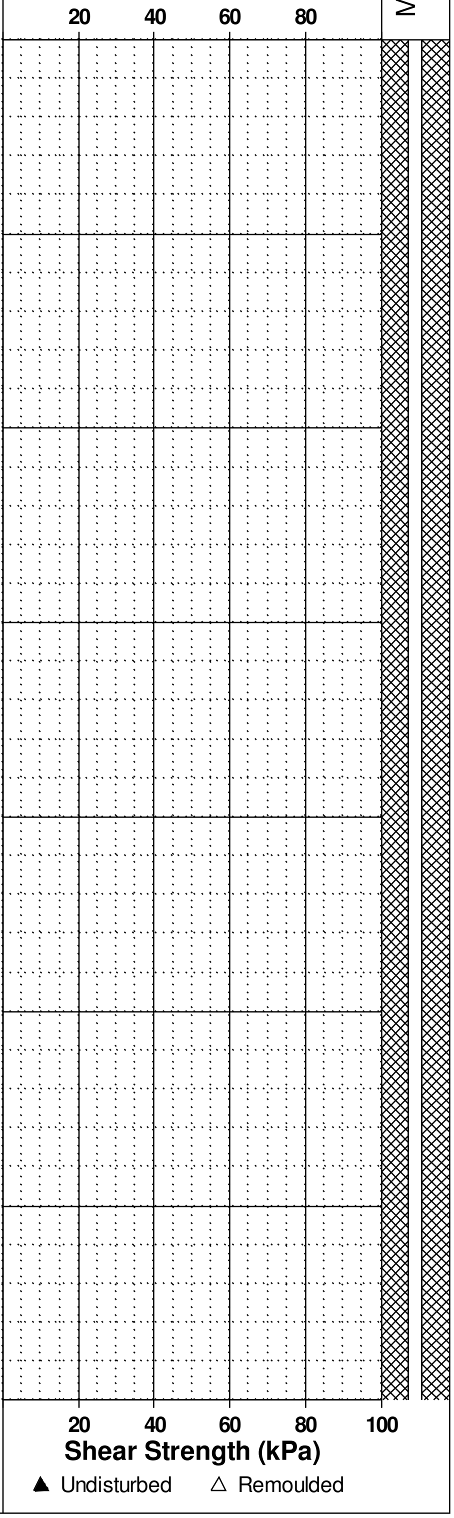
HOLE NO. BH 9

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
OVERBURDEN						7	93.02
						8	92.02
						9	91.02
						10	90.02
						11	89.02
						12	88.02
						13	87.02
						14	86.02

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

○ Water Content %

Monitoring Well Construction



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant. Assumed geodetic elevation = 103.75m as per plan provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

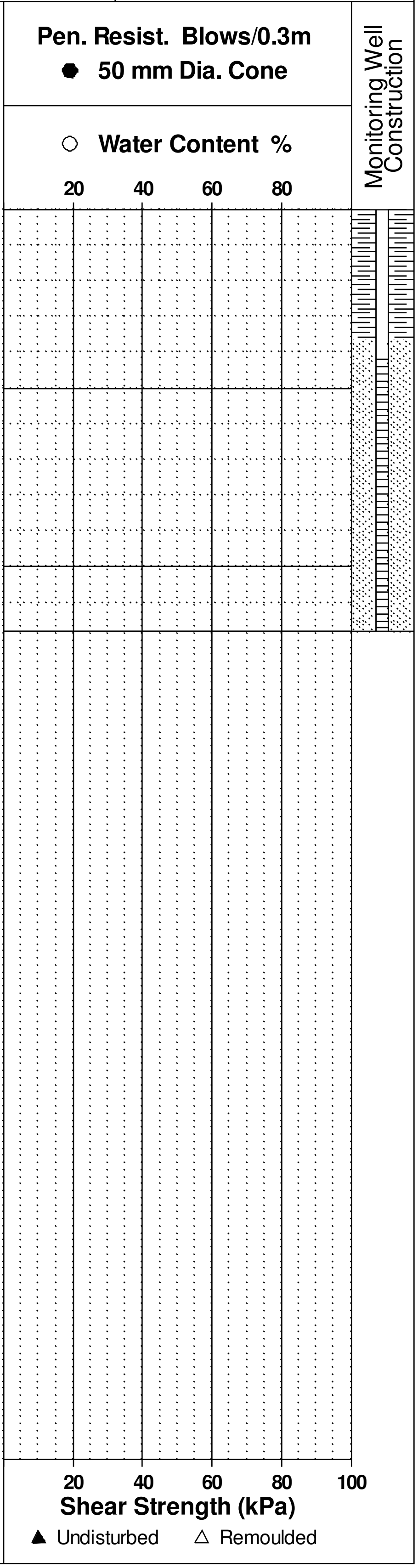
BORINGS BY CME 55 Power Auger

DATE March 2, 2015

FILE NO. PG2846

HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						14	86.02
OVERBURDEN							
14.73							
BEDROCK: Grey sandstone interbedded with dolomite		RC	1	100	0		
						15	85.02
		RC	2	100	50		
						16	84.02
16.36							
End of Borehole (GW L@ 5.48m-March 4, 2015)							



SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top of flange of fire hydrant located at the end of McGarry Terrace. Geodetic elevation = 103.10m, as per plan provided by City of Ottawa.

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 28, 2018

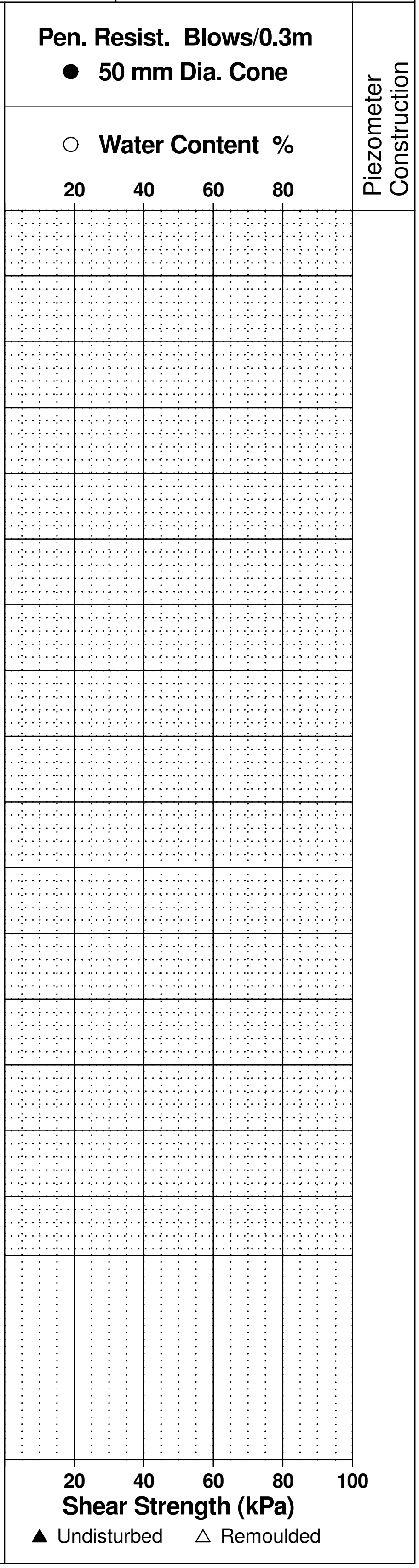
FILE NO. PG2846

HOLE NO. BH 1-18

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	0
OVERBURDEN	1.52
Inferred **SILTY CLAY**	3.35
Inferred **GLACIAL TILL**	12.60
BEDROCK: Interbedded sandstone and dolostone	15.90
End of Borehole	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
RC	1	15	
RC	2	7	
RC	3	10	
RC	4	27	
RC	5	44	
RC	6	79	57
RC	7	100	100

DEPTH (m)	ELEV. (m)
0	98.02
1	97.02
2	96.02
3	95.02
4	94.02
5	93.02
6	92.02
7	91.02
8	90.02
9	89.02
10	88.02
11	87.02
12	86.02
13	85.02
14	84.02
15	83.02



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top of flange of fire hydrant located at the end of McGarry Terrace. Geodetic elevation = 103.10m, as per plan provided by City of Ottawa.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 1, 2018

FILE NO. PG2846

HOLE NO. BH 2-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	97.84		
OVERBURDEN									
						1	96.84		
1.52									
						2	95.84		
						3	94.84		
		RC	1	26					
						4	93.84		
						5	92.84		
						6	91.84		
						7	90.84		
Inferred **GLACIAL TILL**		RC	2	9					
						8	89.84		
						9	88.84		
						10	87.84		
		RC	3	17					
						11	86.84		
						12	85.84		
		RC	4	12					
13.11						13	84.84		
		RC	5	100	24				
						14	83.84		
BEDROCK: Interbedded sandstone and dolostone						15	82.84		
		RC	6	97	74				
16.05						16	81.84		
End of Borehole									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top of flange of fire hydrant located at the end of McGarry Terrace. Geodetic elevation = 103.10m, as per plan provided by City of Ottawa.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 1, 2018

FILE NO. PG2846

HOLE NO. BH 3-18

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
OVERBURDEN	1.52
Inferred **GLACIAL TILL**	14.00
BEDROCK: Interbedded sandstone and dolostone	17.68
End of Borehole	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
RC	1	26	
RC	2	31	
RC	3	33	
RC	4	53	
RC	5	23	
RC	6	20	
RC	7	62	48
RC	8	92	32
RC	9	93	93

DEPTH (m)	ELEV. (m)
0	98.92
1	97.92
2	96.92
3	95.92
4	94.92
5	93.92
6	92.92
7	91.92
8	90.92
9	89.92
10	88.92
11	87.92
12	86.92
13	85.92
14	84.92
15	83.92
16	82.92
17	81.92

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

○ **Water Content %**

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Piezometer Construction

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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
1034 McGarry Terrace and 1117 Longfields Drive
Ottawa, Ontario

DATUM TBM - Top of flange of fire hydrant located at the end of McGarry Terrace. Geodetic elevation = 103.10m, as per plan provided by City of Ottawa.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 2, 2018

FILE NO. PG2846

HOLE NO. BH 4-18

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
OVERBURDEN				
1.52				
	RC	1	22	
Inferred **GLACIAL TILL**	RC	2	20	
	RC	3	46	
	RC	4	8	
13.05				
BEDROCK: Interbedded sandstone and dolostone	RC	5	97	61
	RC	6	100	76
16.08				
End of Borehole				

DEPTH (m)	ELEV. (m)
0	98.83
1	97.83
2	96.83
3	95.83
4	94.83
5	93.83
6	92.83
7	91.83
8	90.83
9	89.83
10	88.83
11	87.83
12	86.83
13	85.83
14	84.83
15	83.83
16	82.83

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

○ **Water Content %**
20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))

TW - Thin wall tube or Shelby tube

PS - Piston sample

AU - Auger sample or bulk sample

WS - Wash sample

RC - Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

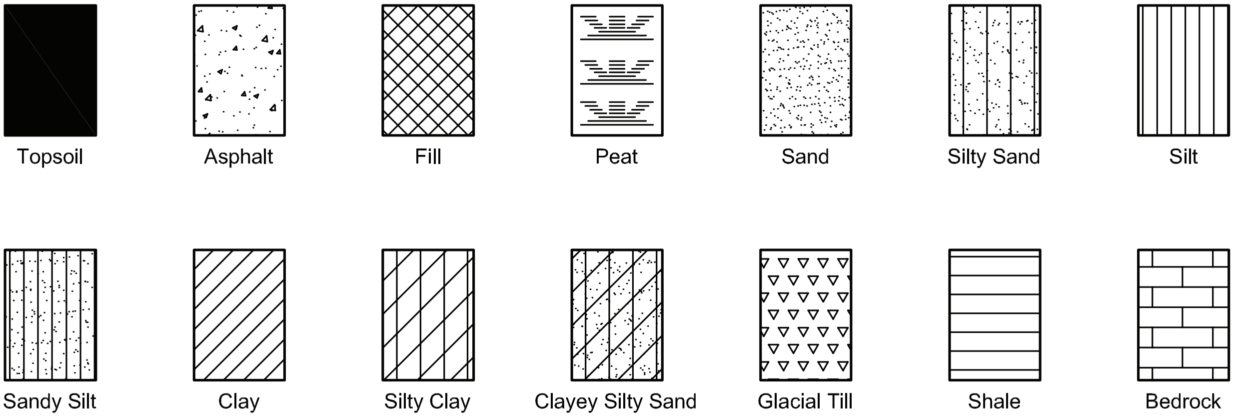
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

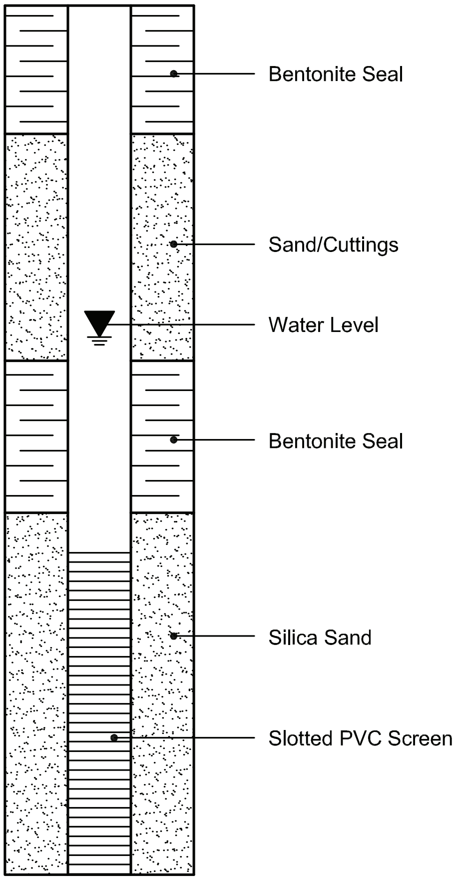
SYMBOLS AND TERMS (continued)

STRATA PLOT

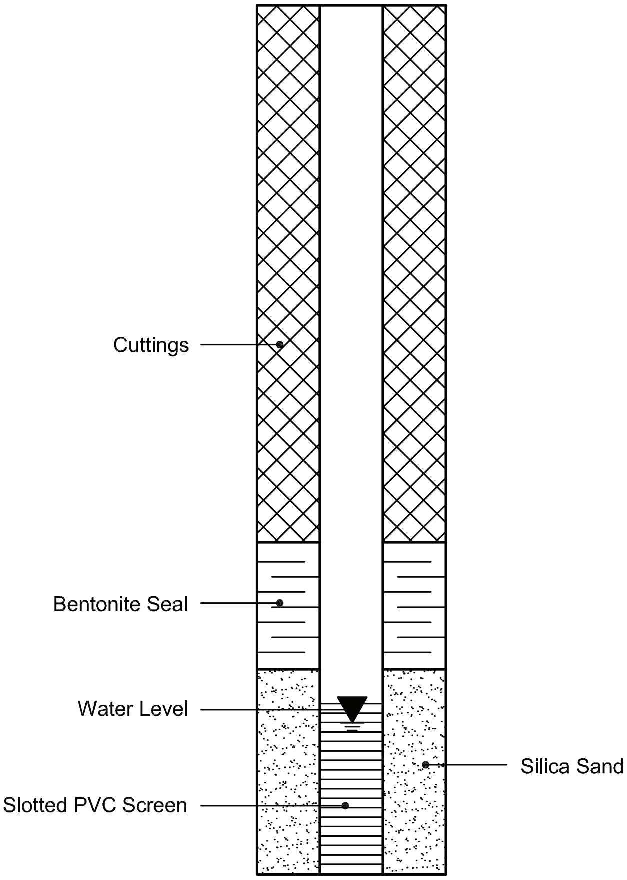


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 13465

Project Description: PG2846

Report Date: 03-Dec-2012

Order Date:28-Nov-2012

	Client ID:	BH1-SS5	-	-	-
	Sample Date:	19-Nov-12	-	-	-
	Sample ID:	1248161-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.82	-	-	-
Resistivity	0.10 Ohm.m	94.7	-	-	-
Anions					
Chloride	5 ug/g dry	10	-	-	-
Sulphate	5 ug/g dry	20	-	-	-

Certificate of Analysis

Report Date: 10-Apr-2013
Order Date:4-Apr-2013

Client: **Paterson Group Consulting Engineers**
Client PO: 13465
Project Description: PG2846

	Client ID:	BH5-SS4	-	-	-
	Sample Date:	03-Apr-13	-	-	-
	Sample ID:	1314180-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	77.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.27	-	-	-
Resistivity	0.10 Ohm.m	4.81	-	-	-
Anions					
Chloride	5 ug/g dry	40	-	-	-
Sulphate	5 ug/g dry	169	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG2846-3 - TEST HOLE LOCATION PLAN

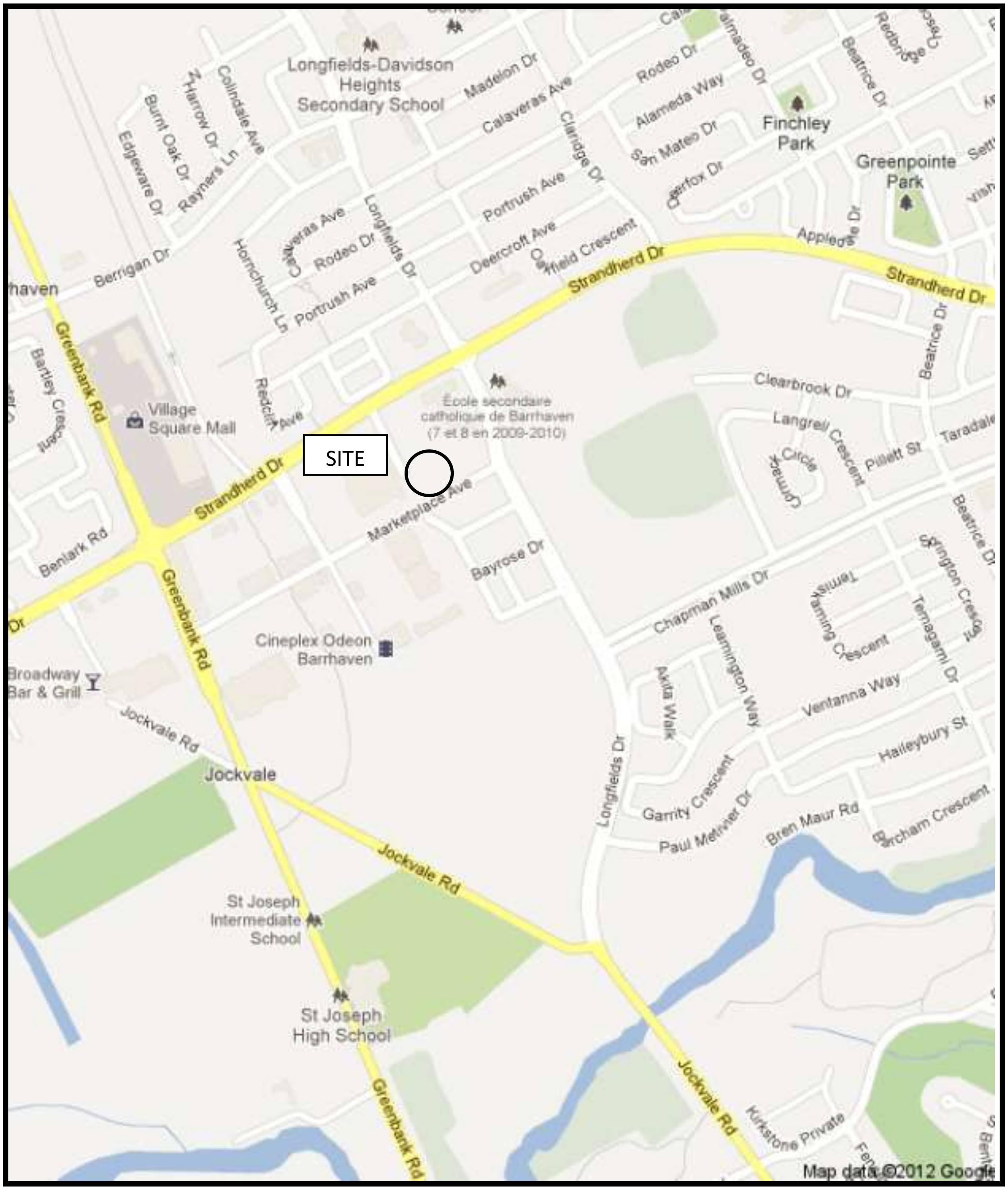


FIGURE 1

KEY PLAN

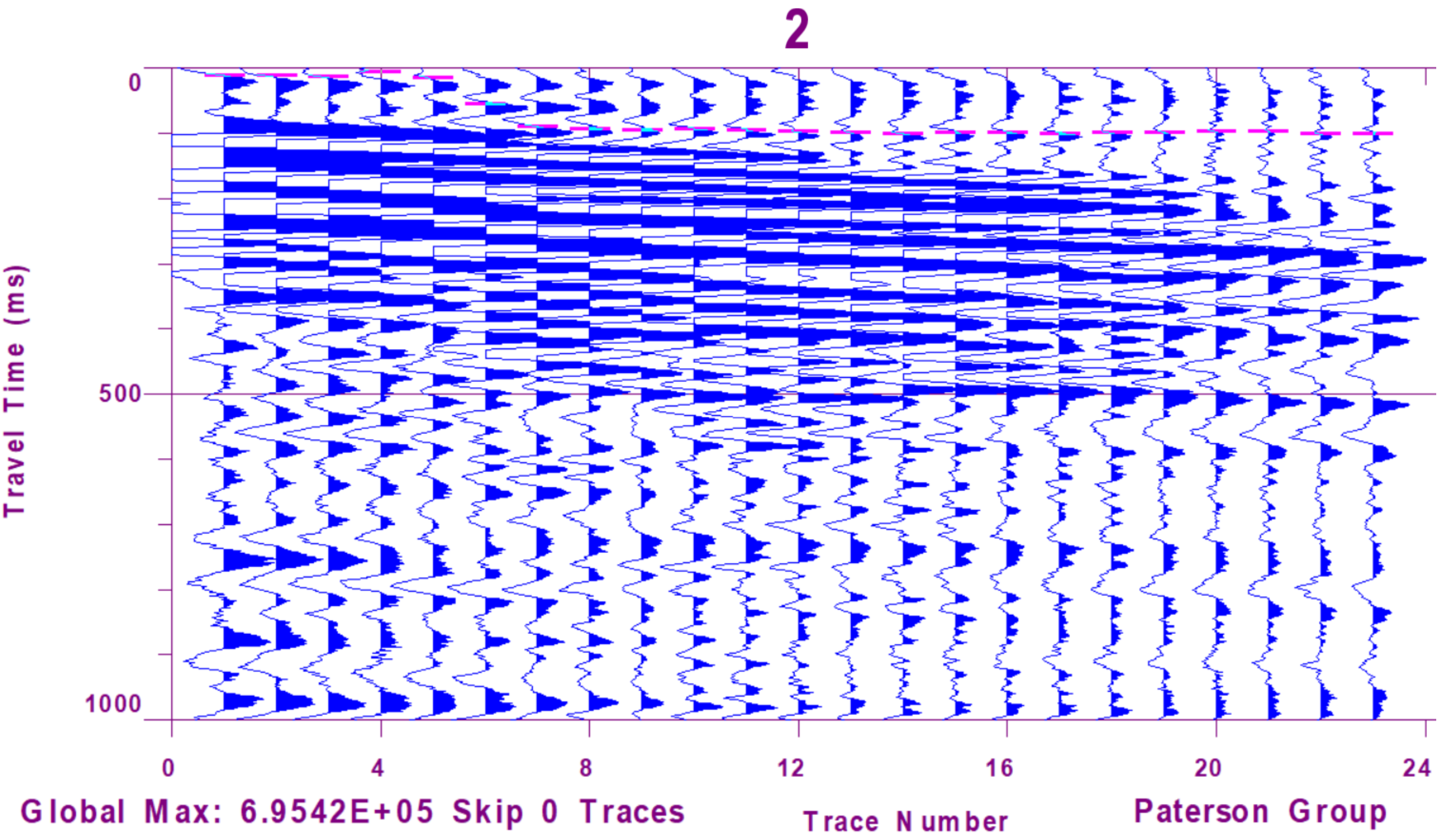


Figure 2 – Shear Wave Velocity Profile at Shot Location -20 m

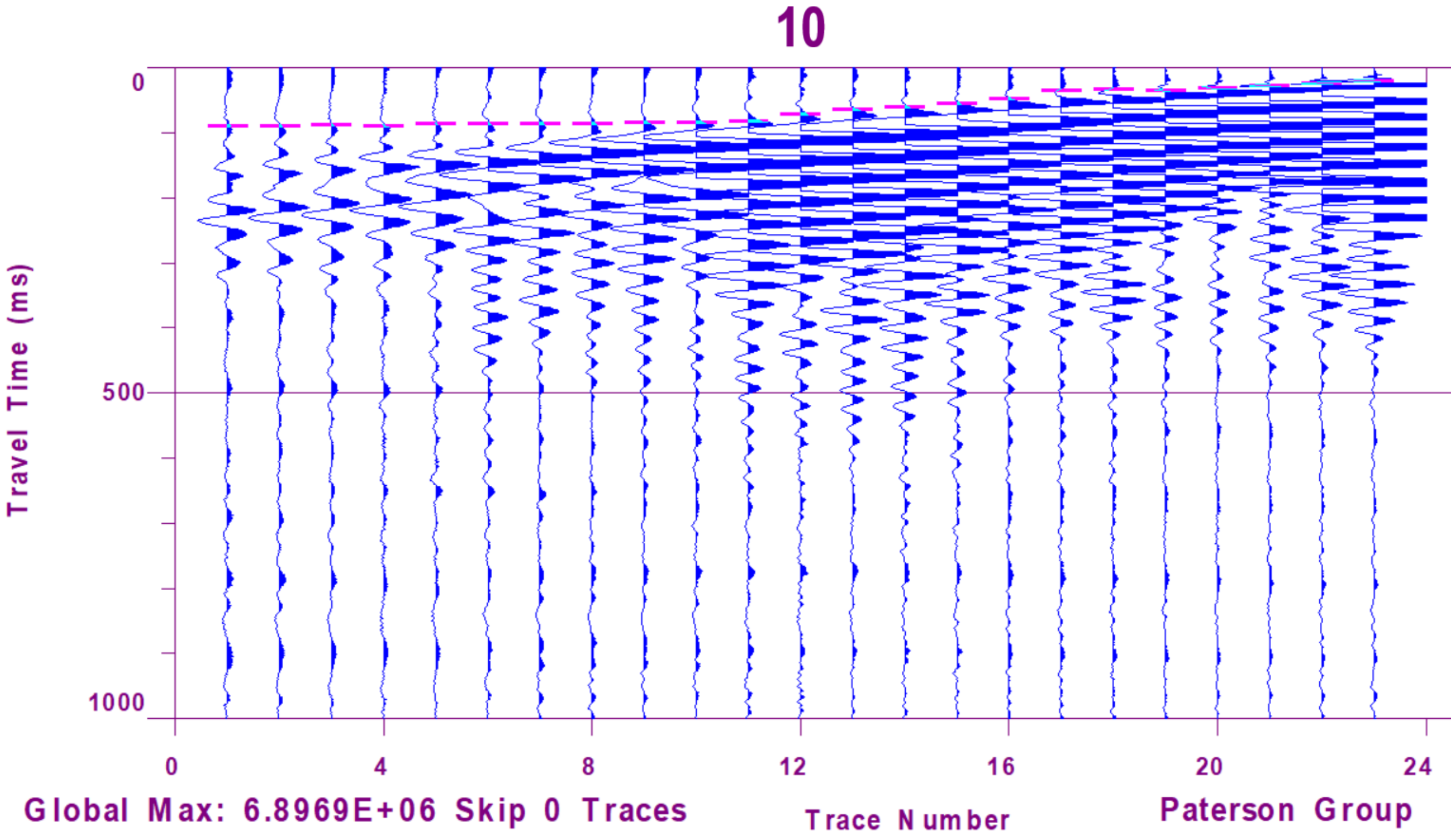
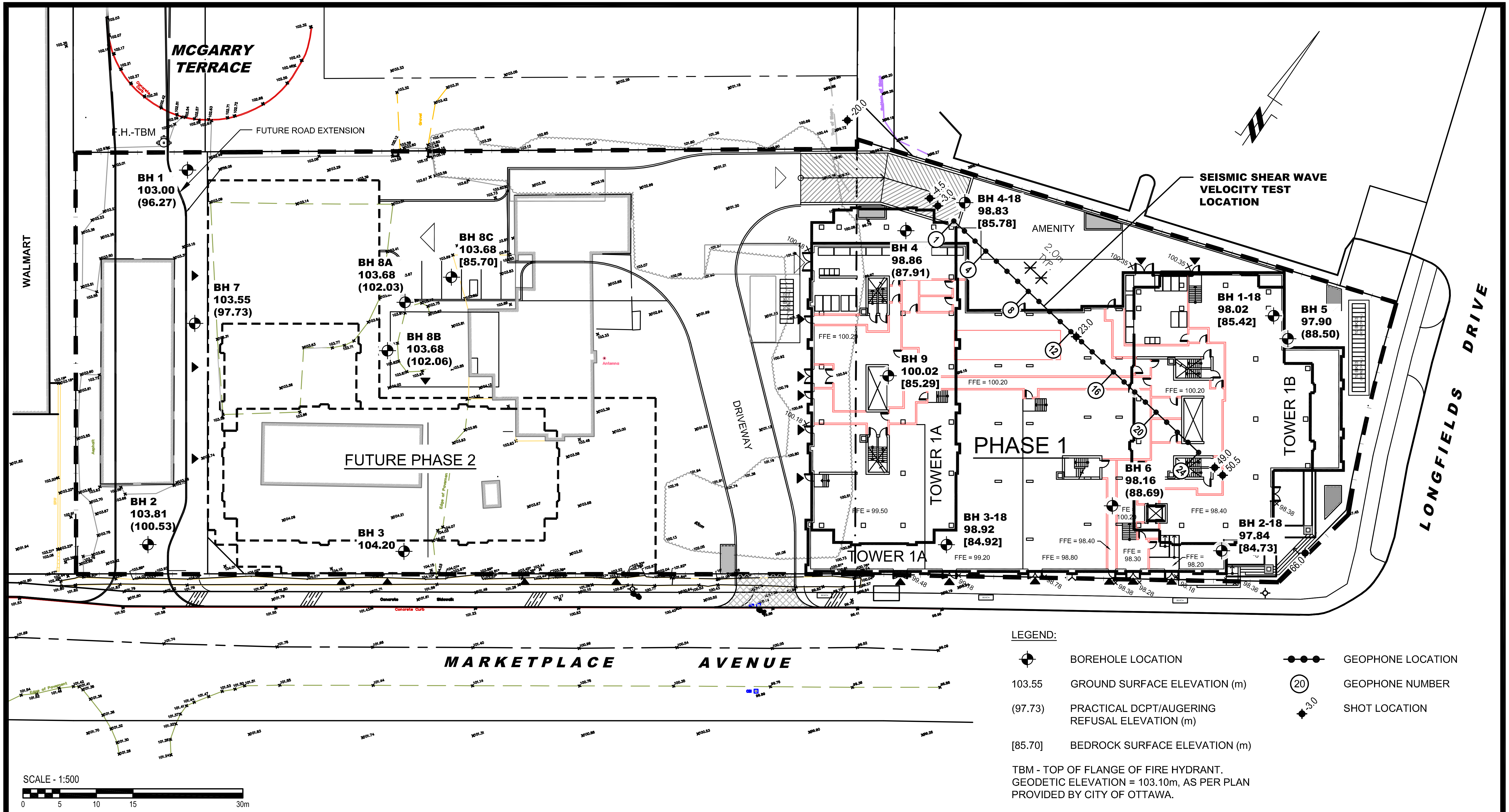


Figure 3 – Shear Wave Velocity Profile at Shot Location 3 m



LEGEND:

⌖	BOREHOLE LOCATION	●●●	GEOPHONE LOCATION
103.55	GROUND SURFACE ELEVATION (m)	⑳	GEOPHONE NUMBER
(97.73)	PRACTICAL DCPT/AUGERING REFUSAL ELEVATION (m)	✦ -3.0	SHOT LOCATION
[85.70]	BEDROCK SURFACE ELEVATION (m)		

TBM - TOP OF FLANGE OF FIRE HYDRANT.
GEODETIC ELEVATION = 103.10m, AS PER PLAN
PROVIDED BY CITY OF OTTAWA.

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NO.	REVISIONS	DATE	INITIAL
2	SEISMIC SURVEY LOCATION ADDED	16/05/2018	NC
1	NEW BASE PLAN & BH 1-18 TO BH 4-18 ADDED	06/03/2018	DJG

1897365 ONTARIO INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
1034 McGARRY TERRACE & 1117 LONGFIELDS DR.
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:500	Date:	03/2015
Drawn by:	MPG	Report No.:	PG2846-2
Checked by:	NZ	Drawing No.:	PG2846-3
Approved by:	DJG	Revision No.:	2

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