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Proposed Commercial Buildings
Block 16 - Citigate 416 Development
Strandherd Drive
Ottawa, Ontario

Prepared For

Tomlinson Group

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Report: PG3563-2 Revision 2

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Tomlinson Group to conduct a geotechnical investigation for the proposed commercial buildings to be constructed within Block 16 of the Citigate 416 Development along Strandherd Drive, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was 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 its design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. The report contains the geotechnical findings and includes 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 Development

Based on the conceptual drawings, it is our understanding that the proposed development will consist of a four (4) storey office building and a one (1) storey testing facility of slab-on-grade construction. The remainder of the subject site will consist of asphalt car parking and access lanes with associated landscaping areas.

It is further understood that significant grade raises are being considered around the perimeter of the structures.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the supplemental investigation was carried out on November 9 to 11, 2015. At that time, a total of thirteen (13) boreholes were drilled to a maximum depth of 6.7 m below existing ground surface. Five (5) boreholes and two (2) test pits were also completed for our previous geotechnical investigations within the subject site. The test hole locations were determined in the field by Paterson personnel. The test hole locations are presented on Drawing PG3563-2 - Test Hole Location Plan included in Appendix 2.

All fieldwork was conducted under the full-time supervision of Paterson personnel with the direction of a senior engineer from the geotechnical department. The boreholes were drilled with hollow stem augering to the required depths at select locations, sampling and testing the overburden. The test pits were excavated using a rubber tired backhoe to the required depth with periodic sampling of the overburden.

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights or a 50 mm diameter split-spoon sampler. Soil samples from the test pits were recovered from the side walls of the open excavation. All soil samples were classified on site, placed in sealed plastic bags and transported to the laboratory for further review. The depths at which the auger, split spoon and grab samples were recovered from the test holes are presented as, AU, SS and G, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Standard Penetration Testing (SPT) was conducted and 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 sample 300 mm into the soil after a 150 mm initial penetration with a 63.5 kg hammer falling from a height of 760 mm. This testing was completed in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

Undrained shear strength testing was conducted at regular intervals in cohesive soils and completed using a MTO field vane apparatus. This testing was done in general accordance with ASTM D2573-08 - Standard Test Method for Field Vane Shear Test in Cohesive Soil.

Overburden thickness was evaluated by a dynamic cone penetration test (DCPT) at BH 2 and BH 5. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip and 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.

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

Groundwater

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

Sample Storage

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

3.2 Field Survey

The test hole locations were chosen by Paterson in a manner to provide general coverage of the proposed buildings taking into consideration site features and underground utilities. The borehole locations and ground surface elevation at each borehole location were surveyed and provided by Tomlinson Group.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in the laboratory to review the field logs. The subsurface soils were classified in general accordance with ASTM D2488-09a, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Transportation of the samples was completed in general accordance with ASTM D4220-95 (2007) - Standard Practice for Preserving and Transporting Soil Samples.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site consists of former agricultural land which is currently grass covered and several young trees. The ground surface across the site is relatively flat and slopes gradually down toward the west. The site was observed to be slightly lower than Nortel Drive to the east and significantly lower than Fallowfield Road bordering the north boundary of the subject site. A 1 to 1.5 m deep drainage ditch running in an east-west direction exists within the central portion of the site. It was also noted during our field investigations, that several fill piles are present within the east portion of the subject site.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of a thin layer of topsoil/fill underlain by a native very stiff to stiff silty clay deposit and a compact to dense glacial till layer. The fine soil matrix within the glacial till layer ranged from a silty clay to silty sand with gravel, cobbles and boulders. Practical refusal to DCPT was noted at BH 2 and BH 5 at 9.2 and 10.1 m, respectively.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Available Geological Mapping

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded sandstone and dolomite of the March formation. The overburden drift thickness is estimated to be between 15 to 25 m depth.

4.3 Groundwater

The measured groundwater levels are summarized below in Table 1 and presented on the Soil Profile and Test Data sheets in Appendix 1. It should be noted that surface water can become perched with a backfilled borehole, which can lead to higher than normal groundwater level readings. The long-term groundwater level can also be estimated based on the recovered soil samples' moisture levels, colouring and consistency. Based on these observations, the long-term groundwater level is anticipated at a 2 to 3 m depth. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Date
BH 1	98.34	1.78	96.56	November 23, 2015
BH 2	98.66	1.44	97.22	November 23, 2015
BH 3	99.09	0.94	98.15	November 23, 2015
BH 4	98.40	1.68	96.72	November 23, 2015
BH 5	98.30	1.20	97.10	November 23, 2015
BH 6	98.30	Dry	-	November 23, 2015
BH 7	98.28	1.45	96.83	November 23, 2015
BH 8	98.23	1.49	96.74	November 23, 2015
BH 9	98.86	Damaged	-	November 23, 2015
BH 10	98.22	1.34	96.88	November 23, 2015
BH 11	98.44	1.58	96.86	November 23, 2015
BH 12	98.30	1.68	96.62	November 23, 2015
BH 13	98.38	2.10	96.28	November 23, 2015
BH 14	98.03	dry	-	November 23, 2015
BH 15	98.70	dry	-	November 23, 2015
BH 16	98.74	dry	-	November 23, 2015
BH 17	98.81	dry	-	November 23, 2015
BH 18	99.48	0.88	98.60	November 23, 2015

5.0 Discussion

5.1 Geotechnical Assessment

Based on the results of our investigation, it is expected that the proposed buildings will be founded over conventional shallow footings placed on an undisturbed, very stiff to stiff silty clay and/or glacial till bearing surface. From a geotechnical perspective, the subject site is satisfactory for the proposed development.

Based on the current conceptual drawings for the proposed development, it is understood that proposed grade raises of up to 6 m are anticipated along the perimeter of the proposed main building and testing facility. Due to the presence of the silty clay deposit, it was recommended that a settlement preload program be completed for the proposed main building footprint to eliminate any long-term settlement associated with the proposed grade raises. The proposed main building footprint was in-filled with an oversized blast rock fill in November 2015 for the settlement preload program. Details of the settlement preload monitoring results are presented in Subsection 5.4.

Details of a slope reinforcement system are presented in Subsection 5.10, which has been designed to eliminate earth pressure effects associated with supporting a 6 m high slope for the foundation walls, where required, and retaining wall segments at the building entrances. Our recommendations are presented in Subsection 5.10.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, deleterious fill and soils containing significant amounts of organics, should be stripped from under any buildings and other settlement sensitive structures. Precautions should be taken to ensure that all bearing surfaces and subgrade soils remain undisturbed during site preparation activities.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts compacted by the tracks of the spreading equipment to minimize voids. If the material is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 95% of the material's 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

Bearing Resistance Values

Footings founded on an undisturbed, stiff silty clay, compact glacial till or approved engineered fill bearing surface can be designed using the bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **350 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. It is understood that a 'raft' style pad footing (approx. 9 m by 18 m) may be required based on preliminary structural design review. It should be noted that the abovenoted bearing resistance values can be used for the 'raft' pad footing, which is approximately 9 m by 18 m.

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, under dry conditions, prior to the placement of concrete for footings.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to an stiff silty clay or compact glacial till when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

Settlement Preload Program / Permissible Grade Raise

For areas where the preload program is not completed, a permissible grade raise of **3.5 m** is recommended for the subject site.

5.4 Settlement Preload Program

The settlement preload program has been designed to eliminate the excessive settlement anticipated due to the proposed grading adjacent to the proposed main building location and the underlying silty clay deposit. The proposed grading information in the site grading plan prepared McIntosh Perry was used to determine the top elevation required for the preload pile. The settlement preload program commenced in late November 2015 for the proposed main building footprint. The outline of the preload pile placed by Tomlinson is presented in Drawing PG3563-2 - Test Hole Location Plan in Appendix 2. The preload pile consists of a blast rock fill and covers the entire building footprint extending at least 5 m horizontally beyond the proposed building perimeter. The top of the preload pile was relatively flat across the building footprint with an approximate geodetic elevation of 104.4 m. Four (4) settlement plates were placed at the proposed building corners as depicted in Drawing PG3563-2 - Test Hole Location Plan in Appendix 2.

A settlement monitoring program was completed by Paterson during the preload program. The initial settlement survey was carried out on December 1, 2015 with a monthly settlement survey completed until March 7, 2016. The results of our settlement surveys are depicted in Figure 4 presented in Appendix 2. Based on our settlement monitoring data for the preload program and available soils information, greater than 90% of primary consolidation for the underlying silty clay deposit has been achieved within the preload area. Therefore, it is recommended that the preload fill be removed at this time and the settlement preload program has been successfully completed.

For design purposes, it is expected that total and differential settlements associated with the combination of grade raises and footing loading conditions will be limited to 25 and 20 mm, respectively. It should be further noted that conventional construction methods are acceptable and no LWF or additional reinforcing within the foundation are required for the proposed building within the preload area from a geotechnical perspective.

5.5 Design for Earthquakes

A seismic shear wave velocity test was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building based on Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity test was completed by Paterson personnel. Two seismic shear wave velocity profiles from the on site testing are presented in Figures 2 and 3 in Appendix 2.

Field Program

The seismic shear wave test was completed along the north property boundary, as presented in Drawing PG3563-2 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly an east-west orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was connected to a 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 strikes an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four to eight times at each shot location to improve signal to noise ratio. The shot locations are completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the centre of the geophone array, as well as 3, 4.5 and 30 m away from the first and last geophone.

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

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was completed by reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m below the structure's foundation. 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 by the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. As bedrock quality increases, the bedrock shear wave velocity also increases. Based on the soils information and testing results, bedrock is present at a 10 m depth.

Based on the test results, the overburden and bedrock seismic shear wave velocities are 164 m/s and 2,950 m/s, respectively. The Vs_{30} was calculated using the standard equation for average shear wave velocity from the Ontario Building Code (OBC) 2012.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\sum\left(\frac{(Depth_i(m))}{Vs_i(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{10m}{164m/s} + \frac{20m}{2{,}950m/s}\right)}$$

$$V_{s30} = 443m/s$$

Based on the seismic test results, the average shear wave velocity, Vs_{30}, for foundations at the subject site is 443 m/s. Therefore, a **Site Class C** is applicable for design of the proposed buildings, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not considered to be susceptible to liquefaction.

5.6 Uplift Resistance

A system, utilizing a concrete footing and the weight of a cone of soil over the concrete footing, could be considered to provide uplift resistance to seismic design forces. Typically, the horizontal load component is resisted by passive earth pressure (actually the net of passive minus active) and the vertical load component is resisted by the weight that can be mobilized by the footing.

Geotechnical parameters for typical backfill materials compacted to 98% of standard Proctor maximum dry density (SPMDD) in 300 mm lift thicknesses are provided in Table 2, along with the associated earth pressure coefficients for horizontal resistance calculations for deadman anchors. General uplift cone or prism angles are provided in Figure 2 - Uplift Cone Angles for Backfill Material in Appendix 2 for cohesion and cohesionless soils. Also, friction factors between concrete and the various subgrade materials are provided in Table 2.

For soil above the groundwater level, the “drained” unit weight should be used and below groundwater level, the “effective” unit weight should be used. Please note that backfilled excavations in low permeability soils can be expected to fill with water and the use of the effective unit weights would be prudent if drainage of the soils and fill adjacent to the concrete footings is not provided.

A sieve analysis and standard Proctor test should be conducted on each of the fill materials proposed to obtain an accurate soil density to be expected, so that the applicable unit weights can be estimated.

Please note that the parameters provided in Table 2 are unfactored and, in the case of passive earth pressure coefficients, are “ultimate” values. As such, the appropriate factor of safety for working stress design, or resistance factor for limit states design (0.4 to 0.8) should be applied.

Table 2 - Geotechnical Parameters for Uplift Resistance Design

Material Description	Unit Weight (kN/m^3)		Friction Angle (°) φ'	Friction Factor, tan δ	Earth Pressure Coefficients			
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P	Seismic Kae
OPSS Granular A Fill (Crushed Stone)	22	13.5	40	0.6	0.22	0.36	4.58	0.3
OPSS Granular B Type II Fill (Crushed Stone)	22.5	14	42	0.6	0.2	0.33	5.04	0.28
Brown silty clay	17	7.2	33	0.5	0.29	0.46	3.39	0.38

Notes:

❑ Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
❑ The earth pressure coefficients provided are for horizontal backfill profile.

5.7 Slab on Grade Construction

With the removal of all topsoil, soils containing significant amounts of organics or deleterious fill within the proposed building footprint, the native soil surface will be considered an acceptable subgrade surface on which to commence backfilling for floor slab construction. Assessment of the blast rock fill currently being imported to site should be reviewed for placement below the sub-slab fill. The upper 200 mm of sub-slab backfill should consist of an OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the material’s SPMDD.

Provision should be provided for proof-rolling the soil subgrade with heavy vibratory compaction equipment prior to placing any fill. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab. The upper 200 mm of sub-slab backfill is recommended to consist of an OPSS Granular A crushed stone.

5.8 Foundation Walls/Retaining Walls

Reinforced Slope Design - Lightweight Fill Backfill

It should be noted that a reinforced slope design with lightweight fill backfill has been prepared by Paterson to eliminate earth pressure against the foundation walls, where required, and cantilevered retaining walls. It is understood that typical soil backfill would result in excessive earth pressure that would require significant reinforcing and wall thickness for walls to resist the loading. Our reinforced slope design recommendations are presented in Subsection 5.10 and Drawing PG3563-6 - Reinforced Slope Detail for Proposed Berm in Appendix 2. Provided the reinforced slope design is properly installed and approved by Paterson at the time of construction, the adjacent foundation walls and cantilevered retaining walls can be designed with an at-rest earth pressure coefficient of zero.

Foundation Walls - Typical Soil Backfill

There are several combinations of backfill materials and retained soils that could be applicable for the foundation walls of the subject structure. The conditions for typical soil backfill can be conservatively represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 21 kN/m^3.

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. Under seismic conditions, $K_o = 0.0$ for areas where the reinforced slope design with lightweight fill backfill is in place.

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

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

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

Sliding Resistance

Sliding horizontal shear resistance of the footings founded on a silty clay subgrade can be computed using a horizontal shear resistance (friction) factor of 0.5. A geotechnical resistance factor of 0.8 should be applied to the abovenoted value as per OBC 2012.

Retaining Walls - Typical Soil Backfill

It is recommended that backfill behind the proposed retaining walls consist of a free draining, non-frost susceptible granular backfill, such as Granular A or Granular B Type II. It is expected that the conditions can be conservatively represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 21 kN/m^3. The retaining walls must be drained. An interface friction angle of 20 degrees between the wall and the backfill material is applicable for the abovenoted parameters. Two (2) distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two (2) conditions are presented below.

Retaining Walls - Static Earth Pressures

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

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

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

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

Retaining Walls - Seismic Earth Pressures

Seismic loading conditions influence the earth pressures that will act on earth retaining structures during seismic events. In Ottawa, the peak ground acceleration (PGA) is 0.32 for the OBC 2012.

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

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

The M-O Method is used to calculate the total active earth pressure (P_{AE}). The resulting force is then split into the static (active) (P_A) and seismic component (ΔP_{AE}).

The total active earth pressure (P_{AE}) can be calculated using $0.5K_{AE}\ \gamma H^2$ where:

K_{AE} - Dynamic active earth pressure coefficient, 0.42, for typical soil backfill.
K_{AE} - Dynamic active earth pressure coefficient, 0.0, where the reinforced slope design with lightweight fill backfill is in place.
γ - unit weight of the fill of the applicable retained soil (kN/m^3)
H - height of the wall (m)

The static component (P_A) can be calculated using $0.5K_A\ \gamma\ H^2$ Cos β where:

K_A = dynamic active earth pressure coefficient, 0.3, for typical soil backfill.
K_A = dynamic active earth pressure coefficient, 0.0, where the reinforced slope design with lightweight fill backfill is in place.
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
β = backslope angle from horizontal

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

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

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

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

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

Retaining Wall - Design and Construction Considerations

The proposed retaining walls should be checked for global stability and designed to maintain an adequate factor of safety in excess of the required 1.5 for static conditions and 1.1 for seismic loading conditions. The internal and external failure modes of the retaining wall sections should also be designed with the same factors of safety provided. The applicable seismic design should incorporate a Peak Ground Acceleration (PGA) of 0.32 for the Ottawa area, as per the Ontario Building Code (OBC 2012).

Geotechnical field review must be completed at the time of excavation, prior to placing the granular bedding layer, to assess the bearing medium under the proposed wall. Based on the underside of wall elevations provided, it is anticipated that the walls will be founded over an engineered fill pad or undisturbed, stiff silty clay bearing surface. A bearing resistance value at serviceability limit states, or allowable bearing pressure, of **200 kPa**, and/or a factored bearing resistance value at ULS of 300 kPa, if required, can be used for design purposes. Engineered fill placed below the proposed retaining wall should consist of a Granular A or Granular B Type II placed in maximum 300 mm loose lifts and compacted to 98% of its standard Proctor maximum dry density (SPMDD).

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.

It is recommended that the geotechnical consultant conduct field reviews of the subgrade for the base of the wall, and testing or visual observations of the compaction methods for the base and backfill during wall construction. It is further recommended that all bedding and backfill materials be placed under dry conditions and in above freezing temperatures. Precautions should be taken to ensure that the bedding material does not freeze before placement of the retaining wall blocks, which could lead to detrimental movement within the retaining wall, once the frost leaves the bedding material.

5.9 Pavement Structure

Minimum Pavement Structures

For design purposes, the minimum pavement structure presented in the following tables could be used for the design of car parking areas and access lanes.

Table 3 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 4 - Recommended Pavement Structure - Truck Parking and Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, 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 I or II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is dependent on the moisture condition of the contact zone between the subgrade material and granular base. Failure to provide adequate drainage under conditions of heavy wheel loading could result in the subgrade fines being pumped into the stone subbase voids, thereby reducing the load bearing capacity.

Due to the impervious nature of the subgrade materials consideration should be provided to installing subdrains during the pavement construction. The subdrains should extend in four orthogonal directions and longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines or the pipe, should be wrapped with suitable filter cloth. The subdrain inverts should be approximately 300 mm below subgrade level and placed in accordance with City of Ottawa standard drawings. The subgrade surface should be shaped to promote water flow to the drainage lines.

5.10 Soil Berm Recommendations

It is understood that the current landscaping concept includes soil berms that extend up to 6 m in height adjacent to the proposed building footprints. Based on the plans provided, the berms will be shaped with a slope profile between 4H:1V and 3H:1V. Several fill options are available for the proposed slopes. The long-term slope stability factor of safety will be greater than 1.5 for the subject slopes provided the fill materials are placed according to Paterson recommendations and approved by Paterson personnel at the time of construction.

Where the slopes will be graded to a 3H:1V profile or greater (4H:1V), several fill options, such as those listed below, are available provided the material is reviewed and approved by Paterson at the time of construction.

Well-Graded Blast Rock Fill

A free draining, suitably fragmented, well-graded blast rock material with a maximum particle size of 300 mm placed in maximum 600 mm loose lifts and compacted by an adequately sized bulldozer making several passes and approved by the geotechnical consultant at the time of placement. Any blast rock greater than 300 mm in diameter should be segregated and hoe rammed into acceptable fragments.

It is recommended that a non-woven geotextile liner, such as a Terrafix 270R or equivalent, be placed over the granular fill surface and topped with a minimum 300 mm thick layer of topsoil.

Brown Silty Clay Fill or Sand and Gravel Fill

Alternatively, a relatively dry workable brown silty clay fill or sand and gravel fill approved by the geotechnical consultant can be used to build up the subject slopes. The material should be placed in maximum 300 mm loose lifts and compacted using suitable compaction equipment for the lift thickness to a minimum of 95% of its SPMDD. It is further recommended that the slope be covered with a minimum thickness of 150 mm of topsoil mixed with a hardy grass seed to minimize surficial erosion.

It is recommended that the geotechnical consultant conduct field reviews, including testing and visual observations of the material placement during backfilling. It is further recommended that all cohesive and frost susceptible backfill materials be placed under dry conditions and in above freezing temperatures.

Reinforced Slope Design and Lightweight Fill Backfill

The reinforced slope design was prepared as previously noted to eliminate earth pressure against the proposed building's foundation walls, where required, and against the cantilevered retaining walls. The proposed design allows the soil behind the walls to be self-supporting and exert no earth pressure on the adjacent wall. Lightweight fill blocks (EPS Type 19) will be placed between the reinforced slope face and adjacent wall to in-fill the void and allow for topsoil placement across the slope face. Details of the design are presented in Drawing PG3563-6 - Reinforced Slope Details for Proposed Berm and Figure 6 - Section A-A' - Profile of Reinforced Slope in Appendix 2.

It is anticipated that several areas along the slope may not provide sufficient room for the recommended horizontal geogrid lengths due to the slope configuration. For these areas, it is recommended that the geogrid be placed to complete a closed basket or 'cell' of the supported soil. Area specific recommendations can be provided by Paterson, as required, at the time of construction.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter foundation drainage system is recommended to be provided for the proposed structures. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for placement as backfill against the foundation walls unless used in conjunction with a composite drainage system, such as Delta Drain 6000 or Miradrain G100N. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be placed for this purpose.

6.2 Protection Against Frost Action

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

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection. The recommended minimum thickness of soil cover is 2.1 m (or equivalent).

6.3 Excavation Side Slopes

The excavations for the proposed development will be through a native silty clay material. The subsurface soil is considered to be mainly a Type 2 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Above the groundwater level, for excavations to depths of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Shallower slopes should be provided for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be installed.

The slope cross-sections recommended above are for temporary slopes. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be maintain safe working distance from the excavation sides.

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

A trench box is recommended to be installed 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 should not remain open for extended periods of time.

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 the SPMDD. The bedding material should extend at a minimum 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 a minimum of 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 dry brown silty clay could be place above the cover material if the excavation and backfilling operations are completed in dry weather conditions. The wet silty clay or glacial till materials could be difficult to place and compact, due to the high water content.

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 consist of the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

6.5 Groundwater Control

The groundwater infiltration into the excavations should be low to moderate depending on the subsurface soil conditions. The contractor should be prepared to collect and pump groundwater infiltration volumes from the excavation trenches.

It is not expected that more than 50,000 L/day will be pumped from open shallow excavations. However, if deeper excavation are contemplated, a temporary MOECC permit to take water (PTTW) may be required 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 MOECC.

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

6.6 Winter Construction

Precautions should be provided if winter construction is considered for this project. The subsurface soil conditions 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 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 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 during construction. 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 analytical test results are presented in Table 5 along with industry standards for the applicable threshold values. The results are indicative that Type 10 Portland cement (Type GU).

Table 5 - Corrosion Potential

Parameter	Laboratory Results	Threshold	Commentary
	BH2-SS4		
Chloride	7 µg/g	Chloride content less than 400 mg/g	Negligible concern
pH	7.79	pH value less than 5.0	Neutral Soil
Resistivity	92.7 ohm.m	Resistivity greater than 1,500 ohm.cm	Low to Moderate Agressive
Sulphate	7 µg/g	Sulphate value greater than 1 mg/g	Negligible Concern

7.0 Recommendations

The following is recommended to be completed once the site plan and development are determined:

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

❏ Observation of all subgrades prior to backfilling.

❏ Field density tests to ensure that the specified level of compaction has been achieved.

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

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

A report confirming the construction has been completed in general accordance with the recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The report recommendations are in accordance with the present understanding of the project. Paterson requests permission to review the grading plan, once available, and recommendations when the drawings and specifications are complete.

The recommendations are based on information gathered at the specific test locations and could 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 the history of the site reflecting natural, construction, and other activities.

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 Tomlinson Group 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.

LICENSED PROFESSIONAL ENGINEER
June 9, 2016
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PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng.

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Buildings - Block 16 Citigate 416
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by Tomlinson Group.

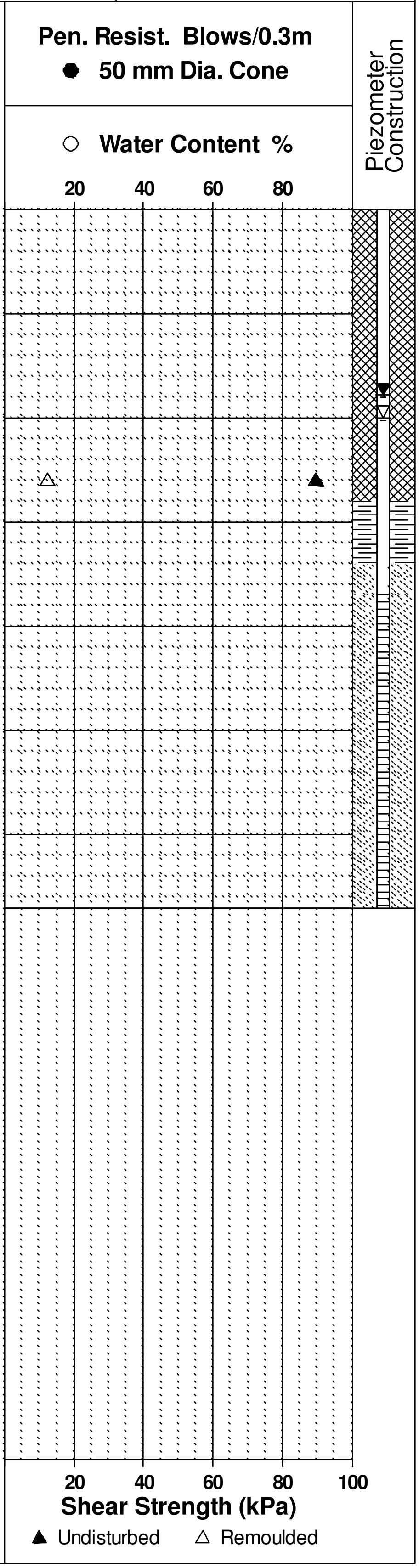
FILE NO. PG3563

REMARKS

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger **DATE** 14 July 2015

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.34
TOPSOIL	0.30	AU	1				
FILL: Brown sandy silt, trace clay and gravel	0.63						
Very stiff to stiff, brown **SILTY CLAY,** some sand - grey by 1.4m depth		SS	2	83	13	1	97.34
		SS	3	100	3	2	96.34
	3.05					3	95.34
GLACIAL TILL: Very loose to compact, grey silty clay with sand and gravel		SS	4	46	6		
		SS	5	25	3	4	94.34
		SS	6	25	1	5	93.34
		SS	7	33	12		
	6.10					6	92.34
GLACIAL TILL: Dense, grey silty sand, some gravel and cobbles, trce clay	6.70	SS	8	71	34		
End of Borehole							
(GWL @ 2.00m-July 28, 2015)							
(GWL @ 1.78m-Nov. 23, 2015)							



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REMARKS

BORINGS BY CME 55 Power Auger

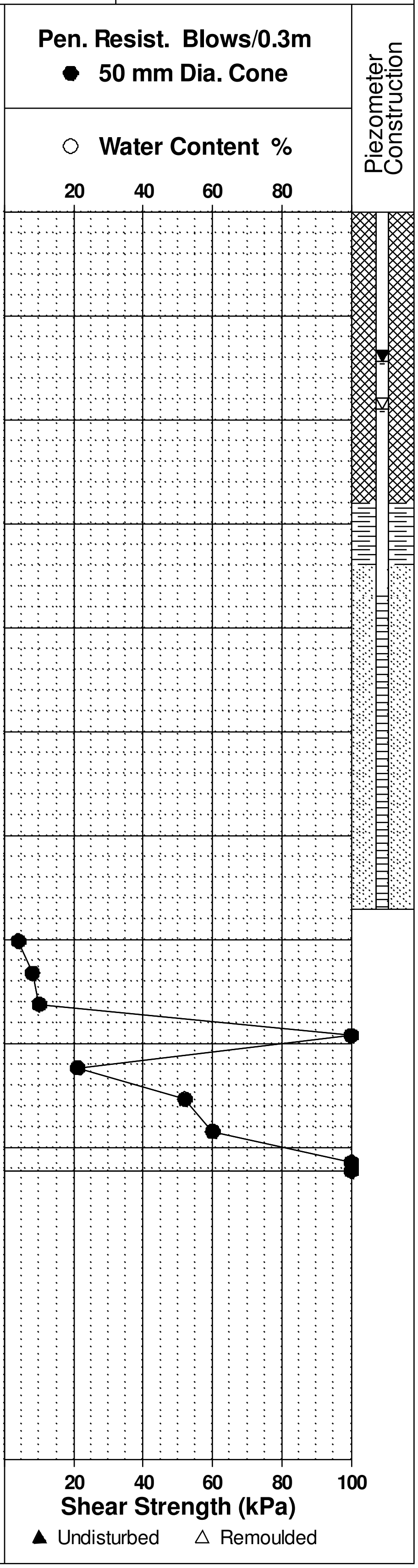
DATE 14 July 2015

FILE NO. PG3563

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.30	AU	1		
FILL: Brown sandy silt, trace clay and gravel 0.76				
Stiff, grey **SILTY CLAY,** some sand 1.35	SS	2	100	4
GLACIAL TILL: Loose to very loose, grey silty sand with gavel, some cobbles and boulders - silty clay matrix in upper part	SS	3	70	23
	SS	4	75	12
	SS	5	46	8
	SS	6	75	5
	SS	7	75	1
	SS	8	100	6
6.70	SS	9	88	3
Dynamic Cone Penetration Test commenced at 6.70m depth. Inferred **GLACIAL TILL** 9.22				
End of Borehole Practical DCPT refusal at 9.22m depth (GWL @ 1.90m-July 28, 2015) (GWL @ 1.44m-Nov. 23, 2015)				

DEPTH (m)	ELEV. (m)
0	98.66
1	97.66
2	96.66
3	95.66
4	94.66
5	93.66
6	92.66
7	91.66
8	90.66
9	89.66



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FILE NO. PG3563

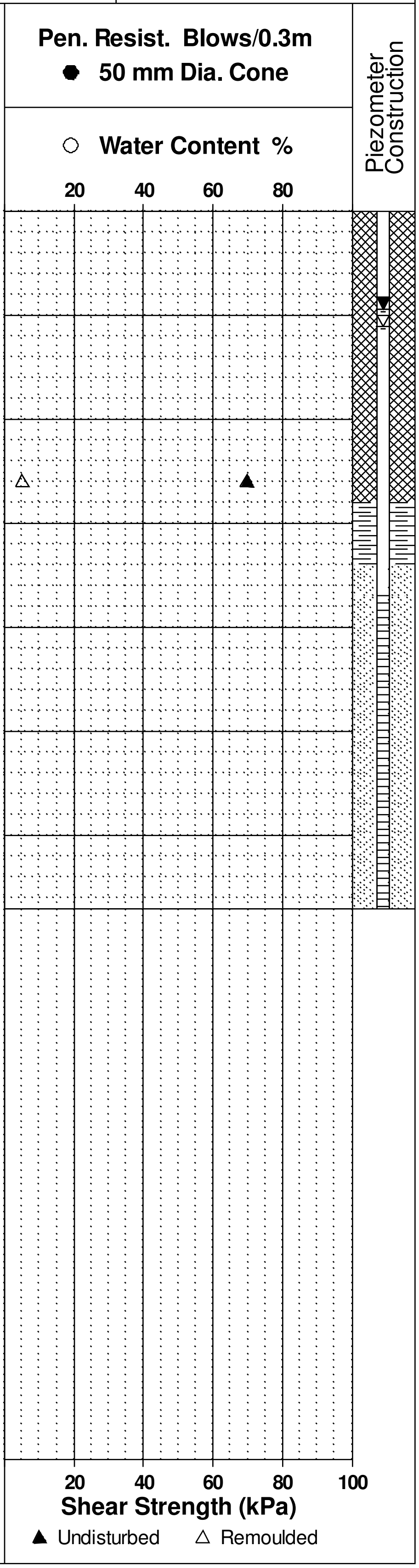
REMARKS

HOLE NO. BH 3

BORINGS BY CME 55 Power Auger

DATE 14 July 2015

SOIL DESCRIPTION	SAMPLE				DEPTH (m)	ELEV. (m)
	TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE					0	99.09
TOPSOIL 0.30	AU	1				
FILL: Brown sandy silt, some gravel, trace cobbles and boulders 1.07	SS	2	58	21	1	98.09
Very stiff, grey **SILTY CLAY,** some sand 2.18	SS	3	100	5	2	97.09
GLACIAL TILL: Very loose to loose, grey silty sand with gravel, trace cobbles - silty clay matrix in upper part					3	96.09
	SS	4	100	2		
	SS	5	58	1	4	95.09
	SS	6	75	9	5	94.09
	SS	7	42	9	6	93.09
6.70	SS	8	100	1		
End of Borehole (GWL @ 1.11m-July 28, 2015) (GWL @ 0.94m-Nov. 23, 2015)						



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REMARKS

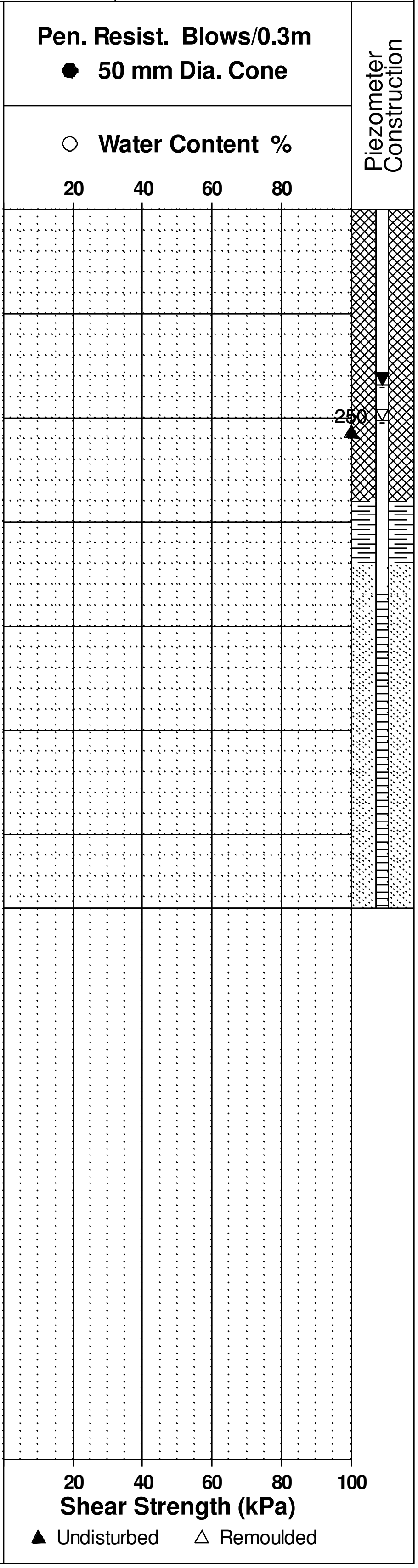
BORINGS BY CME 55 Power Auger

DATE 14 July 2015

FILE NO. PG3563

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE				DEPTH (m)	ELEV. (m)
	TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE					0	98.40
TOPSOIL 0.30						
FILL: Brown sandy silt, trace gravel 0.60	AU	1				
Very stiff, grey **SILTY CLAY,** trace sand 1.52	SS	2	83	10	1	97.40
GLACIAL TILL: Grey silty sand, some gravel, clay, trace cobbles and boulders - silty clay matrix in upper part					2	96.40
	SS	3	50	6		
					3	95.40
	SS	4	25	5		
	SS	5	67	10	4	94.40
	SS	6	96	2	5	93.40
	SS	7	42	1	6	92.40
6.70	SS	8	100	1		
End of Borehole						
(GWL @ 2.03m-July 28, 2015)						
(GWL @ 1.68m-Nov. 23, 2015)						



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FILE NO. **PG3563**

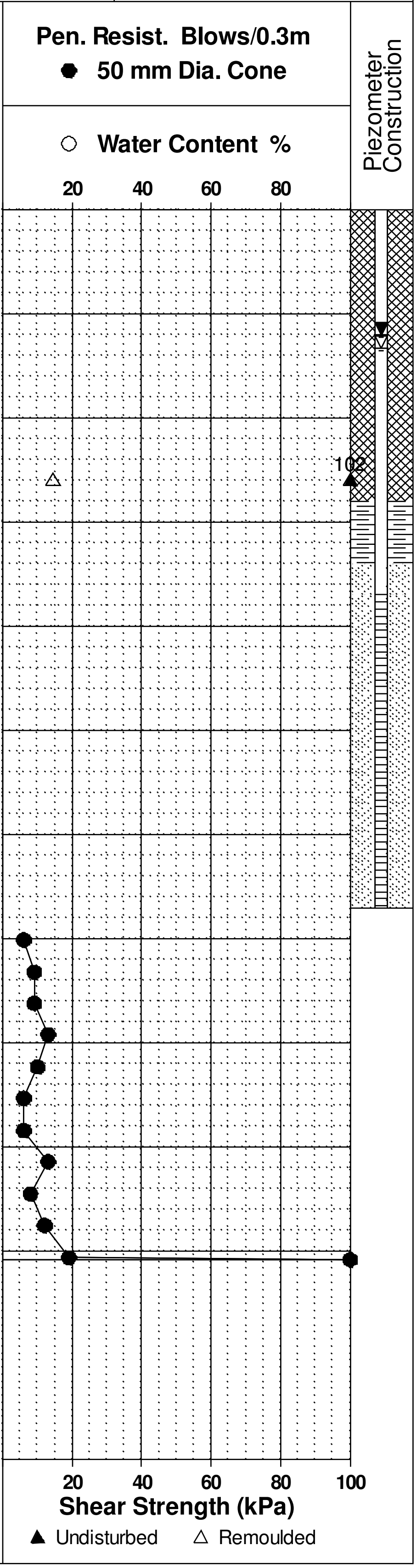
REMARKS

HOLE NO. **BH 5**

BORINGS BY CME 55 Power Auger **DATE** 14 July 2015

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.30	AU	1		
FILL: Brown sandy silt, trace gravel and cobbles 0.71				
Very stiff, grey **SILTY CLAY,** trace sand 2.95	SS	2	75	12
	SS	3	100	4
GLACIAL TILL: Compact, grey silty sand with gravel, some cobbles, trace clay 6.70	SS	4	62	17
	SS	5	75	30
	SS	6	100	18
	SS	7	62	11
	SS	8	58	10
Dynamic Cone Penetration Test commenced at 6.70m depth Inferred **GLACIAL TILL** 10.08				
End of Borehole Practical DCPT refusal at 10.08m depth (GWL @ 1.33m-July 28, 2015) (GWL @ 1.20m-Nov. 23, 2015)				

DEPTH (m)	ELEV. (m)
0	98.30
1	97.30
2	96.30
3	95.30
4	94.30
5	93.30
6	92.30
7	91.30
8	90.30
9	89.30
10	88.30



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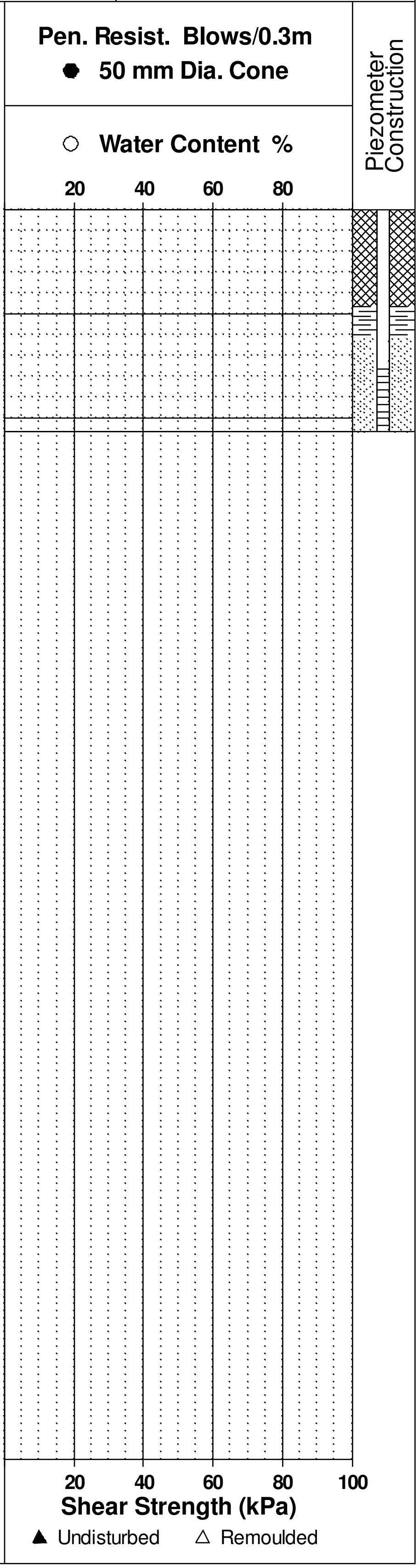
DATE 9 November 2015

FILE NO. PG3563

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.30
FILL: Brown silty clay, some sand and gravel 0.15		AU	1				
Very stiff to stiff, brown **SILTY CLAY**		SS	2	83	10	1	97.30
2.13		SS	3	92	6	2	96.30
End of Borehole (BH dry based on field observations)							



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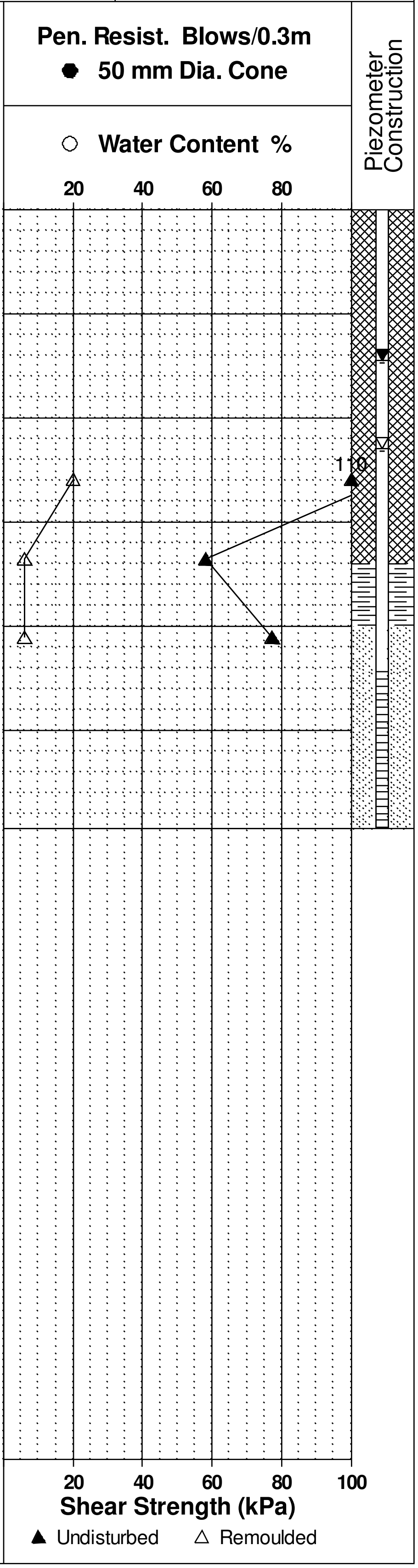
HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL 0.20		AU	1			0	98.28
		SS	2	75	8	1	97.28
Very stiff to stiff, brown **SILTY CLAY**		SS	3	25	6		
						2	96.28
- grey by 2.3m depth							
						3	95.28
- some sand by 3.8m depth 4.27						4	94.28
GLACIAL TILL: Compact, grey silty sand, some gravel, trace cobbles and boulders						5	93.28
5.94		SS	4	71	16		

End of Borehole

(GWL @ 2.3m depth based on field observations)

(GWL @ 1.45m-Nov. 23, 2015)



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REMARKS

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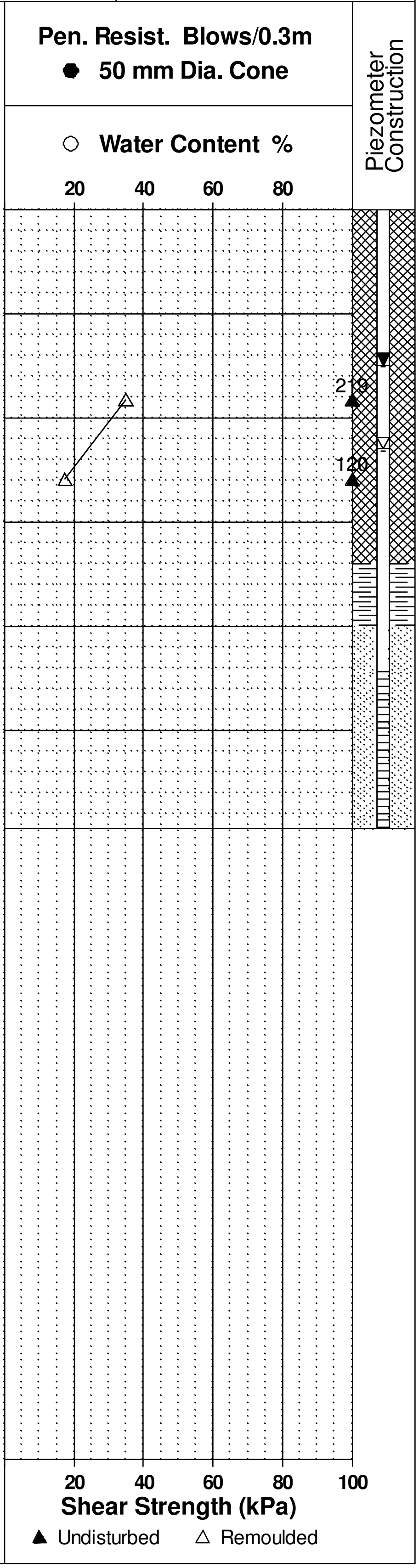
DATE 10 November 2015

FILE NO. PG3563

HOLE NO. BH 8

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.23	AU	1		
Hard to very stiff, brown **SILTY CLAY,** some sand		SS	2	88	8
- grey by 2.2m depth	2.95				
GLACIAL TILL: Grey silty sand, some gravel and cobbles, trace boulders		SS	3	50	10
	5.23	SS	4	4	2
GLACIAL TILL: Grey silty clay with sand, gravel, some cobbles, trace boulders	5.94	SS	5	79	2
End of Borehole					
(GWL @ 2.3m depth based on field observations)					
(GWL @ 1.49m-Nov. 23, 2015)					

DEPTH (m)	ELEV. (m)
0	98.23
1	97.23
2	96.23
3	95.23
4	94.23
5	93.23



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SOIL PROFILE AND TEST DATA

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DATUM Ground surface elevations provided by Tomlinson Group.

FILE NO. PG3563

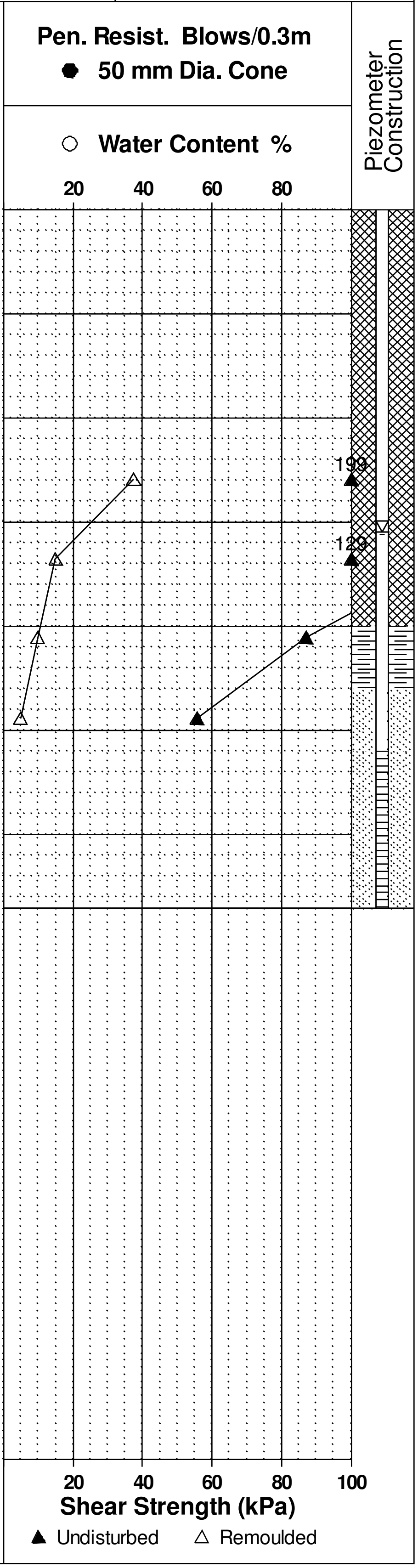
REMARKS

HOLE NO. BH 9

BORINGS BY CME 55 Power Auger

DATE 10 November 2015

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.86
FILL: Brown silty clay with sand		AU	1				
0.66							
TOPSOIL							
0.86							
		SS	2	75	9	1	97.86
		SS	3	50	8		
						2	96.86
Very stiff to stiff, brown **SILTY CLAY**							
						3	95.86
- grey by 3.5m depth							
						4	94.86
						5	93.86
5.18							
		SS	4	75	2		
GLACIAL TILL: Very loose, grey silty sand, some gravel, trace cobbles and boulders						6	92.86
		SS	5	83	2		
6.70							
End of Borehole (GWL @ 3.1m depth based on field observations)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Buildings - Block 16 Citigate 416
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by Tomlinson Group.

FILE NO. **PG3563**

REMARKS

HOLE NO. **BH10**

BORINGS BY CME 55 Power Auger

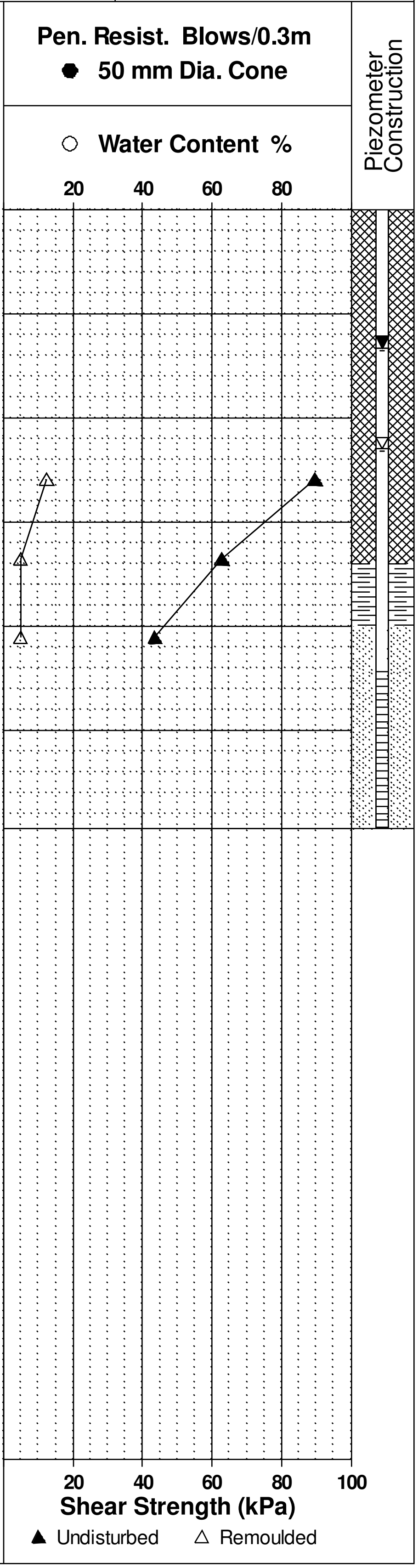
DATE 10 November 2015

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.25	AU	1		
Very stiff to firm, brown **SILTY CLAY,** trace sand		SS	2	71	7
		SS	3	100	4
	4.27				
GLACIAL TILL: Grey silty clay with sand and gravel, some cobbles, trace boulders		SS	4	100	2
	5.94	SS	5	46	8
End of Borehole					

(GWL @ 2.3m depth based on field observations)

(GWL @ 1.338m-Nov. 23, 2015)

DEPTH (m)	ELEV. (m)
0	98.22
1	97.22
2	96.22
3	95.22
4	94.22
5	93.22



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 10 November 2015

FILE NO. PG3563

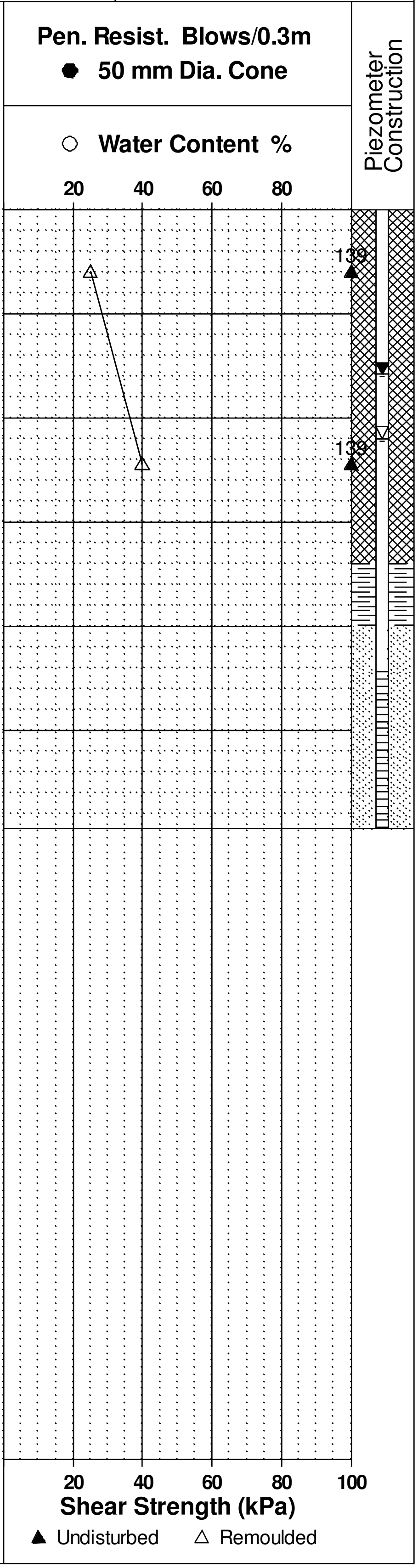
HOLE NO. BH11

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	98.44
TOPSOIL 0.23	AU	1				
Very stiff to stiff, brown **SILTY CLAY,** trace sand	SS	2	100	12	1	97.44
	SS	3	100	7	2	96.44
2.95					3	95.44
GLACIAL TILL: Loose, grey silty sand, some gravel, trace cobbles and boulders	SS	4	58	7	4	94.44
	SS	5	62	4	5	93.44
5.94	SS	6	54	7		

End of Borehole

(GWL @ 2.2m depth based on field observations)

(GWL @ 1.58m-Nov. 23, 2015)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Buildings - Block 16 Citigate 416
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by Tomlinson Group.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 10 November 2015

FILE NO. PG3563

HOLE NO. BH12

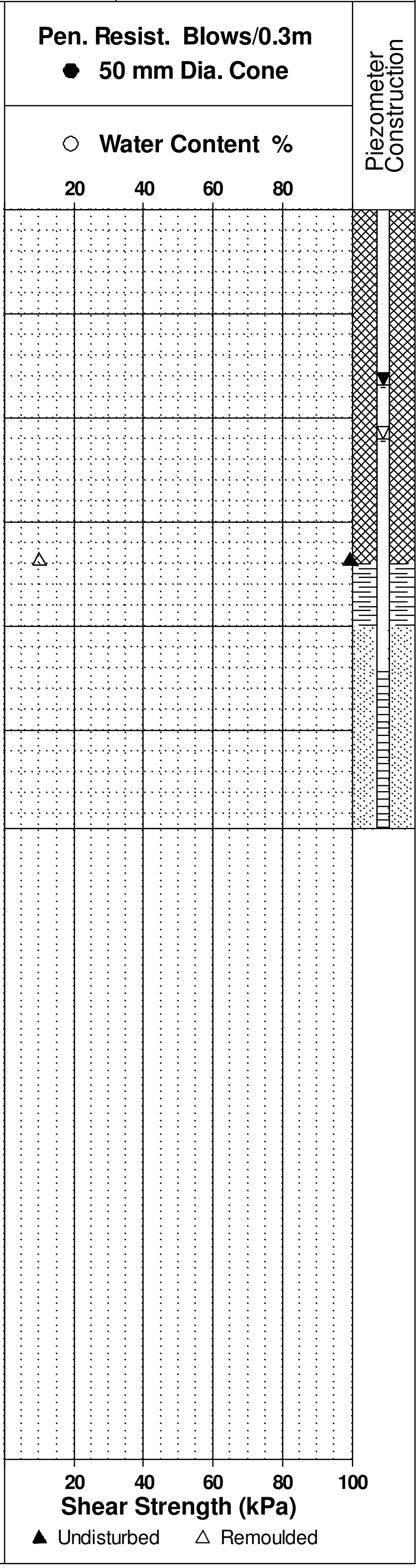
SOIL DESCRIPTION	STRATA PLOT (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Topsoil, trace gravel	0.40	AU	1		
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	79	13
		SS	3	100	7
	3.66	SS	4	100	3
GLACIAL TILL: Loose, grey silty sand with gravel, some cobbles, trace boulders		SS	5	62	4
	5.94	SS	6	54	4

End of Borehole

(GWL @ 2.2m depth based on field observations)

(GWL @ 1.68m-Nov. 23, 2015)

DEPTH (m)	ELEV. (m)
0	98.30
1	97.30
2	96.30
3	95.30
4	94.30
5	93.30



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SOIL PROFILE AND TEST DATA

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DATUM Ground surface elevations provided by Tomlinson Group.

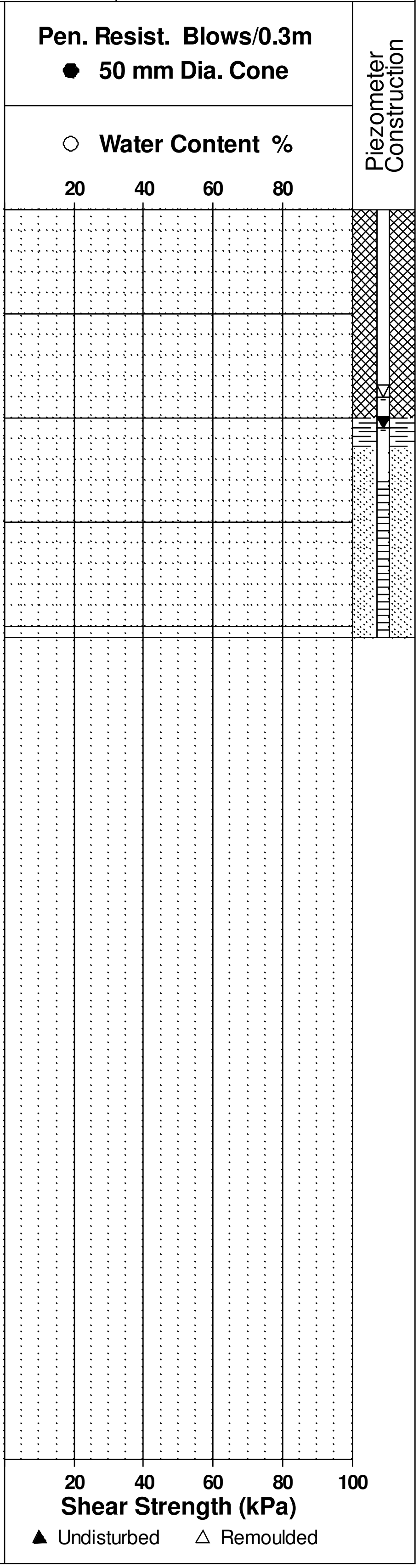
FILE NO. **PG3563**

REMARKS

HOLE NO. **BH13**

BORINGS BY CME 55 Power Auger **DATE** 10 November 2015

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	98.38
TOPSOIL 0.23	AU	1				
Very stiff, brown **SILTY CLAY**	SS	2	67	13	1	97.38
2.23	SS	3	100	6	2	96.38
GLACIAL TILL: Dense, grey silty sand, some gravel, cobbles, trace boulders					3	95.38
	SS	4	67	52		
4.11	SS	5	4	50+	4	94.38
End of Borehole (GWL @ 1.8m depth based on field observations) (GWL @ 2.10m-Nov. 23, 2015)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Buildings - Block 16 Citigate 416
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REMARKS

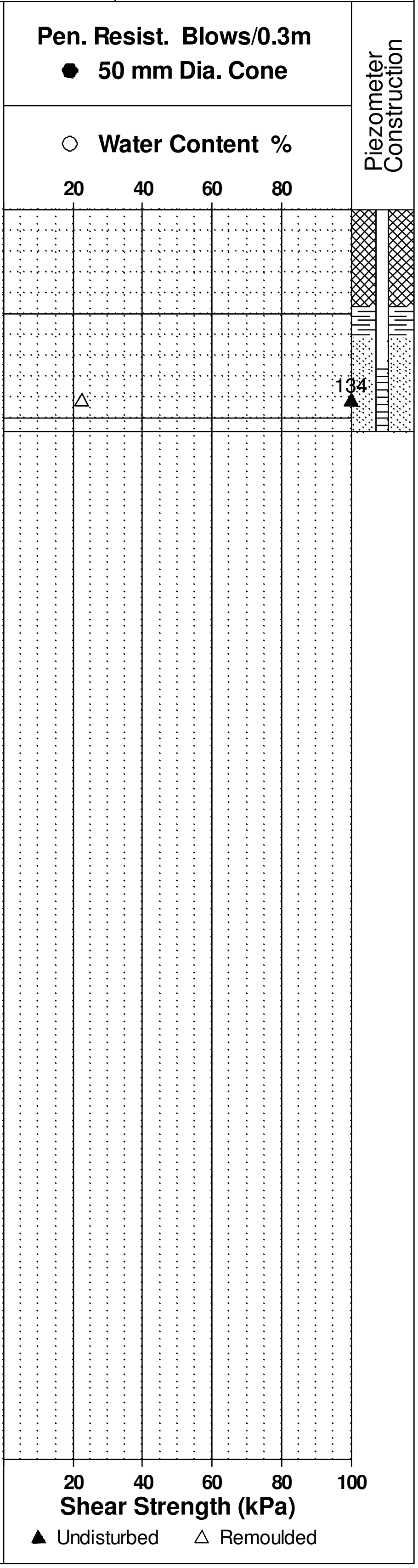
BORINGS BY CME 55 Power Auger

DATE 11 November 2015

FILE NO. PG3563

HOLE NO. BH14

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	98.03
FILL: Topsoil with clay, trace gravel		AU	1				
0.79							
Very stiff, brown **SILTY CLAY**		SS	2	100	10	1	97.03
		SS	3	79	5	2	96.03
2.13							
End of Borehole (BH dry based on field observations) (BH dry - Nov. 23, 2015)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Buildings - Block 16 Citigate 416
Strandherd Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by Tomlinson Group.

REMARKS

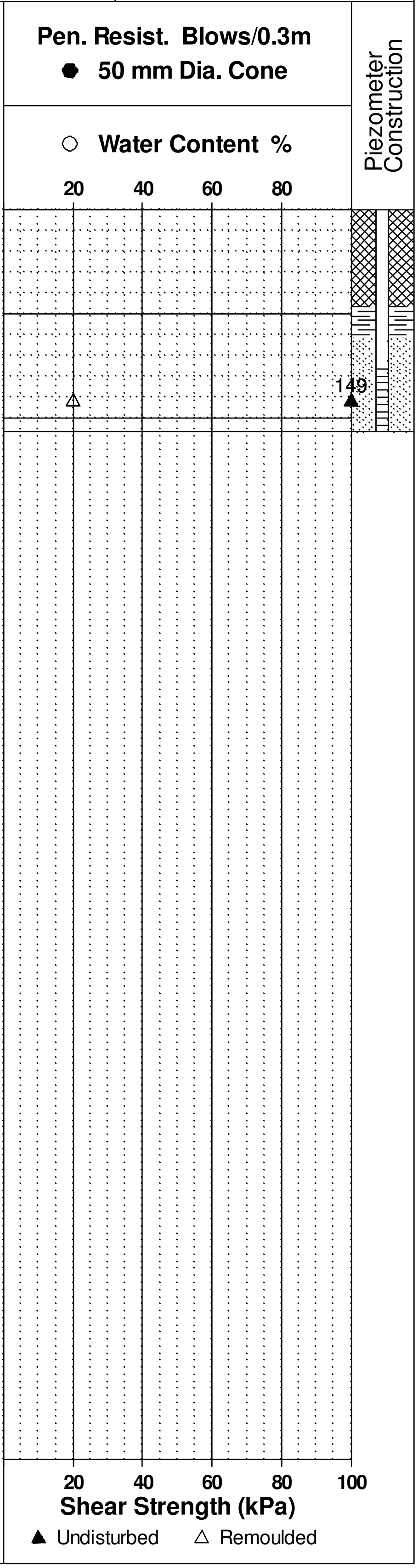
BORINGS BY CME 55 Power Auger

DATE 11 November 2015

FILE NO. PG3563

HOLE NO. BH15

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.70
TOPSOIL 0.25		AU	1				
Very stiff, brown **SILTY CLAY**		SS	2	67	10	1	97.70
- trace sand and grave by 2.0m depth 2.13		SS	3	83	5	2	96.70
End of Borehole (BH dry based on field observations) (BH Dry-Nov. 23, 2015)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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REMARKS

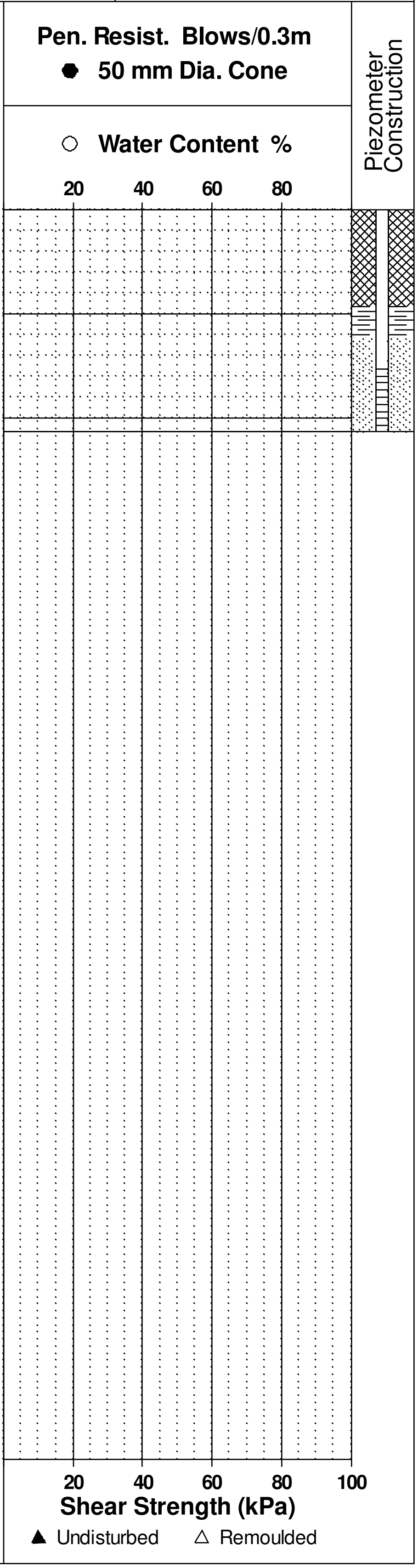
BORINGS BY CME 55 Power Auger

DATE 11 November 2015

FILE NO. PG3563

HOLE NO. BH16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.74
TOPSOIL 0.23		AU	1				
Very stiff, brown **SILTY CLAY** 1.68		SS	2	92	13	1	97.74
GLACIAL TILL: Compact, brown silty sand, some gravel, trace cobbles and boulders 2.13		SS	3	42	21	2	96.74
End of Borehole							
(BH dry based on field observations)							
(BH Dry-Nov. 23, 2015)							



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SOIL PROFILE AND TEST DATA

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REMARKS

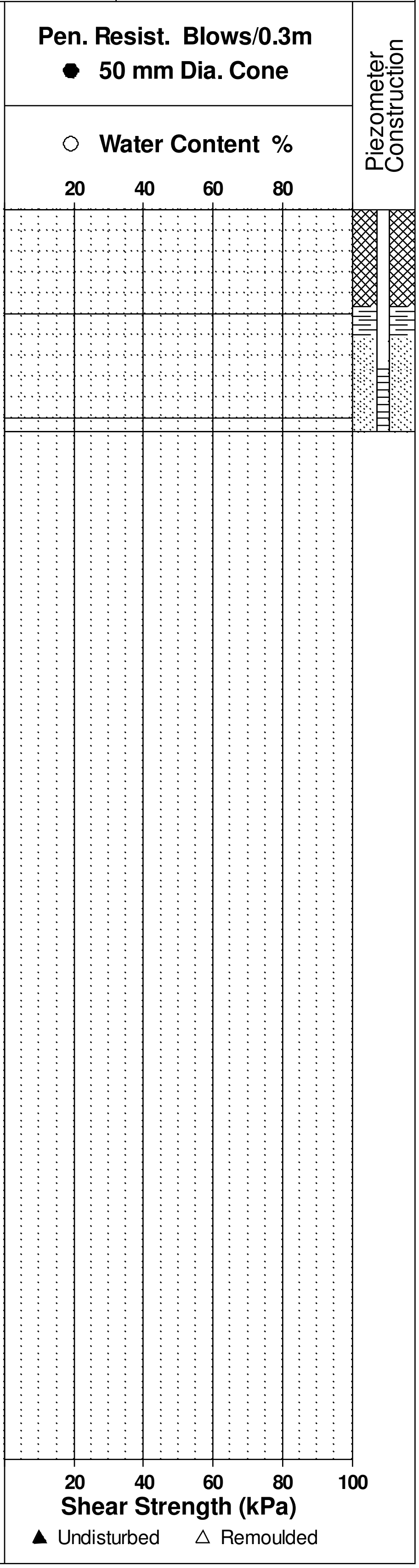
BORINGS BY CME 55 Power Auger

DATE 11 November 2015

FILE NO. PG3563

HOLE NO. BH17

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	98.81
FILL: Topsoil with clay, some gravel and cobbles 0.46	AU	1				
GLACIAL TILL: Compact to loose, brown silty sand, some gravel, trace cobbles and boulders	SS	2	38	10	1	97.81
2.13	SS	3	42	9	2	96.81
End of Borehole						
(BH dry based on field observations)						
(BH Dry-Nov. 23, 2015)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Buildings - Block 16 Citigate 416
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DATUM Ground surface elevations provided by Tomlinson Group.

FILE NO. **PG3563**

REMARKS

HOLE NO. **BH18**

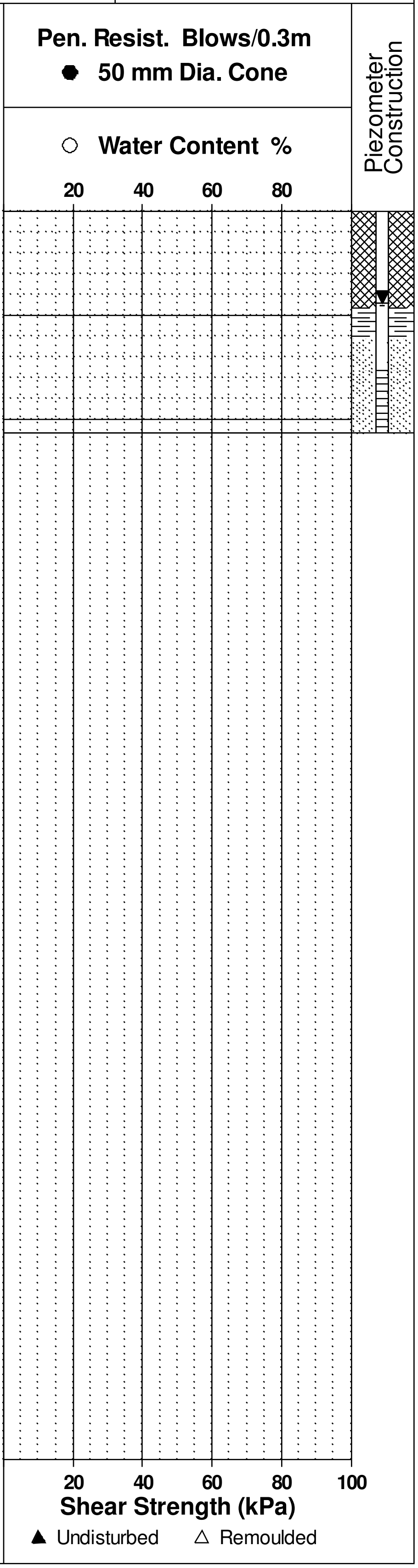
BORINGS BY CME 55 Power Auger

DATE 11 November 2015

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	99.48
FILL: Brown silty sand, some gravel, trace cobbles	AU	1				
1.07	SS	2	46	10	1	98.48
GLACIAL TILL: Compact to dense, brown silty sand, some gravel, trace cobbles and boulders	SS	3	58	30		
2.13					2	97.48
End of Borehole						

(BH dry based on field observations)

(GWL @ 0.88m-Nov. 23, 2015)



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)

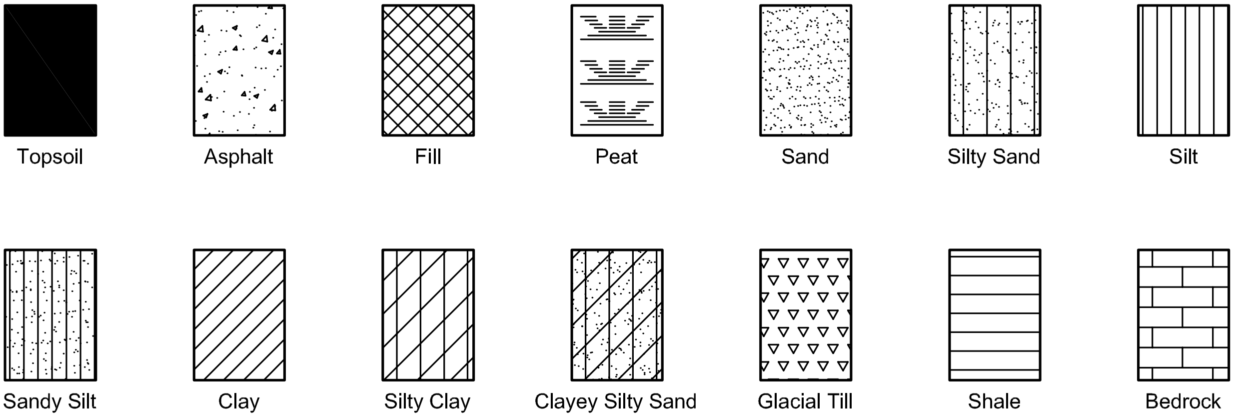
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

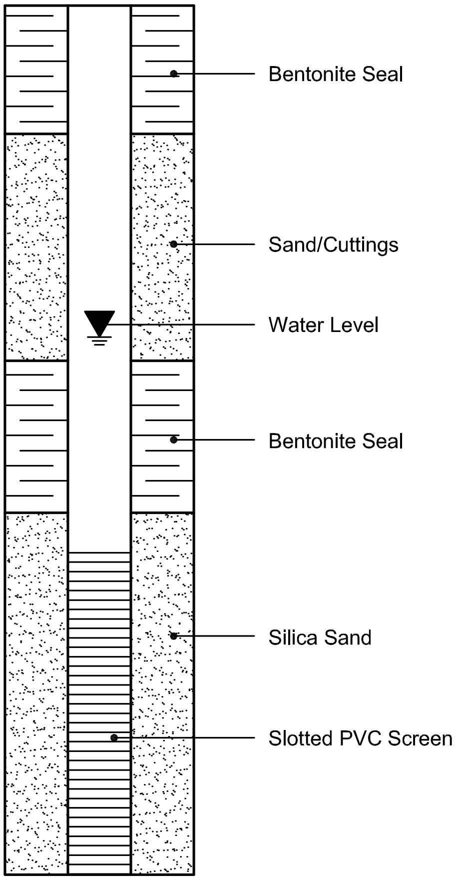
SYMBOLS AND TERMS (continued)

STRATA PLOT

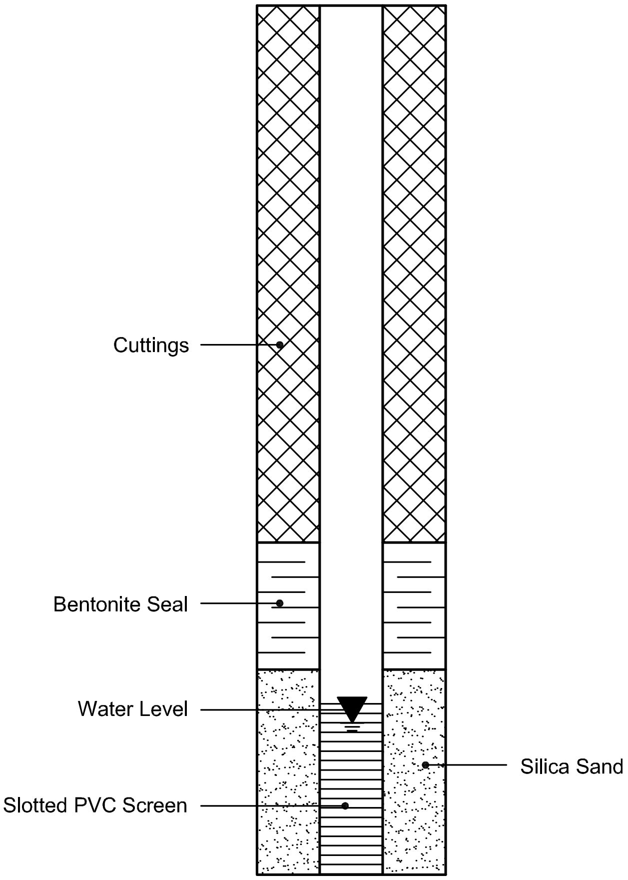


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Client ID:		BH2 SS4	-	-	-
Sample Date:		14-Jul-15	-	-	-
Sample ID:		1529208-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	90.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.79	-	-	-
Resistivity	0.10 Ohm.m	92.7	-	-	-
Anions					
Chloride	5 ug/g dry	7	-	-	-
Sulphate	5 ug/g dry	7	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 - SETTLEMENT PRELOAD MONITORING PROGRAM

FIGURE 5 - UPLIFT CONE ANGLE FOR BACKFILL MATERIAL

DRAWING PG3563-2 - TEST HOLE LOCATION PLAN

DRAWING PG3563-6 - REINFORCED SLOPE DETAILS FOR PROPOSED BERM

FIGURE 6 - SLOPE A-A’ - PROFILE OF REINFORCED SLOPE

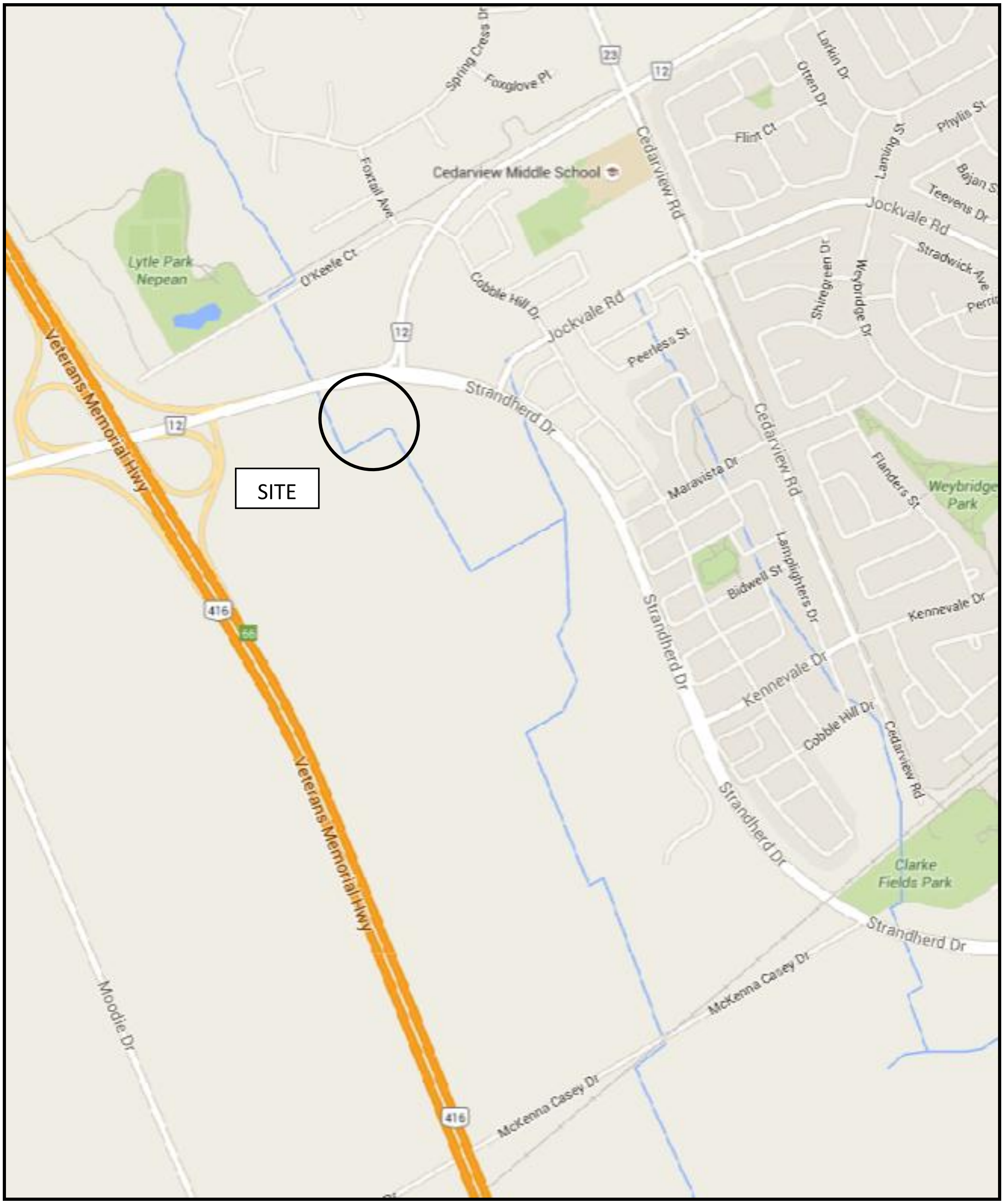


FIGURE 1

KEY PLAN

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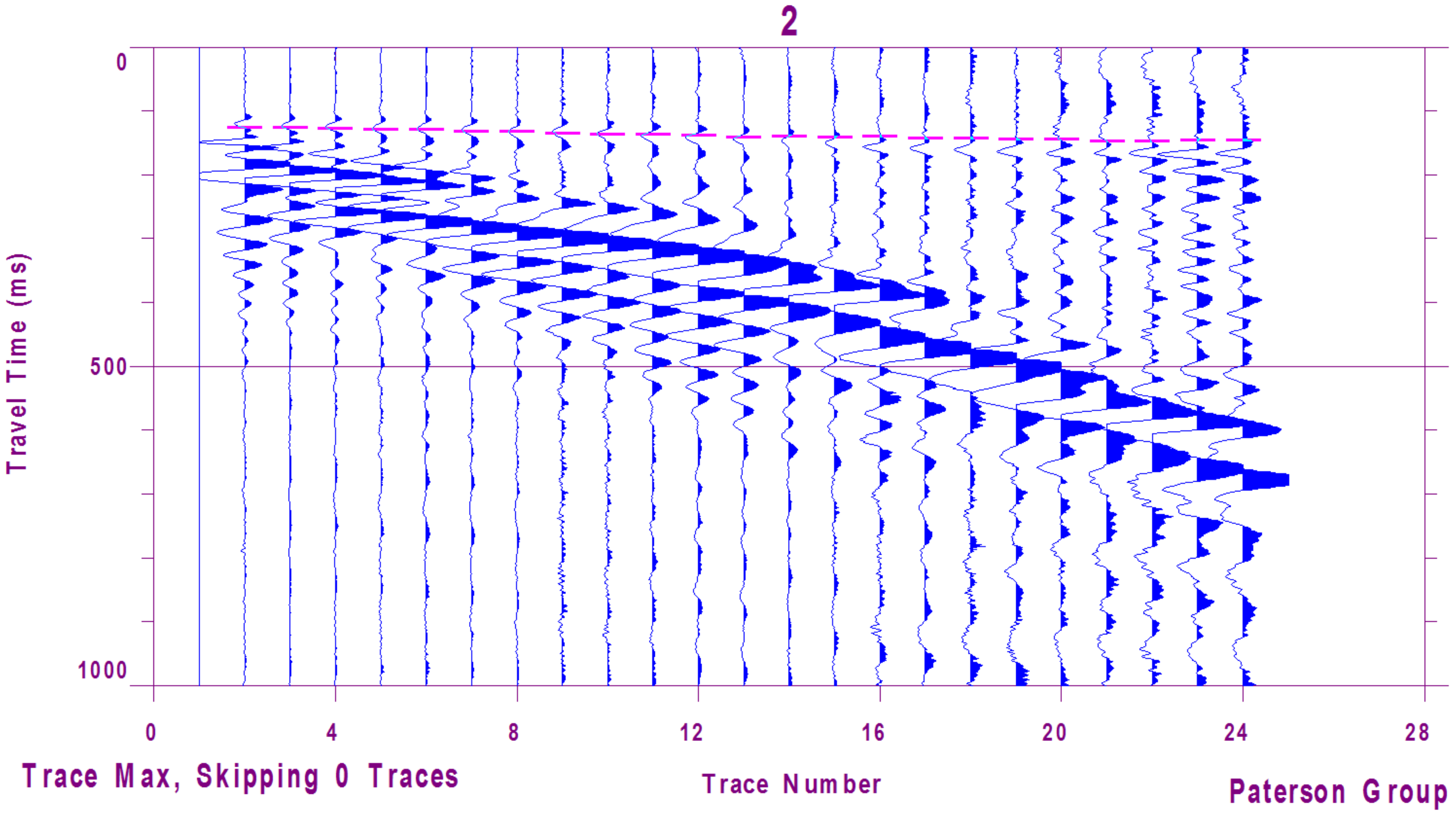


Figure 2 – Shear Wave Velocity Profile at Shot Location -30 m

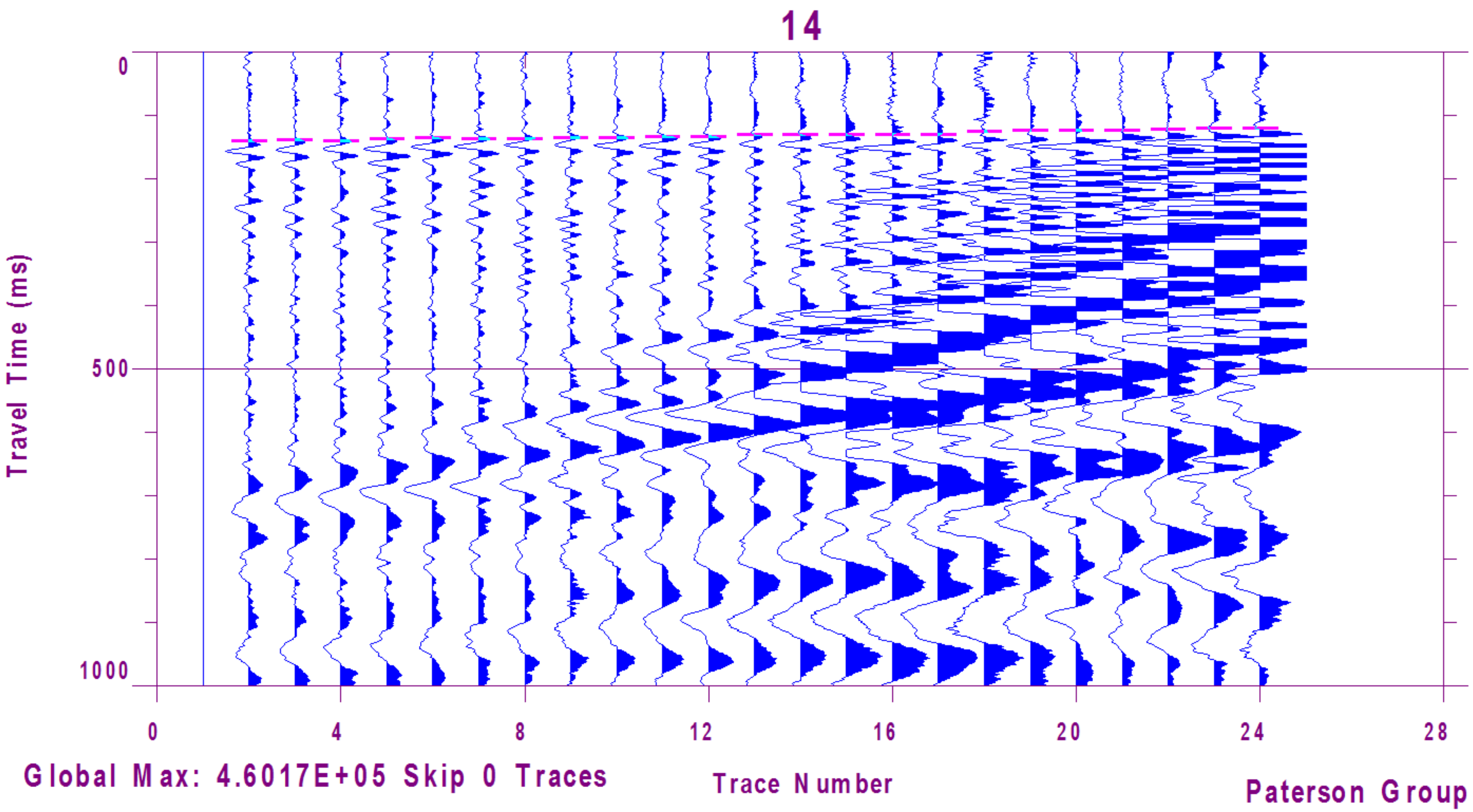
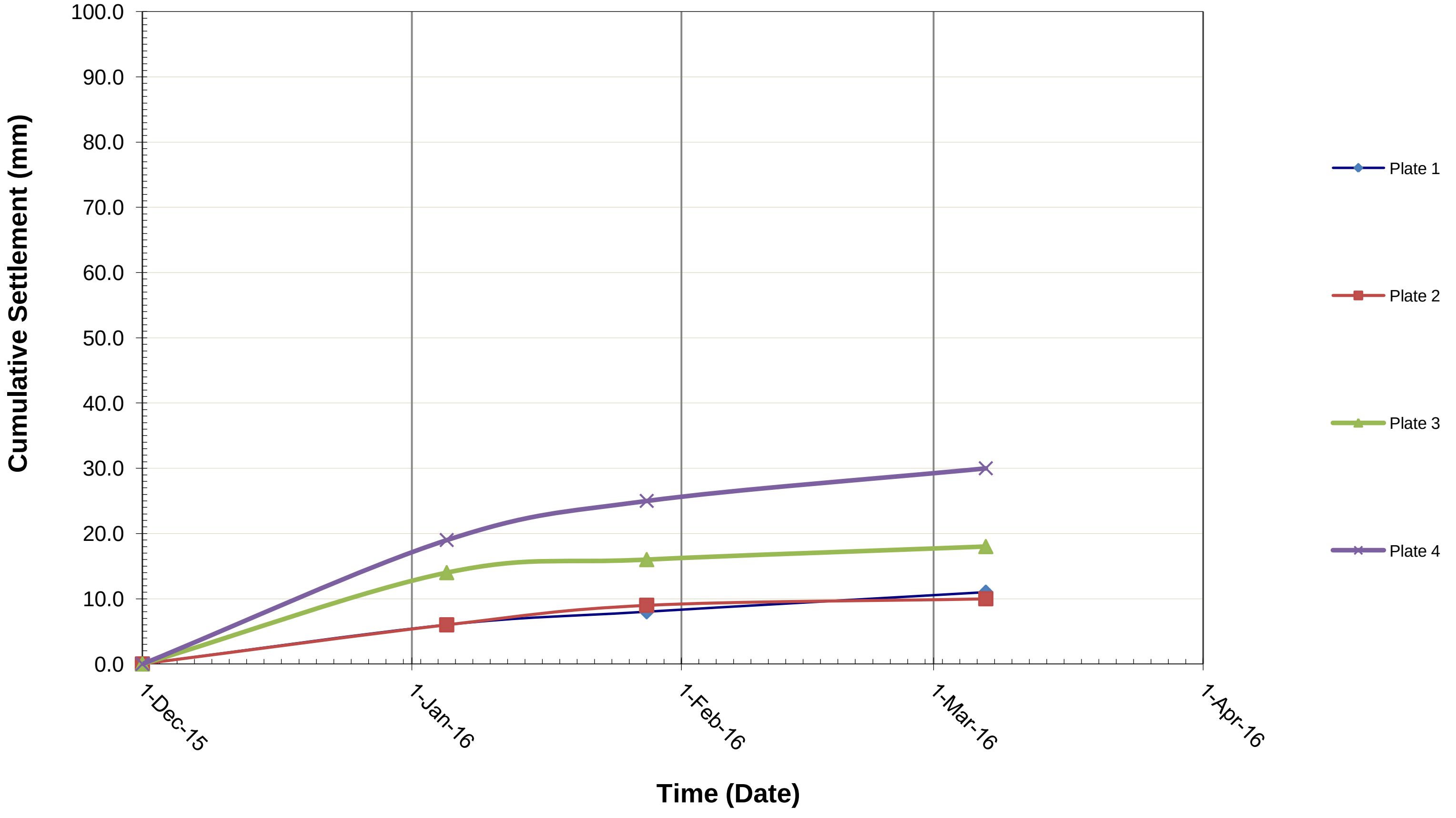
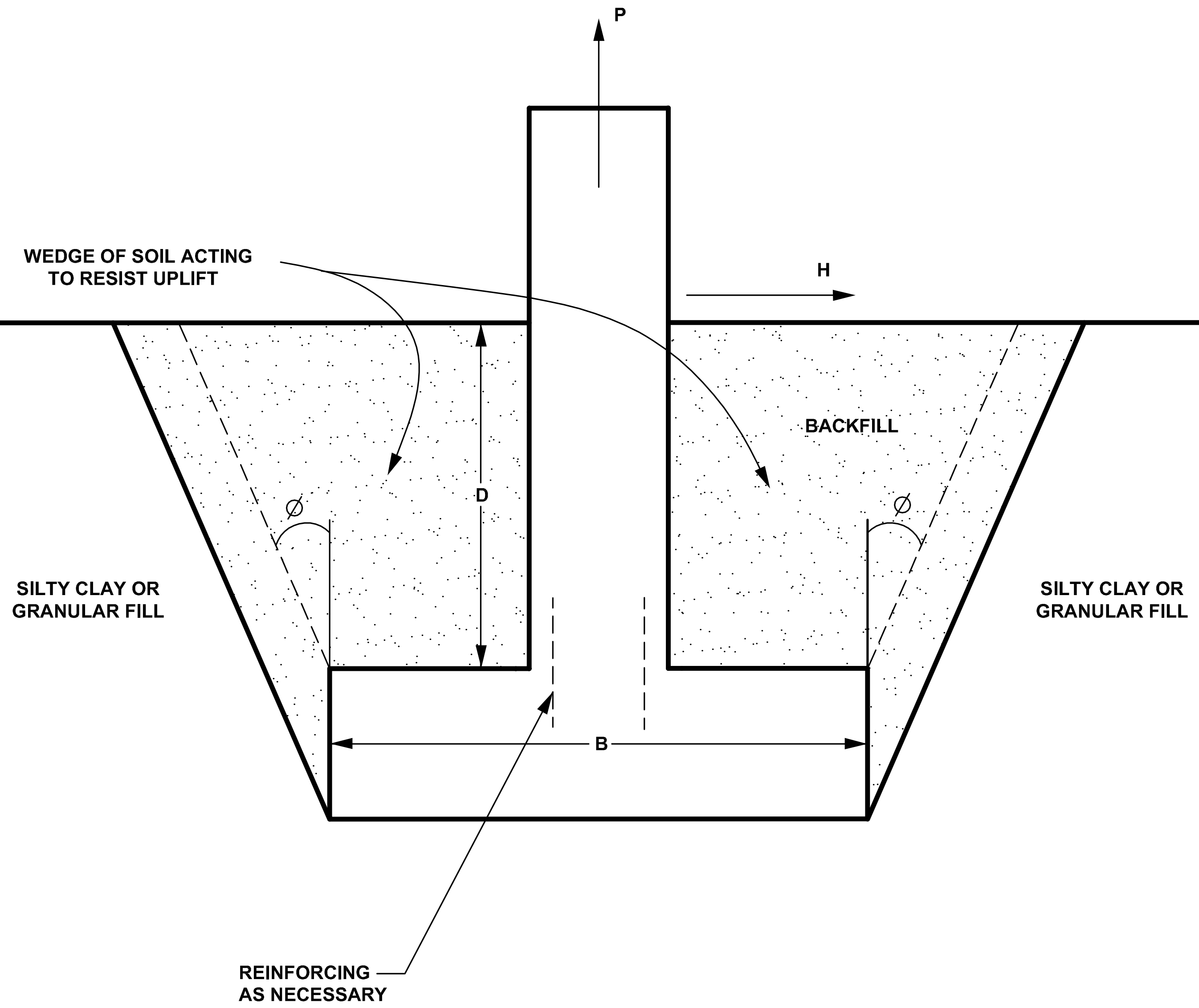


Figure 3 – Shear Wave Velocity Profile at Shot Location 99 m

Figure 4 - Settlement Preload Monitoring Program
Proposed Commercial Buildings - Block 16 - CitiGate 416 Development
Strandherd Drive - Ottawa





WHERE

P = UPLIFT LOAD
H = HORIZONTAL LOAD
B = WIDTH OF FOOTING

∅ - USE 20° FOR COHESIONLESS SOIL (ie.- GRANULAR BACKFILL COMPACTED TO 98% SPMDD)
- USE 30° FOR COHESIVE SOIL (ie.- SILTY CLAY COMPACTED WITH A SHEEPSFOOT ROLLER)

FIGURE 5

UPLIFT CONE ANGLES FOR BACKFILL MATERIAL

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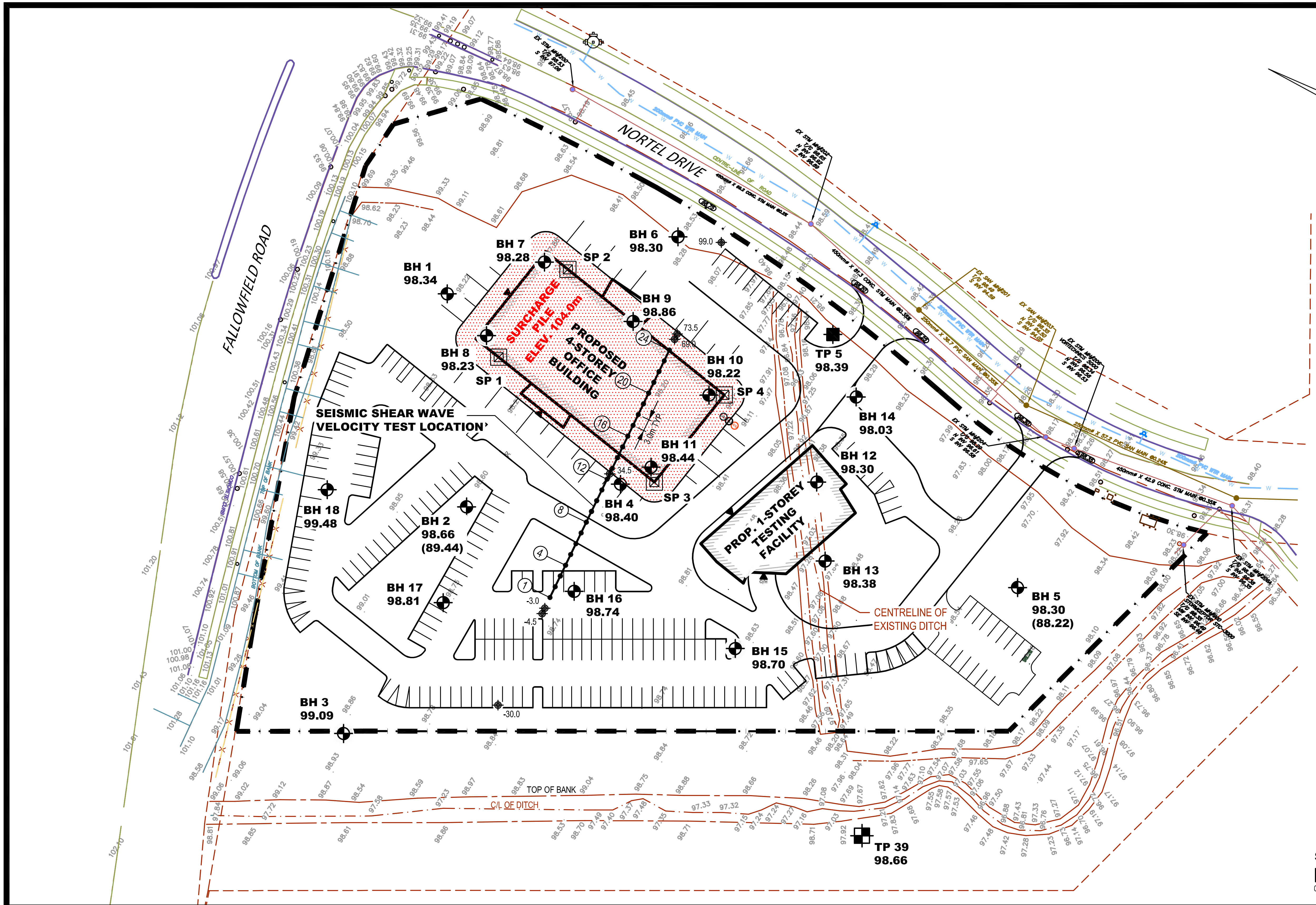
TOMLINSON GROUP

PROPOSED BUILDINGS - BLOCK 16 - CITIGATE
STRANDHERD DRIVE, OTTAWA, ONTARIO

Report No.:	Scale:	
PG3563	N.T.S.	
Date:	Drawn:	Approved:
04/2016	MPG	DJG



LEGEND:

☒	SETTLEMENT PLATE LOCATION
⊕	BOREHOLE LOCATION, CURRENT INVESTIGATION
▚	TEST PIT LOCATION, PREVIOUS INVESTIGATION PATERSON GROUP REPORT G7892
■	TEST PIT LOCATION, PREVIOUS INVESTIGATION PATERSON GROUP REPORT PG2449
98.30	GROUND SURFACE ELEVATION (m)
(88.22)	PRACTICAL DCPT REFUSAL ELEVATION (m)
-●-●-	GEOPHONE LOCATIONS
⑳	GEOPHONE NUMBER
99.0 ◈	SHOT LOCATION

BASE PLAN, BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS PROVIDED BY TOMLINSON GROUP.

SCALE: 1:1000

0 10 20 30 40 50m 75m

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NO.	REVISIONS	DATE	INITIAL
1	SETTLEMENT PLATE LOCATIONS ADDED	12/04/2016	DJG

TOMLINSON GROUP
GEOTECHNICAL INVESTIGATION
PROP. COMMERCIAL BUILDINGS - BLOCK 16 - CITIGATE 416
STRANDHERD DRIVE
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 08/2015
Drawn by: MPG	Report No.: PG3563-2
Checked by: RG	Dwg. No.: PG3563-2
Approved by: DJG	Revision No.: 1

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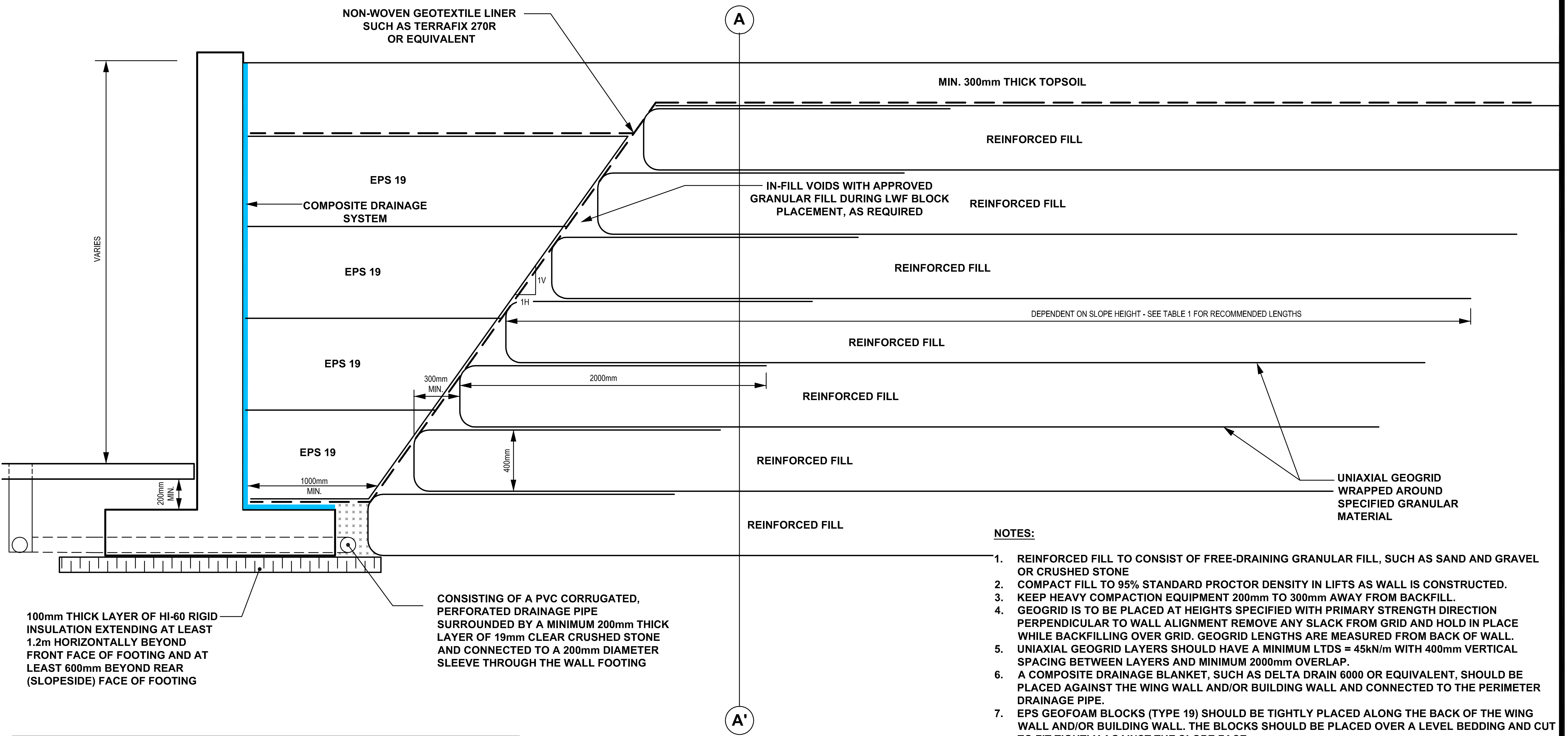

NOTES:

1. REINFORCED FILL TO CONSIST OF FREE-DRAINING GRANULAR FILL, SUCH AS SAND AND GRAVEL OR CRUSHED STONE
2. COMPACT FILL TO 95% STANDARD PROCTOR DENSITY IN LIFTS AS WALL IS CONSTRUCTED.
3. KEEP HEAVY COMPACTION EQUIPMENT 200mm TO 300mm AWAY FROM BACKFILL.
4. GEOGRID IS TO BE PLACED AT HEIGHTS SPECIFIED WITH PRIMARY STRENGTH DIRECTION PERPENDICULAR TO WALL ALIGNMENT REMOVE ANY SLACK FROM GRID AND HOLD IN PLACE WHILE BACKFILLING OVER GRID. GEOGRID LENGTHS ARE MEASURED FROM BACK OF WALL.
5. UNIAXIAL GEOGRID LAYERS SHOULD HAVE A MINIMUM LTDS = 45kN/m WITH 400mm VERTICAL SPACING BETWEEN LAYERS AND MINIMUM 2000mm OVERLAP.
6. A COMPOSITE DRAINAGE BLANKET, SUCH AS DELTA DRAIN 6000 OR EQUIVALENT, SHOULD BE PLACED AGAINST THE WING WALL AND/OR BUILDING WALL AND CONNECTED TO THE PERIMETER DRAINAGE PIPE.
7. EPS GEOFOAM BLOCKS (TYPE 19) SHOULD BE TIGHTLY PLACED ALONG THE BACK OF THE WING WALL AND/OR BUILDING WALL. THE BLOCKS SHOULD BE PLACED OVER A LEVEL BEDDING AND CUT TO FIT TIGHTLY AGAINST THE SLOPE FACE.
8. SHAPE REINFORCED SLOPE FACE TO BE PROFILED AT 1V : 1H PROFILE.
9. PLACE NON-WOVEN GEOTEXTILE LINER, SUCH AS TERRAFIX 270R OR EQUIVALENT, OVER FACE OF REINFORCED SLOPE TO PREVENT LOSS OF TOPSOIL INTO UNDERLYING GRANULAR FILL LAYER.
10. THE WALL BASE DESIGN ASSUMED A BEARING RESISTANCE AT SLS OF 150kPa. THE SITE DEPARTMENTAL REPRESENTATIVE SHOULD OBSERVE THE BEARING CONDITIONS AND ADJUST THE THICKNESS OF THE GRANULAR BASE TO ACCOMMODATE THE SITE CONDITIONS, IF NECESSARY.
11. GEOTECHNICAL FIELD REVIEW IS REQUIRED DURING CONSTRUCTION.

TABLE 1 -

UNIAXIAL GEOGRID LENGTHS	HORIZONTAL GEOGRID LENGTH(m)
SLOPE HEIGHTS BETWEEN 4 TO 6m HIGH	5.0m
SLOPE HEIGHTS BETWEEN 2 TO 3.99m HIGH	3.0m
SLOPE HEIGHTS BETWEEN 0.5 TO 1.99m HIGH	1.5m

NOTE:
DRAINAGE PIPE FOR WING WALL TO BE CONNECTED TO SUB-FLOOR DRAIN BELOW WALKWAY AREA. SUB-FLOOR DRAIN BELOW WALKWAY SHOULD BE CONNECTED TO A NEARBY CATCH BASIN DIRECTED TOWARDS THE STORM SEWER SYSTEM.

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NO.	REVISIONS	DATE	INITIAL
0			

TOMLINSON GROUP
GEOTECHNICAL INVESTIGATION
PROPOSED OFFICE BUILDING - BLOCK 16 - CITIGATE 416
OTTAWA, ONTARIO

Title:
REINFORCED SLOPE DETAIL FOR PROPOSED BERM

Scale: 1:25
Date: 06/2016
Drawn by: MPG
Report No.: PG3563
Checked by: FA
Dwg. No.: PG3563-6
Approved by: DJG
Revision No.: 0

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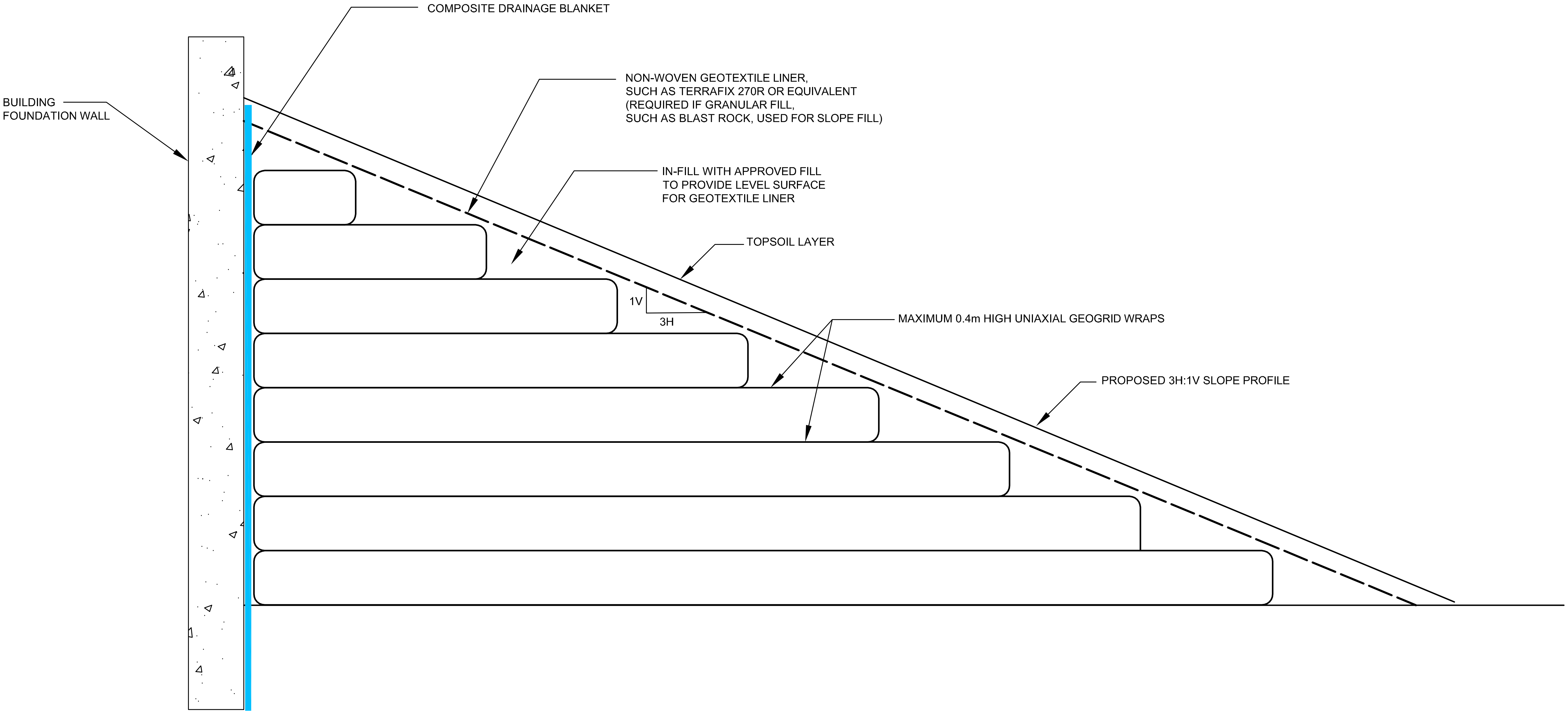

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NO.	REVISIONS	DATE	INITIAL
0			

TOMLINSON GROUP

GEOTECHNICAL INVESTIGATION
PROPOSED OFFICE BUILDING - BLOCK 16 - CITIGATE 416

OTTAWA, ONTARIO

Title:

SECTION A-A' - PROFILE OF REINFORCED SLOPE

Scale: 1:25
Drawn by: RG
Checked by: FA
Approved by: DJG
Date: 06/2016
Report No.: PG3563
Dwg. No.: **FIG. 6**
Revision No.: 0

p:\autocad drawings\geotechnical\pg35xx\pg3563 figure 6(cross section).dwg