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Geotechnical Engineering

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Geotechnical Investigation

Proposed Warehouse Complex
5510 Boundary Road
Ottawa, Ontario

Prepared For

Day & Ross
c/o Avenue 31 Capital Inc.

September 10, 2018

Report PG4592-1

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

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

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Avenue 31 Capital Inc on behalf of Day & Ross to conduct a geotechnical investigation for the proposed warehouse complex to be located at 5510 Boundary Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.

☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. 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

Based on the available drawings, it is understood that the proposed development will consist of a slab-on-grade, single-storey warehouse building with limited office space. The warehouse will be surrounded by associated access lanes, loading dock and parking areas. Truck traffic will be a large component of the vehicle loading on the pavement structure.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on August 1 and 3, 2018. A total of 4 boreholes were drilled and sampled to a maximum depth of 9.75 m below existing ground surface. In addition, 12 test pits were excavated to a maximum depth 3.5 below existing grade. The test hole locations were distributed in a manner to provide general coverage of the subject site taking into consideration site features. The locations of the test holes are shown on Drawing PG4592-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed with a track-mounted auger drill rig operated by a two-person crew where the test pits were excavated using a hydraulic shovel. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The test hole procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler, a Shelby tube, or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon, Shelby tubes and auger samples were recovered from the boreholes are shown as SS, TW and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) at BH1. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

One 32 mm diameter monitoring well was installed in BH 4-18 and 3 flexible standpipe piezometers were installed in the remainder boreholes and the test pits conducted in July 2014 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

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 directed otherwise.

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 site features.

The test hole locations and ground surface elevations were surveyed by Annis, O'Sullivan, Vollebekk Ltd. and are presented on Drawing PG4592-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

A total of 3 Shelby tube samples were submitted for unidimensional consolidation testing from the boreholes completed for our investigation. The results of the consolidation testing are presented on the Unidimensional Consolidation Test Results sheets presented in Appendix 1 and are further discussed in Sections 5.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and generally covered with grass, small brush, forested areas and scattered fill piles. Two gravel covered access points were noted along the east portion of the site. The ground surface across the subject site is relatively flat and at grade with the adjacent Boundary Road. Water ponds were noted along eastern central portion and northern borderline of the site. A small drainage ditch was noted along the center of the site running south-north and a drainage swale was noted bordering the site along the east border line. Based on our review of the historical aerial maps, the center of the site was previously a water ponded area which was in-filled with miscellaneous fills over the course of the last 10 years.

4.2 Subsurface Profile

Overburden

The subsurface profile encountered at the test hole locations consists of fill consisting of silty clay with sand, gravel and cobbles overlying a thin layer of very loose to compact silty sand. The above noted layers were underlain by a deep soft to firm grey silty clay deposit. Practical refusal to DCPT was encountered at BH 1 at 24 m below existing grade. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in this area consists of shale of the Carlsbad formation with an overburden drift thickness of 25 to 35 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring well and piezometers at the borehole locations on August 8, 2018. The measured groundwater level (GWL) readings are presented in Table 1 below. Based on our field observations, experience with the local area, moisture levels and the colouring of the recovered samples, it is expected that the groundwater level is between 2 to 3 m below the existing grade. It should be noted that groundwater levels are subject to seasonal fluctuations and, therefore, groundwater levels could differ at the time of construction.

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elev. (m)	Measured Groundwater Level		Recording Date
		Depth (m)	Elevation (m)	
BH 1 (MW)	78.29	1.30	77.00	August 8, 2018
BH 2	78.67	6.49	72.18	August 8, 2018
BH 3	77.67	1.29	76.38	August 8, 2018
BH 4	77.98	1.08	76.90	August 8, 2018
Summary of Open Hole Groundwater Level (TP)				
TP1	77.87	3.10	74.77	August 1, 2018
TP1	77.56	0.60	76.96	August 1, 2018
TP3	77.51	3.30	74.21	August 1, 2018
TP4	77.66	0.75	76.91	August 1, 2018
TP5	77.93	2.20	75.73	August 1, 2018
TP6	78.45	3.00	75.45	August 1, 2018
TP7	78.00	2.10	75.90	August 1, 2018
TP8	78.00	Dry	n/a	August 1, 2018
TP9	78.17	0.65	77.52	August 1, 2018
TP10	78.27	Dry	n/a	August 1, 2018
TP11	78.07	3.10	74.97	August 1, 2018
TP12	76.25	2.60	73.65	August 1, 2018
Note: Ground surface elevations at the test hole locations were provided by Annis, O'Sullivan, Vollebekk, Ltd. and are assumed to be referenced to a geodetic datum.				

The long term groundwater level is expected to be between elevation 75 and 76 m.

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Options

From a geotechnical perspective, the subject site is considered satisfactory for the proposed warehouse complex. It is expected that the proposed building can be founded on end bearing piles along with spread footing foundations. Also, any light duty external structures can be founded on conventional spread footings bearing on an undisturbed, silty sand or stiff silty clay deposits, or engineered fill placed over an undisturbed, native bearing medium.

Permissible Grade Raise

Due to the presence of a silty clay deposit, the site will be subject to a permissible grade raise restriction. It should be noted that a fill layer ranging in depth between 1.8 to 3.1 m below existing grade was noted during the field investigation. Based on our review of the available historical aerial mapping, for the past 20 years, the site has been used as a fill placement area. Therefore, due to the significant thickness of fill encountered across the site, the permissible grade raise restrictions were calculated based on the native soil surface.

Lightweight Fill Recommendations

Due to the time constraints and thickness of the existing clay deposit, it is understood that surcharging the area to induce long term settlements will not be possible for the subject site based on the final finished grades. Therefore, a combination of granular materials and lightweight fill (EPS Type 19 blocks) will be required to manage the long term settlements associated with a floor slab that will behave independently from the main building structure. The lightweight fill recommendations can be provided once a finalized grading plan is available.

Existing Fill - Original Versus Existing Ground Surface

Based on the current findings, the original ground surface is expected to be at an approximate geodetic elevation of 76 m within the north portion of the site which slopes down to 74.5 m along the south portion of the site. It is suspected that the lower original ground surface may be associated with a drainage area since the surrounding area is relatively flat.

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

5.2 Site Grading and Preparation

Stripping Depth

Consideration can be given to leaving the existing fill, free of significant amounts of deleterious materials, below the proposed building floor slab outside of the lateral support zone of the proposed footings and within the proposed parking areas and access lanes. However, it is recommended that the existing fill be approved by the geotechnical consultant once the subgrade level is determined at design stage and exposed during the construction phase.

The approved existing fill material should be proof-rolled using suitable compaction equipment under dry conditions and reviewed by Paterson personnel. Poor performing areas should be removed and replaced with engineered fill.

Any topsoil remaining on site and fill, containing deleterious or organic materials, should be stripped from under any building, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of the material's 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. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Pile Foundation

A deep foundation system driven to refusal in the bedrock is recommended for foundation support of the proposed building. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance values at ultimate limit states (ULS) are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored at ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance	Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		Factored at ULS (kN)		
245	9	1495	25	40
245	11	1750	24	48.5
245	13	2000	25	56

The minimum centre-to-centre pile spacing is 2.5 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Due to the proposed grade raises at the site, downdrag loads should be considered on the piles. Based on the available subsurface information, it is expected that the piles will be driven through approximately 21 to 26 m of stiff to soft silty clay. The silty clay generally has a cohesion of 20 to 40 kPa. Assigning an adhesion factor of 1.0 to 0.5, the silty clay can be taken to have an ultimate adhesion of 20 kPa against the sides of the piles. As such, the estimated downdrag load for each 245 mm diameter pile is anticipated to vary from 325 kN for a 21 m pile to 400 kN for a 26 m pile.

The downdrag load is effectively applied to each pile at the location of the “neutral plane,” where negative (i.e. downdrag) skin friction becomes positive shaft resistance. In the case of the end-bearing piles at this site, the neutral plane will be located near the bedrock surface.

The downdrag load is a structural pile capacity criterion and does not affect the geotechnical capacity of the piles. The structural axial capacity of the pile is governed by its structural strength at the neutral plane when subjected to the permanent load plus the downdrag load. Transient live load is not to be included. At or below the pile cap, the structural strength of the embedded pile is determined as a short column subjected to the permanent load plus the transient live load, but downdrag load is to be excluded.

At the depth of the neutral plane where the downdrag load is applied, the pile structure is well confined. The 4th edition of the Canadian Foundation Engineering Manual recommends that the allowable structural axial capacity of piles at the neutral plane, for resisting permanent load plus the downdrag load, can be determined by applying a factor of safety of 1.5 to the pile material strength (steel yield and concrete 28 day compressive strength).

Lateral Load Resistance

Lateral loads on the foundations can be resisted using passive resistance on the sides of the foundations. For Limit States Design, the resistance factor to be applied to the ultimate lateral resistance, including passive pressure, is 0.50. The total lateral resistance will be comprised of the individual contributions from up to several material layers, as follows.

Geotechnical parameters for the native sand and for typical backfill materials compacted to 98% of SPMDD in 300 mm lift thicknesses are provided in Table 3, below, along with the associated earth pressure coefficients for horizontal resistance calculations for footings under lateral loads or deadman anchors. Friction factors between concrete and the various subgrade materials are also provided in Table 3, where normal loads allow them to be used.

Where granular soils and/or granular backfill materials are present, the passive pressure can be calculated using a triangular distribution equal to $K_P \cdot \gamma \cdot H$ where:

K_P = factored passive earth pressure coefficient of the applicable retained soil, 1.5
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the equivalent wall or footing side (m)

Note that for cases where the depth to the top of the structure (i.e. footing) pushing against the soil does not exceed 50% of the depth to the base of the structure, the effective value of H in the above noted relationship will be the overall depth to the base of the structure. There will also be “edge effects” where the effective width of soil providing the resistance can be increased by 50% of the effective depth on each side of the pushing structural component.

Note that where the foundation extends below the groundwater level, the effective unit weight should be utilized for the saturated portion of the soil or fill.

Where a component of lateral resistance is to be provided by the EPS foam lightweight fill (LWF) layer, the ultimate passive or lateral resistance will be the compressive strength of the LWF at 5% deformation. A geotechnical resistance factor of 0.5 also applies to this resistance component. In Subsection 5.6 below, the LWF under the slab is recommended to consist of EPS Blocks Type 19, which has a compressive strength at 5% deformation of 90 kPa.

Should additional passive resistance be require, the horizontal component of the axial resistance of battered piles (up to 1H:3V inclination), or anchors can be used in the building foundation design.

Foundation Uplift Resistance

Uplift forces on the proposed foundations can be resisted using the dead weight of the concrete foundations, the weight of the materials overlying the foundations, and the submerged weight of the piles. Unit weights of materials are provided in Table 3.

For soil above the groundwater level, calculate using the “drained” unit weight and below groundwater level use the “effective” unit weight. 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 anchor footings is not provided.

As noted, the piles will generally be located below the groundwater level, so the submerged, or effective, weight of the pile will be available to contribute to the uplift resistance, if required. Considering that this is a reliable uplift resistance, and is really counteracting a dead load, in our opinion, a resistance factor of 0.9 is applicable for the ULS weight component.

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

Table 3 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m^3)		Internal 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
OPSS Granular A Fill (Crushed Stone)	22.0	13.7	38	0.60	0.22	0.36	8.8
OPSS Granular B Type I Fill (Well-Graded Sand-Gravel)	21.5	13.4	36	0.55	0.26	0.41	7.5
OPSS Granular B Type II Fill (Crushed Stone)	22.5	14.0	40	0.62	0.20	0.33	10.3
Granular Working Surface - Coarse Open-Graded Crushed Stone	19.0	12.0	36	0.55	0.26	0.41	7.5
In Situ Silty Sand or Site Excavated Silty Sand Fill	18.0	11.2	32	0.48	0.30	0.46	5.6

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.
☐ Passive pressure coefficients incorporate wall friction of 0.5 φ'.

Conventional Spread Footings (External Structures)

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, firm silty clay and/or compact silty sand bearing surface can be designed using a bearing resistance value at Serviceability Limit States (SLS) of **60 kPa** and a factored bearing resistance value at Ultimate Limit States (ULS) of **100 kPa**, incorporating a geotechnical resistance factor of 0.5.

For larger pad footings designed for uplift resistance, up to 7 m wide, and other footings placed on the undisturbed brown silty sand bearing surface, these footings can be designed using a bearing resistance value at Serviceability Limit States (SLS) of **80 kPa** and a factored bearing resistance value at Ultimate Limit States (ULS) of **140 kPa**, incorporating a geotechnical resistance factor of 0.5.

Footings placed on compacted engineered fill, placed on an undisturbed, brown silty sand bearing surface can be designed using a bearing resistance value at SLS of **80 kPa** and a factored bearing resistance at ULS of **140 kPa**, incorporating a geotechnical resistance factor of 0.5.

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.

Settlement

The total and differential settlements associated with the footing loading conditions using the bearing resistance value at SLS provided are estimated to be 25 and 20 mm, respectively.

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 the soil subgrade medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil.

5.4 Permissible Grade Raise Recommendations

Consideration must be given to potential settlements which could occur due to the presence of the silty clay deposit and the combined loads from the proposed footings, any groundwater lowering effects, and grade raise fill. The foundation loads to be considered for the settlement case are the continuously applied loads which consist of the unfactored dead loads and the portion of the unfactored live load that is considered to be continuously applied.

Consolidation Testing

Generally, the potential long term settlement is evaluated based on the compressibility characteristics of the silty clay. These characteristics are estimated in the laboratory by conducting unidimensional consolidation tests on undisturbed soil samples collected using Shelby tubes in conjunction with a piston sampler. A total of 3 site specific consolidation tests are being carried out for this project. The results of the consolidation tests are included in Appendix 1.

Value p'_c is the preconsolidation pressure of the sample and p'_o is the effective overburden pressure. The difference between these values is the available preconsolidation. The increase in stress on the soil due to the cumulative effects of the fill surcharge, the footing pressures, the slab loadings and the lowering of the groundwater should not exceed the available preconsolidation if unacceptable settlements are to be avoided.

The values C_{cr} and C_c are the recompression and compression indices, respectively, and are a measure of the compressibility of the soil due to stress increases below and above the preconsolidation pressures. The higher values for the C_c, as compared to the C_{cr}, illustrate the increased settlement potential above, as compared to below, the preconsolidation pressure.

It should be noted that the values of p'_c, p'_o, C_{cr} and C_c are determined using standard engineering practices and are estimates only. In addition, natural variations within the soil deposit would also affect the results. Furthermore, the p'_o parameter is directly influenced by the groundwater level. While the groundwater levels were measured at the time of the fieldwork, the levels vary with time and this has an impact on the available preconsolidation. Lowering the groundwater level increases the p'_o and therefore reduces the available preconsolidation. Unacceptable settlements could be induced by a significant lowering of the groundwater level. The p'_o values for the consolidation tests carried out for the present investigation are based on the long term groundwater level being 0.5 m above the bottom of the silty clay crust. The level of the groundwater level is based on the colour and undrained shear strength profile of the silty clay.

Table 3 - Summary of Consolidation Test Results

Borehole No.	Sample	Sample Depth (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c	Q (*)
BH 2	TW 6	5.05	64.7	27.8	0.047	1.293	A
BH 3	TW 5	3.38	49.5	16.3	0.048	1.986	A
BH 4	TW 5	5.69	82.8	31.8	0.058	3.722	G

* - Q - Quality assessment of sample - G: Good A: Acceptable P: Likely disturbed

Settlement

For design purposes, the total and differential settlements associated with the combination of grade raises and footing loading conditions using the bearing resistance values are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed.

Permissible Grade Raise

Preliminary permissible grade raise recommendations were determined for the proposed development based on the consolidation testing results of samples of the silty clay obtained during the geotechnical investigation. Based on our findings, a permissible grade raise of **1.0 to 1.2 m** is recommended for slab-on-grade at the site, using 400 mm of EPS geofoam blocks (Type 19 or higher), to provide compensation for the sustained slab-on-grade loading.

For parking and loading areas away from the building foundation, preliminary permissible grade raise recommendations will be slightly higher at **1.4 m**.

To reduce potential long term liabilities, consideration should be given to provide means to reduce long term groundwater lowering (e.g. clay dykes, restriction on planting around the structures, etc).

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 (OBC) 2012. The shear wave velocity test was completed by Paterson personnel. A seismic shear wave velocity profile from the on site testing is presented in Appendix 2.

Field Program

The seismic array testing location was placed directly to the east of the site in a north-south direction as presented on Drawing PG4592-1 - Test Hole Location Plan 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 3 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, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (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 seated parallel to the geophone array). The shot location was located at the centre of the seismic array.

The methods of testing completed by Paterson are guided by the standard testing procedures used by the expert seismologists at Carleton University and the 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 made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation.

Based on the test results, the average overburden seismic shear wave velocity is 168.7 m/s. Through interpretation, the bedrock has a shear wave velocity of 1,500 m/s. The Vs_{30} was calculated using the standard equation for average shear wave velocity from the Ontario Building Code (OBC) 2012. The overburden thickness below underside of footing is assumed to be 22.4 m, based on an underside of footing at 1.5 m below ground surface.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the OBC 2012, as presented below.

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

$$V_{s30} = \frac{30m}{\left(\frac{23.5m}{168.7\,m/s} + \frac{6.5m}{1{,}500\,m/s}\right)}$$

$$V_{s30} = 209\,m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for foundations placed on the overburden materials is 209 m/s. Therefore, a **Site Class D** is applicable for design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.6 Slab-on-Grade

Under Slab Materials

With the removal of all topsoil and fill (containing deleterious or organic materials), the native soil will be considered an acceptable subgrade surface on which to prepare the profile, provided below, for slab-on-grade construction. Any soft areas should be removed and backfilled with appropriate backfill material.

It is recommended that the sub-slab profile consist of the following:

☐ A thickness yet to be determined of EPS Blocks Type 19 (LWF) overlying the native soil subgrade.

☐ Geotextile or poly separation layer overlying the EPS Blocks.

☐ 400 to 450 mm of OPSS Granular A placed over the separation layer for the support of the slab-on-grade.

Modulus of Subgrade Reaction

Typical values of subgrade modulus for the OPSS Granular A, native silty sand and native silty clay surfaces are provided in Table 4.

Table 4 - Modulus of Subgrade Reaction

Soil Type	Modulus of Soil Reaction (MPa/m)
OPSS Granular A Subgrade	20
Silty Sand	15
Silty Clay Deposit	5

5.7 Pavement Structure

Minimum Pavement Structure Recommendations

For the pavement structures, it's expected that concrete aprons will be used along the loading docks along with dolly pads. The loading docks will most likely be 1.2 m lower than the floor slab. The remainder of the pavement structures will be flexible asphaltic concrete and will be able to comply with the permissible grade raise restrictions provided. The proposed pavement structures are presented in Tables 5 and 6.

Table 5 - 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 6 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

Crushed Concrete for Subbase Layer

It is understood that crushed concrete is introduced as a replacement for the recommended OPSS Granular B Type II subbase layer. From a geotechnical perspective, the use of crushed concrete is acceptable. However, the pavement structure design must be altered to optimise the performance of the new subbase, as follows:

☐ The subbase layer must be increased by 100 mm and should consist of free draining and compacted to a minimum of 98% of the material's SPMDD.

☐ The crushed concrete used should be a minimum of 50 mm in size, well graded and free of any deleterious materials such as, rebar debris and metal.

It should be noted that although the usage of crushed concrete is acceptable for the subbase layer, the crushed concrete will not mirror all the characteristics of the recommended OPSS Granular B Type II. Due to the nature of the concrete, the concrete partially loses its strength when exposed to excessive water over the long run. Also, due to the shallow groundwater across the subject site, it is recommended that the subgrade have a positive drainage to direct any water beneath the pavement to subdrains that will direct the water to positive outlets. The pavement structure drainage recommendations are discussed in the following section.

Periodic granular size distribution testing must be completed during the construction of the parking area by Paterson personnel to confirm the sufficiency of the crushed concrete used within the subbase layer.

Pavement Structure Drainage

The pavement structure performance is dependent on the moisture condition at 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 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. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed building. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

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

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover should be provided for adequate frost protection for heated structures.

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 heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations at the site should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the excavation to be undertaken by open-cut methods.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level.

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

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

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material’s standard Proctor maximum dry density.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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.

A temporary Ministry of the Environment and Climate Change (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 of 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

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 the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters, 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.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

7.0 Recommendations

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

☐ Review of the grading plan from a geotechnical perspective.

☐ Review of LWF recommendations and design.

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

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

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

☐ Observation of all subgrades prior to backfilling.

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

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

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided 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 is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test hole locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 Day & Ross c/o Avenue 31 Capital Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Faisal I. Abou-Seido, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Sept. 10, 2018
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

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

Report Distribution

- ☐ Day & Ross c/o Avenue 31 Capital Inc. (3 copies)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

CONSOLIDATION TEST RESULTS

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

FILE NO. PG4592

REMARKS

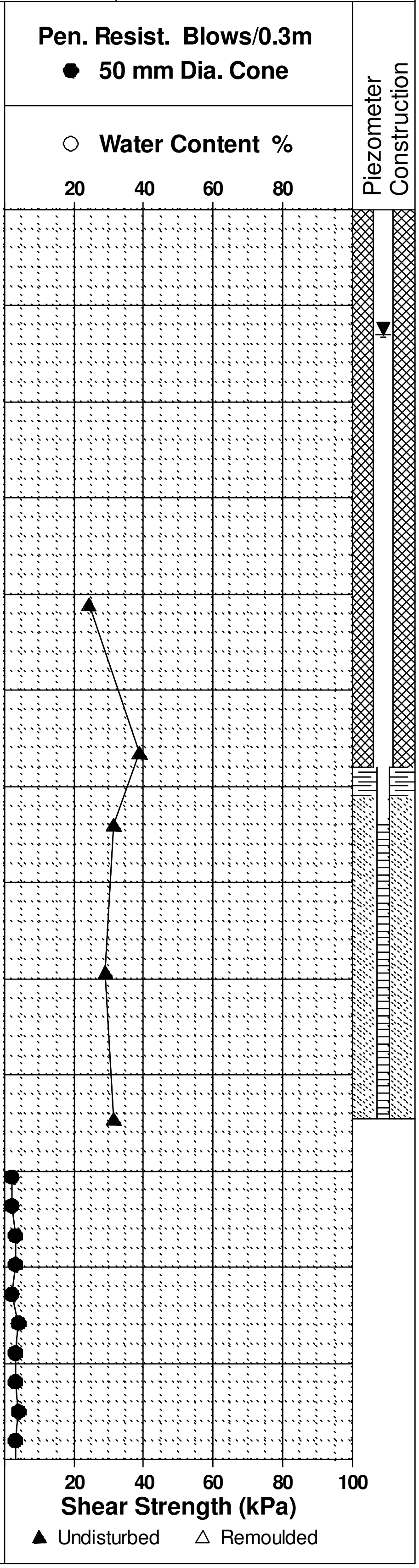
HOLE NO. BH 1

BORINGS BY CME 55 Power Auger **DATE** 1 August 2018

SOIL DESCRIPTION	STRATA PLOT
GROUND SURFACE	
FILL: Brown silty clay, trace sand, gravel, cobbles	2.29
Loose, brown **SILTY SAND**	3.05
Soft to firm, grey **SILTY CLAY**	9.45
Dynamic Cone Penetration Test commenced at 9.45m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	21
SS	3	50	10
SS	4	8	9
SS	5	88	W
SS	6	100	6

DEPTH (m)	ELEV. (m)
0	78.29
1	77.29
2	76.29
3	75.29
4	74.29
5	73.29
6	72.29
7	71.29
8	70.29
9	69.29
10	68.29
11	67.29
12	66.29
13	65.29



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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REMARKS

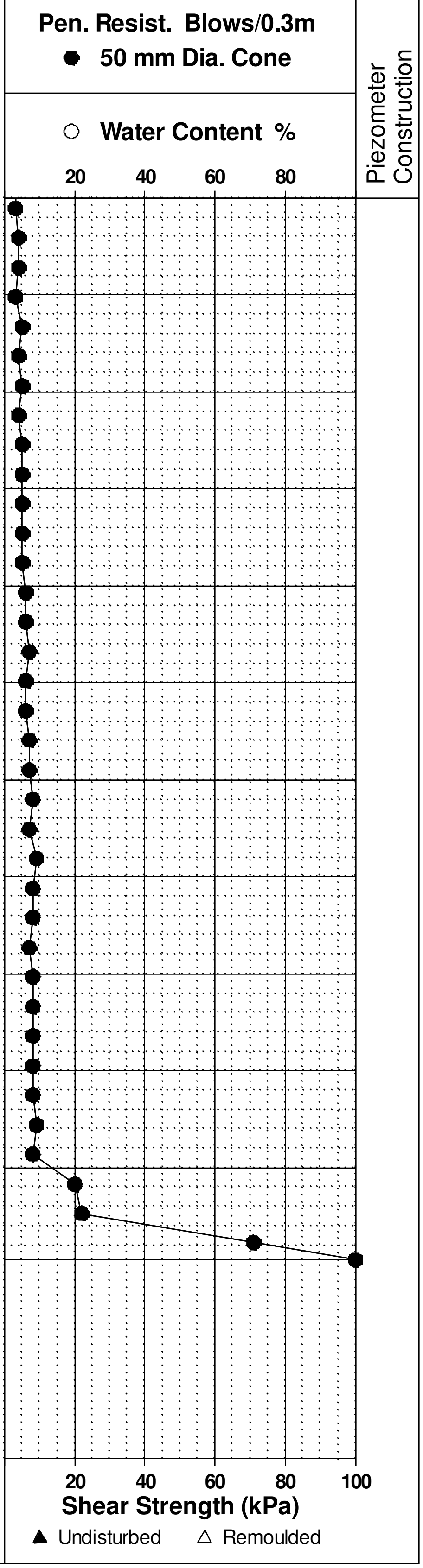
BORINGS BY CME 55 Power Auger

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						13	65.29
						14	64.29
						15	63.29
						16	62.29
						17	61.29
						18	60.29
						19	59.29
						20	58.29
						21	57.29
						22	56.29
						23	55.29
23.95							
End of Borehole Practical DCPT refusal at 23.95m depth (GWL @ 1.30m - Aug. 8, 2018)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

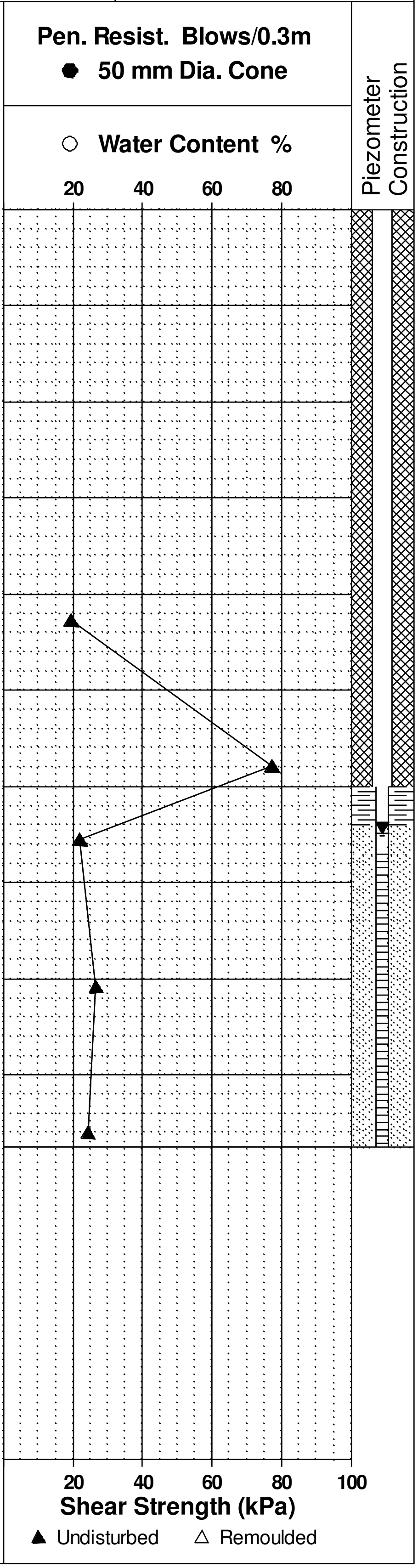
DATE 3 August 2018

FILE NO. PG4592

HOLE NO. BH 2

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty clay with sand, gravel, cobbles		AU	1		
		SS	2	50	17
		SS	3	0	7
	2.59	SS	4	25	3
Very loose, brown **SILTY SAND**	3.50	SS	5	100	3
Soft to firm, grey **SILTY CLAY**		TW	6	100	
	9.75				
End of Borehole					
(GWL @ 6.49m - Aug. 8, 2018)					

DEPTH (m)	ELEV. (m)
0	78.67
1	77.67
2	76.67
3	75.67
4	74.67
5	73.67
6	72.67
7	71.67
8	70.67
9	69.67



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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FILE NO. **PG4592**

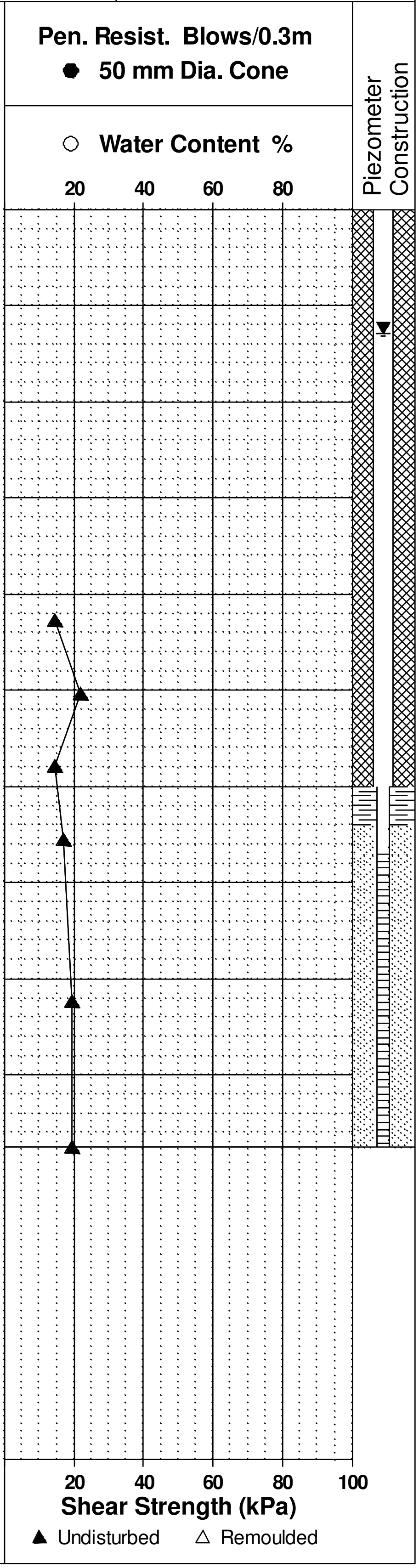
REMARKS

HOLE NO. **BH 3**

BORINGS BY CME 55 Power Auger

DATE 3 August 2018

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	77.67
TOPSOIL 0.05	AU	1				
FILL: Brown silty clay with sand	SS	2	50	9	1	76.67
1.83						
Compact, brown **SAND**	SS	3	50	17	2	75.67
2.44	SS	4	100	W		
					3	74.67
	TW	5	100			
					4	73.67
					5	72.67
Soft, grey **SILTY CLAY**					6	71.67
	TW	6	100		7	70.67
					8	69.67
					9	68.67
9.75						
End of Borehole						
(GWL @ 1.29m - Aug. 8, 2018)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

FILE NO. **PG4592**

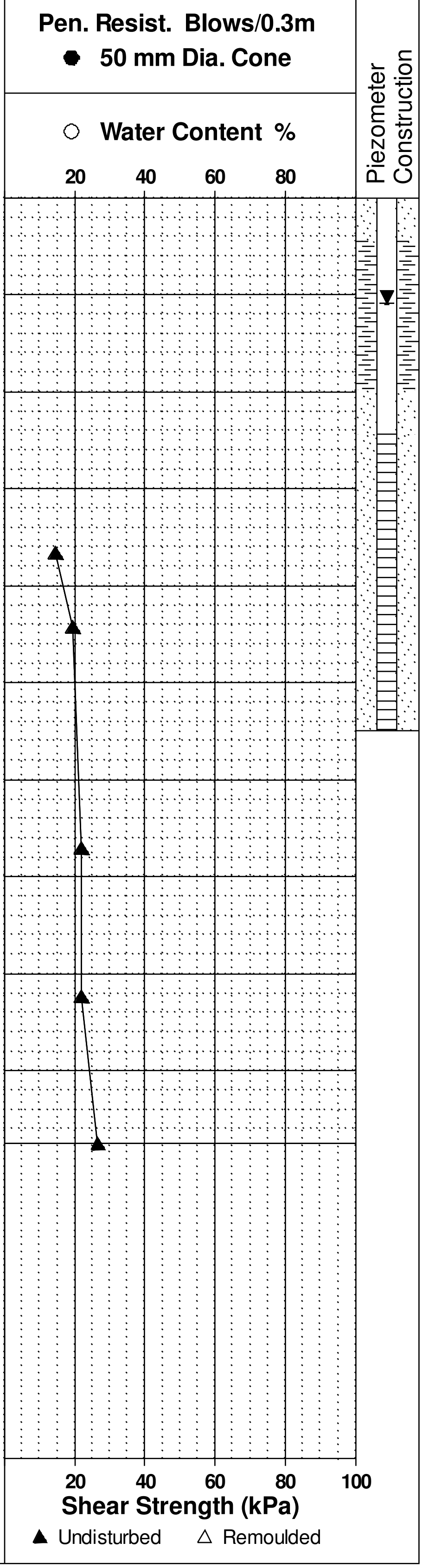
REMARKS

HOLE NO. **BH 4**

BORINGS BY CME 55 Power Auger

DATE 3 August 2018

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	77.98
FILL: Gravelly sand						
0.60						
FILL: Grey silty clay with sand - some gravel by 1.5m depth	SS	1	50	5	1	76.98
	SS	2	58	10	2	75.98
	SS	3	42	2		
3.05					3	74.98
Soft to firm, brown to grey **SILTY CLAY**					4	73.98
	SS	4	100	2	5	72.98
	TW	5	100		6	71.98
					7	70.98
					8	69.98
					9	68.98
9.75						
End of Borehole (GWL @ 1.08m - Aug. 8, 2018)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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FILE NO. **PG4592**

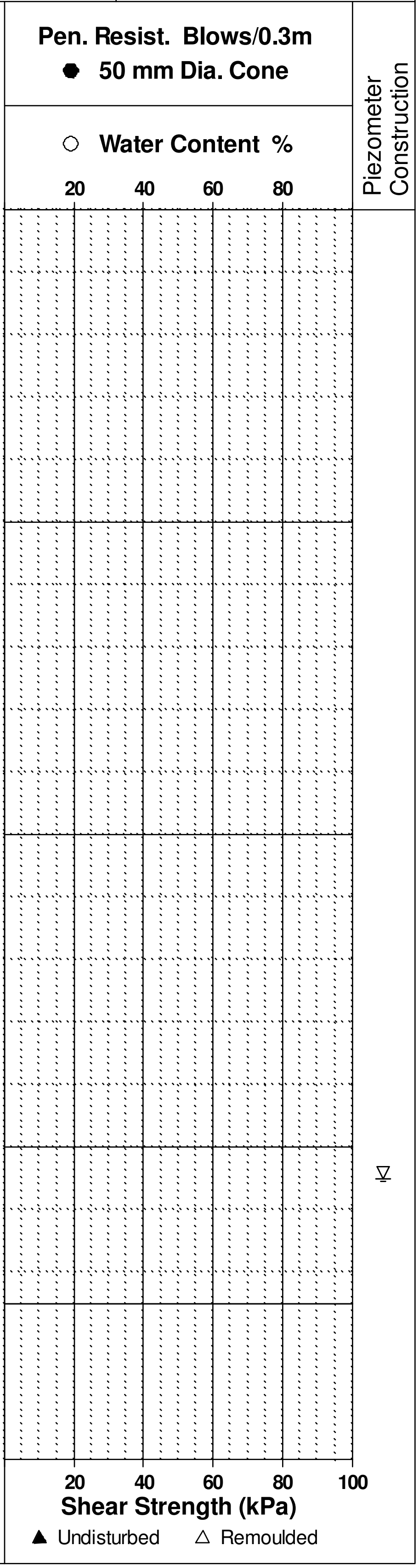
REMARKS

HOLE NO. **TP 1**

BORINGS BY Backhoe

DATE 1 August 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.87
						1	76.87
FILL: Silty sand topsoil with organics, trace gravel, cobbles and boulder		G	1				
						2	75.87
		G	2				
						3	74.87
3.10							
Brown **SILTY SAND**, trace organics		G	3				
3.50							
End of Test Pit (Open hole GWL @ 3.1 m depth)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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REMARKS

BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 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 %	Piezometer Construction
GROUND SURFACE						0	77.56		
TOPSOIL 0.08									
		G	1						
									▽
FILL: Brown silty sand, some gravel, cobbles, boulders, asphalt, concrete, foam, construction debris, metal, plaster, pipe, brick and clay 3.20						1	76.56		
		G	2			2	75.56		
						3	74.56		
Brown **SILTY SAND**, trace organics 3.50		G	3						
End of Test Pit (Open hole GWL @ 0.6 m depth)									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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FILE NO. PG4592

REMARKS

HOLE NO. TP 3

BORINGS BY Backhoe

DATE 1 August 2018

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	77.51		
						1	76.51		
FILL: Brown silty sand, trace brown silty clay, some sand, gravel and cobbles, trace asphalt, brick, organics and concrete.		G	1						
		G	2						
						2	75.51		
2.30									
Brown **SILTY SAND**, trace organics		G	3						
2.90									
Red **SILTY CLAY** -grey by 3.3 m depth		G	4			3	74.51		
		G	5						▽
3.50									
End of Test Pit (Open hole GWL @ 3.3 m depth)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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5510 Boundary Road
Ottawa, Ontario

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FILE NO. PG4592

REMARKS

HOLE NO. TP 4

BORINGS BY Backhoe **DATE** 1 August 2018

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	77.66		
FILL: Brown silty sand, some asphalt, gravel, cobbles, boulders, construction debris and brick		G	1						
									⊻
						1	76.66		
		G	2						
						2	75.66		
		G	3						
						3	74.66		
3.10									
Brown **SILTY SAND**		G	4						
3.30									
Red **SILTY CLAY**		G	5						
3.50									
End of Test Pit (Open hole GWL @ 0.75 m 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

Geotechnical Investigation
5510 Boundary Road
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FILE NO. PG4592

REMARKS

BORINGS BY Backhoe

DATE 1 August 2018

HOLE NO. TP 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.93
FILL: Brown silty sand to brown silty clay, some sand, gravel, cobbles and boulders, trace organics, brick, glass, tile, and construction debris	▨	G	1				
						1	76.93
		G	2				
2.20						2	75.93
Brown **SILTY SAND**		G	3				
3.00						3	74.93
Red **SILTY CLAY** -grey by 3.3 m depth		G	4				
3.50		G	5				
End of Test Pit (Open hole GWL @ 2.2 m depth)							

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

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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REMARKS

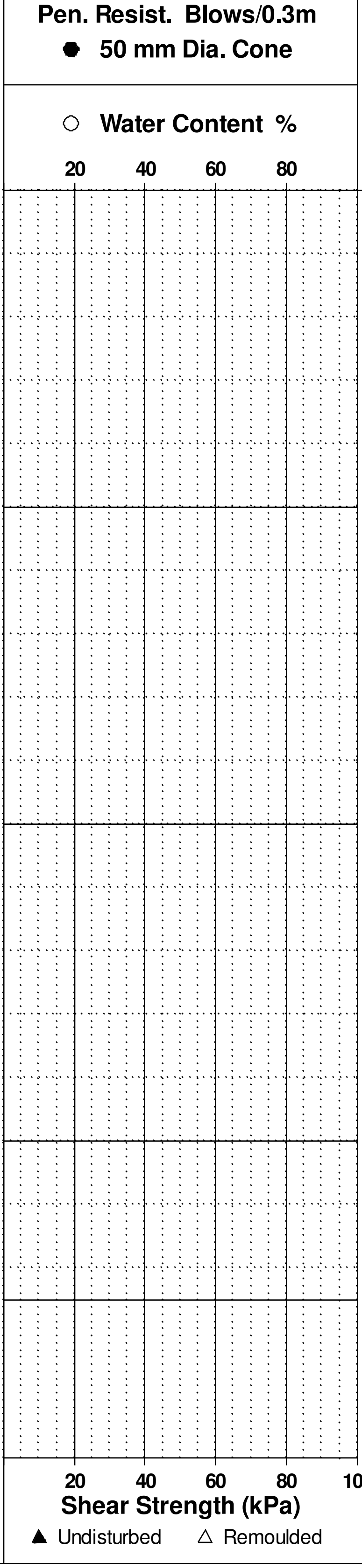
BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
GROUND SURFACE						0	78.45	
FILL: Brown silty clay, some sand, gravel, cobbles, trace organics and construction debris	▨	G	1			1	77.45	
						2	76.45	
2.30 Brown **SILTY SAND**		G	2					
2.50 Red **SILTY CLAY**	▨	G	3			3	75.45	⊻
3.50 End of Test Pit (Open hole GWL @ 3.0 m depth)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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REMARKS

BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	78.00	
FILL: Brown silty clay, some sand, trace gravel, cobbles, boulders, organics, asphalt, concrete, brick and construction debris	▨	G	1					
	▨	G	2			1	77.00	
2.00 Brown **SILTY SAND**		G	3			2	76.00	∇
2.60 Red **SILTY CLAY**	▨	G	4			3	75.00	
3.50 End of Test Pit (Open hole GWL @ 2.1 m depth)								

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
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FILE NO. PG4592

REMARKS

HOLE NO. TP 8

BORINGS BY Backhoe

DATE 1 August 2018

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	78.00		
FILL: Brown silty clay, some sand, gravel, cobbles, trace boulders, organics, asphalt, brick and construction debris	▨	G	1						
						1	77.00		
		G	2						
						2	76.00		
2.05									
Red **SILTY CLAY**	▧	G	3						
2.55									
End of Test Pit (TP dry upon completion)									

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

Geotechnical Investigation
5510 Boundary Road
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REMARKS

BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	78.17
	▨	G	1				
	▨					1	77.17
FILL: Brown silty sand, some clay, gravel, cobbles, boulders, and construction debris	▨	G	2				
	▨					2	76.17
	▨	G	3				
	▨					3	75.17
3.50	▨	G	4				
End of Test Pit (Open hole GWL @ 0.65 m depth)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

▽

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
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

FILE NO. PG4592

REMARKS

HOLE NO. TP10

BORINGS BY Backhoe

DATE 1 August 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	78.27
FILL: Brown silty sand, some gravel and cobbles, trace boulders, brick and organics	▨	G	1				
	▨					1	77.27
	▨	G	2				
2.10	▨					2	76.27
Brown **SILTY SAND** 2.30		G	3				
Red **SILTY CLAY** 2.60	▨	G	4				
End of Test Pit (TP dry upon completion)							

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

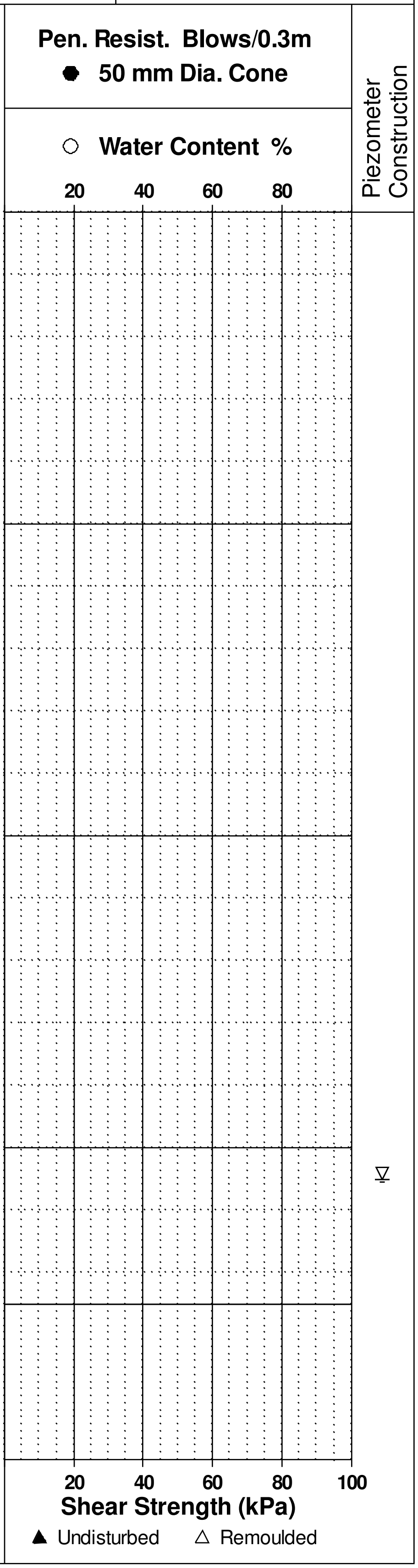
BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP11

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	78.07
FILL: Brown silty sand, some topsoil, trace organics, gravel, cobbles, boulders and clay		G	1				
						1	77.07
		G	2				
2.05						2	76.07
Brown **SILTY SAND**		G	3				
2.25							
Red **SILTY CLAY**		G	4				
-grey by 3.1 m depth						3	75.07
		G	5				
3.50							
End of Test Pit							
(Open hole GWL @ 3.1 m depth)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

FILE NO. PG4592

REMARKS

HOLE NO. TP12

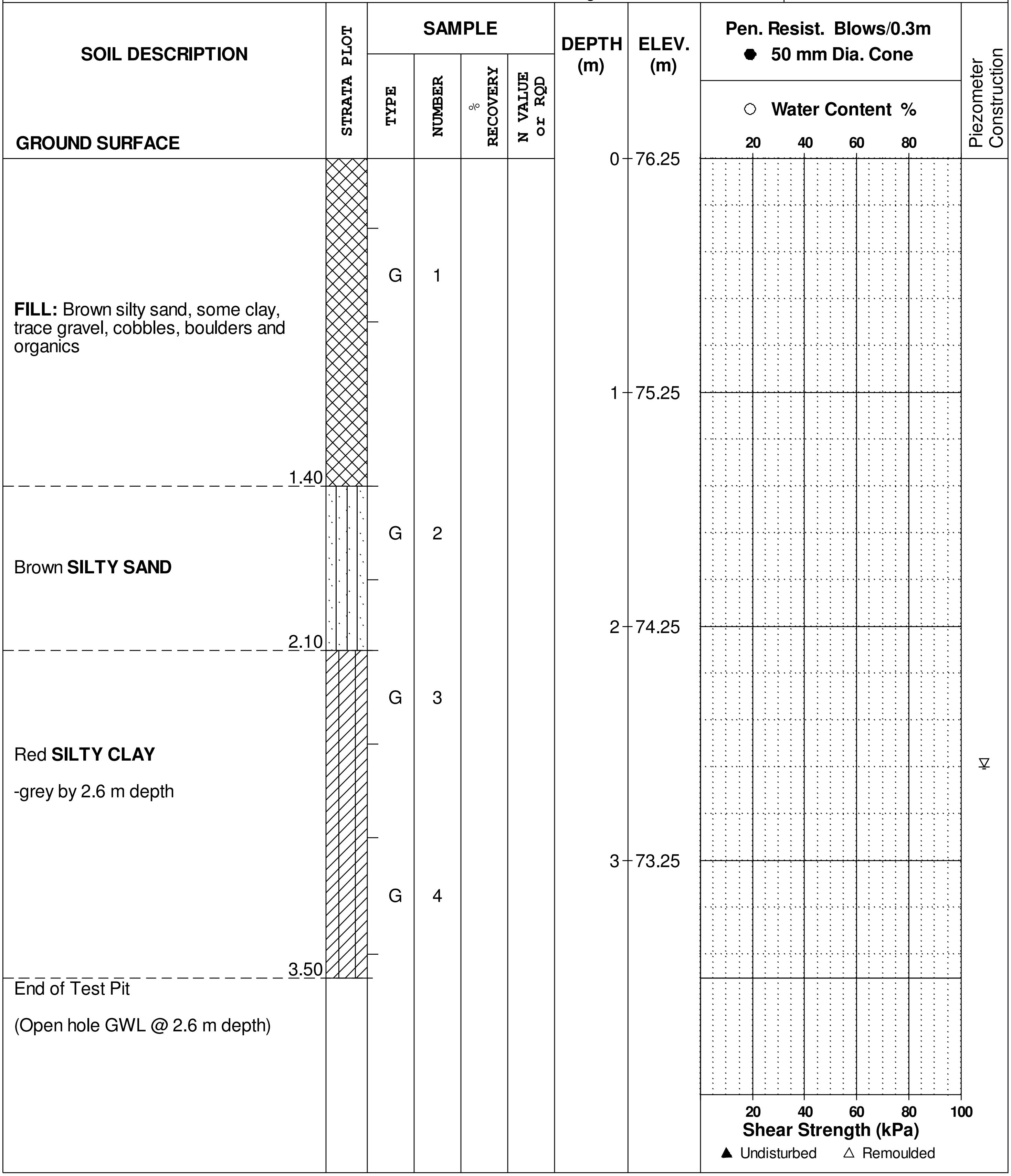
BORINGS BY Backhoe

DATE 1 August 2018

SOIL DESCRIPTION	Depth of base (m)
GROUND SURFACE	
FILL: Brown silty sand, some clay, trace gravel, cobbles, boulders and organics	1.40
Brown **SILTY SAND**	2.10
Red **SILTY CLAY** -grey by 2.6 m depth	3.50
End of Test Pit (Open hole GWL @ 2.6 m depth)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
G	1		
G	2		
G	3		
G	4		

DEPTH (m)	ELEV. (m)
0	76.25
1	75.25
2	74.25
3	73.25



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

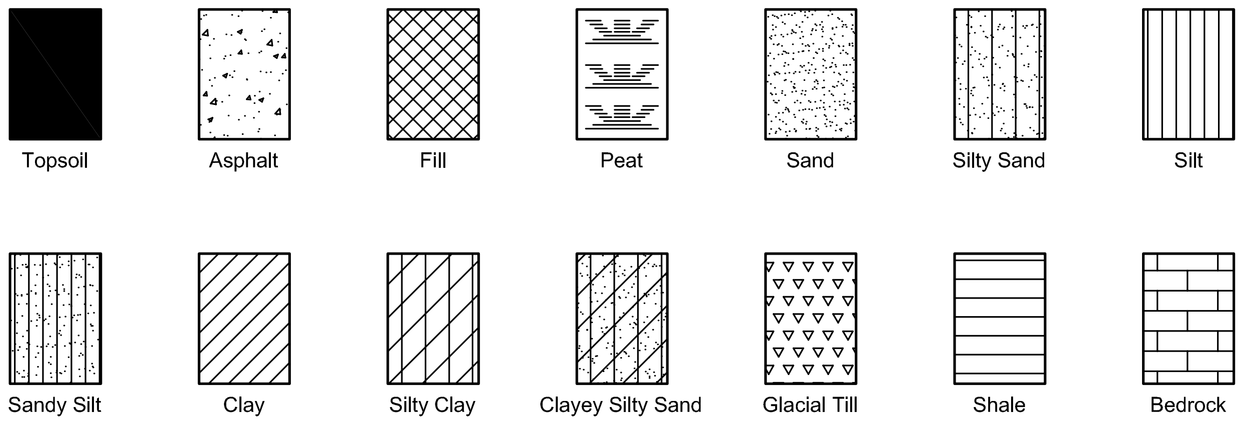
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

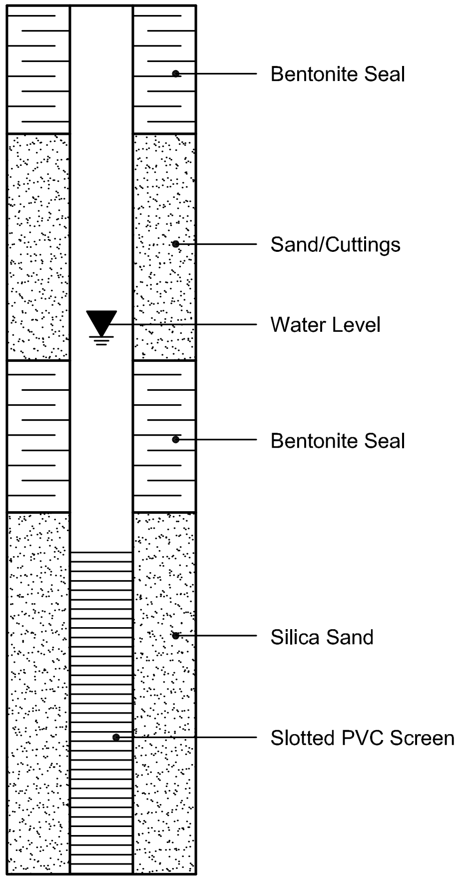
SYMBOLS AND TERMS (continued)

STRATA PLOT

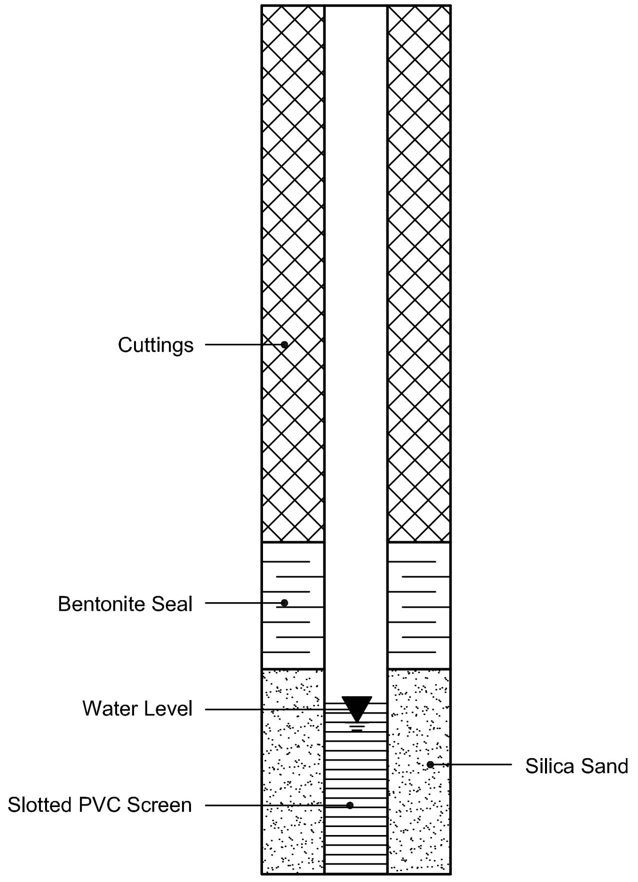


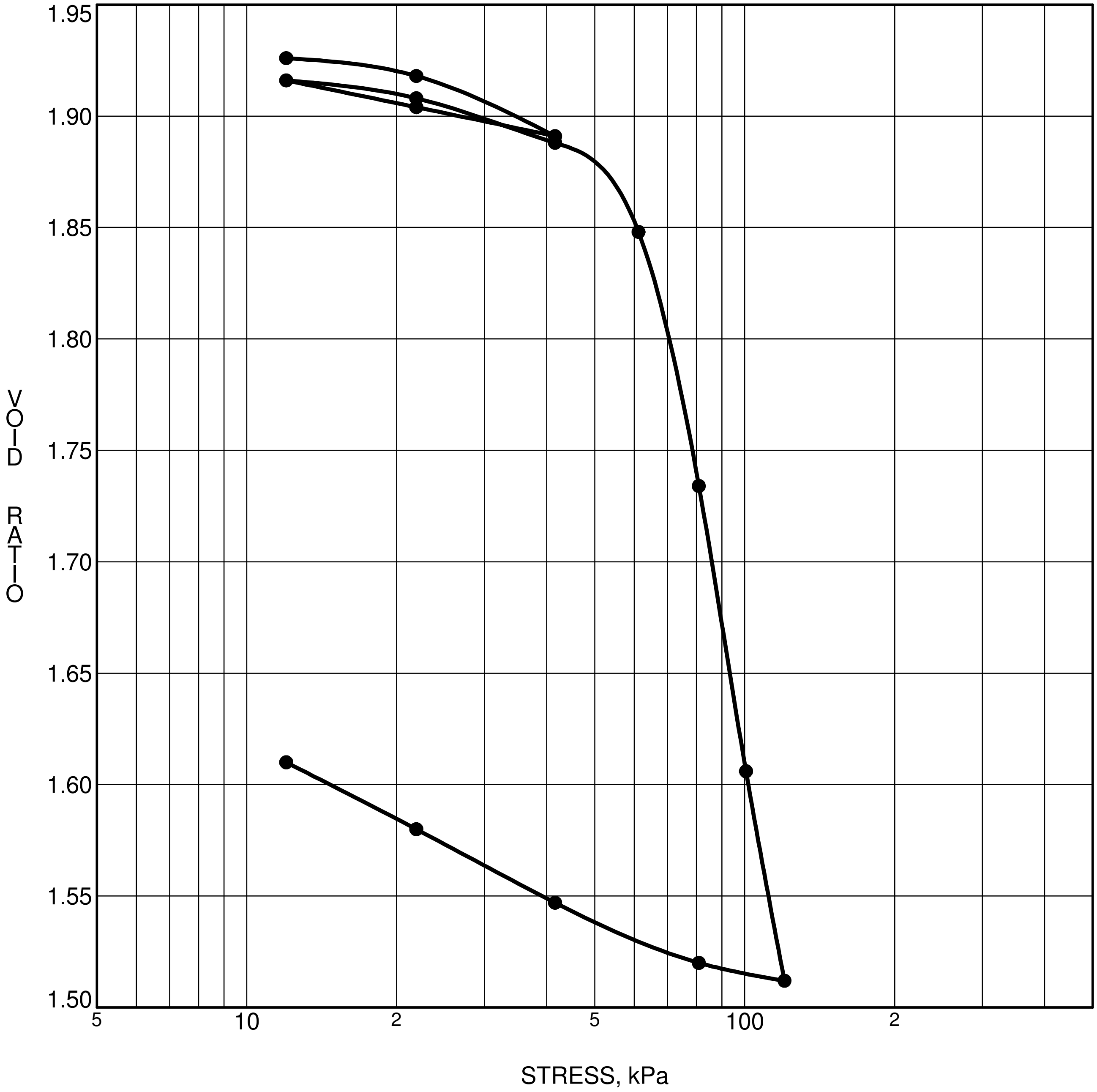
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 2**	p'_o	**27.8** kPa	Ccr	**0.047**
Sample No.	**TW 6**	p'_c	**64.66** kPa	Cc	**1.293**
Sample Depth	**5.05** m	OC Ratio	**2.3**	Wo	**70.6** %
Sample Elev.	**73.62** m	Void Ratio	**1.941**	Unit Wt.	**15.6** kN/m^3

CLIENT **Day and Ross**
PROJECT **Geotechnical Investigation - 5510 Boundary Road**
FILE NO. **PG4592**
DATE **30/08/2018**

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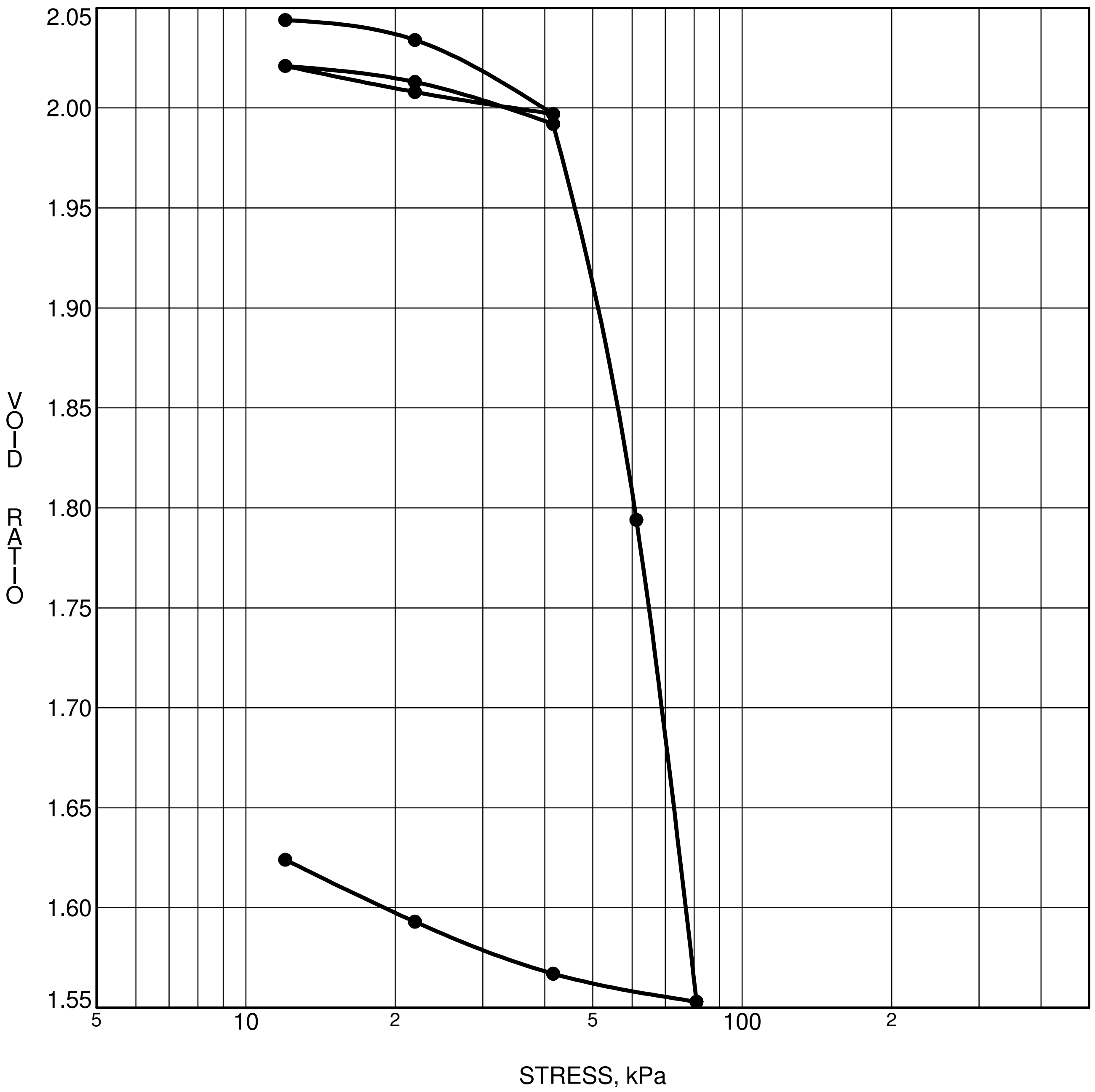
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 3**	p'o	**16.32** kPa	Ccr	**0.048**
Sample No.	**TW 5**	p'c	**50** kPa	Cc	**1.986**
Sample Depth	**3.38** m	OC Ratio	**3.1**	Wo	**75.0** %
Sample Elev.	**74.29** m	Void Ratio	**2.062**	Unit Wt.	**15.4** kN/m ^{3}

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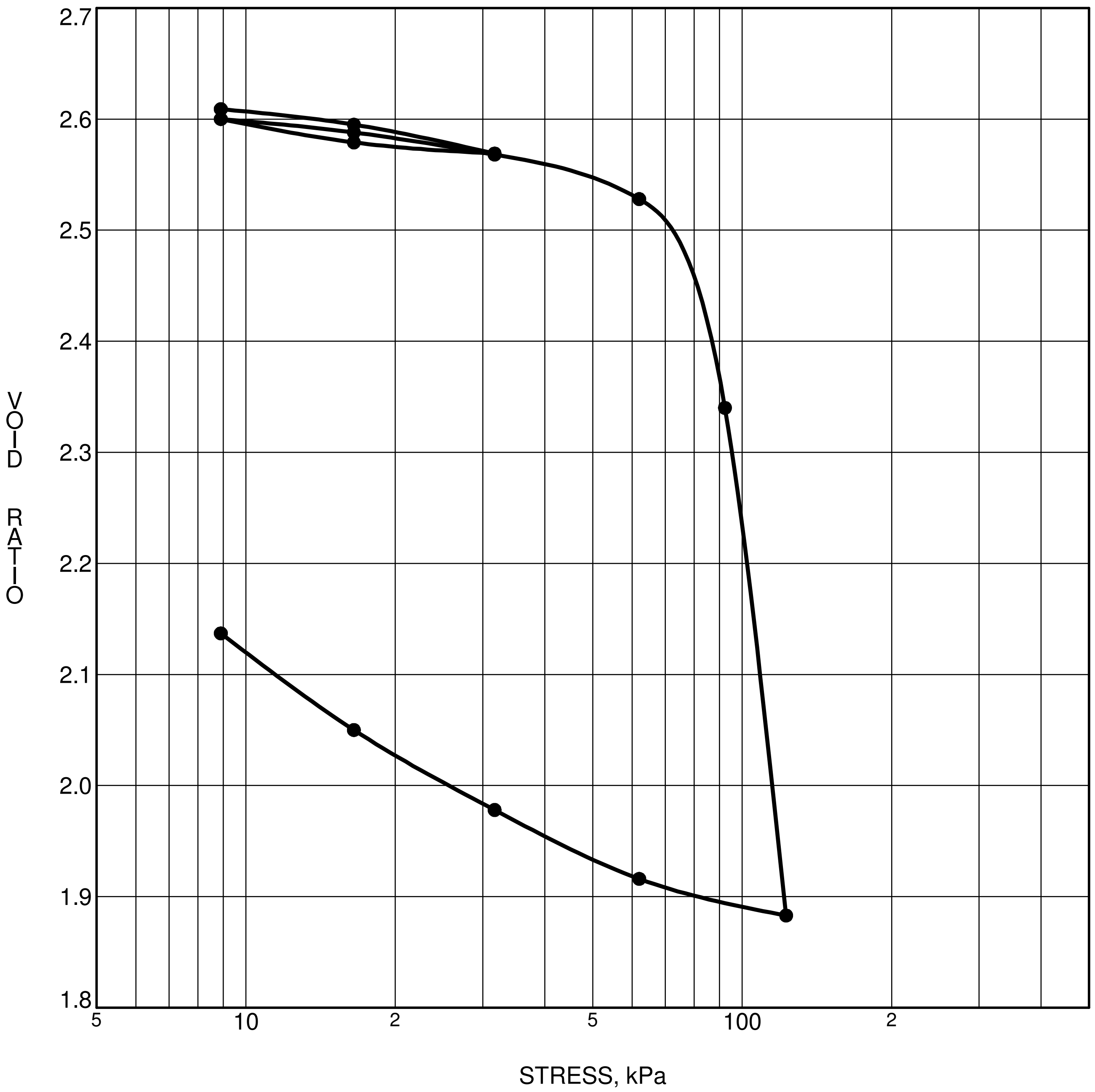
CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY								
Borehole No.	**BH 4**		p'o	**31.82**	kPa	Ccr	**0.058**	
Sample No.	**TW 5**		p'c	**82.79**	kPa	Cc	**3.722**	
Sample Depth	**5.69**	m	OC Ratio	**2.6**		Wo	**94.9**	%
Sample Elev.	**72.29**	m	Void Ratio	**2.61**		Unit Wt.	**14.6**	kN/m^3

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CONSOLIDATION TEST

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - SHEAR WAVE VELOCITY PROFILE AT SHOT LOCATION 34.5 m

DRAWING PG4592-1 - TEST HOLE LOCATION PLAN

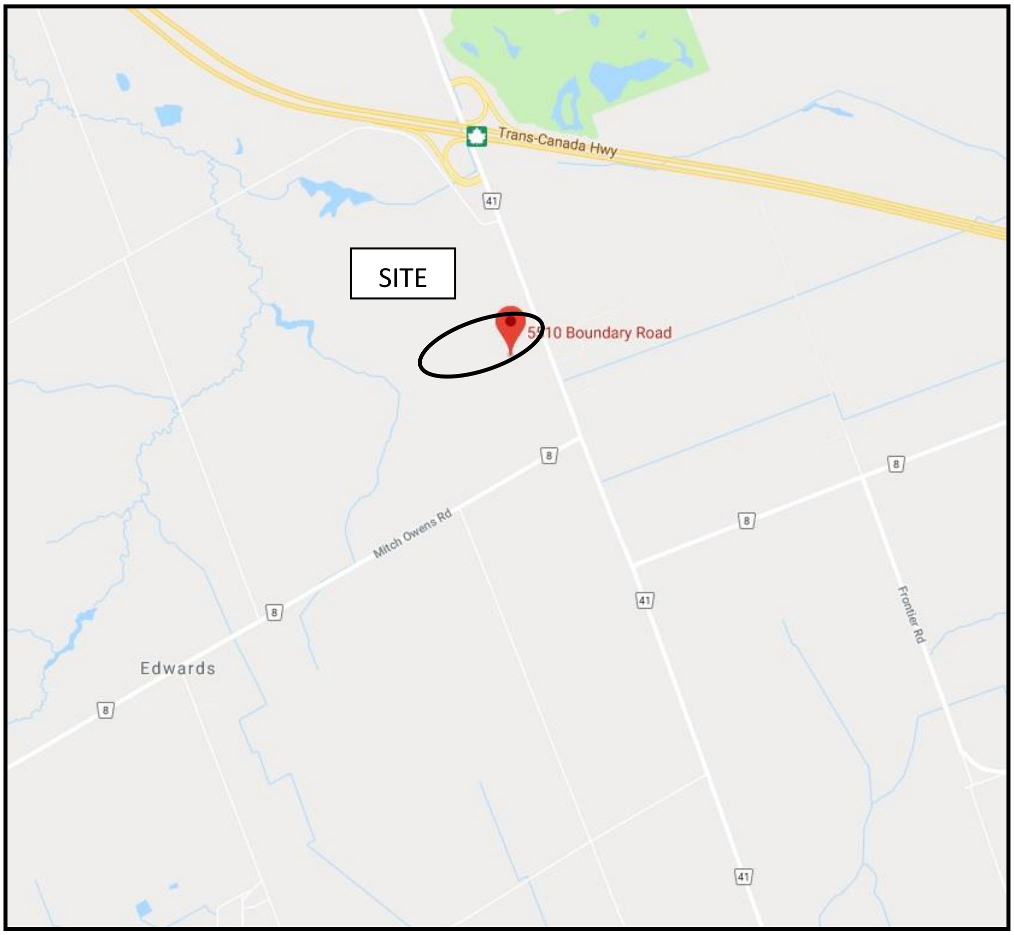

FIGURE 1

KEY PLAN

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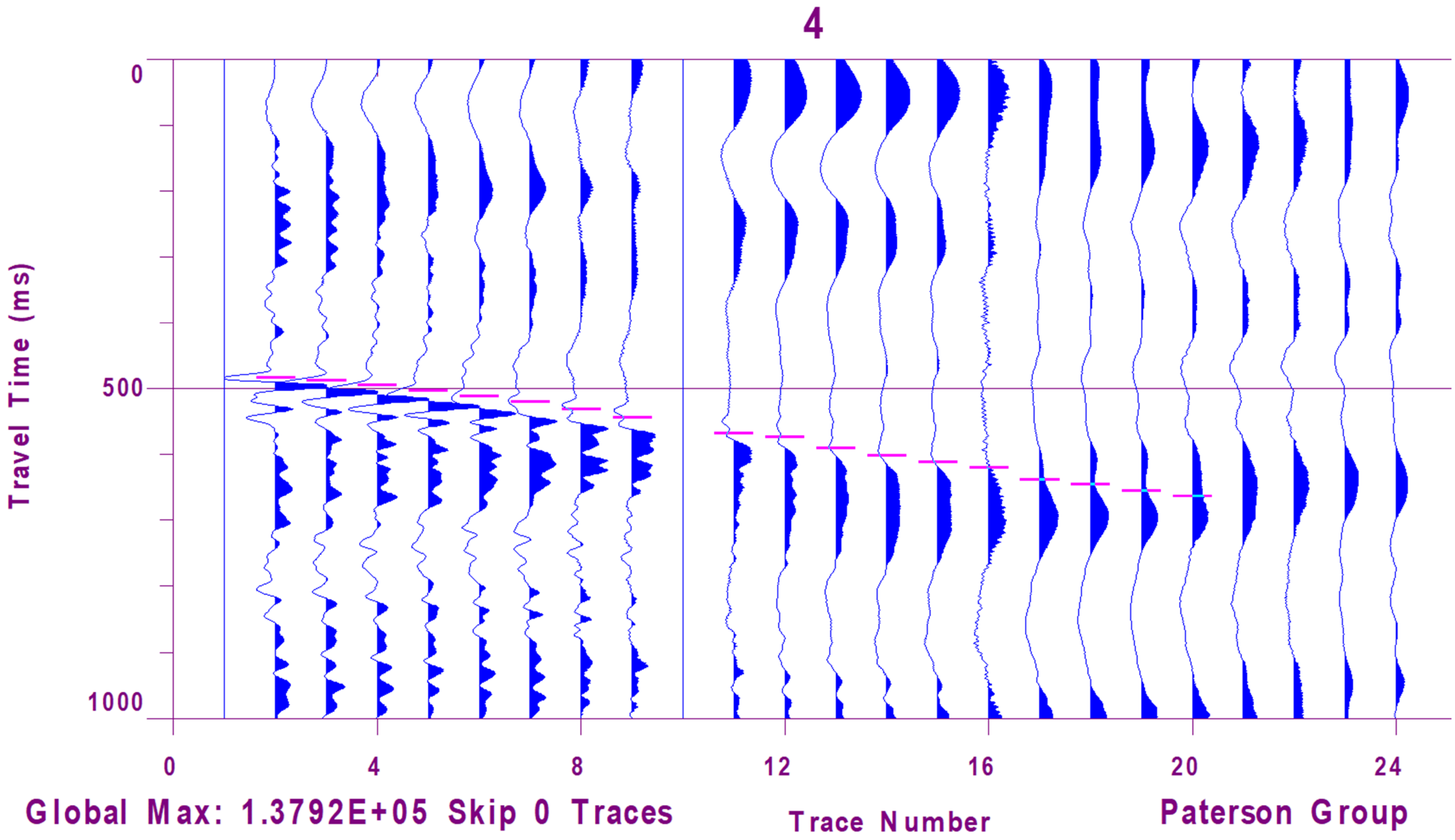


Figure 2 – Shear Wave Velocity Profile at Shot Location -30 m

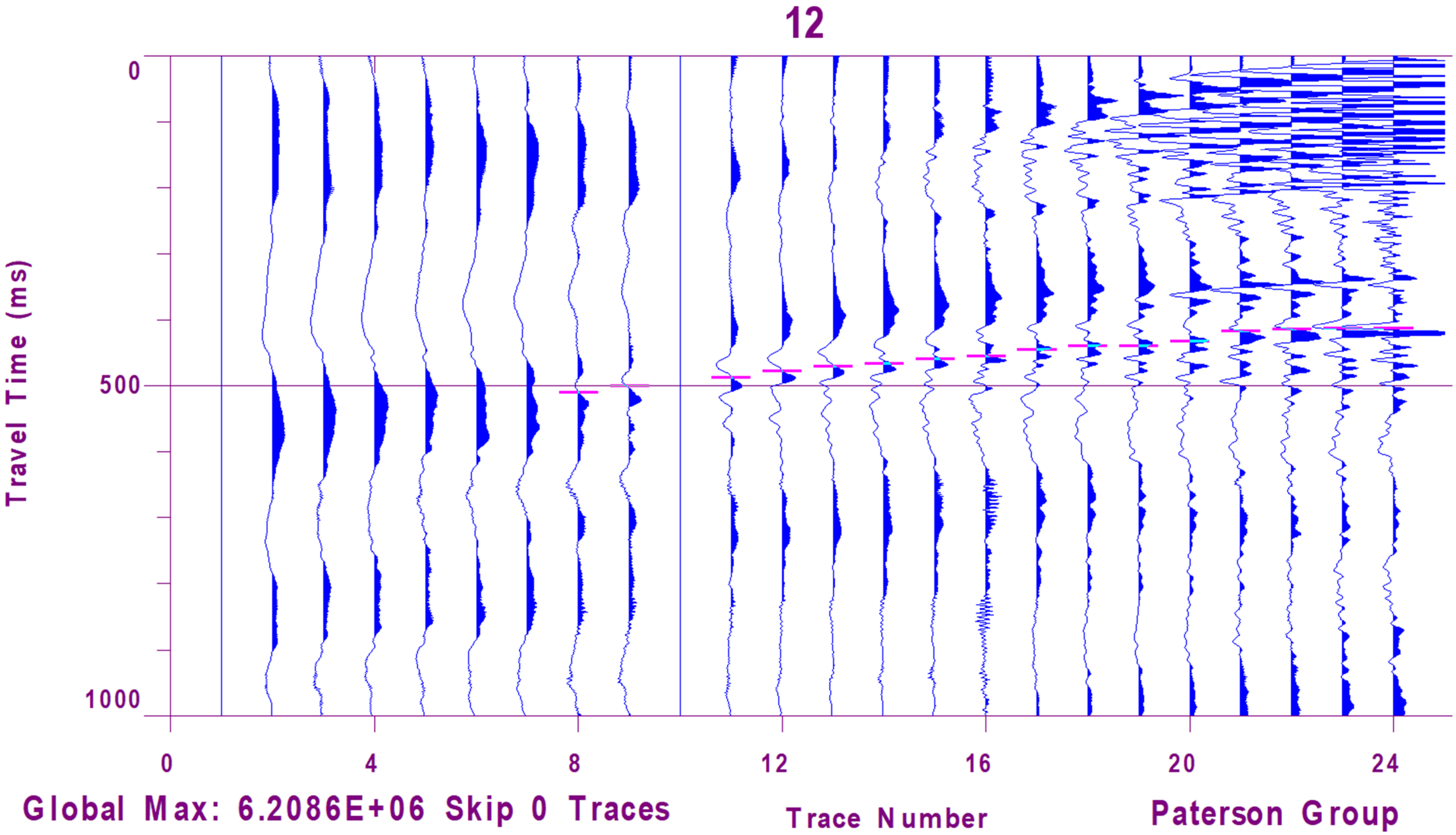
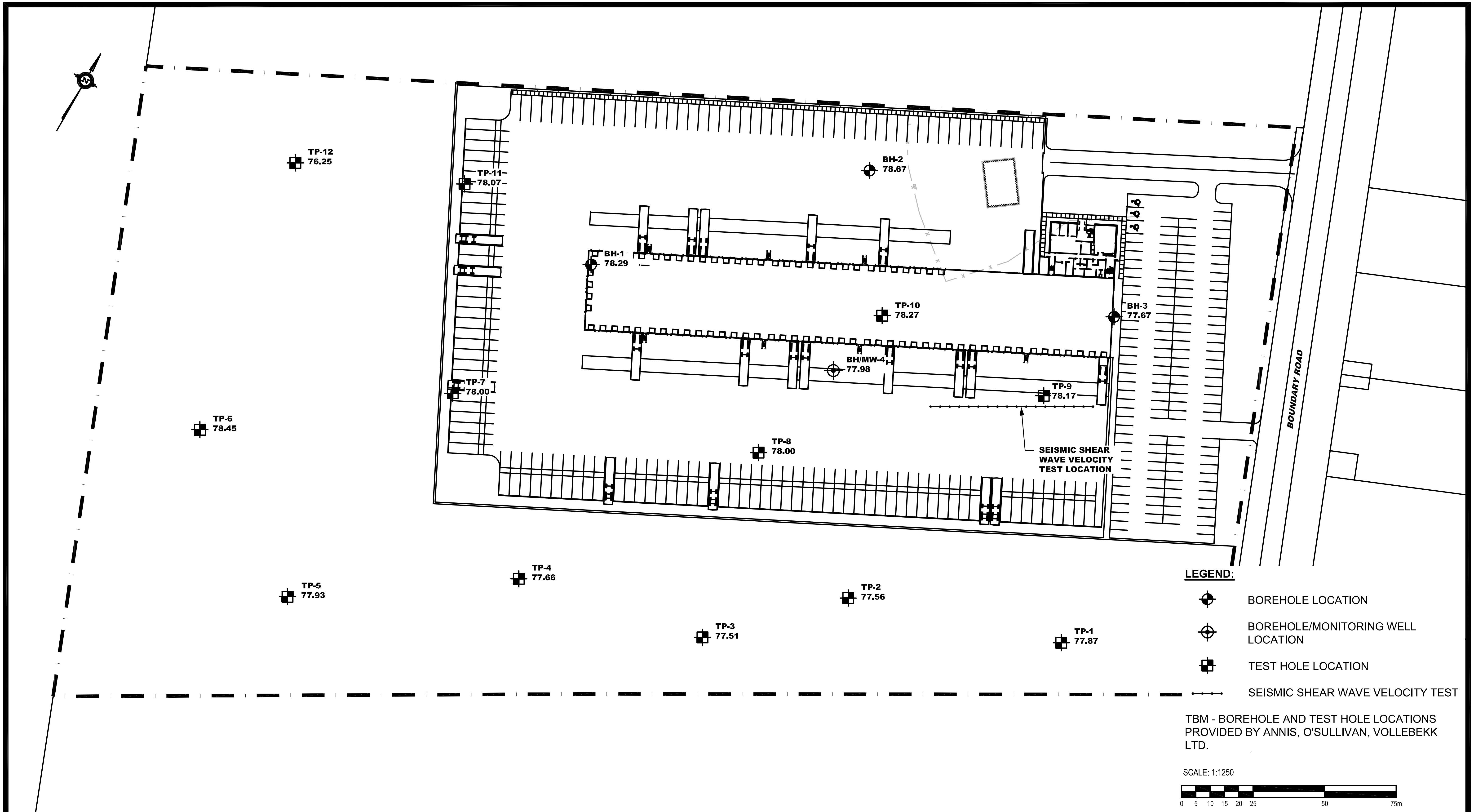


Figure 3 – Shear Wave Velocity Profile at Shot Location 72 m



LEGEND:

⌖	BOREHOLE LOCATION
⊕	BOREHOLE/MONITORING WELL LOCATION
▣	TEST HOLE LOCATION
-·-·-·-	SEISMIC SHEAR WAVE VELOCITY TEST

TBM - BOREHOLE AND TEST HOLE LOCATIONS PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

SCALE: 1:1250

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

ROUTES CAR RENTALS

GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE COMPLEX - 5510 BOUNDARY ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1250
Date: 08/2018
Drawn by: GR
Report No.: PG4592-1
Checked by: FAS
Dwg. No.: PG4592-1
Approved by: DG
Revision No.: 0

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