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Geotechnical Engineering

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Geotechnical Investigation

Proposed Commercial Development - Phase 2
Cope Drive
Ottawa, Ontario

Prepared For

SmartCentres

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Report: PG2950-1R

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TABLE OF CONTENTS

PAGE

APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by SmartCentres to conduct a geotechnical investigation for Phase 2 of the proposed commercial development to be located along the north and south side of Cope Drive at the intersection of Terry Fox Drive in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. Therefore, the present report does not address environmental issues.

2.0 PROPOSED DEVELOPMENT

It is our understanding that the current phase of the proposed development, consists of four (4) commercial buildings of slab-on-grade construction. The remainder of the subject site will consist of car parking and access lanes with some landscaping areas. It is further understood that the subject site will be serviced by municipal water and sewer.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The current field program for the geotechnical investigation was carried out on May 2 and 3, 2013. At that time, a total of four (4) boreholes were placed across the subject site to provide general coverage of the proposed development. Relevant boreholes and laboratory testing completed during previous geotechnical investigations for Phase 1 of the commercial development are also included in the present report. The locations of the test holes are shown on Drawing PG2950-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler or from the auger flights. All soil samples were visually inspected and initially classified on site. The split-spoon samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

Overburden thickness was also evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at BH 1 (PG2263) completed during our previous geotechnical investigation on the adjacent site. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its 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

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

Sample Storage

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

3.2 Field Survey

The test hole locations completed during the present geotechnical investigation were selected in the field by Paterson personnel taking into consideration site features and underground utilities. The ground surface elevations at the test hole locations were surveyed by Paterson personnel and referenced to a temporary benchmark (TBM), consisting of the finished floor slab of the existing Walmart located to the south of the subject site. A geodetic elevation of 98.25 m was provided for the TBM based on plans prepared by Stantec. The location of the TBM, test holes and the ground surface elevation at each test hole location are presented on Drawing PG2950-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

Five (5) Shelby tube samples were submitted for unidimensional consolidation testing, three (3) soil sample for Atterberg limits testing and one (1) soil sample was submitted for grain size analysis during the previous geotechnical investigations.

The results of the consolidation testing are presented in the Consolidation Test Results sheets presented in Appendix 1 and are further discussed in Sections 4 and 5.

3.4 Analytical Testing

One (1) 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 samples were submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 OBSERVATIONS

4.1 Surface Conditions

Generally, crushed stone fill and/or topsoil are noted at ground surface across the subject site. The ground surface is relatively flat and slightly lower than Terry Fox Drive and Cope Drive. The site is bordered to the west by a drainage ditch.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the borehole locations consists of crushed stone fill, silty sand/silty clay fill and/or topsoil underlain by a silty sand layer and/or brown silty clay crust. A grey silty clay layer was noted below the silty clay crust at each borehole location. A DCPT completed at BH 1 (PG2263) for a previous investigation encountered practical refusal at a 35.4 m depth. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for details of the soil profiles encountered at the test hole locations.

Silty Clay

Generally, the upper portion of the silty clay layer has been weathered to a stiff to very stiff brown crust. Sand seams and silt pockets were also observed within the upper portion of the silty clay layer. The brown silty clay crust extends to depths varying between 1.8 and 2.7 m.

Grey silty clay was encountered below the weathered crust at all borehole locations. In situ shear vane tests conducted within the grey silty clay layer yielded undrained shear strength values ranging from 24 to 48 kPa. Generally, these values are indicative of a firm consistency.

Five (5) soil samples from relevant boreholes were subjected to unidimensional consolidation (odometer) testing during the previous geotechnical investigations. The test results are presented in Subsection 5.4 and on the Consolidation Test Results sheets in Appendix 1. The consolidation test results indicate that the silty clay is overconsolidated with overconsolidation ratios (OCR) for the tested samples varying between 1.5 and 2.4. The OCR is the ratio of the preconsolidation pressure to the effective pressure at the sample depth. This is further discussed in Subsection 5.3.

Three (3) silty clay samples collected from relevant boreholes completed for previous investigations were submitted for Atterberg Limits testing. The tested samples from the boreholes were classified as Inorganic Clays of Low Plasticity (CL). The results are summarized in Table 1 and presented on the Atterberg Limits Results sheets in Appendix 1.

Table 1 - Summary of Atterberg Limits Tests

Sample PG1568	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Classification
BH 1 TW 3 (PG1568)	35.2	25	17	8	CL
BH 1 TW3 (PG2263)	38.2	30	17	13	CL
BH 5 TW 4 (PG2263)	47.0	43	20	23	CL

Note:
☐ CL - Inorganic Clays of Low Plasticity

Available Geological Mapping

Based on available geological mapping, the bedrock in this area mostly consists of interbedded limestone and dolomite of the Gull River formation with an overburden drift thickness of 30 to 40 m depth.

4.3 Groundwater

The measured groundwater levels in the piezometers at the borehole locations are presented in Table 2. It should be noted that surface water can become trapped with a backfilled borehole, which can lead to higher than normal groundwater level readings. Long-term groundwater levels can also be estimated based on the observed colour, moisture levels and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater can be expected between 1.5 to 2 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 2 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1-13	97.27	1.39	95.88	May 16, 2013
BH 2-13	97.67	1.68	95.99	May 16, 2013
BH 3-13	97.27	0.98	96.29	May 16, 2013
BH 4-13	98.17	1.56	96.61	May 16, 2013

Note:

❏ The ground surface elevations at the test hole locations were surveyed by Paterson personnel and referenced to a temporary benchmark (TBM), consisting of the finished floor slab of the existing Walmart located to the south of the subject site. A geodetic elevation of 98.25 m was provided for the TBM based on plans prepared by Stantec.

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed commercial buildings. It is expected that the proposed buildings can be supported by conventional shallow footings. Alternatively, the proposed buildings can be founded by concrete in-filled trenches extended to an undisturbed, stiff silty clay bearing surface. Due to the presence of the sensitive silty clay layer, the proposed development will be subjected to grade raise restrictions. Permissible grade raise recommendations are discussed in Subsection 5.3.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be considered to reduce the risks of unacceptable long-term post construction total and differential settlements.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A 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 areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use 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

Due to the relatively thin silty clay crust observed at the borehole locations, it is recommended to keep the footings as high as possible within the soil profile. To ensure footings are founded over an undisturbed, stiff silty clay bearing surface, it is recommended that footings be founded at a geodetic elevation of 96.0 m or higher. Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, founded over an undisturbed, stiff silty clay bearing surface can be designed using the bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**.

A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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.

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 a firm to stiff silty clay above the groundwater table 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 of the same or higher capacity as the bearing medium soil.

Settlement/Grade Raise

Consideration must also 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. For buildings, a minimum value of 50% of the live load is often recommended by Paterson.

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. Five (5) unidimensional consolidation tests, which are relevant to our current investigation, were carried out on selected shelby tube samples as part of our previous geotechnical investigation. The results of the consolidation tests are presented in Table 3 and 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.

Table 3 - Summary of Consolidation Test Results

Borehole No.	Sample	Depth (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c	Q (*)
BH 1 (PG2263)	TW 3	3.3	84	44	0.010	0.543	A
BH 5 (PG2263)	TW 4	9.5	128	84	0.019	0.766	G
BH 1 (PG1568)	TW 3	5.1	98	54	0.001	0.462	A
BH 4 (PG1568)	TW 3	4.3	102	50	0.020	1.227	A
BH 5 (PG1568)	TW 2	3.3	97	40	0.016	1.387	A

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

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 are based on the long term groundwater level being at 0.5 m below the existing groundwater level.

The total and differential settlements will be dependent of the characteristics of the buildings. 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.

The potential post construction total and differential settlements are dependent on the position of the long term groundwater level when building over deposits of compressible silty clay. While efforts can be made to reduce the impacts of the development on the long term level of the groundwater by placing clay dykes in the service trenches, reducing the sizes of paved areas, leaving green spaces to allow for groundwater recharge, limiting planting of trees to areas away from the buildings, it is not economically possible to control the level of the groundwater.

To reduce potential long term liabilities, consideration should be given to accounting for a larger groundwater lowering and to provide means to reduce long term groundwater lowering (e.g. clay dykes, restriction on planting around the dwellings, etc). It should be noted that building on silty clay deposits increases the likelihood of building movements and therefore of cracking. The use of steel reinforcement in foundations placed at key structural locations will tend to reduce foundation cracking as compared to unreinforced foundations.

Permissible grade raise recommendations based on available consolidation testing and in-situ undrained shear strength testing results are defined in Drawing PG2950-2 - Permissible Grade Raise Plan in Appendix 2.

5.4 Design for Earthquakes

Shear wave velocity testing was completed in close proximity to the subject site as part of the previous geotechnical investigation to accurately determine the applicable seismic site classification for the proposed buildings from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity testing at two (2) shot locations are presented in Appendix 2.

Field Program

A shear wave test was completed immediately to the south of the subject site, as presented in Drawing PG2950-1 - Test Hole Location Plan presented 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 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 locations are located at the centre of the geophone array and 3, 4.5 and 30 m away from the first and last geophone.

The methods of testing completed by Paterson are guided by the standard testing procedures used 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 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. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

The overburden and bedrock velocities were calculated to be 150 and 2,049 m/s, respectively. However, the bedrock was noted to greater than 30 m below ground surface. The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012.

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

$$V_{s30} = -\frac{30m}{\sum\left(\frac{30m}{150m/s}\right)}$$

$$V_{s30} = 150m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, is 150 m/s for the subject site. Therefore, a **Site Class E** is applicable for foundation design within the subject site, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Slab on Grade Construction

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed buildings, the undisturbed native soil surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. It is recommended that the upper 300 mm of sub-slab fill consist of OPSS Granular A crushed stone.

5.6 Service Pits - Jiffy Lube Building

Service Pit Perimeter Drainage

It is understood that below grade service pits will be constructed within the proposed Jiffy Lube building. It is recommended that a perimeter drainage system be placed at footing level of the proposed service pit and connected to a positive outlet, such as a gravity connection to the storm sewer or sump pit. Backfill against the exterior sides of the service pit walls should consist of free-draining, non frost susceptible granular materials. Alternatively, the service pit walls and base could be waterproofed using a suitable waterproofing product to prevent water infiltration into the service pit. It is recommended that the geotechnical consultant provide inspection services during the waterproofing installation to ensure the system is adequately installed.

Lateral Earth Pressures

The foundation walls can be designed using the information provided below. There are several combinations of backfill and retained soils that could be applicable for the service pit walls. However, the conditions could be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective unit weight of the retained soil should be 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

Two distinct conditions, static and seismic, should be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Static Conditions

The static horizontal earth pressure (p_o) could be calculated with 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 with 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 Conditions

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}) could be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45\text{-}a_{max}/g)a_{max}$
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 0.36g according to OBC 2012. The vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions could be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions presented above.

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

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

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

5.7 Pavement Structure

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

Table 4 - Recommended Pavement Structure - Car 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 5 - Recommended Pavement Structure - 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
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil	

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 material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

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

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for proposed structures. The system should consist of a 100 to 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system is provided.

It should be noted that if a trench style foundation is used for the proposed buildings, a two ply polyethylene liner should be placed against the outside trench face to limit frost adhesion potential between the foundation concrete and silty clay sidewalls.

It should also be noted that if a perimeter drainage system is not provided and frost susceptible silty clay is left at subgrade level below building's perimeter sidewalks, a potential for frost heave below the sidewalk is present. It is recommended to place a minimum 50 mm thick horizontal layer of SM rigid insulation at a minimum 600 mm depth below finished grade, extending from building face to at least 1 m beyond face of curb to limit frost heave action below the building's sidewalks.

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum 1.5 m thick soil cover (or equivalent) should be provided in this regard. A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials 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 assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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. Flatter slopes could be required for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be used. The subsoil at this site is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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 kept away from the excavation sides.

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

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

Excavation Base Stability

The base of supported excavations can fail by three (3) general modes:

- ❑ Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,
- ❑ Piping from water seepage through granular soils, and
- ❑ Heave of layered soils due to water pressures confined by intervening low permeability soils.

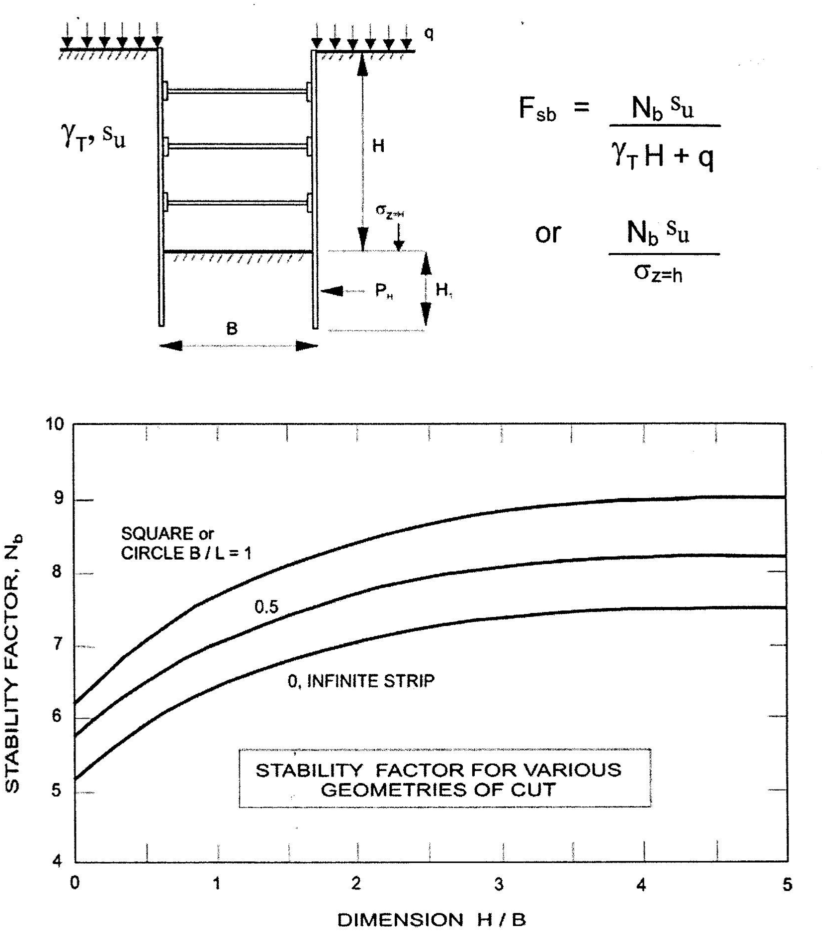
Shear failure of excavation bases is typically rare in granular soils if adequate lateral support is provided. Inadequate dewatering can cause instability in excavations made through granular or layered soils. The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems.

The factor of safety with respect to base heave, FS_b, is:

$$FS_b = N_b s_u/\sigma_z$$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.
s_u - undrained shear strength of the soil below the base level
σ_z - total overburden and surcharge pressures at the bottom of the excavation



$$F_{sb} = \frac{N_b s_u}{\gamma_T H + q}$$

or

$$\frac{N_b s_u}{\sigma_{z=h}}$$

Figure 1 - Stability Factor for Various Geometries of Cut

In the case of soft to firm clays, a factor of safety of 2 is recommended for base stability.

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.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the firm grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extent at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material’s SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. A perched groundwater condition may be encountered within the sandy silt deposit which may produce significant temporary groundwater infiltration levels. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

A temporary MOE permit to take water (PTTW) will be required for this project if more than 50,000 L/day are to be pumped during the construction phase. At least 3 to 4 months should be allowed for completion of the application and issuance of the permit by the MOE. In our opinion, since the site excavations will be relatively shallow and in impervious materials, no significant water infiltration is expected. Therefore, a PTTW would only be as a precautionary measure.

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

The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur. Precautions should be taken if winter construction is considered for this project.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, 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. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. These results are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the chloride content, pH and resistivity indicate the presence of a non-aggressive to slightly aggressive environment for exposed ferrous metals at this site.

7.0 RECOMMENDATIONS

It is recommended that the following be carried out during the site development:

☐ Review detailed grading plan(s) from a geotechnical perspective.

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

☐ 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 placing backfilling materials.

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

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

☐ Suggest foundation alternatives based on the potential long term settlements.

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 material testing and observation program by the geotechnical consultant.

8.0 STATEMENT OF LIMITATIONS

The recommendations made in this report are in accordance with our present understanding of the project. We request permission to review the grading plan once available. Also, our recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

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

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

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Carlos P. Da Silva, P.Eng.

Report Distribution:

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☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

CONSOLIDATION TEST RESULTS

ATTERBERG LIMITS TESTING RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Kanata South Phase 2
Ottawa, Ontario

DATUM TBM - Finished floor level of Walmart. Geodetic elevation = 98.25m, as per plan prepared by Stantec Geomatics.

REMARKS 18T 0431401; 5013975

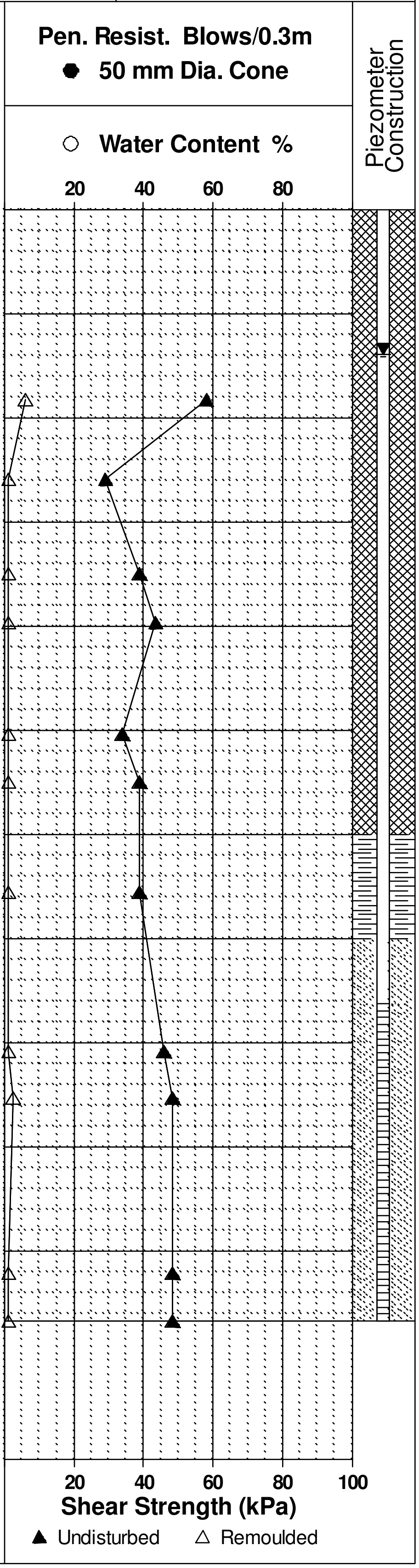
BORINGS BY CME 55 Power Auger

DATE May 2, 2013

FILE NO. PG2950

HOLE NO. BH 1-13

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.27
FILL: Brown to grey silty clay with sand and gravel 0.56		AU	1				
Stiff, brown **SILTY CLAY** to **CLAYEY SILT** with silty sand seams		SS	2	42	3	1	96.27
1.96		SS	3	50	2		
						2	95.27
						3	94.27
						4	93.27
						5	92.27
Firm, grey **SILTY CLAY**						6	91.27
		SS	4	100	1	7	90.27
						8	89.27
		SS	5	100	1	9	88.27
10.67						10	87.27
End of Borehole (GWL @ 1.39m-May 16, 2013)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Kanata South Phase 2
Ottawa, Ontario

DATUM TBM - Finished floor level of Walmart. Geodetic elevation = 98.25m, as per plan prepared by Stantec Geomatics.

REMARKS 18T 0431431; 5013938

BORINGS BY CME 55 Power Auger

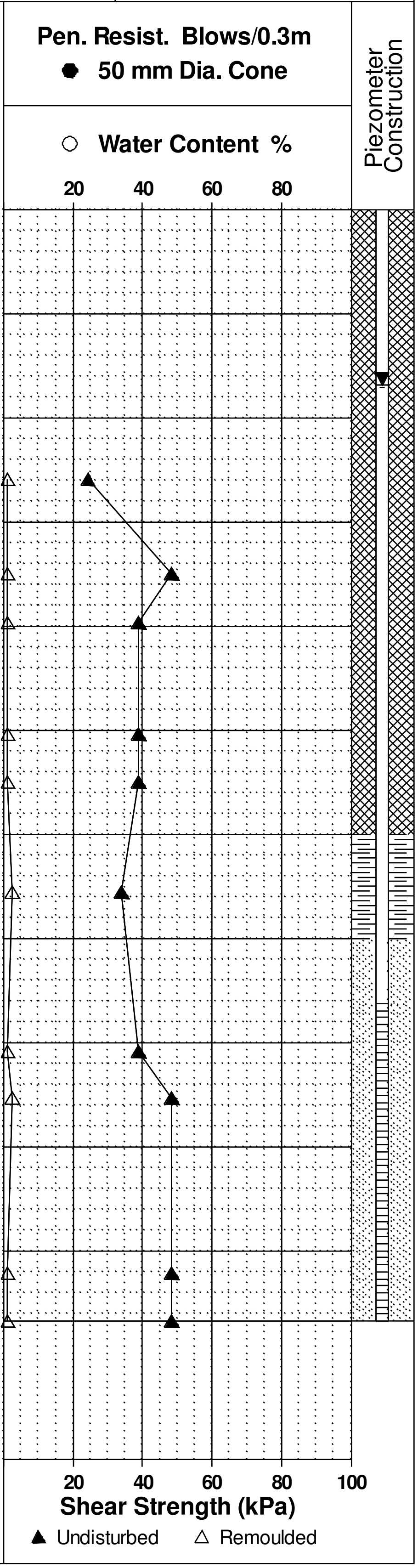
DATE May 2, 2013

FILE NO. PG2950

HOLE NO. BH 2-13

SOIL DESCRIPTION	SAMPLE			
	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Crushed stone with silty sand, trace clay 0.41	AU	1		
Compact, brown **SILTY SAND,** trace clay 1.22	SS	2	42	18
Stiff, brown **SILTY CLAY** to **CLAYEY SILT** with silty sand seams 2.21	SS	3	46	6
Firm, grey **SILTY CLAY** 10.67	SS SS SS	4 5 6	88 100 100	1 1 1
End of Borehole (GWL @ 1.68m-May 16, 2013)				

DEPTH (m)	ELEV. (m)
0	97.67
1	96.67
2	95.67
3	94.67
4	93.67
5	92.67
6	91.67
7	90.67
8	89.67
9	88.67
10	87.67



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Kanata South Phase 2
Ottawa, Ontario

DATUM TBM - Finished floor level of Walmart. Geodetic elevation = 98.25m, as per plan prepared by Stantec Geomatics.

REMARKS 18T 0431482; 5013993

BORINGS BY CME 55 Power Auger

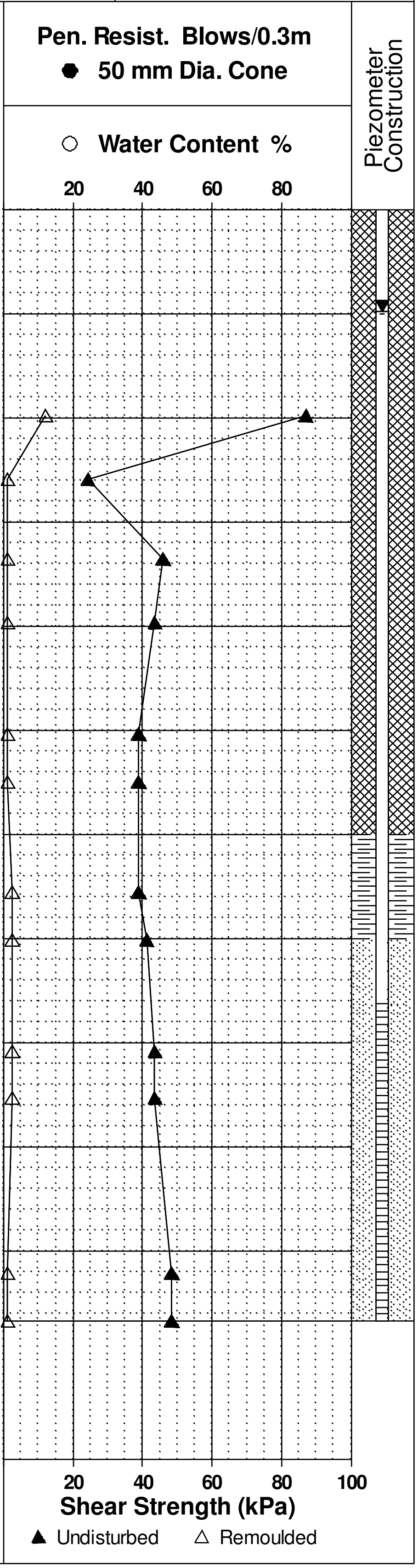
DATE May 2, 2013

FILE NO. PG2950

HOLE NO. BH 3-13

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with gravel	0.41	AU	1		
TOPSOIL	0.81				
Loose, brown **SILTY SAND,** trace clay	1.17	SS	2	83	6
Stiff, brown **SILTY CLAY** to **CLAYEY SILT** with sand seams	1.83	SS	3	46	3
Firm, grey **SILTY CLAY**		SS	4	100	1
	10.67	SS	5	100	1
End of Borehole					
(GWL @ 0.98m-May 16, 2013)					

DEPTH (m)	ELEV. (m)
0	97.27
1	96.27
2	95.27
3	94.27
4	93.27
5	92.27
6	91.27
7	90.27
8	89.27
9	88.27
10	87.27



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Kanata South Phase 2
Ottawa, Ontario

DATUM TBM - Finished floor level of Walmart. Geodetic elevation = 98.25m, as per plan prepared by Stantec Geomatics.

REMARKS 18T 0431548; 5013965

BORINGS BY CME 55 Power Auger

DATE May 2, 2013

FILE NO. **PG2950**

HOLE NO. **BH 4-13**

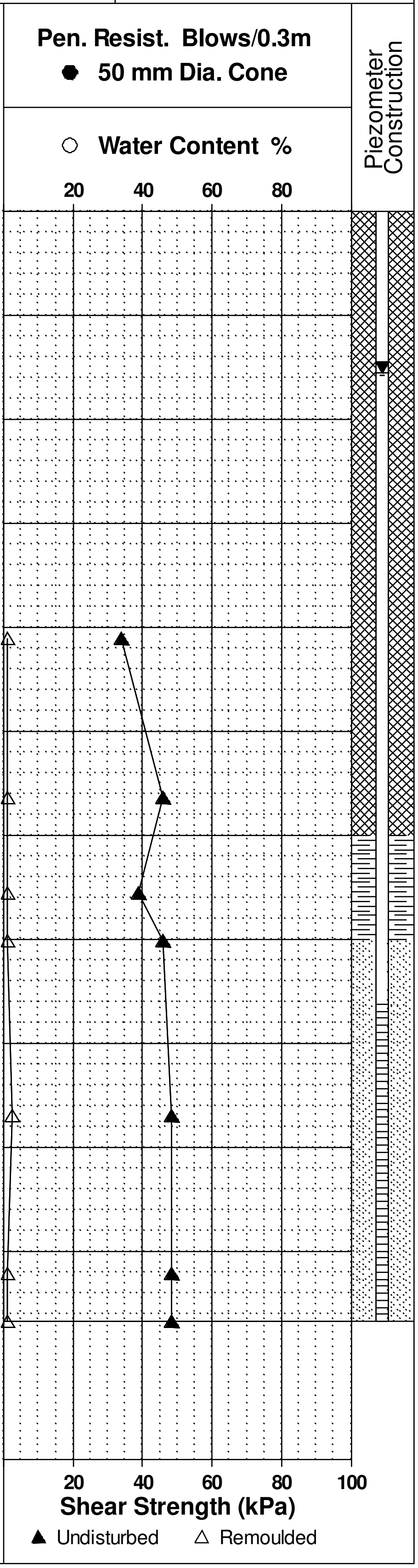
SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.20
FILL: Brown silty sand with gravel and crushed stone	0.91
FILL: Crushed stone, trace sand	1.37
Compact, brown **SILTY SAND,** trace clay	1.98
Stiff, brown **SILTY CLAY** to **CLAYEY SILT** with silty sand seams	2.74
Firm, grey **SILTY CLAY**	10.67

End of Borehole

(GWL @ 1.56m-May 16, 2013)

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	45	50+
SS	2	0	12
SS	3	0	5
SS	4	67	2
SS	5	100	1
SS	6	92	1
SS	7	71	1

DEPTH (m)	ELEV. (m)
0	98.17
1	97.17
2	96.17
3	95.17
4	94.17
5	93.17
6	92.17
7	91.17
8	90.17
9	89.17
10	88.17



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Fernbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 10 December 2010

FILE NO. PG2263

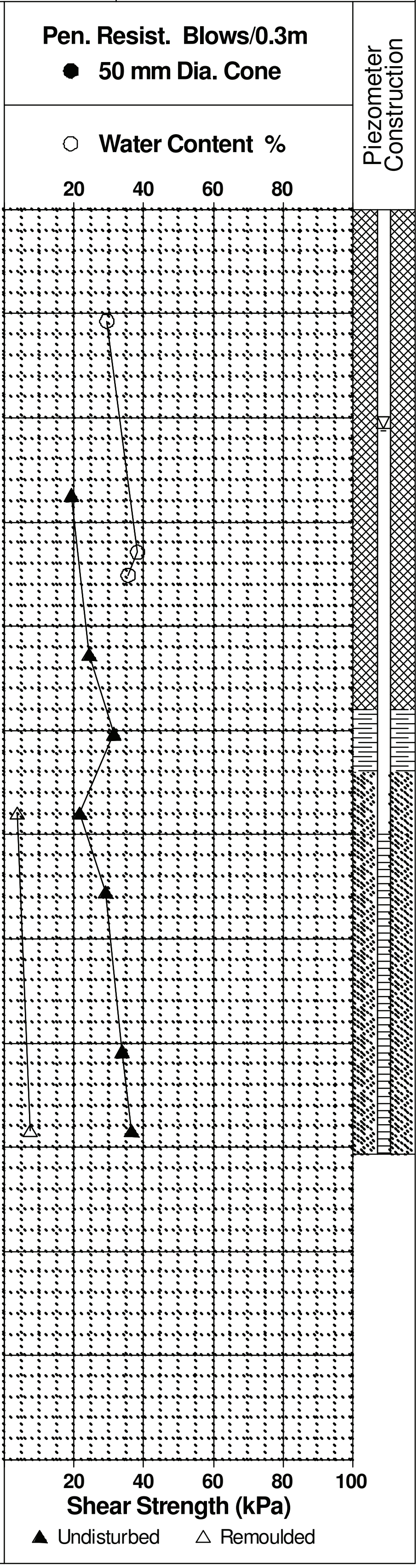
HOLE NO. BH 1

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.49
TOPSOIL	0.30						
Firm, brown **SILTY CLAY,** some sand	1.30	SS	1	25	7	1	96.49
		SS	2	0	2		
						2	95.49
						3	94.49
		TW	3	100			
						4	93.49
Soft to firm, grey **SILTY CLAY**						5	92.49
						6	91.49
		TW	4	100		7	90.49
						8	89.49
	9.07					9	88.49
						10	87.49
						11	86.49
						12	85.49

Dynamic Cone Penetration Test commenced @ 9.07m depth. Cone pushed to 35.36m depth. Practical refusal to DCPT @ 35.38m depth

(GWL @ 2.1m depth based on field observations)

(Piezometer destroyed - Jan. 11/11)



patersongroup **Consulting Engineers**

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Fernbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

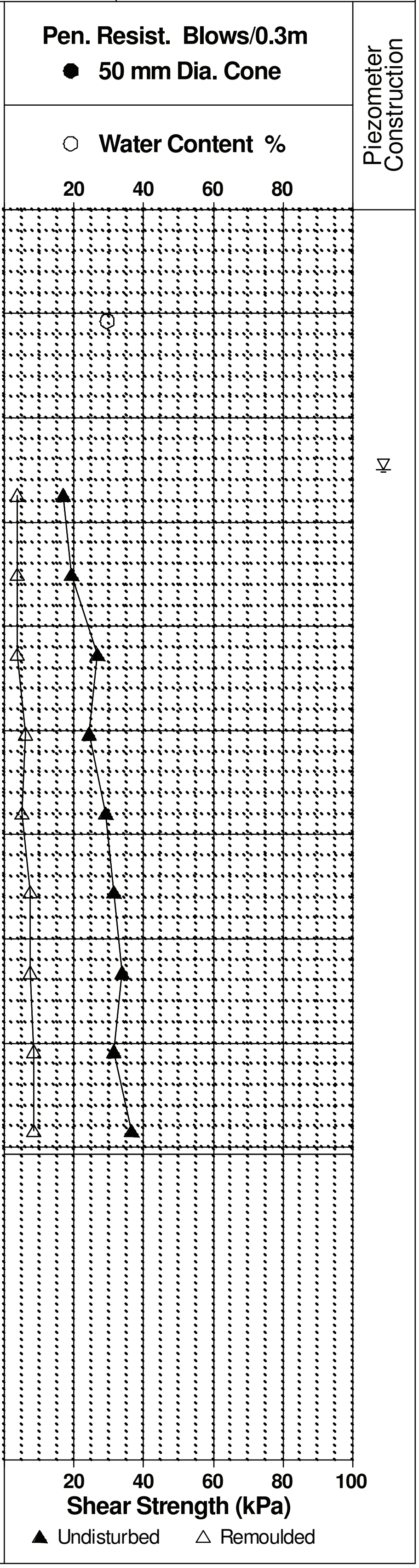
BORINGS BY CME 55 Power Auger

DATE 13 December 2010

FILE NO. PG2263

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.21
TOPSOIL 0.43							
Firm, brown **SILTY CLAY,** some sand 1.60		SS	1	17	10	1	96.21
		SS	2	33	3	2	95.21
						3	94.21
						4	93.21
Soft to firm, grey **SILTY CLAY**						5	92.21
						6	91.21
						7	90.21
						8	89.21
9.07						9	88.21
End of Borehole (GWL @ 2.5m depth based on field observations)							



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Fernbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

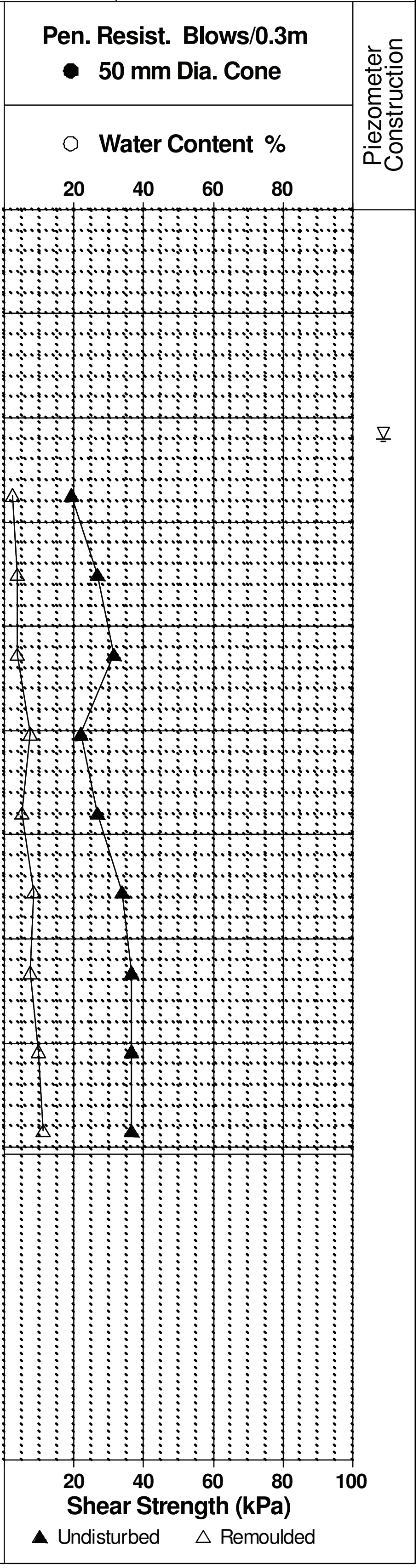
DATE 13 December 2010

FILE NO. PG2263

HOLE NO. BH 3

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.36				
Firm, brown **SILTY CLAY,** some sand	1.60	SS	1	58	7
Soft to firm, grey **SILTY CLAY**	9.07	SS	2	75	3
End of Borehole (GWL @ 2.2m depth based on field observations)					

DEPTH (m)	ELEV. (m)
0	97.42
1	96.42
2	95.42
3	94.42
4	93.42
5	92.42
6	91.42
7	90.42
8	89.42
9	88.42



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Fernbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

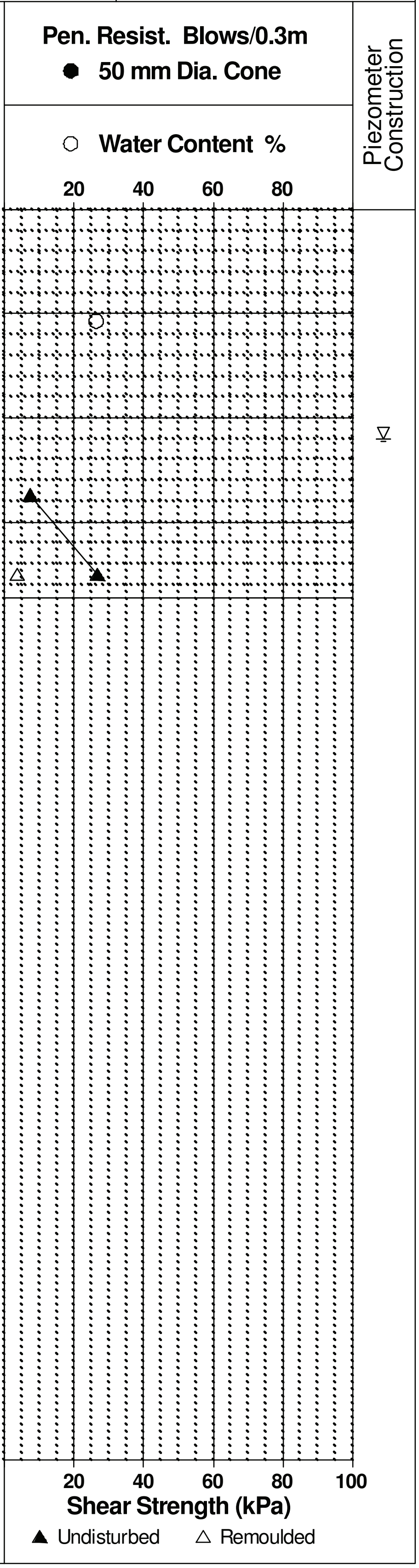
BORINGS BY CME 55 Power Auger

DATE 10 December 2010

FILE NO. PG2263

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.59
TOPSOIL 0.25							
Firm, brown **SILTY CLAY,** some sand 1.40		SS	1	75	4	1	96.59
Very soft to firm, grey **SILTY CLAY** 3.73		SS	2	0	2	2	95.59
						3	94.59
End of Borehole (GWL @ 2.2m depth based on field observations)							



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Fernbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

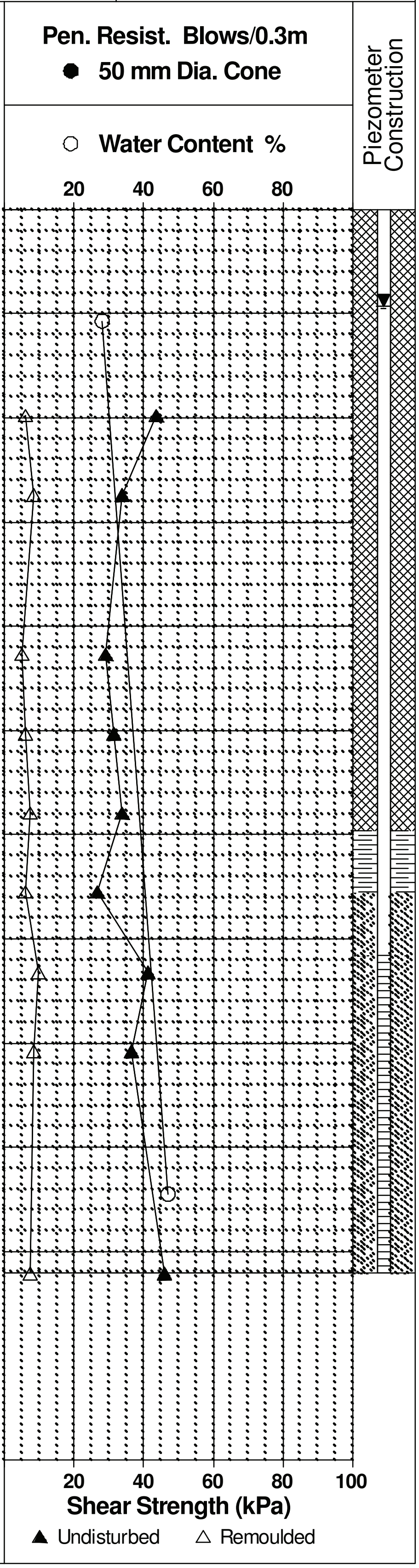
BORINGS BY CME 55 Power Auger

DATE 21 December 2010

FILE NO. PG2263

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.02
TOPSOIL 0.30		AU	1				
Firm to soft, brown **SILTY CLAY,** some sand 1.74		SS	2		4	1	96.02
Firm, grey **SILTY CLAY**						2	95.02
		TW	3			3	94.02
						4	93.02
						5	92.02
						6	91.02
						7	90.02
						8	89.02
		TW	4			9	88.02
10.21						10	87.02
End of Borehole (GWL @ 0.93m-Jan. 11/11)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Fernbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

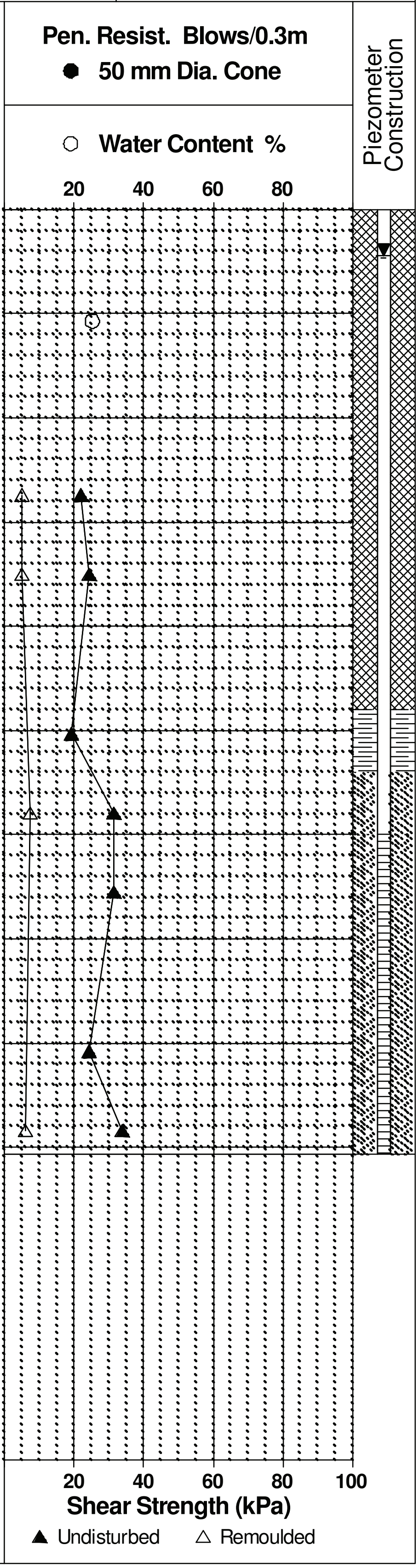
BORINGS BY CME 55 Power Auger

DATE 9 December 2010

FILE NO. PG2263

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	96.90
TOPSOIL 0.30							
Firm to soft, brown **SILTY CLAY,** some sand		SS	1	67	9	1	95.90
		SS	2	0	5		
2.21						2	94.90
						3	93.90
		TW	3	100		4	92.90
Soft to firm, grey **SILTY CLAY**						5	91.90
						6	90.90
		TW	4	100		7	89.90
						8	88.90
9.07						9	87.90
End of Borehole							
(GWL @ 0.44m-Jan. 11/11)							



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Dev.-Terry Fox Dr. at Fernbank Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

FILE NO. **PG1568**

REMARKS

HOLE NO. **BH 1**

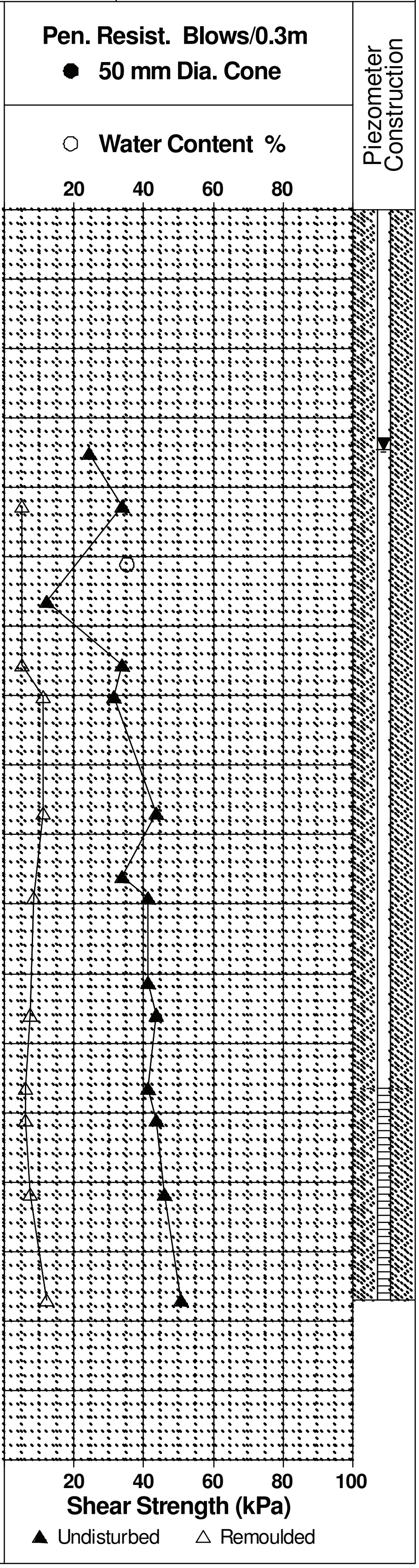
BORINGS BY CME 75 Power Auger

DATE 22 November 2007

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOIL	0.20
Stiff, brown **SILTY CLAY**, some sand seams	2.00
Soft to firm, grey **SILTY CLAY** (GWL @ 3.45m-Nov. 28/07)	15.70
Dynamic Cone Penetration Test commenced @ 15.70m depth, cone pushed to 35m depth through inferred silty clay.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	79	3
TW	2	88	
TW	3	100	
TW	4	100	

DEPTH (m)	ELEV. (m)
0	97.41
1	96.41
2	95.41
3	94.41
4	93.41
5	92.41
6	91.41
7	90.41
8	89.41
9	88.41
10	87.41
11	86.41
12	85.41
13	84.41
14	83.41
15	82.41
16	81.41
17	80.41
18	79.41



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Dev.-Terry Fox Dr. at Fernbank Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

FILE NO. **PG1568**

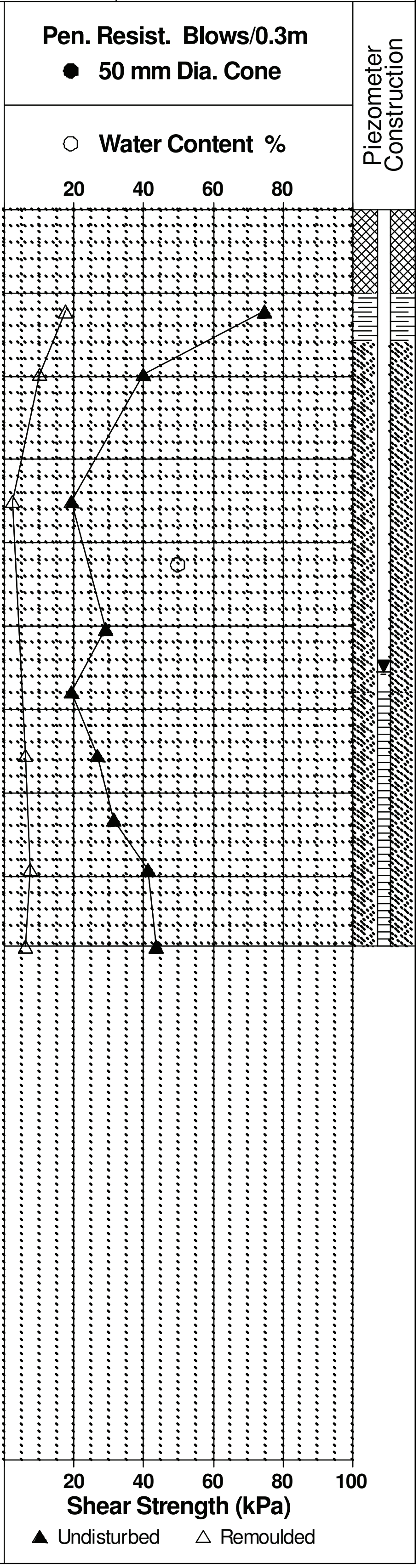
REMARKS

HOLE NO. **BH 4**

BORINGS BY CME 85 Power Auger

DATE 26 October 2009

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.00
Stiff, brown **SILTY CLAY,** some sand	▨	☒ AU	1				
						1	96.00
1.52							
Firm, grey **SILTY CLAY**	▨					2	95.00
		TW	2	79			
						3	94.00
		TW	3	100		4	93.00
						5	92.00
						6	91.00
						7	90.00
						8	89.00
8.84							
End of Borehole (GWL @ 5.55m-Nov. 2/09)							



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Dev.-Terry Fox Dr. at Fernbank Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

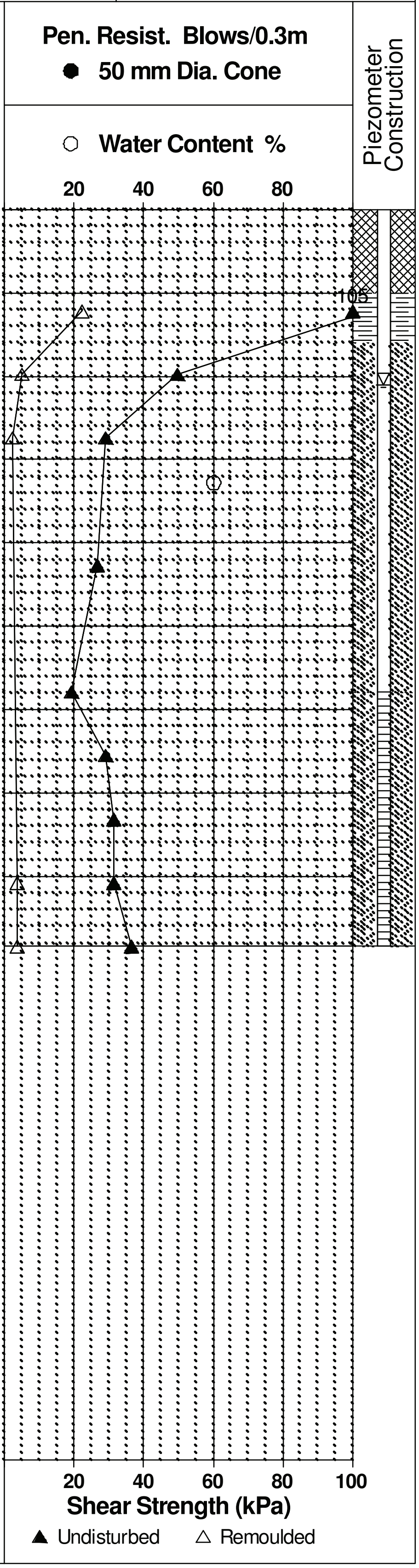
BORINGS BY CME 85 Power Auger

DATE 26 October 2009

FILE NO. PG1568

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.20
Very stiff to stiff, brown **SILTY CLAY**						1	96.20
2.13						2	95.20
		TW	1	100		3	94.20
		TW	2	100		4	93.20
Firm, grey **SILTY CLAY**						5	92.20
						6	91.20
						7	90.20
						8	89.20
8.84							
End of Borehole (GWL @ 2.10m based on field observations)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

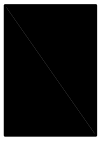
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

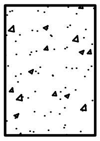
PERMEABILITY TEST

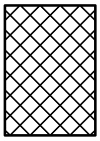
k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during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

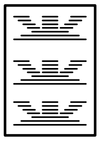
SYMBOLS AND TERMS (continued)

STRATA PLOT


Topsoil


Asphalt


Fill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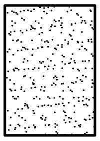
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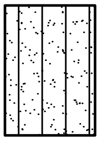
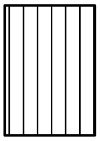
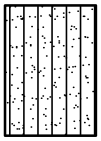
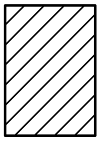
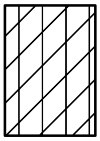
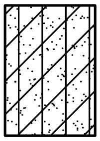

Silty Sand

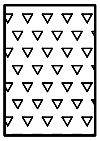

Silt

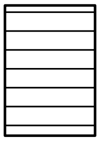

Sandy Silt

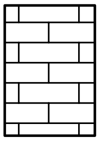

Clay


Silty Clay


Clayey Silty Sand

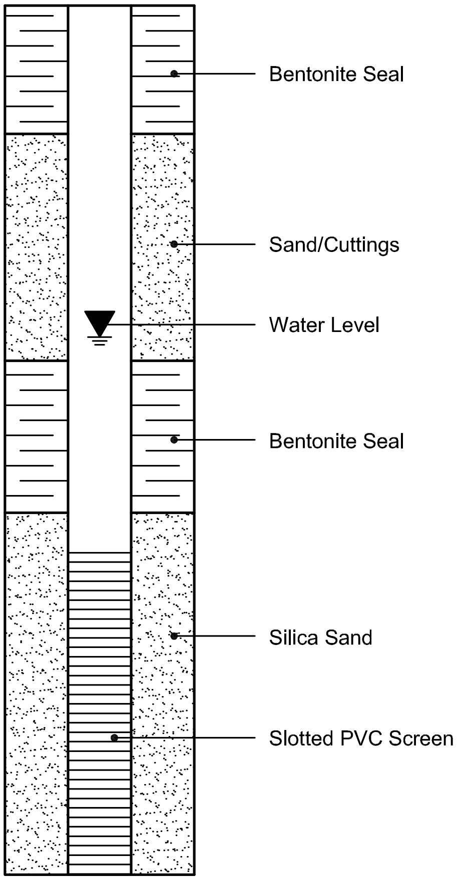

Glacial Till


Shale

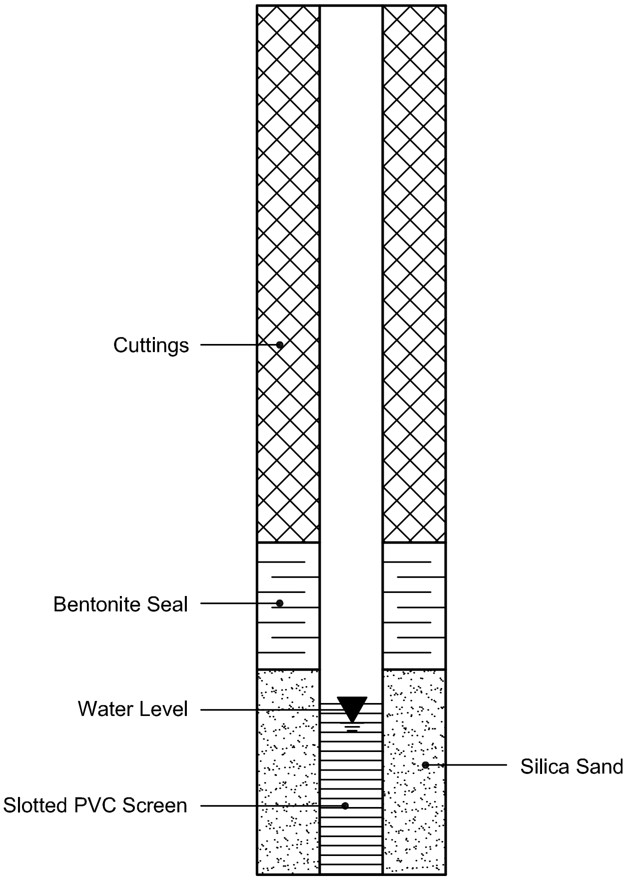

Bedrock

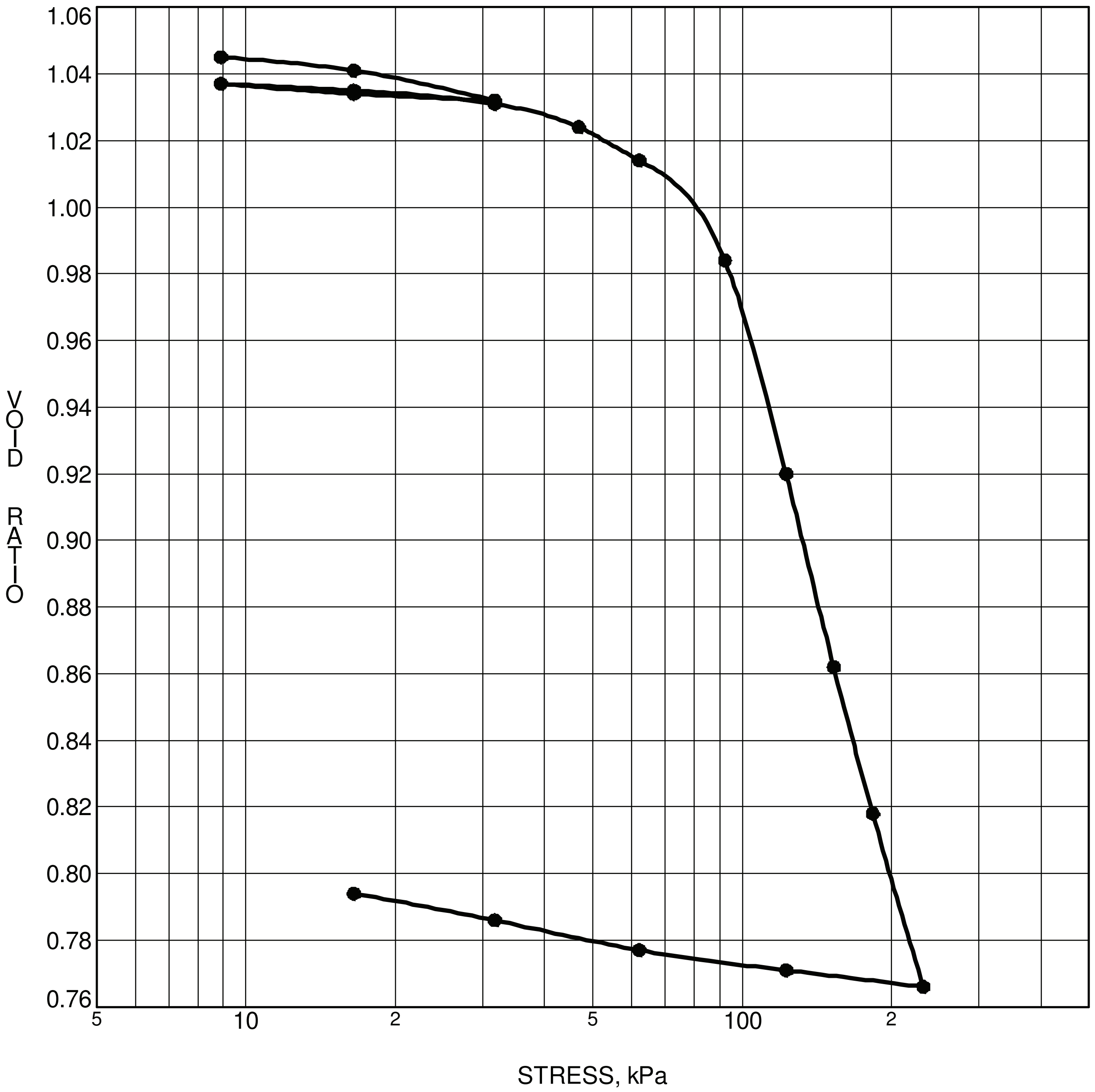
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





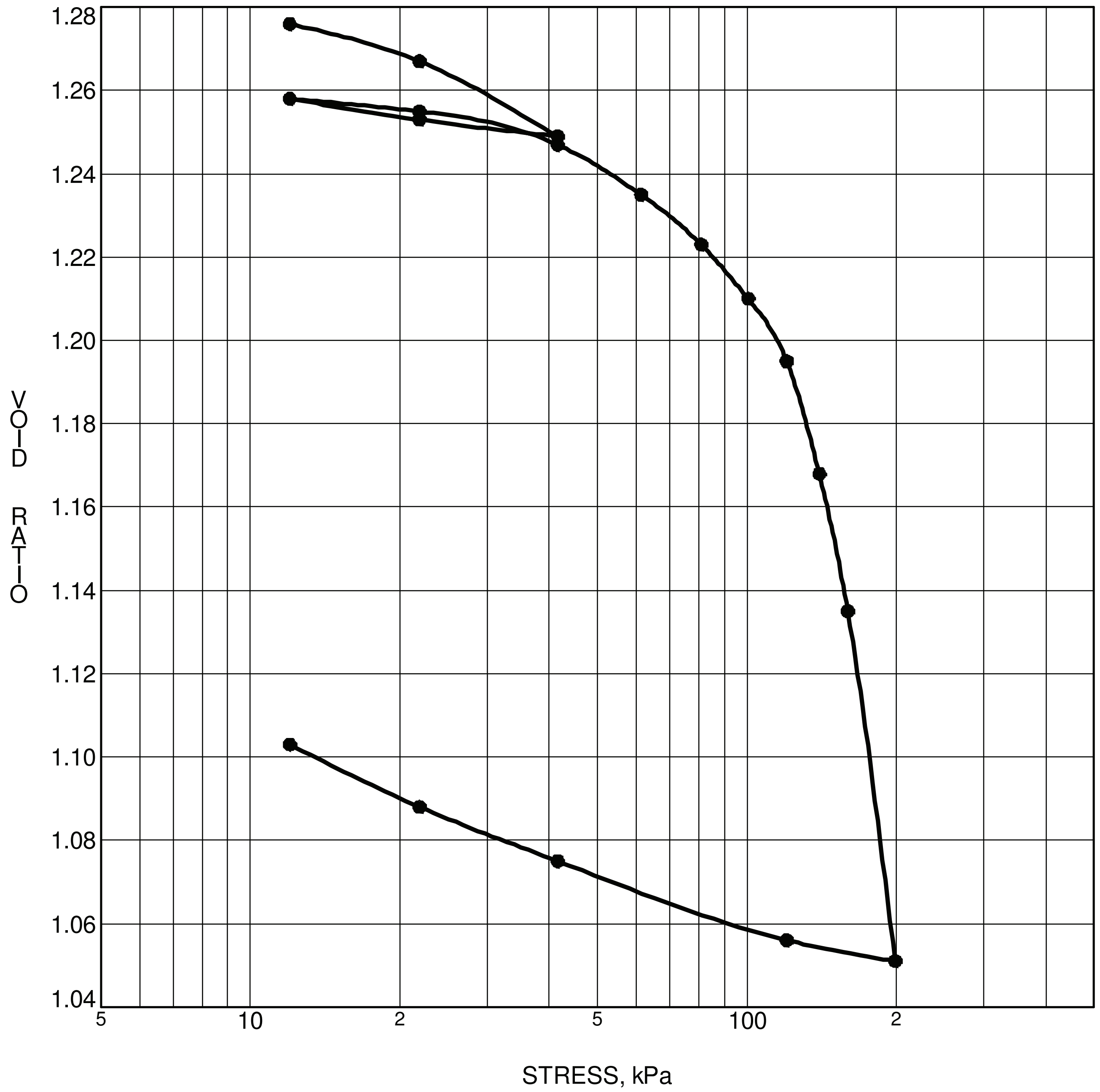
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 1**	p'_o	**44** kPa	Ccr	**0.010**
Sample No.	**TW 3**	p'_c	**84** kPa	Cc	**0.543**
Sample Depth	**3.29** m	OC Ratio	**1.9**	Wo	**38.2** %
Sample Elev.	**94.20** m	Void Ratio	**1.051**	Unit Wt.	**18.2** kN/m ^{3}

CLIENT	**Smart Centres**	FILE NO.	**PG2263**
PROJECT	**Geotechnical Investigation - Proposed Commercial Development - Fernbank Road**	DATE	**1/12/2011**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 5**	p'_o	**84** kPa	Ccr	**0.019**
Sample No.	**TW 4**	p'_c	**128** kPa	Cc	**0.766**
Sample Depth	**9.45** m	OC Ratio	**1.5**	Wo	**47.0** %
Sample Elev.	**87.57** m	Void Ratio	**1.292**	Unit Wt.	**17.3** kN/m^3

CLIENT **Smart Centres**

FILE NO. **PG2263**

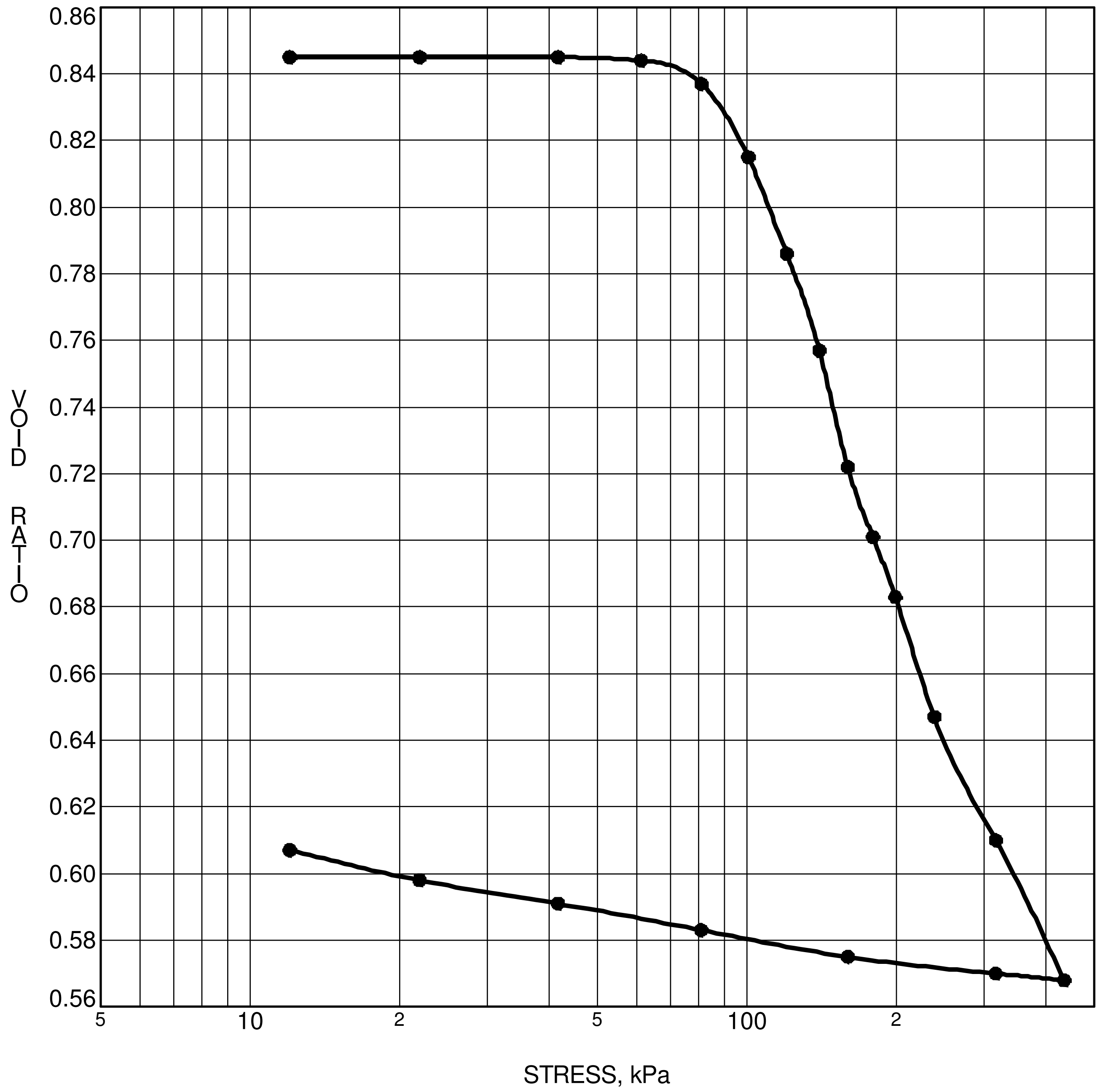
PROJECT **Geotechnical Investigation - Proposed Commercial Development - Fernbank Road**

DATE **1/12/2011**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 1**	p'$_o$	**54** kPa	Ccr	**0.001**
Sample No.	**TW 3**	p'$_c$	**98** kPa	Cc	**0.462**
Sample Depth	**5.11** m	OC Ratio	**1.8**	Wo	**35.2** %
Sample Elev.	**92.30** m	Void Ratio	**0.846**	Unit Wt.	**18.9** kN/m^3

CLIENT **Smart Centres**

FILE NO. **PG1568**

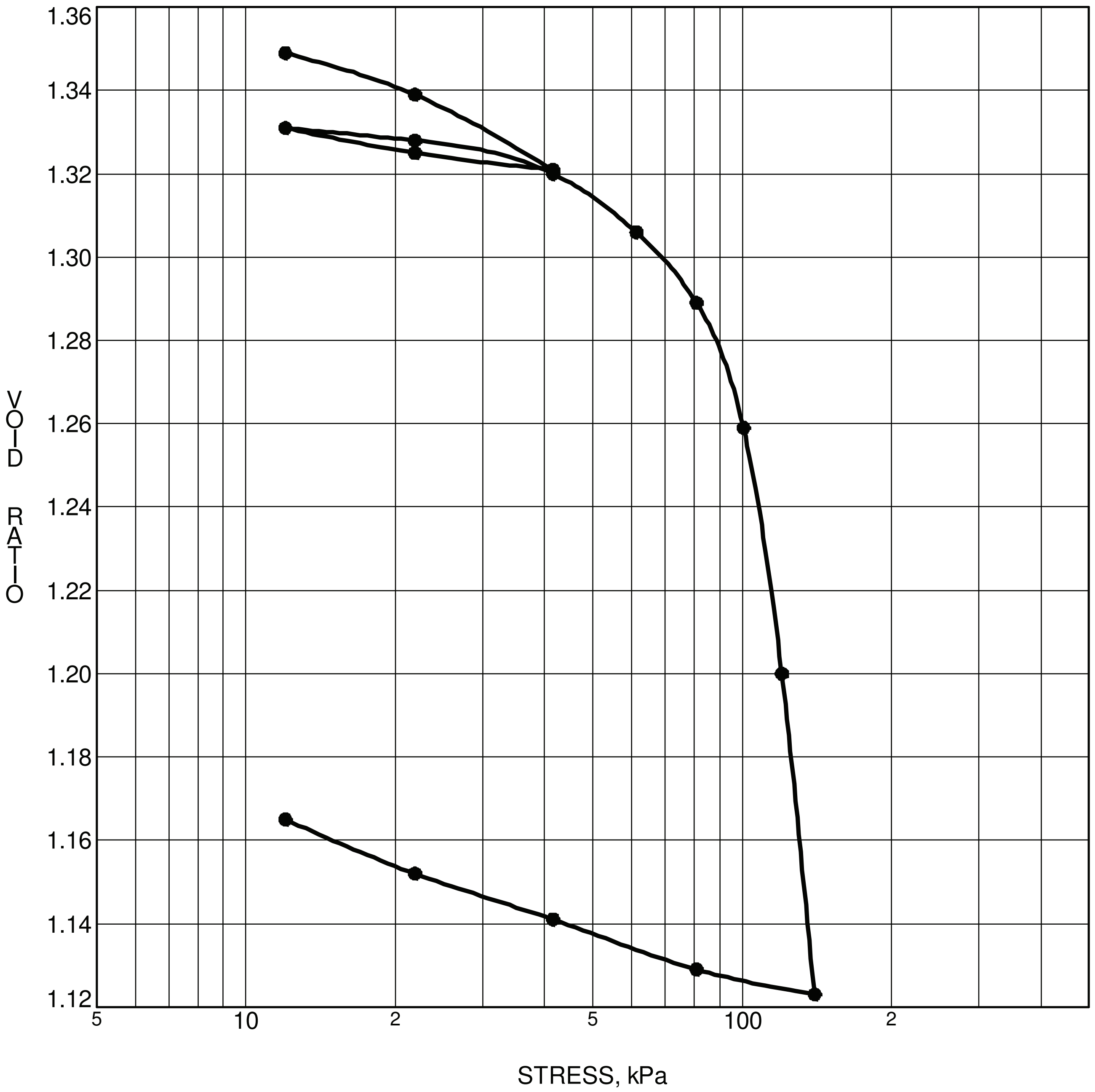
PROJECT **Geotechnical Investigation - Prop. Commercial Dev.-Terry Fox Dr. at Fernbank Rd.**

DATE **03/12/2007**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 4**	p'_o	**50** kPa	Ccr	**0.020**
Sample No.	**TW 3**	p'_c	**102** kPa	Cc	**1.227**
Sample Depth	**4.26** m	OC Ratio	**2.0**	Wo	**49.8** %
Sample Elev.	**92.74** m	Void Ratio	**1.369**	Unit Wt.	kN/m ^{3}

CLIENT **Smart Centres**

FILE NO. **PG1568**

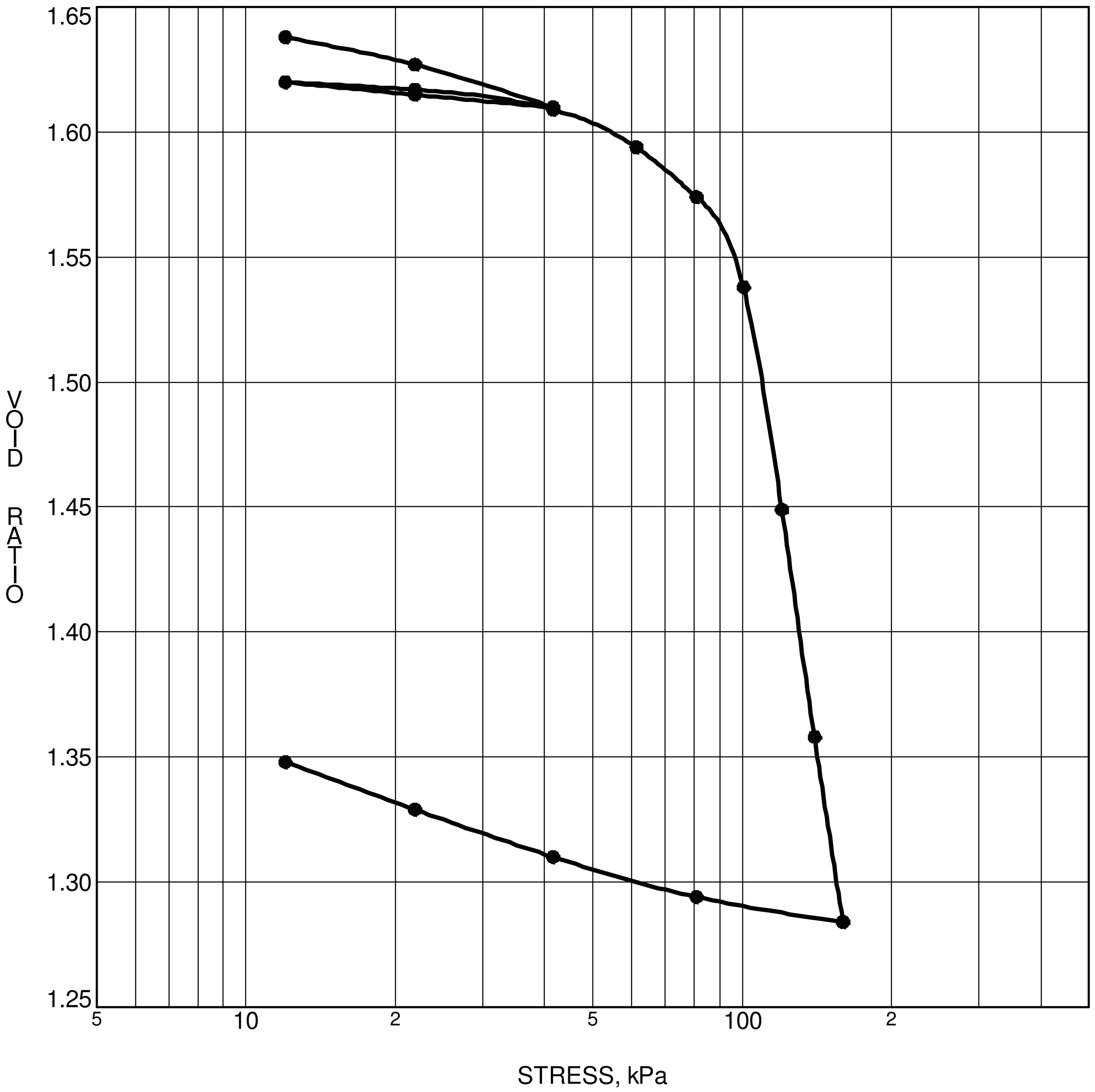
PROJECT **Geotechnical Investigation - Prop. Commercial Dev.-Terry Fox Dr. at Fernbank Rd.**

DATE **11/5/2009**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



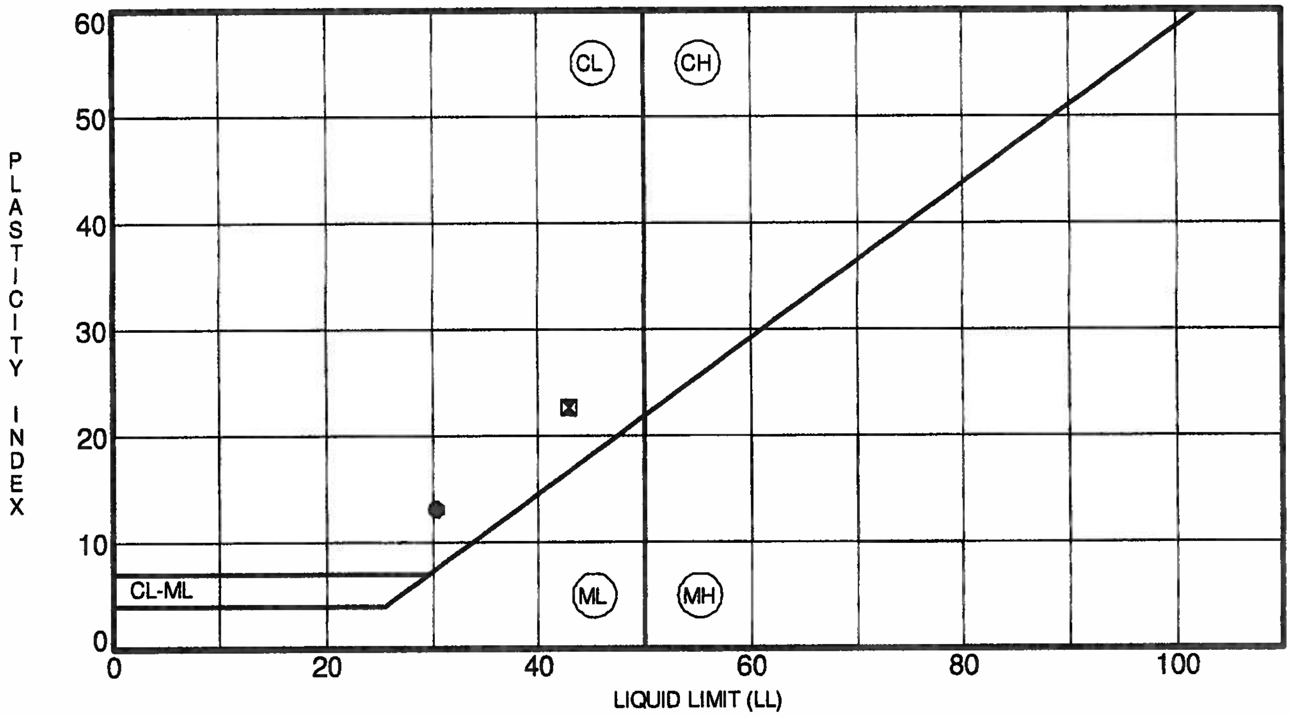
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 5**	p'_o	**40** kPa	Ccr	**0.016**
Sample No.	**TW 2**	p'_c	**97** kPa	Cc	**1.387**
Sample Depth	**3.28** m	OC Ratio	**2.4**	Wo	**60.1** %
Sample Elev.	**93.92** m	Void Ratio	**1.652**	Unit Wt.	**16.3** kN/m ^{3}

CLIENT	**Smart Centres**	FILE NO.	**PG1568**
PROJECT	**Geotechnical Investigation - Prop. Commercial Dev.-Terry Fox Dr. at Fernbank Rd.**	DATE	**10/30/2009**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1**	**TW 3**	**30**	**17**	**13**		**CL- Inorganic clays, low to medium plasticity**
☒	**BH 5**	**TW 4**	**43**	**20**	**23**		**CL-Inorganic clays, low to medium plasticity**

CLIENT **Smart Centres**

PROJECT **Geotechnical Investigation - Proposed Commercial Development - Fernbank Road**

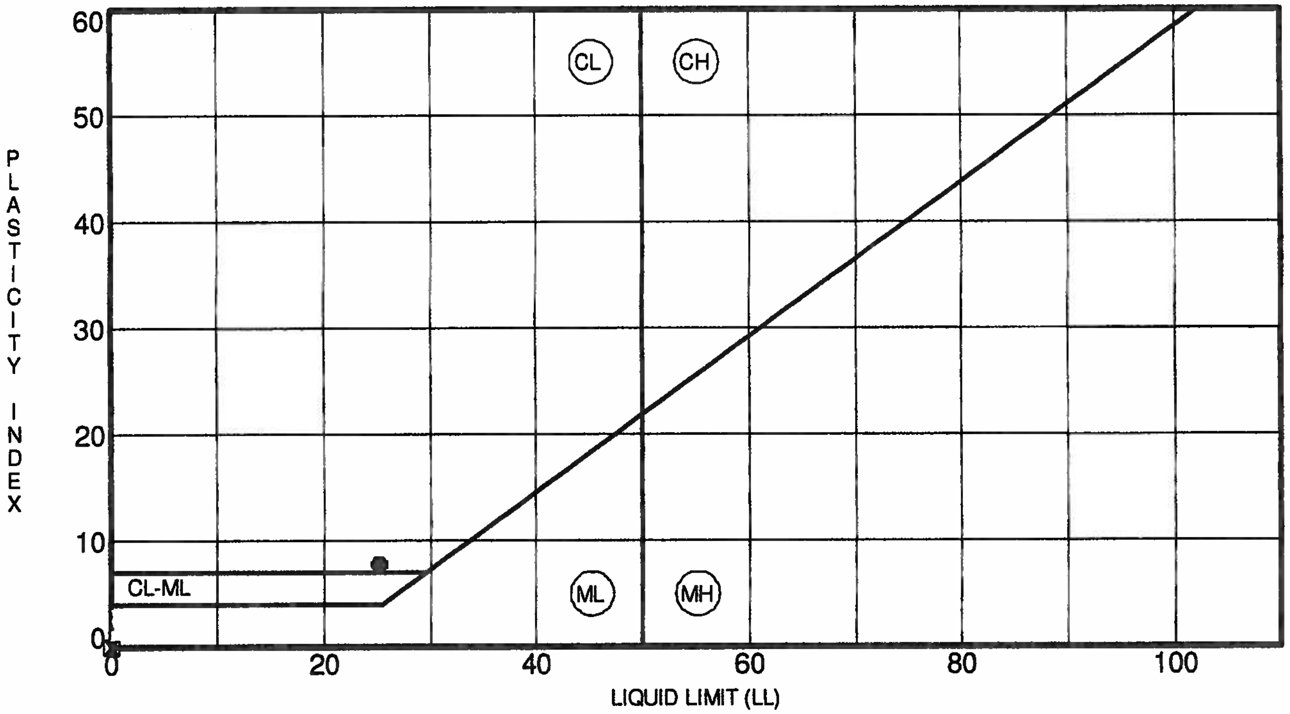
FILE NO. **PG2263**

DATE **17 Dec 10**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

ATTERBERG LIMITS' RESULTS



	Specimen Identification		LL	PL	PI	Fines	Classification
●	BH 1	TW 3	25	17	8		CL-Inorganic clays, low to medium plasticity

CLIENT **Smart Centres**

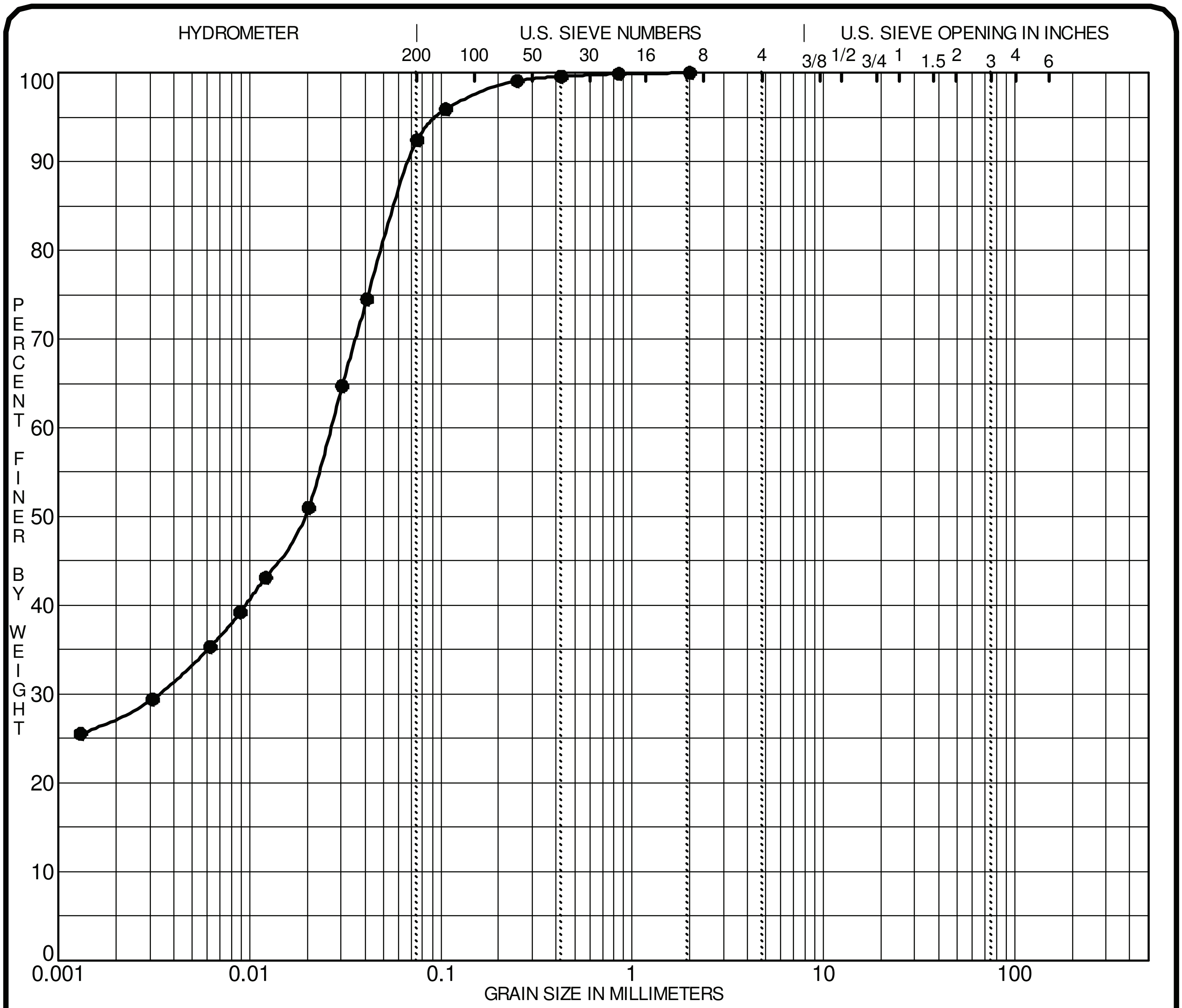
PROJECT **Geotechnical Investigation - Prop. Commercial**
Dev.-Terry Fox Dr. at Fernbank Rd.

FILE NO. **PG1568**

DATE **21 Nov 07**

patersongroup **Consulting Engineers**
28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

ATTERBERG LIMITS' RESULTS



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 1**	**TW 3**	**CL- Inorganic clays, low to medium plasticity**		**30**	**17**	**13**		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 1**	**TW 3**	**2.00**	**0.03**	**0.003**		**0.0**	**7.6**	**92.4**	

CLIENT **Smart Centres**

FILE NO. **PG2263**

PROJECT **Geotechnical Investigation - Proposed Commercial Development - Fernbank Road**

DATE **10 Dec 10**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

GRAIN SIZE DISTRIBUTION

Certificate of Analysis

Report Date: 10-May-2013
Order Date:7-May-2013

Client: **Paterson Group Consulting Engineers**
Client PO: 13939
Project Description: PG2950

	Client ID:	BH2 SS3	-	-	-
	Sample Date:	05-May-13	-	-	-
	Sample ID:	1319060-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	80.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.43	-	-	-
Resistivity	0.10 Ohm.m	25.8	-	-	-
Anions					
Chloride	5 ug/g dry	120	-	-	-
Sulphate	5 ug/g dry	86	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SHEAR WAVE VELOCITY PROFILES

DRAWING PG2950-1 - TEST HOLE LOCATION PLAN

DRAWING PG2950-2 - PERMISSIBLE GRADE RAISE PLAN

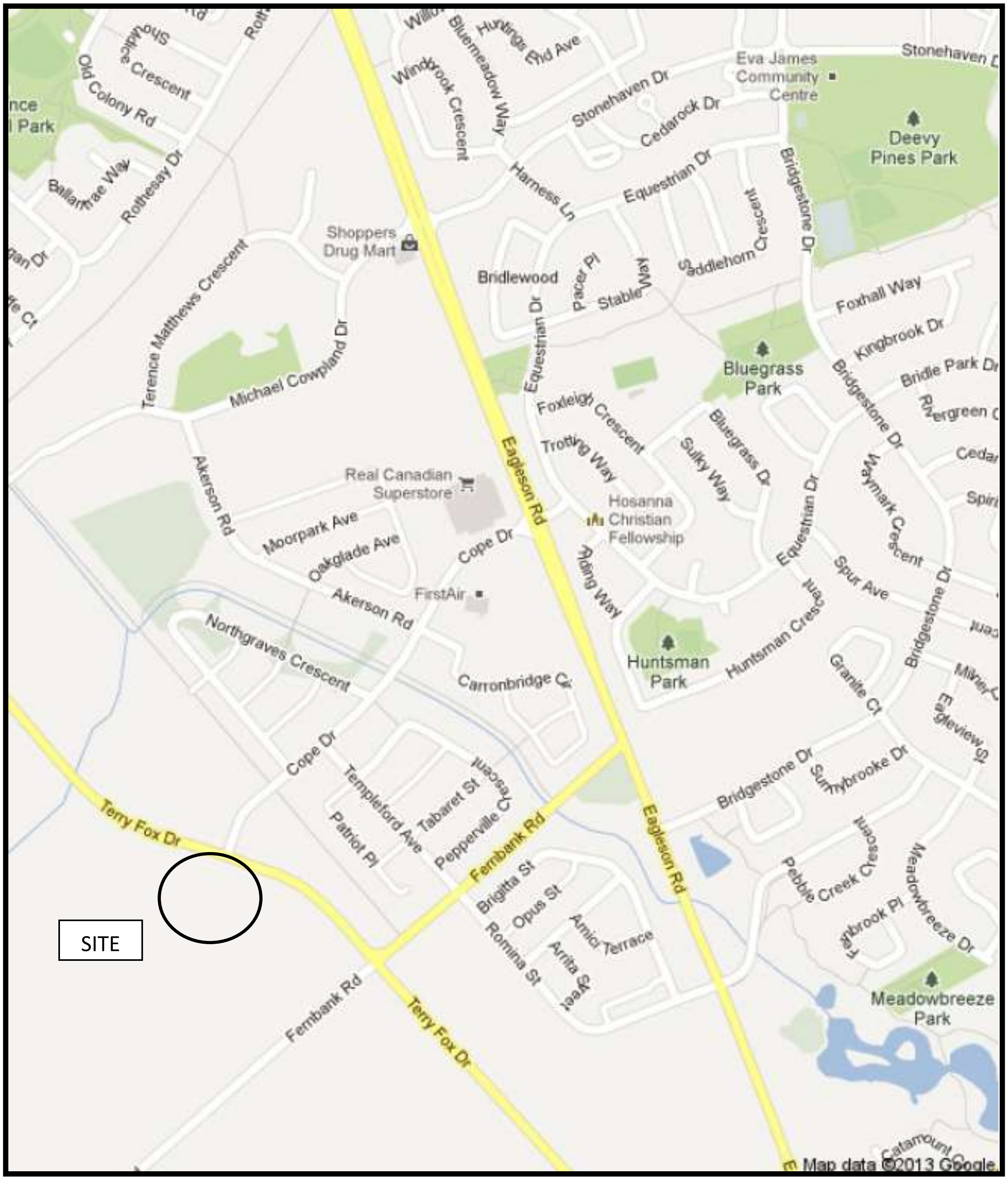


FIGURE 1

KEY PLAN

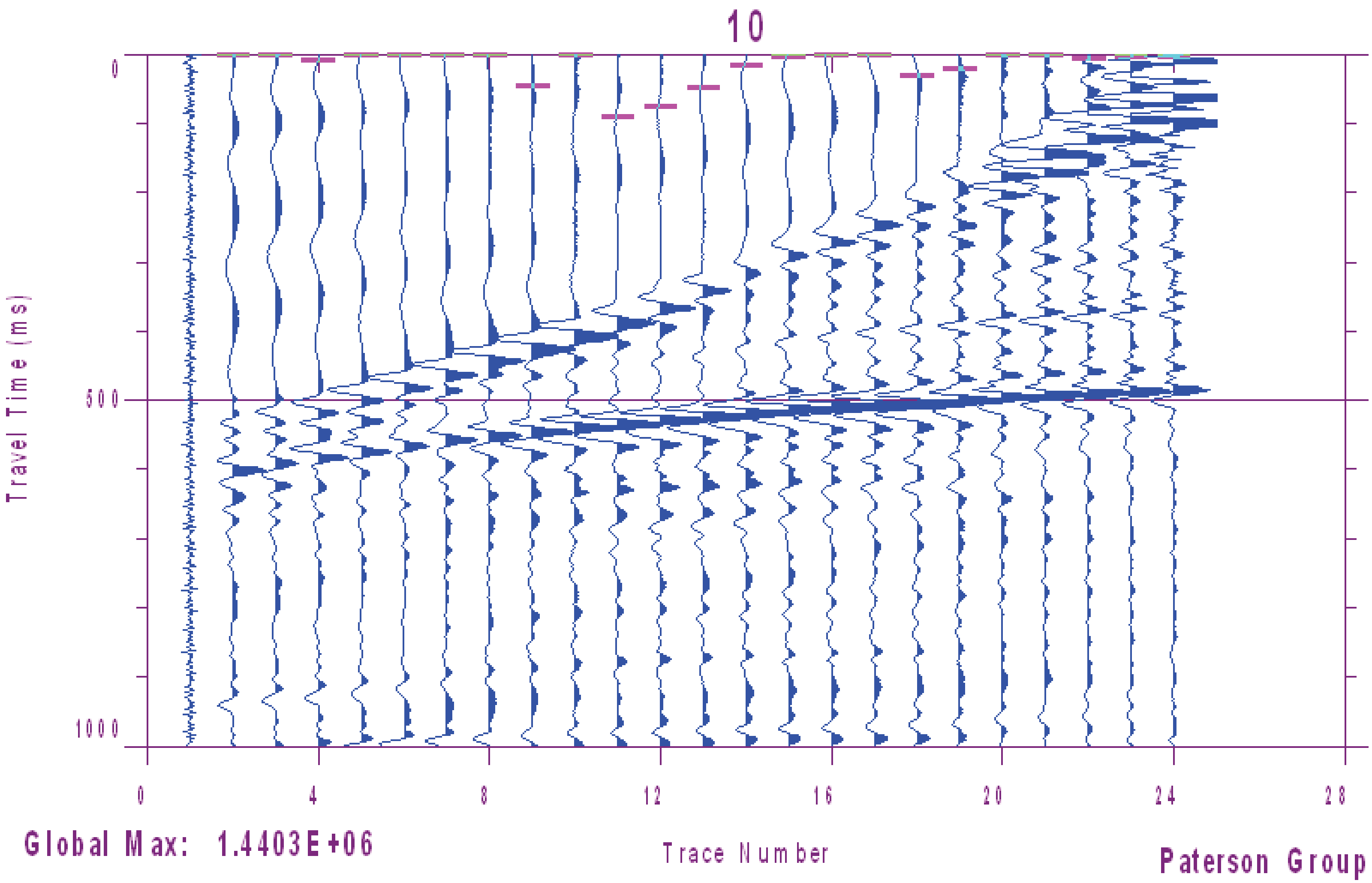


Figure 2 – Shear Wave Velocity Profile at Shot Location 72 m

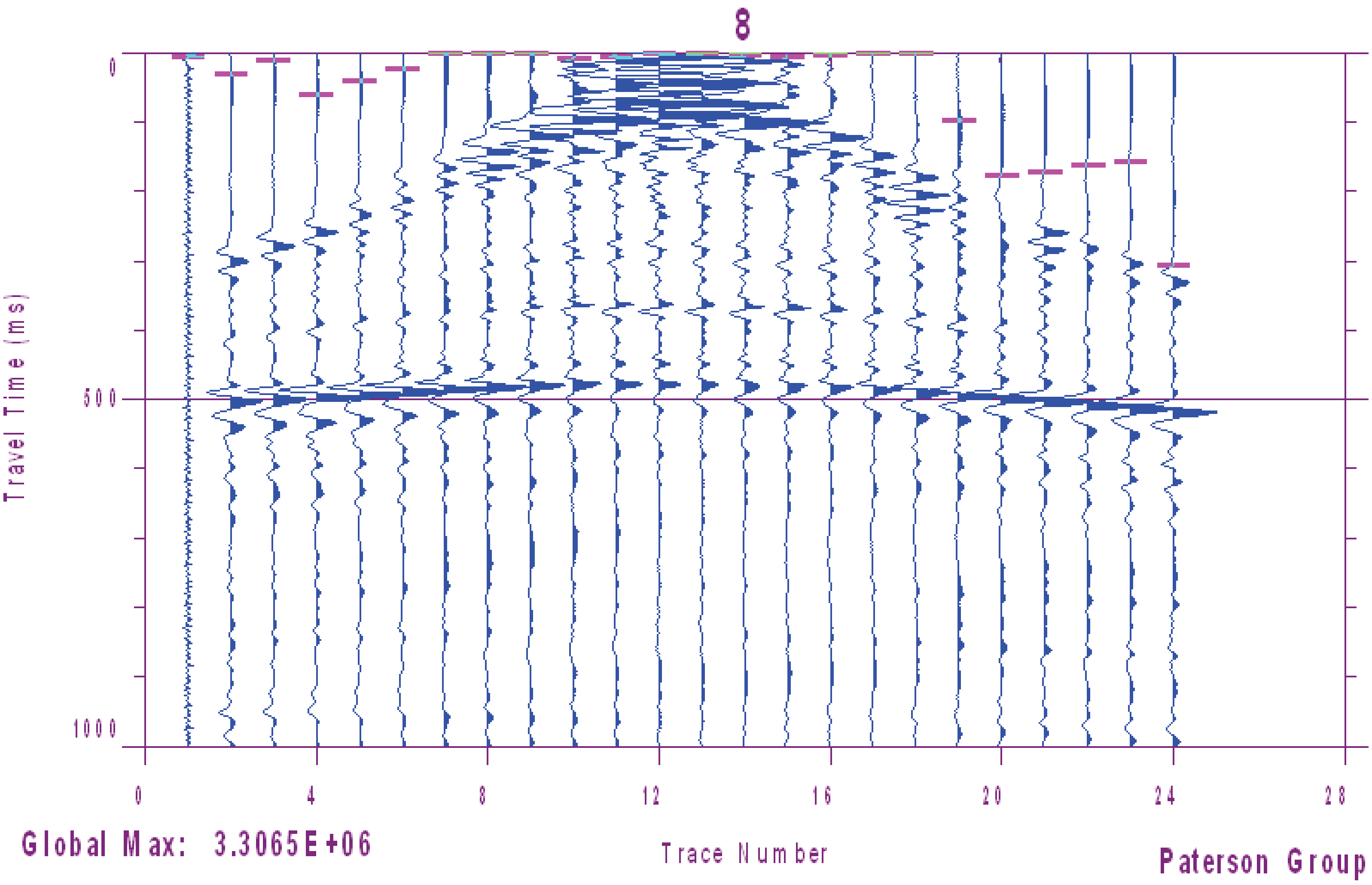
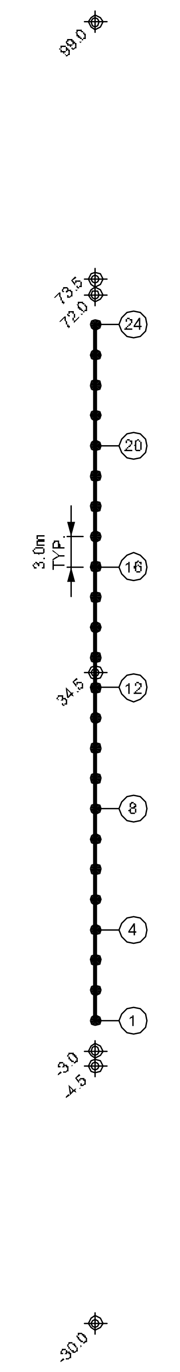
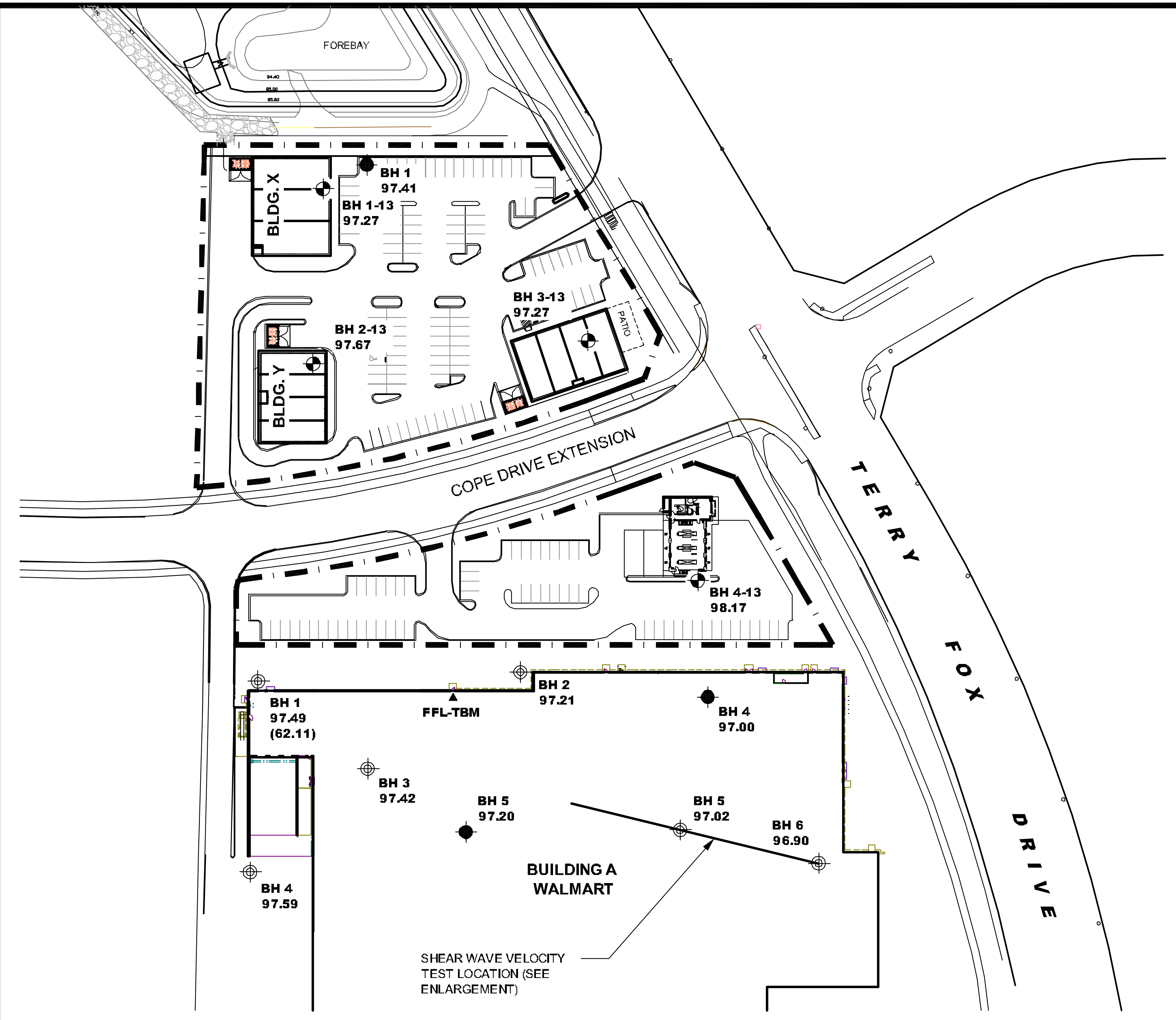


Figure 3 – Shear Wave Velocity Profile at Shot Location 34 m



ENLARGEMENT



LEGEND:

◆	BOREHOLE LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP REPORT PG2263
●	BOREHOLE LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP REPORT PG1568
97.49	GROUND SURFACE ELEVATION (m)
(62.11)	PRACTICAL DCPT REFUSAL ELEVATION (m)
•—•—•	GEOPHONE LOCATIONS
20	GEOPHONE NUMBER
73.5 ⊕	SHOT LOCATION

TBM - FINISHED FLOOR LEVEL OF WALMART. GEODETIC ELEVATION = 98.25m, AS PER PLAN PREPARED BY STANTEC GEOMATICS LIMITED.

SCALE - 1:1250
0 5 10 15 25 50m

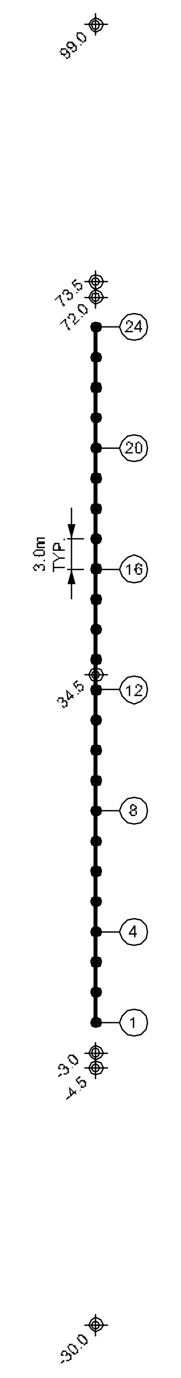
paterson group
consulting engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

Scale: 1:1250
Des.: RG
Dwn.: MPG
Chkd.: DG

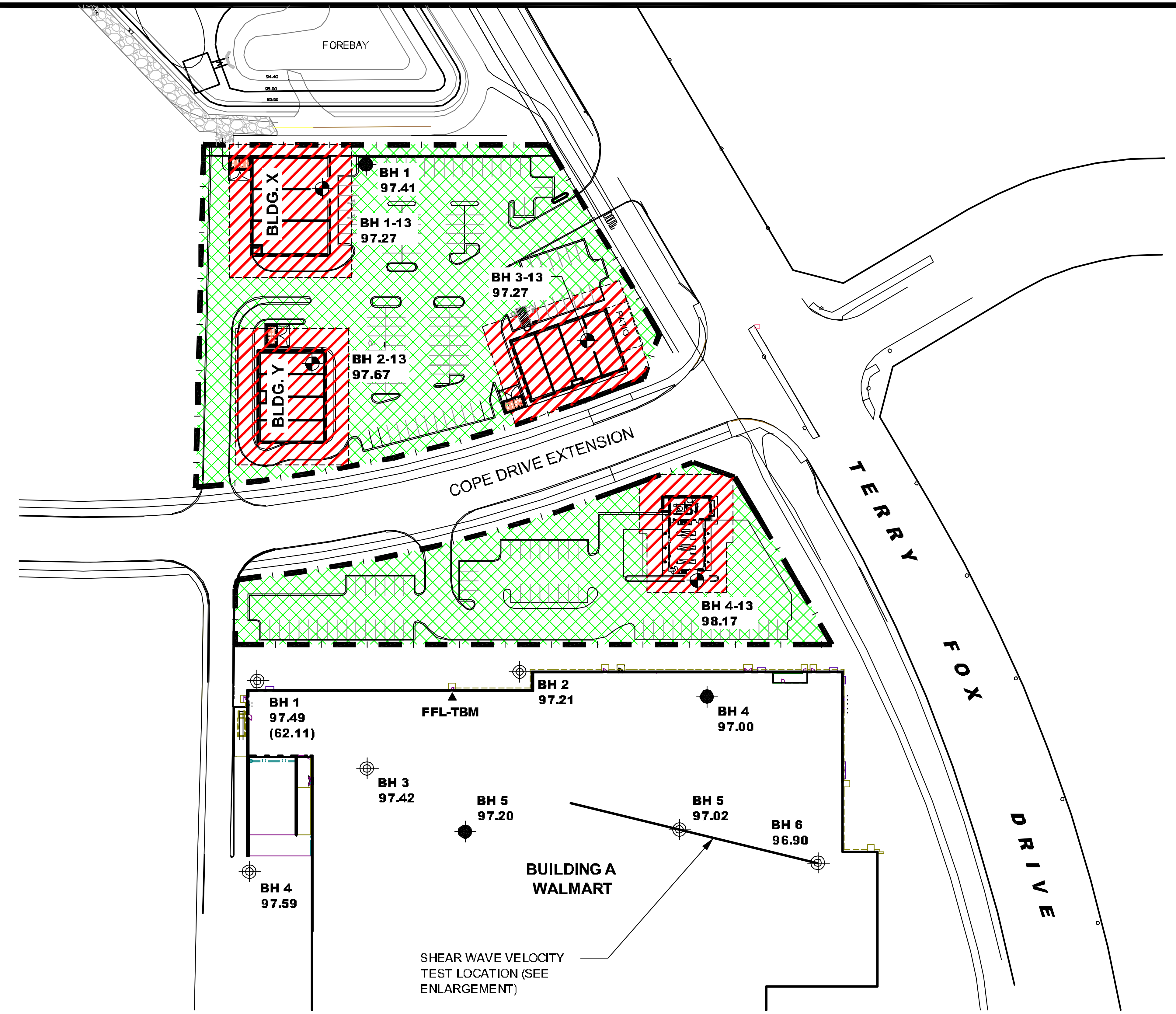
SMART CENTRES
GEOTECHNICAL INVESTIGATION
COMMERCIAL DEVELOPMENT - KANATA SOUTH PHASE 2
OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No. **PG2950-1**
Report No.: PG2950-1
Date: 05/2013



ENLARGEMENT



🟥	UP TO GEODETIC ELEVATION OF 98.5m
🟩	UP TO GEODETIC ELEVATION OF 99.0m

PERMISSIBLE GRADE RAISE RECOMMENDATIONS FOR BUILDING LOCATIONS BASED ON 80% OF AVAILABLE PRECONSOLIDATION PRESSURE, 0.5m GROUNDWATER LOWERING AND ANTICIPATED BUILDING LOADS.

PERMISSIBLE GRADE RAISE RECOMMENDATIONS FOR PARKING AREAS AND ACCESS LANES BASED ON 90% OF AVAILABLE PRECONSOLIDATION PRESSURE AND 0.5m GROUNDWATER LOWERING.

LEGEND:

⊕	BOREHOLE LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP REPORT PG2263
●	BOREHOLE LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP REPORT PG1568
97.49	GROUND SURFACE ELEVATION (m)
(62.11)	PRACTICAL DCPT REFUSAL ELEVATION (m)
•—•—•	GEOPHONE LOCATIONS
20	GEOPHONE NUMBER
73.5 ⊕	SHOT LOCATION

TBM - FINISHED FLOOR LEVEL OF WALMART. GEODETIC ELEVATION = 98.25m, AS PER PLAN PREPARED BY STANTEC GEOMATICS LIMITED.

SCALE - 1:1250
0 5 10 15 25 50m

paterson group
consulting engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

Scale:	1:1250
Des.:	RG
Dwn:	MPG
Chkd:	DG

SMART CENTRES
GEOTECHNICAL INVESTIGATION
COMMERCIAL DEVELOPMENT - KANATA SOUTH PHASE 2
OTTAWA, ONTARIO

PERMISSIBLE GRADE RAISE PLAN

Dwg. No.	PG2950-2
Report No.:	PG2950-1
Date:	05/2013