Geotechnical Engineering

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Geological Engineering

Materials Testing

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# Geotechnical Investigation

Kanata West Business Park
Proposed Office Building
Campeau Drive at Palladium Drive
Ottawa, Ontario

Prepared For

Taggart Realty Management


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Report: PG3115-6

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## Appendices

# 1.0 Introduction

Paterson Group (Paterson) was commissioned by Taggart Realty Management to conduct a geotechnical investigation for the proposed office building within the Kanata West Business Park located along Campeau Drive, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the investigation was to:

❏ determine the subsoil and groundwater conditions at this site by means of test holes.

❏ 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.

# 2.0 Proposed Development

Based on available drawings, it is understood that the proposed development will consist of a 5 storey office building of slab-on-grade construction along with associated access lanes, parking areas and landscaped areas. It is also understood that the proposed building will be municipally serviced.

# 3.0 Method of Investigation

## 3.1 Field Investigation

The field program for the geotechnical investigation was carried out by Paterson between January 13 and 14, 2014. At that time, three boreholes of the overall investigation were completed within the current phase of the development. The boreholes were advanced to a maximum depth of 6.1 m below existing grade. The test hole locations were distributed in a manner to provide general coverage of the subject site. The locations of the test holes are shown on Drawing PG3115-7 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track-mounted auger drill rig operated by a two person crew. The test pits were completed by a rubber tired backhoe at the selected locations across the site. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test hole procedures consisted of augering or excavating to the required depths at the selected locations and sampling the overburden.

**Sampling and In Situ Testing**

Soil samples were recovered during drilling from the auger flights or a 50 mm diameter split-spoon sampler. The split-spoon samples were classified on site and placed in sealed plastic bags. Grab samples were collected from the test pits at selected intervals and classified on site. All samples from all field investigations were transported to our laboratory. The depths at which the auger flight, split-spoon and grab samples were recovered are depicted as AU, SS, and G, respectively, on the Soil Profile and Test Data sheets 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.

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

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

**Groundwater**

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

## 3.2 Field Survey

The borehole and test hole locations for the field investigations were selected by Paterson and laid out in the field by Stantec Geomatics. It is understood that the elevations are referenced to a geodetic datum. The locations of the test holes and the ground surface elevation at each test hole location are presented on Drawing PG3115-7 - 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.

# 4.0 Observations

## 4.1 Surface Conditions

The subject site is currently predominantly vacant land stripped of topsoil and overgrown with brush with a gravel access road running east-west along the north border. The site is bordered to the east by Palladium Drive and further by an active construction site, to the south by Campeau Drive and similarly vacant land to the north and west. Generally, the ground surface is relatively flat with and approximately 1.0 m below the grade observed along Campeau Drive and Palladium Drive.

## 4.2 Subsurface Profile

**Overburden**

Generally, the subsurface profile encountered at the test hole locations within this portion of the Kanata West Business development consists of topsoil overlying a very stiff to stiff layer of brown silty clay underlain by loose to dense grey-brown silty fine sand. Practical refusal to augering was encountered at all boreholes at depths ranging from 4.6 to 6.1 m below ground surface. 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 site is located in an area where the bedrock consists of interbedded limestone and shale of the Verulam formation. Also, the bedrock surface is expected at depths ranging from 5 to 7 m.

## 4.3 Groundwater

Groundwater levels were measured in the boreholes and observations of the open test pits were made during previous investigations. The measured groundwater level (GWL) readings are presented in the Soil Profile and Test Data sheets in Appendix 1. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole. Groundwater levels can also be estimated based on recovered soil samples' moisture levels, coloring and consistency. Based on these observations, the long-term groundwater level can be estimated between 2.5 to 3.5 m below existing grade. Groundwater level observations are presented in the Soil Profile and Test Data sheets in Appendix 1. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

# 5.0 Discussion

## 5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. A seismic shear wave velocity test was completed for the subject site. The testing indicates that the seismic shear wave velocity,Vs30, is 994 m/s, which qualifies for a seismic site Class B provided the foundation is extended within 3 m of the bedrock surface. Therefore, to achieve the higher seismic site class (Class B), consideration should be given to placing footings over a lean concrete in-filled near vertical, zero entry trench. The lean concrete in-filled trench should be extended to a geodetic elevation of 100.0 m to be located within 3 m of the bedrock surface. Extending the lean concrete trench to elevation100.0 m will allow the proposed building to qualify for a seismic site Class B. Additional details regarding the seismic site class are presented in Subsection 5.4. It is expected that the lean concrete trench will be placed over an undisturbed, very stiff silty clay and/or a compact silty sand bearing surfaces.

Permissible grade raise restriction areas are also required due to the silty clay deposit. A permissible grade raise restriction of **2 m** is recommended for areas where settlement sensitive structures are founded over the silty clay deposit.

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

## 5.2 Site Grading and Preparation

**Stripping Depth**

Topsoil 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 footprint, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It 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 area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. 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. 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.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 95% of its SPMDD.

## 5.3 Foundation Design

### Bearing Resistance Values (Shallow Foundation)

Footings for the proposed buildings can be designed with the following bearing resistance values presented in Table 1.

**Table 1 - Bearing Resistance Values**

| Bearing Surface | Bearing Resistance Value at SLS (kPa) | Factored Bearing Resistance Value at ULS (kPa) |
|---|---|---|
| Lean Concrete infilled trench over undisturbed, stiff Silty Clay | 150 | 250 |
| Lean Concrete in-filled trench over undisturbed, compact Silty Sand | 150 | 250 |

**Note:** Strip footings, up to 3 m wide, and pad footings, up to 8 m wide, placed over a lean concrete in-filled trench over an undisturbed, silty clay bearing surface can be designed using the above noted bearing resistance values.
**-** It is recommended that the near vertical, zero entry trench be in-filled with a minimum 15 MPa lean concrete. The trench sidewalls should extend at least 150 mm beyond the footing faces. Paterson personnel should be on site during the excavation of the lean concrete trench to confirm the bearing resistance values provided.
**-** A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The above-noted bearing resistance values at SLS for soil bearing surfaces will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

The bearing resistance values are provided on the assumption that the footings are placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

**Lean Concrete Filled Trenches**

Typically, the excavation sidewalls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying soils. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

**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 stiff silty clay or compact silty sand above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

**Permissible Grade Raise Restriction**

Based on the current borehole information, a **permissible grade raise restriction of 2 m** is recommended for the proposed building and settlement sensitive structures where founded over a silty clay deposit. A post-development groundwater lowering of 0.5 m was assumed for our calculations.

## 5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building modifications in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The results of the shear wave velocity testing are attached to the present report.

**Field Program**

The shear wave testing was located within the proposed building footprint, as presented in Drawing PG3115-7 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly a north-south 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 of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is repeated at each shot location to provide an average shear wave velocity, $Vs_{30}$, of the upper 30 m profile immediately below the proposed building foundations. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock shear wave velocity due to the increasing quality of bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on available geological mapping, the local bedrock consists of interbedded limestone and shale of the Verulam formation with an anticipated overburden thickness of 5 to 7 m. Based on the test results, the average overburden shear wave velocity is **249 m/s**. Through interpretation, the bedrock shear wave velocity is **2,475 m/s**.

The $Vs_{30}$ was calculated using the standard equation for average shear wave velocity provided in the OBC 2012, and as presented below.

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

$$V_{s30} = \frac{30m}{\left(\frac{5m}{249m/s} + \frac{25m}{2475m/s}\right)}$$

$$V_{s30} = 994m/s$$

Based on the results of the seismic shear wave velocity testing, the average shear wave velocity, $Vs_{30}$, was calculated to be **994 m/s**. Therefore, a **Site Class B** is applicable for design of the proposed office building as per Note 1 of Table 4.1.8.4.A of the OBC 2012, “Site Classes A and B, hard rock and rock are not to be used if there is more than 3 m of softer materials between the rock and the underside of footing or mat foundations.” If footings are placed on a lean-concrete filled trench extending to geodetic elevation 100.0 m, which is within 3 m of the bedrock surface, a **Site Class B** is applicable for design of the proposed building. 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 materials, within the footprint of the proposed building, the native soil or approved fill surface will be considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab. The upper 200 mm of sub-slab fill should consist of an OPSS Granular A crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

## 5.6 Pavement Design

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

**Table 2 - Recommended Pavement Structure - Car Only Parking Areas**

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

**Table 3 - Recommended Pavement Structure - Local Roadways and Heavy Truck Loading Areas**

| 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 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.

**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. For areas where silty clay is encountered at subgrade level, it is recommended that subdrains be installed 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.

## 5.7 Percolation Rates

Infiltration galleries are anticipated to be located beneath the asphaltic parking areas within the subject site. Paterson has completed a detailed hydrogeological investigation of the lands south of the subject site as part of previous phases of the Kanata West Business Park in order to establish hydraulic conductivity and percolation time of in-situ materials.

Varying strata at the base of the galleries will be encountered during the installation and will affect the rate of stormwater infiltration into the underlying material. The calculations for the infiltration galleries should be reviewed to correspond with the appropriate percolation rates given the appropriate strata. The percolation rate was interpreted from the hydraulic conductivity which was estimated based on the range of grain size distribution for the proposed development area. Based on these values, the average percolation rate (T-Time) was estimated to be within the ranges in Table 4.

**Table 4 - Estimated Percolation Rates**

| Material | Hydraulic Conductivity - k (m/sec) | Percolation (T-time) - (mins/cm) |
|---|---|---|
| Silty Clay [1] | **$3 \times 10^{-6}$ to $1 \times 10^{-10}$** | **35 to 50+** |
| Silty Fine Sand / Sandy Silt [1] | **$1 \times 10^{-7}$ to $1 \times 10^{-8}$** | **20 to 50** |

[1] - Values are based upon site specific testing carried out at a nearby phase of the development

# 6.0 Design and Construction Precautions

## 6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structure in areas where snow cleared sidewalks are to be placed along the exterior of the structure. 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.

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

## 6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

## 6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be 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).

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

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

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

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.

## 6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & 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 bedding for sewer and water pipes when placed on soil subgrade. 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 (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material’s SPMDD.

Generally, it should be possible to re-use the moist, not wet, silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

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 that are located in the areas underlain by silty clay. 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

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

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

## 6.6 Winter Construction

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

The subsoil conditions at this site 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 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.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

## 6.7 Landscaping Considerations

**Tree Planting Restrictions**

Based on the subsurface profile encountered at the borehole locations, the silty clay deposit, consists of a brown silty clay ‘crust’ with a very stiff to stiff consistency extending to a depth of 2.1 to 3.7 m below existing ground surface. The long-term groundwater level within the subject site is 2.5 to 3.5 m below existing ground surface based on our observations. It should be noted that the low moisture levels of the silty clay crust provide a more desiccated layer, which is less prone to shrinkage than a grey silty clay below the long-term groundwater level, based on our experience with clays in the Ottawa area. It should also be noted that the results of the atterberg limit tests completed for adjacent blocks to the subject site were found to be below the 40% threshold required for the new tree planting setbacks. Therefore, the current phase of the proposed development is located within an area of low to medium sensitivity silty clay deposits with respect to tree planting restrictions.

It should be further noted that the proposed building foundation is expected to extend below the silty clay deposit to meet the seismic Site Class B requirements as discussed in Subsection 5.0. Therefore, the City Guidelines for Tree Planting in Sensitive Marine Clay Soils is not considered applicable since no sensitive marine clay is present within 2 m of design underside of foundation.

# 7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

- ❏ Review master grading plan from a geotechnical perspective, once available.
- ❏ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

# 8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

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

**Paterson Group Inc.**

LICENSED PROFESSIONAL ENGINEER
July 6, 2019
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, EIT

David Gilbert, P.Eng.

**Report Distribution:**

- ❑ Taggart Realty Management (3 copies)
- ❑ Paterson Group (1 copy)

# APPENDIX 1

## SOIL PROFILE & TEST DATA SHEETS

## SYMBOLS AND TERMS

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial Development - Huntmar Road**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Stantec Geomatics Ltd.

**REMARKS**

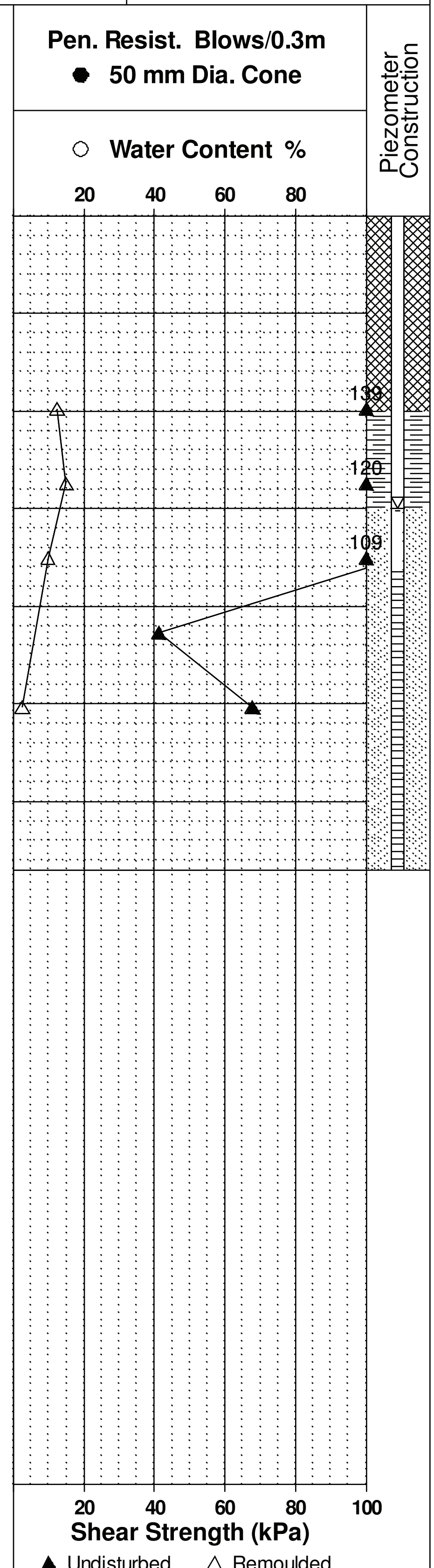
**BORINGS BY** CME 55 Power Auger

**DATE** January 13, 2014

**FILE NO.** PG3115

**HOLE NO.** BH13

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 103.03 |
| **TOPSOIL** 0.28 | | | | | | | |
| | | AU | 1 | | | | |
| | | SS | 2 | 100 | 5 | 1 | 102.03 |
| Very stiff to stiff, brown **SILTY CLAY** | | | | | | 2 | 101.03 |
| | | | | | | 3 | 100.03 |
| - firm to stiff and grey-brown by 3.7m depth | | | | | | 4 | 99.03 |
| 5.30 | | | | | | 5 | 98.03 |
| Grey-brown **CLAYEY SILT** | | SS | 3 | 83 | 10 | 6 | 97.03 |
| 6.70 | | SS | 4 | 83 | 6 | | |
| End of Borehole | | | | | | | |
| (GWL @ 3.0m depth based on field observations) | | | | | | | |



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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
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**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Stantec Geomatics Ltd.

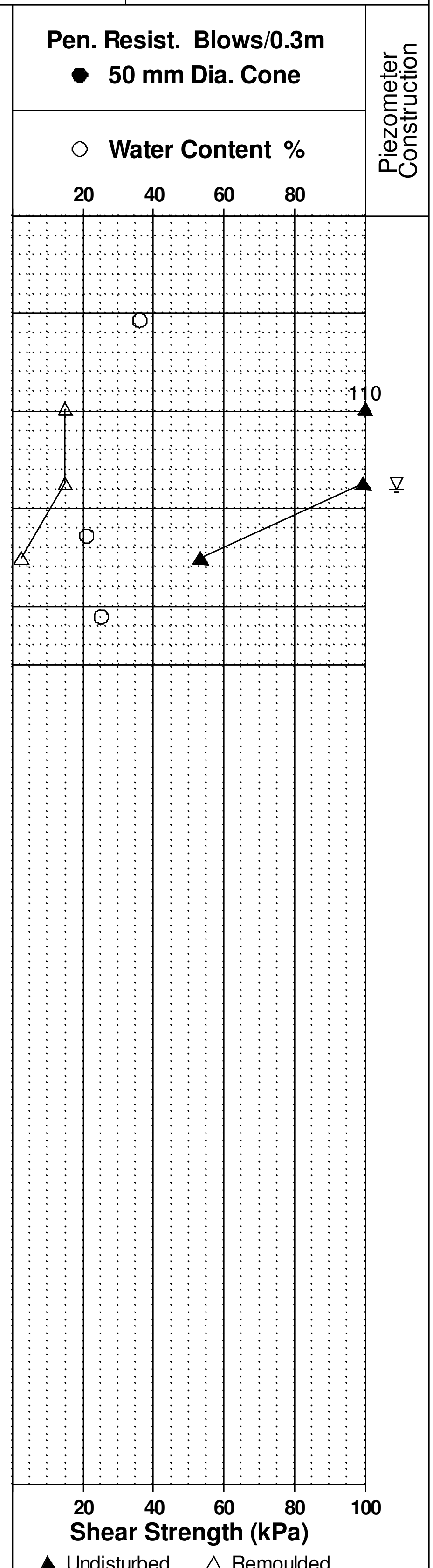
**FILE NO.** **PG3115**

**REMARKS**

**HOLE NO.** **BH14**

**BORINGS BY** CME 55 Power Auger **DATE** January 13, 2014

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 103.55 |
| TOPSOIL 0.28 | | | | | | | |
| Very stiff to stiff, brown **SILTY CLAY** | | SS | 1 | 100 | 3 | 1 | 102.55 |
| | | | | | | 2 | 101.55 |
| 3.66 | | | | | | 3 | 100.55 |
| Grey-brown **SILTY FINE SAND** | | SS | 2 | 71 | 6 | 4 | 99.55 |
| 4.60 | | SS | 3 | 0 | 50+ | | |
| End of Borehole<br><br>Practical refusal to augering at 4.60m depth<br><br>(GWL @ 2.8m depth based on field observations) | | | | | | | |



**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial Development - Huntmar Road**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Stantec Geomatics Ltd.

**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** January 14, 2014

**FILE NO.** PG3115

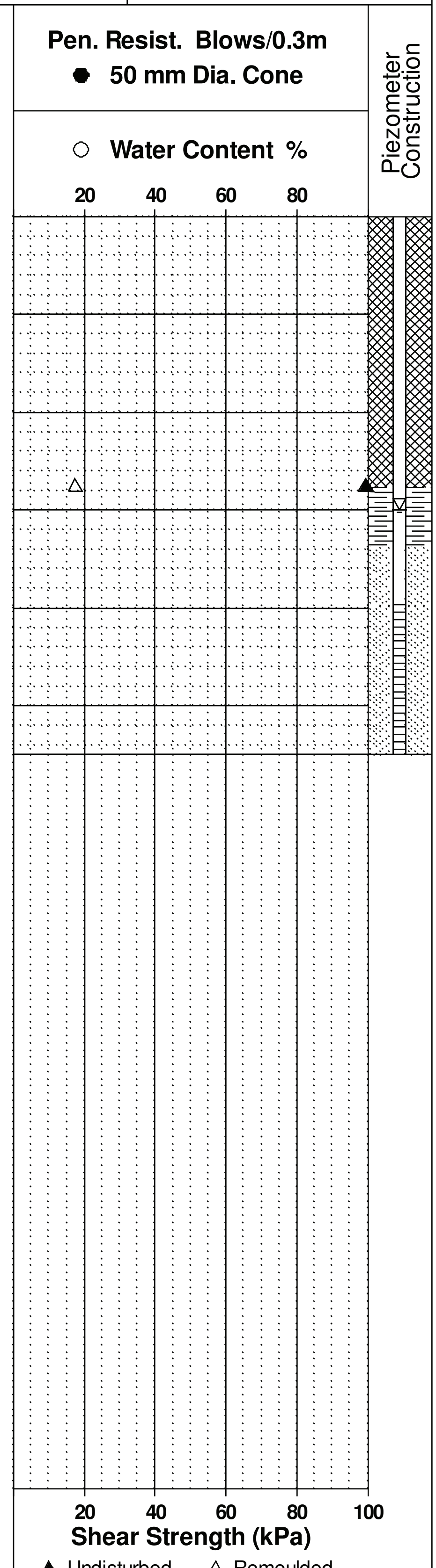
**HOLE NO.** BH15

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 104.43 |
| TOPSOIL 0.20 | | | | | | | |
| Stiff, brown **SILTY CLAY** | | AU | 1 | | | | |
| | | SS | 2 | 83 | 4 | 1 | 103.43 |
| | | SS | 3 | 100 | 4 | 2 | 102.43 |
| 2.90 | | | | | | 3 | 101.43 |
| Grey-brown **SILTY FINE SAND** | | SS | 4 | 67 | 14 | | |
| | | SS | 5 | 42 | 3 | 4 | 100.43 |
| | | SS | 6 | 83 | 5 | 5 | 99.43 |
| 5.49 | | SS | 7 | 0 | 50+ | | |

End of Borehole

Practical refusal to augering at 5.49m depth

(GWL @ 3.0m depth based on field observations)



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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial Development - Huntmar Road**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Stantec Geomatics Ltd.

**FILE NO.** PG3115

**REMARKS**

**HOLE NO.** BH20

**BORINGS BY** CME 55 Power Auger **DATE** January 14, 2014

| SOIL DESCRIPTION | Depth of strata change (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 104.66 |
| **TOPSOIL** | 0.33 | | | | | | |
| Brown **SILTY CLAY** with sand | 0.69 | AU | 1 | | | | |
| Compact, brown **SILTY FINE SAND** | | SS | 2 | 83 | 16 | 1 | 103.66 |
| - grey-brown by 2.2m depth | | SS | 3 | 67 | 15 | 2 | 102.66 |
| - grey by 3.0m depth | | SS | 4 | 83 | 12 | 3 | 101.66 |
| - some gravel below 4.1m depth | | SS | 5 | 67 | 14 | 4 | 100.66 |
| | 5.64 | SS | 6 | 75 | 10 | 5 | 99.66 |

End of Borehole

Practical refusal to augering at 5.64m depth

(GWL @ 2.3m depth based on field observations)

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

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## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**Proposed Commercial Development - Huntmar Road**
**Ottawa, Ontario**

**DATUM** Ground surface elevations provided by Stantec Geomatics Ltd.

**FILE NO.** PG3115

**REMARKS**

**HOLE NO.** BH21

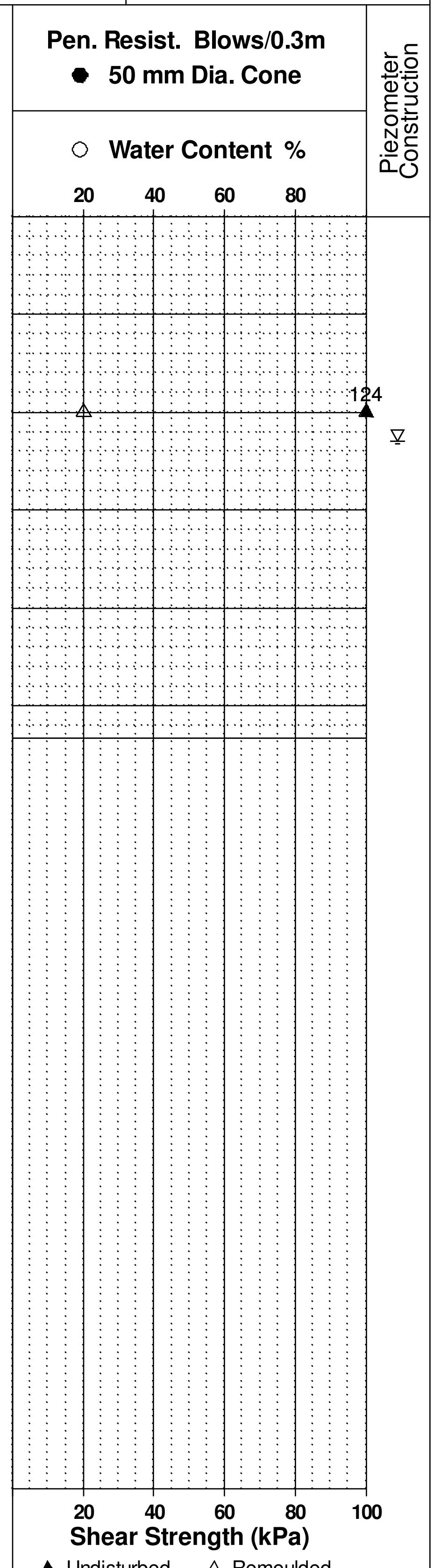
**BORINGS BY** CME 55 Power Auger

**DATE** January 14, 2014

| SOIL DESCRIPTION | TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | 0 | 104.24 |
| **TOPSOIL** 0.30 | AU | 1 | | | | |
| | AU | 2 | | | | |
| Very stiff, brown **SILTY CLAY** with sand seams | SS | 3 | 83 | 5 | 1 | 103.24 |
| 2.13 | | | | | 2 | 102.24 |
| Compact to loose, brown **SILTY FINE SAND** | SS | 4 | 67 | 15 | | |
| - grey-brown by 3.0m depth | SS | 5 | 62 | 9 | 3 | 101.24 |
| | | | | | 4 | 100.24 |
| 5.33 | SS | 6 | 71 | 10 | 5 | 99.24 |
| End of Borehole | | | | | | |

Practical refusal to augering at 5.33m depth

(GWL @ 2.3m depth based on field observations)



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# SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development - Huntmar Road
Ottawa, Ontario

**DATUM** Ground surface elevations provided by Stantec Geomatics Ltd.

**FILE NO.** PG3115

**REMARKS**

**HOLE NO.** BH22

**BORINGS BY** CME 55 Power Auger

**DATE** January 14, 2014

| SOIL DESCRIPTION | Depth to base (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|---|
| GROUND SURFACE | | | | | |
| **TOPSOIL** | 0.30 | AU | 1 | | |
| Very stiff to stiff, brown **SILTY CLAY** | | SS | 2 | 100 | 3 |
| | 3.66 | SS | 3 | 100 | 2 |
| Grey-brown **SILTY FINE SAND** | | SS | 4 | 83 | 2 |
| | | SS | 5 | | 1 |
| | 6.12 | SS | 6 | | 50+ |
| End of Borehole | | | | | |

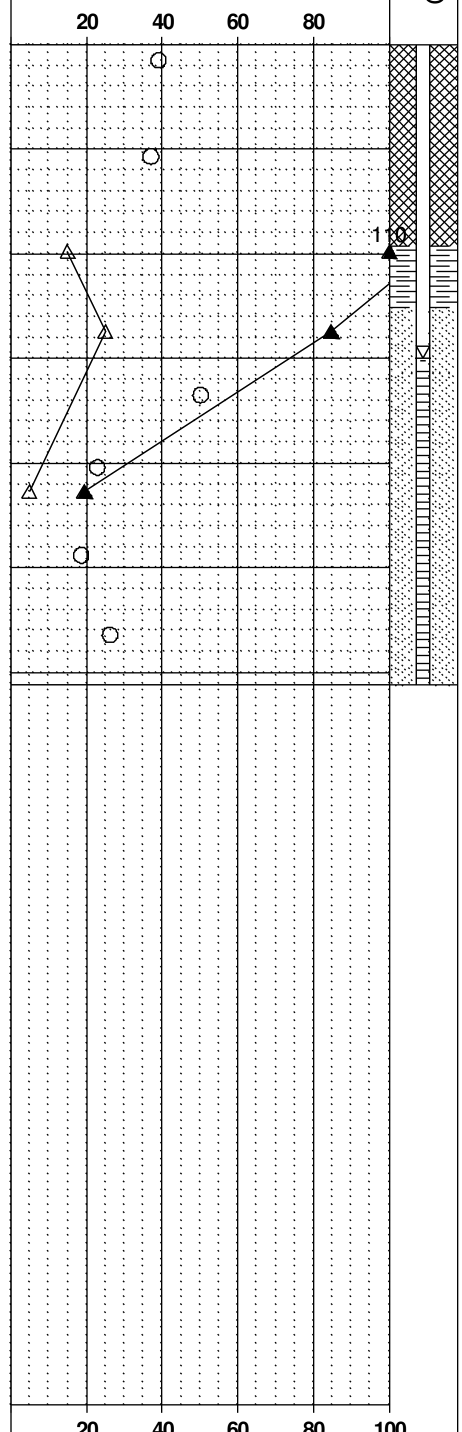
Practical refusal to augering at 6.12m depth

(GWL @ 3.0m depth based on field observations)

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 103.22 |
| 1 | 102.22 |
| 2 | 101.22 |
| 3 | 100.22 |
| 4 | 99.22 |
| 5 | 98.22 |
| 6 | 97.22 |

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

○ Water Content %




Shear Strength (kPa)

▲ Undisturbed △ Remoulded

# 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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

| Compactness Condition | ‘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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

| 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 |

## SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, $S_t$, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

| | |
|---|---|
| Low Sensitivity: | $S_t < 2$ |
| Medium Sensitivity: | $2 < S_t < 4$ |
| Sensitive: | $4 < S_t < 8$ |
| Extra Sensitive: | $8 < S_t < 16$ |
| Quick Clay: | $S_t > 16$ |

## 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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler |
| G | - | "Grab" sample from test pit or surface materials |
| AU | - | Auger sample or bulk sample |
| WS | - | Wash sample |
| RC | - | Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits. |

## SYMBOLS AND TERMS (continued)

## PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

| | | |
|---|---|---|
| WC% | - | Natural water 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 at which xx% of the soil, by weight, is of finer grain sizes<br>These grain size descriptions are not used below 0.075 mm grain size |
| D10 | - | Grain size at which 10% of the soil is finer (effective grain size) |
| D60 | - | Grain size at which 60% of the soil is finer |
| Cc | - | Concavity coefficient = $(D30)^2$ / (D10 x D60) |
| Cu | - | Uniformity coefficient = D60 / D10 |

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

## CONSOLIDATION TEST

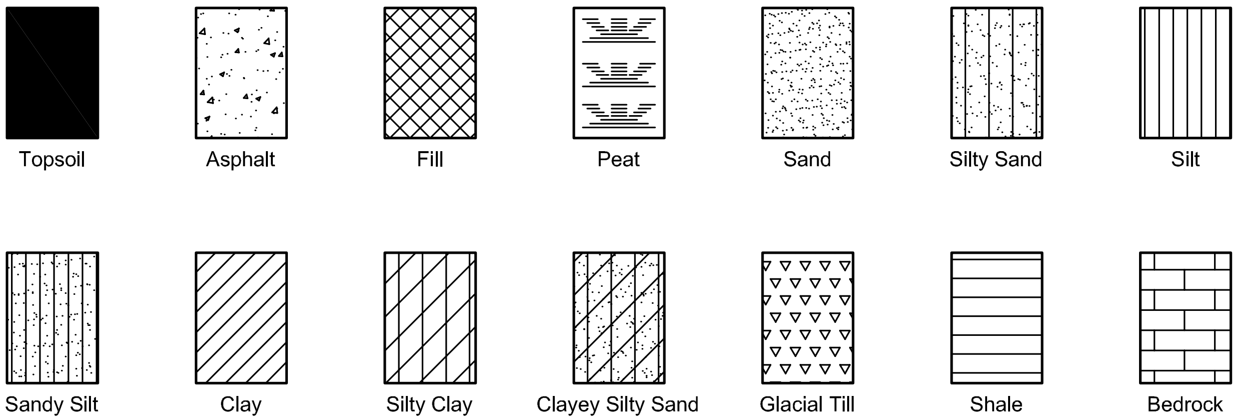
| | | |
|---|---|---|
| $p'_o$ | - | Present effective overburden pressure at sample depth |
| $p'_c$ | - | Preconsolidation pressure of (maximum past pressure on) sample |
| Ccr | - | Recompression index (in effect at pressures below $p'_c$) |
| Cc | - | Compression index (in effect at pressures above $p'_c$) |
| OC Ratio | | Overconsolidaton ratio = $p'_c / p'_o$ |
| Void Ratio | | Initial sample void ratio = volume of voids / volume of solids |
| Wo | - | Initial water content (at start of consolidation test) |

## PERMEABILITY TEST

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

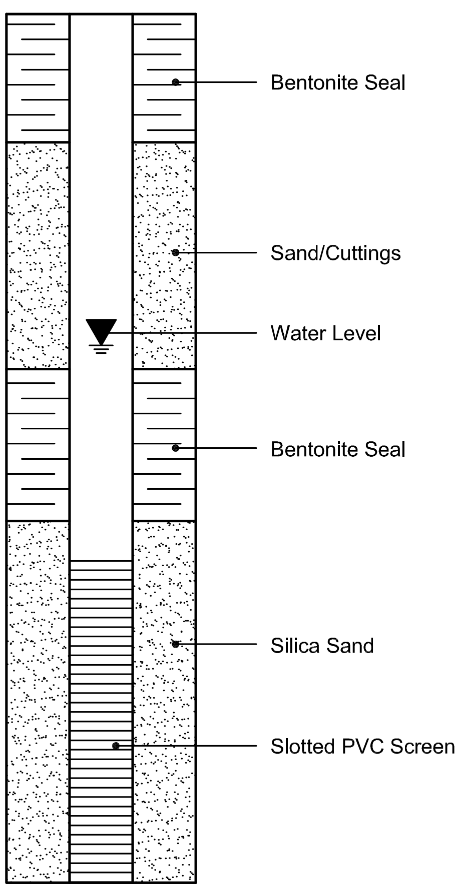
## SYMBOLS AND TERMS (continued)

### STRATA PLOT

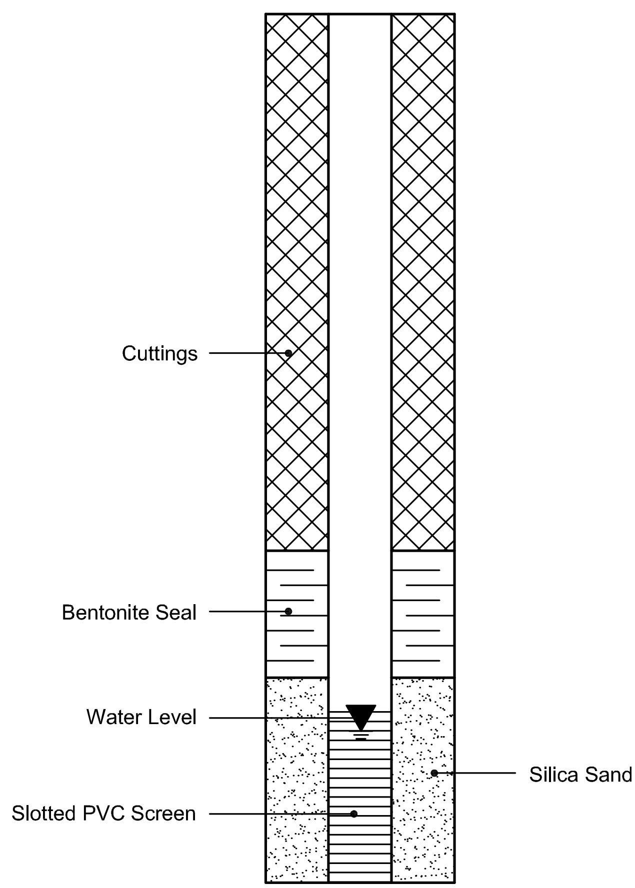



### MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION




PIEZOMETER CONSTRUCTION

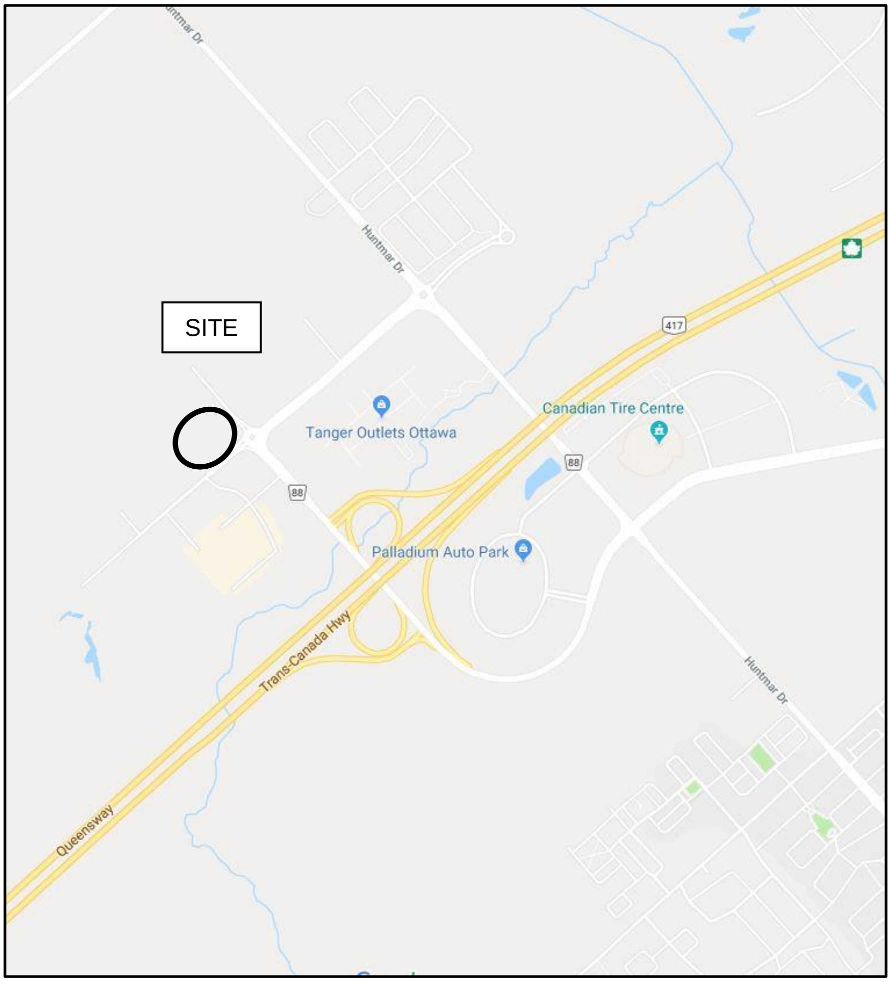


# APPENDIX 2

**FIGURE 1 - KEY PLAN**

**FIGURES 2 AND 3 - SHEAR WAVE VELOCITY PROFILES**

**DRAWING PG3115-7 - TEST HOLE LOCATION PLAN**




# FIGURE 1

## KEY PLAN

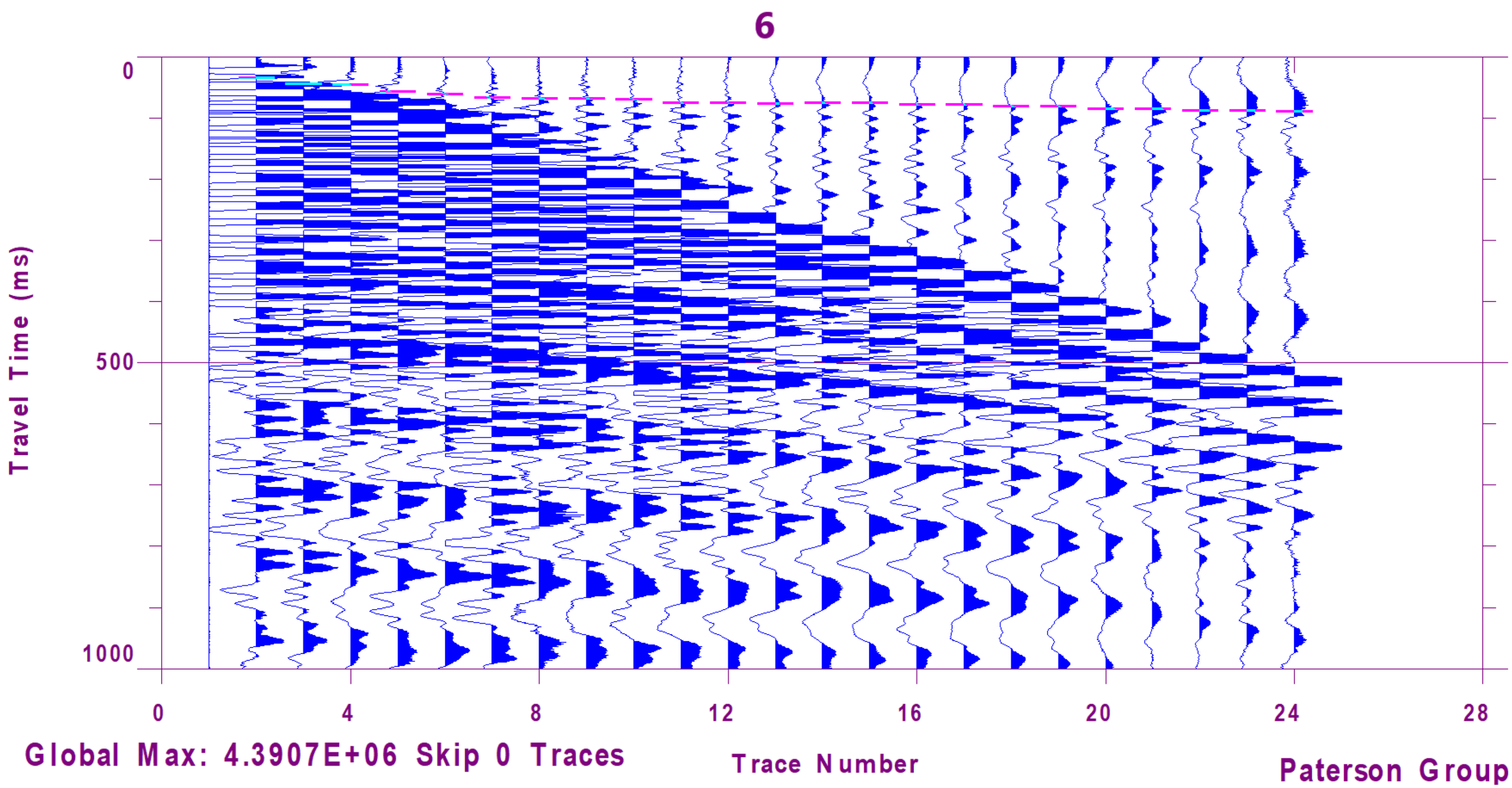



**FIGURE 2 – Shear Wave Velocity Profile at Shot Location -4.5 m**

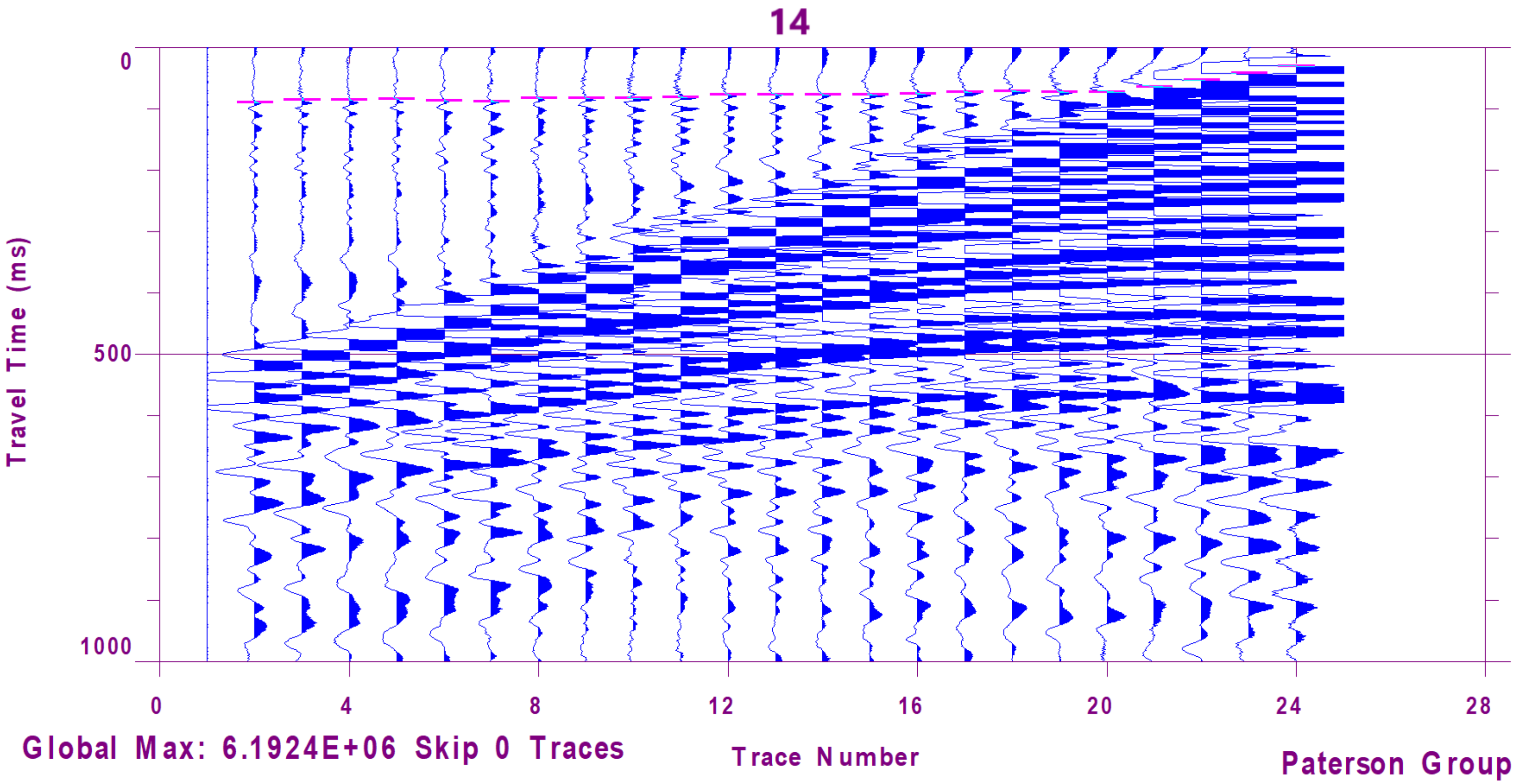
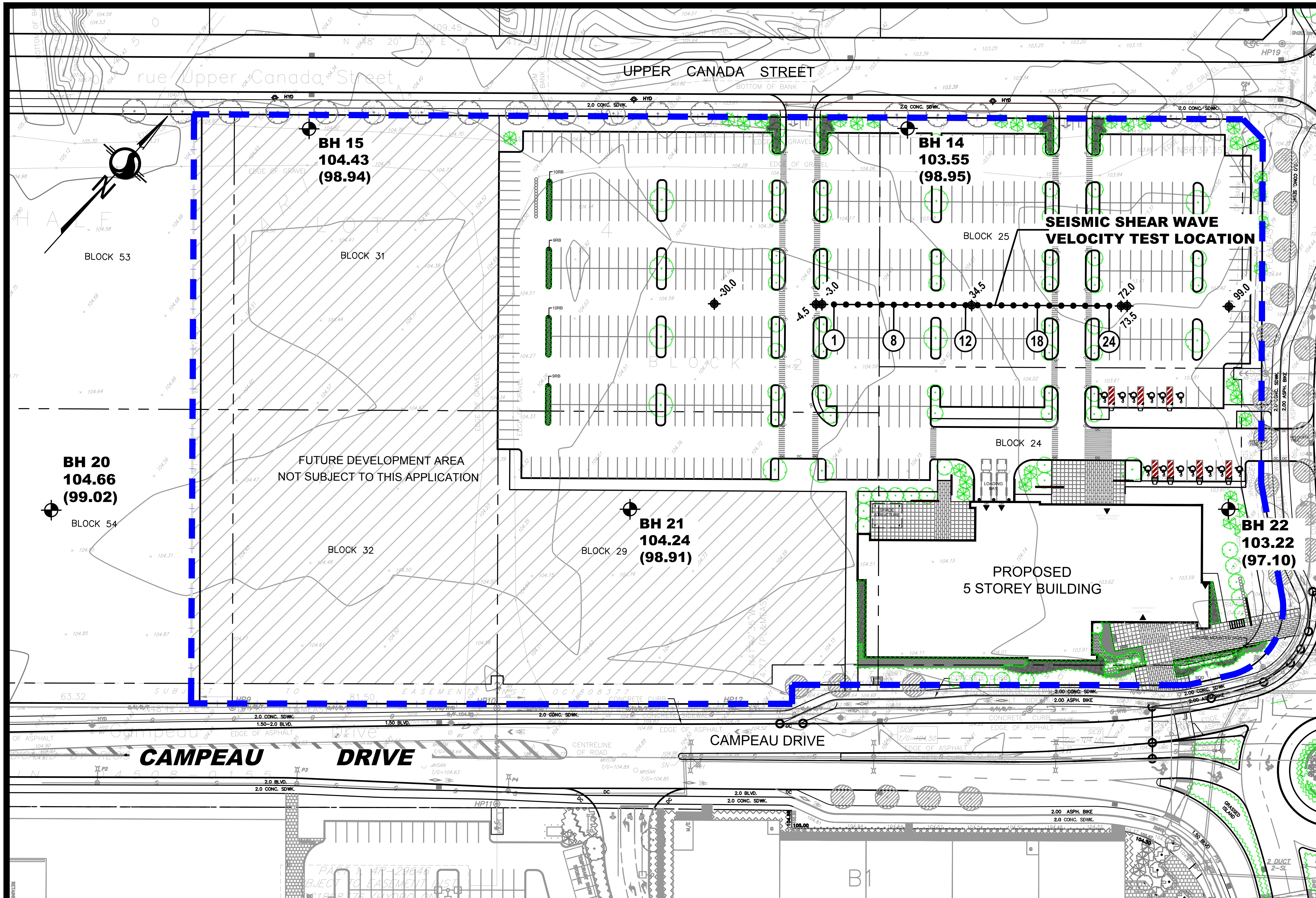



FIGURE 3 – Shear Wave Velocity Profile at Shot Location +73.5 m




LEGEND:

| | |
|---|---|
| ⌖ | BOREHOLE LOCATION |
| 105.99 | GROUND SURFACE ELEVATION (m) |
| (97.38) | PRACTICAL REFUSAL TO DCPT/AUGERING ELEVATION (m) |
| ●—●—● | GEOPHONE LOCATIONS |
| (24) | GEOPHONE NUMBER |
| 3.0 | SHOT LOCATION |

BASE PLAN, TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS AT TEST HOLE LOCATIONS PROVIDED BY STANTEC GEOMATICS LTD.

SCALE: 1:1000

0 10 20 30 40 50m 75m

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| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| 1 | BASE PLAN UPDATED | 09/07/2019 | DJG |

TAGGART REALTY MANAGEMENT

GEOTECHNICAL INVESTIGATION
PROPOSED OFFICE BUILDING - KANATA WEST BUSINESS PARK - CAMPEAU DR.
OTTAWA, ONTARIO

Title:

# TEST HOLE LOCATION PLAN

| | |
|---|---|
| Scale: 1:1000 | Date: 09/2018 |
| Drawn by: RCG | Report No.: PG3115-1 |
| Checked by: CB | Dwg. No.: PG3115-7 |
| Approved by: DJG | Revision No.: 1 |

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