

Geotechnical  
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## Geotechnical Investigation

Proposed Multi-Storey Building  
800 Eagleson Road,  
Ottawa, Ontario

Prepared For

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## Table of Contents

	<b>Page</b>
<b>1.0 Introduction</b>	<b>1</b>
<b>2.0 Proposed Development</b>	<b>1</b>
<b>3.0 Method of Investigation</b>	
3.1 Field Investigation	2
3.2 Field Survey	3
3.3 Laboratory Testing	4
3.4 Analytical Testing	4
<b>4.0 Observations</b>	
4.1 Surface Conditions	5
4.2 Subsurface Profile	5
4.3 Groundwater	8
<b>5.0 Discussion</b>	
5.1 Geotechnical Assessment	9
5.2 Site Grading and Preparation	9
5.3 Foundation Design	10
5.4 Permissible Grade Raise Recommendations	11
5.5 Design for Earthquakes	12
5.6 Basement Slab	14
5.7 Basement Wall	14
5.8 Pavement Structure	15
<b>6.0 Design and Construction Precautions</b>	
6.1 Foundation Drainage and Backfill	17
6.2 Protection of Foundations Against Frost Action	18
6.3 Excavation Side Slopes	19
6.4 Pipe Bedding and Backfill	19
6.5 Groundwater Control	20
6.6 Winter Construction	20
6.7 Corrosion Potential and Sulfate	21
6.8 Landscaping Considerations	21
<b>7.0 Recommendations</b>	<b>23</b>
<b>8.0 Statement of Limitations</b>	<b>24</b>

## Appendices

**Appendix 1** Soil Profile and Test Data Sheets:

BH1-18 to BH3-18

Soil Profile and Test Data Sheets (by others):

BH1 to BH8 (Kollaard Associates - February, 2018)

Symbols and Terms

Unidimensional Consolidation Tests

Analytical Test Results

**Appendix 2** Figure 1 - Key Plan

Figure 2 and 3 - Seismic Shear Wave Velocity Profiles

Drawing PG4692-1 - Test Hole Location Plan

## 1.0 Introduction

Paterson Group (Paterson) was commissioned by Ironclad Developments to conduct a geotechnical investigation for the proposed multi-storey building to be located on the aforementioned site situated at the southwest corner of the intersection of Eagleson Road and Fernbank Road in the City of Ottawa, Ontario (refer to Figure 1, Key Plan, in Appendix 2).

The objectives of the investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of boreholes to supplement the previous geotechnical investigation completed by others.
- ☐ 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

It is our understanding that the proposed development will consist of a 6 storey wood framed residential building with one level of underground parking occupying the majority of the north portion of the site. It is expected that the proposed building will be serviced with municipal water and sewer.

Associated surface car parking areas, access lanes and landscaped areas are also anticipated.

## **3.0 Method of Investigation**

### **3.1 Field Investigation**

#### **Field Program**

The fieldwork program for the current geotechnical investigation was completed on October 9 and 10, 2018. During that time, a total of 3 boreholes (BH1-18, BH2-18 and BH3-18) were drilled and sampled to a maximum depth of 15.5 m below existing ground surface. The boreholes were located in the field by Paterson personnel to provide general coverage of the proposed development taking into consideration existing site features, underground utilities and existing test holes completed during the previous geotechnical investigation. The previous test holes completed by others have been included as part of the current geotechnical investigation. The locations of the test holes are shown on Drawing PG4692-1 - Test Hole Location Plan, included in Appendix 2.

The boreholes were drilled using a track-mounted power auger drill rig, operated by a crew of two. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The drilling procedure consisted of augering to the required depths at the selected locations along with regularly sampling and testing of the overburden soil.

#### **Sampling and In Situ Testing**

Soil samples were recovered from the auger flights, vane blade, and using a 50 mm diameter split-spoon sampler or a thin walled Shelby tube in combination with a fixed piston sampler. The split-spoon samples were placed in sealed plastic bags and the Shelby tubes were sealed at both ends on site. All the samples were transported to our laboratory. The depths at which the auger, split-spoon and Shelby tube samples were recovered from the boreholes are shown as AU, SS and TW, 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.

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

The thickness of the overburden was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at BH2-18. The DCPT consists of driving a steel 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 of depth. Practical refusal to augering on inferred boulder was encountered during the previous geotechnical investigation at BH1 by others.

Bedrock sample was recovered at BH2 during the previous geotechnical investigation to confirm bedrock and assess its quality. The depth at which the rock core sample was recovered from BH2 is shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

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.

### **Groundwater**

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

### **Sample Storage**

All samples from the current investigation 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 ground surface elevations recovered during the current geotechnical investigation was referenced to temporary benchmark (TBM). The TBM consisted of the top of spindle of the fire hydrant located on the south side of Fernbank Road with a geodetic elevation of 96.59 m, as shown on the survey plan prepared by Stantec Geomatics Ltd. The location of the TBM, boreholes and the ground surface elevation at each borehole location are presented on Drawin PG4692-1- Test Hole Location Plan, in Appendix 2.

### **3.3 Laboratory Testing**

All the soil samples recovered from the subject site were visually examined in our laboratory to review the field logs.

A total of 3 representative soil samples recovered using a thin walled Shelby tubes as part of the current geotechnical investigation were submitted for unidimensional consolidation testing. The results of the unidimensional consolidation testing are further discussed in Subsection 5.4 and presented in Appendix 1.

A total of 5 representative soil samples were submitted for Atterberg limits testing and 6 representative soil samples were submitted for grain size distribution analysis (hydrometer testing) as part of the previous geotechnical investigation. The results of the Atterberg Limits Testing are presented in Subsection 4.2 and the results of the Grain Size Distribution analysis (hydrometer testing) is presented in Appendix 1.

### **3.4 Analytical Testing**

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analysed to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are shown in Appendix 1 and are further discussed in Subsection 6.7.

## **4.0 Observations**

### **4.1 Surface Conditions**

Generally, the subject site was observed to be vegetated with grass and sparsely occupied by mature trees and brush. The site was gradually observed to slope down towards the existing Monahan Drain bordering the southwest boundary of the site. Based on the topographic information provided on the survey plan prepared by Stantec Geomatics Ltd., the ground surface at the site was observed to be approximately 1.5 to 2 m below Eagleson Road and approximately 2 to 2.5 m below Fernbank Road.

Based on the information provided by Kollaard Associates as part of the previous geotechnical investigation, the normal water level of the neighbouring Monahan Drain is approximately 1.8 m below the average ground surface elevation of the site. It was further noted that the 1:100 year flood level of the Monahan Drain reported by the City of Ottawa may extend upward to 94.62 m which is approximately 0.4 m above the average existing ground surface of the site.

### **4.2 Subsurface Profile**

Generally, the subsoil conditions encountered at the test hole locations consist of an overlying layer of organic topsoil/peat overlying a relatively deep silty clay deposit extending to depths varying between 25 to 28 m below existing ground surface. A compact to dense glacial till was encountered below the silty clay deposit which in turn was overlying bedrock.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each test hole location.

#### **Organic Topsoil/Peat**

An overlying organic topsoil/peat was encountered at all borehole locations completed during the geotechnical investigations and was observed to vary between 0.3 and 1 m in thickness across the site overlying a very stiff to stiff brown silty clay/clayey silt and/or sandy silt with clay.



## **Clayey Silt/Sandy Silt with clay**

Pockets of a clayey silt/sandy silty with clay was observed at BH1-18 during the current geotechnical investigation and at BH2, BH3 and BH5 completed during the previous investigation underlying the organic topsoil/peat. The clayey silt/sandy silt with clay was observed to decrease in silt content at depth and extended to a maximum depth of 1.5 m below existing ground surface. Based on the results of the Standard Penetration Test (SPT) conducted in conjunction with the recovery of the split-spoon samples, the material is considered to be in a loose to stiff state.

## **Silty Clay**

The upper portion of the relatively deep silty clay deposit encountered at the borehole locations during both geotechnical investigations was observed to be weathered to a brown silty clay crust. The brown silty clay crust at the borehole locations extended between 1.5 and 2.1 m below existing ground surface. Based on the results of the Standard Penetration Test (SPT) conducted in conjunction with the recovery of the split-spoon samples and undrained shear strength testing, the weathered brown silty clay crust was evaluated to be of a very stiff to stiff state.

The grey silty clay located within the long term groundwater table encountered below the weathered brown silty clay crust was determined to extend approximately 25 to 28 m below existing ground surface. The strength of the underlying grey silty clay deposit was evaluated during the course of the field portion of the geotechnical investigation by conducting undrained shear strength testing at regular intervals throughout the silty clay deposit. The undrained shear strengths recovered during the current geotechnical investigation generally varied between 25 and 60 kPa which is indicative of a firm to stiff silty clay.

<b>Table 1 - Summary of Atterberg Limits Tests (by others)</b>					
<b>Sample</b>	<b>Moisture Content %</b>	<b>Liquid Limit %</b>	<b>Plastic Limit %</b>	<b>Plasticity Index %</b>	<b>Classification</b>
BH 1 - SS3	28.0	32	17	16	CL
BH 1 - SS10	46.5	36	23	13	CL
BH 2 - SS8	36.3	25	16	9	CI
BH 5 - SS6	36.6	32	20	12	CL
BH 7 - SS2	26.4	32	17	14	CL

## **Glacial Till**

A glacial till deposit was encountered directly below the silty clay deposit at BH1 and BH 2 during the previous geotechnical investigation. The glacial till was evaluated to consist of a mixture of grey silty clay with sand and gravel with occasional cobbles and boulders. The SPT results recovered in conjunction with the recovery of the split-spoon samples ranged from 7 to 29 blows per 0.3 m with an average value of 18 blow per 0.3 m which is indicative of a compact state.

One representative soil sample recovered from the glacial till deposit was submitted for Grain Size Distribution Analysis as part of the previous geotechnical investigation and presented in Appendix 1.

## **Bedrock**

The thickness of the overburden was evaluated during the course of the current investigation by a dynamic cone penetration test (DCPT) at BH2-18 which was terminated to practical refusal at a depth of 25 m below existing ground surface. It is suspected that the dynamic cone penetration test terminate terminated at practical refusal on inferred boulders within the compact to dense glacial till deposit.

Bedrock sample was recovered from a depth of 33 to 34 m below existing ground surface at BH2 during the previous geotechnical investigation to confirm bedrock and assess its quality. A recovery value and a Rock Quality Designation (RQD) value was calculated for the drilled section (core run) of bedrock. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. Based on the results presented in the previous geotechnical investigation, the bedrock is indicative of a sound quality bedrock.

Based on available geological mapping, the bedrock in this area consists of interbedded limestone and dolomite of the Gull River formation and it is expected to be encountered at depths ranging from 15 to 50 m.

### 4.3 Groundwater

Groundwater levels were measured in the standpipes on October 18, 2018 and are summarized in Table 2 - Summary of Groundwater Levels. It is important to note that groundwater readings at piezometers can be influenced by surface water perched within the borehole backfill material. Groundwater conditions can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, it is estimated that groundwater can be expected between elevation of 91.5 and 92.5 m at depths between 1 to 2 m below existing ground surface.

<b>Table 2 - Summary of Groundwater Level Readings</b>				
<b>Borehole Number</b>	<b>Ground Elevation (m)</b>	<b>Groundwater Levels (m)</b>		<b>Recording Date</b>
		<b>Depth</b>	<b>Elevation</b>	
BH 1-18	93.87	5.18	88.69	October 18, 2018
BH 2-18	93.35	0.57	92.78	October 18, 2018
BH 3-18	93.99	4.81	89.18	October 18, 2018
<b>Note:</b> The ground surface elevations completed during the current geotechnical investigation was referenced to temporary benchmark (TBM). The TBM consisted of the top of spindle of the fire hydrant located on the south side of Fernbank Road with a geodetic elevation of 96.59 m, as shown on the survey plan prepared by Stantec Geomatics Ltd..				

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could be different at the time of construction.

## **5.0 Discussion**

### **5.1 Geotechnical Assessment**

The subject site is underlain by a deep compressible sensitive silty clay deposit. From a geotechnical perspective, the subject site is suitable for the proposed development provided the foundation is designed to ensure the long term total and differential settlements will be within tolerable ranges.

Based on recent discussions with the design team, it's expected that the building can be founded on conventional shallow footings placed at an elevation of approximately 93.0 to 93.5 m within the upper portion of the weathered stiff silty clay crust. Perimeter grades exceeding permissible grade raises will require light weight fill (LWF) in the design to lower the stresses for the perimeter of the raft slab.

The proposed building excavation will be mainly through clayey silty to silty clay. It is expected that unsupported excavations may not be possible along the north property boundary as a result of the depth and proximity to the property boundary.

Due to the presence of a layer of sensitive silty clay, the subject site will be subjected to permissible grade raise restrictions.

The above and other considerations are further discussed in the remainder of this report.

### **5.2 Site Grading and Preparation**

#### **Stripping Depth**

Topsoil, organic matter and existing fill should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

#### **Fill Placement**

Fill used for grading under the base and subbase layers of paved areas should consist of clean imported select subgrade material. 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 paved areas should be compacted to at least 95% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to reduce 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.

## **5.3 Foundation Design**

### **Conventional Spread Footings**

Strip footings, up to 3.5 m wide, and pad footings, up to 6.5 m wide, placed over an undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **105 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **175 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance values at ULS.

### **Settlement Considerations**

The bearing resistance values were provided to ensure that the anticipated post-construction total and differential settlements of the foundation supporting the building are within acceptable range for the structure. The total and differential settlements at the building will be limited to 25 and 20 mm, respectively.

### **Other Foundation Design Considerations**

The potential post construction total and differential settlements are dependent on the position of the long term groundwater level when building over thick 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.

## **5.4 Permissible Grade Raise Recommendations**

Consideration must be given to potential settlements which could occur due to the presence of the silty clay deposit and the combined loads from the proposed footings, any groundwater lowering effects, and grade raise fill.

## Consolidation Testing

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

Value  $\sigma'_p$  is the preconsolidation pressure of the sample and  $\sigma'_{vo}$  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  $C_r$  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  $C_r$ , illustrate the increased settlement potential above, as compared to below, the preconsolidation pressure.

It should be noted that the values of  $\sigma'_p$ ,  $\sigma'_{vo}$ ,  $C_r$  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  $\sigma'_{vo}$  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  $\sigma'_{vo}$  and therefore reduces the available preconsolidation. Unacceptable settlements could be induced by a significant lowering of the groundwater level.

<b>Table 4 - Summary of Consolidation Test Results</b>							
<b>Borehole No.</b>	<b>Sample</b>	<b>Sample Depth (m)</b>	<b><math>\sigma'_p</math> (kPa)</b>	<b><math>\sigma'_{vo}</math> (kPa)</b>	<b><math>C_r</math></b>	<b><math>C_c</math></b>	<b>Q</b>
BH1-18	TW4	5.82	114	47	0.026	0.917	A
BH1-18	TW5	6.58	87	52	0.024	0.593	P
BH3-18	TW4	6.58	110	76	0.018	0.906	A
* - Q - Quality assessment of sample - G: Good      A: Acceptable      P: Likely disturbed							

## **Permissible Grade Raise**

Permissible grade raise recommendations have been determined for the proposed development based on the consolidation testing results of samples of the silty clay obtained during the geotechnical investigation. Based on our findings, a permissible grade raise of **1.0 m** is recommended for grading at the site within 4.8 m of the proposed building.

In soft and hard landscaped areas and asphalt covered parking areas, some minor settlement can be tolerated greater than 4.8 m from the proposed building. For these areas, the permissible grade raise can be increased to **1.4 m**.

## **5.5 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 in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the seismic shear wave interpretation are presented in Appendix 2.

### **Field Program**

The shear wave velocity testing array was placed across the site, oriented approximately southwest-northeast as shown on Drawing PG4692 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the ground surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 3 m intervals and connected by a geophone spread cable to a Geode 24 channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-beam seated into the ground surface parallel to the geophone array, which creates a polarized shear wave. The hammer shots are repeated 4 to 8 times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e. striking both sides of the I-beam). The shot locations are located 3, 4.5 and 14.5 m from the first geophone, 3, 4.5 and 7.5 m from the last geophone, and at the centre of the seismic array.

## Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is repeated at each shot location to provide an average shear wave velocity,  $V_{s30}$ , 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 due to the increasing quality of bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on the test results, the average silty clay overburden shear wave velocity is **151 m/s**, the average glacial till overburden shear wave velocity is **319 m/s** and the bedrock shear wave velocity is **1,500 m/s**. It is understood that the currently proposed design includes one level of underground parking. The analysis details are presented below.

The  $V_{s30}$  was calculated using the standard equation for the average shear wave velocity calculation from the OBC 2012. Assuming the bottom of building foundation is placed at 91 m, the following equation applies:

$$V_{s30} = \frac{Depth_{OfInterest} (m)}{\left( \frac{Depth_{Layer1} (m)}{Vs_{Layer1} (m/s)} + \frac{Depth_{Layer2} (m)}{Vs_{Layer2} (m/s)} + \frac{Depth_{Layer3} (m)}{Vs_{Layer3} (m/s)} \right)}$$

$$V_{s30} = \frac{30m}{\left( \frac{20.5m}{151m/s} + \frac{8m}{319m/s} + \frac{1.5m}{1500m/s} \right)}$$

$$V_{s30} = 185m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity,  $V_{s30}$  for a raft foundation at the subject site is 185 m/s. Therefore, a **Site Class D** is applicable for design of the proposed building.

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



## 5.6 Basement Slab

### Expected Subgrade

With the removal of all topsoil, organic matter, and fill within the footprint of the proposed building, the native silty clay surface will be considered to be an acceptable subgrade surface on which to construct the basement floor slab.

### Concrete Mud Slab for Protection of Subgrade

The subgrade material will consist of sensitive silty clay. It's suggested that a 100 mm thick concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment and to prevent dewatering of the surrounding areas. The concrete mud slab should be placed in sections as the excavation is being completed to avoid exposing large areas of the silty clay to potential disturbance and drying.

## 5.7 Basement Wall

The basement walls may be designed using a triangular earth pressure distribution with a maximum stress value at the base of the wall equal to  $K \gamma H$  where:

- $K = 0.33$  - where slight movement is permissible (1 to 4% of  $H$ )
- $K = 0.5$  - where no movement is permissible (normal basement wall)
- $\gamma = 20 \text{ kN/m}^3$ , unit weight of the soil above the groundwater level  
 $10.8 \text{ kN/m}^3$ , effective unit weight below the groundwater level
- $H =$  height of the basement wall, m

An additional pressure having a magnitude equal to  $K q$  and acting on the entire height of the wall should be added to the above diagram for any surcharge loading,  $q$ , that may be placed at ground surface adjacent to the wall. Hydrostatic water pressures should be applied for waterproofed foundation walls.

It is recommended that the foundation walls be waterproofed in order to allow for the groundwater to be used to contribute to the compensation of the foundation weight. A foundation drainage system can be provided at elevation 94.62 m. The level of the waterproofing should extend a minimum of 0.5 m above the level of the drainage system (i.e. el. 95.12 m).

## 5.8 Pavement Structure

The recommended pavement structures for the subject site are shown in Tables 5 and 6.

<b>Table 5 - Recommended Pavement Structure Car Only Parking Areas</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
50	<b>Wear Course</b> - Superpave 12.5 Asphaltic Concrete
150	<b>BASE</b> - OPSS Granular A Crushed Stone
300	<b>SUBBASE</b> - OPSS Granular B Type II
<b>SUBGRADE</b> - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

<b>Table 6 - Recommended Pavement Structure Access Lanes and Heavy Truck Parking Areas</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
40	<b>Wear Course</b> - Superpave 12.5 Asphaltic Concrete
50	<b>Binder Course</b> - Superpave 19.0 Asphaltic Concrete
150	<b>BASE</b> - OPSS Granular A Crushed Stone
450	<b>SUBBASE</b> - OPSS Granular B Type II
<b>SUBGRADE</b> - 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 sub-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 100% of their SPMDD with 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 and promote good subbase drainage.

## **6.0 Design and Construction Precautions**

### **6.1 Foundation Drainage and Backfill**

#### **Foundation Drainage**

Since the basement level of the proposed building will be well above the long-term groundwater level, it's recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 150 mm diameter, geotextile-wrapped, perforated and corrugated plastic pipe surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. It's expected that the exterior perimeter drainage system will drain by gravity to the storm sewer. If gravity flow is not possible, it's recommended that 150 mm diameter sleeves at approximately 10 m spacing on centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the basement area.

#### **Underfloor Drainage**

For design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at each bay. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

#### **Foundation Backfill**

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

It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the foundation drainage level, and be hydraulically connected to the perimeter drainage pipe. It is our understanding that the perimeter drainage system can have gravity drainage to the storm sewer system.

## **6.2 Protection of Foundations Against Frost Action**

Perimeter foundations of heated structures are required to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

Exterior unheated foundations, such as those for unheated sunken terrace areas over the raft extensions and/or isolated exterior piers, do not benefit from heat loss from the heated areas of the building and require additional protection, such as a soil cover of 2.1 m or combination of soil cover and foundation insulation.

## **6.3 Excavation Side Slopes**

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

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

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

## **6.4 Pipe Bedding and Backfill**

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. Where the invert of the excavation is into the grey silty clay, as is expected, the thickness of the bedding should be increased to 300 mm. 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. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

The surficial sand material should be suitable to use for backfill if it is not saturated. The wet grey silty clay soil will be difficult to re-use as its high water content will make compacting this materials impractical without an extensive drying period.

Where hard surfaces 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, seepage barriers (clay dykes) should be provided at the property limits and at no greater than 60 m intervals in the service trenches where the excavation is below the groundwater level. The barriers should be at least 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. Generally, the dykes 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 300 mm thick loose layers compacted to a minimum of 95% of the material's SPMDD.

## **6.5 Groundwater Control**

### **Groundwater Control for Building Construction**

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.

Infiltration levels are anticipated to generally be low through the excavation face, although seepage is to be expected from within the silty sand, perched over the silty clay. The groundwater infiltration should be controllable with open sumps and pumps.

## **Permit to Take Water**

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 Category 3 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.

## **Impacts on Neighbouring Structures**

Based on our observations, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal temporary groundwater lowering. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

## **6.6 Winter Construction**

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

The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. As well, pavement construction is difficult during winter. Additional information can be provided, if required.

## **6.7 Corrosion Potential and Sulphate**

The test results show that the sulphate content is less than 0.1%. This result is indicative that GU (general use) Portland cement, formerly Type 10 cement, would be appropriate for buried concrete structures at this site.

The chloride content and the pH of the tested samples indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive to very aggressive corrosive environment.

## **6.8 Landscaping Considerations**

### **Tree Planting Restrictions and Tree Planting Setbacks**

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. The results of our testing are presented in Table 1 in Subsection 4.1 and in Appendix 1.

Based on the results of our review, the site is located in a low/medium sensitivity clay soil according to the City of Ottawa Tree Planting in Sensitive Marine Clay Soils.



### **Low/Medium Sensitivity Clay Soil**

A low to medium sensitivity clay soil was encountered between design underside of footing elevations and 3.5 m below finished grade as per City Guidelines. Based on our Atterberg Limits test results, the modified plasticity limit generally does not exceed 40% and the following tree setbacks are recommended. Large trees (mature height over 14 m) can be planted provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits may be reduced to 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m).

## 7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation services/field review program, including the following aspects, be performed by the geotechnical consultant.

- ☐ Grading plan review from a geotechnical perspective, once the final grading plan is available.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Address LWF requirements and observation of the placement of LWF materials during construction.
- ☐ Sampling and testing of the concrete and fill materials used.
- ☐ Observation of the installation of waterproofing materials.
- ☐ 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 and follow-up field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

## 8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project.

A geotechnical investigation 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. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests notification immediately in order to permit reassessment of the 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) or entities other than Ironclad Developments, 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.



Richard Groniger, C. Tech.



Carlos P. Da Silva, P.Eng., ing., QP<sub>ESA</sub>



### Report Distribution:

- ☐ Ironclad Developments Inc. (3 copies)
- ☐ Paterson Group Inc. (1 copy)

# **APPENDIX 1**

**SOIL PROFILE AND TEST DATA SHEETS**

**SOIL PROFILE AND TEST DATA SHEETS (by others)**

**SYMBOLS AND TERMS**

**GRAIN SIZE DISTRIBUTION ANALYSIS (by others)**

**ANALYTICAL TESTING RESULTS**

**DATUM** TBM - Top spindle of fire hydrant. Geodetic elevation = 96.59m, as per survey plan prepared by Stantec Geomatics Ltd.

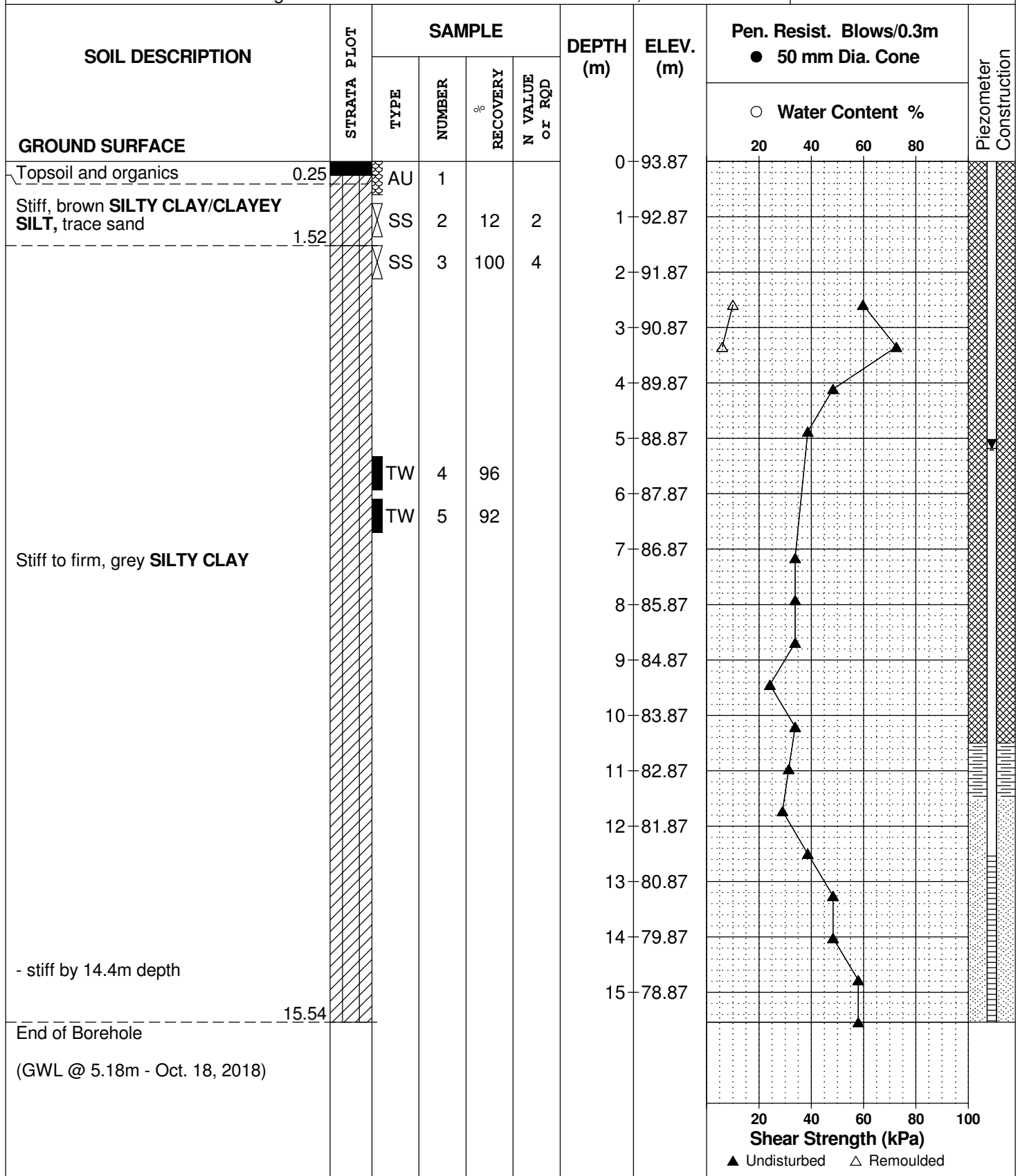
**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** October 9, 2018

**FILE NO.**  
**PG4692**

**HOLE NO.**  
**BH 1-18**



## SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Prop. Residential Development - 800 Eagleson Road  
Ottawa, Ontario

**DATUM** TBM - Top spindle of fire hydrant. Geodetic elevation = 96.59m, as per survey plan prepared by Stantec Geomatics Ltd.

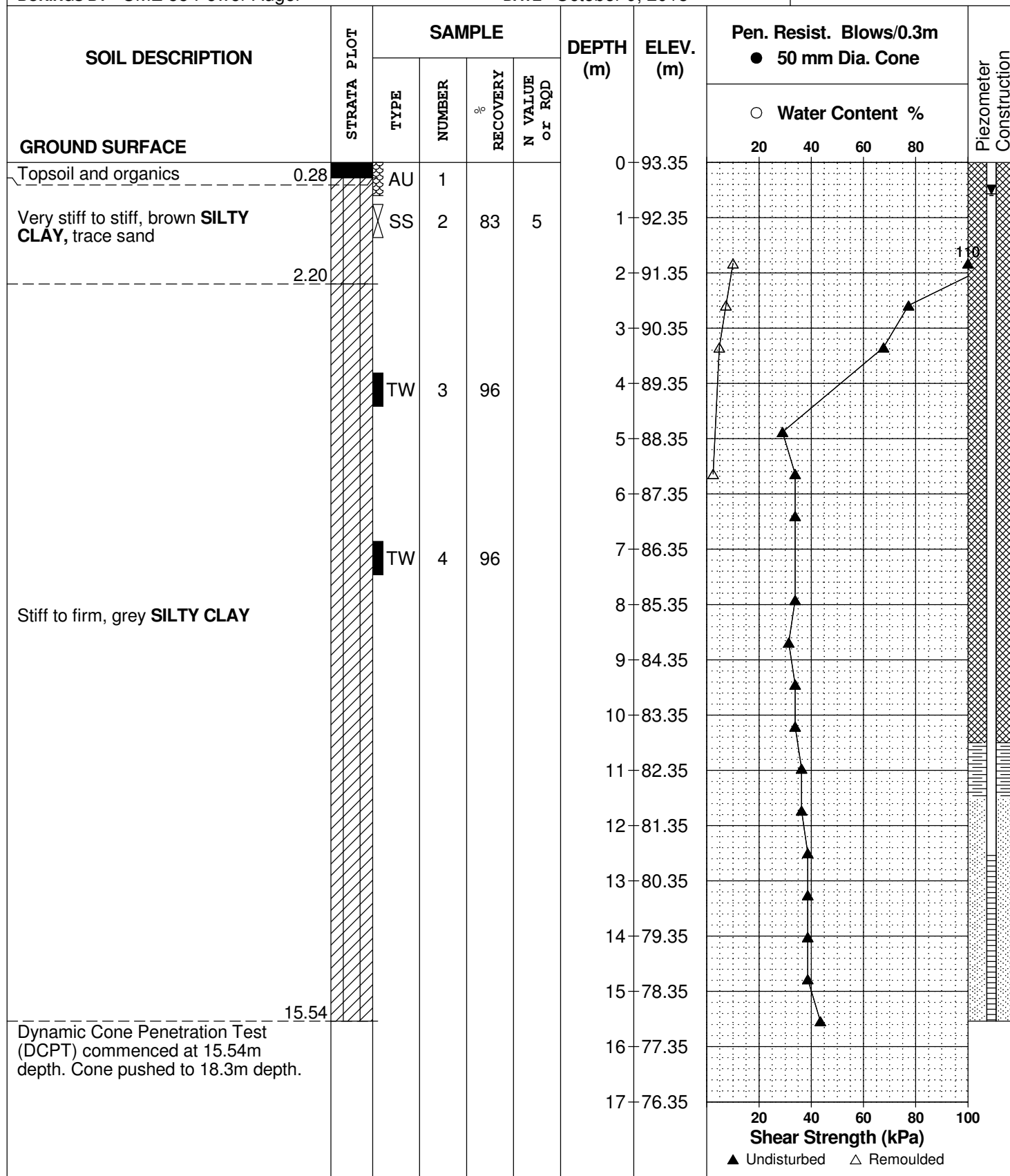
**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** October 9, 2018

**FILE NO.**  
**PG4692**

**HOLE NO.**  
**BH 2-18**



## SOIL PROFILE AND TEST DATA

Geotechnical Investigation  
Prop. Residential Development - 800 Eagleson Road  
Ottawa, Ontario

**DATUM** TBM - Top spindle of fire hydrant. Geodetic elevation = 96.59m, as per survey plan prepared by Stantec Geomatics Ltd.

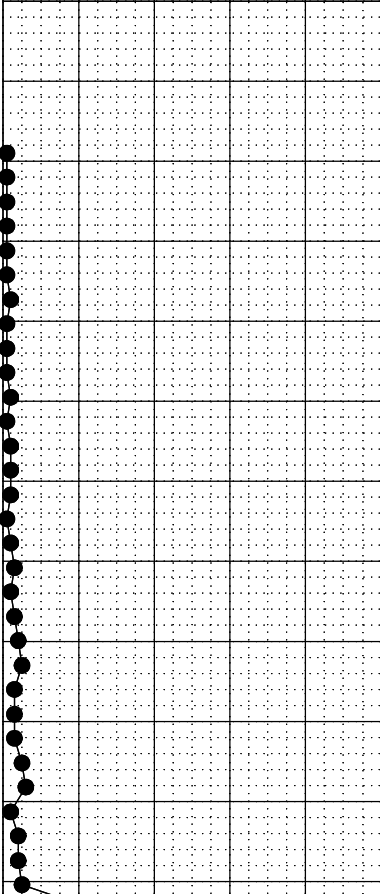
**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** October 9, 2018

**FILE NO.**  
**PG4692**

**HOLE NO.**  
**BH 2-18**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						17	76.35					
End of Borehole  Practical DCPT refusal at 28.40m depth  (GWL @ 0.57m - Oct. 18, 2018)						18	75.35					
						19	74.35					
						20	73.35					
						21	72.35					
						22	71.35					
						23	70.35					
						24	69.35					
						25	68.35					
						26	67.35					
						27	66.35					
						28	65.35					

**DATUM** TBM - Top spindle of fire hydrant. Geodetic elevation = 96.59m, as per survey plan prepared by Stantec Geomatics Ltd.

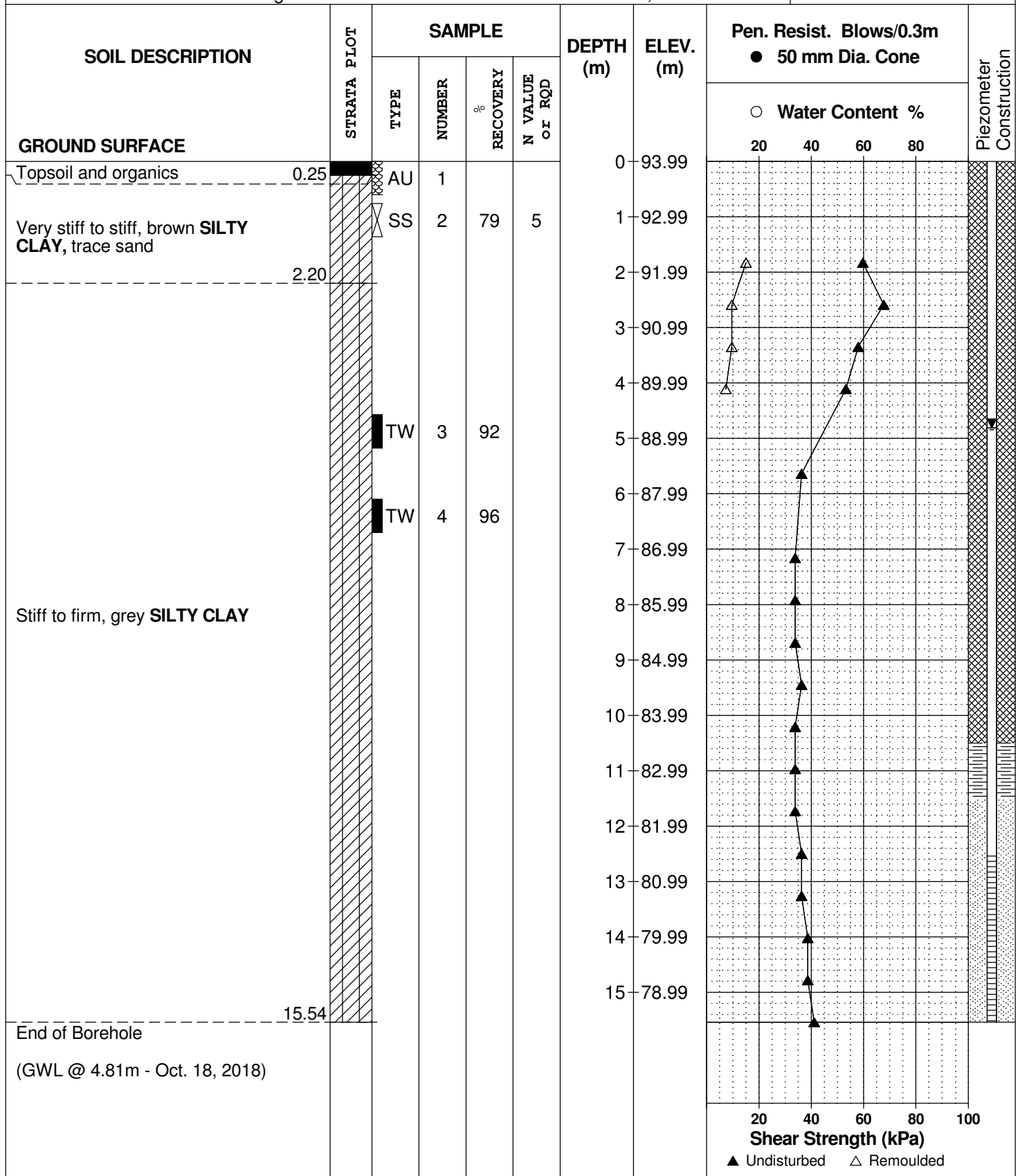
**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** October 10, 2018

**FILE NO.**  
**PG4692**

**HOLE NO.**  
**BH 3-18**





# RECORD OF BOREHOLE BH1

**PROJECT:** Proposed Residential Development

**CLIENT:** Ironclad Developments Inc.

**LOCATION:** 800 Eagleson Road

**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 180084

**DATE OF BORING:** February 13, 2018

**SHEET** 1 of 2

**DATUM:**

DEPTH SCALE (meters)	SOIL PROFILE			SAMPLES			UNDIST. SHEAR STRENGTH				DYNAMIC CONE PENETRATION TEST					ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	Cu, kPa				blows/300 mm						
							REM. SHEAR STRENGTH										
							Cu, kPa										
0	Ground Surface		94.30												MC%		
	Black, Peaty TOPSOIL		0.00	1	SS	4									27	<div><div></div></div> <p>Water observed in borehole at approximately 2.1 metres below the existing ground surface on February 13, 2018.</p>	
			93.70														
1	Stiff grey brown SILTY CLAY, trace sand		0.60	2	SS	3											
	Firm to soft grey SILTY CLAY		93.24														
			1.06														
2				3	SS	10											
3				4	SS	9									31		
4																	
5				5	SS	4									40		
6																	
7				6	SS	2									44		
8																	
9				7	SS	WH									39		
10																	
11																	
12																	
13				9	SS	WH									37		
14																	
15																	
16				10	SS	WH											
17																	

**DEPTH SCALE:** 1 to 55

**BORING METHOD:** Power Auger

**AUGER TYPE:** 200 mm Hollow Stem

**LOGGED:** DT

**CHECKED:** SD

Water observed in borehole at approximately 2.1 metres below the existing ground surface on February 13, 2018.

RECORD OF BOREHOLE BH1

**PROJECT:** Proposed Residential Development

**CLIENT:** Ironclad Developments Inc.

**LOCATION:** 800 Eagleson Road

**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 180084

**DATE OF BORING:** February 13, 2018

**SHEET** 2 of 2

**DATUM:**

DEPTH SCALE (meters)	SOIL PROFILE		SAMPLES				UNDIST. SHEAR STRENGTH				DYNAMIC CONE PENETRATION TEST					ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	Cu, kPa				blows/300 mm						
							REM. SHEAR STRENGTH										
							Cu, kPa										
18																	
19				11	SS	1									56		
20																	
21																	
22				12	SS	4									54		
23																	
24																	
25			69.16														
25	Grey silty clay, trace sand and gravel, becoming bouldery with depth (GLACIAL TILL)		25.14	13	SS	7									15		
26																	
27	Practical refusal to augering in Glacial Till. Advanced borehole by coring through cobbles and boulders		67.61														
28			26.69														
29	End of Borehole		65.45														
29			28.85														
30																	
31																	
32																	
33																	
34																	
35																	

DEPTH SCALE: 1 to 55

BORING METHOD: Power Auger

AUGER TYPE: 200 mm Hollow Stem












































LOGGED: DT

CHECKED: SD

# RECORD OF BOREHOLE BH2

**PROJECT:** Proposed Residential Development  
**CLIENT:** Ironclad Developments Inc.  
**LOCATION:** 800 Eagleson Road  
**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 180084  
**DATE OF BORING:** February 13, 2018  
**SHEET** 1 of 2  
**DATUM:**

DEPTH SCALE (meters)	SOIL PROFILE			SAMPLES			UNDIST. SHEAR STRENGTH				DYNAMIC CONE PENETRATION TEST					ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	Cu, kPa				blows/300 mm						
							REM. SHEAR STRENGTH										
							Cu, kPa										
0	Ground Surface		93.90												MC%		
	Black, Peaty TOPSOIL		0.00	1	SS	6											
			93.30												32		
	Grey brown SILTY SAND, some clay		0.60														
1			92.79	2	SS	5											
	Stiff grey brown SILTY CLAY, trace sand		1.11														
			92.35														
	Firm to soft grey SILTY CLAY		1.55	3	SS	10											
2																	
																	
3																	
																	
4				4	SS	WH									47		
																	
5																	
				5	SS	WH									44		
6																	
																	
7																	
																	
8																	
				7	SS	WH									45		
9																	
																	
10																	
																	
11																	
																	
12																	
																	
13																	
																	
14																	
																	
15																	
																	
16																	
																	
17																	
																	
																	
																	
																	
																	
																	
																	
																	

DEPTH SCALE: 1 to 55

BORING METHOD: Power Auger

AUGER TYPE: 200 mm Hollow Stem

LOGGED: DT

CHECKED: SD

Water observed in borehole at approximately 1.5 metres below the existing ground surface on February 13, 2018.

# RECORD OF BOREHOLE BH2

**PROJECT:** Proposed Residential Development  
**CLIENT:** Ironclad Developments Inc.  
**LOCATION:** 800 Eagleson Road  
**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 180084  
**DATE OF BORING:** February 13, 2018  
**SHEET** 2 of 2  
**DATUM:**

DEPTH SCALE (meters)	SOIL PROFILE		SAMPLES			UNDIST. SHEAR STRENGTH				DYNAMIC CONE PENETRATION TEST					ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH  (M)	NUMBER	TYPE	BLOWS/0.3m	Cu, kPa				blows/300 mm						
							REM. SHEAR STRENGTH										
							20	40	60	80	10	30	50	70			90
18																	
19				11	SS	2									58		
20																	
21																	
22				12	SS	1									35		
23																	
24	Grey silty clay, trace sand and gravel, becoming bouldery with depth (GLACIAL TILL)		69.82														
25			24.08	13	SS	13									15		
26																	
27																	
28				14	SS	29									12		
29																	
30																	
31																	
32	Advanced corehole through BEDROCK		61.75														
33			32.15	15	RC												
34	End of Borehole		59.61														
35			34.29														

**DEPTH SCALE:** 1 to 55

**BORING METHOD:** Power Auger

**AUGER TYPE:** 200 mm Hollow Stem

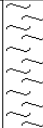


**LOGGED:** DT

**CHECKED:** SD

# RECORD OF BOREHOLE BH3

**PROJECT:** Proposed Residential Development  
**CLIENT:** Ironclad Developments Inc.  
**LOCATION:** 800 Eagleson Road  
**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 180084  
**DATE OF BORING:** February 16, 2018  
**SHEET** 1 of 1  
**DATUM:**

DEPTH SCALE (meters)	SOIL PROFILE			SAMPLES			UNDIST. SHEAR STRENGTH				DYNAMIC CONE PENETRATION TEST					ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	Cu, kPa				blows/300 mm								
							×	20	40	60	80	×	10	30	50			70	90
							REM. SHEAR STRENGTH												
						○	20	40	60	80	○								
0	Ground Surface		93.80													MC%			
	Black, Peaty TOPSOIL		0.00	1	SS	2													
1			92.74	2	SS	2													
	Grey brown SILTY SAND, trace clay		1.06																
	Grey SILTY CLAY			3	SS	3										35			
2				4	SS	9										24			
				5	SS	6										28			
3	End of Borehole		90.75																
			3.05																
4																			
5																			
6																			
7																			
8																			
9																			

Borehole dry,  
February 16,  
2018.

**DEPTH SCALE:** 1 to 55

**BORING METHOD:** Power Auger

**AUGER TYPE:** 200 mm Hollow Stem

**LOGGED:** DT

**CHECKED:** SD

RECORD OF BOREHOLE BH4

PROJECT: Proposed Residential Development

CLIENT: Ironclad Developments Inc.

LOCATION: 800 Eagleson Road

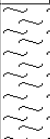

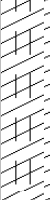




PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180084

DATE OF BORING: February 16, 2018

SHEET 1 of 1

DATUM:

DEPTH SCALE (meters)	SOIL PROFILE			SAMPLES			UNDIST. SHEAR STRENGTH					DYNAMIC CONE PENETRATION TEST					ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	Cu, kPa					blows/300 mm						
							REM. SHEAR STRENGTH											
							Cu, kPa											
0	Ground Surface		93.90														MC%	
	Black, Peaty TOPSOIL		0.00	1	SS	2												
1	Grey brown SILTY CLAY		92.89	2	SS	4										34		
	Grey SILTY CLAY		1.01															
2				3	SS	6										28		
																		
																		
				4	SS	3										33		
3	End of Borehole		91.01															
			2.89															
4																		
5																		
6																		
7																		
8																		
9																		

Water observed in borehole at about 2.1 metres below existing ground surface, February 16, 2018.

Water observed  
in borehole at  
about 2.1  
metres below  
existing ground  
surface,  
February 16,  
2018.

DEPTH SCALE: 1 to 55

BORING METHOD: Power Auger

AUGER TYPE: 200 mm Hollow Stem


LOGGED: DT

CHECKED: SD

# RECORD OF BOREHOLE BH5

**PROJECT:** Proposed Residential Development  
**CLIENT:** Ironclad Developments Inc.  
**LOCATION:** 800 Eagleson Road  
**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 180084  
**DATE OF BORING:** February 16, 2018  
**SHEET** 1 of 1  
**DATUM:**

DEPTH SCALE (meters)	SOIL PROFILE		SAMPLES			UNDIST. SHEAR STRENGTH				DYNAMIC CONE PENETRATION TEST					ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	Cu, kPa				blows/300 mm						
							REM. SHEAR STRENGTH										
							20	40	60	80	10	30	50	70			90
0	Ground Surface		94.10											MC%			
	Black, Peaty TOPSOIL		0.00	1	SS	8											
	Grey brown SILT, trace sand		93.55														
	Grey brown SILTY CLAY, trace sand		0.55														
1				2	SS	4								28			
			92.55														
	Grey SILTY CLAY		1.55	3	SS	3								28			
2																	
				4	SS	2								29			
3																	
				5	SS	2								39			
4																	
			89.68	6	SS	1											
	End of Borehole		4.42														
5																	
6																	
7																	
8																	
9																	

Water measured in standpipe at approximately 0.5 metres below the existing ground surface on February 21, 2018.



Water measured in standpipe at approximately 0.5 metres below the existing ground surface on February 21, 2018.

**DEPTH SCALE:** 1 to 55

**BORING METHOD:** Power Auger

**AUGER TYPE:** 200 mm Hollow Stem

**LOGGED:** DT

**CHECKED:** SD

RECORD OF BOREHOLE BH6

**PROJECT:** Proposed Residential Development

**CLIENT:** Ironclad Developments Inc.

**LOCATION:** 800 Eagleson Road











**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 180084

**DATE OF BORING:** February 16, 2018

**SHEET** 1 of 1

**DATUM:**

DEPTH SCALE (meters)	SOIL PROFILE			SAMPLES			UNDIST. SHEAR STRENGTH					DYNAMIC CONE PENETRATION TEST					ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	Cu, kPa					blows/300 mm						
							REM. SHEAR STRENGTH											
							Cu, kPa											
0	Ground Surface		94.60														MC%	
	Black, Peaty TOPSOIL		0.00	1	SS	15												
	Grey brown SILTY CLAY, trace sand		94.15															
			0.45															
1				2	SS	6										31		
																		
																		
2			92.59	3	SS	2										28		
	Grey SILTY CLAY		2.01															
																		
				4	SS	2										35		
3	End of Borehole		91.71															
			2.89															
4																		
5																		
6																		
7																		
8																		
9																		

Water observed in borehole at about 2.1 metres below existing ground surface, February 16,

**DEPTH SCALE:** 1 to 55

**BORING METHOD:** Power Auger

**AUGER TYPE:** 200 mm Hollow Stem

**LOGGED:** DT









**CHECKED:** SD



# RECORD OF BOREHOLE BH7

PROJECT: Proposed Residential Development  
 CLIENT: Ironclad Developments Inc.  
 LOCATION: 800 Eagleson Road  
 PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180084  
 DATE OF BORING: February 16, 2018  
 SHEET 1 of 1  
 DATUM:

DEPTH SCALE (meters)	SOIL PROFILE			SAMPLES			UNDIST. SHEAR STRENGTH					DYNAMIC CONE PENETRATION TEST					ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	Cu, kPa					blows/300 mm							
							×	20	40	60	80	×	10	30	50	70			90
							○	20	40	60	80	○							
0	Ground Surface		93.60												MC%				
	Black, Peaty TOPSOIL		0.00	1	SS	5													
			93.02																
	Grey brown SILTY CLAY, trace sand		0.58																
1				2	SS	5													
																			
			91.77																
2	Grey SILTY CLAY		1.83	3	SS	5									41				
																			
				4	SS	2									30				
3	End of Borehole		90.71																
			2.89																
4																			
5																			
6																			
7																			
8																			
9																			

DEPTH SCALE: 1 to 55

BORING METHOD: Power Auger

AUGER TYPE: 200 mm Hollow Stem

LOGGED: DT

CHECKED: SD

Borehole dry,  
 February 16,  
 2018.

RECORD OF BOREHOLE BH8

**PROJECT:** Proposed Residential Development

**CLIENT:** Ironclad Developments Inc.

**LOCATION:** 800 Eagleson Road





**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 180084

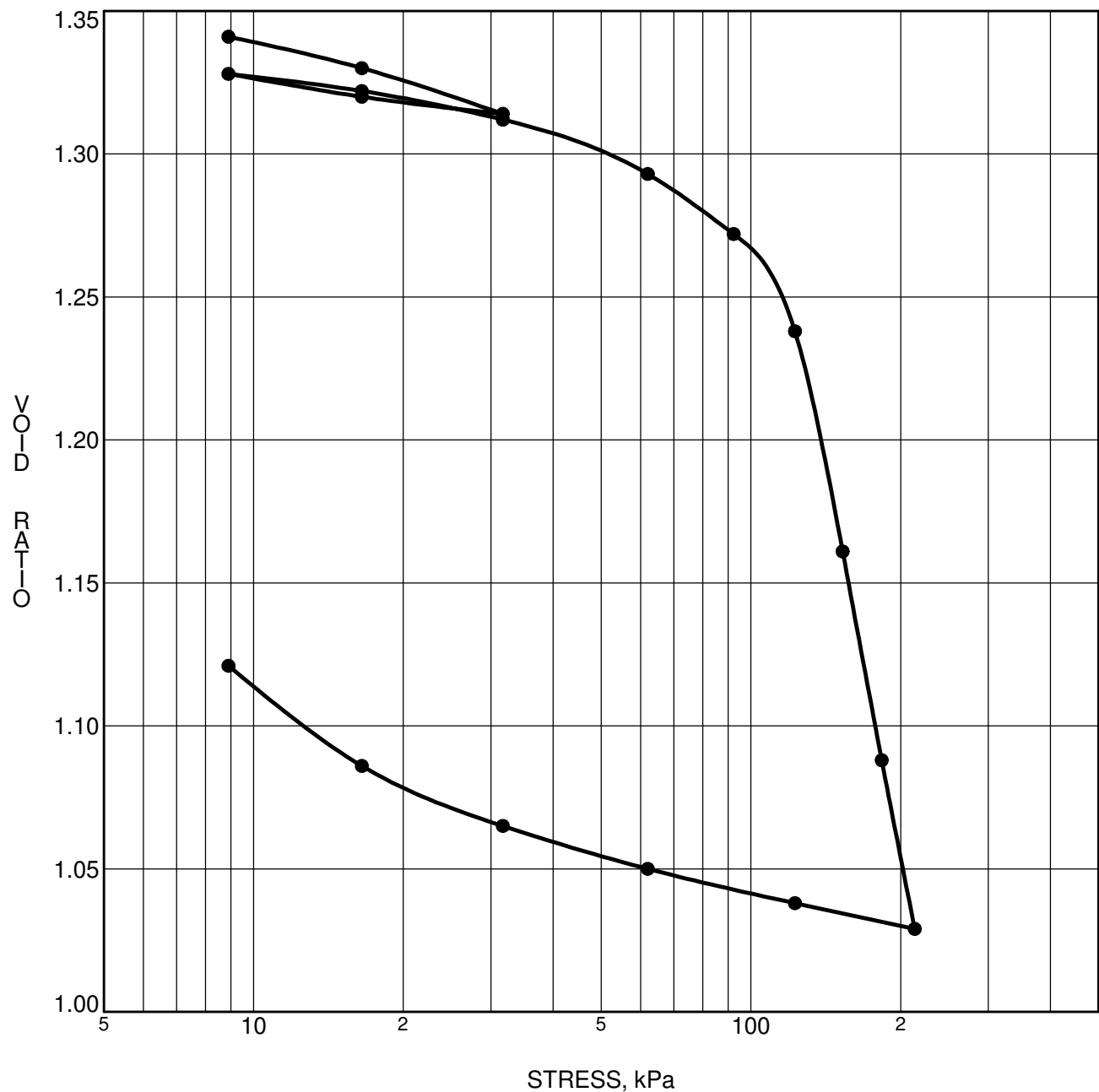
**DATE OF BORING:** February 16, 2018

**SHEET** 1 of 1

**DATUM:**

DEPTH SCALE (meters)	SOIL PROFILE		SAMPLES			UNDIST. SHEAR STRENGTH					DYNAMIC CONE PENETRATION TEST					ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	Cu, kPa					blows/300 mm						
							REM. SHEAR STRENGTH											
							Cu, kPa											
	Ground Surface		94.10												MC%			
0	Black, Peaty TOPSOIL		0.00	1	SS	4												
	Grey brown SILTY CLAY, trace sand		93.52 0.58															
1				2	SS	6									29			
	Grey SILTY CLAY		92.73 1.37															
2				3	SS	4									30			
				4	SS	2									33			
3	End of Borehole		91.21 2.89															
4																		
5																		
6																		
7																		
8																		
9																		

Water observed in borehole at about 1.4 metres below existing ground surface, February 16, 2018.



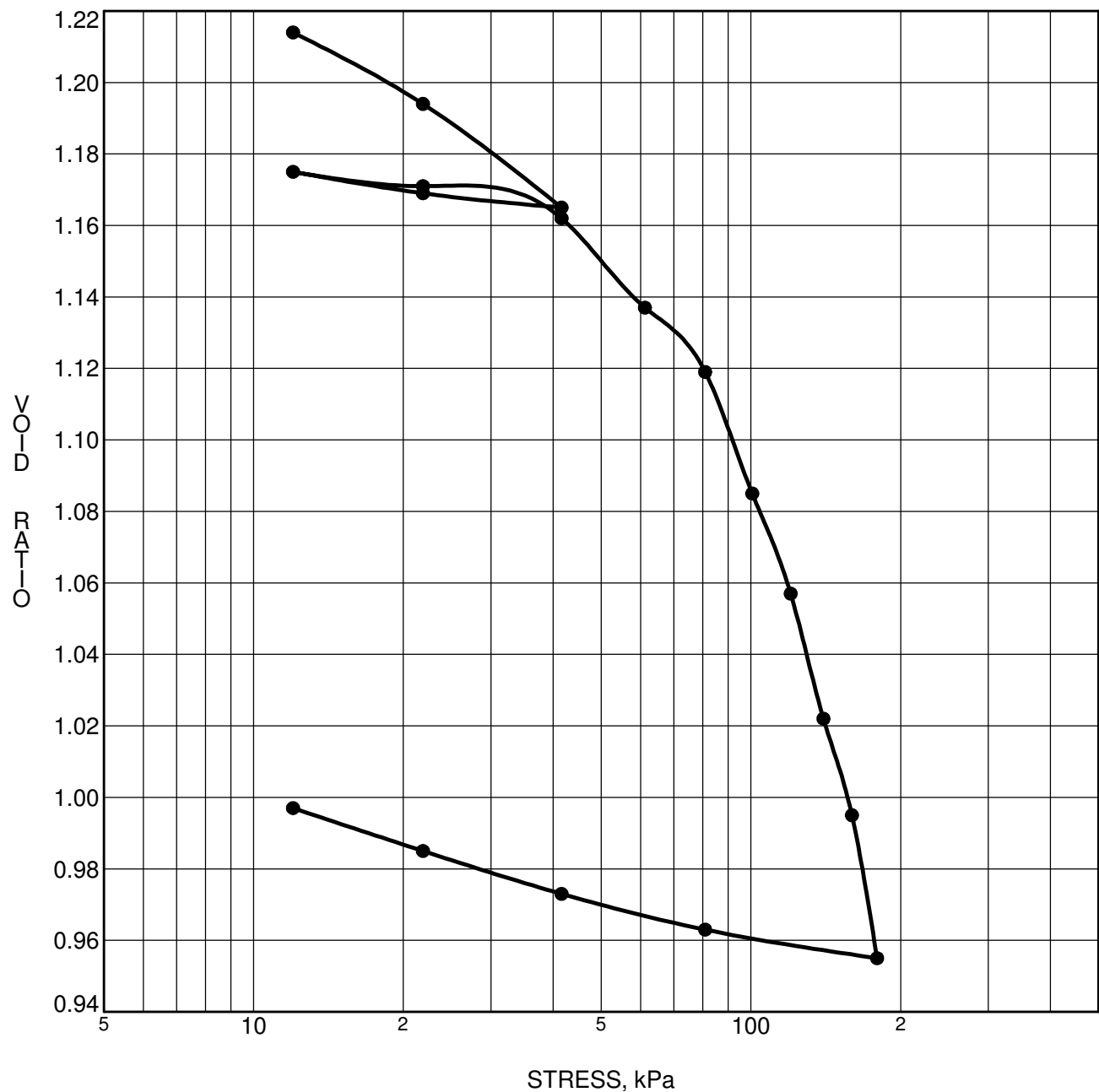
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	<b>BH 1-18</b>	$p'_o$	<b>47.34</b> kPa	$C_{cr}$	<b>0.026</b>
Sample No.	<b>TW 4</b>	$p'_c$	<b>114</b> kPa	$C_c$	<b>0.917</b>
Sample Depth	<b>5.82</b> m	OC Ratio	<b>2.4</b>	$W_o$	<b>48.8</b> %
Sample Elev.	<b>88.05</b> m	Void Ratio	<b>1.341</b>	Unit Wt.	<b>17.1</b> kN/m <sup>3</sup>

CLIENT **Ironclad Developments**  
 PROJECT **Geotechnical Investigation - Prop. Residential**  
**Development - 800 Eagleson Road**

FILE NO. **PG4692**  
 DATE **19/10/2018**

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

**CONSOLIDATION  
TEST**



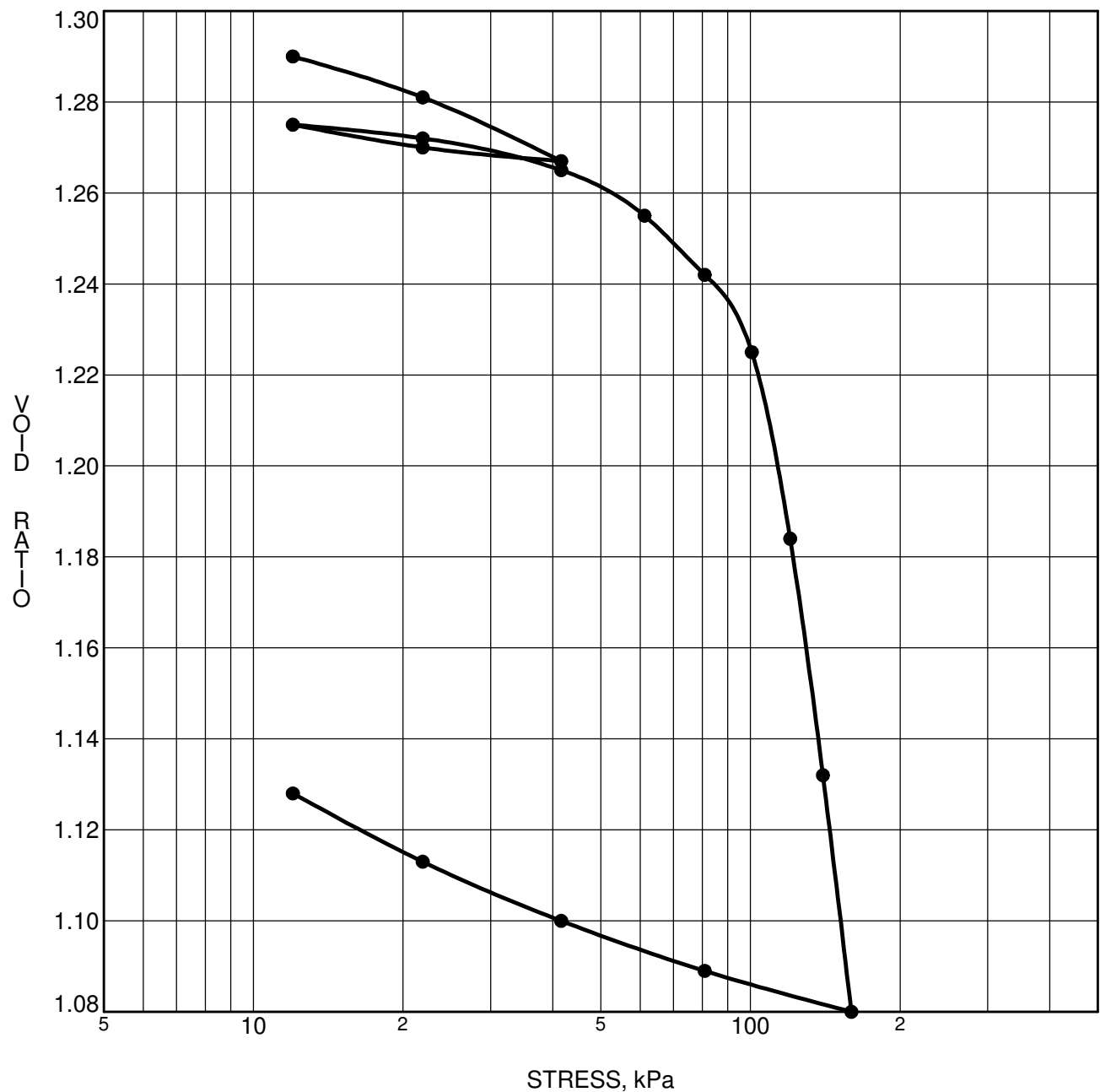
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	<b>BH 1-18</b>	$p'_o$	<b>52.06</b> kPa	$C_{cr}$	<b>0.024</b>
Sample No.	<b>TW 5</b>	$p'_c$	<b>87</b> kPa	$C_c$	<b>0.593</b>
Sample Depth	<b>6.58</b> m	OC Ratio	<b>1.7</b>	$W_o$	<b>45.3</b> %
Sample Elev.	<b>87.29</b> m	Void Ratio	<b>1.246</b>	Unit Wt.	<b>17.4</b> kN/m <sup>3</sup>

CLIENT **Ironclad Developments**  
 PROJECT **Geotechnical Investigation - Prop. Residential**  
**Development - 800 Eagleson Road**

FILE NO. **PG4692**  
 DATE **19/10/2018**

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

**CONSOLIDATION  
TEST**



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	<b>BH 3-18</b>	$p'_o$	<b>76.41</b> kPa	$C_{cr}$	<b>0.018</b>
Sample No.	<b>TW 4</b>	$p'_c$	<b>110</b> kPa	$C_c$	<b>0.906</b>
Sample Depth	<b>6.58</b> m	OC Ratio	<b>1.4</b>	$W_o$	<b>47.5</b> %
Sample Elev.	<b>87.41</b> m	Void Ratio	<b>1.305</b>	Unit Wt.	<b>17.2</b> kN/m <sup>3</sup>

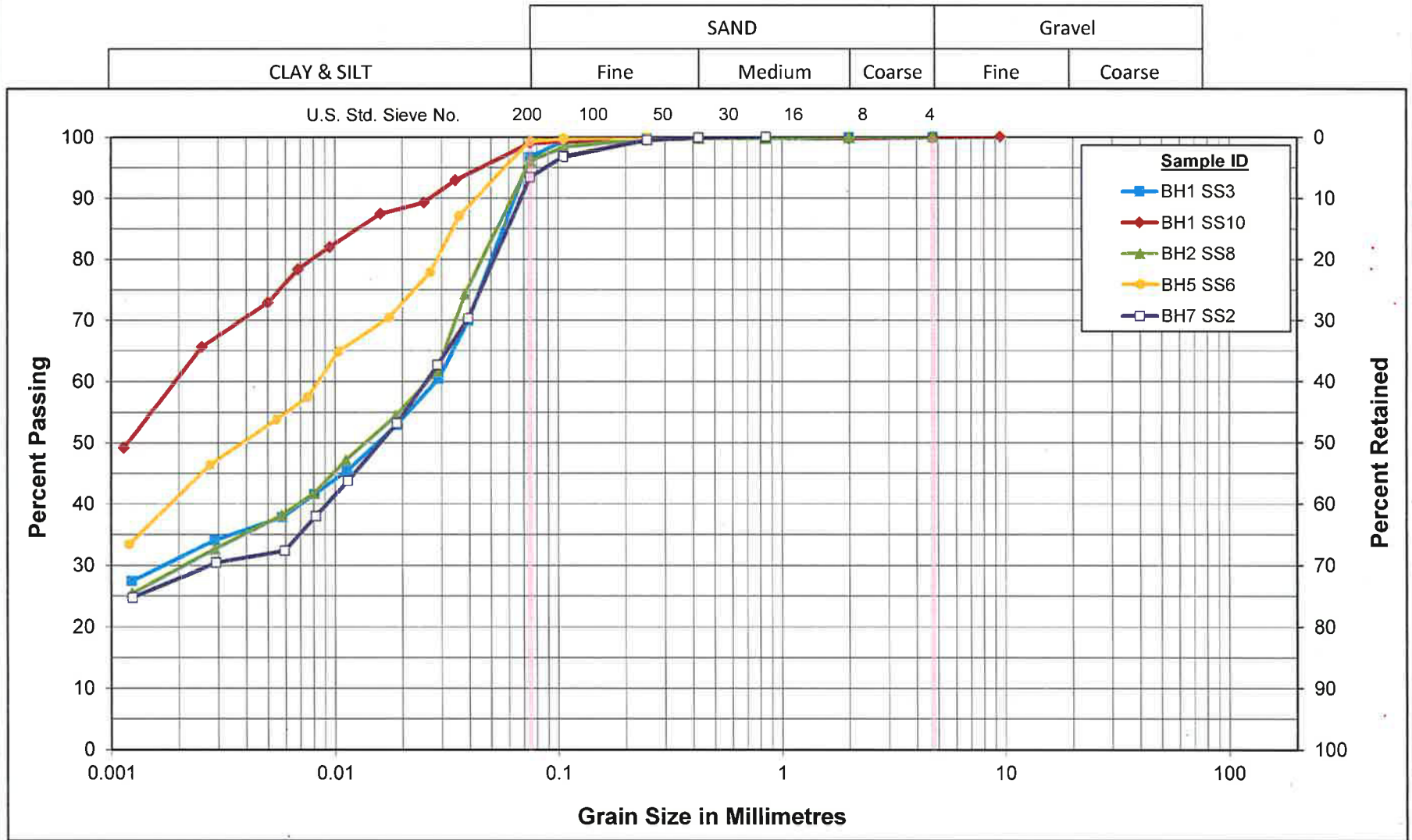
CLIENT **Ironclad Developments**  
 PROJECT **Geotechnical Investigation - Prop. Residential**  
**Development - 800 Eagleson Road**

FILE NO. **PG4692**  
 DATE **19/10/2018**

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

**CONSOLIDATION  
TEST**

# Unified Soil Classification System



## GRAIN SIZE DISTRIBUTION

Kollaard Associates Engineers, File #180084

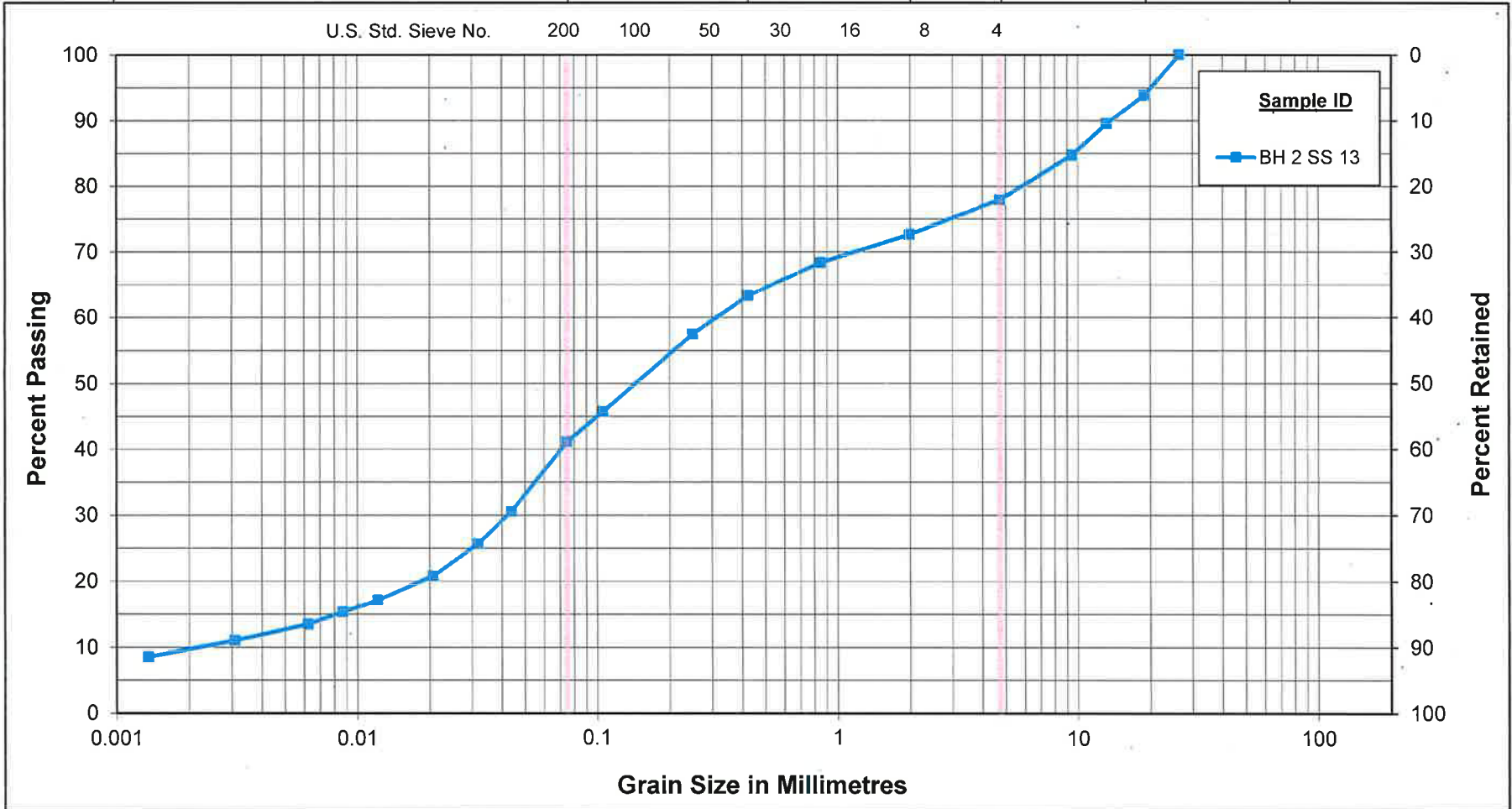
800 Eagleson Road, Ottawa

Figure No.

Project No. 122410003

# Unified Soil Classification System

CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse



## GRAIN SIZE DISTRIBUTION

Kollaard Associates Engineers, File #180084  
800 Eagleson Road, Ottawa

Figure No.

Project No. 122410003

**Certificate of Analysis**

**Client: Paterson Group Consulting Engineers**

**Client PO: 24731**

Report Date: 16-Oct-2018

Order Date: 10-Oct-2018

**Project Description: PG4692**

<b>Client ID:</b>	BH3-18-SS2	-	-	-
<b>Sample Date:</b>	10/10/2018 00:00	-	-	-
<b>Sample ID:</b>	1841311-01	-	-	-
<b>MDL/Units</b>	Soil	-	-	-

**Physical Characteristics**

% Solids	0.1 % by Wt.	82.3	-	-	-
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**General Inorganics**

pH	0.05 pH Units	7.51	-	-	-
Resistivity	0.10 Ohm.m	13.7	-	-	-

**Anions**

Chloride	5 ug/g dry	314	-	-	-
Sulphate	5 ug/g dry	149	-	-	-



# 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.

<b>RQD %</b>	<b>ROCK QUALITY</b>
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 = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = $D_{60} / D_{10}$

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		Overconsolidation 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.
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## SYMBOLS AND TERMS (continued)

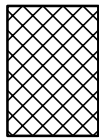
### STRATA PLOT



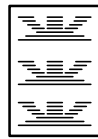
Topsoil



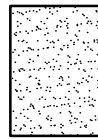
Asphalt



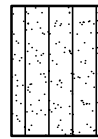
Fill



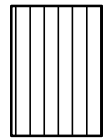
Peat



Sand



Silty Sand



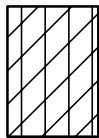
Silt



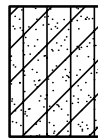
Sandy Silt



Clay



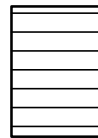
Silty Clay



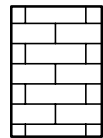
Clayey Silty Sand



Glacial Till



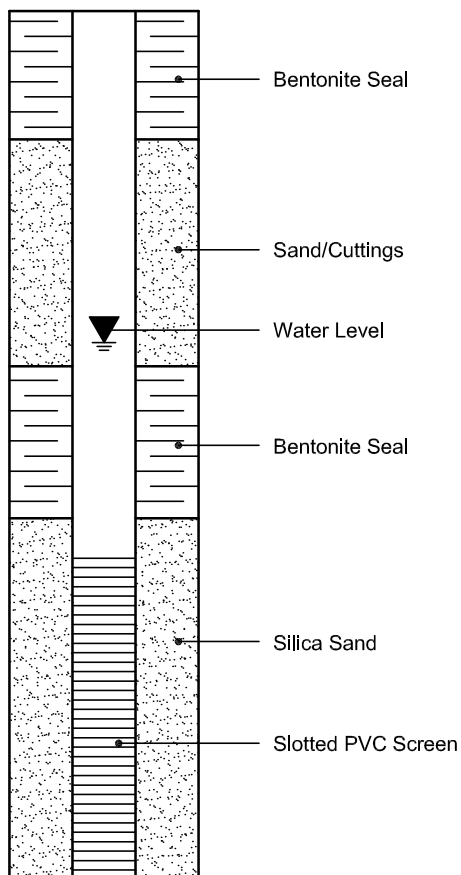
Shale



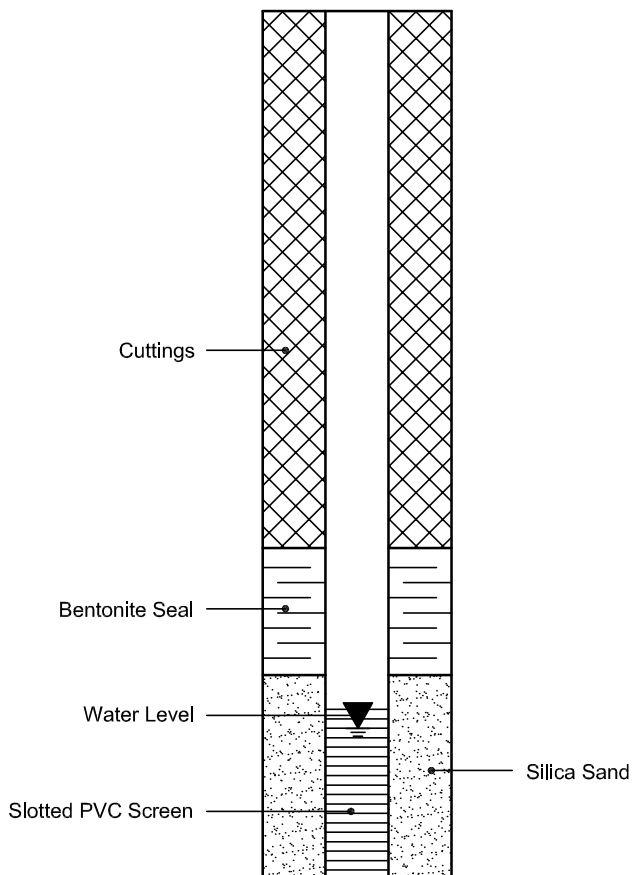
Bedrock

### MONITORING WELL AND PIEZOMETER CONSTRUCTION

#### MONITORING WELL CONSTRUCTION



#### PIEZOMETER CONSTRUCTION

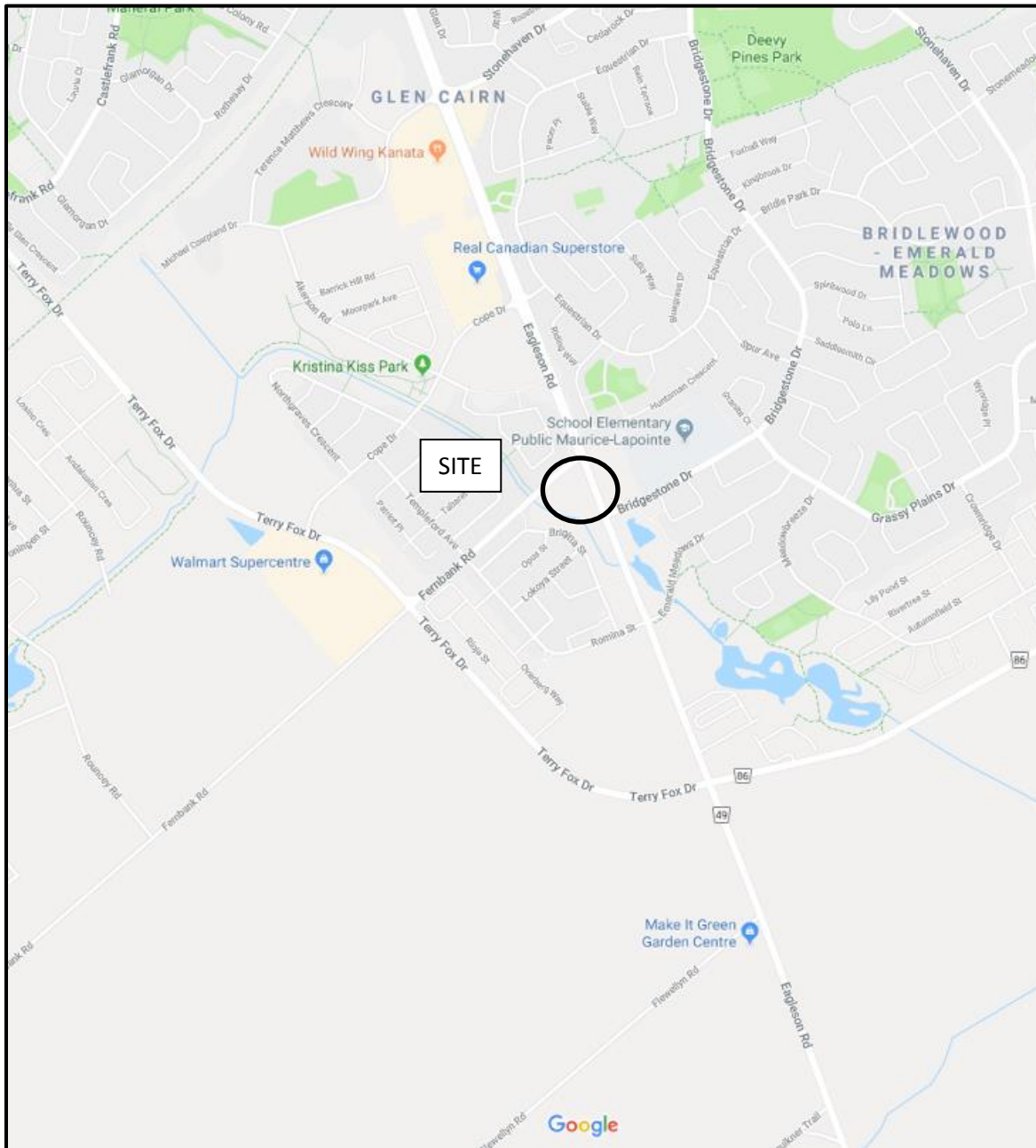


# **APPENDIX 2**

**FIGURE 1 KEY PLAN**

**FIGURE 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES**

**DRAWING PG4692-1 - TEST HOLE LOCATION PLAN**



**FIGURE 1**

**KEY PLAN**

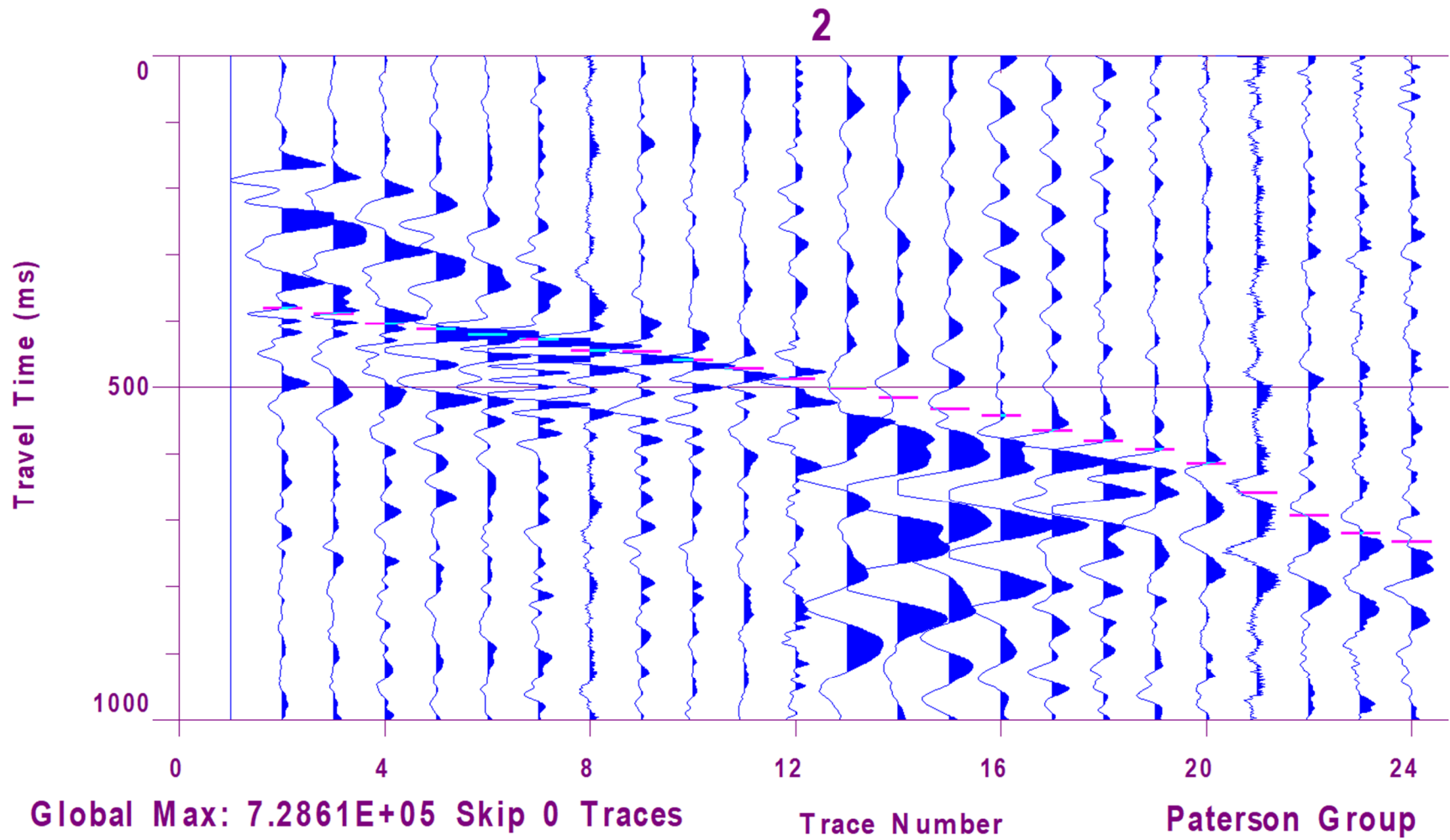


Figure 2 – Shear Wave Velocity Profile at Shot Location -14.5 m

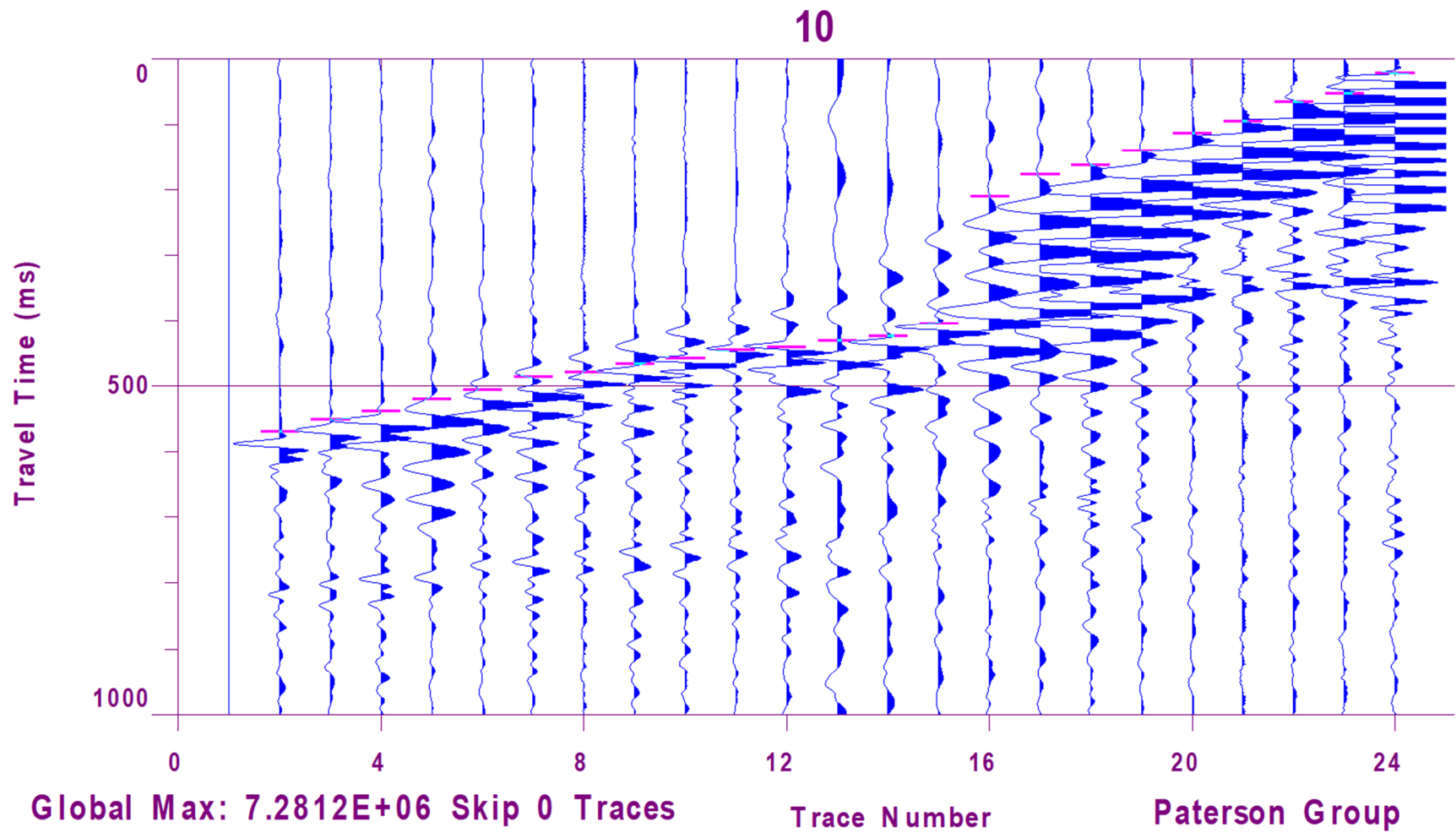
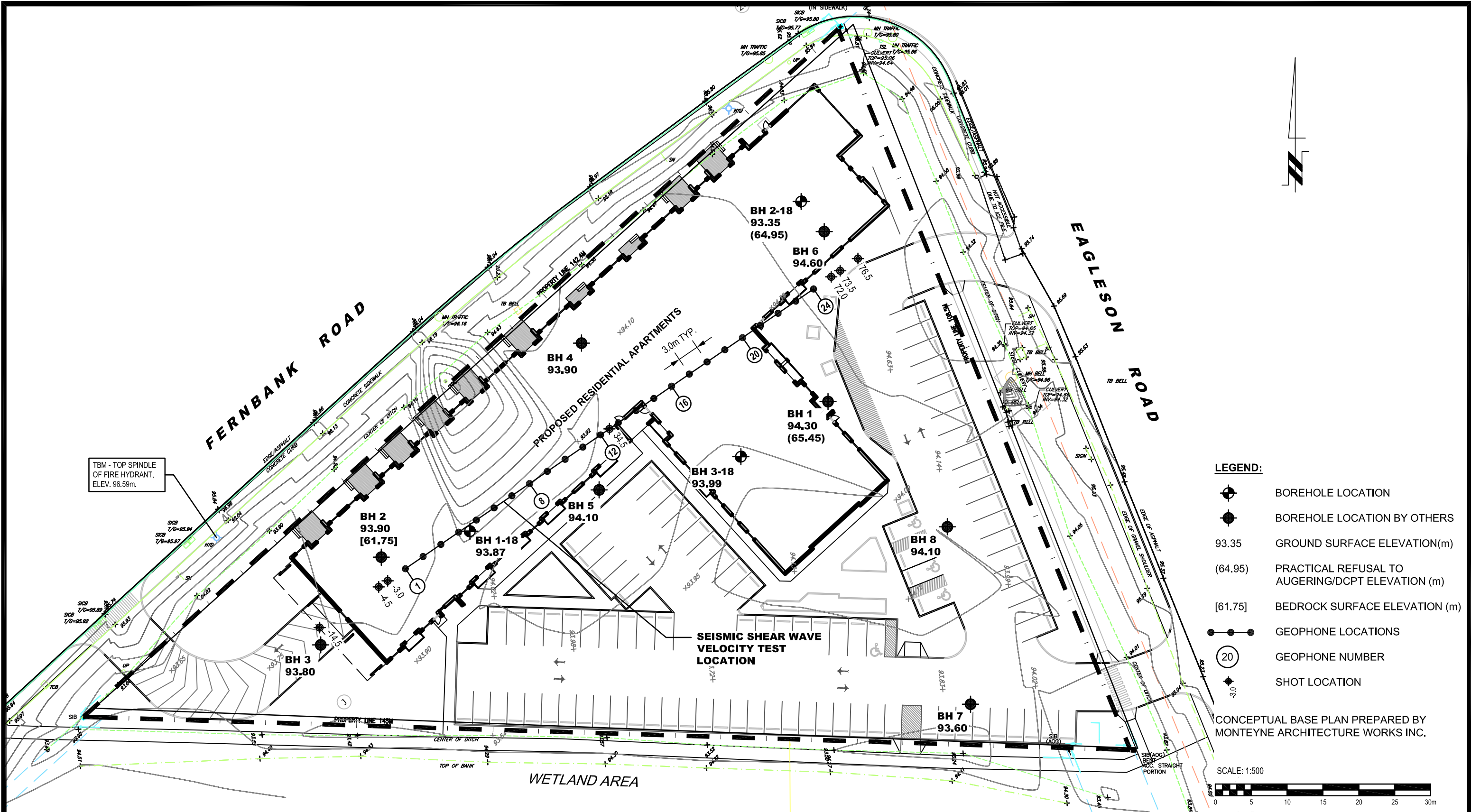


Figure 3 – Shear Wave Velocity Profile at Shot Location 72 m





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OTTAWA,  
Title:

IRONCLAD DEVELOPMENTS INC.  
GEOTECHNICAL INVESTIGATION  
800 EAGLESON ROAD

ONTARIO

## TEST HOLE LOCATION PLAN

Scale:	1:500	Date:	10/2018
Drawn by:	RCG	Report No.:	PG4692-1
Checked by:	RG	Dwg. No.:	<b>PG4692-1</b>
Approved by:	DJG	Revision No.:	0