Geotechnical Engineering

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Geotechnical Investigation

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

Prepared For

Ironclad Developments Inc.

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Report PG4692-1 Revision 1

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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 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 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 contend 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 an 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 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.

Table 1 - Summary of Atterberg Limits Tests (by others)

Sample	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Classification
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 til 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.

Table 2 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
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

Note: 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 view point, 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 the results of the undrained shear strengths conducted at the boreholes and expected building loads for the multi-storey structure, it is expected that a shallow footing foundation will not be permissible, as the footing stresses would be high and could lead to excessive settlement. A deep foundation, such as toe-bearing piles would not be economical due to the depths to the inferred bedrock surface. As such, a conventional spread footing and/or raft foundation design has been evaluated in conjunction with the use of lightweight fill (LWF) to address grade raise issues.

Based on the analyses, described under Subsection 5.3 of this report, it is expected that the building can be founded on a compensating raft foundation ("floating" foundation) placed at an inferred elevation of 91.5 m for a raft slab with an expected thickness of approximately 0.9 to 1.2 m. The founding elevation of the proposed raft slab will have a depth of approximately 4.5 m below the average ground surface of Fernbank Road. 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.

To reduce the risk of lowering the groundwater, it is recommended that the basement portion of the building be waterproofed and tanked to take advantage of the buoyancy effect. The building should be designed to resist the uplift forces due the hydrostatic pressures, that will also partly compensate for the building weight.

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.

It is recommended that the bottom of the excavation be covered with a lean concrete mud slab at founding elevation to reduce the risk of soil disturbance as the exposed silty clay will be susceptible to disturbance under the traffic of workers and equipment as the form work and reinforcing steel for the raft, and raft-embedded services are installed.

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 m wide, and pad footings, up to 6 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.

If conventional spread footings are considered, it's recommended that the bearing surface be covered by a 100 mm thick concrete mud slab consisting of 15 MPa (28-day strength) lean concrete. The concrete mud slab will act as an impermeable layer, suppressing the dewatering of the surrounding areas and will protect the bearing surface from potential disturbance caused by construction and worker traffic. Furthermore, the added concrete will improve the capacity of the bearing medium.

Compensating Raft Foundation (Floating Foundation)

In order to reduce the building pressure onto the underlying silty clay, a compensating raft foundation can be considered. Essentially, the principle of the raft foundation is that the entire floor of the building is designed as one large footing thereby allowing the use of a smaller unit bearing pressure, compared to a standard footing foundation, as the total bearing area is greatly increased. In addition, the building load can be compensated totally or partially by the excavation of the basement level(s).

The expected thickness of the raft slab will be approximately 1.5 m. Considering that the proposed basement floor elevation is inferred at an approximate elevation of 93 m, the proposed raft slab will be founded at an inferred elevation of approximately 91 m (1.5 m raft slab plus granular material and final floor slab).

Equivalent Subgrade Modulus

The structural design of the raft is based on the use of elastic analyses using a soil subgrade modulus to model the response of the soil bearing medium. The silty clay bearing medium is not a truly elastic material, however, and is not easily represented by an elastic parameter, such as the subgrade modulus.

As such, an equivalent subgrade modulus, based on the results of detailed consolidation settlement analysis, can be estimated based on an iterative process between the geotechnical and structural analyses components. A limiting subgrade modulus is determined for the soil bearing medium that is valid for the range of contact pressures that the raft will apply to the bearing medium.

For the subject building and raft, the subgrade modulus was determined to be approximately **3,000 kN/m^3** for total deflections (settlements) of up to 50 mm and differential deflections (settlements) of up to 25 mm for the overall structural raft analyses and the range of bearing resistance at SLS as described in the next section.

Bearing Resistance (Contact Pressure) at SLS

Grey firm to stiff silty clay is expected at the anticipated founding levels of elevation 91 m. The shear strength increases with increasing depth.

The unit weights in Table 3, below, in combination with a groundwater level of elevation 92.5 m, were used to determine the stress relief resulting from the excavation of the basement level and the raft slab.

Table 3 - Unit Weights for Raft Compensation

Material	Total Unit Weight	Effective Unit Weight
Grade Raise Fill	19.0 kN/m^3	----
Clayey Silty	16.5 kN/m^3	6.7 kN/m^3
Silty Clay	15.7 kN/m^3	5.9 kN/m^3
Groundwater (Buoyancy)	9.8 kN/m^3	----

Based our iterative analysis between the structural and geotechnical disciplines during previous buildings of similar design, the SLS contact pressures from the raft (including the raft weight) generally ranges between **90 kPa** and **120 kPa**. As a result, this generally requires that the raft slab be extended well beyond the vertical face of the structure which could be used as sunken terraces and basements or will require exterior LWF to be provided.

Settlement Considerations

Settlement analyses are an important part of the iterative process that leads to a raft design that suits the geotechnical conditions. The raft has to be designed to ensure that the anticipated post-construction total and differential settlements of the raft slab foundation supporting the building are within acceptable range for the structure. The total and differential settlements at the building will be limited to 50 and 25 mm, respectively.

The SLS raft contact pressures for the settlement analyses correspond to 100% of the dead load and 50% of the live load to estimate the continuously applied load. The total and differential settlements within the building footprint were estimated using a settlement model developed using Settle-3D software. The soil characteristics component of the model was based on the results of the shear strength profiles at the boreholes, correlated with the results of the consolidation tests.

The proposed grades for the development were inferred based on the existing ground surface elevations of the adjacent roadways was used to model the grading and should be confirmed once detail drawing are prepared.

Once the preliminary grading and building design are available, settlement analyses will be conducted using the raft dimensions and SLS stress distributions provided by the structural engineer. The settlement analyses will use 90% of the estimated overconsolidation of the clay, based on a shear strength to preconsolidation pressure ratio of 0.28.

The results of the preliminary settlement analysis indicates that lightweight fill (LWF) should be used as backfill over the raft extensions and within 2 to 3 m of the exterior foundation walls. The LWF should consist of EPS (expanded polystyrene) geofoam blocks, which allow for raising the grade without adding a significant load to the underlying soils. The strength of EPS to be used will depend on the use of the ground surface above the EPS.

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.

To reduce potential long term liabilities, the following means to reduce long term groundwater lowering are recommended:

❏ Clay dykes or seepage barriers should be installed in the trenches on all service connections.

❏ The lower portion of the basement level should be waterproofed and foundation drainage should be provided to elevation of approximately 1 m above the long term groundwater table, on the assumption that a minimum of 0.5 m of fill cover will be provided over the top of the EPS to provide ballast against uplift.

❏ Restriction on planting around the building, as described in Subsection 6.8, Landscaping Considerations, should be incorporated into the development.

Frictional and Passive Resistance

For foundation walls backfilled with LWF, as recommended above, the passive resistance will be limited to the 50% of the compressive strength of the LWF at 5% deformation, which accounts for a geotechnical resistance factor of 0.5.

The factored frictional sliding resistance at the bottom of the raft bearing on the silty clay is 20 kPa, which accounts for a geotechnical resistance factor of 0.5.

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 σ'_p is the preconsolidation pressure of the sample and σ'_{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 σ'_p, σ'_{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 σ'_{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 σ'_{vo} and therefore reduces the available preconsolidation. Unacceptable settlements could be induced by a significant lowering of the groundwater level.

Table 4 - Summary of Consolidation Test Results

Borehole No.	Sample	Sample Depth (m)	σ'_p (kPa)	σ'_{vo} (kPa)	C_r	C_c	Q
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 m** is recommended for grading at this site.

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, 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 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 Vs_{30} 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, Vs_{30} 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 Raft Slab and 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 raft slab.

Concrete Mud Slab for Protection of Subgrade

The subgrade material will consist of sensitive silty clay. It is recommended 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.

Pressure Relief Chamber

To allow pouring in relatively dry conditions and managing groundwater prior to developing full hydrostatic pressure and buoyancy, at the founding level, a pressure relief chamber will be installed along with collection pipes within excavated within the silty clay deposit. The collection pipe trenching should extend along the proposed building perimeter and lead to the pressure relief chamber. It is suggested that the pressure relief chamber be incorporated below the concrete mud slab and in close proximity to the proposed sump pit(s). Figure 4 - Pressure Relief Chamber in Appendix 2 provides an example of the required pressure relief chamber. Once the pressure relief chamber and associated piping is installed, the proposed raft slab can be constructed. The purpose of the pressure relief chamber will be as follows:

- Manage any water infiltration along the founding surface during the excavation program.
- Manage the water infiltration during the pouring of the raft slab to prevent water flow in the fresh concrete.
- Manage water infiltration below the raft slab until sufficient load is applied to resist any potential hydrostatic uplift.
- Regulate the discharge valve to control water infiltration once the raft slab is in place and over the long term to manage the hydrostatic pressure to permit any repairs associated with any water infiltration.
- Once sufficient load is applied to the raft slab, the pressure relief valve will be fully closed to prevent any further dewatering.

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 kN/m^3, unit weight of the soil above the groundwater level
10.8 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.

Table 5 - Recommended Pavement Structure
Car Only Parking Areas

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

Table 6 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking 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
450	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - 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

Waterproofing of the foundation up to elevation of 95.2 m (0.5 m above 1:100 year flood level of 94.62 m) is recommended for the proposed structure. Specific details of the foundation waterproofing system can be discussed in further detail with the design team.

It is further recommended that a perimeter foundation drainage system be provided for the proposed structure at elevation 94.6 m when considering a raft foundation.

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 Side Slopes

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.

Temporary Shoring

The design and approval of the temporary shoring system, should it be required, will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring system is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. The design of the temporary shoring system should take into consideration hydrostatic conditions where applicable.

The temporary shoring system could consist of steel sheet piles or a soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced. The shoring system is also recommended to be adequately supported to resist toe failure.

The earth pressures acting on the temporary shoring system may be calculated with the following parameters.

Table 7 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Drained Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no wall movement is permissible. The drained unit weight should be used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the drained unit weight for the soil should be used, with no hydrostatic groundwater pressure component.

For design purposes, a minimum factor of safety of 1.5 should be provided.

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 Planing in Sensitive Marine Clay Soils.

Low/Medium Sensitivity Clay Soils

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.

☐ Review the raft slab design and assist the design team in assessing bearing resistance values and long term settlements.

☐ 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.

Nathan F. S. Christie, P.Eng.

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

LICENSED PROFESSIONAL ENGINEER
C. P. DA SILVA
PROVINCE OF ONTARIO

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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

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

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

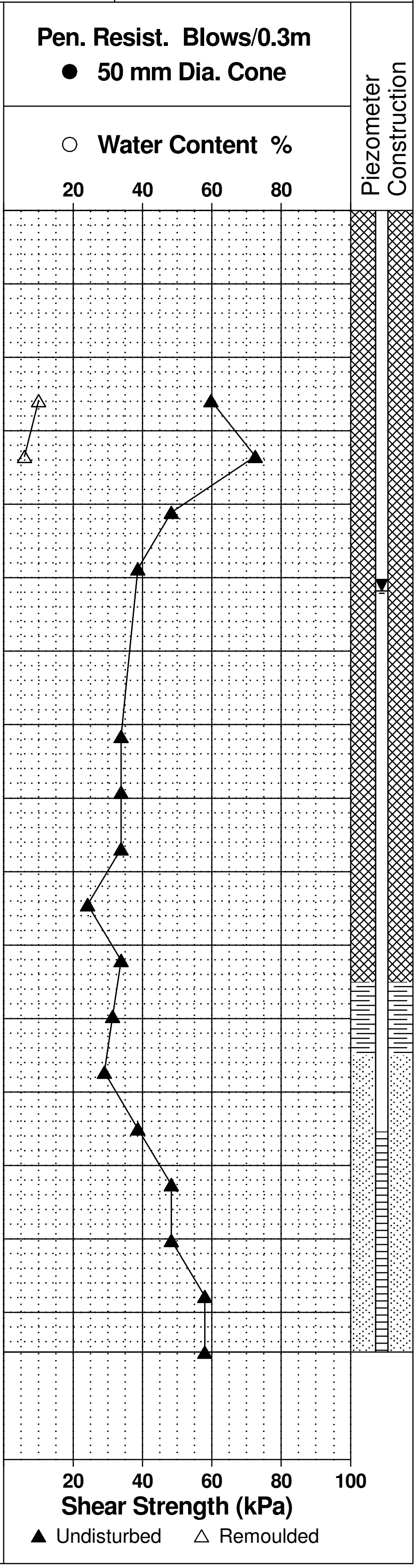
BORINGS BY CME 55 Power Auger

DATE October 9, 2018

FILE NO. PG4692

HOLE NO. BH 1-18

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	93.87
Topsoil and organics 0.25	AU	1				
Stiff, brown **SILTY CLAY/CLAYEY SILT,** trace sand 1.52	SS	2	12	2	1	92.87
	SS	3	100	4	2	91.87
					3	90.87
					4	89.87
					5	88.87
	TW	4	96		6	87.87
	TW	5	92			
Stiff to firm, grey **SILTY CLAY**					7	86.87
					8	85.87
					9	84.87
					10	83.87
					11	82.87
					12	81.87
					13	80.87
					14	79.87
- stiff by 14.4m depth					15	78.87
15.54						
End of Borehole						
(GWL @ 5.18m - Oct. 18, 2018)						



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

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

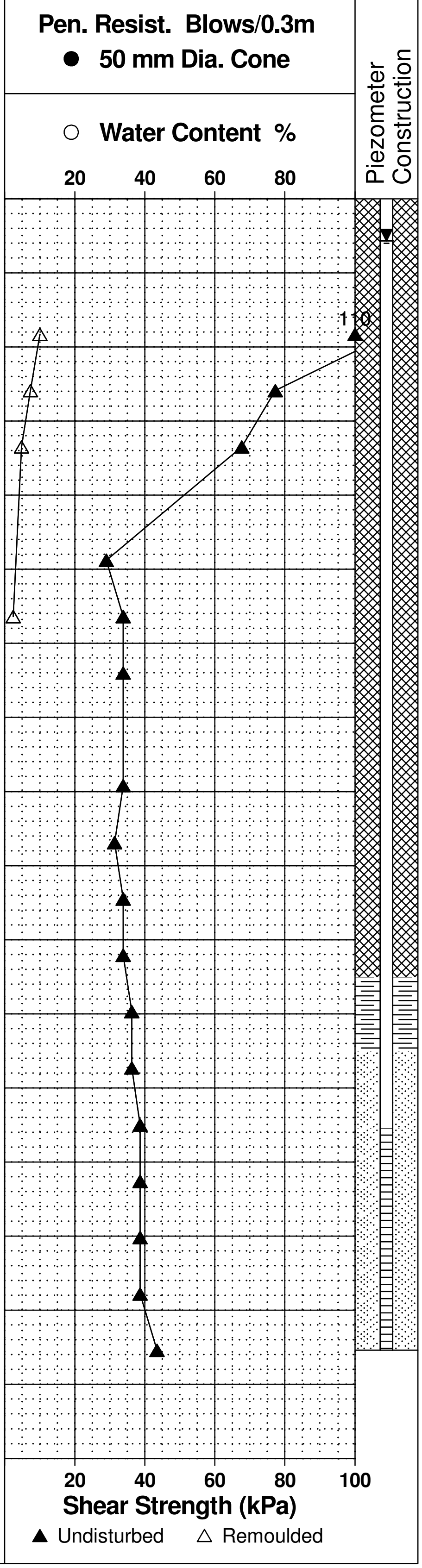
BORINGS BY CME 55 Power Auger

DATE October 9, 2018

FILE NO. PG4692

HOLE NO. BH 2-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.35
Topsoil and organics 0.28		AU	1				
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	2	83	5	1	92.35
2.20						2	91.35
						3	90.35
		TW	3	96		4	89.35
						5	88.35
						6	87.35
		TW	4	96		7	86.35
Stiff to firm, grey **SILTY CLAY**						8	85.35
						9	84.35
						10	83.35
						11	82.35
						12	81.35
						13	80.35
						14	79.35
						15	78.35
15.54							
Dynamic Cone Penetration Test (DCPT) commenced at 15.54m depth. Cone pushed to 18.3m depth.						16	77.35
						17	76.35



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

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 TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						17	76.35
						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
28.40							

End of Borehole

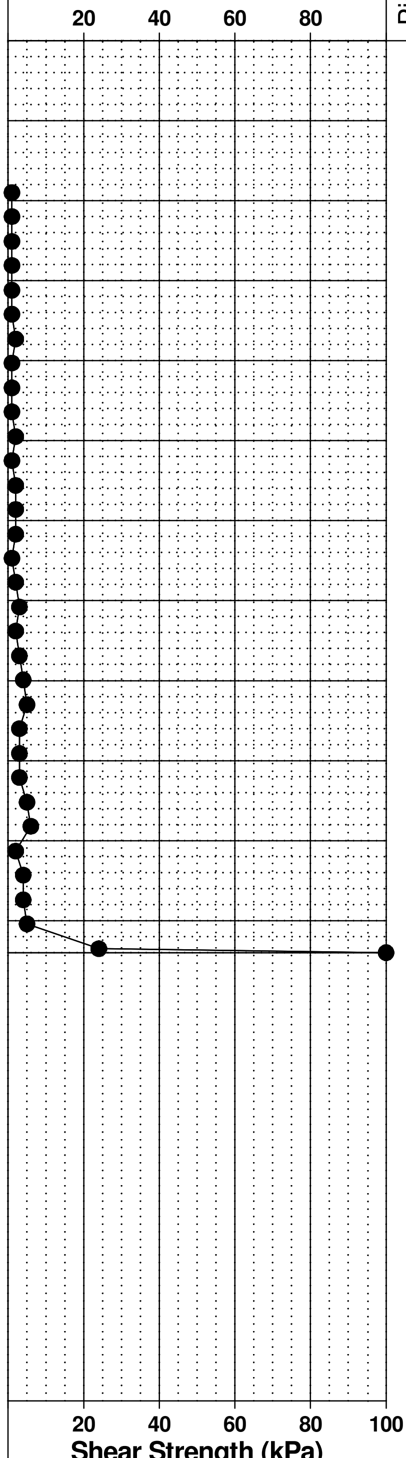
Practical DCPT refusal at 28.40m depth

(GWL @ 0.57m - Oct. 18, 2018)

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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 10, 2018

FILE NO. **PG4692**

HOLE NO. **BH 3-18**

SOIL DESCRIPTION

GROUND SURFACE

Topsoil and organics 0.25

Very stiff to stiff, brown **SILTY CLAY,** trace sand 2.20

Stiff to firm, grey **SILTY CLAY** 15.54

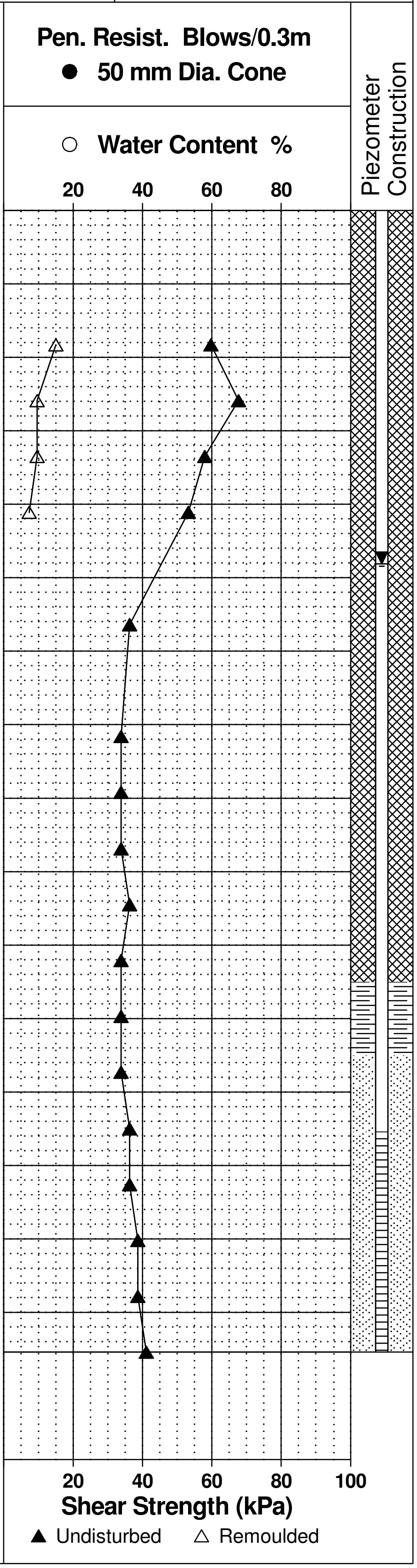
End of Borehole

(GWL @ 4.81m - Oct. 18, 2018)

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	79	5
TW	3	92	
TW	4	96	

DEPTH (m)	ELEV. (m)
0	93.99
1	92.99
2	91.99
3	90.99
4	89.99
5	88.99
6	87.99
7	86.99
8	85.99
9	84.99
10	83.99
11	82.99
12	81.99
13	80.99
14	79.99
15	78.99



RECORD OF BOREHOLE BH1

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

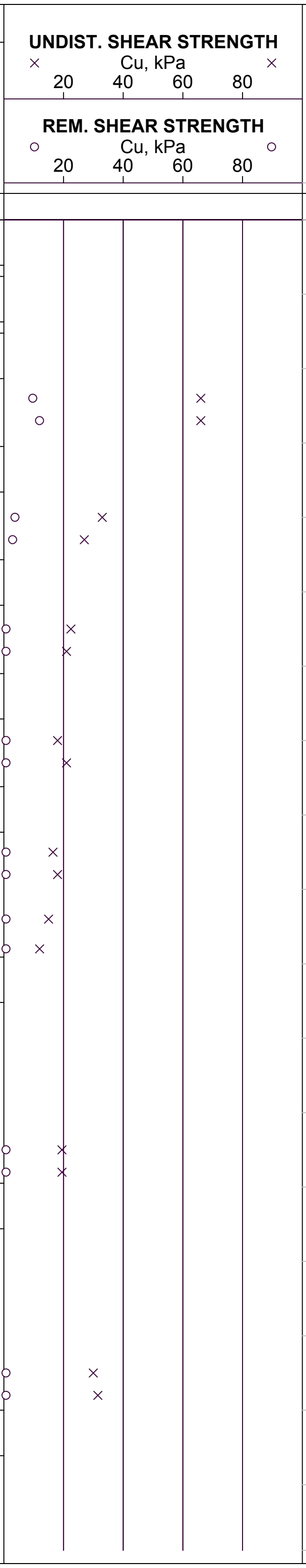
PROJECT NUMBER: 180084
DATE OF BORING: February 13, 2018
SHEET 1 of 2
DATUM:

SOIL PROFILE

DESCRIPTION	ELEV. DEPTH (M)
Ground Surface	94.30
Black, Peaty TOPSOIL	0.00 93.70
Stiff grey brown SILTY CLAY, trace sand	0.60 93.24
Firm to soft grey SILTY CLAY	1.06

SAMPLES

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING MC%
1	SS	4	27
2	SS	3	
3	SS	10	
4	SS	9	31
5	SS	4	40
6	SS	2	44
7	SS	WH	39
8	SS	WH	41
9	SS	WH	37
10	SS	WH	



DYNAMIC CONE PENETRATION TEST
blows/300 mm
10 30 50 70 90

PIEZOMETER OR STANDPIPE INSTALLATION

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

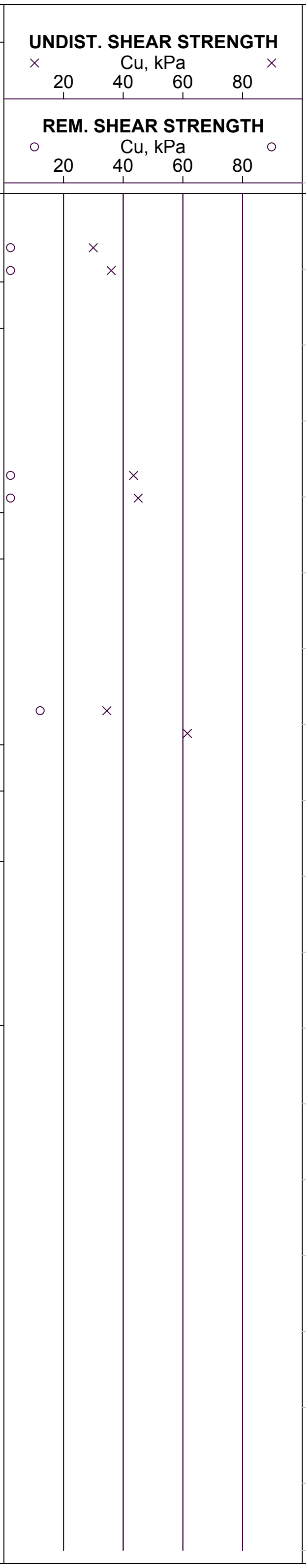
DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

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:

SOIL PROFILE		SAMPLES				
DESCRIPTION	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		11	SS	1	56	
		12	SS	4	54	
Grey silty clay, trace sand and gravel, becoming bouldery with depth (GLACIAL TILL)	69.16 25.14	13	SS	7	15	
Practical refusal to augering in Glaciall Till. Advanced borehole by coring through cobbles and boulders	67.61 26.69	14	RC			
End of Borehole	65.45 28.85					



DYNAMIC CONE PENETRATION TEST
blows/300 mm
10 30 50 70 90

DEPTH SCALE (meters): 18 19 20 21 22 23 24 25 26 27 28 29 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:

SOIL PROFILE

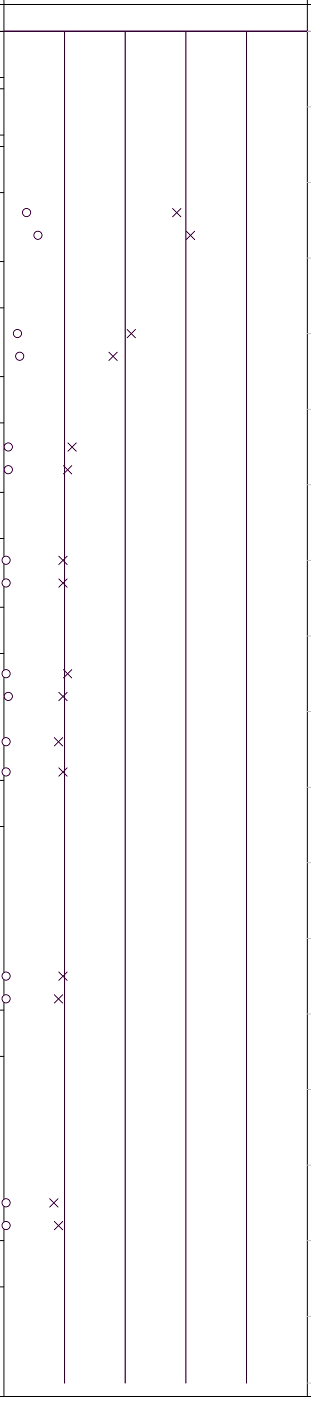
DESCRIPTION	ELEV. DEPTH (M)
Ground Surface	93.90
Black, Peaty TOPSOIL	0.00
Grey brown SILTY SAND, some clay	93.30 0.60
Stiff grey brown SILTY CLAY, trace sand	92.79 1.11
Firm to soft grey SILTY CLAY	92.35 1.55

SAMPLES

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING MC%
1	SS	6	32
2	SS	5	
3	SS	10	
4	SS	WH	47
5	SS	WH	44
6	SS	WH	44
7	SS	WH	45
8	SS	WH	
9	SS	WH	43
10	SS	WH	52

UNDIST. SHEAR STRENGTH × Cu, kPa ×
REM. SHEAR STRENGTH ○ Cu, kPa ○

DYNAMIC CONE PENETRATION TEST blows/300 mm



DEPTH SCALE (meters): 0 to 17

PIEZOMETER OR STANDPIPE INSTALLATION

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

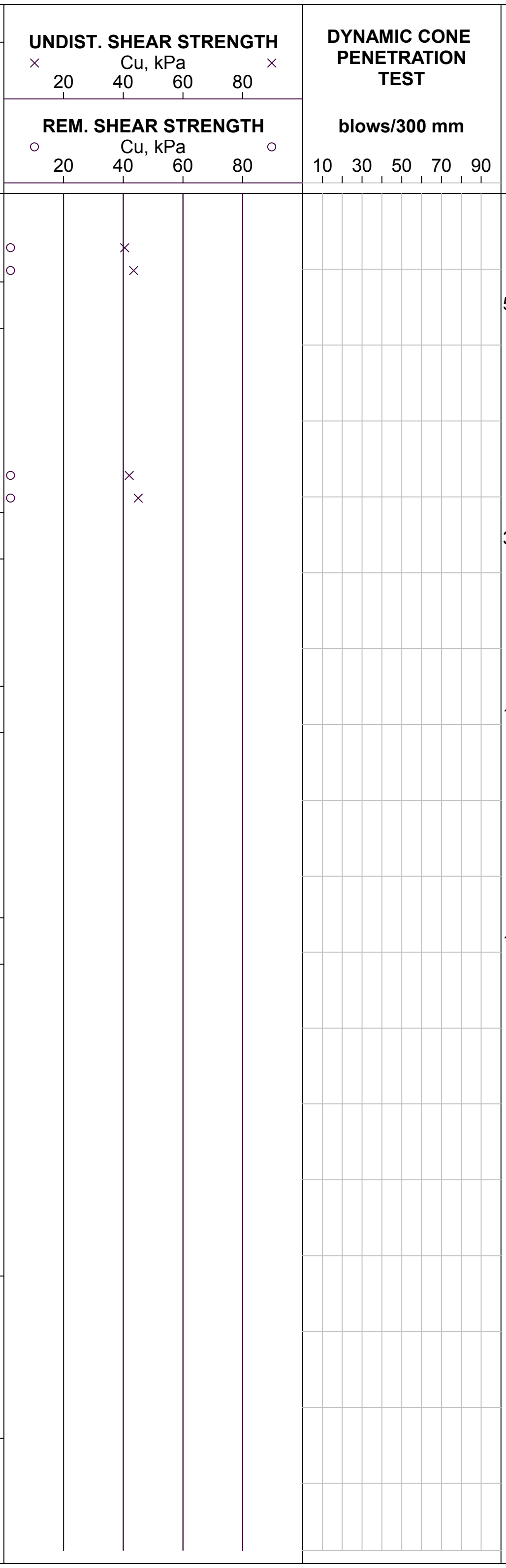
DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

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 2 of 2
DATUM:

SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		11	SS	2	58	
		12	SS	1	35	
Grey silty clay, trace sand and gravel, becoming bouldery with depth (GLACIAL TILL)	69.82 24.08	13	SS	13	15	
		14	SS	29	12	
Advanced corehole through BEDROCK	61.75 32.15	15	RC			
End of Borehole	59.61 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 × Cu, kPa × 20 40 60 80 REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m				
	Ground Surface		93.80						MC%	
0	Black, Peaty TOPSOIL		0.00	1	SS	2				
				2	SS	2				
1	Grey brown SILTY SAND, trace clay		92.74 1.06							
	Grey SILTY CLAY			3	SS	3			35	Borehole dry, February 16, 2018.
2				4	SS	9			24	
				5	SS	6			28	
3	End of Borehole		90.75 3.05							
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

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 DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80 REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		93.90						MC%	
0	Black, Peaty TOPSOIL		0.00	1	SS	2				
			92.89							
1	Grey brown SILTY CLAY		1.01	2	SS	4			34	
	Grey SILTY CLAY									
				3	SS	6			28	
2										▼
				4	SS	3			33	
			91.01							
3	End of Borehole		2.89							Water observed in borehole at about 2.1 metres below existing ground surface, February 16, 2018.
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

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:

SOIL PROFILE		SAMPLES				
DESCRIPTION	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING	
Ground Surface	94.10				MC%	
Black, Peaty TOPSOIL	0.00	1	SS	8		
	93.55					
Grey brown SILT, trace sand	0.55					
Grey brown SILTY CLAY, trace sand		2	SS	4	28	
	92.55					
Grey SILTY CLAY	1.55	3	SS	3	28	
		4	SS	2	29	
		5	SS	2	39	
		6	SS	1		
	89.68					
End of Borehole	4.42					

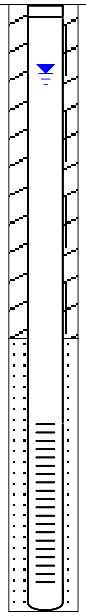
DEPTH SCALE (meters): 0 1 2 3 4 5 6 7 8 9

UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80

REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80

DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90

PIEZOMETER OR STANDPIPE INSTALLATION



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 × Cu, kPa × 20 40 60 80 REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m				
	Ground Surface		94.60						MC%	
0	Black, Peaty TOPSOIL		0.00 94.15	1	SS	15				
1	Grey brown SILTY CLAY, trace sand		0.45	2	SS	6			31	
			92.59	3	SS	2			28	
2	Grey SILTY CLAY		2.01 91.71	4	SS	2			35	▼ Water observed in borehole at about 2.1 metres below existing ground surface, February 16,
3	End of Borehole		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

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 × Cu, kPa × 20 40 60 80 REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3m				
	Ground Surface		93.60						MC%	
0	Black, Peaty TOPSOIL		0.00							
				1	SS	5				
			93.02							
	Grey brown SILTY CLAY, trace sand		0.58							
1				2	SS	5				
			91.77							
	Grey SILTY CLAY		1.83	3	SS	5			41	Borehole dry, February 16, 2018.
2										
				4	SS	2			30	
			90.71							
3	End of Borehole		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

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 DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80 REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		94.10						MC%	
0	Black, Peaty TOPSOIL		0.00 93.52	1	SS	4				
1	Grey brown SILTY CLAY, trace sand		0.58 92.73	2	SS	6			29	
2	Grey SILTY CLAY		1.37	3	SS	4			30	▼ Water observed in borehole at about 1.4 metres below existing ground surface, February 16, 2018.
			91.21	4	SS	2			33	
3	End of Borehole		2.89							

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

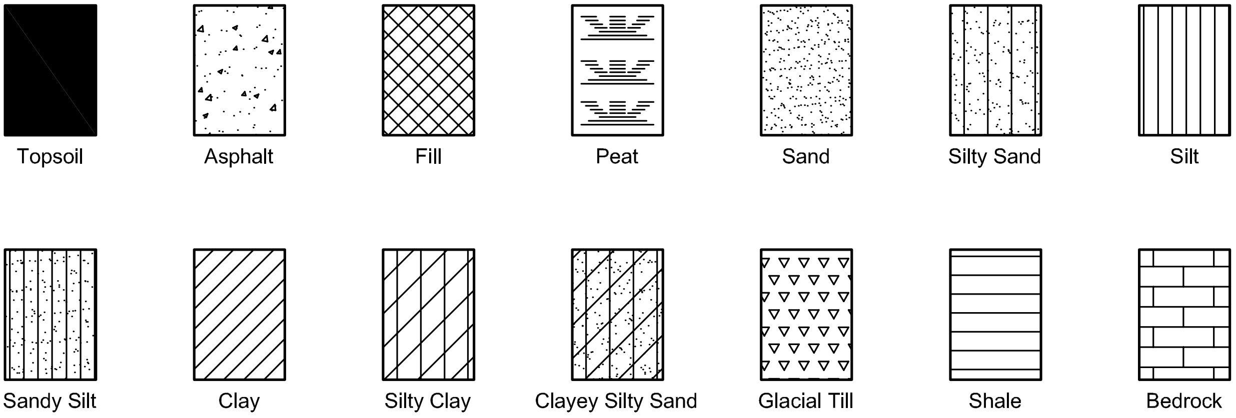
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

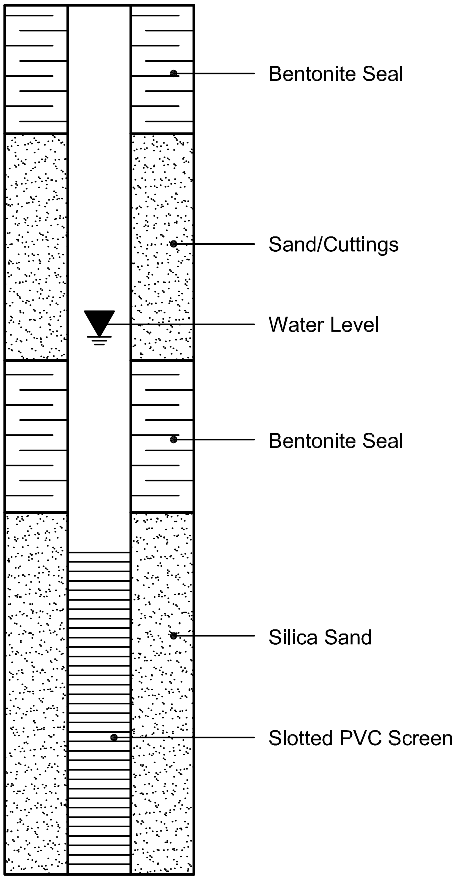
SYMBOLS AND TERMS (continued)

STRATA PLOT

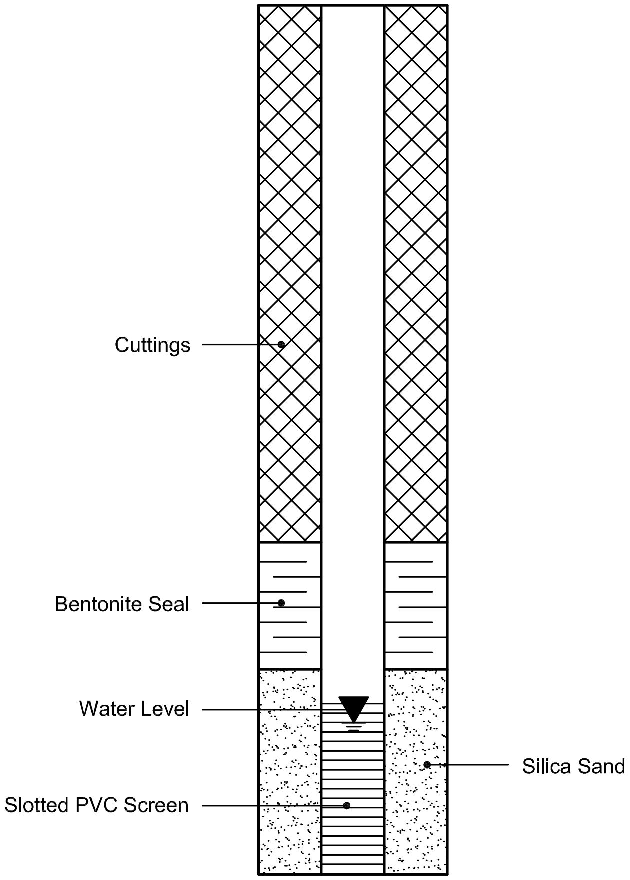


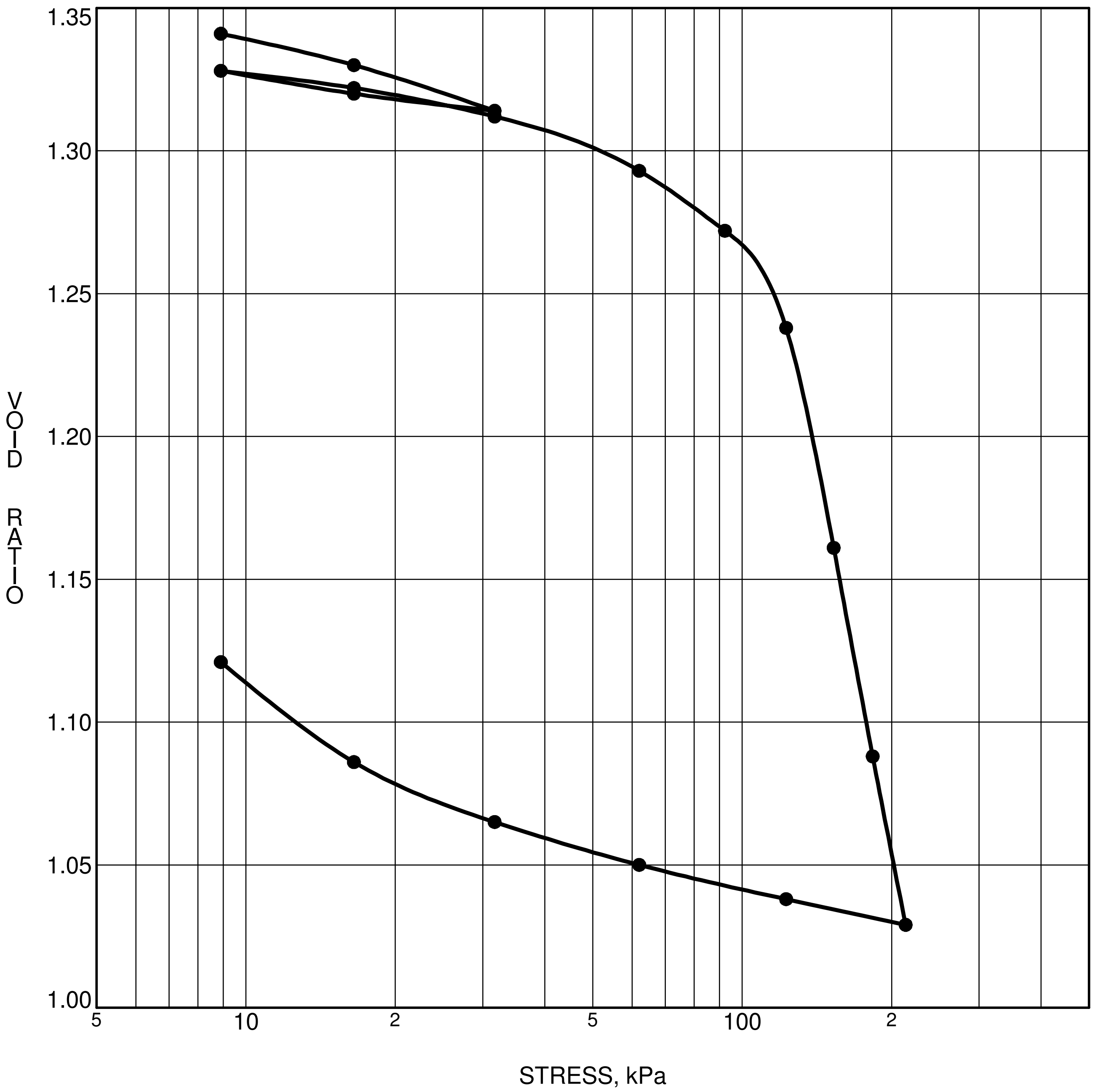
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 1-18**	p'o	**47.34** kPa	Ccr	**0.026**
Sample No.	**TW 4**	p'c	**114** kPa	Cc	**0.917**
Sample Depth	**5.82** m	OC Ratio	**2.4**	Wo	**48.8** %
Sample Elev.	**88.05** m	Void Ratio	**1.341**	Unit Wt.	**17.1** kN/m ^{3}

CLIENT	**Ironclad Developments**	FILE NO.	**PG4692**
PROJECT	**Geotechnical Investigation - Prop. Residential Development - 800 Eagleson Road**	DATE	**19/10/2018**

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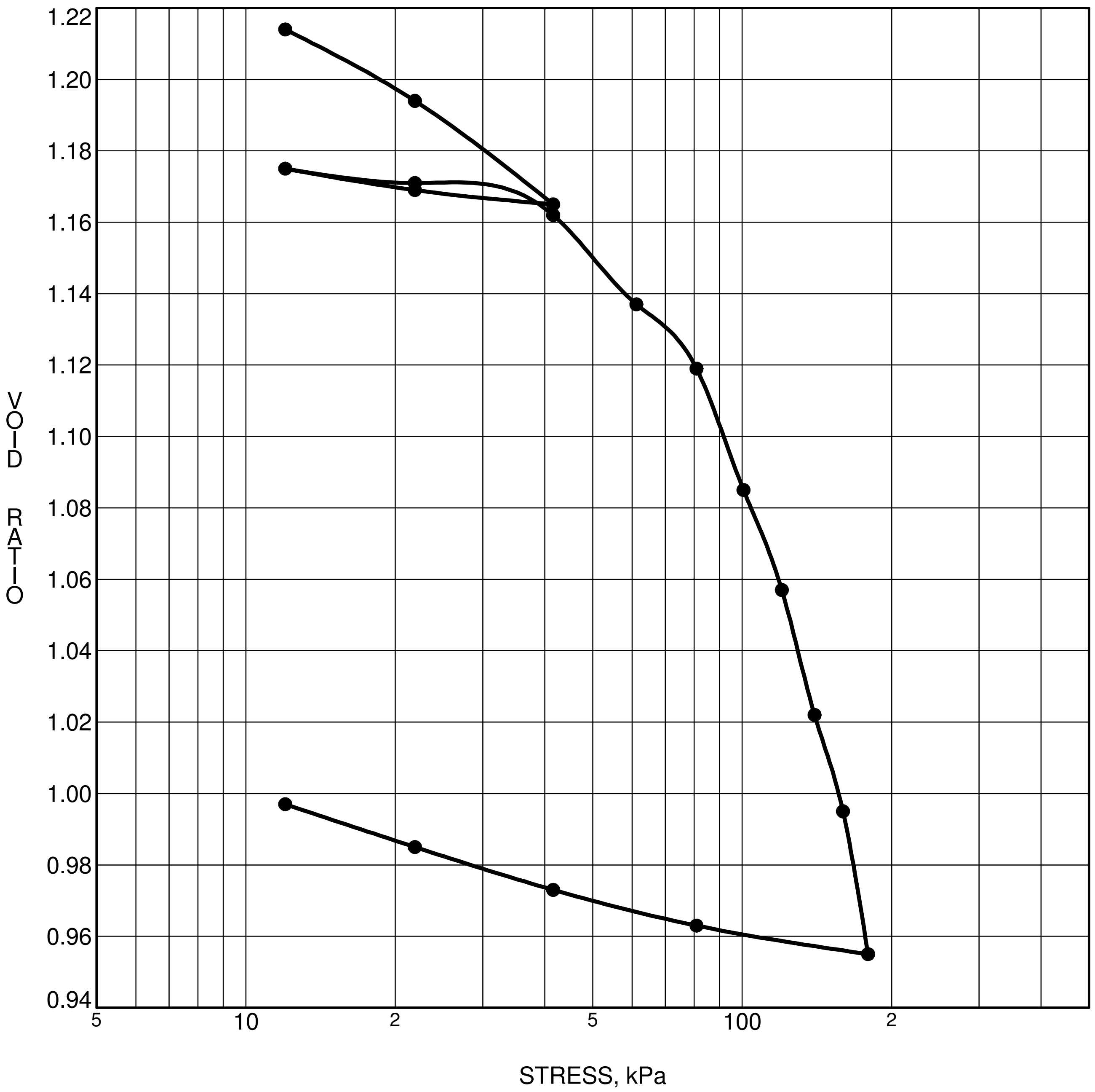
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



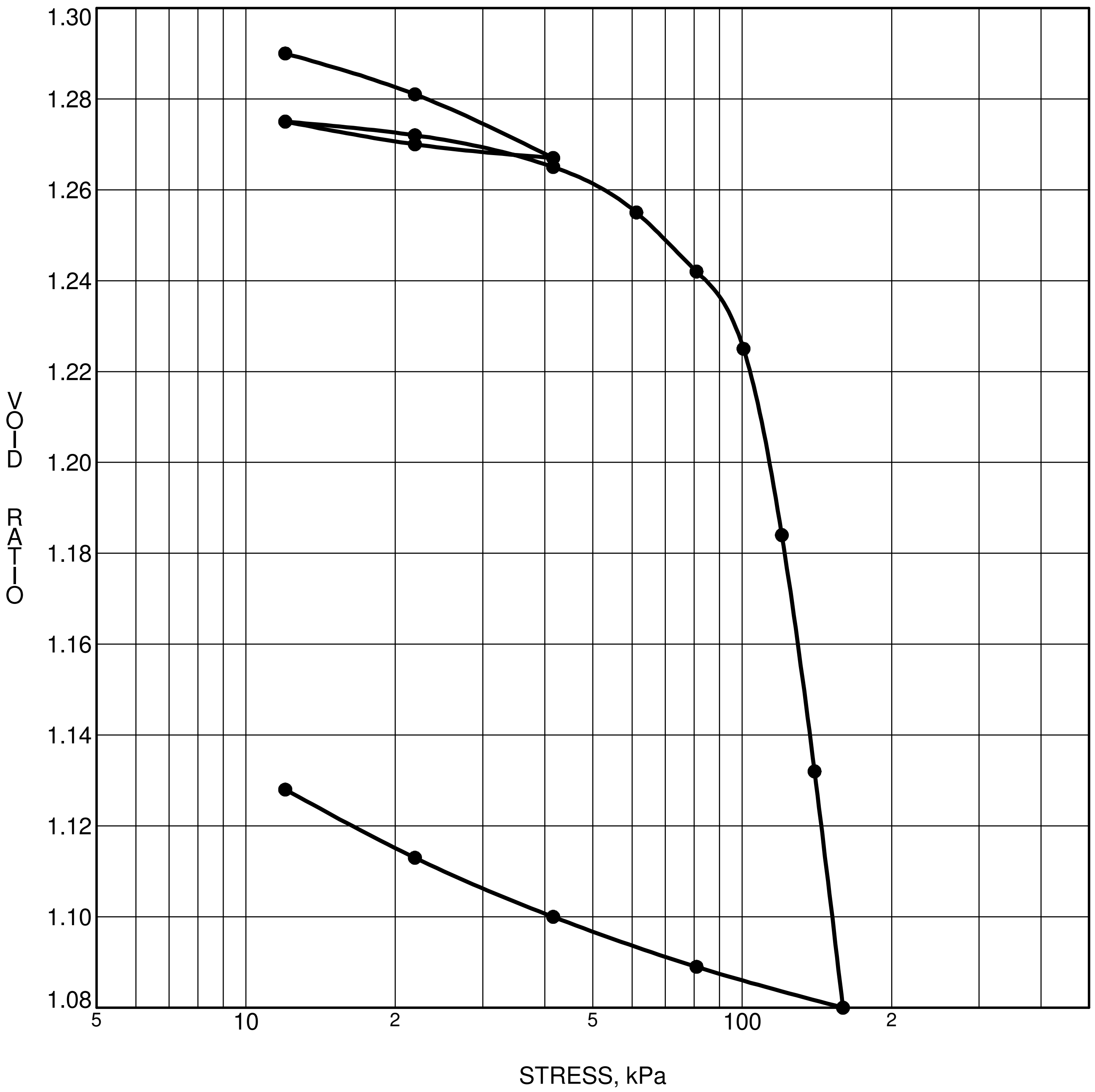
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 1-18**	p'o	**52.06** kPa	Ccr	**0.024**
Sample No.	**TW 5**	p'c	**87** kPa	Cc	**0.593**
Sample Depth	**6.58** m	OC Ratio	**1.7**	Wo	**45.3** %
Sample Elev.	**87.29** m	Void Ratio	**1.246**	Unit Wt.	**17.4** kN/m ^{3}

CLIENT	**Ironclad Developments**	FILE NO.	**PG4692**
PROJECT	**Geotechnical Investigation - Prop. Residential Development - 800 Eagleson Road**	DATE	**19/10/2018**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 3-18**	p'o	**76.41** kPa	Ccr	**0.018**
Sample No.	**TW 4**	p'c	**110** kPa	Cc	**0.906**
Sample Depth	**6.58** m	OC Ratio	**1.4**	Wo	**47.5** %
Sample Elev.	**87.41** m	Void Ratio	**1.305**	Unit Wt.	**17.2** kN/m ^{3}

CLIENT	**Ironclad Developments**	FILE NO.	**PG4692**
PROJECT	**Geotechnical Investigation - Prop. Residential Development - 800 Eagleson Road**	DATE	**19/10/2018**

patersongroup **Consulting Engineers**

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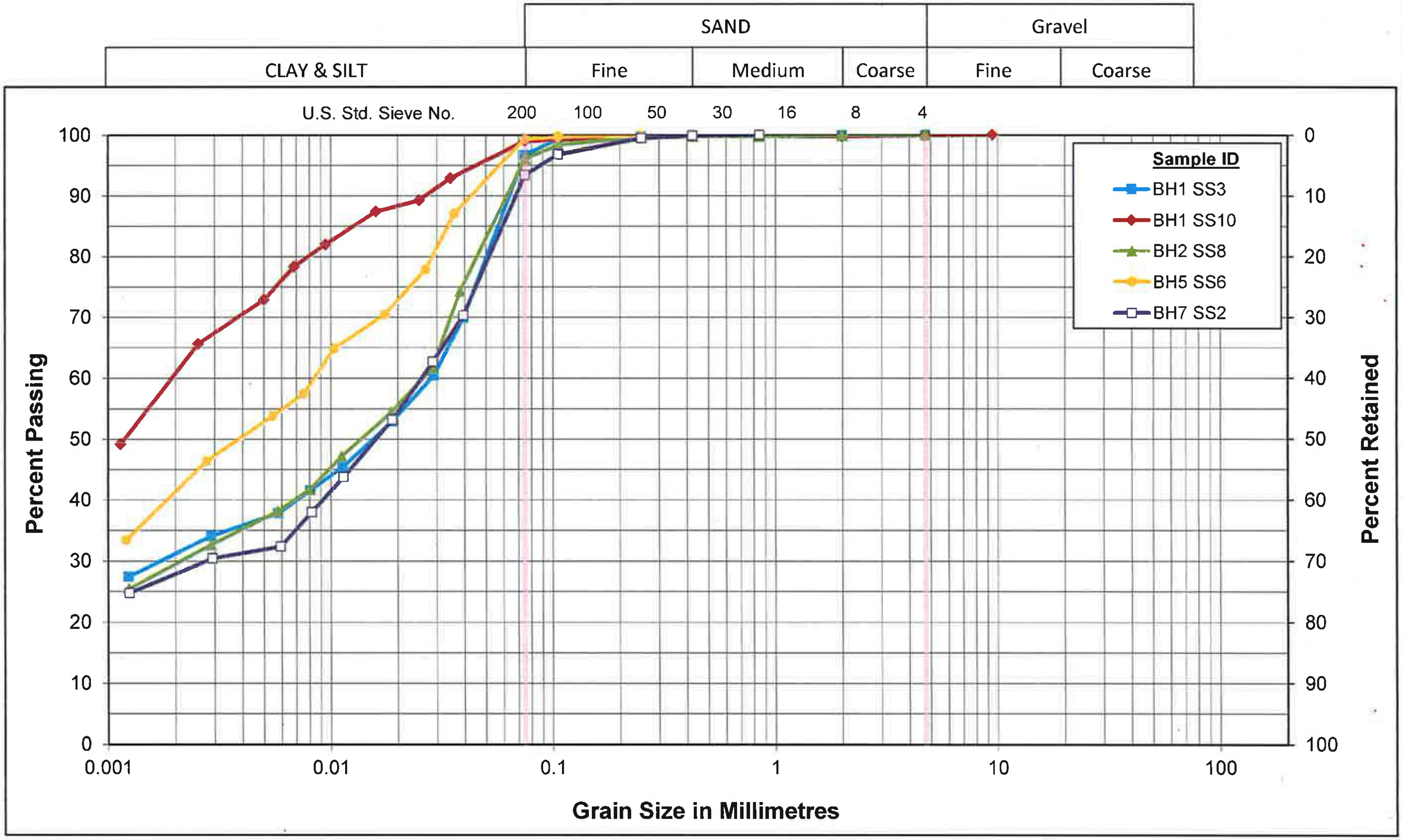
CONSOLIDATION TEST

Unified Soil Classification System



Stantec

GRAIN SIZE DISTRIBUTION

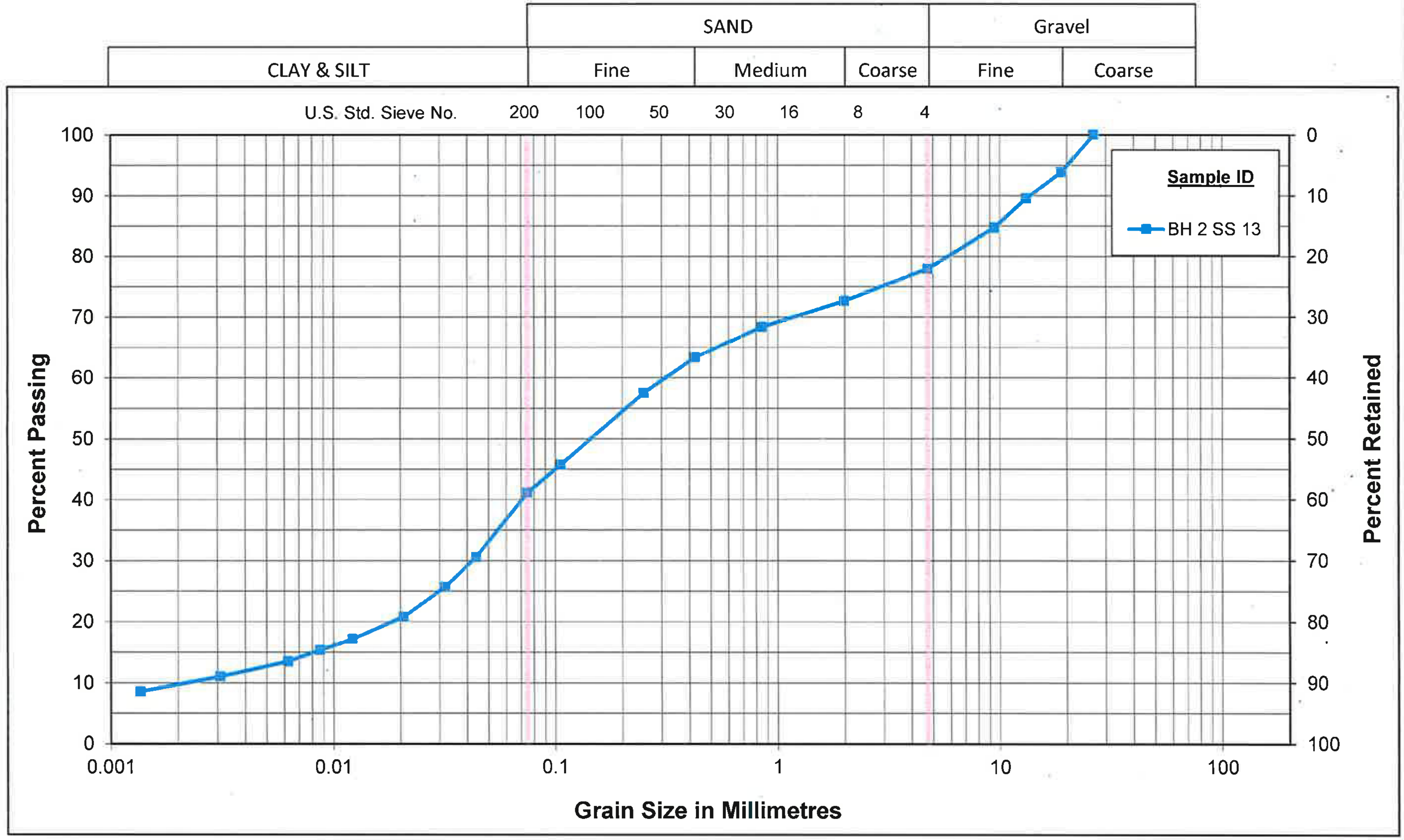
Kollaard Associates Engineers, File #180084

800 Eagleson Road, Ottawa

Figure No.

Project No. 122410003

Unified Soil Classification System



Stantec

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

	Client ID:	BH3-18-SS2	-	-	-
	Sample Date:	10/10/2018 00:00	-	-	-
	Sample ID:	1841311-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	82.3	-	-	-
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	-	-	-

APPENDIX 2

FIGURE 1: KEY PLAN

FIGURE 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4: PRESSURE RELIEF CHAMBER

DRAWING PG4692-1 - TEST HOLE LOCATION PLAN

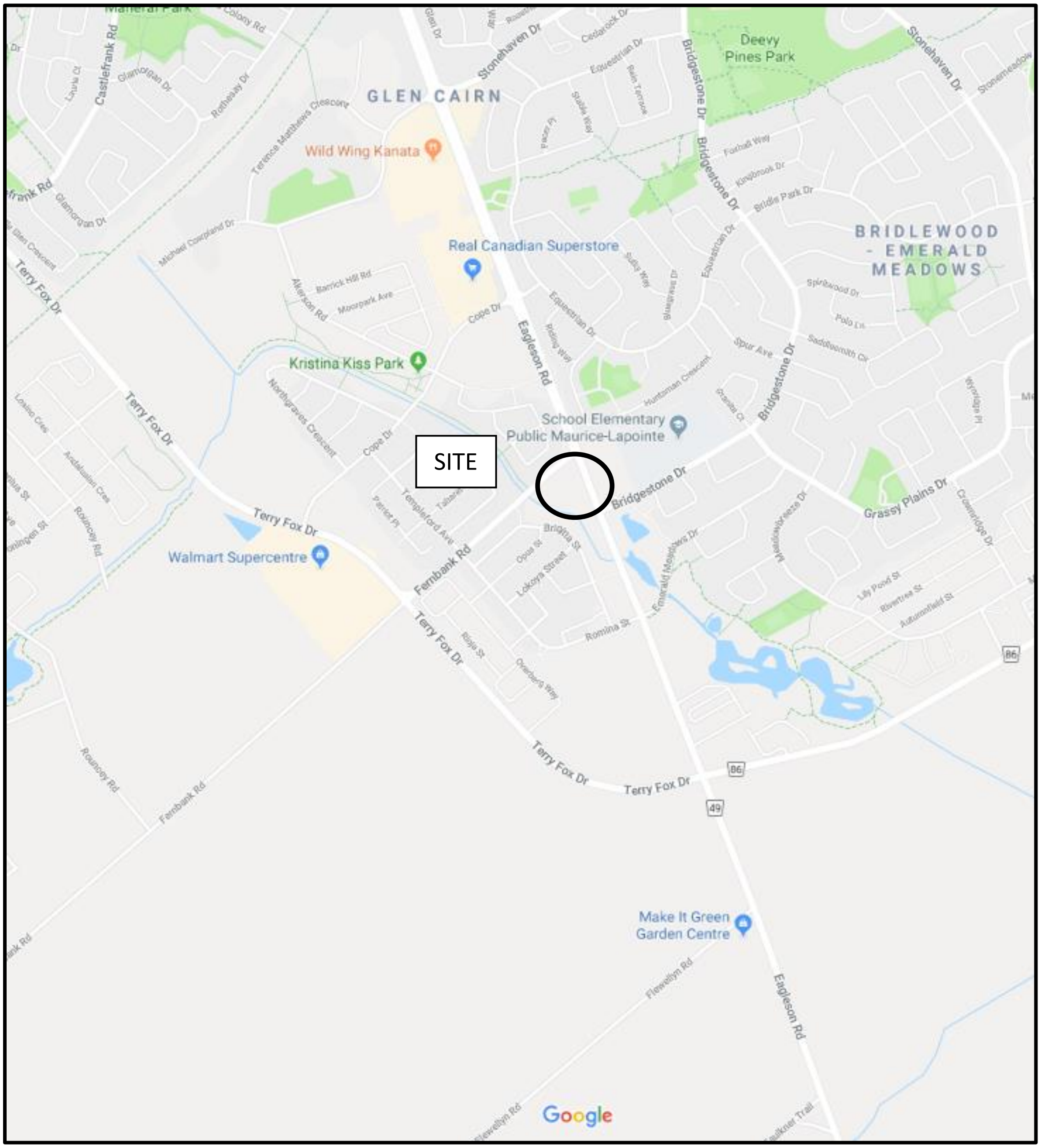


FIGURE 1

KEY PLAN

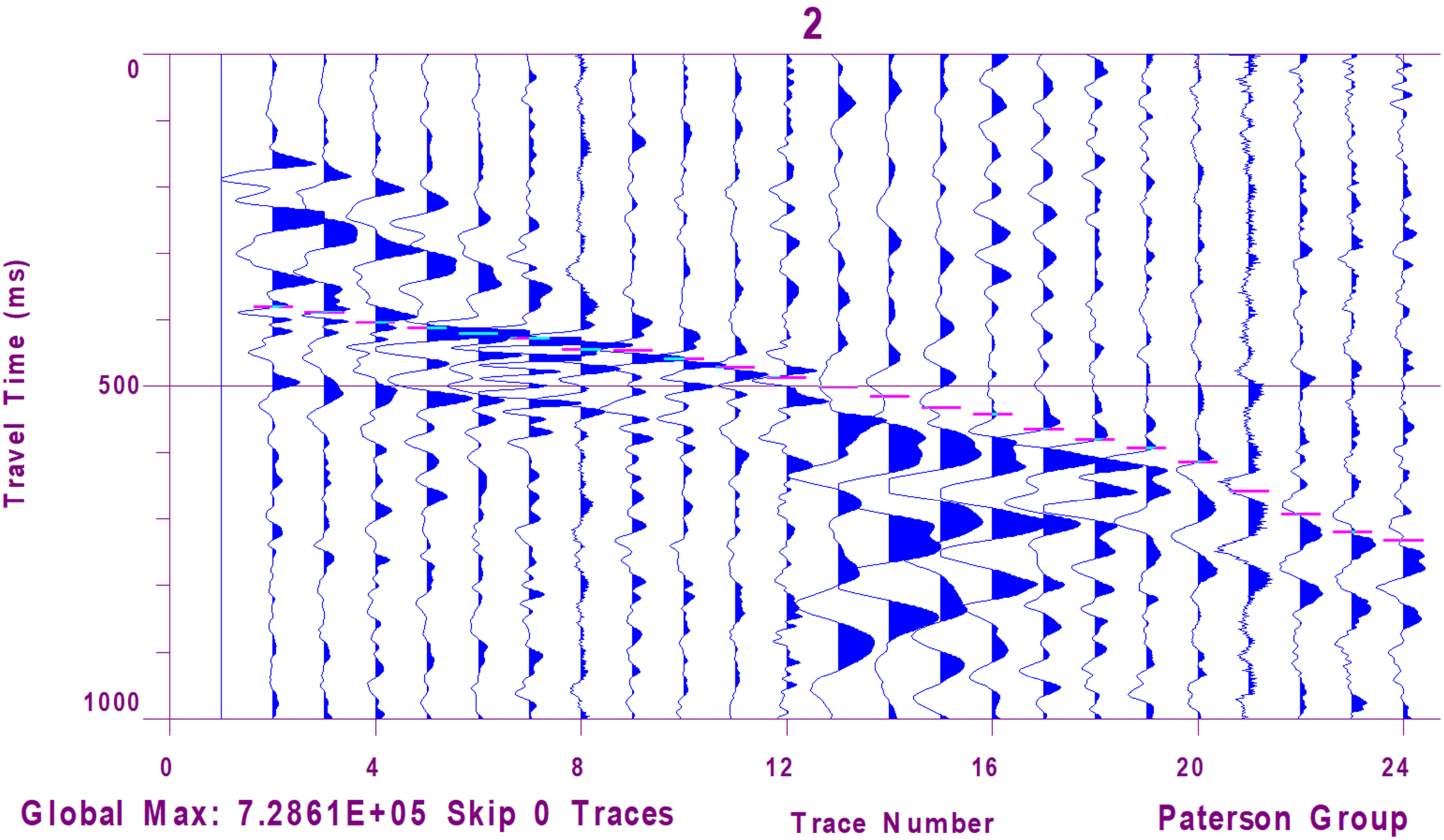


Figure 2 – Shear Wave Velocity Profile at Shot Location -14.5 m

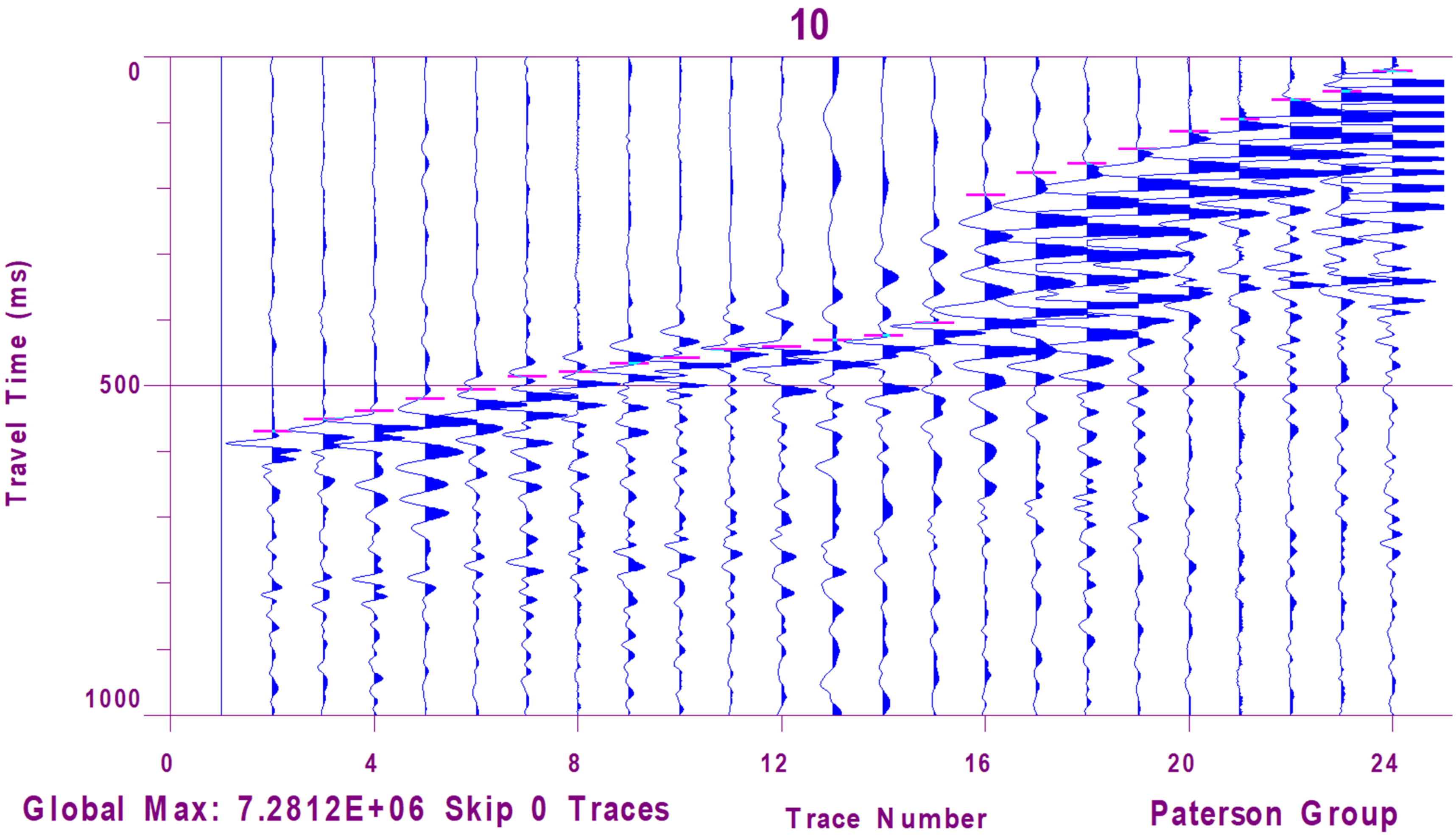


Figure 3 – Shear Wave Velocity Profile at Shot Location 72 m

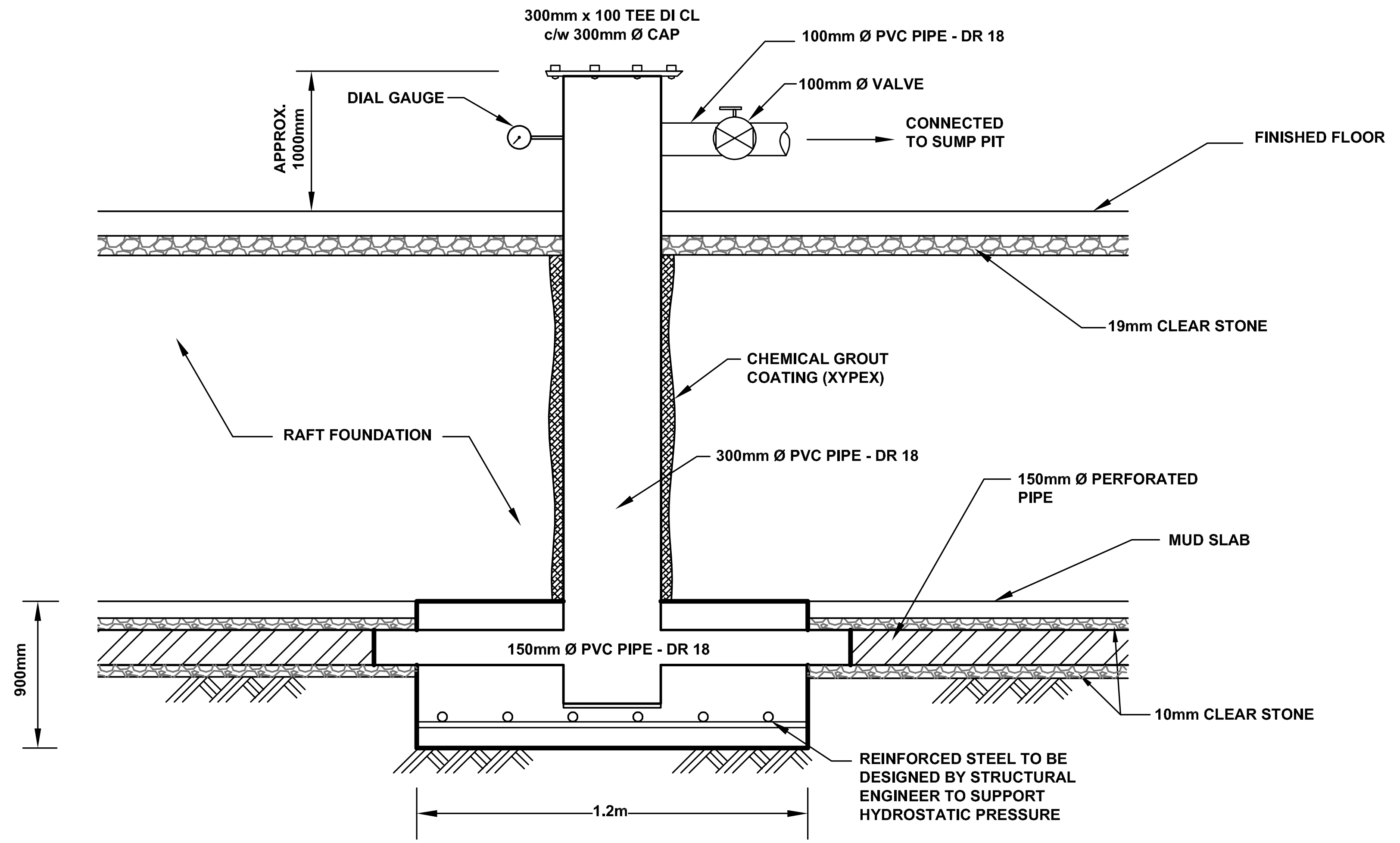


FIGURE 4
PRESSURE RELIEF CHAMBER

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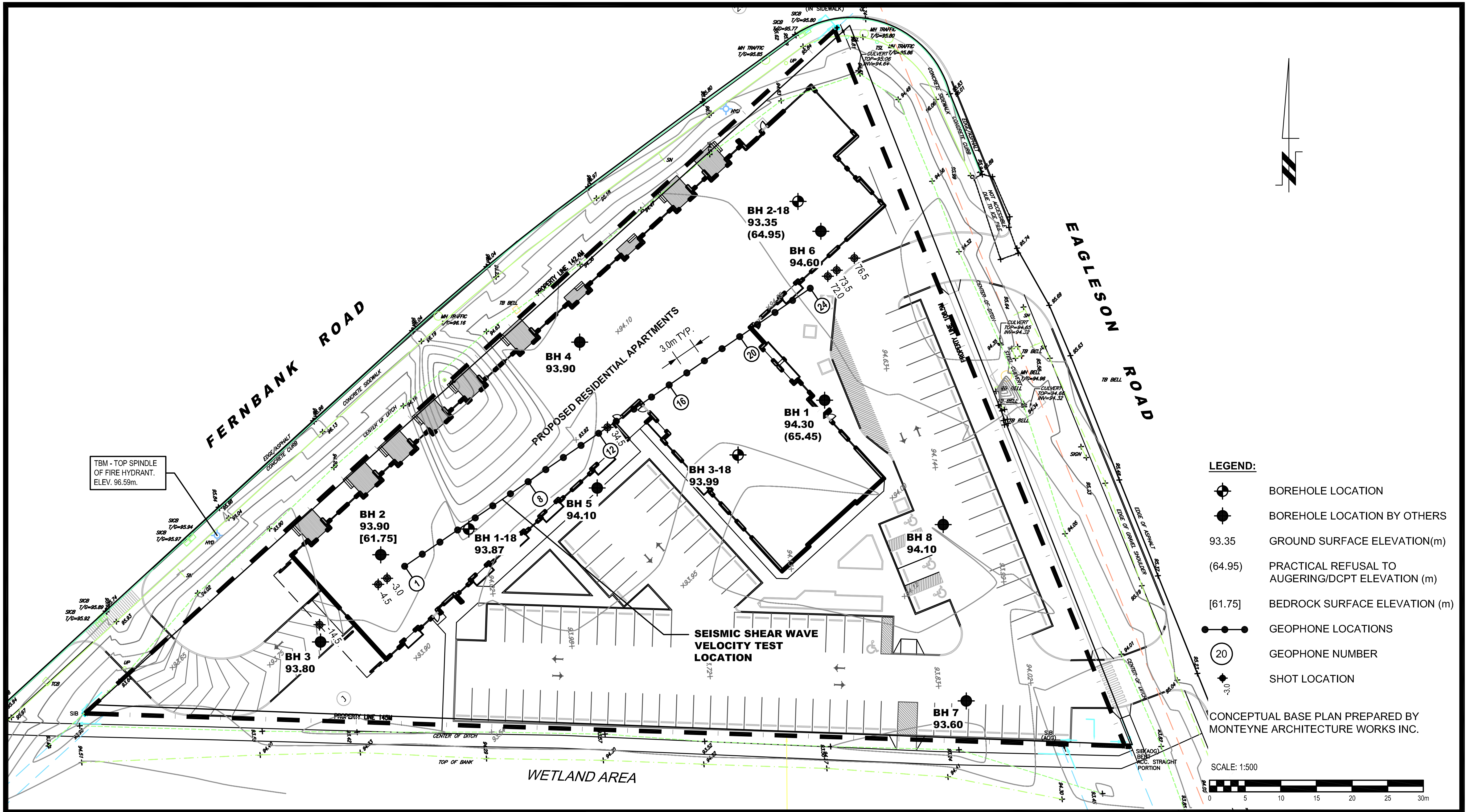
p:\autocad drawings\figures\pg4692 fig2 pressure relief chamber.dwg



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NO.	REVISIONS	DATE	INITIAL
0			

IRONCLAD DEVELOPMENTS INC.
GEOTECHNICAL INVESTIGATION
800 EAGLESON ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

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

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