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Geotechnical Investigation Report

231 Cobourg Street
Ottawa, Ontario

Project No. 94071611

Prepared for

TEN-2-FOUR ARCHITECTURE INC.

January 17, 2017

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LICENSED PROFESSIONAL ENGINEER
F. AGHARAZI
PROVINCE OF ONTARIO

January 17, 2017

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1. INTRODUCTION

AATech Scientific Inc. (ASI) was retained by Ten-2-Four Architecture Inc. to conduct a geotechnical investigation for a proposed new building located at 231 Cobourg St., Ottawa, ON. The site location is shown in Figure 1. This report contains the findings of the geotechnical field investigation and laboratory test results, recommendations and considerations in relation to the design of foundations and the construction of the new building.

The site investigation scope included sub-surface exploratory drilling, field logging, soil sampling, laboratory testing, engineering analysis, and preparation of this report as detailed in the following sections.

Figure 1. Site location (Courtesy of Google Maps)

2. SITE DESCRIPTION AND PROJECT SCOPE

The site is located in Sandy Hill area in Ottawa, Ontario, approximately 270 m west of Rideau River. The area located in a residential community consists of a flat land with no apparent hills nearby. The site is of rectangular shape and was observed to be boarded by street sidewalk on the south and west sides, while the property limits are bound by existing residential dwellings on north and west sides.

It is our understanding that the existing two-story building has been planned to be demolished and replaced by a new three-story building with an approximately similar foot print as the existing structure. The scope of the geotechnical investigation is summarized as follows:

1. Provide necessary arrangements to verify the location of underground utilities with *Ontario One Call.* A gas line was the only underground utility that was reported and was located near the south facade of the building.

2. Conduct geotechnical field investigations including drilling two boreholes with maximum exploration depth of 11.9 m to identify soil type and condition.

3. Perform required laboratory tests on disturbed and undisturbed soil samples obtained during field investigation.

4. Prepare geotechnical report comprising factual findings of filed investigation, geotechnical consideration and recommendation for different foundation supports and temporary excavations.

3. FIELD INVESTIGATION AND LABORATORY TESTING

3.1. FIELD INVESTIGATION

A field investigation carried out on December 13th of 2016, advanced two boreholes referred to as BH-1 and BH-2. A CME-75 track mounted drill rig was used for the investigation by Ohlmann Geotechnical Services (OGS) Inc. Drilling operations, soil sampling, data logging and field testing were supervised by an ASI engineer. Typical testing and sampling procedures were carried out at different depths, including SPT testing, field vane shear tests, pocket penetrometer, as well as disturbed (split-spoon) and undisturbed sample (Shelby tube samples) collection for further laboratory testing.

In order to assess the density of the deeper soil layers as well as to identify bedrock elevation, a Becker penetration test (BPT) was advanced beyond the termination depth of borehole BH-2. Data was recorded for every 300 mm advancing of the casing (blow-count per 300 mm). The raw BPT test results (uncorrected) is included in Appendices. Borehole locations are shown in Appendix 1 and Table 1 provides a cursory summary of the boreholes information.

Table 1: Summary of the borehole information

Borehole No.	Depth (m)	Groundwater depth (m)	Bedrock depth (m)	Northing (m)	Easting (m)
BH-1	9.8	4.2	Not encountered	5030937	447131
BH-2	11.9	4.8	19.5 m (interpreted from BPT results)	5030915	447141

3.2. SOIL PROFILE

The soil profile encountered at the site consisted of surficial topsoil, vegetation roots and organics (about 100 mm), over a thin layer of sand fill underlain by a relatively thick plastic clay deposit down to bedrock at a depth of approximately 19.5 m. Detailed soil stratigraphy and field testing and sampling are summarized in borehole logs, placed in Appendix 2. The soil strata is described in detail below.

3.2.1. Sand fill

A sand fill layer was encountered beneath the surficial top soil layer at both boreholes and extended up to a depth of 1.8 m below grade. The sand fill can generally be classified as poorly-graded (SP), low plasticity, loose to compact, slightly silty and dry. Two SPT N-values were obtained on the sand fill layer indicating N-value ranging between 9 to 10.

3.2.2. Plastic clay

Plastic clay was encountered beneath the sand fill layer and it extended to the end of exploration depth (9.8 m) in BH-1 and to 11.9 m in BH-2. The clay can be classified as high plastic (CH) as illustrated in the plasticity chart (see Appendix 2). It is grey, moist to wet, firm to stiff and contains traces of organic materials.

Based on laboratory and field test results and observations, this clay appears to be medium to high sensitive with sensitivity ratios ranging from about 4 to 9 with an average value of 5. These sensitive marine clays are common to the region due to their deposition history, and are known as Champlain Sea clays or “Leda” clays. Based on CFEM (2006, Errata) very sensitive clays (or quick clays) are defined by having a high ratio (over 16) between the measured undisturbed and disturbed undrained shear strength following standardized remolding effort.

The moisture content of the clay samples obtained from elevations above the groundwater table varied between 55% to 59% with an averaged moisture content of 57%; while the moisture content of the samples collected from below the groundwater elevation varied between 65% to 81% with an average value of 71%. Pocket penetrometer (PP) test on the wet clay samples generally ranged between 10 kPa to 65 kPa, while moist clay samples obtained just below the sand fill layer presented higher PP values ranged from 90 kPa to 270 kPa. Undrained shear strength (S_u) values were obtained from field vane per ASTM D2573-15. S_u values ranged from 41 kPa to 101 kPa with an average value of 65 kPa. S_u values obtained from field vane on remolded samples varied between 8 kPa to 22 kPa with mean value of 13 kPa.

3.2.3. Bedrock

Bedrock was not encountered in the drilled boreholes. As noted earlier, a Becker penetration test was advanced at the end of termination of borehole BH-2 to locate the bedrock elevation and soil consistency at deeper elevations. High driving resistance was noted below depth of 18.3 m, while complete refusal was observed at depth of 19.5 m. Based on ASI experience in Ottawa region the refusal depth can be considered as bedrock with a thin layer of weathered bedrock (or clay till) overlain.

The rock type in the study region can be classified as dark grey almost dark limestone according to available publications.

3.3. GROUNDWATER

Groundwater level was noted during drilling of boreholes BH-1 and BH-2 at depths of approximately 4.3 m and 4.9 m below existing grade, respectively. Soil samples retrieved from the split-spoon were observed quite wet below the water table, compared to that of above water table. No standpipe piezometers were installed to monitor the long-term groundwater level. The groundwater level noted during exploration can be considered as stabilized condition at this site; however, higher groundwater elevation should be expected in response to seasonal fluctuation such as heavy rainfall and melted snow. To account for such circumstances, groundwater depth of 3.0 m may be used for design purposes.

3.4. LABORATORY TESTING

Collected samples from the split-spoon were used for soil classification testing (water content and Atterberg limits). Undisturbed samples extracted from the Shelby tubes were subjected to consolidated drained direct shear tests in order to determine the shear strength property of the plastic clay layer. A summary of the laboratory tests and referenced standard is provided in Table 2. Complete results of laboratory tests are provided in Appendix 2.

Table 2. Summary of laboratory tests

Test	Standard	Sample condition	Number of samples
Natural water content	ASTM D4959-16	Disturbed	5
Atterberg limits	ASTM D4318-10	Undisturbed	5
Direct shear	ASTM D3080-11	Undisturbed	1
Sulphate concentration	ASTM C1580-15	Disturbed	1

3.4.1. Atterberg Limits

Atterberg limits were measured in the laboratory for different clay samples obtained from boreholes BH-1 and BH-2, between the depths of 2.4 m and 5.6 m in BH-1 and 2.4 m to 8.8 m in BH-2. There is a consistency between the values obtained from the two boreholes. The plasticity index (PI) ranges from 45 to 58 with a mean value of 52, while liquid limit (LL) varies between 73 and 88 with an average value of 82. In-situ gravimetric water content values ranged from 55 % to 81 % with an average value of 65 %. The proximity between natural water content and liquid limit is an indication of the soil sensitivity. A summary of the test results are presented in Appendix 3.

3.4.2. Direct shear test

Direct shear tests were conducted on samples extracted from Shelby tube (ST-1) of borehole BH-1 at depths of approximately 4.1 m below grade. Samples were subjected to consolidated drained direct shear tests. Three samples were consolidated under normal stress of 63 kPa, 117 kPa, and 171 kPa for each test, then sheared in a direct shear test equipment at a constant displacement rate of 0.03 mm/min. Shear rate was adopted slow enough to allow pore water pressure dissipation at the shear surface, simulating drained conditions in the soil. Mohr-Coulomb failure envelope was plotted with respect to the peak shear stress values obtained from the shear stress versus shear strain curves. The internal friction angle and cohesion is calculated as 13 degree and 23 kPa, respectively. Detailed results are provided in Appendix 3.

4. GEOTECHNICAL RECOMMENDATIONS AND CONSIDERATIONS

4.1. SITE PREPARATION AND DRAINAGE

The present elevation and grade of the site are estimated to be near the design specifications with only minor excavation/backfilling and fine grading work being required at surface level. It is our understanding that the new building will be built on the old building footprint and therefore existing basement area may still be served for construction of new basement; however, some additional excavation may be required to accommodate new basement elevation as well as frost depth requirements.

Appropriate surface drainage should be provided during and after construction, and be consistent with the existing drainage regime of the site and adjacent areas. It is recommended that a minimum surface gradient of 2% facing away from the proposed structures footprint is provided so as to promote surface runoff without water accumulation.

Foundation drains should be provided at the base of each below-grade wall. It consists of a perforated PVC drainage pipe or tiles with minimum 150 mm diameter, surrounded by at least 300 mm of free drainge materials. Filtration membrane consisting of a layer of geotextile fabric should be placed between the drain pipe and free drainge zone; as well as between the free drainge zone and native clay.

The exterior of foundation walls below-grade elevation should be damp proof in order to avoid moisture and water penetration into the basement wall.

4.2. FROST PENETRATION DEPTH

The soil type near the ground elevation encountered in boreholes was sand fill. The qualitative frost susceptibility as recommended by CFEM (2006) was employed for frost design soil classification. In this classification soils are listed in four categorizes, F1 to F4, in approximate increasing order of frost susceptibility and loss of strength during thaw. The sand fill encountered at this site is classified to have moderate to high frost susceptibility (F3). The native plastic clay encountered at this site can also be classified as moderate to high frost susceptibility (F3).

Based on the graphs published by Ontario Ministry of Transportation and Communications (MTC), the design freezing index for Ottawa area has been estimated as 1,016 degree-days Celsius (1,829 degree-days Fahrenheit) based on the period from 1931 to 1970. Consequently, the depth of frost penetration in the city of Ottawa is estimated about 1.8 m for unprotected and snow free surface. Presence of vegetation and/or snow may reduce the depth of frost penetration.

5. SHALLOW FOUNDATION

Shallow foundations in form of spread and strip footings founded on native undisturbed clay are considered suitable for lightly loaded foundation. Shallow foundation should be placed at or below a depth of 1.8 m from the existing ground surface for unheated structures and 1.4 m for continuously heated structures.

The foundations may be placed at depths shallower than the depth of seasonal frost action, if horizontal insulation around foundation walls are provided. In this case, the foundation should be underlain by 100 mm rigid polystyrene (Styrofoam HI, or equivalent) extending 2.0 m from the face of the walls. A minimum burial depth of 300 mm should be considered. The insulation should be sandwiched between two layers of bedding sand and should be sloped away from the face of the wall with minimum 1% slope.

All footings should be founded on undisturbed native firm to stiff plastic clay encountered in this site. A minimum layer of 250 mm thick, 25 mm minus crushed gravel bedding such as OPSS approved Granular A (see Appendix 4) should be placed below the footings. The crushed gravel bedding should be compacted uniformly to a minimum 98% of the Standard Proctor Maximum Dry Density (SPMDD). A factored ultimate soil bearing pressure (ULS) of 115 kPa and a serviceability limit state bearing pressure (SLS) of 75 kPa may be used for the design of the footings to be placed on approved native soils. This bearing pressure may be reassessed as necessary based on the actual soil conditions encountered during construction. Also, if the footings are subjected to eccentric loading, then equivalent footing width should be considered as per CFEM (2006). Estimated settlements for footings designed based on the SLS bearing pressure are expected to be less than 25 mm.

It is our understanding that some parts of the new footings of the proposed building may not be located at the exact location of the existing footings. In such cases, some differential settlements are expected to occur between the newly loaded and pre-loaded areas. It is recommended to provide adequate reinforcements in the basement walls as well the underneath footing to accommodate the induced moment and displacement as a result of such differential settlements.

Where local soft clay zones or unsuitable soils (fills) are encountered in the footing elevation, it is necessary to replace the soft materials with a compacted Engineered fill. All disturbed materials should be removed prior to footing placement. Engineered fill consisting of approved, well-graded, sandy gravel should be compacted to minimum 98 percent of the SPMDD, as verified by compaction tests.

Bearing soil shall be protected from excessive wetting, weathering and frost action during footing construction. The footing excavations should be inspected by qualified geotechnical personnel to ensure that the footings are located in suitable clay soils. When bearing soils have been approved, the footing concrete shall be placed as soon as possible.

5.1. DEEP FOUNDATION

Deep foundation system such as skin friction cast-in-place (CIP) and helical (screw) piles may be employed to carry the large loads exerted by the proposed structure. Driven steel pipe piles and H-piles are considered to be impractical due to pretense of vibration sensitive residential old buildings in the immediate vicinity of the proposed structure.

5.1.1. Skin friction cast-in-place piles

Deep foundations in the form of cast-in-place skin friction piles is considered suitable for the proposed structure. Skin friction on the pile shafts for the first 2.0 m below grade must not be included in pile capacity calculation due to presence of fill material and frost penetration depth. A minimum pile embedement of 6 m is recommended to resist frost action. End bearing resistance should not be included in the design calculations of a friction pile since a dry, clean base might not be achieved during pile construction. A minimum shaft diameter of 400 mm is recommended to prevent voids from forming during concrete pouring. A minimum spacing of 3 times the shaft diameter between the adjacent piles should be applied. Skin friction has to be reduced and re-calculated as a group of piles if this is not the case. Due to presence of groundwater at shallow depth, temporary casing should be available on site to eliminate the water intrusion into the drilled holes during construction of CIP concrete piles. The concrete must be poured immediately after the drilling to reduce the potential for sloughing or seepage. A minimum percentage of longitudinal reinforcement (0.5% of the sectional area of the pile shaft) is required for the upper 6 m pile length to resist potential uplift forces on the pile due to frost action.

The recommended skin friction values for the design of CIP concrete piles are provided in Table 3. The design values are estimated based on the guidelines provided by CFEM (2006). A geotechnical resistance factor of 0.4 and 0.3 should be applied to the recommended values in Table 3, for the piles subjected to axial compressive and uplift loading, respectively.

Table 3. Recommended Parameters for Cast-in Place Concrete Piles

Soil Unit	Depth Below Grade (m)	Unfactored ULS Skin Friction (kPa)	Unfactored ULS End Bearing (kPa)
Sand fill	0 - 2	0	-
Native clay	2 - 12	40	-

5.1.2. Helical (Screw) piles

Helical piles are typically designed and installed by specialty contractors. For preliminary design purposes, the ultimate capacity of an end-bearing helical pile can be evaluated as the sum of the capacities of each individual helical plate(s). Therefore, the helical pile capacity is determined by calculating the unit bearing capacity of the soil and applying it to the individual helical plate(s) areas, as follows (CFEM, 2006):

$$Q_h = A_h(s_u N_c + \gamma D_h N_q + 0.5\gamma B N_\gamma)$$

Where:

Q_h = Individual helix ultimate bearing capacity

A_h = Projected helix area

S_u = Undrained shear strength of the soil

γ = Unit weight of the soil

D_h = Depth to helical bearing plate

B = Diameter of helix bearing plate

N_c, N_q, N_γ = Bearing capacity factors for local shear conditions

The total ultimate helical pile capacity for piles with more than one helix, Q_t, may be calculated using below equation provided that at least three times of the helix diameter spacing is provided between the adjacent helices :

$$Q_t = \Sigma q_h$$

An undrained shear strength of 65 kPa and unit weight of 17.0 kN/m^3 may be used in bearing capacity calculation for preliminary design purposes. The minimum embedment depth of 1.8 m plus one helix diameter should be respected. Ad-freeze stresses of 100 kPa acting along the pile shaft should be considered within the depth that would be subject to frost heave, which are 1.4 m and 1.8 m below-grade for piles supporting heated and unheated structures, respectively. Helical piles should be spaced at least three time of the helix diameter center to center. Minimum factor of safety of 2.5 and 3.0 should be used for calculation of allowable bearing capacity of the helical piles subjected to axial compressive and tension loads, respectively.

5.2. CONCRETE SLAB-ON-GRADE

An non-structured slab-on-grade is feasible for the proposed construction. It is expected that any slab-on-grade will be sited on the native clay encountered at this site. It should be installed on a leveled layer of 200 mm thick, well-graded, 25 mm minus crushed gravel bedding such as OPSS approved Granular A (see Appendix 4). The crushed gravel should be compacted uniformly throughout the area of the slab. The crushed gravel bedding should be compacted uniformly to a minimum 98% of the SPMDD, while maintaining a moisture content at placement of ±3% of its optimum moisture content (OMC). A poly vapour barrier should be installed between the above mentioned gravel bedding and the floor slab according to its manufacture specifications.

Any soft or loose material should be sub-excavated and replaced by well compacted granular material. Before gravel placement the condition of sub-grade should be inspected by a qualified geotechnical engineer.

5.3. SHORING AND TEMPORARY EXCAVATIONS

Any excavation more than 1.2 m should be properly sloped or temporarily supported by an adequate shoring system. No open excavation (unsupported) is allowed within the sand fill layer encountered within the first 1.8 m below grade. Temporary shallow excavation executed in the native clay material should be sloped no steeper than 2.0 H:1.0 V. Based on groundwater observation during drilling works, it is expected that all the excavations less than 3.0 m depth will be located above water table. Therefore, no significant seepage is expected within the excavation area; however, control of surface water should be implemented by directing water away from the excavations.

Temporary shoring such as soldier pile and lagging system is considered to be feasible for this site up to 3.0 m excavation. Active earth pressure may be used for flexible walls with allowable free movement of the top of the wall up to 0.002 times of the shoring height. It is recommended to use at-rest soil pressure for rigid walls. Triangular earth pressure distribution may be used for any shallow shoring system. Any additional surcharge loads induced by machinery, traffics, heavy equipment and live loads close to the shoring line should be added in applied earth pressure on the shoring system. A summary of the coefficient of lateral earth pressure is recommended in Table 4.

Lateral earth pressure for a concrete wall may be calculated by using following equation. This equation only considered the earth and surcharge pressure. It should be noted that full drainage of the wall is required, so that no hydrostatic pressure is developed behind the wall.

$$P=K(\gamma.H+q)$$

where,

P = Lateral earth pressure (kPa)

K = Coefficient of earth pressure as per Table 4

γ = Unit weight of soil as per Table 4 (kN/m^3)

H = Depth below ground surface (m)

q = surface surcharge (kPa)

Table 4: Coefficient of lateral earth pressure

Soil type	K_a	K_0	K_p	γ (kN/m^3)
Compacted granular fill	0.31	0.47	3.25	19.5
Sand fill (loose to compact)	0.40	0.58	2.40	17.5
Native clay (firm to stiff)	0.49	0.66	2.05	17.0

Any backfill behind the concrete walls should be placed gradually with help of man-operated compaction equipment. Concrete of the foundation walls should reach to at least 75% of the 28 days design compressive strength. Proposed lateral earth pressure for compacted granular material is based on the compaction of 98% of SPMDD. Higher lateral earth pressure should be used if above noted compaction criteria is not achieved.

5.4. CEMENT REQUIREMENTS AND CORROSION POTENTIAL

One soil sample from the native clay layer encountered at borehole BH-1 at an average depth of 2.4 m below grade, was tested for water soluble sulphate concentrations in accordance with CSA A23.2-3B7. No significant amount of sulphate in the soil sample was measured, hence that the concrete in contact with the soil encountered at this site is not subjected to sulphate attack, as per CSA A23.1-09, Table 3. It is recommended that a soil sample will be obtained from the foundation elevation at the time of construction and subjected to sulphate test in order to verify the obtained results and related exposure class

A similar soil sample was subjected to electrical resistivity test. The electrical resistivity was measured 2900 ohm-cm indicating that the steel structures with exposed surface in contact with the clay soil encountered at the site can be subjected to a moderate corrosion potential, as per AASHTO classification criteria.

Any imported soils should be tested with regard to water soluble sulphate concentration and associated sulphate exposure level should be determined accordingly.

5.5. PAVEMENT RECOMMENDATIONS

It is our understanding that the west area of the proposed building might be used as a parking lot as well as north side areas adjacent to the building may be surfaced with concrete pavement. The parking lot may be subjected to light to medium heavy duty traffic loads.

Any surficial organic and unsuitable materials should stripped off from the intended pavement area. The pavement materials should be placed on approved subgrade materials. The subgrade may consist of sand fill and/or native clay. The exposed subgrade should be proofrolled and compacted to a minimum 98 % of SPMDD. Any unsuitable or weak materials encountered at the surface of subgrade must be further excavated and replaced by appropriate compacted materials.

OPSS approved Granular B Type I (sand and gravel), placed in maximum 300 mm loose lifts and compacted to a minimum 98 % of SPMDD, can be used to raise the subgrade elevation, if needed.

The base and subbase course must be compacted to a minimum 98 % of SPMDD. Base gravels should consist of approved, well graded sandy gravel meeting the recommended gradation as shown in Appendix 4.

The asphalt components must be placed and compacted to 93 % of the Maximum Relative Marshall density of the mix design being used. The finished pavement must be sloped to provide adequate surface drainge toward the catch basins.

In the absence of any specific design data, the following pavement guidelines (Table 5) are suggested for parking/driveway areas.

The quality and performance of any pavement construction depends upon adequate subgrade preparation. Continuous supervision of pavement construction by a qualified engineer is recommended.

Table 5 - Preliminary recommended pavement thickness

Material	Light duty traffic (mm)	Medium duty traffic (mm)	Heavy duty traffic (mm)
HL 3 surface asphalt	30	35	40
HL 8 surface asphalt	40	70	80
Granular Basecourse OPSS Granular “A” (25 mm)	150	150	150
Granular Subbase OPSS Granular “B” (50 mm)	200	300	400

5.6. SEISMIC CONSIDERATIONS

The seismic response of the site is classified according to the National Building Code of Canada 2010 (NBCC), which categorizes the soil conditions into six types - Class ‘A’ to ‘F’. This classification is based on the average shear wave velocity, energy-corrected SPT N values, or undrained shear strength over the top 30 m of the soil profile.

Based on the soil conditions encountered at the boreholes and the geology of the study area, the project site is classified as Site Class D in accordance with the site classification as per Table 4.1.8.4A of the National Building Code (NBCC 2005).

5.7. CONSTRUCTION INSPECTION

It is recommended that foundation installations, pavement construction and other earthworks are executed under a full-time supervision of a qualified geotechnical personnel to confirm that they are installed in competent bearing material and that the soil condition is similar to those that have been assumed for the design.

6. LIMITATIONS

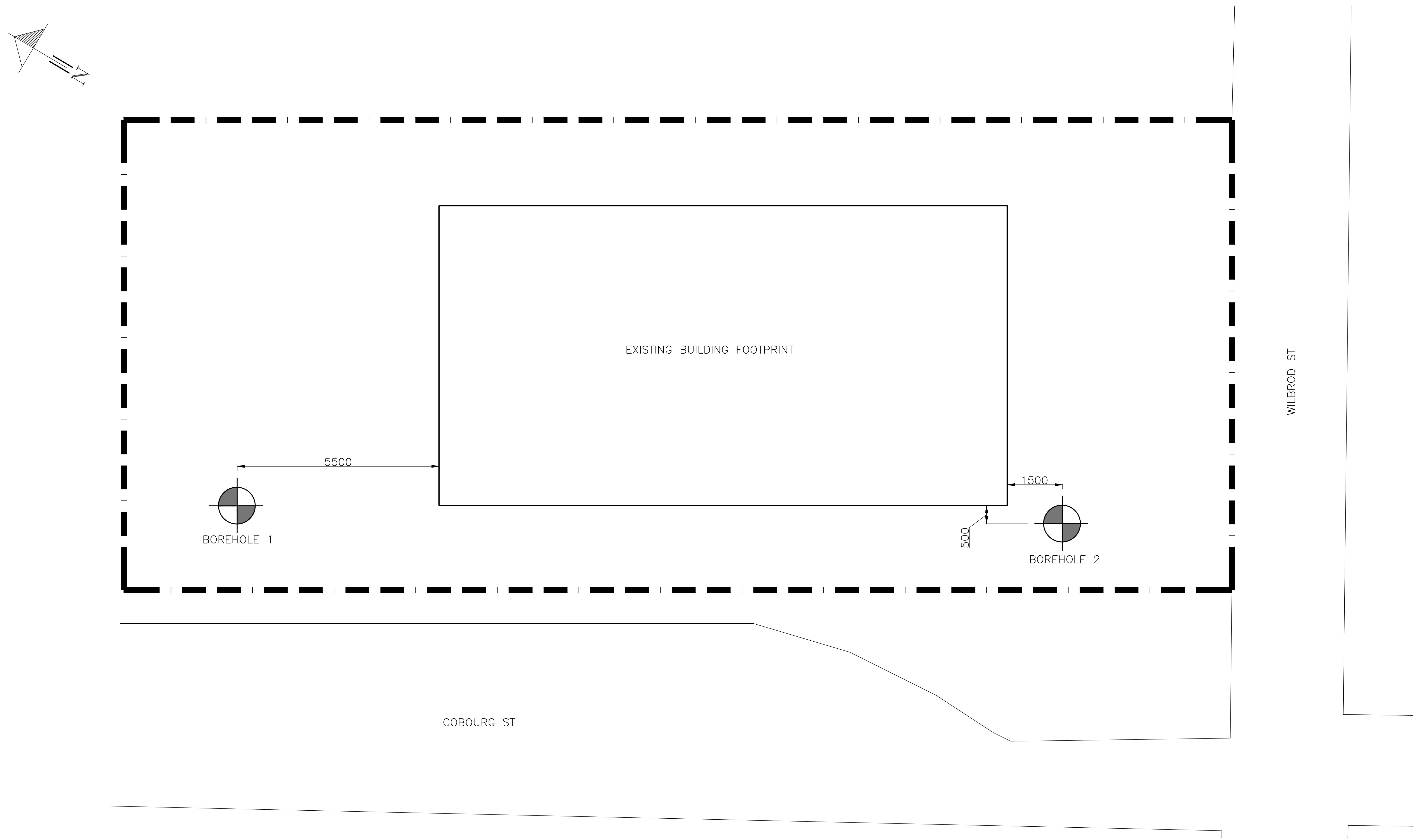
This report was prepared for the sole purpose of proposed development at the specified location. The report was prepared in accordance with generally accepted geotechnical engineering principles and practices, and based on subsurface conditions encountered at this site. The analyses, recommendations and considerations are based on the subsurface soil and groundwater conditions encountered at the borehole locations were considered to be representative of the site conditions at the time of preparation of this report. Should site or soil conditions be discovered at the time of construction differ from those presented herein, ASI should be notified to reassess the soil condition and to provide additional recommendations where deemed necessary.

7. REFERENCES

- National Building Code of Canada (NBCC, 2015), National Research Council of Canada (NRCC).
- Canadian Foundation Engineering Manual (CFEM, 2006), 4th Edition, BiTech, Vancouver, British Columbia, Canada.

Appendix 1

Boreholes Location Plan



LEGEND

— — · — PROPERTY LIMIT

Dimensions in millimeters

NO.	DATE	REVISION	BY	APVD

CLIENT: **TEN 2 FOUR ARCHITECTURE INC.**

PROJECT: **231 COBOURG STREET**
Ottawa, Ontario

TITLE: **BOREHOLES LOCATION**

AATech Scientific Inc.
589 RIDEAU St., SUITE 212,
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TELEPHONE: 613-789-6333 FAX: 613-789-5333

DESIGNED BY	F Agharazi	DRAWING NO.	ASI-94071611-01
DRAWN BY	R. Guertin	SCALE:	1:100
CHECKED BY	F Agharazi	DATE	Jan 17, 2017
		ISSUED	

Appendix 2

Geological Borehole Logs

589 Rideau St., Unit 212
Ottawa, ON, K1N 7G2
(613) 789-6333

Borehole Log

Project / Projet:	231 Cobourg St., Ottawa, ON	DRILLING INFORMATION	Borehole No. / Forage No.:	BH-1
Project No. / Projet No.:	94071611	Downing, Truck - Mounted Drill Rig	Elevation / Niveau:	
Client:	TEN-2-FOUR INC.	CME 75	Depth / Profondeur:	9.8 m
Logged by / Enregistré par:	A.G.		Groundwater depth:	4.2 m
Reviewed by / Inspecté par:	F.A.		Prof. de nappe phréatique:	
Date:	DEC. 13, 2016		Weather / Météo:	Sunny,-2 degree Celsius

Undrained Vane Shear Strength (kPa) 0–150; Remolded Vane Shear Strength (kPa) 0–150; SPT (N / 0.3m) 0–25; Pocket Penetrometer (kPa) 0–200

Depth (m)	Soil Description	Sample type and No.	PL	WC	LL
0.00–0.10	Grass and organic materials				
0.10–1.80	Sand/Dry/Loose to compact	1.20–1.80 Split Spoon 1-1			
1.80–4.20	Clay/Grey/Moist/Firm to stiff	1.80–2.10			
		2.10–2.70 Split Spoon 1-2	30.9	58.6	88.4
	Stiff	2.70–3.70			
		3.70–4.30 Shelby Tube#1			
4.20–9.80	Clay/Grey/Wet/Stiff	4.30–4.60			
		4.60–5.20 Split Spoon 1-3			
		5.20–5.30	27.8	80.8	82.9
		5.30–5.90 Split Spoon 1-4			
		5.90–7.30			
		7.30–7.90 Split Spoon 1-5			
	Firm	7.90–8.50			
		8.50–9.10 Split Spoon 1-6			
9.80	End of borehole				

Piez./ Water Level m

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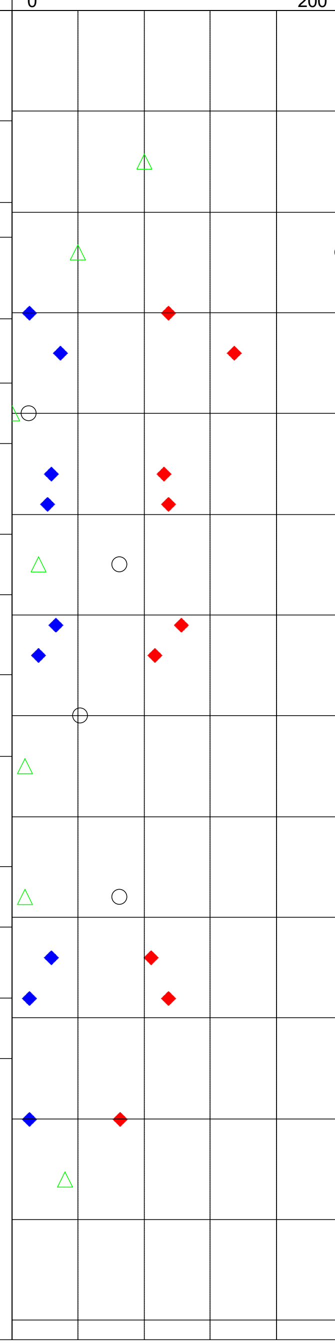
Borehole Log

Project / Projet:	231 Cobourg St., Ottawa, ON	DRILLING INFORMATION	Borehole No. / Forage No.:	BH-2
Project No. / Projet No.:	94071611	Downing, Truck - Mounted Drill Rig	Elevation / Niveau:	
Client:	TEN-2-FOUR INC.	CME 75	Depth / Profondeur:	11.9 m
Logged by / Enregistré par:	A.G.		Groundwater depth:	4.8 m
Reviewed by / Inspecté par:	F.A.		Prof. de nappe phréatique:	
Date:	DEC. 13, 2016		Weather / Météo:	Sunny,-2 degree Celsius

Depth (m)	Soil Description	Sample type and No.	PL	WC	LL
0.00–0.10	Grass and organic materials				
0.10–1.80	Sand/Dry/Loose to compact	1.20–1.80 Split Spoon 2-1			
		1.80–2.10			
1.80–4.80	Clay/Grey/Moist/very stiff	2.10–2.70 Split Spoon 2-2	35.3	54.7	87.4
	Stiff	2.70–3.70			
		3.70–4.30 Split Spoon 2-3			
		4.30–5.20			
4.80–11.90	Clay/Grey/Wet/Stiff	5.20–5.80 Split Spoon 2-4	27	65.4	79
		5.80–6.70			
		6.70–7.30 Split Spoon 2-5			
		7.30–8.50			
		8.50–9.10 Split Spoon 2-6	28.3	67.2	73.1
		9.10–9.80			
		9.80–10.40 Shelby Tube 2-1			
	Firm to Stiff				
11.90	End of borehole				

Undrained Vane Shear Strength (kPa) 0–150; Remolded Vane Shear Strength (kPa) 0–150; SPT (N / 0.3m) 0–25; Pocket Penetrometer (kPa) 0–200



Piez./Water Level m

Appendix 3

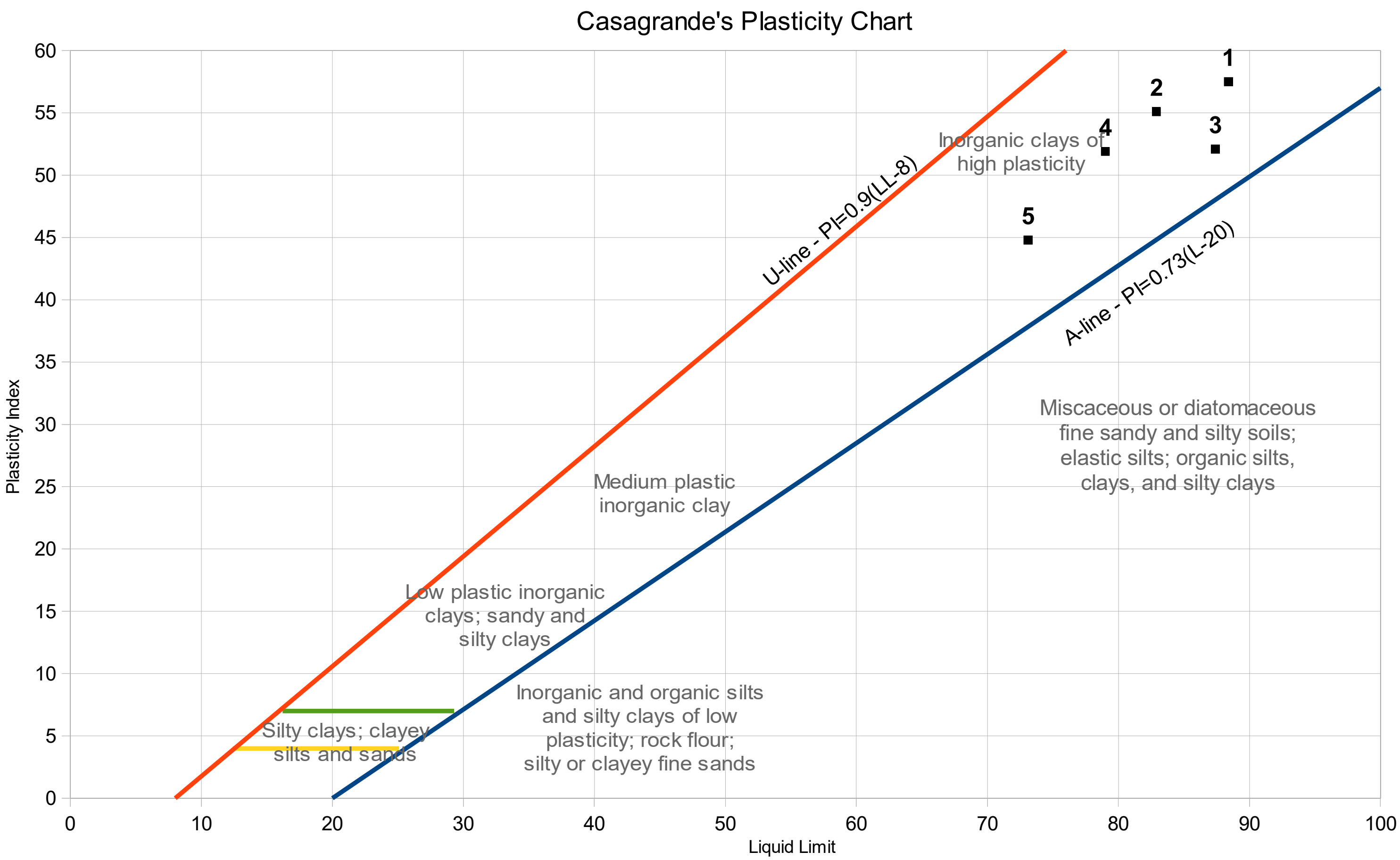
Laboratory Test Results

AATECH AATech Scientific Inc. (ASI)	589 Rideau St, Unit 212 Ottawa, ON, K1N 7G7 (613) 789-6333	**Atterberg Limits Report (LL, PL, PI, In-Situ w%)**	Project:	231 Cobourg St., Ottawa, ON
			Project No.:	940711611
			Client:	Ten-2-Four Architecture Inc.
			Test Carried out by:	N.T.
			Reviewed by:	F.A.
			Date of test:	2016-12-15

Borehole – Split Spoon #	Depth (m)	In situ Water Content w %	Liquid Limit	Plastic Limit	Plastic Index	Soil Classification
BH1 – SS 1	2.4	59	88	31	58	CH
BH1 – SS 2	5.6	81	83	28	55	CH
Borehole – Split Spoon #	Depth (m)	In situ Water Content w %	Liquid Limit	Plastic Limit	Plastic Index	Soil Classification
BH2 – SS 1	2.4	55	87	35	52	CH
BH2 – SS 2	5.5	65	79	27	52	CH
BH2 – SS 3	8.8	67	73	28	45	CH

Figure 1: Atterberg limits test results

AATECH AATech Scientific Inc. (ASI)	**Ottawa (Head Office)** 589 Rideau St., Unit 212 Ottawa, ON - K1N 6A1 Tel: 613.789.6333 Fax: 613.789.5333 Toll Free: 1.877.789.6333 Email: info@aatechscientific.com Web: www.aatechscientific.com	**Calgary** 100, 111 - 5 Avenue SW Suite 312 Calgary, AB - T2P 3Y6 Tel: 403.261.0023 Fax: 403.261.0024 **New York** 26000 U.S RT 11, Suite 194 Evans Mills, NY 13637 Tel: 315.703.9677 Fax: 315.703.9668	**Atterberg Limits Report (LL, PL, PI, In-situ w%)**	
			Project/Projet:	Geotechnial Investigation 231 Cobourg St., Ottawa
			Project No./Projet No.:	94071611
			Client:	Ten-2-Four Inc.
			Test Carried out by:	N.T.
			Reviewed by:	F.A.
			Date of test:	Dec 14-15 2016



Legend

Marker	Sample
1	BH1(7 – 9)'
2	BH1(17.5 – 19.5)'
3	BH2(7 – 9)'
4	BH2(17 –19)'
5	BH2(28 – 30)'

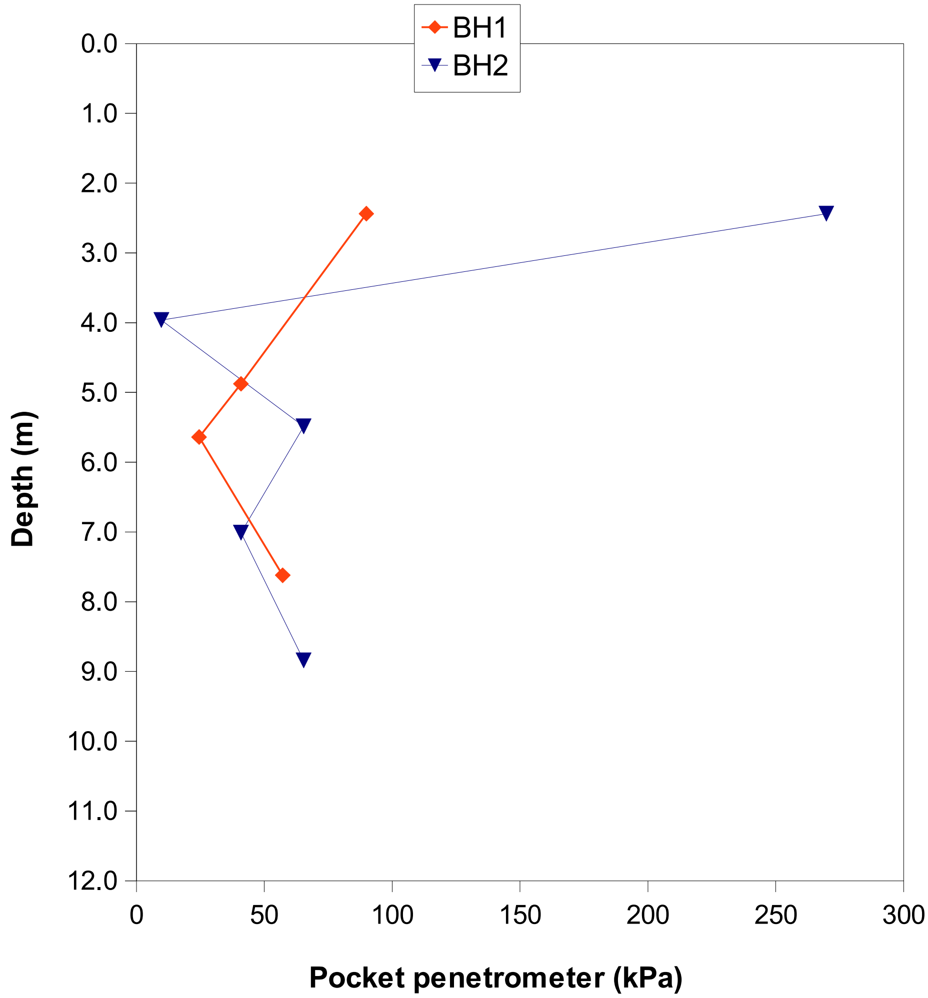


Figure 2: Pocket penetrometer versus depth below grade

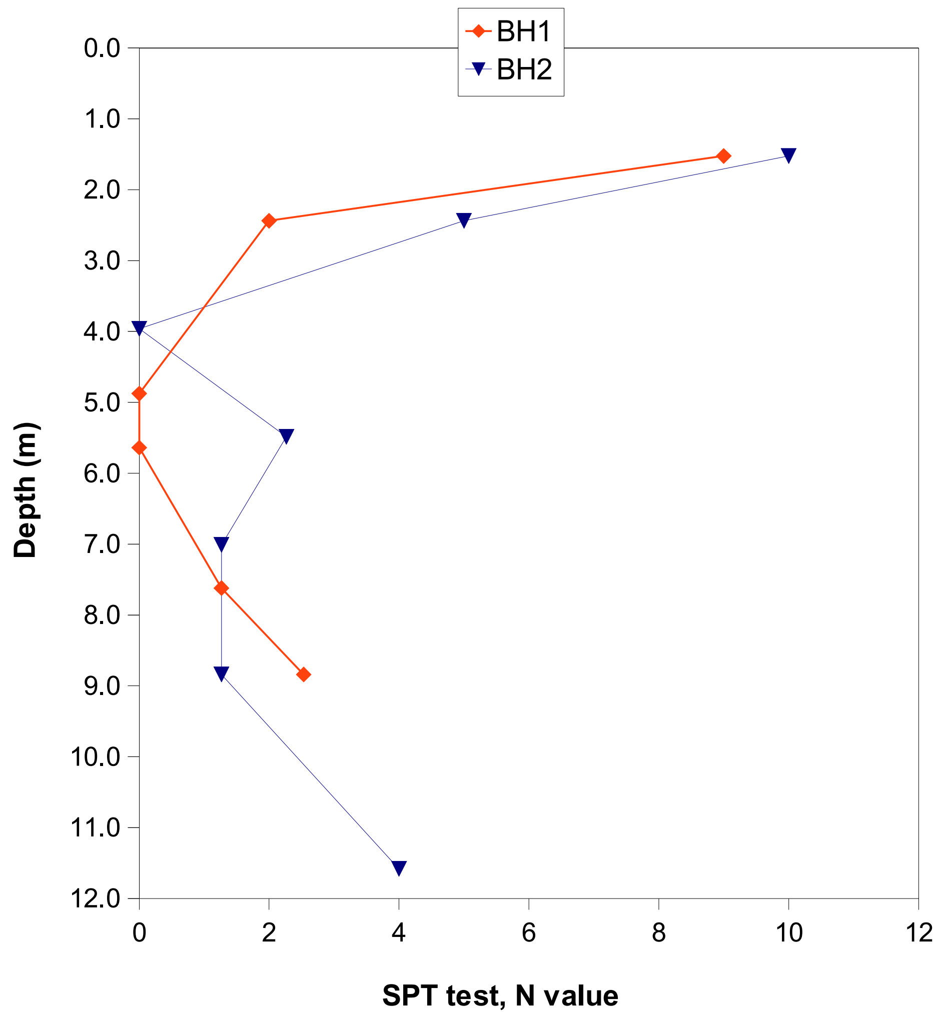


Figure 3: SPT versus depth below grade

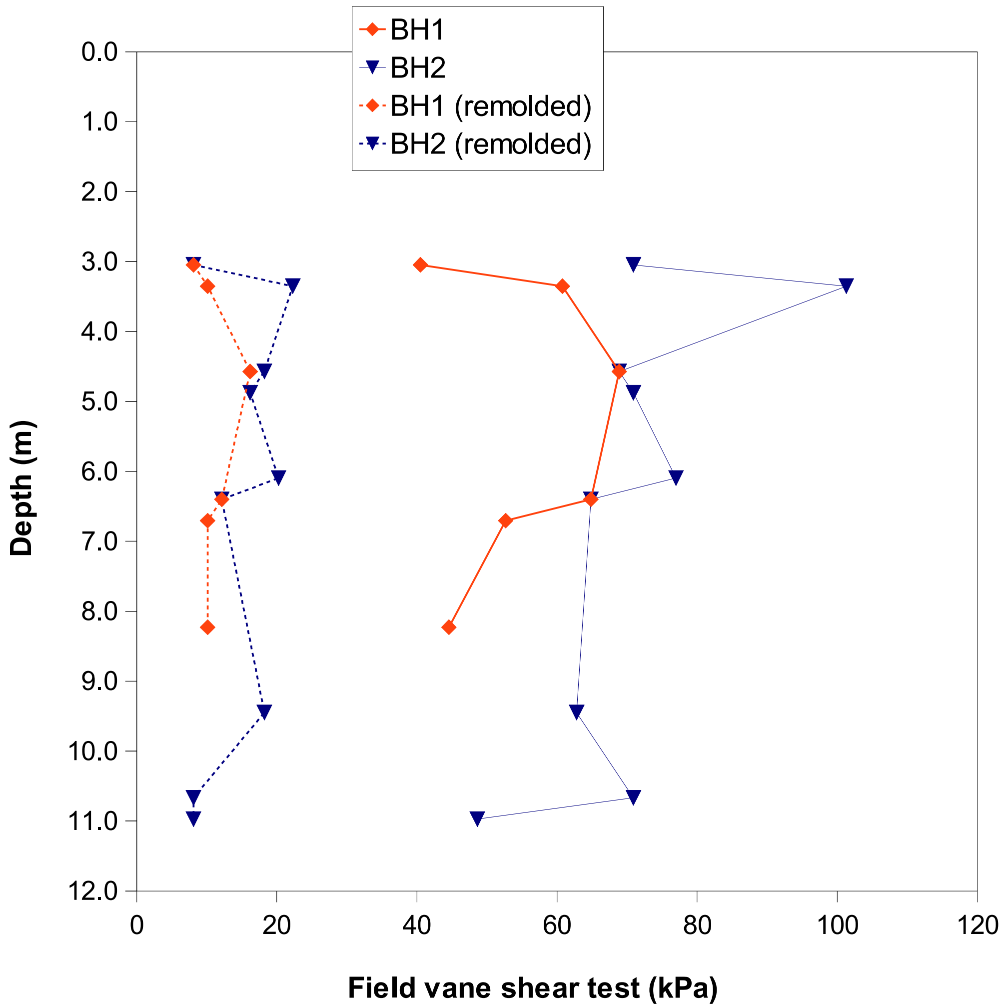


Figure 4: Undrained shear strength versus depth below grade

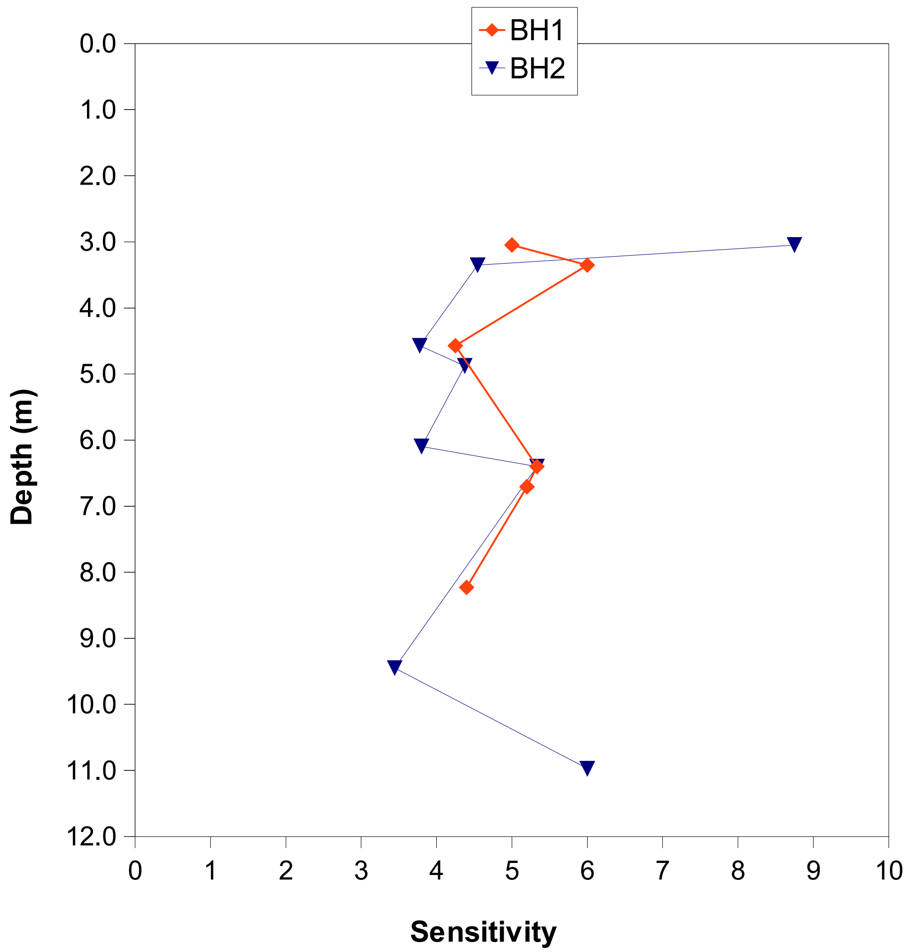


Figure 5: Soil sensitivity versus depth below grade

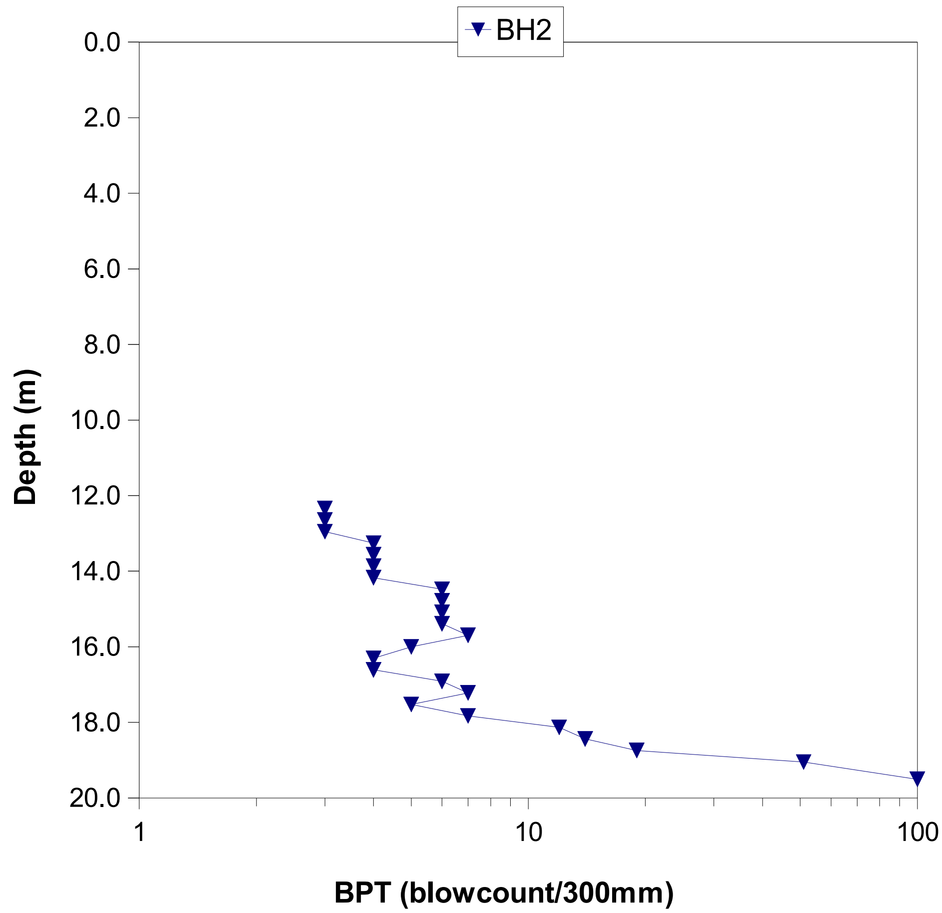


Figure 6: Becker penetration test (uncorrected) versus depth below grade

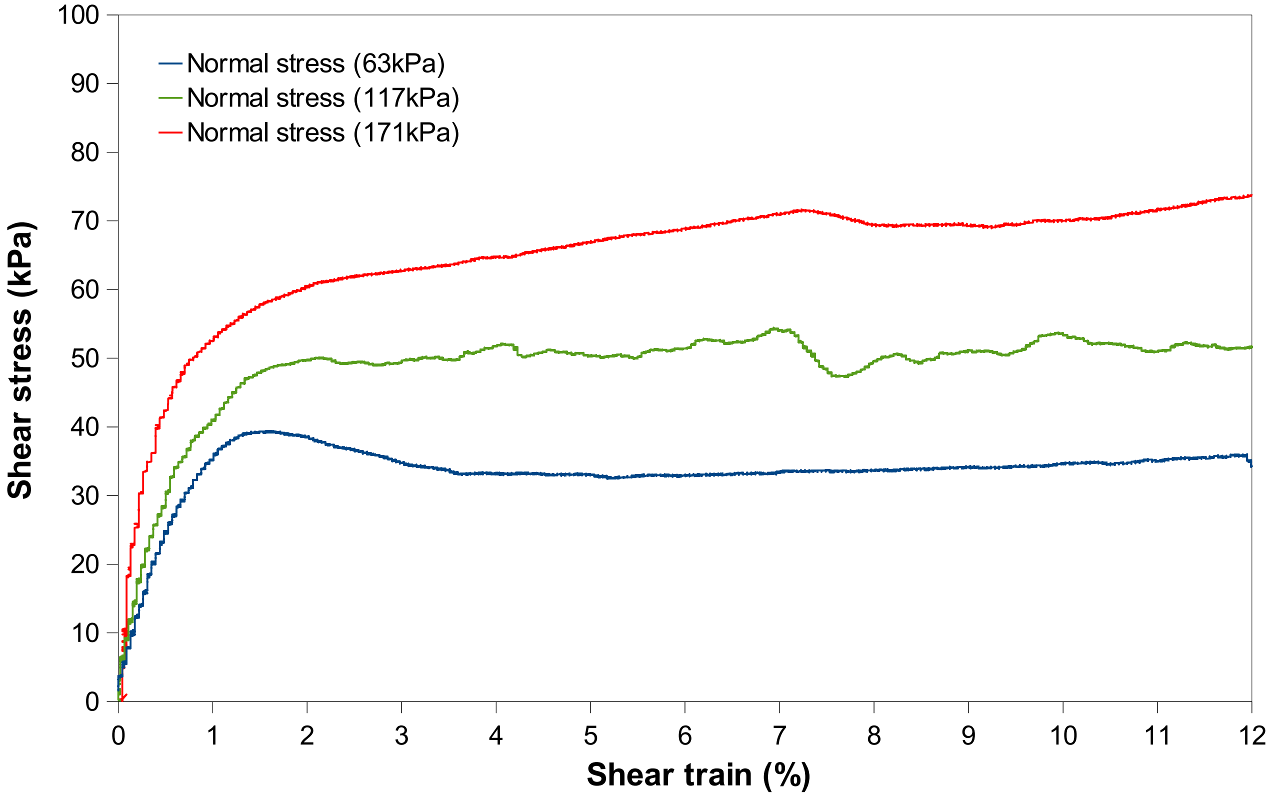


Figure 7: Stress-strain curves obtained from consolidated drained direct shear test

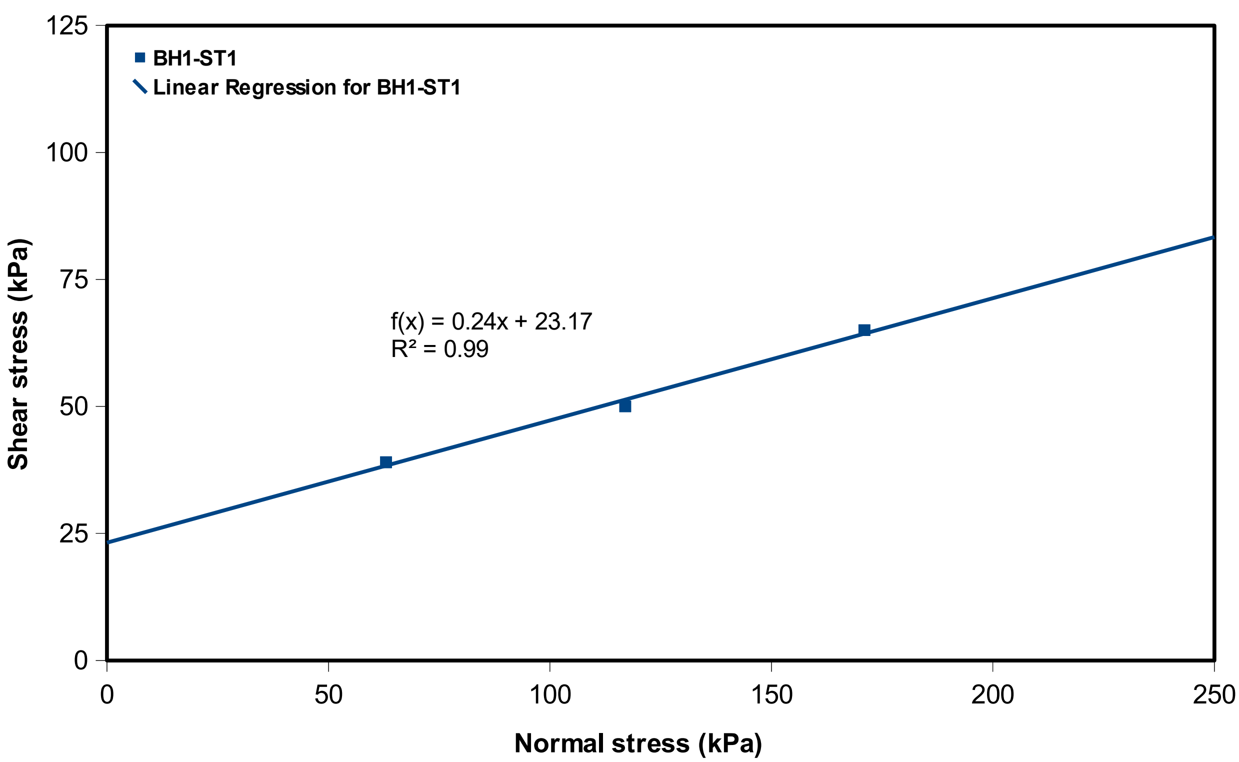


Figure 8: Direct shear test results for sample obtained from BH-1, depth 4.1 m

Maxxam Job #: B706849
Report Date: 2017/01/17

AATech Scientific Inc.
Sampler Initials: TM

RESULTS OF ANALYSES OF SOIL

Maxxam ID		DTA110	DTA110				
Sampling Date							
COC Number		64259	64259				
	UNITS	UHC BH1	UHC BH1 Lab-Dup			RDL	QC Batch
Calculated Parameters							
Resistivity	ohm-cm	2900					4822585
Inorganics							
Conductivity	umho/cm	339				2	4824268
Soluble (20:1) Sulphate (SO4)	ug/g	ND	ND			20	4824286

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
ND = Not detected

Maxxam Analytics International Corporation o/a Maxxam Analytics 32 Colonnade Rd, Unit #1000, Nepean, ON K2E 7J6 Phone: 613 274-0573 Fax: 613 274-0574 Website: www.maxxam.ca

Appendix 4

Typical Gradation Requirements

GRADATION REQUIREMENTS***
(ONTARIO PROVINCIAL STANDARD)

MTO SIEVE DESIGNATION	PERCENTAGE PASSING BY MASS				
	GRANULAR A	GRANULAR B		GRANULAR M	SELECT SUBGRADE MATERIAL
		TYPE I	TYPE II		
150 mm	N/A	100	100	N/A	100
37.5 mm	N/A	N/A	N/A	N/A	N/A
26.5 mm	100	50-100	50-100	N/A	50-100
19 mm	85-100 87-100*	N/A	N/A	100	N/A
13.2 mm	65-90 75-95*	N/A	N/A	75-95	N/A
9.5 mm	50-73 60-83*	N/A	N/A	55-80	N/A
4.75 mm	35-55 40-60*	20-100	20-55	35-55	20-100
1.18 mm	15-40	10-100	10-40	15-40	10-100
300 µm	5-22	2-65	5-22	5-22	5-95
150 µm	N/A	N/A	N/A	N/A	2-65
75 µm	2-8 2-10**	0-8 0-10**	0-10	2-8 2-10**	0-25

* Where the aggregate is obtained from an iron blast furnace slag source.
** Where the aggregate is obtained from a quarry or slag source.
*** MTO Lab Test No. LS 602.

Appendix 5

Site Photos

Photo 1: BH-1

Photo 1: BH-2