# Geotechnical Investigation

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Proposed Gas Bar
2983 Navan Road, Ottawa, ON

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# Executive Summary

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed gas bar to be located at 2983 Navan Road, Ottawa, Ontario (Figure 1). Authorization to proceed with the geotechnical investigation was provided by 12714001 Canada Inc.

The proposed gas bar will consist of a single-story building with no basement. The development will also include a pump island (gas bar) and underground fuel storage tanks that will be set at an assumed 4.3 m to 5.2 m depths below existing grade. It is assumed the underground fuel storage tanks will be designed to withstand hydrostatic uplift pressure by tie-down straps connected to precast concrete deadman anchors set on the founding soil. The underground tanks should also be installed in accordance with the Technical Standards and Safety Authority (TSSA) regulation.

Based on the Grading and Ponding Plan dated August 30,2024 (Revision No. 2) and prepared by J.L. Richards, the finished floor slab of the proposed building will be at Elevation 86.00 m and the elevation of the footings will be at Elevation 83.66 m. The site grade raise will be up to 1.3 m. The proposed development will have paved access roads and parking lots and will be serviced by municipal services.

EXP conducted a geotechnical investigation in 2021 for the residential development site to be located on the large property for which the proposed gas bar forms part of the entire site and is situated at the west end of the large property. From the 2021 geotechnical investigation, two (2) boreholes (EXP and Paterson Group Inc.) are located at the gas bar site and two (2) EXP test holes (borehole and piezocone test hole) are located just on the adjacent property next to the east property line of the gas bar site. The information from these test holes and from laboratory test results from the 2021 EXP geotechnical investigation were used to provide geotechnical engineering comments and recommendations in this report for the design and construction of the proposed gas bar site.

The fieldwork for the geotechnical investigation for the gas bar site and large property east of the gas bar site was completed from April 28 to 30, 2021 and consists of ten (10) boreholes (Borehole Nos. 1 to 10) drilled and advanced by the dynamic cone penetration test (DCPT) to termination depths ranging from 6.2 m to 30.5 m below the existing ground surface. For the proposed gas bar site, Borehole Nos. 9 and 10, located near the east property line of the gas bar site and within the gas bar site were advanced to 6.2 m and 7.0 m termination depths below existing grade. A piezocone test hole with seismic shear wave and pore pressure measurements (SCPTu-9) was located next to Borehole No. 9 located near the east property line of the gas bar site and extends to a termination depth of 32.5 m. The fieldwork was supervised on a full-time basis by a representative from EXP.

The borehole information indicates the subsurface conditions at the gas bar site consists of surficial topsoil underlain by native compact silty sand to a 1.5 m depth followed by a deep silty clay to clay deposit with an upper stiff to very stiff desiccated brown crust underlain by a firm to stiff grey silty clay to clay. The silty clay to clay lowers in strength with depth. The depth of the groundwater table is at 1.3 m depth (Elevation 83.4 m).

Based on the results from the piezocone test hole with seismic shear wave and pore pressure measurements (SCPTu-9) conducted next to Borehole No. 9, the average shear wave velocity of the site ($V_s$) from ground surface to a 30.0 m depth below ground surface is 123 m/s. The Table 4.1.8.4.A in the 2012 Ontario Building Code (as amended May 2, 2019) indicates that for an average Vs value less than 180 m/s, the site classification for seismic site response is **Class E**. Therefore, for design purposes, the site classification for seismic response for the site is **Class E.** The subsurface soils are not susceptible to liquefaction during a seismic event.

The site grade raise of 1.3 m is considered acceptable using approved soil fill, provided the bearing pressure at serviceability limit state (SLS) and factored geotechnical resistance at ultimate limit state (ULS) for footings for the proposed building and for the deadman anchors provided in this report are respected.

Based on a review of the borehole information, it is considered feasible to support the proposed building on strip and spread footings founded on the native compact zone of the silty sand and the very stiff desiccated brown crust of the silty clay to clay. Strip footings having a maximum width of 1.5 m and founded on the native compact silty sand or very stiff brown silty clay to clay to a maximum 1.0/1.1 m depth below existing grade at Elevation 83.66 m may be designed for a bearing pressure at serviceability limit state (SLS) of 75 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 115 kPa. Square pad footings having a maximum width and length of 3.0 m and founded on the native compact silty sand or very stiff brown silty clay to clay to a maximum 1.0/1.1 m depth at Elevation 83.66 m below existing grade may be designed for a bearing pressure at SLS of 90 kPa and factored geotechnical resistance at ULS of 135 kPa. The factored geotechnical resistance at ULS includes a resistance factor of 0.5. The SLS and factored ULS values are valid provided the 1.3 m grade raise is respected and the footings are founded no deeper than 1.0/1.1 m below existing grade on the native soils. If the footing depth below existing grade or the site grade raise will be different than noted above, EXP should be contacted to provide updated SLS and factored ULS values for the footings. Settlements of footings designed for the above SLS bearing pressures are expected to be within the tolerable limits of 25 mm total and 19 mm differential.

It is assumed the proposed underground tanks will be founded at a 4.3 m to 5.2 m depths below existing grade on the firm grey silty clay to clay and will be designed to withstand hydrostatic uplift pressure by tie-down straps connected to precast concrete deadman anchors. The deadman anchors founded on the firm grey silty clay to clay may be designed for a bearing pressure at SLS of 30 kPa and factored geotechnical resistance at ULS of 45 kPa provided the 1.3 m site grade raise is respected and the deadman anchors are founded no deeper than 5.2 m below existing grade on the native soils. If the deadman anchor depth below existing grade or the site grade raise will be different than noted above, EXP should be contacted to provide updated SLS and factored ULS values for the anchors. The total and differential settlements of well designed and constructed deadman anchors placed in accordance with the above recommendations are expected to be less than 25 mm and 19 mm respectively.

The floor slab of the proposed building may be designed as a slab-on-grade founded on a bed of 200 mm thick, 19 mm sized clear stone placed on top of a minimum 300 mm thick compacted Granular B Type II pad placed on the native soils and constructed in accordance with Section 9 of this report. The clear stone would prevent the capillary rise of moisture from the sub-soil to the floor slab. Adequate saw cuts should be provided in the floor slabs to control cracking.

Perimeter drains should be provided for the proposed building. An underfloor drainage system is not required.

Excavation for the construction of the proposed building foundations are anticipated to extend into the silty sand and the silty clay to clay to a 1.0/1.1 m depth and will likely be above or near the groundwater level. The assumed 3.0 m and 5.2 m deep excavations for underground services and the underground fuel storage tanks respectively will likely extend below the silty sand and deeper into the silty clay to clay and are anticipated to be below the groundwater level. The excavations may be undertaken by conventional heavy equipment. All excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA), Ontario Reg. 213/91. Based on the definitions provided in OHSA, the subsurface soils on site are considered to be Type 3 and as such must be cut back at 1H:1V from the bottom of the excavation. Within zones of seepage and below the groundwater level, the excavation side slopes are expected to slough and eventually stabilize at a slope of 3H:1V. If side slopes cannot be achieved due to space restrictions on site such as the proximity of open cut excavations to the property limits, existing infrastructure or to foundations of adjacent existing buildings, excavations would have to be undertaken within the confines of an engineered support system (shoring system).

It is anticipated that the majority of fill required for construction will have to be imported to the site and conform to the Ontario Provincial Standard Specification (OPSS) requirements for Granular B Type II and Select Subgrade Material (SSM).

The above and other related considerations are discussed in greater detail in the attached report.

# 1. Introduction

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed gas bar to be located at 2983 Navan Road, Ottawa, Ontario (Figure 1). Authorization to proceed with the geotechnical investigation was provided by 12714001 Canada Inc.

The proposed gas bar will consist of a single-story building with no basement. The development will also include a pump island (gas bar) and underground fuel storage tanks that will be set at an assumed 4.3 m to 5.2 m depths below existing grade. It is assumed the underground fuel storage tanks will be designed to withstand hydrostatic uplift pressure by tie-down straps connected to precast concrete deadman anchors set on the founding soil. The underground tanks should also be installed in accordance with the Technical Standards and Safety Authority (TSSA) regulation.

Based on the Grading and Ponding Plan dated August 30,2024 (Revision No. 2) and prepared by J.L. Richards, the finished floor slab of the proposed building will be at Elevation 86.00 m and the elevation of the footings will be at Elevation 83.66 m. The site grade raise will be up to 1.3 m. The proposed development will have paved access roads and parking lots and will be serviced by municipal services.

EXP conducted a geotechnical investigation in 2021 for the residential development site to be located on the large property for which the proposed gas bar forms part of the entire site and is situated at the west end of the large property. From the 2021 geotechnical investigation, two (2) boreholes (EXP and Paterson Group Inc.) are located on the gas bar site and two (2) EXP test holes (borehole and piezocone test hole) are located just on the adjacent large property next to the east property line of the gas bar site. The information from these test holes and from laboratory test results from the 2021 EXP geotechnical investigation were used to provide geotechnical engineering comments and recommendations in this report for the design and construction of the proposed gas bar site.

This geotechnical investigation was undertaken to:

a) Establish the subsurface conditions from the four (4) test holes of the 2021 geotechnical investigation located at the gas bar site and near the east property line of the gas bar site,

b) Provide classification of the site for seismic design in accordance with requirements of the 2012 Ontario Building Code (OBC) as amended May 2, 2022 and assess the liquefication potential of the subsurface soils in a seismic event,

c) Comment on grade-raise restrictions for the site,

d) Provide recommendations on the most suitable type of foundations, founding depth and Serviceability Limit State (SLS) bearing pressures and Ultimate Limit State (ULS) factored geotechnical resistances for the proposed building and underground tank deadman anchors as well as anticipated total and differential settlements,

e) Discuss slab-on-grade construction and the need for permanent drainage systems,

f) Discuss excavation conditions and dewatering requirements during construction,

g) Provide pipe bedding requirements,

h) Comment on backfilling requirements and suitability of the on-site soils for backfilling purposes;

i) Provide pavement structure thicknesses for parking lots and access roads,

j) Discuss subsurface concrete and steel requirement; and

k) Comment on tree planting restrictions.

The comments and recommendations given in this report are based on the assumption that the above-described design concepts will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations, or it may require additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

## 2. Site Description

The site is located on the east side of the Brian Coburn Boulevard West and Navan Road intersection in Ottawa, Ontario. The site is densely covered with trees.

Based on a review of the spot elevations of the existing ground surface ranging from Elevation 84.3 m to Elevation 85.7 m, the topography of the site is relatively flat.

# 3. Available Information

The following geotechnical reports were available for use as reference material in the preparation of this geotechnical report. The two (2) geotechnical investigations discuss test hole findings and provide geotechnical engineering comments and recommendations for the proposed development of the gas bar site and at the site of the proposed commercial/residential development located along the east property line of the gas bar site.

- *Geotechnical Investigation, Proposed Residential Development, 2983, 3053 and 3079 Navan Road, Ottawa, Ontario* dated August 19,2021 and prepared by EXP (Project No. OTT-21004743-B0) - The locations of the test holes on the gas bar site and on the adjacent parcel of land near the east property line of the gas bar site (Borehole Nos. 9,10 and SCPTu-9) are shown in Figure 2 and the test hole logs are shown in Appendix A.
- *Geotechnical Investigation, Proposed Commercial Development, Brian Coburn Boulevard at Navan Road, Ottawa, Ontario (Report: PG4415-1 Revision 1)* dated November 13,2018 and prepared by Paterson Group Inc. (Paterson) – The location of the borehole, Borehole No.1, located within the gas bar site is shown on the Borehole Location Plan, Figure 2, and the borehole log for Borehole No. 1 is shown in Appendix B.

# 4. Site Geology

## 4.1 Surficial Geology

The surficial geology map (Map 1506A – Surficial Geology, Ontario-Quebec, Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1982) indicates that beneath any fill material, the site is underlain by off-shore marine deposits consisting of silt, silty clay and clay.

## 4.2 Bedrock Geology

The bedrock geology map (Map 1508A – Generalized Bedrock Geology, Ottawa-Hull, Ontario and Quebec, Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1979) indicates the site is underlain by shale bedrock of the Billings formation.

The drift thickness map (Figure 3 Drift Thickness Trend, Ottawa-Hull Ontario and Quebec Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1979) indicates the trend in the overburden drift thickness or depth to bedrock in the vicinity of the site ranges from approximately 18.0 m to 55.0 m.

# 5. Procedure

## 5.1 Fieldwork

The fieldwork for the geotechnical investigation for the gas bar site and large property east of the gas bar site was completed from April 28 to 30, 2021 and consists of ten (10) boreholes (Borehole Nos. 1 to 10) drilled and advanced by the dynamic cone penetration test (DCPT) to termination depths ranging from 6.2 m to 30.5 m below the existing ground surface.

For the proposed gas bar site, Borehole Nos. 9 and 10 located near the east property line of the gas bar site and within the gas bar site were advanced to 6.2 m and 7.0 m termination depths below existing grade. A piezocone test hole with seismic shear wave and pore pressure measurements (SCPTu-9) was located next to Borehole No. 9 located near the east property line of the gas bar site and extends to a termination depth of 32.5 m. The fieldwork was supervised on a full-time basis by a representative from EXP.

The locations and geodetic elevations of the test holes (borehole and piezocone test hole) were established by a survey crew from EXP and are shown on Figure 2.

Prior to the fieldwork, the locations of the boreholes were cleared of any public and private underground services. The boreholes were drilled using a track mounted drill rig equipped with hollow stem augers operated by a drilling specialist subcontracted to EXP. Standard penetration tests (SPTs) were performed in all the boreholes at 0.75 m to 1.5 m depth intervals and the soil samples were retrieved by the split-spoon sampler. Shelby tube samples of the silty clay soil were retrieved from selected depths in some of the boreholes. The undrained shear strength of the cohesive soils was measured by conducting penetrometer and in-situ vane tests.

A 19 mm diameter standpipe (with slotted section) was installed in Borehole Nos. 9 and 10 for long-term monitoring of the groundwater level. The standpipes were installed in accordance with EXP standard practice and the installation configuration is documented on the respective borehole logs. The boreholes were backfilled upon completion of drilling and the installation of the standpipes.

All soil samples were visually examined in the field for textural classification, logged, preserved in plastic bags and identified. Similarly, the Shelby tube samples were sealed and labelled. On completion of the fieldwork, all the soil samples were transported to the EXP laboratory in Ottawa where they were visually examined by a geotechnical engineer and the borehole logs were prepared and the Shelby tube samples stored.

## 5.2 Laboratory Testing Program

A summary of the soil laboratory testing program completed for the geotechnical investigation of the large property east of the gas bar site and for the gas bar site is shown in Table I.

Table I: Summary of Laboratory Testing Program

| Type of Test | Number of Tests Completed |
|---|---|
| **Soil Samples** | |
| Moisture Content Determination | 54 |
| Unit Weight Determination | 10 |
| Grain Size Analysis | 6 |
| Atterberg Limit Determination | 5 |
| One-Dimensional Consolidation Test | 3 |
| Corrosion Analysis (pH, sulphate, chloride and resistivity) | 5 |

# 6. Subsurface Conditions and Groundwater Levels

A detailed description of the subsurface conditions and groundwater levels for the boreholes located within the gas bar site and on the adjacent property near the east property line of the gas bar site (EXP Borehole Nos. 9 and 10 and Paterson Borehole No. 1) and for the EXP piezocone test hole with seismic shear wave and pore pressure measurements (SCPTu) are given on the attached borehole and test hole logs shown in Appendix A (EXP test hole logs) and Appendix B (Paterson borehole log).

The borehole and test hole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted.

Boreholes and test hole were drilled to provide representation of subsurface conditions as part of a geotechnical exploration program and are not intended to provide evidence of potential environmental conditions.

It should be noted that the soil boundaries indicated on the borehole and test hole logs are inferred from non-continuous sampling and observations during drilling operations. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The “Note on Sample Descriptions” preceding the borehole logs form an integral part of this report and should be read in conjunction with this report.

A review of the three (3) borehole logs (EXP Borehole Nos. 9 and 10 and Paterson Borehole No. 1) and piezocone test hole indicates the following subsurface conditions with depth and groundwater level measurements at the gas bar site. The laboratory test results of samples from boreholes located on the property adjacent to the gas bar site undertaken as part of the 2021 EXP geotechnical investigation are also included in this section of the report, since subsurface conditions are similar at both sites; the gas bar site and the adjacent large property site. The location of all of the boreholes and test hole on the large property adjacent to the gas bar site and the complete version of the laboratory test results are included in the 2021 EXP geotechnical report.

## 6.1 Topsoil

A surficial 200 mm and 300 mm thick layer of topsoil was encountered in all three (3) boreholes.

## 6.2 Silty Sand

Silty sand was contacted beneath the topsoil in the EXP Borehole No. 10 and the Paterson Borehole No. 1 and extends to depths of 0.5 m and 1.5 m (Elevation 84.2 m and Elevation 83.8 m). The N values from the standard penetration test (SPT) of 15 indicates the silty sand is in a compact state.

The results from the grain-size analysis conducted on one (1) sample of the silty sand from Borehole No. 1 located on the large property east of the gas bar site are summarized in Table II.

Table II: Summary of Results from Grain-Size Analysis – Silty Sand Sample

| Borehole (BH) No. – Sample (SS) No. | Depth (m) | Grain-Size Analysis (%) | | | Soil Classification (USCS) |
|---|---|---|---|---|---|
| | | Gravel | Sand | Fines (Silt and Clay) | |
| BH 1 – SS1 | 0.8 – 1.4 | 0 | 84 | 16 | Silty Sand (SM) |

Based on a review of the results from the grain size analysis, the soil sample may be classified as a silty sand (SM) in accordance with the Unified Soil Classification System (USCS).

## 6.3 Silty Clay to Clay

The topsoil and native silty sand are underlain by a sensitive marine clay contacted in all three (3) boreholes at a 0.3 m to 1.5 m depths (Elevation 84.4 m to Elevation 83.8 m). The marine clay consists of an upper desiccated brown silty clay to clay crust underlain by a lower strength un-desiccated grey silty clay to clay.

### 6.3.1 Upper Brown Desiccated Silty Clay to Clay Crust

The upper desiccated brown silty clay to clay crust extends to depths of 1.5 m to 3.7 m (Elevation 83.2 m to Elevation 81.6 m). The undrained shear strength of the crust is 120 kPa indicating a very stiff consistency. From boreholes located on the large property east of the gas bar site and on the gas bar site, the natural moisture content of the brown silty clay to clay is 44 percent to 49 percent and, the natural unit weight of the brown silty clay to clay is 17.1 kN/m3 to 19.1 kN/m3.

The results from the grain-size analysis and Atterberg limit determination conducted on two (2) selected samples of the brown silty clay to clay located on the large property east of the gas bar site is summarized in Table III.

Table III: Summary of Results from Grain-Size Analysis and Atterberg Limit Determination
Brown Silty Clay to Clay Samples

| Borehole (BH) No. – Sample (SS) No. | Depth (m) | Grain-Size Analysis (%) | | | Atterberg Limits (%) | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Gravel | Sand | Fines (Silt and Clay) | Moisture Content | Liquid Limit | Plastic Limit | Plasticity Index | Soil Classification (USCS) |
| BH 1 – SS2 | 1.5-2.1 | 0 | 1 | 99 | 34 | 54 | 21 | 33 | Clay of High Plasticity (CH) |
| BH 7– SS2 | 1.5-2.1 | 0 | 1 | 99 | 44 | 55 | 19 | 36 | Clay of High Plasticity (CH) |

Based on a review of the results of the grain-size analysis and Atterberg limits, the soil may be classified as a clay of high plasticity (CH) in accordance with the USCS.

### 6.3.2 Grey Silty Clay to Clay

The upper brown desiccated silty clay to clay crust in all boreholes is underlain by the un-desiccated grey silty clay to clay contacted at 1.5 m to 3.7 m (Elevation 83.2 m to Elevation 81.6 m). The grey silty clay to clay in Borehole No. 10 contains sand seams. All boreholes terminated within the grey clay to silty clay at 6.2 m to 7.0 m depths (Elevation 78.9 m to Elevation 77.7 m).

The undrained shear strength of the silty clay to clay ranges from 29 kPa to 53 kPa indicating a firm to stiff. The sensitivity values of 5.5 to 8.0 indicate that the clay is sensitive. The grey silty clay to clay has natural moisture contents of 28 percent to 74 percent. Locally in Borehole No. 10, the zone of the silty clay to clay that contains sand seams and has a natural moisture content of 28 percent. From boreholes located on the large property east of the gas bar site, the natural unit weight of the silty clay to clay is 15.0 kN/m3 to 15.3 kN/m3.

The results from the grain-size analysis and Atterberg limit determination conducted on five (5) selected samples of the grey silty clay to clay from Borehole No. 10 located on the as bar site and Borehole Nos. 2,3,4 and 6 located on the large property east of the gas bar site is summarized in Table IV.

Table IV: Summary of Results from Grain-Size Analysis and Atterberg Limit Determination
Grey Silty Clay to Clay Samples

| Borehole (BH) No. – Sample (SS) No. | Depth (m) | Grain-Size Analysis (%) | | | Atterberg Limits (%) | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Gravel | Sand | Fines (Silt and Clay) | Moisture Content | Liquid Limit | Plastic Limit | Plasticity Index | Soil Classification (USCS) |
| BH 2 – SS7 | 6.1 – 6.7 | 0 | 0 | 100 | 76 | 50 | 25 | 25 | Clay of High Plasticity (CH) |
| BH 3 – SS3 | 2.3 – 2.9 | 0 | 3 | 97 | 65 | 32 | 17 | 15 | Silty Clay of Low Plasticity (CL) |
| BH 4 – SS6 | 4.7 – 5.3 | 0 | 0 | 100 | 62 | 58 | 27 | 31 | Clay of High Plasticity (CH) |
| BH 6 – SS8 | 9.1 – 9.7 | 0 | 0 | 100 | 78 | 45 | 26 | 19 | Silty Clay of Medium Plasticity (CL) |
| BH 10 – SS4 | 3.2 – 3.8 | 0 | 2 | 98 | 71 | 50 | 22 | 28 | Clay of High Plasticity (CH) |

Based on a review of the results of the grain-size analysis and Atterberg limits, the soil may be classified as a silty to clay of low to high plasticity (CL and CH) in accordance with the USCS.

One-dimensional consolidation tests were performed on three (3) Shelby tube samples of the silty clay to clay from boreholes located on the large property east of the gas bar site. The test results are shown in the August 19,2021 EXP geotechnical report and summarized in Table V.

Table V: One-Dimensional Oedometer Test Results – Grey Silty Clay to Clay Samples

| Borehole No.- Sample No. | Sample Depth (Elevation) (m) | $\sigma'_{v0}$ (kPa) | $W_c$ (%) | $\gamma$ (kN/m$^3$) | $\sigma'_p$ (kPa) | $e_o$ | $C_r$ | $C_c$ | OCR |
|---|---|---|---|---|---|---|---|---|---|
| BH 6 –TW10 | 12.2 – 12.8<br>(69.0 – 68.4) | 94 | 74 | 15.0 | 120 | 2.131 | 0.04 | 1.5 | 1.3 |
| BH 7 – TW6 | 4.6 – 5.2<br>(79.5 – 78.9) | 52 | 73 | 15.3 | 150 | 2.056 | 0.03 | 1.4 | 2.9 |
| BH 8 – TW4 | 3.0 -3.6<br>(79.3 – 78.7) | 39 | 75 | 15.1 | 70 | 2.127 | 0.03 | 1.4 | 1.8 |

$\sigma'_{v0}$ = calculated effective overburden pressure (kPa); $W_c$: natural moisture content (%), $\gamma$: estimated natural unit weight $\sigma'_p$ = pre-consolidation pressure (kPa), $e_o$ = initial void ratio; $C_r$ = re-compression index; $C_c$ = compression index; OCR - Over-Consolidation Ratio

The pre-consolidation pressure of the silty clay to clay samples at similar elevations in Borehole Nos. 7 and 8 range from 70 kPa to 150 kPa and is 120 kPa at a deeper depth (elevation) in Borehole No. 6. Based on a review of the consolidation test results, the silty clay to clay samples are over-consolidated by a factor of 1.3 to 2.9.

## 6.4 Piezocone Penetration Test Hole (SCPTu)

One (1) piezocone penetration test with seismic shear wave and pore pressure measurements was carried out at the location of Borehole No. 9 (SCPTu-9) from ground surface to a termination depth of 32.5 m (Elevation 52.2 m). The SCPTu results are shown in Appendix A. The SCPTu results indicate the silty clay to clay is present to the termination depth of the SCPTu, with sand layers from 29.5 m depth to the 32.5 m termination depth.

## 6.5 Groundwater Levels

Groundwater level measurements were taken on June 19, 2021 in the standpipes installed in all boreholes. The groundwater level measurements in the boreholes located on the gas bar site and on the large property near the east property line of the gas bar site are summarized in Table VI.

Table VI: Summary of Groundwater Level Measurements

| Borehole No. (BH) | Ground Surface Elevation (m) | Date of Measurement (Elapsed Time in Days from Date of Installation) | Groundwater Depth Below Ground Surface (Elevation), m |
|---|---|---|---|
| BH 9 | 84.70 | June 19, 2021 (50 days) | Damaged |
| BH 10 | 84.72 | June 19, 2021 (51 days) | 1.3 (83.4) |

Based on a review of the groundwater level measurement in Borehole No. 10 located on the gas bar site, the groundwater level is at a 1.3 m depth (Elevation 83.4 m).

Groundwater levels were determined in the boreholes at the times and under the conditions stated in the scope of services. Note that fluctuations in the level of groundwater may occur due to a seasonal variation such as precipitation, snowmelt, rainfall activities, and other factors not evident at the time of measurement and therefore may be at a higher level during wet weather periods.

# 7. Site Classification for Seismic Site Response and Liquefaction Potential of Soils

## 7.1 Site Classification for Seismic Site Response

Based on the results from the piezocone penetration test hole with seismic shear wave and pore pressure measurements (SCPTu) conducted next to Borehole No. 9, the average shear wave velocity of the site (Vs) from ground surface to a 30.0 m depth below ground surface is 123 m/s. The Table 4.1.8.4.A in the 2012 Ontario Building Code (as amended May 2, 2022) indicates that for an average Vs value less than 180 m/s, the site classification for seismic site response is Class E. Therefore, for design purposes, the site classification for seismic response for the site is Class E.

## 7.2 Liquefaction Potential of Soils

Based on a review of the borehole information, the subsurface soils are not considered to be liquefiable during a seismic event.

# 8. Grade Raise Restrictions

The site is underlain by a sensitive marine clay deposit that is prone to consolidation settlement if overstressed by loads imposed on it by site grade raise, foundations and by the permanent lowering of the groundwater level following construction. Overstressing of the clay will result in its consolidation and subsequent settlement of foundations, which may exceed tolerable limits of the structure resulting in cracking of the structure.

A review of the spot elevations shown on the Grading and Ponding Plan dated August 30,2024 (Revision No. 2) and prepared by J.L. Richards indicates the final grades at the site will be set at Elevation 85.5 m to Elevation 86.0 m resulting in a site grade raise of up to 1.3 m.

The site grade raise of 1.3 m is considered acceptable using approved soil fill, provided the bearing pressure at serviceability limit state (SLS) and factored geotechnical resistance at ultimate limit state (ULS) for footings for the proposed building and for the deadman anchors shown in Section 10 of this report are respected.

An allowance for permanent groundwater lowering was not required as part of the review, since the foundations for the proposed building will be at or slightly below the groundwater level and measures will be employed in new service trenches to minimize the permanent lowering of the groundwater level at the site (use of clay seals), as recommended in Section 13 of this report.

Should the design grade raise at the site exceed or be less than the 1.3 m, EXP should be contacted to review the acceptability of the proposed new grade raise and to provide updated bearing pressure at serviceability limit state (SLS) and factored geotechnical resistance at ultimate limit state (ULS) for the building foundations.

# 9. Site Grading

Site grading within the floor slab area of the proposed building should consist of the excavation and removal of all existing topsoil and organic stained soils down to the native undisturbed native silty clay to clay or silty sand. The exposed silty sand should be proofrolled in order to consolidate any loose pockets. The silty sand and silty clay to clay subgrades should be examined by a geotechnician. Any soft, wet or loose zones of the exposed subgrade soils following proofrolling, should be removed and replaced with Ontario Provincial Standard Specification (OPSS) Granular B Type II compacted to 98 percent standard Proctor maximum dry density (SPMDD). The site grades within the floor slab area may then be raised to the design subgrade level of the floor slab using OPSS Granular B Type II compacted to 98 percent SPMDD.

For paved parking lot and access road areas, all topsoil should be excavated and removed to the native silty sand or silty clay to clay. The exposed silty sand should be proofrolled. The proofrolled silty sand subgrade and the exposed silty clay to clay subgrade should be examined by a geotechnician. Any loose, wet or soft zones identified in the subgrade should be excavated and removed and replaced with OPSS select subgrade material (SSM) compacted to 95 percent SPMDD. Once the subgrade has been approved, the site grades may be raised to the design subgrade level for the paved areas using OPSS select subgrade material compacted to 95 percent SPMDD. In wet areas, crusher-run granular type material may be required in the lower levels of the required fill to stabilize the subgrade.

In-place density tests should be performed on each lift of placed material to ensure that it has been compacted to the project specifications.

# 10. Foundation Considerations

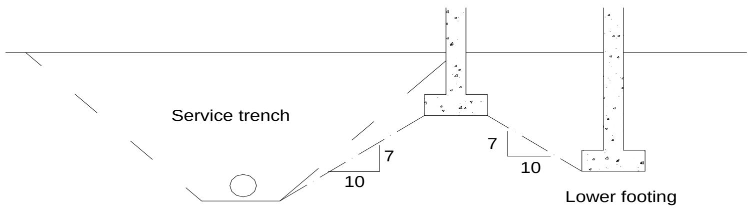
## 10.1 Building Foundations

Based on a review of the borehole information, it is considered feasible to support the proposed building on strip and spread footings founded on the native compact zone of the silty sand and the very stiff desiccated brown crust of the silty clay to clay.

Strip footings having a maximum width of 1.5 m and founded on the native compact silty sand or very stiff brown silty clay to clay to a maximum 1.0/1.1 m depth below existing grade at Elevation 83.66 m may be designed for a bearing pressure at serviceability limit state (SLS) of 75 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 115 kPa. Square pad footings having a maximum width and length of 3.0 m and founded on the native compact silty sand or very stiff brown silty clay to clay to a maximum 1.0 m/1.1 m depth below existing grade at Elevation 83.66 m may be designed for a bearing pressure at SLS of 90 kPa and factored geotechnical resistance at ULS of 135 kPa. The factored geotechnical resistance at ULS includes a resistance factor of 0.5. The SLS and factored ULS values are valid provided the 1.3 m grade raise is respected and the footings are founded no deeper than 1.0/1.1 m below existing grade on the native soils. If the footing depth below existing grade or the site grade raise will be different than noted above, EXP should be contacted to provide updated SLS and factored ULS values for the footings.

Settlements of footings designed for the above SLS bearing pressures are expected to be within the tolerable limits of 25 mm total and 19 mm differential.

Footings, which are to be placed at different elevations, should be located such that the higher footing is set below a line drawn up at 10 horizontal to 7 vertical from the near edge of the lower footing, as indicated on the following sketch:



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

All footing beds should be examined by a geotechnical engineer to ensure that the founding surfaces are capable of supporting the design bearing pressure at SLS and that the footing beds have been properly prepared. The surface of the exposed silty sand subgrade should be proofrolled and examined by a geotechnical engineer prior to concrete placement.

The exposed surface of the silty sand and silty clay to clay is expected to be susceptible to disturbance due to movement of workers and construction equipment. It is therefore recommended that the approved subgrade in the footing beds must be covered with a 50 mm thick concrete mud slab to prevent disturbance to the clay subgrade.

A minimum of 1.5 m of earth cover should be provided to the exterior foundations of heated structures to protect them from damage due to frost penetration. The frost cover should be increased to 2.1 m for unheated structures if snow will not be removed from their vicinity and to 2.4 m if snow will be removed from the vicinity of the structure. When earth cover is less than the required cover, an equivalent thermal combination of earth cover and rigid insulation or rigid insulation alone should be provided. EXP can provide additional comments in this regard, if required.

## 10.2 Installation of Underground Tanks

The underground tanks should be installed in accordance with the Technical Standards and Safety Authority (TSSA) regulation.

It is assumed the proposed underground tanks will be founded at a 4.3 m to 5.2 m depths below existing grade on the firm grey silty clay to clay and will be designed to withstand hydrostatic uplift pressure by tie-down straps connected to precast concrete deadman anchors.

The deadman anchors founded on the firm grey silty clay to clay may be designed for a bearing pressure at SLS of 30 kPa and factored geotechnical resistance at ULS of 45 kPa provided the 1.3 m site grade raise is respected and the deadman anchors are founded no deeper than 5.2 m below existing grade on the native soils. If the deadman anchor depth below existing grade or the site grade raise will be different than noted above, EXP should be contacted to provide updated SLS and factored ULS values for the deadman anchors.

The total and differential settlements of well designed and constructed deadman anchors placed in accordance with the above recommendations are expected to be less than 25 mm and 19 mm respectively.

The subgrade for the deadman anchors should be examined by a geotechnical engineer to confirm that the founding silty clay to clay is capable of supporting the bearing pressure at SLS.

It should be noted that the exposed silty clay to clay is susceptible to disturbance due to movement of workers and construction traffic and the prevailing weather conditions during construction. To prevent disturbance to the silt to clayey silt subgrade for the deadman anchors, the subgrade should be protected by covering it with a 50 mm thick concrete mud slab following examination and approval of the founding soil.

The proposed underground tanks should be designed to withstand hydrostatic uplift pressure assuming the groundwater level is at the ground surface.

The bedding and backfilling requirements for the proposed underground tanks should be in accordance with the tank manufacturer specifications.

## 10.3 Additional Comment

The recommended bearing pressure at SLS and factored geotechnical resistances at ULS have been calculated by EXP from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field monitoring provided by an experienced geotechnical engineer to validate the information for use during the construction stage.

# 11. Floor Slab and Drainage Requirements

The floor slab of the proposed building may be designed as a slab-on-grade founded on a bed of 200 mm thick, 19 mm sized clear stone placed on top of a minimum 300 mm thick compacted Granular B Type II pad placed on the native soils and constructed in accordance with Section 9 of this report. The clear stone would prevent the capillary rise of moisture from the sub-soil to the floor slab. Adequate saw cuts should be provided in the floor slabs to control cracking.

Perimeter drains should be provided for the proposed building. Based on the finished floor slab elevation at Elevation 86.00 m and groundwater level at Elevation 83.4 m, an underfloor drainage system is not required.

The ground floor slab should be set at least 150 mm above the surrounding exterior grades and the exterior grades should be sloped away from the proposed buildings to prevent ponding of surface water close to the exterior walls of the proposed buildings.

# 12. Excavations and De-Watering Requirements

## 12.1 Excess Soil Management

Ontario Regulation 406/19 specifies protocols that are required for the management and disposal of excess soils. As set forth in the regulation, specific analytical testing protocols need to be implemented and followed based on the volume of soil to be managed and the requirements of the receiving site. The testing protocols are specific as to whether the soils are stockpiled or in situ. In either scenario, the testing protocols are far more onerous than have been historically carried out as part of standard industry practices. These decisions should be factored in and accounted for prior to the initiation of the project-defined scope of work. EXP would be pleased to assist with the implementation of a soil management and testing program that would satisfy the requirements of Ontario Regulation 406/19.

## 12.2 Excavations

Excavation for the construction of the proposed building foundations are anticipated to extend into the silty sand and the silty clay to clay to a 1.0/1.1 m depth and will likely be above or near the groundwater level. The assumed 3.0 m and 5.2 m deep excavations for underground services and the underground fuel storage tanks respectively will likely extend below the silty sand and deeper into the silty clay to clay and are anticipated to be below the groundwater level.

The excavations may be undertaken by conventional heavy equipment.

All excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA), Ontario Reg. 213/91. Based on the definitions provided in OHSA, the subsurface soils on site are considered to be Type 3 and as such must be cut back at 1H:1V from the bottom of the excavation. Within zones of seepage and below the groundwater level, the excavation side slopes are expected to slough and eventually stabilize at a slope of 3H:1V.

If side slopes cannot be achieved due to space restrictions on site such as the proximity of open cut excavations to the property limits, existing infrastructure or to foundations of adjacent existing buildings, excavations would have to be undertaken within the confines of an engineered support system (shoring system).

The need for a shoring system, the most appropriate type of shoring system and the design and installation of the shoring system should be determined by the contractors bidding on this project. The design of the shoring system should be undertaken by a professional engineer experienced in shoring design and the installation of the shoring system should be undertaken by a contractor experienced in the installation of shoring systems. The shoring system should be designed and installed in accordance with latest edition of Ontario Regulation 213/91 under the OHSA and the 2006 Fourth Edition of the Canadian Foundation Engineering Manual (CFEM). For tiebacks that may be required to laterally support the shoring system and will extend onto neighboring properties, permission may need to be obtained from the neighboring property owners. The shoring system will need to be monitored on a periodic basis for movement.

A pre-construction condition survey of buildings and infrastructure within the influence zone of the construction should be undertaken prior to start of construction activities including shoring installation activity.

Base heave type failure is not anticipated for excavations that extend to a 5.2 m depth below existing grade with the excavation bases located in the grey firm silty clay to clay.

Many geologic materials deteriorate rapidly upon exposure to meteorological elements. Unless otherwise specifically indicated in this report, walls and floors of excavations must be protected from moisture, desiccation, and frost action throughout the course of construction.

## 12.3 De-Watering Requirements

Seepage of the surface and subsurface water into the shallow excavations for the proposed building and deeper excavations for the municipal services and underground tanks is anticipated. However, due to the relatively impermeable nature of the silty clay to clay, it should be possible to collect water entering the excavations at low points and to remove it by conventional pumping techniques. In areas of high infiltration where more permeable soils exist such as the silty sand or in areas where more permeable soil layers may exist, such as sand seams within the silty clay to clay, a higher seepage rate should be anticipated. Therefore, high-capacity pumps, operating 24 hours, seven (7) days a week, may be required to keep the excavation dry.

For construction dewatering, an Environmental Activity and Sector Registry (EASR) approval may be obtained for water takings greater than 50 m3 and less than 400 m3 per day. If more than 400 m3 per day of groundwater are generated for dewatering purposes, then a Category 3 Permit to Take Water (PTTW) must be obtained from the Ministry of the Environment, Conservation and Parks (MECP). A Category 3 PTTW would require a complete hydrogeological assessment and would take at least 90 days for the MECP to process once the application is submitted.

Although this investigation has estimated the groundwater levels at the time of the fieldwork, and commented on dewatering and general construction problems, conditions may be present which are difficult to establish from standard boring and excavating techniques and which may affect the type and nature of dewatering procedures used by the contractor in practice. These conditions include local and seasonal fluctuations in the groundwater table, erratic changes in the soil profile, thin layers of soil with large or small permeabilities compared with the soil mass, etc. Only carefully controlled tests using pumped wells and observation wells will yield the quantitative data on groundwater volumes and pressures that are necessary to adequately engineer construction dewatering systems.

## 13. Pipe Bedding Requirements

The depth at which municipal services will be installed is anticipated to be at a 3.0 m depth below existing grade. Therefore, the subgrade for the underground service pipes is expected to be within the brown and grey silty clay to clay.

It is recommended that the bedding for the underground services including material specifications, thickness of cover material and compaction requirements conform to municipal requirements and/or Ontario Provincial Standard Specification and Drawings (OPSS and OPSD).

The pipe subgrade material is anticipated to consist of firm to very stiff silty clay to clay. In this case, it is recommended the pipe bedding consist of 300 mm thick of OPSS Granular A bedding material. The bedding material should be compacted to at least 98 percent SPMDD.

The bedding thickness may be further increased in areas where the silty clay to clay subgrade becomes disturbed or below the water table. Trench base stabilization techniques, such as removal of loose/soft material, placement of crushed stone sub-bedding (Granular B Type II) that is completely wrapped in a non-woven geotextile, may also be used if trench base disturbance becomes a problem in wet or soft areas.

To minimize settlement of the pavement structure over services trenches, the trench backfill material within the frost zone, to 1.2 m depth below final grade, should match the existing material along the trench walls to minimize differential frost heaving of the subgrade soil, provided this material is compactible. Otherwise, frost tapers may be required.

If the backfill in the service trenches will consist of granular fill, clay seals should be installed in the service trenches at select intervals (spacing) as per City of Ottawa Drawing No. S8. The seals should be 1 m wide, extend over the entire trench width and from the bottom of the trench to the underside of the pavement structure. The clay should be compacted to 95 percent SPMDD. The purpose of the clay seals is to prevent the permanent lowering of the groundwater level.

The municipal services should be installed in short open trench sections that are excavated and backfilled the same day.

# 14. Backfilling Requirements and Suitability of On-Site Soils for Backfilling Purposes

The soils to be excavated from the site will comprise of topsoil, silty sand and silty clay to clay. From a geotechnical perspective, these soils are not considered suitable for reuse as backfill material in the interior of the building. It may be possible to use portions of these soils above the groundwater level outside the building area and in landscaped areas, subject to additional testing at time of construction. However, these soils are subject to moisture absorption due to precipitation and must be protected at all times from the elements and are considered to be limited in quantity.

It is anticipated that the majority of the material required for engineered fill, backfilling purposes, or as subgrade fill for the project would have to be imported and should preferably conform to the following specification:

- Engineered fill under slab-on-grade - OPSS 1010 Granular B Type II placed in 300 mm thick lifts and each lift compacted 98 percent SPMDD.
- Backfill in footing trenches and against foundation walls – OPSS 1010 Granular B Type II placed in 300 mm thick lifts and each lift compacted to 98 percent of the SPMDD inside the building and 95 percent SPMDD outside the building respectively.
- Backfill in services trenches inside building – OPSS 1010 Granular B Type II placed in 300 mm thick lifts and each lift compacted to 98 percent of the SPMDD.
- Backfill in exterior services trenches – OPSS 1010 Select Subgrade Material (SSM) placed in 300 mm thick lifts and each lift compacted to 95 percent of the SPMDD.
- Landscaped areas – Clean fill that is free of organics and deleterious material and is placed in 300 mm thick lifts with each lift compacted to 92 percent of the SPMDD.

The bedding and backfilling requirements for the proposed underground tanks should be in accordance with the tank manufacturer specifications.

## 15. Access Roads and Parking Lots

Pavement structures for the surface parking lots and access roads are given on Table VII below for the anticipated silty sand, silty clay to clay and OPSS Select Subgrade Material (SSM) subgrades. The pavement structures are based upon the assumption that the subgrade will be properly prepared and assumes a functional design life of 15 to 18 years. The proposed functional design life represents the number of years to the first rehabilitation, assuming regular maintenance is carried out.

Table VII: Recommended Pavement Structure Thicknesses

| Pavement Layer | Compaction Requirements | Computed Pavement Structure | |
|---|---|---|---|
| | | Light Duty Traffic (cars only) | Heavy Duty Traffic (cars and trucks) |
| Asphaltic Concrete (PG 58-34) | 92 to 97 percent MRD | 65 mm HL3/SP12.5 mm/ Cat. B | 50 mm HL3/SP12.5 Cat. B<br>0 mm HL8/SP 19 Cat. B |
| OPSS 1010 Granular A Base (crushed limestone) | 100 percent SPMDD | 150 mm | 150 mm |
| OPSS 1010 Granular B Type II Sub-base | 100 percent SPMDD | 450 mm | 600 mm |

Notes:
1. SPMDD denotes standard Proctor maximum dry density, ASTM, D-698-12e2.
2. MRD denotes Maximum Relative Density, ASTM D2041.
3. The upper 300 mm of the subgrade fill must be compacted to 98 percent SPMDD.

It is recommended that the heavy-duty pavement structure be used in areas where the heavy fuel tanker trucks (used to fill the underground fuel storage tanks) will travel on the site.

The foregoing pavement structure design assumes that construction is carried out during dry periods and that the subgrade is stable under the load of construction equipment. If construction is carried out during wet weather, and heaving or rolling of the subgrade is experienced, additional thickness of granular material and/or geotextile may be required.

Additional comments on the construction of the parking lots and access roads are as follows:

1. As part of the subgrade preparation, the proposed parking area and access roads should be stripped of topsoil and other obviously unsuitable material. The subgrade should be properly shaped, crowned, then proofrolled with a heavy vibratory roller in the full-time presence of a representative of this office. Any soft or spongy subgrade areas detected should be sub excavated and properly replaced with suitable approved backfill compacted to 95 percent SPMDD (ASTM D698-12e2).
2. The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved. The need for adequate drainage cannot be over-emphasized. Subdrains should be installed on both sides of the access road(s). Subdrains must be installed in the proposed parking area at low points and should be continuous between catchbasins to intercept excess surface and subsurface moisture and to prevent subgrade softening. This will ensure no water collects in the granular course, which could result in pavement failure during the spring thaw. The location and extent of subdrains required within the paved areas should be reviewed by this office in conjunction with the proposed site grading.
3. To minimize the problems of differential movement between the pavement and catchbasins/manholes due to frost action, the backfill around the structures should consist of free-draining granular material preferably conforming to OPSS Granular B Type II material. Weep holes should be provided in the catchbasins/manholes to facilitate drainage of any water that may accumulate in the granular fill.

4. The most severe loading conditions on light-duty pavement areas and the subgrade may occur during construction. Consequently, special provisions such as restricted lanes, half-loads during paving, temporary construction roadways, etc., may be required, especially if construction is carried out during unfavorable weather.
5. The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum cross fall of 2 percent) to provide effective surface drainage towards catch basins. Surface water should not be allowed to pond adjacent to the outside edges of paved areas.
6. Relatively weaker subgrade may develop over service trenches at the subgrade level. These areas may require the use of thicker/coarser sub-base material and the use of a geotextile at the subgrade level. If this is the case, it is recommended that additional 150 mm thick granular sub-base, OPSS Granular B Type II, should be provided in these areas, in addition to the use of a geotextile at the subgrade level.
7. The granular materials used for pavement construction should conform to Ontario Provincial Standard Specifications (OPSS 1010) for Granular A and Granular B Type II and should be compacted to 100 percent of the SPMDD.

The asphaltic concrete used, and its placement should meet OPSS 1150 or 1151 requirements. It should be compacted from 92 percent to 97 percent of the MRD (ASTM D2041). Asphalt placement should be in accordance with OPSS 310 and OPSS 313.

It is recommended that EXP be retained to review the final pavement structure design and drainage plans prior to construction to ensure they are consistent with the recommendations of this report.

# 16. Subsurface Concrete and Steel Requirements

Chemical tests limited to pH, chloride, sulphate and resistivity were performed on one (1) selected soil sample obtained from Borehole No. 10 located at the gas bar site. The results are summarized in Table VIII.

Table VIII: Results of pH, Chloride, Sulphate and Resistivity Tests on Soil Sample

| Borehole No. (Sample No.) | Soil Type | Depth (m) | pH | Sulphate (%) | Chloride (%) | Resistivity (ohm-cm) |
|---|---|---|---|---|---|---|
| BH 10 – SS2 | Grey Silty Clay to Clay | 1.5-2.1 | 8.35 | 0.0016 | 0.0003 | 9620 |

The results indicate the clay has a sulphate content of less than 0.1 percent. This concentrations of sulphate in the silty clay to clay would have a negligible potential of sulphate attack on subsurface concrete. The concrete should be designed in accordance with Table Nos. 3 and 6 of CSA A.23.1-14. However, the concrete should be dense, well compacted and cured.

Based on a review of the resistivity test results, the grey silty clay to clay sample is considered to be mildly corrosive to bare steel as per the National Association of Corrosion Engineers (NACE). Appropriate measures should be undertaken to protect buried steel elements from corrosion.

## 17. Tree Planting Restrictions

The site is underlain by marine clay. The test results of the native upper brown and lower grey clay of the marine clay deposit was compared with the document titled, Tree Planting in Sensitive Marine Clay Soils – 2017 City of Ottawa Guidelines (2017 Guidelines) and indicate the upper brown clay and the lower grey clay have a low/medium potential for soil volume change. For soils that have a low/medium potential for soil volume change, the 2017 Guidelines indicate that the tree to foundation setback distance and tree planting restrictions should be in accordance with the 2017 guidelines.

A landscape architect should be consulted to ensure the setbacks and tree planting restrictions are in accordance with the 2017 Guidelines.

## 18. General Comments

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for the design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

The information contained in this report is not intended to reflect on environmental aspects of the soils. Should specific information be required, including for example, the presence of pollutants, contaminants or other hazards in the soil, additional testing may be required.

We trust that the information contained in this report will be satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Sincerely,

LICENSED PROFESSIONAL ENGINEER
09.12.2024
S.M. POTYONDY
37232568
PROVINCE OF ONTARIO

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Senior Geotechnical Engineer
Earth & Environment

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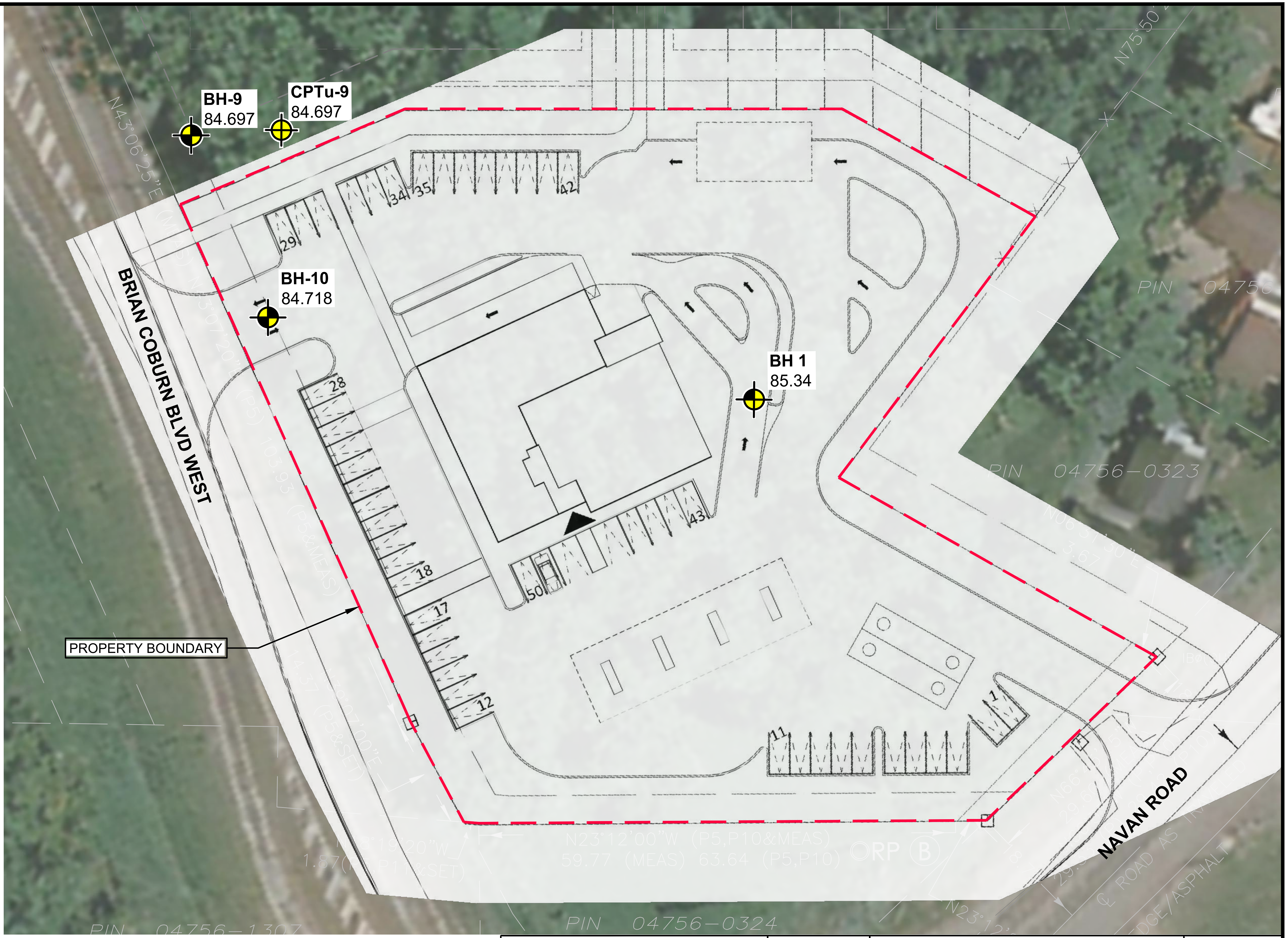
Senior Manager, Eastern Region
Earth and Environment

# Figures



## LEGEND

| | | |
|---|---|---|
| ⊕ | **BH-1**<br>81.115 | BOREHOLE NO. & LOCATION SURFACE ELEVATION ABOVE SEA LEVEL (m) |
| ⊕ | **CPTu-9**<br>81.115 | PIEZOCONE PENETROMETER BOREHOLE LOCATION AND GROUND SURFACE ELEVATION |
| ⊕ | **BH-1**<br>81.115 | *PATERSON* BOREHOLE NO. & LOCATION,GROUND SURFACE ELEVATION (m) (REFERENCE: PATERSON REPORT DATED NOVEMBER 13, 2018, REPORT: PG4415-1 REVISION 1). |

## NOTES:

1. THE BOUNDARIES AND SOIL TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. TOPSOIL QUANTITIES SHOULD NOT BE ESTABLISHED FROM THE INFORMATION PROVIDED AT THE BOREHOLE LOCATIONS.
4. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOT(S) GRADES.
5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
6. BASE PLAN INFORMATION OBTAINED FROM McROBIE ARCHITECTS, SHEET NO. SP-A01, DATED MAY 12, 2022.

0 5m 10m 20m
HORIZONTAL 1:500

**exp** Services Inc.
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Ottawa, ON K2B 8H6
www.exp.com

DESIGN IT/SP
DRAWN TM/AS
DATE MARCH 2023
FILE NO OTT-21004743-B0

GEOTECHNICAL INVESTIGATION
PROPOSED GAS BAR
2983 NAVAN ROAD, OTTAWA, ON

BOREHOLE LOCATION PLAN

SCALE 1:500
SKETCH NO FIG 2

Filename: E:\OTT\OTT-21004743-B0\60 Execution\65 Drawings\geo\21004743-B0_Geo.dwg
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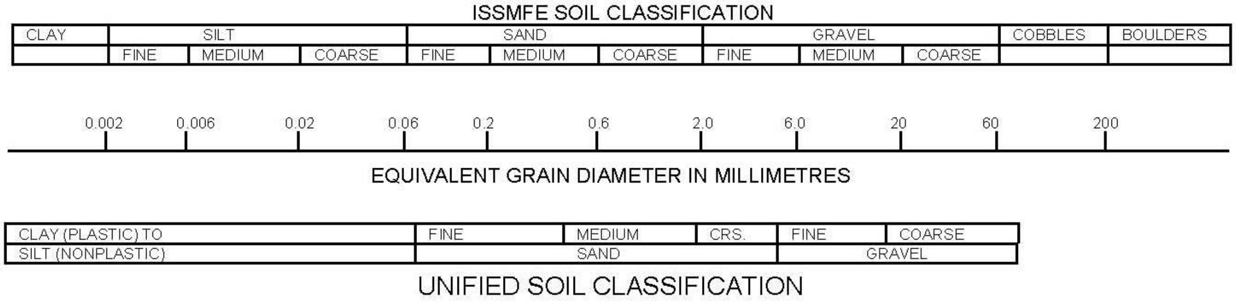
# Appendix A – 2021 EXP Borehole Logs for Borehole Nos. 9 and 10 and Seismic Piezocone Test Hole Log and Data

# Appendix A-1: – 2021 EXP Borehole Logs for Borehole Nos. 9 and 10

# Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Societv for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by **exp** Services Inc. also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

# Log of Borehole BH-09

Project No: OTT-21004743-B0

Project: Proposed Residential Development

Location: 2983, 3053 and 3079 Navan Road, Ottawa, Ontario

Figure No.

Page. 1 of 1

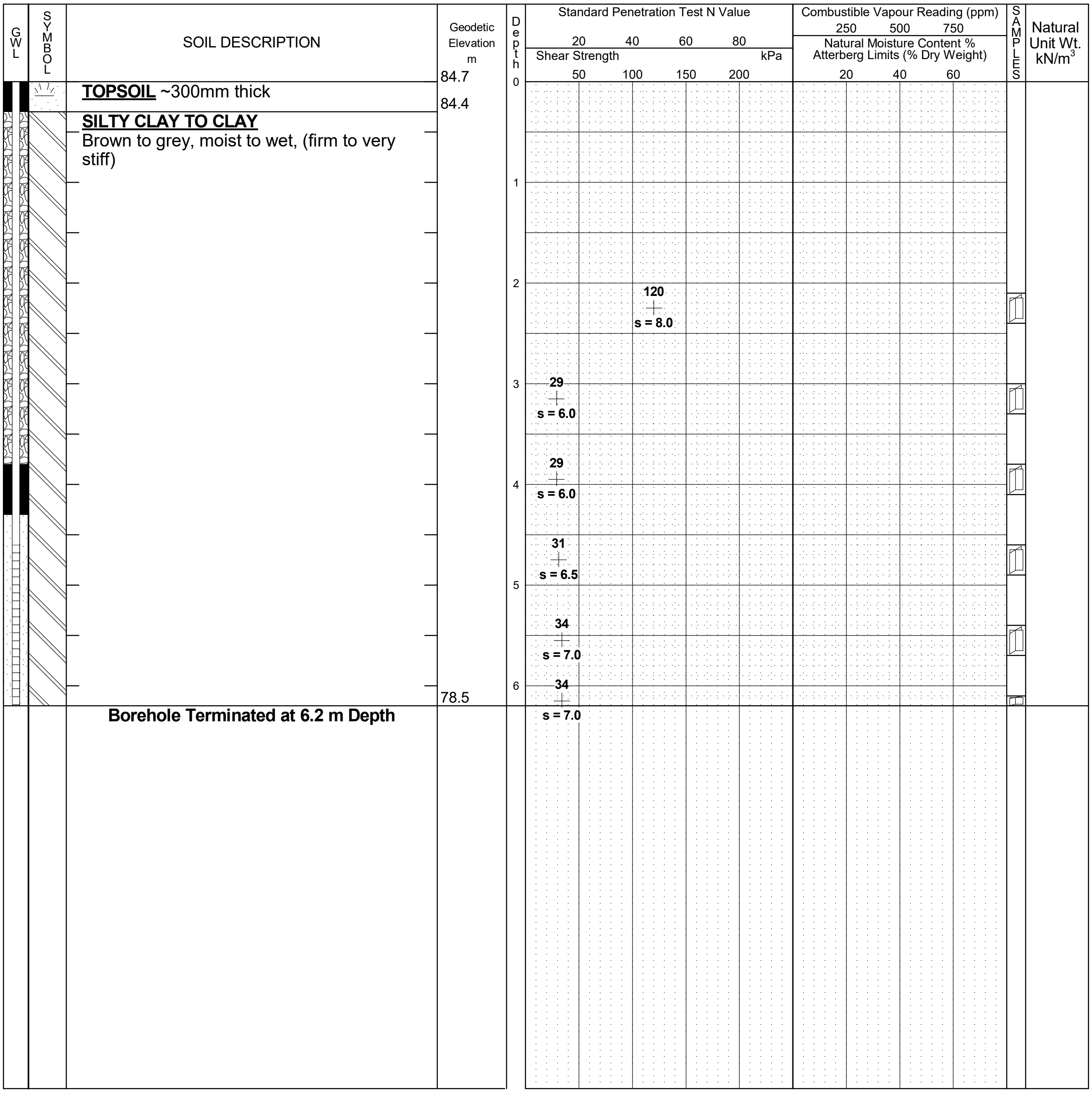
Date Drilled: 'April 30, 2021

Drill Type: CME-850 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

| | | | |
|---|---|---|---|
| Split Spoon Sample | ☒ | Combustible Vapour Reading | ☐ |
| Auger Sample | ◨ | Natural Moisture Content | × |
| SPT (N) Value | ○ | Atterberg Limits | ├──○ |
| Dynamic Cone Test | ─── | Undrained Triaxial at % Strain at Failure | ⊕ |
| Shelby Tube | ■ | | |
| Shear Strength by Vane Test | + S | Shear Strength by Penetrometer Test | ▲ |



**TOPSOIL** ~300mm thick

**SILTY CLAY TO CLAY**
Brown to grey, moist to wet, (firm to very stiff)

**Borehole Terminated at 6.2 m Depth**

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21004743-B0

WATER LEVEL RECORDS

| Date | Water Level (m) | Hole Open To (m) |
|---|---|---|
| Completion | 1.8 | 6.1 |

CORE DRILLING RECORD

| Run No. | Depth (m) | % Rec. | RQD % |
|---|---|---|---|

# Log of Borehole BH-10

Project No: OTT-21004743-B0

Project: Proposed Residential Development

Location: 2983, 3053 and 3079 Navan Road, Ottawa, Ontario

Figure No. ______

Page. 1 of 1

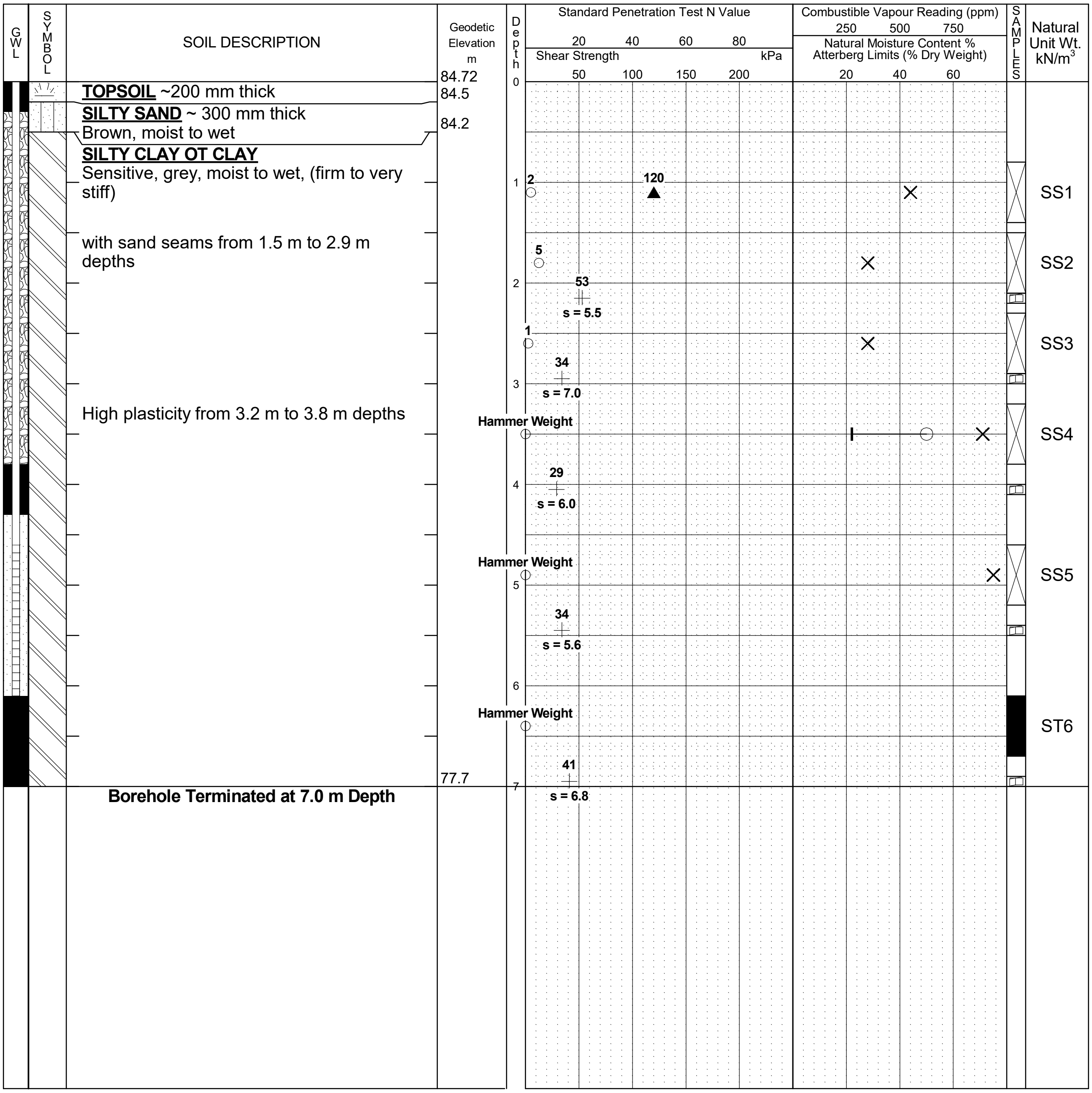
Date Drilled: 'April 29, 2021

Drill Type: CME-850 Track Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: ML Checked by: SMP

| | | | |
|---|---|---|---|
| Split Spoon Sample | ☒ | Combustible Vapour Reading | ☐ |
| Auger Sample | ◧ | Natural Moisture Content | ✕ |
| SPT (N) Value | ○ | Atterberg Limits | ├──○ |
| Dynamic Cone Test | ── | Undrained Triaxial at % Strain at Failure | ⊕ |
| Shelby Tube | ■ | Shear Strength by Penetrometer Test | ▲ |
| Shear Strength by Vane Test | +<br>S | | |



**TOPSOIL** ~200 mm thick

**SILTY SAND** ~ 300 mm thick
Brown, moist to wet

**SILTY CLAY OT CLAY**
Sensitive, grey, moist to wet, (firm to very stiff)

with sand seams from 1.5 m to 2.9 m depths

High plasticity from 3.2 m to 3.8 m depths

**Borehole Terminated at 7.0 m Depth**

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter standpipe installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21004743-B0

WATER LEVEL RECORDS

| Date | Water Level (m) | Hole Open To (m) |
|---|---|---|
| Completion | 1.8 | 6.1 |

CORE DRILLING RECORD

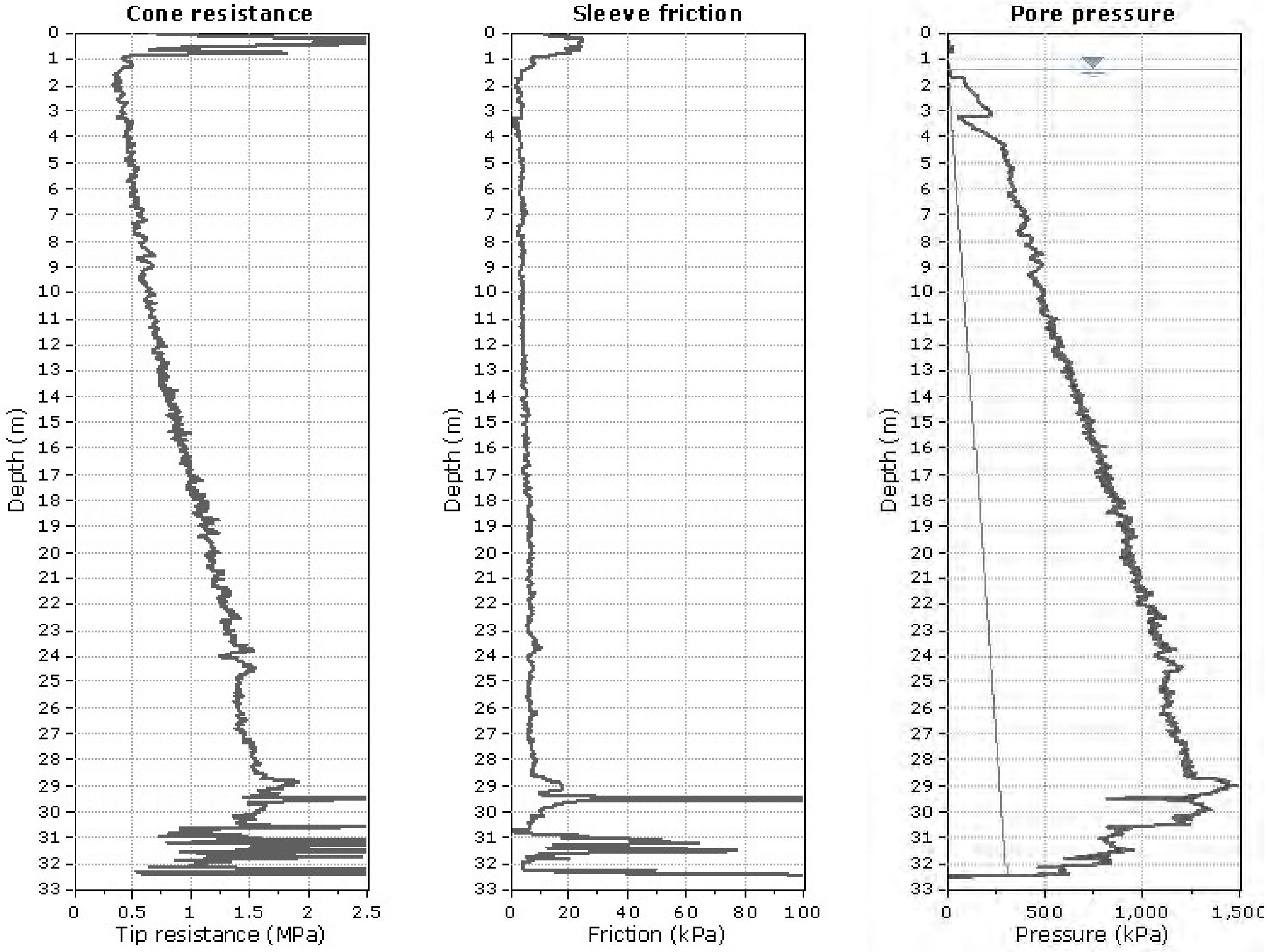
| Run No. | Depth (m) | % Rec. | RQD % |
|---|---|---|---|
| | | | |

# Appendix A-2: 2021 EXP Seismic Piezocone Test Hole Log and Data (SCPTu-9)

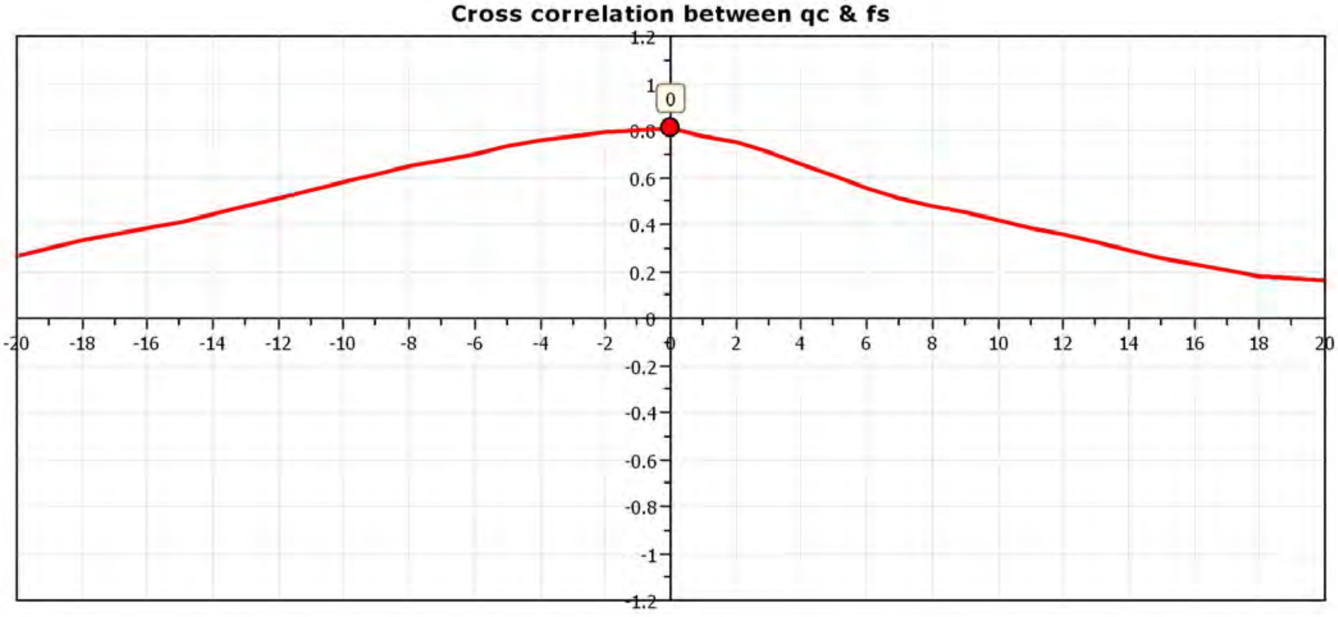
**EXP**
2650 Queensview Dr Suite 100
Ottawa, Ontario,  K2B 8H6
https://ww.exp.com

**CPT: SCPTu-9**
Total depth: 32.51 m
Surface Elevation: 84.70 m
Cone Type: Vertek 4544 - 5t
Cone Operator: Kevin Simoneau, P.Eng, M.Sc.

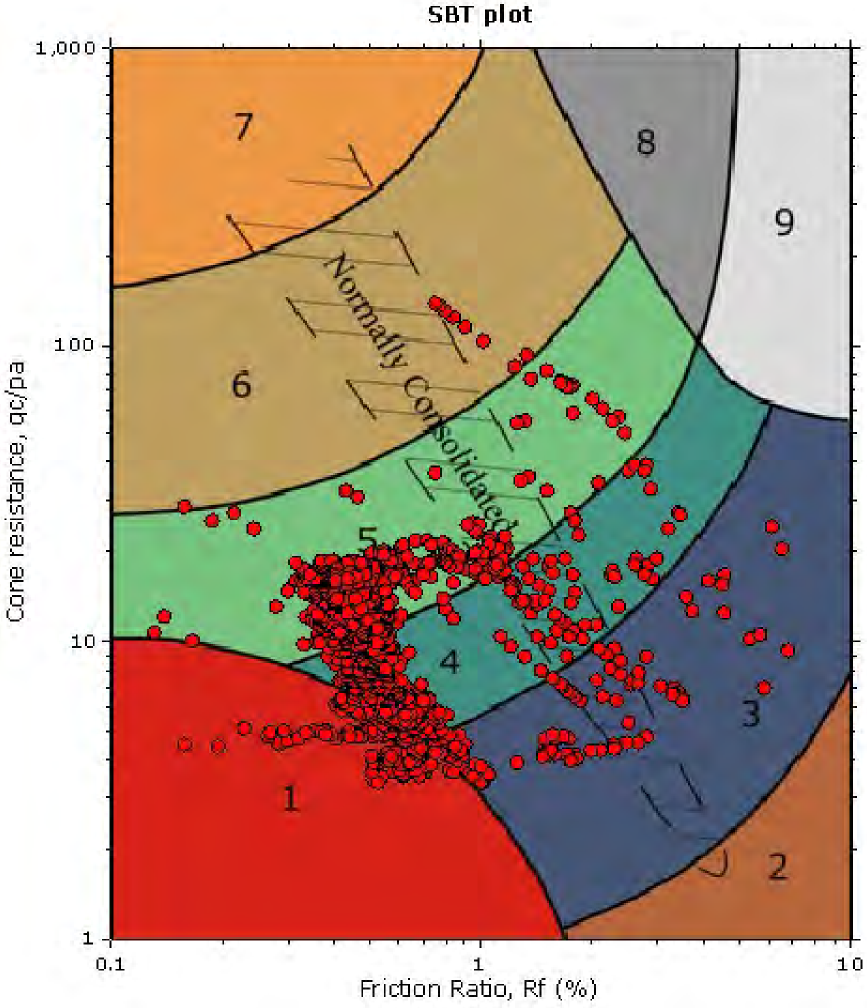
**Project:** **3053 & 3079 Navan Road**
**Location:** **Navan / Pagé Roads, Ottawa**

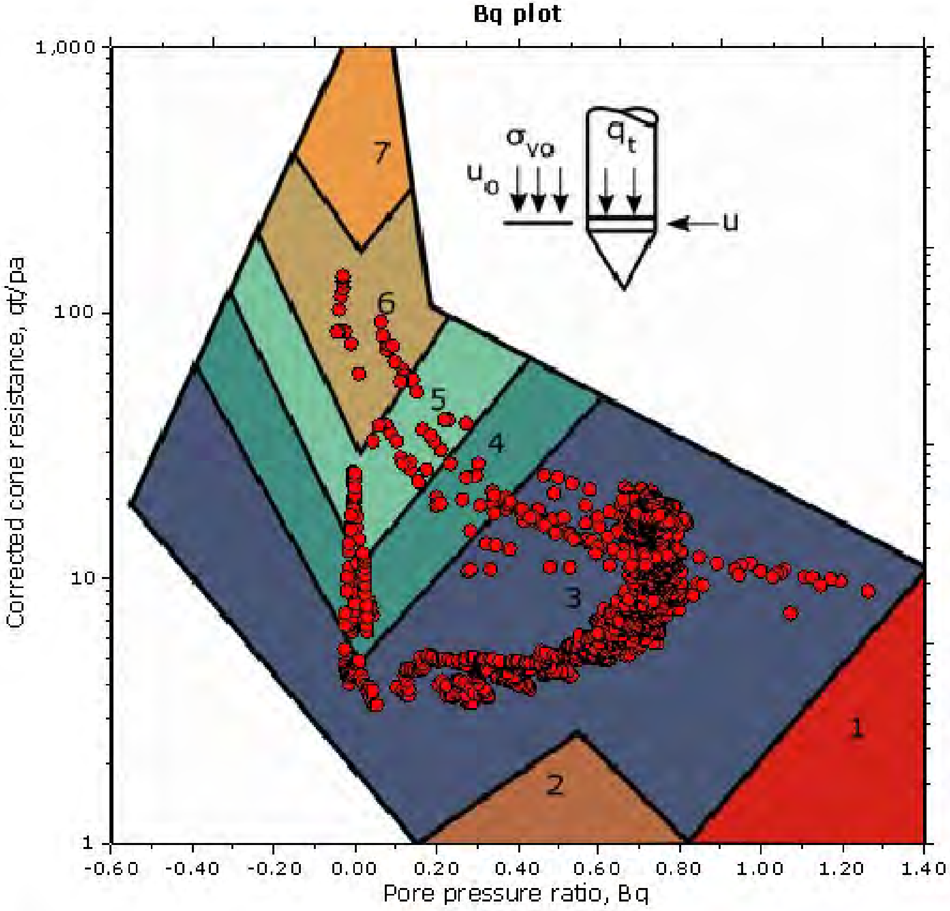


The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



## SBT - Bq plots



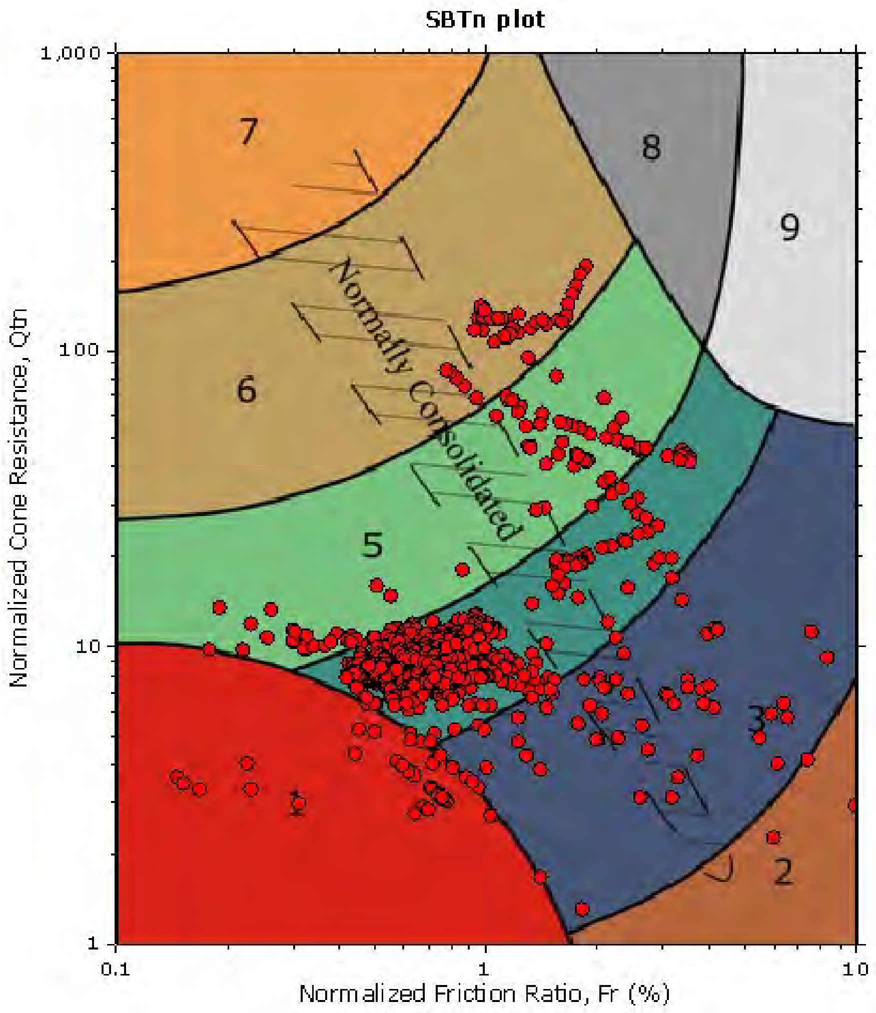


**SBT legend**

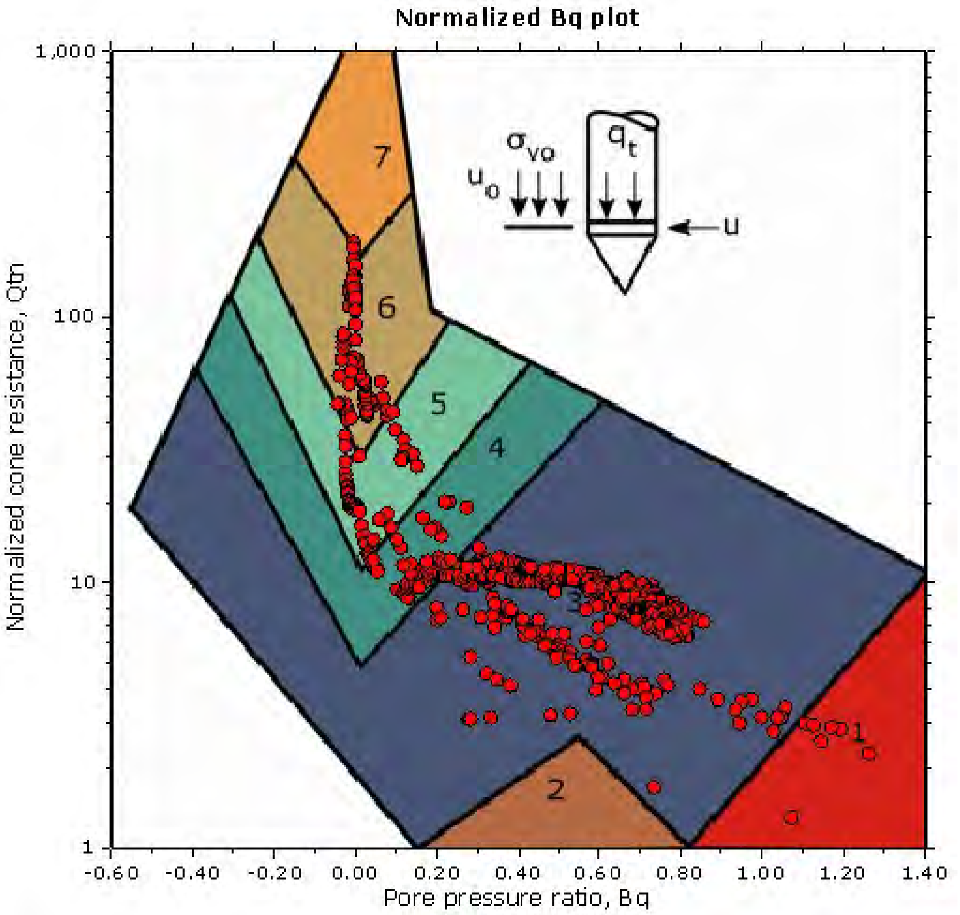
- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

## SBT - Bq plots (normalized)

SBTn plot



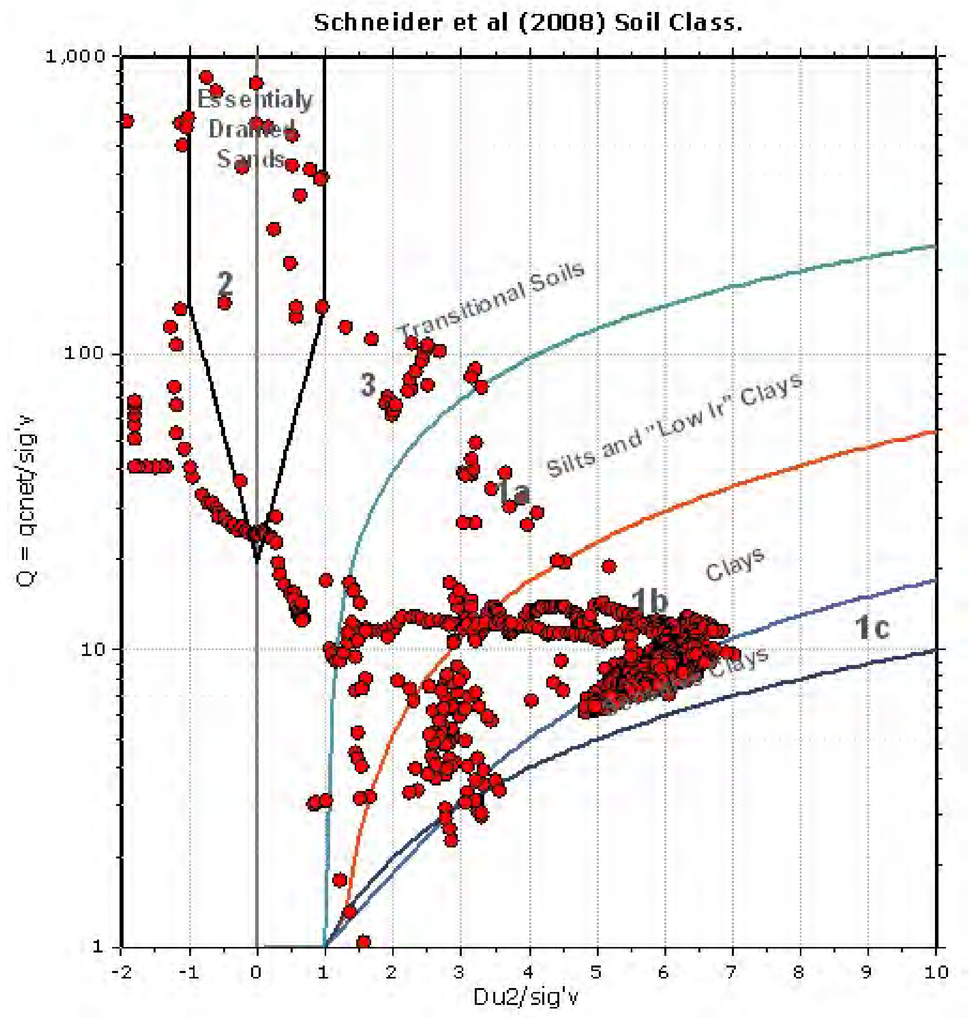
Normalized Bq plot

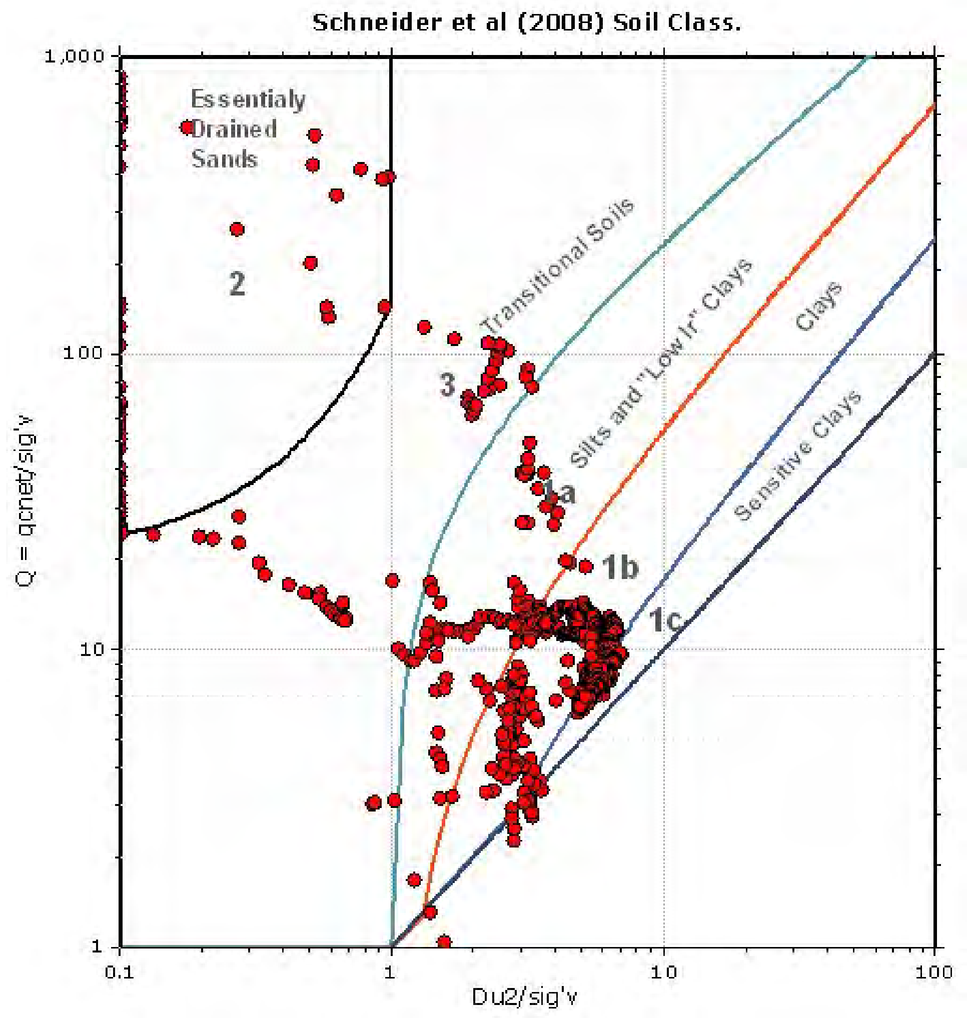


**SBTn legend**

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬜ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

## Bq plots (Schneider)





**CPT: SCPTu-9**
Total depth: 32.51 m
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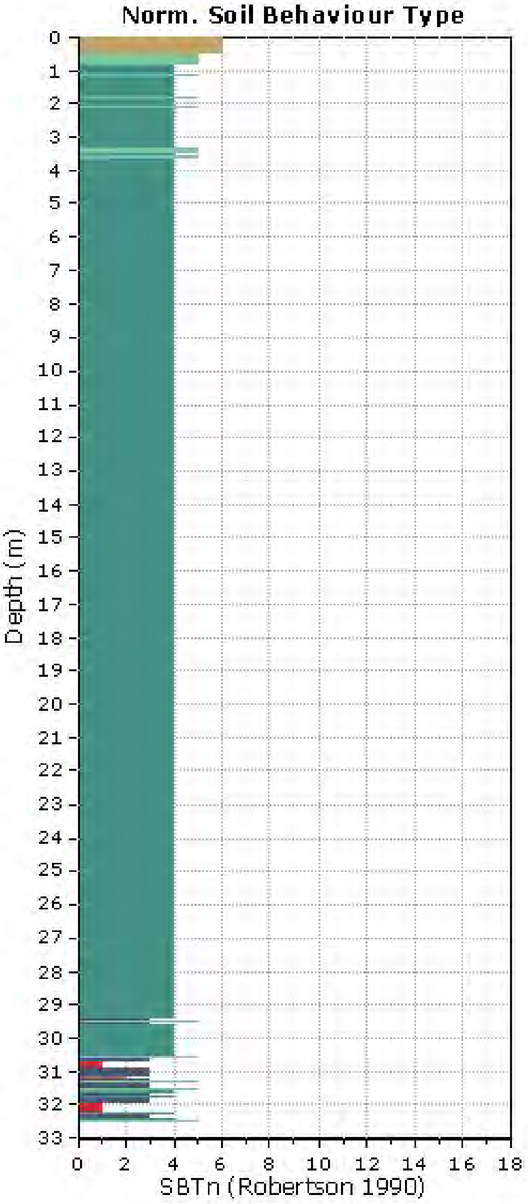
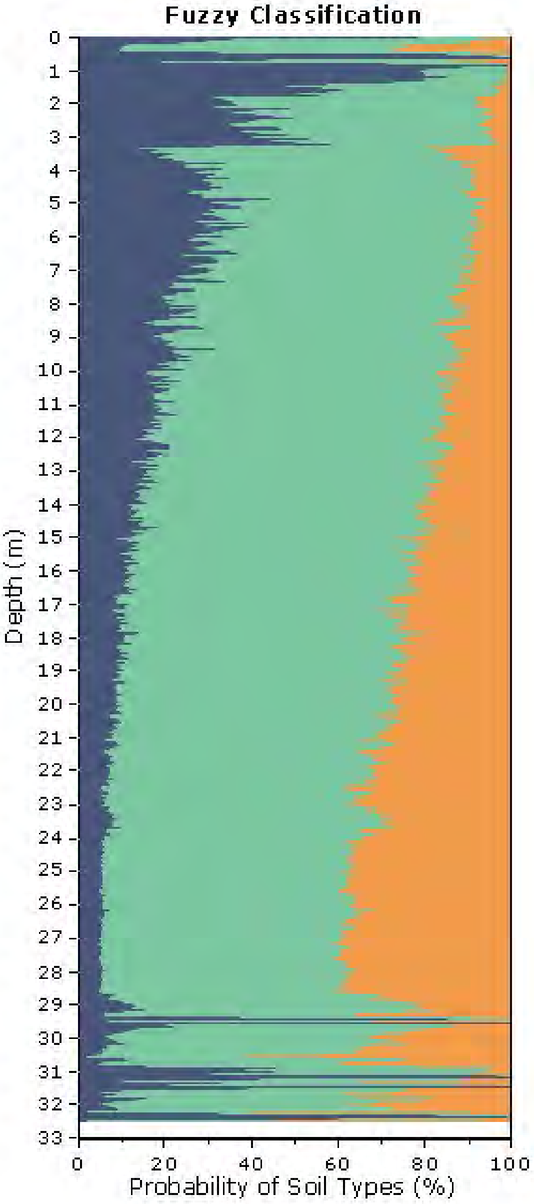
Cone Type: Vertek 4544 - 5t
Cone Operator: Kevin Simoneau, P.Eng, M.Sc.





**EXP**
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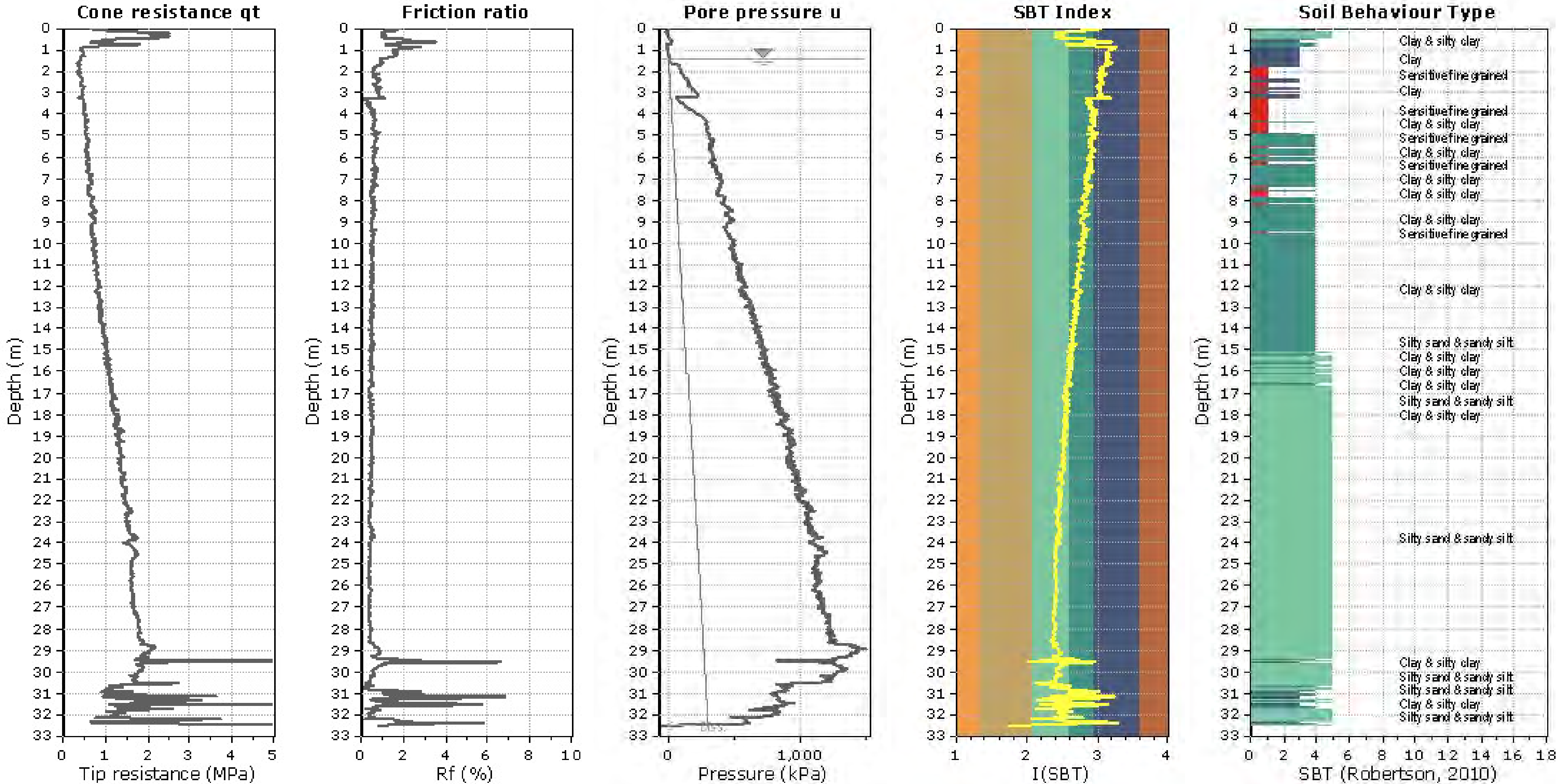
Ottawa, Ontario, K2B 8H6
https://ww.exp.com

**Project:** **3053 & 3079 Navan Road**
**Location:** **Navan / Pagé Roads, Ottawa**

**CPT: SCPTu-9**
Total depth: 32.51 m
Surface Elevation: 84.70 m
Cone Type: Vertek 4544 - 5t
Cone Operator: Kevin Simoneau, P.Eng, M.Sc.



**SBT legend**

| | | |
|---|---|---|
| 🟥 1. Sensitive fine grained | 🟩 4. Clayey silt to silty clay | 🟧 7. Gravely sand to sand |
| 🟫 2. Organic material | 🟩 5. Silty sand to sandy silt | ⬛ 8. Very stiff sand to clayey sand |
| 🟦 3. Clay to silty clay | 🟨 6. Clean sand to silty sand | ⬜ 9. Very stiff fine grained |

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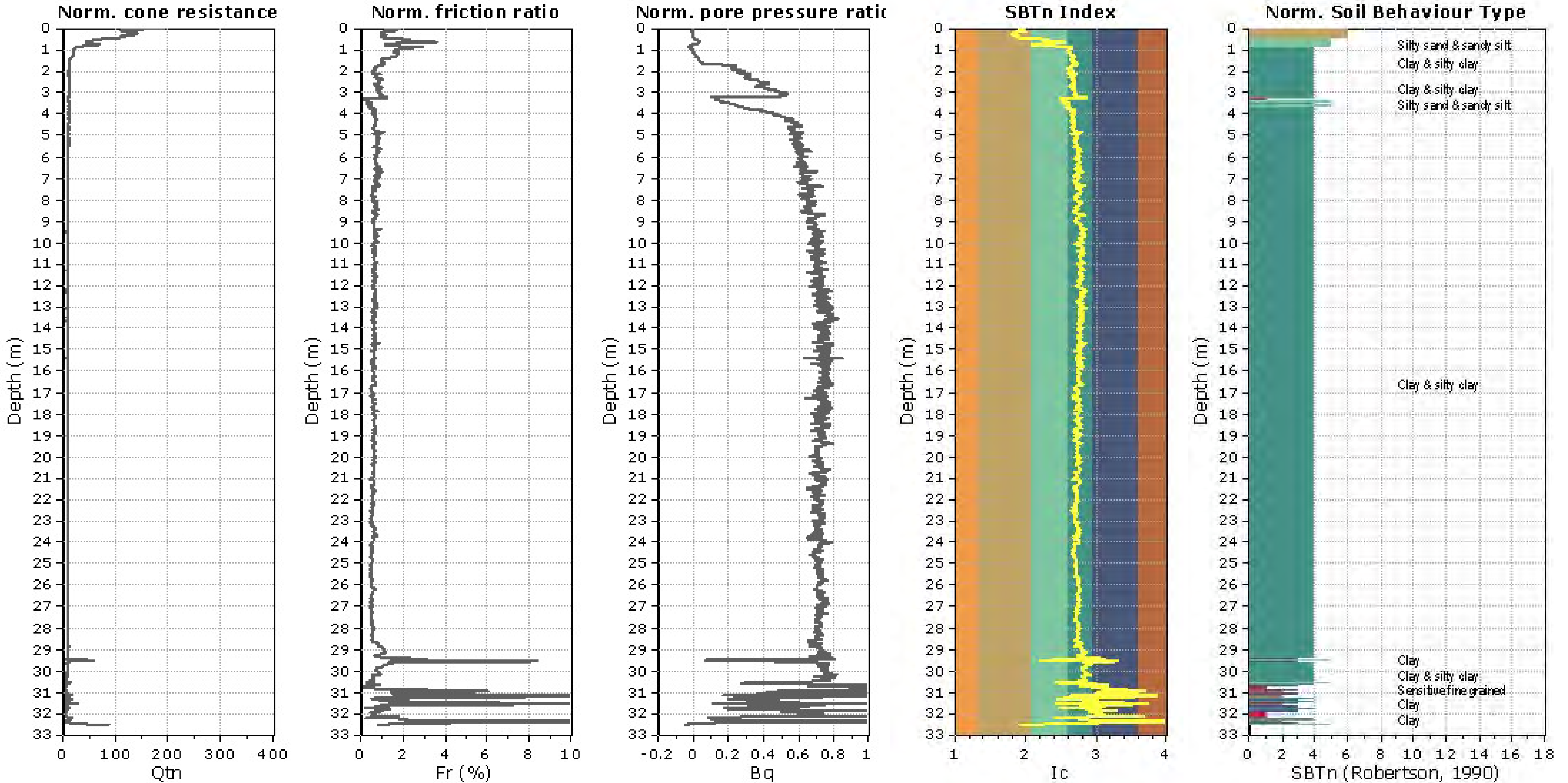
Cone Operator: Kevin Simoneau, P.Eng, M.Sc.



**SBTn legend**

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬜ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

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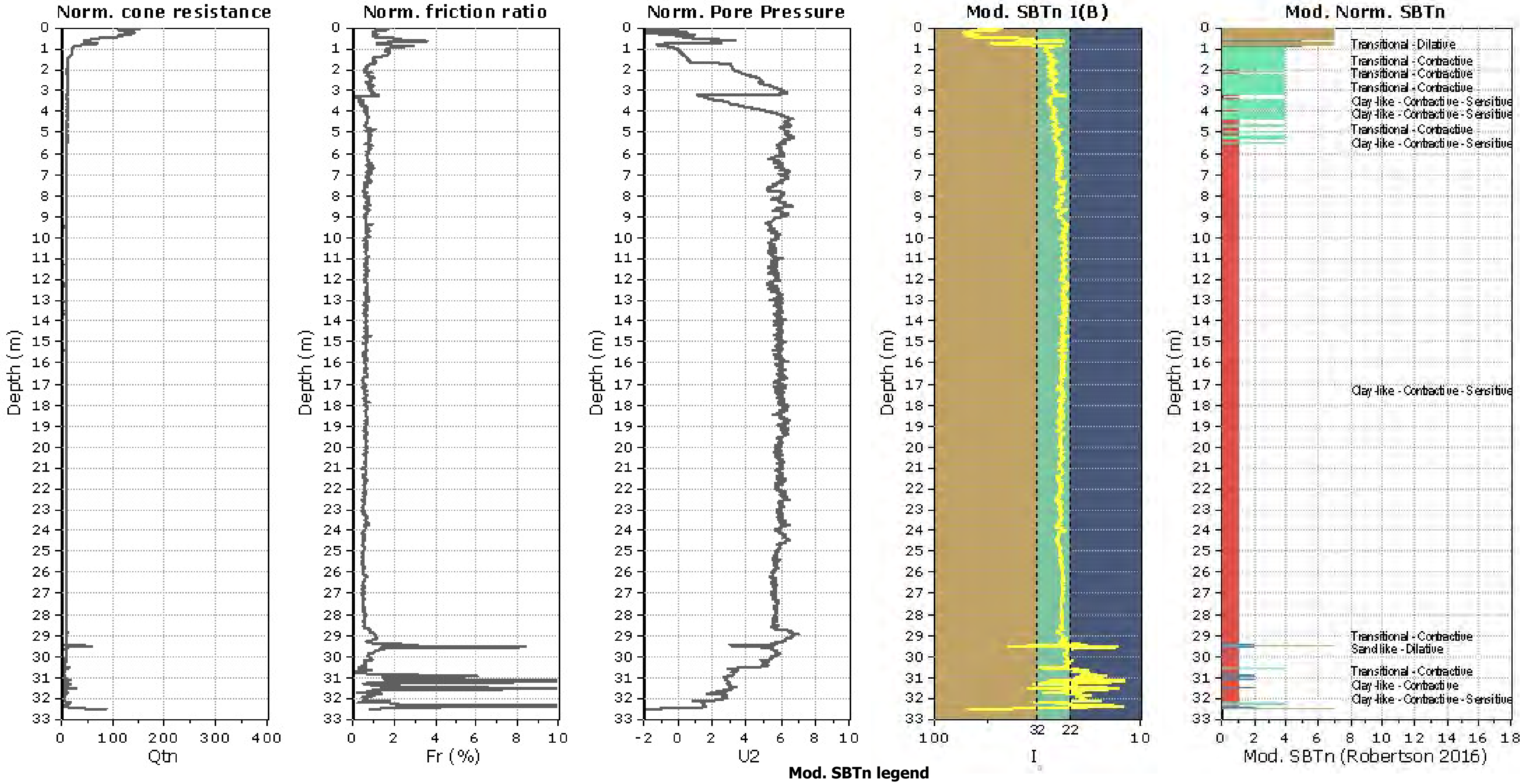
Cone Operator: Kevin Simoneau, P.Eng, M.Sc.

**Project:** **3053 & 3079 Navan Road**
**Location:** **Navan / Pagé Roads, Ottawa**



**Mod. SBTn legend**

| | |
|---|---|
| 🟥 | 1. CCS: ClayLike - Contractive, Sensitive |
| 🟦 | 2. CC: Clay-like - Contractive |
| 🟦 | 3. CD: Clay-Like: Dilative |
| 🟩 | 4. TC: Transitional - Contractive |
| 🟩 | 5. TD: Transitional - Dilative |
| 🟧 | 6. SC: Sand-like - Contractive |
| 🟫 | 7. SD: Sand-like - Dilative |

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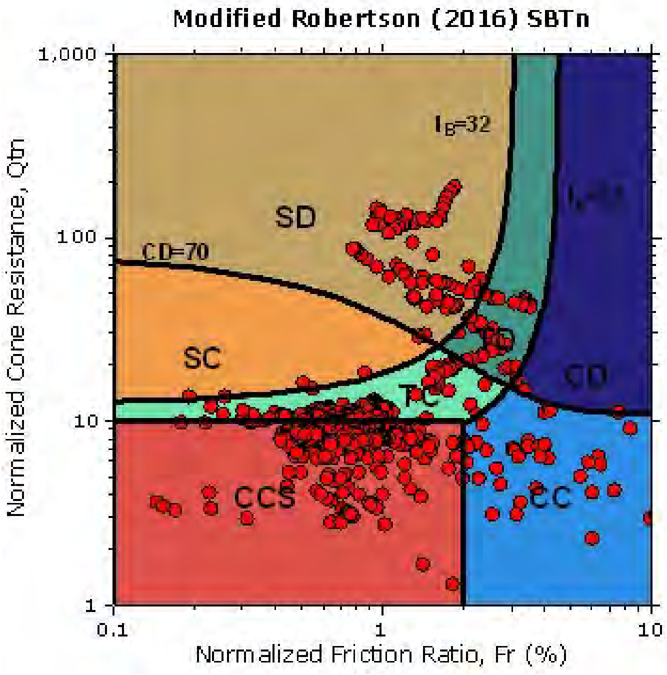
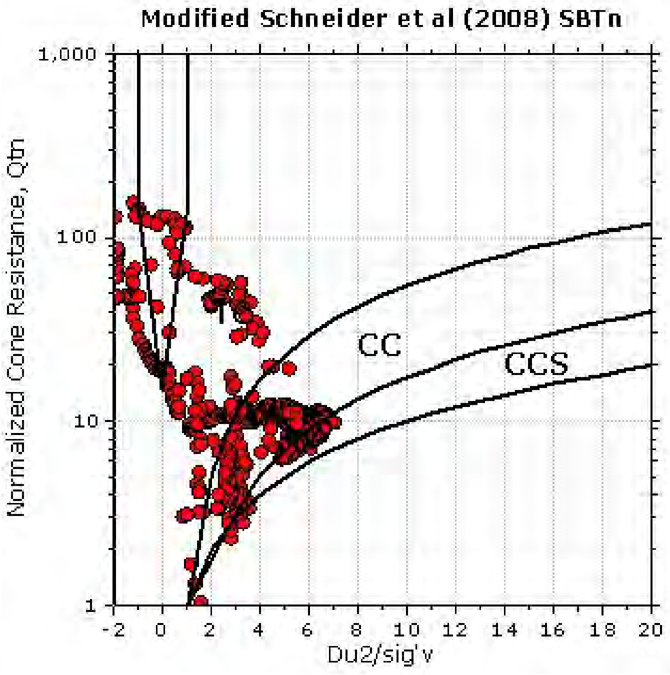
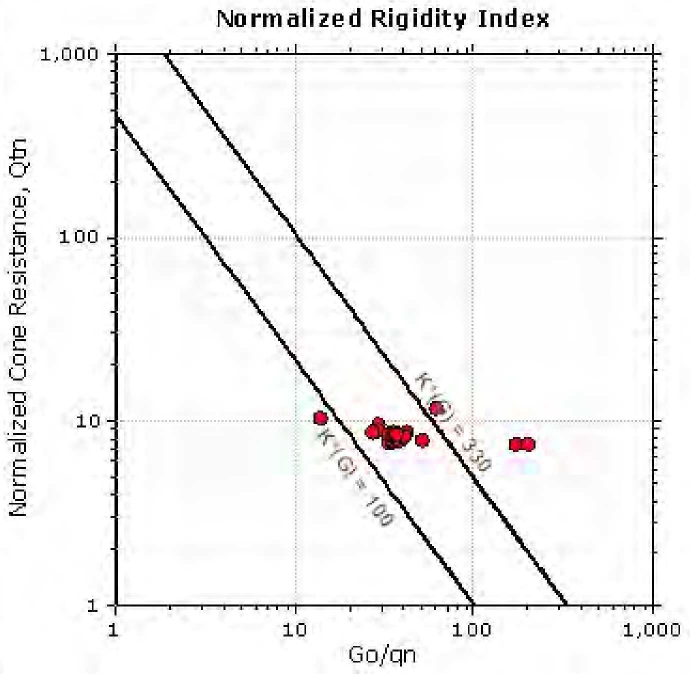
Cone Type: Vertek 4544 - 5t
Cone Operator: Kevin Simoneau, P.Eng, M.Sc.

**Project:** **3053 & 3079 Navan Road**
**Location:** **Navan / Pagé Roads, Ottawa**

# Updated SBTn plots


Modified Robertson (2016) SBTn


Modified Schneider et al (2008) SBTn


Normalized Rigidity Index

CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

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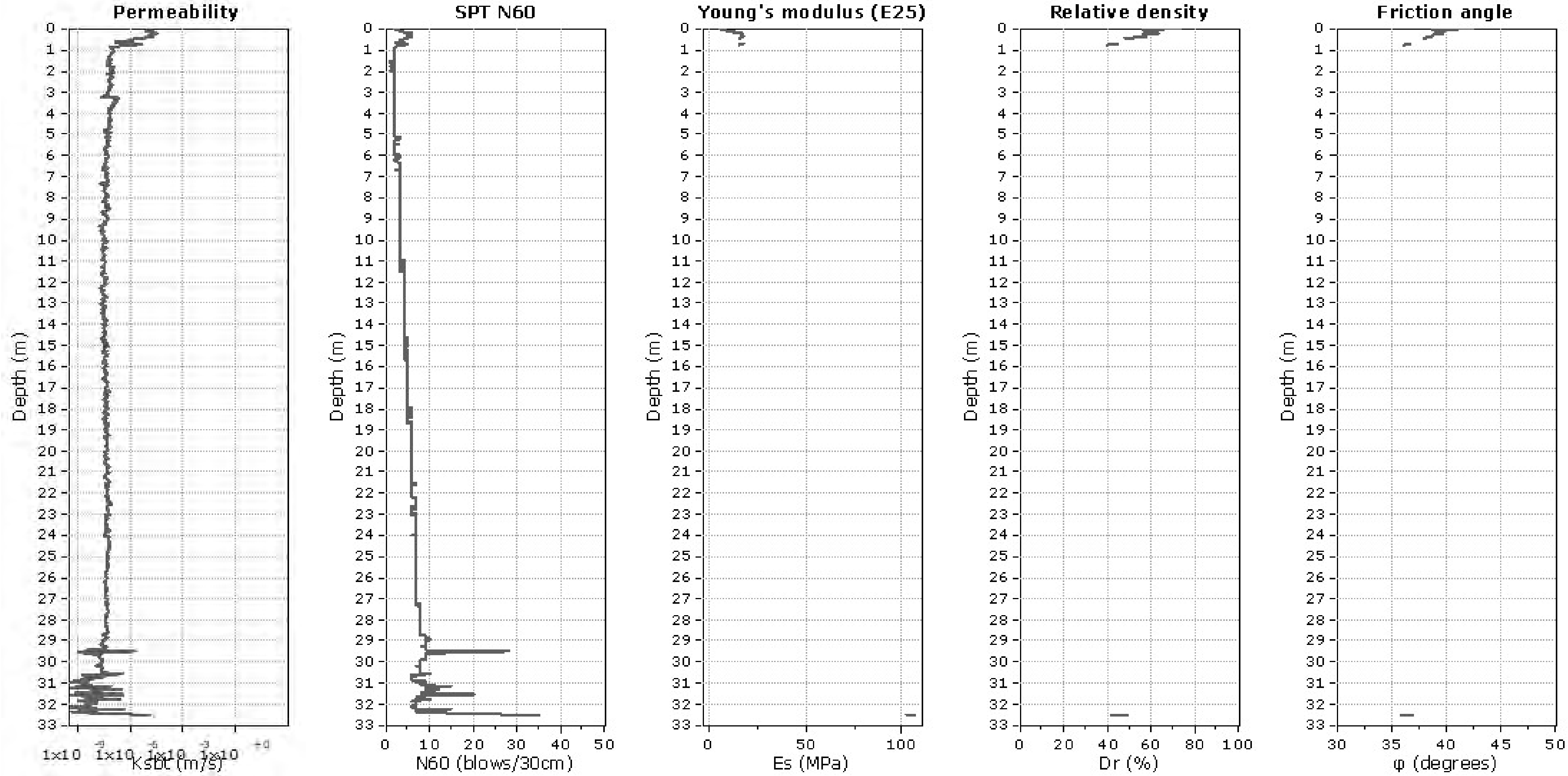
**Project:** **3053 & 3079 Navan Road**
**Location:** **Navan / Pagé Roads, Ottawa**

**CPT: SCPTu-9**
Total depth: 32.51 m
Surface Elevation: 84.70 m
Cone Type: Vertek 4544 - 5t
Cone Operator: Kevin Simoneau, P.Eng, M.Sc.



**Calculation parameters**

Permeability: Based on $SBT_n$
SPT $N_{60}$: Based on $I_c$ and $q_t$
Young's modulus: Based on variable alpha using $I_c$ (Robertson, 2009)

Relative density constant, $C_{Dr}$: 350.0
Phi: Based on Kulhawy & Mayne (1990)
—●— User defined estimation data

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**CPT: SCPTu-9**
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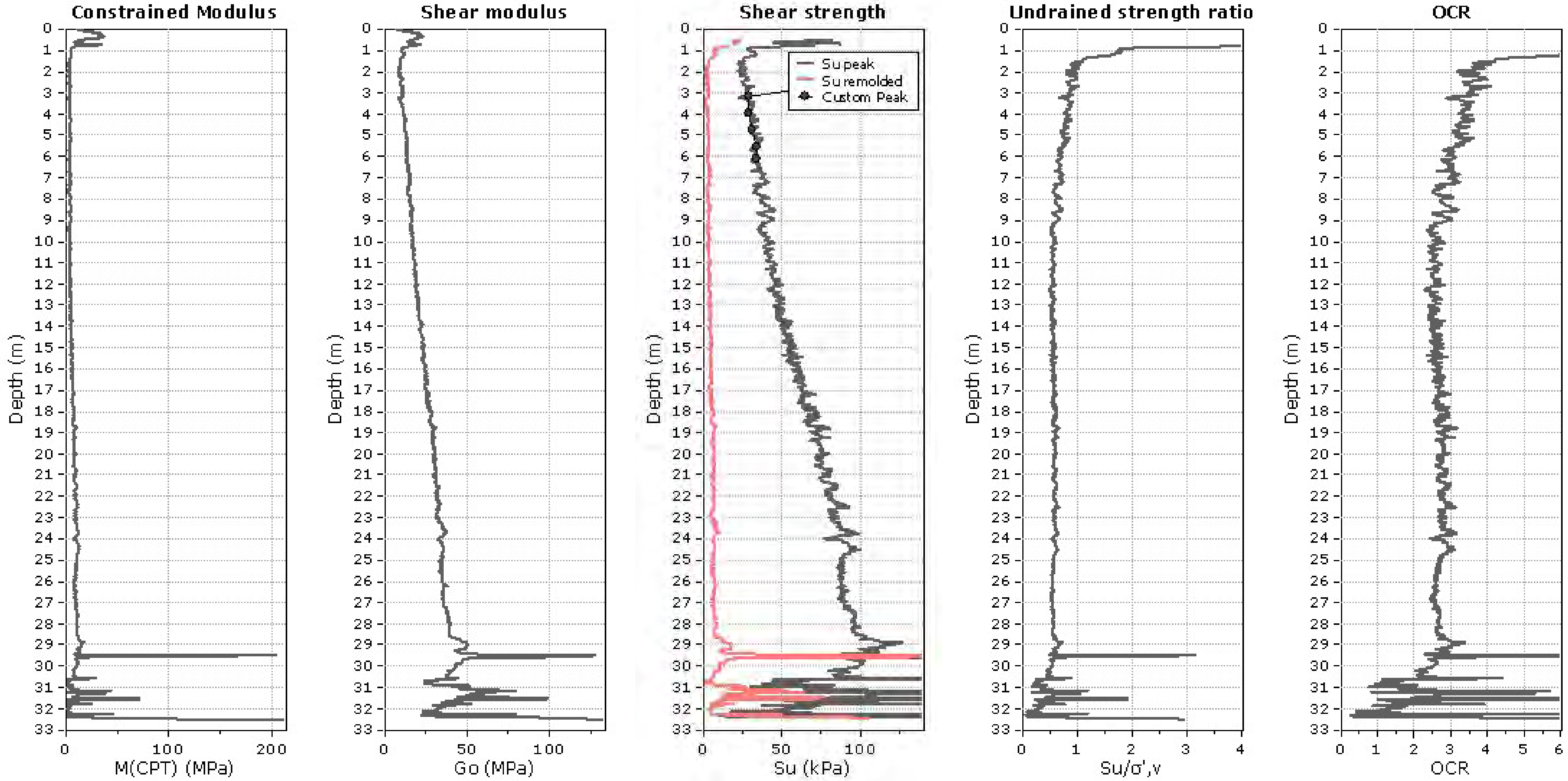
Surface Elevation: 84.70 m
Cone Type: Vertek 4544 - 5t
Cone Operator: Kevin Simoneau, P.Eng, M.Sc.



**Calculation parameters**

Constrained modulus: Based on variable alpha using $I_c$ and $Q_{tn}$ (Robertson, 2009)
Go: Based on variable alpha using $I_c$ (Robertson, 2009)
Undrained shear strength cone factor for clays, $N_{kt}$: 14

OCR factor for clays, $N_{kt}$: 0.33
🔴 User defined estimation data
🔵 Flat Dilatometer Test data

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https://ww.exp.com

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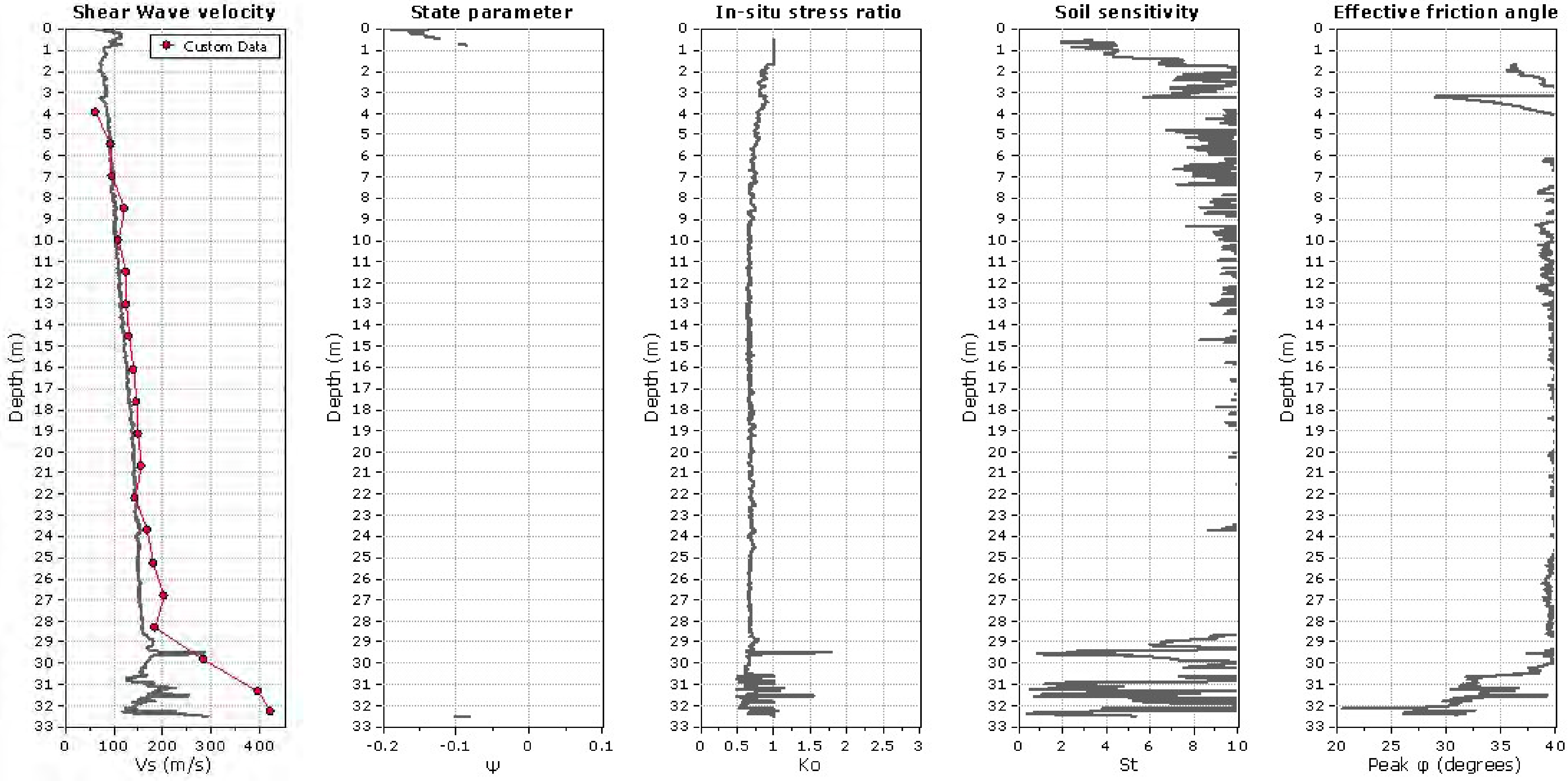
**Location:** **Navan / Pagé Roads, Ottawa**

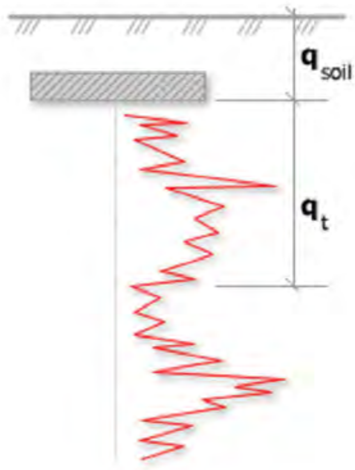
**CPT: SCPTu-9**
Total depth: 32.51 m
Surface Elevation: 84.70 m
Cone Type: Vertek 4544 - 5t
Cone Operator: Kevin Simoneau, P.Eng, M.Sc.



**Calculation parameters**

Soil Sensitivity factor, $N_S$: 7.00

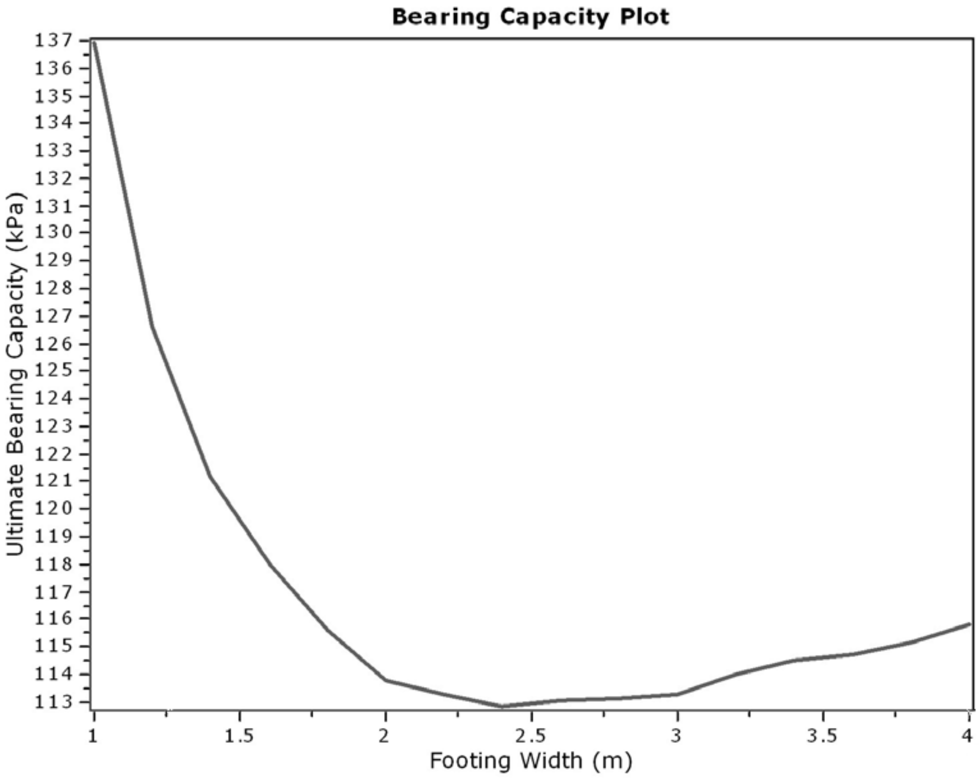
—●— User defined estimation data



Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
$R_k$: Bearing capacity factor
$q_t$: Average corrected cone resistance over calculation depth
$q_{soil}$: Pressure applied by soil above footing



**:: Tabular results ::**

| No | B (m) | Start Depth (m) | End Depth (m) | Ave. $q_t$ (MPa) | $R_k$ | Soil Press. (kPa) | Ult. bearing cap. (kPa) |
|---|---|---|---|---|---|---|---|
| 1 | 1.00 | 0.50 | 2.00 | 0.64 | 0.20 | 9.50 | 136.93 |
| 2 | 1.20 | 0.50 | 2.30 | 0.59 | 0.20 | 9.50 | 126.65 |
| 3 | 1.40 | 0.50 | 2.60 | 0.56 | 0.20 | 9.50 | 121.16 |
| 4 | 1.60 | 0.50 | 2.90 | 0.54 | 0.20 | 9.50 | 117.98 |
| 5 | 1.80 | 0.50 | 3.20 | 0.53 | 0.20 | 9.50 | 115.57 |
| 6 | 2.00 | 0.50 | 3.50 | 0.52 | 0.20 | 9.50 | 113.75 |
| 7 | 2.20 | 0.50 | 3.80 | 0.52 | 0.20 | 9.50 | 113.28 |
| 8 | 2.40 | 0.50 | 4.10 | 0.52 | 0.20 | 9.50 | 112.83 |
| 9 | 2.60 | 0.50 | 4.40 | 0.52 | 0.20 | 9.50 | 113.01 |
| 10 | 2.80 | 0.50 | 4.70 | 0.52 | 0.20 | 9.50 | 113.10 |
| 11 | 3.00 | 0.50 | 5.00 | 0.52 | 0.20 | 9.50 | 113.28 |
| 12 | 3.20 | 0.50 | 5.30 | 0.52 | 0.20 | 9.50 | 113.97 |
| 13 | 3.40 | 0.50 | 5.60 | 0.52 | 0.20 | 9.50 | 114.47 |
| 14 | 3.60 | 0.50 | 5.90 | 0.53 | 0.20 | 9.50 | 114.73 |
| 15 | 3.80 | 0.50 | 6.20 | 0.53 | 0.20 | 9.50 | 115.17 |
| 16 | 4.00 | 0.50 | 6.50 | 0.53 | 0.20 | 9.50 | 115.79 |

# Dissipation Tests Results

## Dissipation tests

Dissipation tests consists of stopping the piezocone penetration and observing porepressures (u) with elapsed time (t). The data are automatic recorded by the field computer and should take place until a minimum of 50% dissipation.

The porepressures are plotted as a function of square root of (t). The graphical technique suggested by Robertson and Campanella (1989), yields a value for $t_{50}$, which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction $c_h$ was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{T \times r^2 \times I_r^{0.5}}{t_{50}}$$

where:

T: time factor given by Houlsby and Teh's (1988) theory corresponding to the porepressure position
r: piezocone radius
$I_r$: stiffness index, equal to shear modulus G divided by the undrained strength of clay ($S_u$).
$t_{50}$: time corresponding to 50% consolidation

## Permeability estimates based on dissipation test

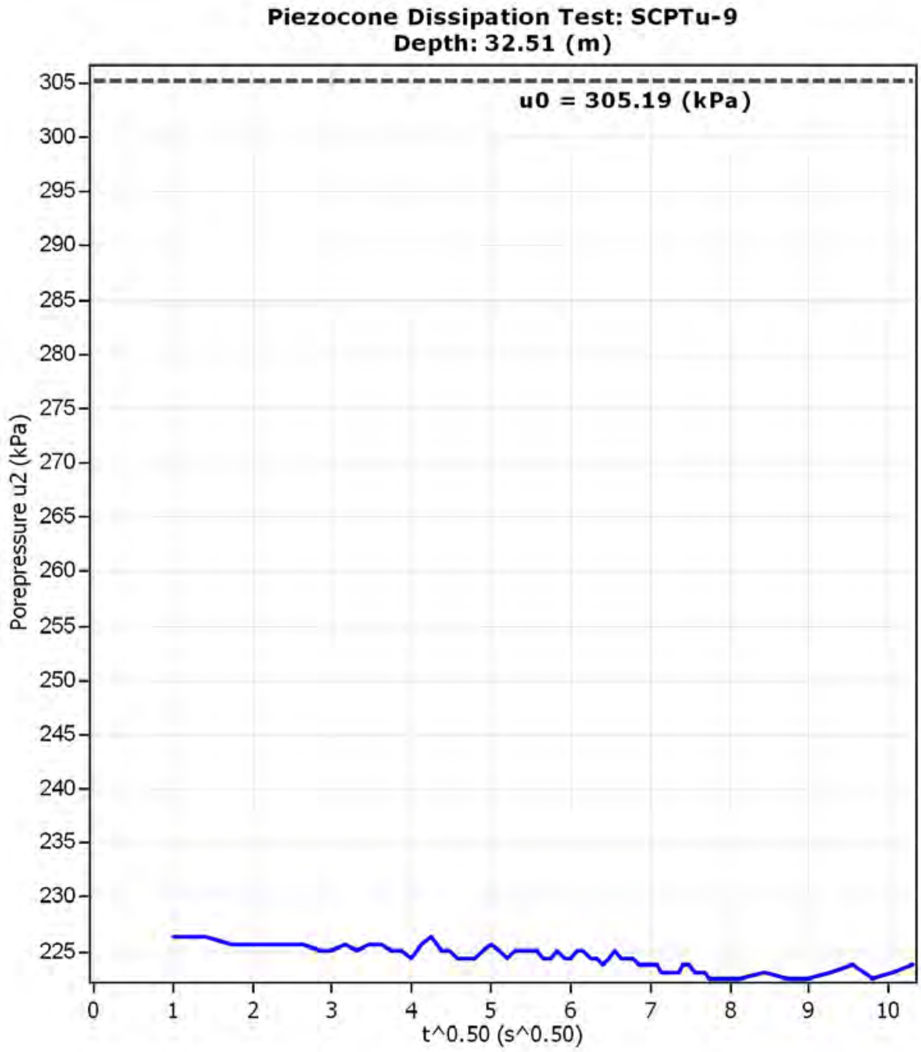
The dissipation of pore pressures during a CPTu dissipation test is controlled by the coefficient of consolidation in the horizontal direction ($c_h$) which is influenced by a combination of the soil permeability ($k_h$) and compressibility (M), as defined by the following:

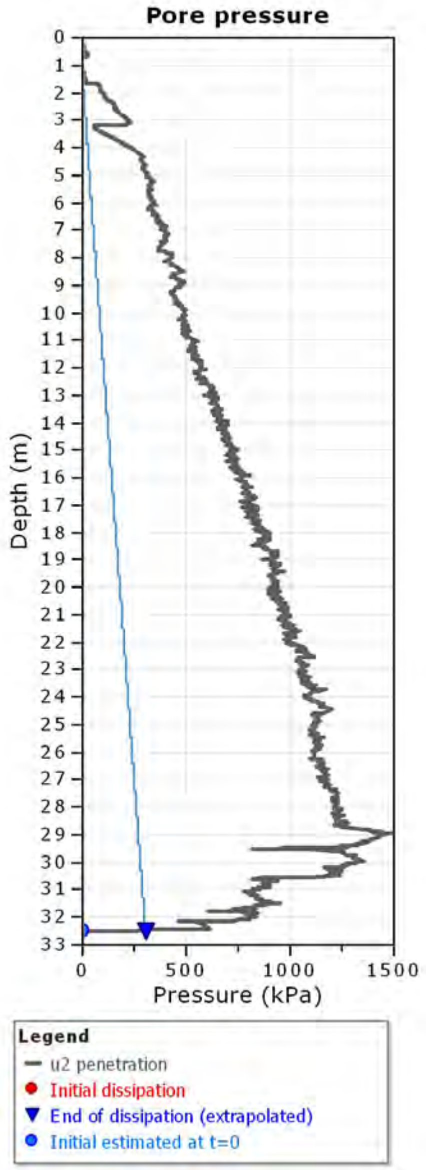
$$k_h = c_h \times \gamma_w / M$$

where: M is the 1-D constrained modulus and $\gamma_w$ is the unit weight of water, in compatible units.

### Tabular results

| CPTU Borehole | Depth (m) | $(t_{50})^{0.50}$ | $t_{50}$ (s) | $t_{50}$ (years) | $G/S_u$ | $c_h$ ($m^2/s$) | $c_h$ ($m^2/year$) | M (MPa) | $k_h$ (m/s) |
|---|---|---|---|---|---|---|---|---|---|
| SCPTu-9 | 32.51 | 0.0 | 0 | 0.00E+000 | 100.00 | 0.00E+000 | 0 | 187.13 | -1.00E+004 |


Piezocone Dissipation Test: SCPTu-9
Depth: 32.51 (m)
u0 = 305.19 (kPa)
Porepressure u2 (kPa)
305
300
295
290
285
280
275
270
265
260
255
250
245
240
235
230
225
t^0.50 (s^0.50)
0
1
2
3
4
5
6
7
8
9
10


Pore pressure
Depth (m)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
Pressure (kPa)
0
500
1000
1500
Legend
u2 penetration
Initial dissipation
End of dissipation (extrapolated)
Initial estimated at t=0

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

**:: Unit Weight, g (kN/m³) ::**

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where $g_w$ = water unit weight

**:: Permeability, k (m/s) ::**

$$I_c < 3.27 \text{ and } I_c > 1.00 \text{ then } k = 10^{0.952-3.04 \cdot I_c}$$

$$I_c \leq 4.00 \text{ and } I_c > 3.27 \text{ then } k = 10^{-4.52-1.37 \cdot I_c}$$

**:: $N_{SPT}$ (blows per 30 cm) ::**

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

**:: Young's Modulus, Es (MPa) ::**

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

**:: Relative Density, Dr (%) ::**

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to $SBT_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

**:: State Parameter, ψ ::**

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

**:: Drained Friction Angle, φ (°) ::**

[illegible]

(applicable only to $SBT_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

**:: 1-D constrained modulus, M (MPa) ::**

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$
[illegible]

**:: Small strain shear Modulus, Go (MPa) ::**

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

**:: Shear Wave Velocity, Vs (m/s) ::**

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

**:: Undrained peak shear strength, Su (kPa) ::**

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to $SBT_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

**:: Remolded undrained shear strength, Su(rem) (kPa) ::**

$S_{u(rem)} = f_s$ (applicable only to $SBT_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

**:: Overconsolidation Ratio, OCR ::**

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to $SBT_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

**:: In situ Stress Ratio, Ko ::**

$$K_O = (1 - \sin \varphi') \cdot OCR^{\sin \varphi'}$$

(applicable only to $SBT_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

**:: Soil Sensitivity, $S_t$ ::**

$$S_t = \frac{N_S}{F_r}$$

(applicable only to $SBT_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

**:: Peak Friction Angle, φ' (°) ::**

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot \left(0.256 + 0.336 \cdot B_q + \log Q_t\right)$$

(applicable for $0.10 < B_q < 1.00$)

**References**

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

# Appendix B – 2018 Paterson Group Inc. Borehole No. 1

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

## SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Development - Navan Road
Ottawa, Ontario

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

**REMARKS**

**BORINGS BY** CME 18 Power Auger

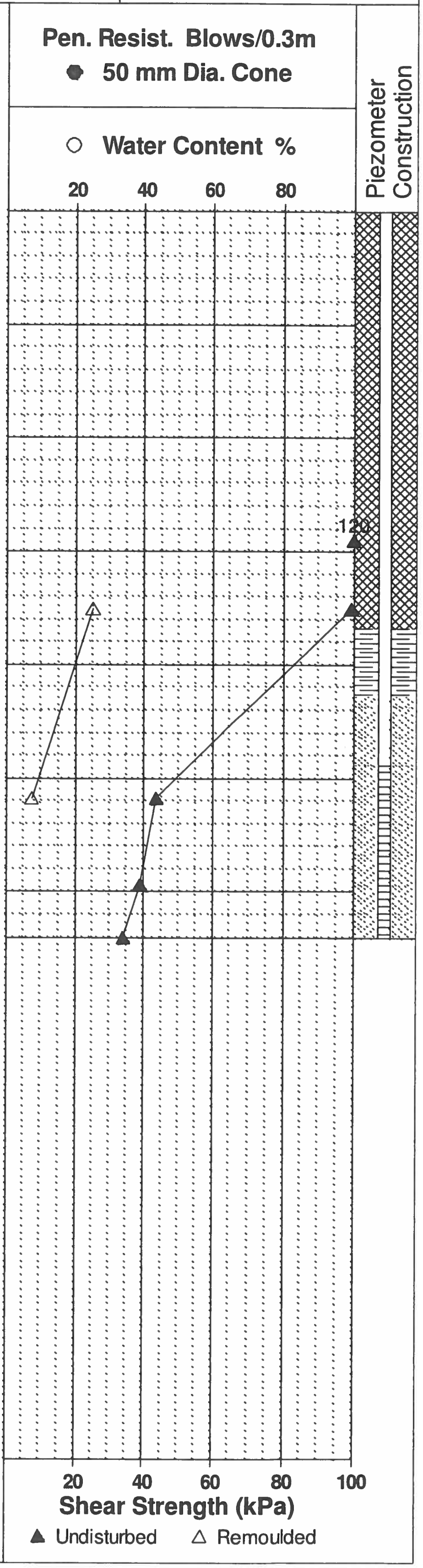
**DATE** 22 May 2018

**FILE NO.** PG4415

**HOLE NO.** BH 1

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD |
|---|---|---|---|---|---|
| GROUND SURFACE | | | | | |
| TOPSOIL 0.20 | | AU | 1 | | |
| Compact, brown **SILTY SAND,** trace gravel, organics 1.45 | | SS | 2 | 83 | 15 |
| Very stiff to stiff, brown **SILTY CLAY** | | SS | 3 | 96 | 5 |
| - stiff to firm and grey by 3.7m depth 6.40 | | SS | 4 | 100 | W |
| End of Borehole | | | | | |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 85.34 |
| 1 | 84.34 |
| 2 | 83.34 |
| 3 | 82.34 |
| 4 | 81.34 |
| 5 | 80.34 |
| 6 | 79.34 |



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