REPORT

Foundation Investigation and Design County Road 31 Underpass Replacement Site No. 31-204, Highway 401 Morrisburg, Ontario

G.W.P 4073-14-00
W.P. 4415-01-01

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Report No. 12-1121-0099/1750
Geocres No. 31B-92

November 2018

Distribution List

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by WSP Canada Group Ltd. (WSP) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the design-build procurement of bridge and culvert replacements at various locations in the Eastern Region of Ontario as part of the 22 Structures MEGA 2 project. Foundation investigation and detail design for several bridge replacements was added to the overall scope of work following award of the project. This report presents the results of the detailed foundation investigation conducted for the replacement of the County Road 31 underpass, Site No. 31-204 (GWP 4073-14-00 and WP 4415-01-01), located on Highway 401 in Morrisburg, Ontario.

The purpose of the current foundation investigation was to assess the subsurface conditions for the proposed bridge replacement by drilling seven boreholes and carrying out in situ and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal (RFP) dated April 2012 for the MEGA 2 project and the work specific to this site was carried out in accordance with Golder's change proposals to WSP, dated October 22, 2014, November 2, 2016, and July 7, 2017. In addition to the above boreholes, two Seismic Cone Penetration Testing (SCPT) holes were added to the geotechnical investigation in 2016, as requested by WSP and MTO for the detailed design of the replacement structures consistent with the 2014 CHBDC (CAN/CSA-S6-14) requirements. In 2017, two additional SCPTs were put down by others at the site in 2017 as part of the peer review of the preliminary design.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 General

The County Road 31 underpass is located on Highway 401 in Morrisburg, Ontario. The existing bridge (Site No. 31-204) is located at about Station 12+808 on Highway 401 (see Key Plan in Drawing 1).

The existing bridge consists of a four-span concrete deck with abutments founded on piles and piers supported on spread footings. The existing structure is aligned approximately north-south and is about 73 m long and 18 m wide. It is understood that the structure was built in 1961.

The natural ground surface varies from about Elevation 81 to 82 m north and south of Highway 401.

In the vicinity of the site, Highway 401 is a four-lane, divided highway and County Road 31 will become a two-lane roadway. In the area of the underpass, County Road 31 has been constructed on embankments that are up to about 6 to 7 m in height above Highway 401 and the natural ground level, with the pavement surface ranging from about Elevation 87.7 to 88.6 m in the vicinity of the bridge. The County Road 31 embankment side slopes are oriented at about 2.5 horizontal to 1 vertical (2.5H:1V). Based on visual observation at the time of the site investigation, the existing embankment side slopes appear to be performing satisfactorily.

It is understood that the bridge replacement will be along a new alignment immediately adjacent to the east of the existing structure.

2.2 Regional Geological Conditions

The site is located in the physiographic region known as the Glengarry Till Plain, just east of the Edwardsburg Sand Plain, as delineated in The Physiography of Southern Ontario.[1]

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[1] Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*. Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

The Glengarry Till Plain is characterized by an undulating to rolling ground surface where the depth to bedrock is typically less than 30 m and glacial till is typically less than 7 m deep.[1]

3.0 INVESTIGATION PROCEDURES

3.1 2015/2016 Investigation (Golder)

The initial subsurface investigation for the proposed underpass bridge replacement was carried out between November 24 and December 16, 2015 at which time seven boreholes (numbered 15-1 to 15-7, inclusive and 15-4-1) were advanced at the locations shown on Drawing 1 and outlined below. On December 20, 2016 two Seismic Cone Penetration Test (SCPT) holes (numbered SCPT 16-108B and SCPT 16-109A) were advanced from the Highway 401 grade to effective refusal. These SCPT test holes are shown on Drawing 1 and outlined below.

- Boreholes 15-1, 15-2, 15-6, and 15-7 were located adjacent to the toes of the existing County Road 31 approach embankments along the proposed bridge realignment to the east. The boreholes were advanced using portable drilling equipment supplied and operated by OGS Inc. of Almonte, Ontario with near-continuous sampling procedures to depths of about 7.3 to 7.5 m (Elevations 73.8 to 74.5 m) below the existing ground surface.

- Boreholes 15-3 and 15-5 were located within the northbound lane of County Road 31 adjacent to the abutments. The boreholes were advanced using 105 mm inside diameter continuous-flight hollow-stem augers and/or wash boring using HW casing with a truck-mounted drill rig, supplied and operated by CCC Geotechnical & Environmental Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of about 31.2 and 32.8 m (Elevations 56.5 and 55.9 m) below the existing pavement surface in the overburden, to the bedrock surface. The boreholes were then cored between about 3.1 to 3.4 m into the bedrock using HQ-size coring equipment.

- Boreholes 15-4 and 15-4-1 were located within the median of Highway 401 adjacent to the eastern side of the central pier. Borehole 15-4 was advanced to a depth of about 25.1 m (Elevation 56.8 m) below the existing ground surface using a combination of 105 mm inside diameter continuous-flight hollow-stem augers and wash boring using HW casing through the overburden to the bedrock surface. The borehole was then cored about 5.5 m into the bedrock using HQ-size coring. Borehole 15-4-1 was advanced without sampling using a dynamic cone penetration test (DCPT) to a depth of about 25.0 m (Elevation 56.8 m) below the existing ground surface where refusal was encountered. The upper 3 m of the borehole was then augered using the equipment described above to allow for installation of a groundwater level monitoring device. Both of these boreholes were also carried out with a truck-mounted drill rig, supplied and operated by CCC Geotechnical & Environmental Drilling Ltd. of Ottawa, Ontario.

- The location of SCPT16-108B was pre-drilled on the right shoulder lane of Highway 401 westbound (adjacent to County Road 31 north abutment) and the location of SCPT16-109A was pre-drilled on the right shoulder lane of Highway 401 eastbound (adjacent to County Road 31 south abutment). Both test holes were pre-drilled by auger to 5.2 metres below ground surface (mbgs) and backfilled with sand for subsequent SCPT testing. SCPT16-108B was advanced to cone refusal at 21.8 mbgs (Elevation 59.9 m) and SCPT16-109A was advanced to cone refusal at 25.5 mbgs (Elevation 56.5 m). The pre-drilling was carried out with a truck-mounted drill rig, supplied and operated by CCC Geotechnical & Environmental Drilling Ltd. of Ottawa, Ontario and the SCPT test holes were completed with a CPT truck rig (C3) supplied and operated by ConeTec Investigations Ltd. of Richmond Hill, Ontario.

Soil samples in the boreholes were obtained at vertical intervals of about 0.6 to 3.0 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

Standpipe piezometers were installed in Boreholes 15-2 and 15-4-1 to monitor the groundwater level at the site. The standpipes consist of either 19 or 25 mm inside diameter rigid PVC pipe with up to a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The water levels in the standpipe piezometers were measured on April 1, 2016. The ground water levels at the SCPT locations were inferred based on pore water pressure measurement taken during SCPT advancement. Pore pressure dissipation tests were also carried out in SCPT16-109A. The summary and plots of the pore pressure dissipation tests are included in Conetec report in Appendix C.

At borehole 15-4 a 60 mm inside diameter rigid PVC casing was grouted for the full advancement depth (i.e., through the overburden and into the bedrock) to allow for future Vertical Seismic Profile testing to support the selection of a seismic Site Class for the site and in the assessment of liquefaction potential. Shear wave velocity testing was carried out as part of seismic cone penetration testing. A built in geophone within the cone penetration probe recorded seismic wave traces from a surface source as the CPTs were advanced. Measurements were recorded at roughly one meter intervals from depths between 5.9 and 21.8 to 25.5 meters. A more detailed description of the test methodology is provided in Conetec report in Appendix C.

The boreholes were backfilled with bentonite pellets, mixed with native soils in the overburden and bentonite pellets in the bedrock, except as indicated previously for standpipe piezometers. The site conditions were substantially restored following completion of work.

The 2015/2016 field work was supervised by members of Golder's technical and engineering staff, who located the boreholes and SCPTs, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratory in Ottawa for further examination. Index and classification tests consisting of grain size distribution, Atterberg limits, organic content and water content testing were carried out on selected soil samples at the Golder Ottawa laboratory. Axial point load tests and unconfined compressive strength tests were carried out on selected rock core samples in the Golder Mississauga laboratory. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

Prior to drilling, the locations of the boreholes, SCPT 16-108B and SCPT 16-109A were staked and surveyed by Golder personnel using a Trimble R8 GPS unit. The boreholes and locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawing 1.

Test Hole Number	Test Hole Location	Northing[1] (m)	Easting[1] (m)	Ground Surface Elevation[2] (m)
15-1	Adjacent to South Embankment	4975936.6	407704.1	81.8
15-2	Adjacent to South Embankment	4975981.8	407686.7	81.3
15-3	South Abutment	4976011.7	407653.6	87.7
15-4 and 15-4-1	Central Pier (Within the median of Highway 401)	4976053.3	407638.9	81.8
15-5	North Abutment	4976093.0	407614.7	88.6
15-6	Adjacent to North Embankment	4976137.3	407615.7	81.8

Test Hole Number	Test Hole Location	Northing[1] (m)	Easting[1] (m)	Ground Surface Elevation[2] (m)
15-7	Adjacent to North Embankment	4976181.6	407590.4	81.3
SCPT 16-108B	Adjacent to County Road 31 North Abutment	4976066.7	407613.0	81.7
SCPT 16-109A	Adjacent to County Road 31 South Abutment	4976031.2	407637.7	82.0

[1] Northing and Easting coordinates shown are relative to the MTM NAD83 (Zone 9) coordinate system.
[2] Ground surface elevations shown are relative to Geodetic Datum.

3.2 2017 Investigation (Thurber)

A supplementary subsurface investigation was carried out by Thurber Engineering Ltd. (Thurber) on May 24, 2017 at which time an additional three SCPTs (numbered SCPT 17-01, SCPT 17-02, and SCPT 17-02B) were put down through the embankments from the County Road 31 grade. These SCPT test holes are also shown on Drawing 1 and outlined below.

- The location of SCPT 17-01 was pre-drilled on the southbound lane of County Road 31 (about 14 m north of the north abutment) and the location of SCPT 17-02/02B was pre-drilled on the southbound lane of County Road 31 (about 15 m south of the south abutment). SCPT 17-01 was pre-drilled to a depth of about 1.6 m below the existing County Road 31 grade prior to CPT advancement. SCPT 17-02 was pre-drilled to about 10.7 mbgs but the CPT probe met effective refusal at a depth of about 11.6 mbgs and was terminated. SCPT 17-02B was pre-drilled at an adjacent location to a depth of about 12.0 mbgs, and the CPT probe advanced with no further issue.
- The pre-drilling was carried out with a truck-mounted drill rig, supplied and operated by Downing Drilling Ltd. of Hawkesbury, Ontario and the SCPT test holes were completed with a CPT truck rig (C3) supplied and operated by ConeTec Investigations Ltd. of Richmond Hill, Ontario. The field work was carried out on behalf of Thurber.

Pore pressure dissipation tests were carried out at various depths, and shear wave velocity measurements were obtained out at roughly one metre intervals during advancement of the SCPTs. A more detailed description of the test methodologies carried out during advancement of SCPT 17-01 and SCPT17-02B is provided in the Conetec report in Appendix D.

The locations of the SCPTs, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawing 1. The coordinates and ground surface elevations were provided by others.

Test Hole Number	Test Hole Location	Northing[1] (m)	Easting[1] (m)	Ground Surface Elevation[2] (m)
SCPT 17-01 [3]	County Road 31 North Embankment	4976090.1	407604.9	88.3
SCPT 17-02B [3]	County Road 31 South Embankment	4976000.6	407648.0	87.6

[1] Northing and Easting coordinates shown are relative to the MTM NAD83 (Zone 9) coordinate system.
[2] Ground surface elevations shown are relative to Geodetic Datum.
[3] Coordinates and ground surface elevations provided by others.

4.0 SITE STRATIGRAPHY

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of related in situ and laboratory testing are given on the Record of Borehole and Drillhole sheets contained in Appendix A. The results of geotechnical laboratory testing are also included in Appendix B. The results of the 2016 and 2017 seismic cone penetration testing are provided in Appendices C and D, respectively, which include the result of shear wave velocity tests and pore pressure dissipation tests.

The interpreted stratigraphic conditions along the centreline of the proposed bridge realignment are shown on Drawing 1. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic section included on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the surficial soils at the location of the existing bridge consist of the embankment fill at the abutments and grade fill at the central pier location, where the surficial soils along the proposed alignment of the new approach embankments consist of topsoil, alluvium and peat, underlain by thin layers silty sand and silty clay. Similar layers were also encountered beneath the embankment fill, but of limited thickness. The embankment fill and surficial deposits are underlain by glacial till, overlying limestone and dolostone bedrock.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.1 Pavement Structure and Embankment Fill

The County Road 31 pavement structure was penetrated in the northbound lane at Boreholes 15-3 and 15-5. At the borehole locations, the pavement structure consists of about 200 mm of asphaltic concrete overlying about 1.0 to 1.2 m of sand and gravel base/subbase. The granular base/subbase is underlain by about 7.0 m of embankment fill. The embankment fill generally consists silty sand to sandy silt with varying amounts of clay and gravel. The embankment fill was fully penetrated to depths of about 8.3 to 8.5 m (Elevations 79.4 to 80.1 m) at Boreholes 15-3 and 15-5, respectively.

The grade fill within the median of Highway 401 was penetrated at Borehole 15-4. At the borehole location, the pavement structure consisted of about 1.2 m of crushed stone over 1.7 m of silty sand fill. Gravel, organic matter and cobbles were also encountered within the silty sand fill.

Standard Penetration Test (SPT) “N” values measured in the embankment and grade fills ranged from 4 to 33 blows per 0.3 m of penetration indicating a loose to dense relative density.

The results of grain size distribution testing carried out samples of the embankment fill are provided on Figure B1 in Appendix B. The measured water content of selected samples of the embankment fill ranges from approximately 5 to 12 percent.

4.2 Topsoil, Alluvium and Peat

Surficial layers of topsoil, organic clayey silt, alluvium and/or peat were encountered at ground surface at Boreholes 15-1, 15-2, 15-6, and 15-7 and beneath the embankment fill at Borehole 15-5. The surficial layers extend to depths between about 1.1 and 2.4 m (Elevations 79.4 and 80.2 m) below the existing ground surface. The layer of alluvium encountered beneath the embankment fill at Borehole 15-5 is about 0.4 m thick (Elevation 80.1 m).

SPT "N" values ranging from 3 to 16 blows per 0.3 m of penetration were as measured within the deposits.

The measured organic content of the alluvium ranged from about 8 to 11 percent, where one organic content of about 56 percent was measured in the peat layer. The water content of selected samples of the organic deposits range from approximately 22 to 242 percent.

4.3 Silty Sand and Sand

The organic deposits, where present, are underlain by layers of silty sand to sand. The sandy layers vary in thickness from about 0.1 to 1.3 m and were encountered between Elevations 79.4 and 80.2 m.

SPT "N" values ranging from 11 to 15 blows per 0.3 m of penetration were measured within the sandy deposits indicating a compact sate of packing.

The results of grain size distribution testing carried out on one sample of the silty sand are provided on Figure B2 in Appendix B. The measured water content of the sample of silty sand was approximately 20 percent.

4.4 Silty Clay to Clay

A layer of silty clay to clay was encountered beneath the embankment fill or silty sand at Boreholes 15-1, 15-3, and 15-5. The deposit varies in thickness from about 0.2 to 0.7 m and was encountered between Elevations 78.7 and 79.4 m.

Two SPT "N" values of 11 and 18 blows per 0.3 m of penetration were recorded within the deposit, indicating a very stiff consistency.

Atterberg limit testing carried out on three samples of the deposit measured plasticity indices of about 13, 26, and 37 percent and a liquid limits from about 30, 45, and 58 percent, as shown on Figure B3, indicating that the deposit has an low to high plasticity. The measured natural water content of the samples of this material range from about 26 to 38 percent.

4.5 Glacial Till

The fill, silty sand, sand, silty clay and clayey silt, where present, are underlain by a deposit of glacial till. In general, the glacial till is a heterogeneous mixture of gravel, cobbles and boulders in a matrix of silty sand to sandy silt trace clay. The surface of the till deposit was encountered between Elevations 78.2 and 79.3 m. The deposit was fully penetrated in Boreholes 15-3, 15-4 and 5-5 to depths between about 22.9 and 32.8 m (Elevations 58.9 to 55.8 m). At these borehole locations, the glacial till had a thickness between about 20.0 and 22.7 m. Boreholes 15-1, 15-2, 15-6, and 15-7 were terminated in the glacial till at depths between about 7.3 to 7.5 m (Elevations 74.5 to 73.8 m).

SPT "N" values measured in the glacial till range from about 1 to greater than 63 blows per 0.3 m of penetration. The SPT "N" values measured in the upper portion of the glacial till deposit (above about Elevation 73.9 to 77.3 m) range from 12 to greater than 63 blows per 0.3 m of penetration, indicating a compact to very dense state of packing. Within the lower portion of the glacial till deposit (below about Elevation 73.9 to 77.3 m), SPT "N" values range from 1 to 15 blows per 0.3 m of penetration indicating a very loose to compact state of packing. The dynamic cone penetration test (DCPT) carried out within Borehole 15-4-1 further indicates this increased resistance down to Elevation 73.0 m with no further increase below this elevation. The DCPT encountered refusal at Elevation 56.8 m, the bedrock elevation in the adjacent Borehole 15-4.

The measured tip resistances within the glacial till at the SCPT locations were generally in the range of 630 to 2,500 kPa, with localized “spikes” in the tip resistance of up to 20,200 kPa which likely reflect coarse grained layers (e.g., gravel, cobbles or boulders within the deposit). The measured shear wave velocity (V_s) in the glacial till ranges between 330 and 560 m/s. A more detailed information of the results of the SCPT testing is provided in Appendices C and D.

At borehole 15-5, rotary diamond drilling techniques were required to advance through the glacial till deposit between about Elevation 55.8 and 57.0 m.

The results of grain size distribution testing carried out on multiple samples of the glacial till are provided on Figures B4a and B4b in Appendix B. These test results do not reflect the cobble/boulder or full gravel content of the material, since the samples were retrieved using a 50 mm outside diameter split-spoon sampler.
The measured natural water contents of selected samples of the till ranged from about 6 to 19 percent.

The results of Atterberg limit testing carried out on samples of the glacial till indicate plasticity indices of about 3 to 8 percent and liquid limits of about 15 to 20 percent, as shown on Figure B5, indicating the deposit has slight to low plasticity (note: samples sieved through No. 40 sieve beforehand).

An approximately 0.4 m thick layer of gravel, cobbles and boulders was inferred to have been excavated with the glacial till in boreholes 15-2 and 15.7 at about Elevation 77.0 m.

An approximately 0.2 m thick layer of silty clay was encountered within the glacial till deposit at Borehole 15-3. The results of Atterberg limit testing carried out on one sample of this deposit indicates a plasticity index of about 25 percent and a liquid limit of about 48 percent, as shown on Figure B3, indicating a silty clay of intermediate plasticity. The measured natural water content of the same sample of the deposit was about 38 percent.

4.6 Lower Silty Clay

An approximately 1.5 m thick layer of clayey silt was encountered beneath the glacial till deposit at Borehole 15-4 with its surface at about Elevation 58.9 m and its base at about Elevation 57.4 m.

One measured SPT “N” value within this deposit was 5 blows per 0.3 m of penetration, indicating a very stiff consistency. The results of Atterberg limit testing carried out on one sample of this deposit indicates a plasticity index of about 16 percent and a liquid limit of about 35 percent, as shown on Figure B3, confirming that the deposit is a silty clay of low to intermediate plasticity. The measured natural water content of the same sample of the deposit was about 30 percent.

4.7 Lower Sand

The glacial till at Borehole 15-3 and the silty clay at Borehole 15-4 are underlain by an about 0.7 to 1.0 m thick layer of sand. The sand was encountered at depths of about 30.2 and 24.4 m (Elevations 57.5 and 57.4 m) at Boreholes 15-3 and 15-4, respectively.

Two SPT “N” values of 17 and 35 blows per 0.3 m of penetration were measured within this deposit, indicating a compact state of packing.

The results of grain size distribution testing carried out on one sample deposit are shown on Figure B2 in Appendix B. The measured natural water content of two samples of the deposit were about 8 and 14 percent.

4.8 Refusal and Bedrock

Bedrock was encountered beneath the glacial till and lower sand deposits at Boreholes 15-3, 15-4 and 15-5, where it was cored for lengths between 3.1 and 5.5 m. Dynamic cone penetration refusal was encountered in Boreholes 15-4-1 at a depth of 25.0 m (Elevation 56.8 m); this refusal likely represents the bedrock surface.

The following table summarizes the bedrock surface depths and elevations as encountered at the borehole locations.

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
15-3	87.7	31.2	56.5
15-4	81.8	25.1	56.8
15-4-1	81.8	25.0[1]	56.8[1]
15-5	88.6	32.8	55.9

[1] Depth and elevation to bedrock inferred from DCPT refusal.

The bedrock encountered in the boreholes typically consists of dark grey limestone with thin to medium thick interbeds of shale. The bedrock is fresh, thinly to thickly bedded, fine to medium grained, slightly porous and typically medium strong.

Dolostone bedrock was encountered beneath the limestone bedrock at Borehole 15-4 at about Elevation 52.4 m. The dolostone bedrock is light green to dark grey in colour with thin to medium limestone interbeds. The bedrock is fresh, medium bedded, fine grained, non-porous and medium strong to strong.

The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples range from about 73 to 100 percent, indicating fair to excellent quality rock. One lower RQD value of 47 was measured within the upper 1.6 m of the bedrock at Borehole 15-4 indicating a poor quality rock. The discontinuities observed in the rock core are associated with the joints and bedding of the bedrock.

Laboratory axial point load index testing as well as unconfined compressive strength testing was carried out on selected specimens of the bedrock core. The results of the testing are summarized on Figure B6 and B7 in Appendix B. The correlated compressive strengths from the point load index testing carried out on three samples of the bedrock were about 6, 19 and 105 MPa. The results of the unconfined compressive strength testing on three sample of the bedrock indicate values of 38, 42 and 48 MPa.

4.9 Groundwater Conditions

The groundwater levels measured in the standpipe piezometers in Borehole 15-2, 15-4-1 and those measured during the SCPT testing are presented in the table below.

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
15-2	81.3	0.5	80.8	April 1, 2016
15-4-1	81.8	0.5	81.3	April 1, 2016
SCPT 16-108B	81.7	0.9	79.8[1]	December 20, 2016
SCPT 16-109A	82.0	0.9	81.2[1]	December 20, 2016
SCPT 17-01[2]	88.3	6.9	81.4[1]	May 24, 2017

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
SCPT 17-02 [2]	87.6	6.9	80.7[1]	May 24, 2017
SCPT 17-02B [2]	87.5	6.9	80.6[1]	May 24, 2017

[1] Level inferred from SCPT pre-drilled hole and SCPT data.
[2] Ground surface elevations provided by others.

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.

5.0 CLOSURE

This Foundation Design Report was prepared by Mr. Matt Kennedy, P.Eng., and reviewed by Mr. Michael Snow, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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PART B

Foundation Design Report
County Road 31 Underpass Replacement
Site 31-204, Highway 401
Morrisburg, Ontario
G.W.P. 4073-14-00
W.P. 4415-01-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing County Road 31 underpass (Site No. 31-204) on Highway 401 in Morrisburg, Ontario.
The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigations. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the foundations for the replacement structure. It is understood that the bridge is to be designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC). In accordance with Section 4.4.2 of the CHBDC, we understand that the proposed bridge structure has an importance category of *other* bridge.

Where comments are made on construction, they are provided to highlight those aspects that could affect the detail design of the project, and for which special provisions may be required in the contract documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing bridge is shown in plan on Drawing 1 and consists of a four-lane, four-span concrete deck with abutments on piles and piers supported on spread footings. The two middle spans are about 22.9 m long, and the two outer spans are about 12.2 m long. It is understood that the existing bridge, constructed in 1961, is to be replaced with a two lane, two-span structure with a partial shift in the alignment to east of the existing structure. The new underpass will be founded on integral abutments located within or adjacent to the existing abutment foundation footprints. The proposed pavement grades at the new structure will be up to about 1.8 m higher than the existing pavement grades.

6.2 Seismic Design

6.2.1 Site Seismicity and Importance Category

The site falls within the Western Quebec Seismic Zone (WQSZ) according to the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montréal to Témiscaming. Within the WQSZ, recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the WQSZ includes the 1935 Témiscaming event which had a magnitude (i.e., a measure of the intensity of the earthquake, M_{bLg} or MN) of 6.2 and the 1944 Cornwall-Massena event which had a magnitude of 5.6. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

The CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC). The GSC has developed a new set of seismic hazard maps (referred to as the 5th generation seismic hazard maps) that were made available for public use in December 2015.

6.2.2 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the current field investigation and laboratory testing. The Seismic Cone Penetration Test results and measured shear wave velocity of the soil profile were used to select a seismic site classification in accordance with Table 4.1 of the CHBDC. Based on these results a Site Class C designation would be assigned.

However, Table 4.1 of the CHBDC also specifies circumstances for which a Site Class of F is applicable and a site-specific response evaluation must be carried out; the presence of liquefiable soils is one of those conditions. As discussed below (see Section 6.2.6), the soil at this site may be considered to be non-liquefiable for design and, therefore, a Site Class C designation is considered appropriate.

6.2.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the bridge (latitude 44.917 and longitude 75.198), the following are the reference Site Class C (reference) peak seismic hazard values based on the 5th generation seismic hazard maps published by the GSC.

Seismic Hazard Values for Reference Ground Condition Site Class C

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.355
PGV (m/s)	0.235
Sa (0.2) (g)	0.555
Sa (0.5) (g)	0.285
Sa (1.0) (g)	0.135
Sa (2.0) (g)	0.062
Sa (5.0) (g)	0.016
Sa (10.0) (g)	0.006

As indicated in Section 6.2.2, the fundamental period of the structure is expected to be greater than 0.5 s which in consideration of its *other* importance category and the site-specific seismic hazard values given above, would indicate that the bridge structure falls in Seismic Performance Category 2 in accordance with Table 4.10 of the CHBDC. Based on this Seismic Performance Category and the *regular* geometry of the bridge, it is understood that the structure will be designed using a "force based approach" as defined in the CHBDC.

6.2.4 Site-Specific Ground Response Analysis

During the initial design stages of the project, portions of the site soils were considered to be potentially liquefiable during the design earthquake event and a site-specific ground response analysis was carried out to satisfy the requirements of Site Class F outlined in Table 4.1 of the CHBDC. Since the subsequent updated liquefaction assessment indicated that the site soils may be considered to be non-liquefiable for design, a site-specific ground response analysis is no longer strictly required. However, the results of the analyses remain applicable to the site for other aspects of design.

The site-specific seismic assessment was carried out to model the dynamic ground response at the site as input to the updated liquefaction assessment and to develop a site-specific design spectra under the 2,475 return period earthquake. The assessment was based on the ground motion hazard parameters defined in Section 6.2.3. Further details on the development of the spectrum-compatible input acceleration time histories, and the one-dimensional ground response analyses are included in the following sections.

6.2.4.1 Spectrum-Compatible Time Histories

The CHBDC describes two approaches to scaling input time histories to match a target spectrum: linear scaling and spectral matching. Linear scaling involves simply scaling the ordinates of the record to achieve the best fit to the target response spectrum over the period range of interest. Linear scaling provides input time histories that are more representative of the original records of ground shaking (i.e. less modification), but can be difficult to match the target spectrum over a large period range. Spectral matching involves changing the frequency and phase contents of the record to match the target spectrum. Spectral matching allows for development of input records that provide a closer match to the target spectrum over a broad range of periods but involves more modification of the original records since no real earthquake spectrum will match the entire target spectrum.

The period range of interest for selection and matching of time histories to the target spectra was taken as 0.15 s to 1.5 s, based on the anticipated range of the fundamental period of the bridge provided by WSP and the guidance outlined in Section C4.4.3.6 of the Commentary to the CHBDC.

The target spectra used to scale the input time histories was developed using the 5th generation seismic hazard values given in Section 6.2.3 for Site Class C and modified to the site-specific seismic site classification of Site Class B, representative of the bedrock conditions in accordance with Section 4.4.3.3 of the CHBDC.

Target Seismic Hazard Values for Ground Motion Scaling for Site Class B (bedrock)

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.275
PGV (m/s)	0.140
Sa (0.2) (g)	0.463
Sa (0.5) (g)	0.171
Sa (1.0) (g)	0.072
Sa (2.0) (g)	0.032
Sa (10.0) (g)	0.003

Both linearly-scaled and spectrally matched time histories were used in the site-specific ground response analysis to develop the minimum of 11 sets horizontal ground motions as required by Section 4.4.3.6 of the CHBDC. As part of site-specific ground response analyses previously carried out at a site with a similar seismic hazard, orthogonal pairs of records from five earthquakes retrieved from the Pacific Earthquake Engineering Research (PEER) Center NGA-West2 database were spectrally matched to a target spectrum for Site Class B ground motions similar to that at the County Road 31 underpass site for period range of interest. An additional five simulated time histories were selected from the Engineering Seismology Toolbox (EST) database as outlined in Section C4.4.3.6 of the Commentary to the CHBDC, and linearly scaled to achieve the best fit to the Site Class B target response spectrum within the period range of interest (see Figure E1 in Appendix E).

A summary of the earthquake records used is provided in the table below. Plots of the Site Class B scaled spectral accelerations of the input time histories are shown on Figure D1 along with the target Site Class B spectra.

Summary of Input Time History Earthquake Events

Database	Event Name	Event Year	Station / Suite Name	Mag.	Dist. (km)	Scaling Method
PEER	San Fernando	1971	Lake Hughes #4 (H1)	6.6	19.5	Spectral Matching
PEER	San Fernando	1971	Lake Hughes #4 (H2)	6.6	19.5	Spectral Matching
PEER	N. Palm Springs	1986	Winchester Bergman (H1)	6.1	48.9	Spectral Matching
PEER	N. Palm Springs	1986	Winchester Bergman (H2)	6.1	48.9	Spectral Matching
PEER	Coyote Lake	1979	Gilroy Array #1 (H1)	5.7	10.2	Spectral Matching

Database	Event Name	Event Year	Station / Suite Name	Mag.	Dist. (km)	Scaling Method
PEER	Coyote Lake	1979	Gilroy Array #1 (H2)	5.7	10.2	Spectral Matching
PEER	Northridge	1994	LA - Wonderland Ave (H1)	6.7	15.1	Spectral Matching
PEER	Northridge	1994	LA – Wonderland Ave (H2)	6.7	15.1	Spectral Matching
PEER	Nahanni	1985	Site 3 (H1)	6.8	4.9	Spectral Matching
PEER	Nahanni	1985	Site 3 (H2)	6.8	4.9	Spectral Matching
EST	Motion #09	-	East6a2 Suite	6.0	16.9	Linear Scaling
EST	Motion #11	-	East6a2 Suite	6.0	21.1	Linear Scaling
EST	Motion #22	-	East6a2 Suite	6.0	26.1	Linear Scaling
EST	Motion #35	-	East6a2 Suite	6.0	24.8	Linear Scaling
EST	Motion #36	-	East6a2 Suite	6.0	24.8	Linear Scaling

6.2.5 One-Dimensional Ground Response Analyses

One-dimensional ground response analyses were undertaken to assess the ground response at the site. The ground response analyses were carried out based on the subsurface stratigraphy and using the 2,475-year input ground motions described above.

Based on the results of the 2015/2016 SCPT profiles and boreholes, and an assumed average shear wave velocity of 760 m/s for the bedrock, representative index properties and shear wave velocity variations of overburden soil and rock encountered were developed and are summarized in the table below.

Summary of Representative Stratigraphy and Material Properties

Soil Unit	Depth Below Ground Surface (m)	γ (kN/m^3)	V_s (m/s)
Granular Fill	0 – 3.0	21.5	350
Sandy Silt to Silty Sand (Glacial Till)	3.0 – 24.5	21	400 – 559
Sand	24.5 – 25.0	19	500
Bedrock	> 25.0	26	760

Where required for analysis, the small-strain shear modulus (G_{max}) for the site soils encountered within the depth of investigation were estimated using the site-specific shear wave velocity (V_s) measurements obtained from the results of the SCPT testing. The values of G_{max} and V_s are related through the following expression:

$$G_{max} = \rho\,(V_s)^2 \text{, where } \rho = \text{material density.}$$

6.2.5.1 Shake Analysis Models

The one-dimensional soil columns and soil parameters described above were used for the ground response analyses. For all soil columns, the input motions established for the site were applied at the top of the bedrock as outcropping motions to account for the overburden effects. All ground response analyses were carried out using the software Shake2000 (Version 99.99.93, released June 2015, part of the Professional Suite of ground response software by GeoMotions, LLC).

The modulus reduction and damping verses shear strain curves used for the main soil strata are as follows:

- Granular Fill: Seed and Idriss (1970) lower and upper-bound curves for shear modulus and damping, respectively;
- Glacial Till: Seed and Idriss (1970) lower and upper-bound curves for shear modulus and damping, respectively;
- Sand: Seed and Idriss (1970) lower and upper-bound curves for shear modulus and damping, respectively; and,
- Bedrock: EPRI, 1993.

6.2.5.2 Analysis Results

As noted in the previous section, the ground response analyses were carried out to obtain the acceleration time histories at the ground surface for the 2,475-year ground motions. The spectral accelerations of the output time histories are shown on Figure E2. For comparison, the output time histories for the design earthquake event are plotted relative to the Site Class C design response spectrum.

The average ground response from all 12 output motions are shown on Figure E3 along with the Site Class C hazard spectrum. Figure E3 also shows the proposed design spectrum selected for this bridge project.

6.2.6 Liquefaction Assessment

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface settlements) and under undrained conditions generate excess pore pressures. The excess pore pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

Where the calculated shear stress is greater than the shear resistance, liquefaction of the soil with an associated significant strength loss is predicted to occur. This methodology considers that the soil behaves as a “sand-like” material and is applicable to assessment of liquefaction of cohesionless soils.

However, post-seismic strength loss may occur as a result of similar but different cyclic mechanisms. Cohesionless soil are also susceptible to cyclic mobility which, in contrast to liquefaction, can still occur when the static shear stress is less than the shear resistance of the soil. The deformations associated with cyclic mobility failure develop incrementally during the earthquake event. Further, soils that are predominantly fine-grained typically do not respond with liquefaction or cyclic mobility, but they can experience strength reduction as a result of prolonged shaking knows as cyclic softening.

The liquefaction potential at the site was initially assessed using the approach outlined in the CHBDC and by Idriss and Boulanger (2008) which is appropriate for granular deposits and soil that will behave as a “sand-like” material and involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil. The results of these liquefaction analyses indicated the potential for liquefaction at the site.

However, the behaviour of the glacial till at this site is not straightforward. It is considered to be Fort Covington till that is in the range of 12,500 to 25,000 years old (Prest and Hode-Keyser, 1977) and, given its age and geological origin, is not typically considered to behave as a potentially liquefiable, “sand-like” soil. Table C4.4 in the Commentary to the CHBDC suggests that glacial till deposits > 500 years old generally have a “very low” liquefaction potential.

Further interpretation of the results of the CPTs put down at the site suggest that the glacial till will exhibit a more “clay-like” behaviour:

- The normalized small-strain rigidity index values at all CPT locations are generally greater than about 1500, indicating that the soil exhibits some aging and has a significant micro-structure and inter-particle bonds;
- The measured porewater pressure response was typically very high indicating a fine-grained, clay-like soil response (rather than a drained, sand-like response);
- Relatively high average fines content (between about 40 and 55 percent, see Figures B4a and B4b in Appendix B) and low estimated permeability from dissipation tests suggest clay-like behaviour; and,
- The Soil Behaviour Type Index (Ic) was generally above the accepted boundary of clay-like behaviour (Ic = 2.6).

Based on the above, it is considered that the glacial till will not behave as a sand-like material and may be considered to be non-susceptible to flow liquefaction. However, the behaviour of the soil under cyclic loading is complex and it is recommended that a reduced post-seismic strength of 25% be considered in design to account for potential cyclic softening.

6.2.7 Seismic Load on Abutment Walls

Section C4.6.4 of the Commentary to the CHBDC provides guidance on appropriate methods to represent the backfill passive pressure force resisting movement at the abutments during seismic loading. For vertical bridge abutment walls up to about 1.7 m high, a spring stiffness based on the near-field conditions immediately behind the abutment wall is recommended (Caltrans, 2013). However, for abutment walls that are taller than 1.7 m, consideration of the influence of the far field conditions on the backfill stiffness is recommended (Carvajal, 2011). It is understood that the proposed abutment walls at the County Road 31 site will be about 5 m high (from underside of approach slab to the underside of the abutment “pile cap”).

At a nearby, similar highway overpass replacement site (Highway 401 underpass bridge at Post Road, Site 31-179), dynamic Soil Structure Interaction (SSI) analyses of the bridge structure, abutment backfill, and embankments was carried out to model the ground response behind the abutment walls. The results of the SSI analyses were compared to those calculated using the simplified analysis methodology proposed by Carvajal that considers the near field and far field response on the earth pressures due to shaking, as well as the results using the Caltrans methodology. At that site, it was recommended that the compliance spring to represent the passive resistance at the abutment wall be based on the total force vs. displacement computed using the Carvajal approach as it agreed well with the results of the SSI analyses.

Though a similar rigorous SSI analysis has not been carried out for the County Road 31 site, it is considered that a compliance spring to represent the passive resistance at the abutment wall based on the Carvajal approach would be appropriate given the similarities of the proposed structures, the level of shaking of the design earthquake, and subsurface conditions at each site.

The far-field response is dependent on the length of the approach embankment, which was assumed to be about 20 m long with a 13 m wide abutment. The abutment wall height was assumed to be 5.0 m high for the near-field response, and the overall embankment height assumed to be 6.4 m high for the far-field response.

The maximum passive pressure that can be mobilized with the lateral movements of the abutment was also estimated using the Richards & Elms (1979) approach based on the Mononobe-Okabe (M-O) formulation. The soil reaction at the abutment should be limited to this maximum passive force.

Therefore, it is recommended that the combined near and far-field response of the soils (i.e. the “Total Force” shown on Figure E4 in Appendix E) be considered to represent the soil passive reaction at the abutment, with a maximum limiting value defined by the maximum passive force (i.e. the “Limiting Passive Dynamic Soil Force” shown on Figure E4 in Appendix E). The soil spring should be applied at mid-height of the wall. Since the skew of the bridge at the abutments is relatively small (i.e. the abutment walls are generally perpendicular to the longitudinal axis of the bridge) the transverse component of the passive force is expected to be negligible.

6.3 Foundations - General

The existing County Road 31 underpass is a four-span structure with a reinforced concrete deck and non-integral abutments. The existing underpass bridge is understood to be in fair condition. Based on the 1961 design drawings (Drawings TWP #29-204-1-A to #29-204-13-A, inclusive), the existing foundations beneath the abutments are understood to consist of 324 mm diameter piles.

Each abutment foundation consists of 22 piles on each of two rows: the inner row battered at 1H:4V and the outer row vertical. The design load on each pile was about 268 kN (30 tons). Both abutment pile caps are perched within the existing embankments with the top of the pile cap at about Elevation 85.6 m at the north abutment and Elevation 85.0 m at the south abutment.

The existing pier foundations are understood to consist of spread footings that measure about 2.4 m by 17.7 m, which are likely founded on underlying glacial till.

6.3.1 Foundation Options

Based on the subsurface conditions, only deep foundation options have been considered for the replacement of the existing County Road 31 underpass, as shallow foundations would not provide sufficient bearing resistances or acceptable settlement performance for the structure. A summary of the advantages and disadvantages associated with each foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, constructability and relative costs is provided in Table 1 following the text of this report.

- **Driven steel H-piles:** Steel H-piles driven to refusal on the limestone bedrock could be feasible for support of the replacement bridge structure. This option would provide high geotechnical resistances and minimal post-construction settlements. In addition, this option would permit the use of conventional, semi integral or integral abutments. The use of driving shoes is recommended to minimize damage while penetrating the glacial till deposit (which may contain cobbles and boulders) and seating onto the limestone bedrock.
- **Driven steel pipe (tube) piles:** Closed-ended steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments and central pier. This foundation option would have similar advantages to steel H-piles in terms of high geotechnical resistances and minimal settlements. This option would permit the use of conventional, semi-integral and potentially integral abutments. Occasional cobble or boulder-sized particles were encountered at the borehole locations. Pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation if cobbles and/or boulders are encountered within the till deposit during driving.

- **Rock socketed steel pipe (tube) piles:** Socketed steel pipe piles installed using the down-the-hole hammer method could also be considered as a deep foundation option for support of the abutments. This foundation option would also have similar advantages to those above of high geotechnical resistances and minimal settlements. This option would permit the use of conventional, semi-integral and potentially integral abutments. This foundation type would also penetrate any cobbles or boulders encountered within the glacial till deposit during installation.
- **Rock Socketed Drilled Concrete Caissons:** Caissons deriving their support from bearing within the limestone bedrock are also feasible for this site. Caissons would require the use of temporary or permanent liners to mitigate the potential risks of ground loss from potential water-bearing cohesionless layers within the till soils during construction. In addition, the caissons would have to be socketed at least nominally into the bedrock to permit cleaning of the caisson bases, and such sockets would have to be advanced by rock coring and/or chisel drilling into the medium strong to strong limestone bedrock. This foundation option is considered feasible at both the abutments and pier. However given the larger diameter of caissons (when compared to H-piles or pipe piles), removal of about 2 to 4 of the existing abutment piles will likely be required, since the western portion of the new abutment is to be located at approximately the same location as the existing abutment. This is of lesser concern at the location of the central pier, where the existing foundations are supported on spread footings. Rock socketed caissons would also have the required stiffness to resist the additional seismic lateral loads as well as provide 'fixity' at the interface with the bedrock surface during a seismic event.

Based on the above considerations, the preferred options from a geotechnical/foundations perspective is to support the abutments and centre pier on driven steel H-piles or pipe piles for the bridge replacement.

6.3.2 Feasibility of Integral and Semi-Integral Abutments

As outlined in MTO's report SO-96-01, integral abutment bridges are single span or multiple span continuous deck type bridges with a movement system composed primarily of abutments on flexible integral foundations and approach slabs, in lieu of movable deck expansion joints and bearings at abutments. The feasibility of integral abutments is influenced by a number of factors including geometry and subsurface conditions. The primary criterion is the need to support the abutments on relatively flexible piles. Where the load bearing stratum is near the surface or where the use of short piles or caissons (less than 5 m in length) is planned, the site is not considered suitable for integral abutment bridges. Geometric constraints on the use of integral abutments are also applicable and include: overall bridge length less than 150 m; skew angle less than 35º; and abutment wall heights less than 6 m without a retained soil system.

An integral abutment arrangement using a deep foundation option described above is considered feasible at this site since the flexible pile-supported abutment foundations would meet MTO's foundation criteria for integral abutments.

6.3.3 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the proposed underpass structure and foundation system may be classified as having medium traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a "typical" consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in Sections 6.4 to 6.6 below.

6.4 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

6.4.1 Founding Elevations

The foundations for the replacement bridge may be supported on steel H-piles or closed-ended pipe (tube) piles driven to found on the limestone or dolostone bedrock. Based on the geotechnical investigations carried out at the site, the bedrock surface is considered to be relatively flat and was encountered between about Elevation 55.9 m and 56.8 m at the borehole locations. Based on the borehole results, and assuming about 0.1 m of penetration into the bedrock to allow for some weathering in the upper portion of the rock, the following pile tip elevations are recommended for design of steel H-piles or pipe piles:

Foundation Element	Borehole Numbers	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)
North Abutment	15-5	55.9	55.8
Central Pier	15-4	56.8	56.7
South Abutment	15-3	56.5	56.4

The pile caps should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).*

At the abutment and pier locations, the bedrock surface was encountered at elevations ranging from about 55.9 to 56.8 m. Based on the results of the investigations, steel H-piles driven to the bedrock surface would be greater than 5 m in length and are therefore considered to be feasible for use in an integral abutment configuration. However, the upper portion of the piles should be cased in a loose sand filled, corrugated steel pipe (or similar) to provide suitable flexibility of steel H-piles.

Cobbles or boulders were encountered within the glacial till deposit at the borehole locations. A layer of cobbles and boulders estimated to be about 0.3 m thick was encountered within the upper 2.0 m of the glacial till deposit at borehole 15-2. Cobbles and boulders were encountered or inferred within the glacial till based on recovered samples or rig response during drilling at all borehole locations. For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the glacial till. In this regard, steel H-piles are preferred over closed-ended steel pipe piles as pipe piles are considered to pose a higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. Each pile should be reinforced at the tip with a driving shoe to improve seating of the piles on the bedrock and to reduce the potential for damage to the piles during driving through soils that may contain boulders, in accordance with OPSS 903 *(Deep Foundations).* If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II *(Steel Tube Pile Driving Shoe).*

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A Non-Standard Special Provision for vibration monitoring should be included in the contract documents and has been included in Appendix F of this report. A maximum peak particle velocity of 100 mm/s is recommended at the existing abutments. The piles further from the existing structure should be driven first, in order to check the vibration level at the existing structures and, if necessary, alter the pile driving criteria for the remaining piles.

The piles for the widened abutments may need to be driven in close proximity to the battered piles supporting the existing abutments. Depending on the preferred location of the abutment foundations for the west side of the structure, the piles may be driven behind or in front of the existing pile caps and piled foundations. Consideration may also be given to driving the new abutment piles adjacent to (or in between) the existing piles following removal of the existing pile cap and exposure of the existing piles or to extracting the existing piles that are in conflict with the new foundations.

6.4.2 Axial Geotechnical Resistance

For design of HP 310x110 piles driven to the estimated tip elevations provided in Section 6.5.1, the factored axial geotechnical resistance at Ultimate Limit States (ULS) may be taken as 2,000 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the limestone or dolomite bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. Similar axial geotechnical resistances may be used in the design of HP 310x132 piles, or closed-end, concrete-filled, 324 mm diameter steel pipe piles having a minimum wall thickness of 9.5 mm.

Pile installation should be in accordance with OPSS 903 *(Deep Foundations).* The drawings should incorporate the appropriate note stating that the piles should be equipped with driving shoes and should be driven to bedrock. For piles driven to refusal on bedrock, and as described in OPSS 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

6.4.3 Downdrag Load (Negative Skin Friction)

No significant raise in the grade of Highway 401 is expected and, therefore, downdrag loads are not anticipated on pile foundations at the new pier. At the abutments, negligible settlement is expected of the underlying glacial till deposit due to the placement additional/new embankment fill. Provided that the any organic matter encountered within the footprint of the widened embankments is removed prior to placement of any fill, downdrag loads are not anticipated on piled foundations at the abutments.

6.4.4 Lateral Geotechnical Resistance

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.11.2.2.1 of the *Commentary to the CHBDC.*

The ULS lateral resistance of a pile group may be estimated as the sum of the individual pile resistances across the face of the pile group, perpendicular to the direction of the applied lateral force.

For an HP 310x110 or an HP 310x132 pile, the factored ULS lateral resistance may be taken as 300 kN. The ULS resistances obtained using the above parameter represents a factored value, considering a resistance factor of 0.5 in accordance with the *CHBDC*. This value provides a limit on the lateral geotechnical resistance offered using the p-y curves below in the upper 6 pile diameters below the pile cap.

6.4.5 Lateral Soil-Structure Interaction Springs

The foundation lateral soil-structure interaction springs required for the dynamic analysis of the bridge pier and abutments were computed based on the available subsurface information on the soil layers surrounding the foundations and the pile dimensions.

The soil-structure interaction between the bridge foundations and the surrounding soils was modeled using the load transfer method. The lateral load-displacement behaviour of the piles can be modeled using p-y curves (CFEM, 2006). P-y curves relate the lateral deflection of a single pile to the corresponding soil and bedrock reactions at any depth below ground surface. The P-y curves were generated internally using the commercially available software programs LPILE Plus (Version 5.0.29), produced by ENSOFT Inc.

For all loading conditions, a pinned connection was assumed between the pile head and the pile cap. Both static and cyclical loading conditions were considered in the lateral analyses. The strength of the glacial till was reduced by 25% to account for cyclic softening under the cyclical loading condition. At the abutments, the strength parameters of the soil in the upper 3 m were modified to represent loose sand within the Corrugated Steel Pipes (CSPs) to be installed with each pile as part of the integral abutment configuration.

The families of static and cyclic P-y curves calculated at 0.5 to 1.0 m increments of depth for a single, vertical 310 x 110 H-Pile at the abutments and pier are attached in tabular format and graphically in Appendix F. The values shown on Figure F1 and F3 reflect the lateral resistance under static conditions (at abutments and pier, respectively) and Figures F2 and F4 reflect the reduced strength cyclic loading condition (at abutments and pier, respectively). The P-y curves presented in Appendix F may also be used for design of a single, vertical 310 x 132 H-Pile.

For piles arranged in closely spaced groups, the pile-soil-pile interaction causes the individual piles in a group to be less effective than a single pile. Theses “group effects” can be incorporated into the design using a method that modifies the single pile p-y curves by some factor (i.e. a p-reduction factor). Generalized p-multipliers (i.e. p-reduction factors) for a range of pile spacings are provided in Section C6.11.3.4 of CHBDC.

6.5 Socketed Steel Pipe Pile Foundations

6.5.1 Founding Elevations

Alternatively, the abutments and central pier for the replacement bridge may be supported on steel pipe piles installed to found on the bedrock then socketed into the limestone bedrock using the down-the-hole hammer method. Based on the geotechnical investigations carried out at the site, the bedrock surface is considered to be relatively flat and was encountered between about Elevation 55.9 and 56.8 m at the borehole locations. It is recommended that the steel pipe piles be socketed 2.0 m into the bedrock for axial resistance considerations. Therefore the following socket founding elevations are recommended for design.

Foundation Element	Borehole Numbers	Bedrock Surface Elevation (m)	Design Socket Founding Elevation (m)
North Abutment	15-5	55.9	53.9
Central Pier	15-4	56.8	54.8
South Abutment	15-3	56.5	54.5

The pile caps should be constructed at a minimum depth of 1.7 m for frost protection purposes, per Ontario Provincial Standard Drawing (OPSD) 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).*

Occasional cobbles and boulders were encountered within the glacial till deposit at the borehole locations, but should not be problematic where a down the hole hammer is used.

Vibration monitoring should be carried out during pile installation as described in Section 6.4.1.

Consideration should be given to the proximity of the new piles to the battered piles supporting the existing abutments when selecting the pile locations and driving order, as described in Section 6.4.1.

6.5.2 Axial Geotechnical Resistance

The following foundation design recommendations have been based on the side-wall (shaft) resistance of the rock socket and a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,100 kPa. For a 350 mm diameter concrete-filled pipe pile socketed 2.0 m into the limestone bedrock this would equate to a factored geotechnical resistance at ULS of 2,400 kN (geotechnical resistance factor of 0.5). We understand that pipe piles on the order of 600 mm to 750 mm in diameter are being considered. For socket depths and diameters that differ from above, the ULS resistance can be pro-rated based on the resulting socket side-wall surface area for sockets up to 5 m deep and diameters up to 750 mm. The ULS resistance considers the RQD values recorded for the bedrock as well as the compressive strength data for the rock core. This value is applicable provided that the socket is within competent bedrock and that the side wall of the socket is cleaned of any smeared material.
In addition, a smaller diameter pipe pile should be placed at the bottom of the rock socket and extend at least 2 m above the bedrock surface to provide additional lateral support and full fixity given the liquefiable soils at this site.

Pile installation should be in accordance with OPSS.PROV 903 *(Deep Foundations).*

6.5.3 Downdrag Load (Negative Skin Friction)

No downdrag loads are expected to be imposed on the piles, as described in Section 6.4.3.

6.5.4 Lateral Geotechnical Resistance

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.11.2.2.1 of the *Commentary* to the CHBDC.

The ULS lateral resistance of a pile group may be estimated as the sum of the individual pile resistances across the face of the pile group, perpendicular to the direction of the applied lateral force.

As outlined in Section C6.11.2.2.1 of the *Commentary* to the CHBDC, the SLS lateral resistance of the piles can be determined by either empirical equations based on theory of elasticity or by using the P-y method, and should be considered if socketed pipe pile foundations are selected for design.

6.6 Caisson Foundations

6.6.1 Founding Elevations

Alternatively, support of the abutments or central pier may be provided by caisson foundations. Due to the relatively high water table and the potential difficulty in socketing a liner into the strong bedrock, it may not be feasible to dewater and clean the base of the caisson and, as such, full end-bearing support may not be developed. The axial geotechnical resistance for rock-socketed caissons should be based on the side-wall (shaft) resistance of the rock socket rather than end-bearing. For design purposes, it is recommended that the caissons be founded at the following elevations (i.e., a rock socket of approximately 2 m).

Foundation Element	Borehole Numbers	Bedrock Surface Elevation (m)	Design Socket Founding Elevation (m)
North Abutment	15-5	55.9	53.9
Central Pier	15-4	56.8	54.8
South Abutment	15-3	56.5	54.5

The use of a temporary or permanent liner or casing will be required to advance the caissons through the potential water-bearing cohesionless layers within the glacial till deposit while minimizing loss of ground. The casing should be extended so that it is “seated” a minimum of 300 mm into the bedrock.

If caisson caps are to be included as part of the design, they should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).*

Similar to pile installation, vibration monitoring should be carried out during caisson installation to ensure that the vibration levels at the existing structure are maintained below tolerable level, as described in Section 6.4.1.

For this deep foundation option, consideration must also be given to removal of the existing abutment piles, as the western portion of the new abutment is to be located at approximately the same location as the existing abutment; while piles may be able to be located so as to avoid conflict with the existing piles, larger diameter caissons would likely necessitate removal of the 2 to 4 existing piles which may be in conflict with the new foundations.

6.6.2 Axial Geotechnical Resistance

The *factored* geotechnical side wall (shaft) resistance at ULS can be taken as 1,100 kPa provided that the caisson socket is formed within competent bedrock. This value assumes that the side wall of the socket will be cleaned of any smeared material. To provide full fixity, the caissons should be provided with a minimum socket length equal to 2 times the caisson diameter. The structural engineer should check that the shear strength of the concrete is adequate to support these loads.

For a 1.0 m diameter caisson socketed 2 m into the competent bedrock, this would equate to a factored axial geotechnical resistance at ULS of about 7,000 kN (geotechnical resistance factor of 0.5). For socket depths and diameters that differ from above the ULS resistance can be pro-rated based on the resulting socket side-wall circumference for sockets up to 5 m deep and diameter up to 1.2 m. SLS resistances do not apply to caissons founded within the limestone bedrock, because the SLS resistance for 25 mm of settlement will be greater than the factored axial geotechnical resistance at ULS.

6.6.3 Downdrag Load (Negative Skin Friction)

No downdrag loads are expected to be imposed on the piles, as described in Section 6.4.3.

6.6.4 Lateral Geotechnical Resistance

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.11.2.2.1 of the *Commentary* to the CHBDC.

As outlined in Section C6.11.2.2.1 of the *Commentary* to the CHBDC, the SLS lateral resistance of the caissons can be determined by either empirical equations based on theory of elasticity or by using the P-y method, and should be considered if caisson foundations are selected for design.

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls and any associated wing walls (if required) will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls:

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 *(Aggregates)* Granular A or Granular B Type II, should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 *(Compacting)*. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 *(Walls, Abutment, Backfill, Minimum Granular Requirement),* OPSD 3121.150 *(Walls, Retaining, Backfill, Minimum Granular Requirement)*, and 3190.100 *(Walls, Retaining and Abutment, Wall Drain).*
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.7 m behind the back of the wall (Case (a) on Figure C6.20 of the *Commentary* to the CHBDC). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or pile cap (Case (b) on Figure C6.20 of the *Commentary* to the CHBDC).

6.7.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- For Case (a), the pressures are based on the proposed embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM):

Material	Earth Fill or SSM
Soil Unit Weight:	20 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_P	 0.33 0.50 3.0

- For Case (b), the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II
Soil Unit Weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_P	 0.27 0.43 3.7	 0.27 0.43 3.7

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
  - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
  - Horizontal translation of 0.001 times the height of the wall; or,
  - A combination of both.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.7.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the wall. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given in Section 6.7.1, above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA.
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case (a) and Case (b)) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

	Design Earthquake	Site PGA	Granular A	Granular B Type II	SSM
Yielding Wall	**2,475 Yr**	0.36	0.38	0.38	0.46
Non-Yielding Wall	**2,475 Yr**	0.36	0.54	0.54	0.64

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H\text{-}d), \text{ yielding walls}$$

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H\text{-}d), \text{ non-yielding walls}$$

Where:	$\sigma_h(d)$	is the (static plus seismic) lateral earth pressure at depth, d, (kPa);
	K_a	is the static active earth pressure coefficient;
	K_o	is the static at-rest earth pressure coefficient;
	K_{AE}	is the seismic active earth pressure coefficient;
	γ	is the unit weight of the backfill soil (kN/m^3), as given previously;
	d	is the depth below the top of the wall (m); and,
	H	is the total height of the wall (m).

6.8 Approach Embankments

It is understood that the proposed pavement grades at the new structure will up to about 1.8 m higher than the existing County Road 31 grades particularly at the south end of the structure. In addition, the alignment of County Road 31 will also be shifted to the east, therefore the existing embankments will require a maximum widening of about 7.9 m to accommodate this shift.

In general, the surficial soils at the location of the existing County Road 31 bridge embankments consist of pavement structure underlain by embankment fill, where the surficial soils along the proposed alignment of the new approach embankments consist of topsoil, organic clayey silt and peat, underlain by thin layers silty sand and silty clay. Similar layers were also encountered beneath the embankment fill, but of limited thickness.
The embankment fill and surficial deposits are underlain by glacial till, overlying limestone and dolostone bedrock.

6.8.1 General Embankment Construction

The topsoil, alluvium and peat are compressible soils that are expected to experience settlement under increased load. Therefore it is recommended that all the topsoil, alluvium and peat present within the footprint of the embankment widening be stripped prior to placement of the new embankment fill. The topsoil, alluvium and peat material should be stripped to the underlying silty sand or glacial till.

The new embankment fill associated with the grade raise and widening for the bridge replacement should be placed and compacted in accordance with OPSS.PROV 206 *(Earth Excavation and Grading)* and OPSS.PROV 501 *(Compacting).* Benching of the existing County Road 31 embankment side slopes should be carried out to “key in” the new fill materials in areas where the embankment is widened, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil *(OPSS 802 – Topsoil)* and seeding *(OPSS.PROV 804 – Seed and Cover)* or pegged sod *(OPSS 803 – Sodding)* is recommended as soon as practicable after construction of the embankments.

6.8.2 Global Stability

Static and seismic slope stability analyses of the embankments with the proposed grade raise and widening were carried out with the commercially available slope stability analysis software, SlopeW (part of the software package, Geo-Studio 2007 Version 7, produced by Geo-Slope International Ltd.), to verify that a minimum factor of safety of 1.3 is achieved under static conditions and 1.1 under seismic conditions. These minimum factors of safety are considered appropriate for the proposed bridge approach embankments, considering the design requirements and the available field and laboratory testing data.

A Morgenstern-Price method was used to determine the factor of safety. The analyses were based on the existing topographic information provided by the design team and the available subsurface information.

The soil and bedrock stratigraphy between the borehole locations is based on our interpretation of the geological conditions of the area and consequently the actual conditions may vary from that used in our model. The soil parameters used in the analyses were based on in situ and laboratory testing data as well as published correlations and are given in the table below.

Provided that the approach embankment side slopes are maintained no steeper than 2H:1V, and the existing embankment side slopes are benched in accordance with OPSD 208.010 (Benching of Earth Slopes), to “key in” any new fill materials placed on the slopes to accommodate the overall grade, the embankments should have an adequate minimum factor of safety of at least 1.3 under static conditions.

The stability of the embankments was also evaluated under seismic loading conditions. The minimum factor of safety value that is typically required against instability during a seismic event is 1.1. A horizontal seismic coefficient of 0.18 (i.e., 50% of the PGA) was used for the pseudo-static analyses. Furthermore, the static soil shear strengths were increased by 10% under seismic loading conditions to account for strain-rate effects. Under these loading conditions the factor of safety of the proposed 8 m high embankments was in excess of 1.5.

Soil Stratum	Bulk Unit Weight (kN/m^3)	Static[1] Effective Friction Angle (ϕ)
Existing Embankment Fill	21.5	35°
New Embankment Fill	21.5	31°
Organic Deposits (Topsoil, Alluvium and Peat)	17	28°
Dense Glacial Till (Upper)	23	33°
Very Loose to Loose Glacial Till (Lower)	23	33°[2]
Bedrock	-	-

[1] For pseudo-static analyses tan ϕ was increased by 10 percent for stain rate effects.

[2] For the post-earthquake analyses a friction angle of 23° was used to account for a 25% strength reduction as a result of cyclic softening (see Section 6.2.6).

The ‘post-earthquake’ stability of the embankments was also evaluated under static loading conditions, using the static shear strengths of the non-liquefied deposits, and using softened residual shear strength to consider cyclic softening in the glacial till. A minimum factor of safety of 1.3 is considered acceptable against static undrained deep-seated embankment instability under these conditions. The minimum factor of safety of 1.3 reflects greater gravity load certainty of the post-earthquake condition and general practice as we understand it. MTO may wish to review this criteria given the transient nature of the softened strengths. Based on the nature of the deposits a post-cyclic friction angle of 23 degrees was assigned to the glacial till to account for a 25% strength reduction (see Section 6.2.6). Under these conditions a factor of safety of greater than 1.3 was obtained.

If side slopes steeper than 2H:1V are to be considered or the grade raise is to be increased more than 1.8 m above the existing grades, the embankment side-slope stability will have to be reassessed.

6.8.3 Static Settlement

Settlement of the existing embankments has likely occurred over time since the original bridge construction. Negligible (i.e., less than 25 mm) additional settlement is expected of the underlying glacial till deposit due to the 1.8 m grade raise and 8 m embankment widening, provided any organic matter encountered within the footprint of the widened embankments is removed prior to placement of any fill. The settlement of the native soils will be elastic in nature and should therefore occur during construction.

Additional settlement of the embankments will occur as a result of compression of the new and existing embankment fill. The magnitude of compression of the new fill may range from 0.5 to 1 percent of its thickness, assuming approximately 95 percent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. Some nominal compression of the existing fill (less than 0.5 percent of its thickness) is expected to occur under the increased loading. Provided that granular fill is used to raise the grade, settlement of the new fill is expected to occur essentially during embankment construction. Similarly, settlement of the existing embankment fill will be elastic in nature and should occur essentially immediately following placement of the new fill.

Seismic settlements as discussed in Section 6.2.6 would be in addition to these values.

6.9 Construction Considerations

The following sections identify construction issues that should be considered at this stage as they may impact the detail design of the project and provisions in the Contract Documents.

6.9.1 Excavation and Temporary Protection Systems

The excavations for pile caps would extend a minimum of 1.7 m deep (for frost protection purposes) into the existing grade fill at the central pier location. At the abutments, the excavations for pile caps could be maintained at a higher elevation, within the approach embankments. Excavations for the removal of the surficial organic deposits beneath the new approach embankments would extend up to about 2.4 m in depth.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill organic material above the water table would be classified as Type 3 soils according to OHSA, therefore excavations in these materials should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). however, the organic deposits below the water table would be classified as Type 4 soil, based on OSHA, and excavations in these materials should be sloped no steeper than 3H:1V; however, with appropriate groundwater control, it is anticipated that temporary excavation slopes through these materials can also be formed at 1H:1V.

If the above open-cut excavation side slopes cannot be accommodated, then a temporary protection system (i.e., temporary excavation shoring) will be required. Where shoring is required, the protection system should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539.

The selection and design of the protection system will be the responsibility of the Contractor. However, the following comments are provided to aid in the costing and assessment of temporary protection system options for this site:

- It is considered that a soldier pile and lagging system would be feasible at this site.
- The soldier piles would have to be socketed to sufficient depth to provide the necessary passive resistance for the retained soil height of up to a few metres. The soldier pile and lagging or sheetpiling would be supported against lateral movement using walers, tie backs and/or internal struts/braces.

6.9.2 Groundwater Control

Based on the groundwater level encountered in the standpipe piezometers installed within Boreholes 15-2 and 15-4 (at the toe of the south approach embankment and the centre pier, respectively) the groundwater level is expected to be within about 0.5 m of the existing Highway 401 grade at the central pier location and the original ground surface at the toe of the existing embankments.

The excavation required for construction of the pile cap at the central pier is anticipated to extend up to about 1.2 m below the groundwater level for frost protection requirements. The excavations for the removal of the organic deposits below the footprint of the new embankments is anticipated to extend up to about 1.9 m below the groundwater level. Some groundwater or surface water inflow into the excavations should be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps in the bottom of the excavations provided suitably sized pumps are used. Additional groundwater flow is expected during periods of sustained wet weather and the groundwater control methods used may need to be adjusted accordingly during and immediately following such events. Dewatering will be required to lower the groundwater level to approximately 0.5 m below the pile cap founding level pile cap formation. The water-bearing till at this site is relatively fine-grained (silty) and therefore will have a lower permeability. This relatively small drawdown is not expected to have an adverse impact on the existing or new structure foundations at this site.

According to Ontario Regulation 63/16 and Ontario Regulation 387/04, a Permit to Take Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water greater than 400,000 L/day is pumped from the excavations. If the volume of water to be pumped will be less than 400,000 L/day, but more than 50,000 L/day, the water taking will not require a PTTW, but will need to be registered in the Environmental Activity and Sector Registry (EASR) as a prescribed activity. It is estimated that less than 400,000 L/day but possibly more than 50,000 L/day of water may require handling during excavation for the central pier and removal of the organic materials. Therefore, registration of EASR may be required for construction.

Surface water should be directed away from the excavation areas, to prevent ponding of water that could result in disturbance and weakening of the subgrade.

6.9.3 Vibration Monitoring During Foundation Installation

Vibration monitoring of the existing structure is recommended during pile/caisson installation to assist in maintaining vibration levels within tolerable ranges for the existing portions of the bridge in close proximity to County Road 31. A NSSP has been provided in Appendix G to address this requirement.

A maximum peak particle velocity of 100 mm/s is recommended at the existing structure foundations. The piles/caissons furthest from the existing structure should be installed first, in order to check the vibration level at the existing structure and, if necessary, alter the installation procedures for the remaining piles/caissons.

6.9.4 Ground/Groundwater Control and Obstructions for Deep Foundation Installation

Where caissons are adopted, the use of temporary or permanent liners will be required to minimize loss of ground through the water-bearing cohesionless till deposit.

The presence of cobbles and boulders in the glacial till could affect the installation of deep foundations or protection system elements. If caissons are to be used, appropriate drilling techniques will be required to advance the caissons through the glacial till.

An NSSP is provided in Appendix G, for inclusion in the Contract Documents to alert the Contractor to these conditions.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Susan Trickey, P.Eng., and reviewed by Mr. Michael Snow, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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Table 1 – Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles driven to bedrock	▪ Feasible at piers and abutments ▪ Preferred from a foundations perspective	▪ Abutment pile caps could be within embankments, reducing depth of excavation and temporary excavation support requirements ▪ Higher geotechnical resistances and negligible settlement ▪ Less potential for interference with existing piles (vs. pipe piles) ▪ Preferred foundation option for integral abutment construction	▪ Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in piles "hanging up" in the glacial till deposit and lower geotechnical resistances ▪ Temporary protection systems may be required at the central pier ▪ Some groundwater control would be required at the central pier	▪ Moderate cost	▪ Potential risk of driven H-piles "hanging up" in glacial till

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Closed-ended steel pipe (tube) piles, driven to found in bedrock	- Feasible but not preferred	- Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary protection system - Higher geotechnical resistances and negligible settlement - Allows for semi-integral and potentially integral abutment configuration - Higher pile stiffness compared to H-piles to resist seismic lateral loads	- Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving resulting in more piles “hanging up”, lower geotechnical resistances, and greater potential for interference with existing piles - Temporary protection systems may be required at the central pier - Some groundwater control would be required at the central pier	- Moderate cost	- Higher risk of pipe piles “hanging up” in glacial till

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Socketed steel pipe piles installed in the bedrock	▪ Feasible for support of bridge replacement	▪ Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary protection system ▪ High geotechnical resistances and negligible settlement ▪ Allows for semi-integral abutment configuration ▪ Rock socket would provide toe fixity of the pile and further resistance to seismic lateral loads	▪ Requires specialized equipment for penetrating through cobbles and boulders and for forming the rock socket ▪ Temporary protection systems may be required at the central pier ▪ Some groundwater control would be required at the central pier	▪ High	

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Rock Socketed Caissons	▪ Feasible for the bridge replacement	▪ High geotechnical resistances and negligible settlement ▪ Allows for semi-integral abutment configuration ▪ Rock socket would provide toe fixity of the pile and further resistance to seismic lateral loads ▪ Could eliminate the need for pile caps at the central pier, and allow for structural continuity between caissons and piers ▪ Construction from existing grade would reduce excavation and groundwater control requirements at center pier	▪ Temporary or permanent liners required to control ground and groundwater in the glacial till deposit ▪ Rock coring, churn drilling or chisel drilling required to form rock sockets in medium strong to strong bedrock ▪ Conflict with existing abutment piles likely, requiring removal of existing piles	▪ High	▪ Some risk of difficulty in removing existing abutment piles to avoid conflict with new caissons

MINISTRY OF TRANSPORTATION, ONTARIO

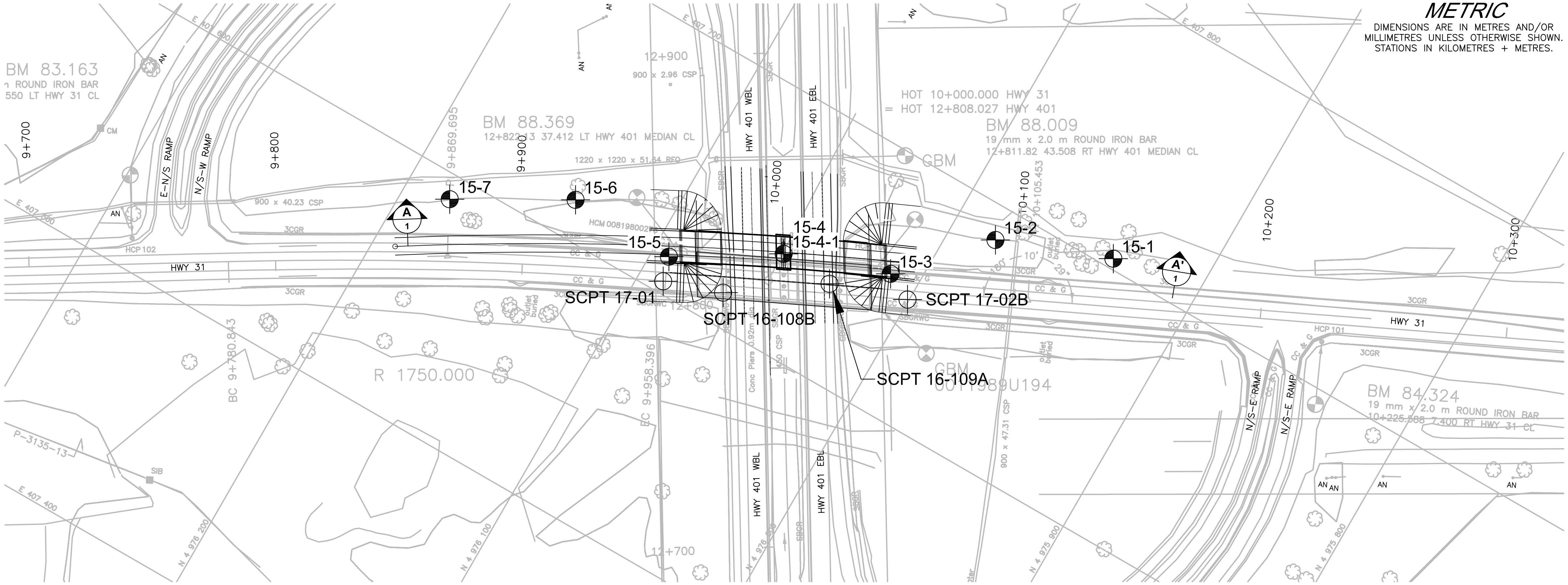
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 4415-01-01

SHEET 61

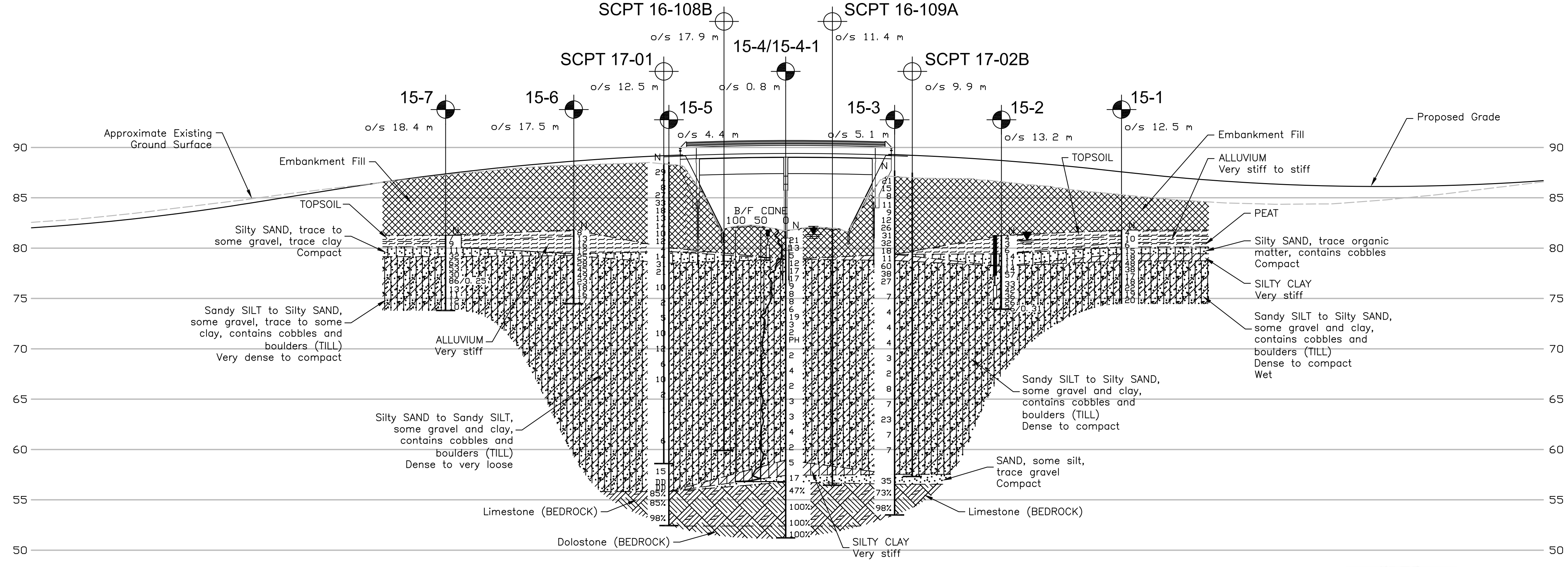
HIGHWAY 401 – BRIDGE REPLACEMENT COUNTY ROAD 31 INTERCHANGE BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER



PLAN

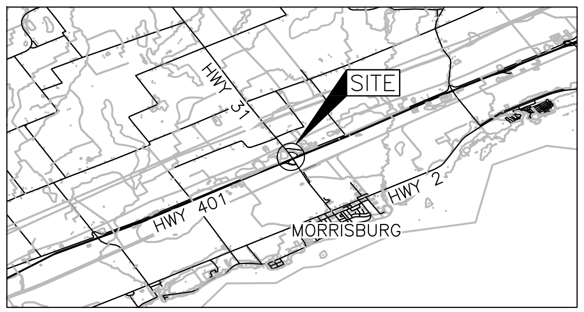
SCALE
20 0 20 40 m



A-A'

PROFILE ALONG HWY 31

HORIZ. SCALE
20 0 20 40 m
5 0 5 10 m
VERT. SCALE



KEY PLAN

SCALE 2 0 2 4 KM

LEGEND

⊕	Borehole – Current Investigation
⊕	Cone Penetration Test – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
DD	Rotary Diamond Drill Coring
100%	Total Core Recovery (REC)
▼	WL in piezometer
▮	Seal
▯	Piezometer

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
15–1	81.8	4975936.6	407704.1
15–2	81.3	4975981.8	407686.7
15–3	87.7	4976011.7	407653.6
15–4	81.8	4976053.3	407638.9
15–4–1	81.8	4976053.3	407638.9
15–5	88.6	4976093.0	407614.7
15–6	81.8	4976137.3	407615.7
15–7	81.3	4976181.6	407590.4
16–108B	81.7	4976066.7	407613.0
16–109A	82.0	4976031.2	407637.7
17–01	88.3	4976090.1	407604.9
17–02B	87.5	4976000.6	407648.0

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by WSP, drawing file no. 3412039–7B–301–001_General_Arrangement.dwg, received August 7, 2018.

M. J. KENNEDY
Nov 14, 2018

F. J. HEFFERNAN
NOV. 14, 2018

NO.	DATE	BY	REVISION

Geocres No. 31B–92

HWY. 401		PROJECT NO. 12–1121–0099–1750	DIST. Eastern
SUBM'D. SAT	CHKD. SAT	DATE: 11/14/2018	SITE: 31X–0204–B0
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 1

PLOT DATE: November 15, 2018
FILENAME: N:\Active\2012\1121 – Geotechnical\12-1121-0099 MRC 22 Structures Eastern Region\Spatial IM\CAD\Phase 1750\1211210099-1750-02.dwg

APPENDIX A

Borehole Records

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

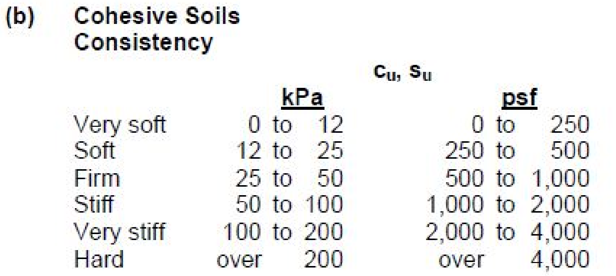
Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Compactness Condition	**N Blows/300 mm or Blows/ft**
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50



(b) Cohesive Soils

Consistency	**c_u, s_u kPa**	**psf**
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	**Modifier**	**Example**
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

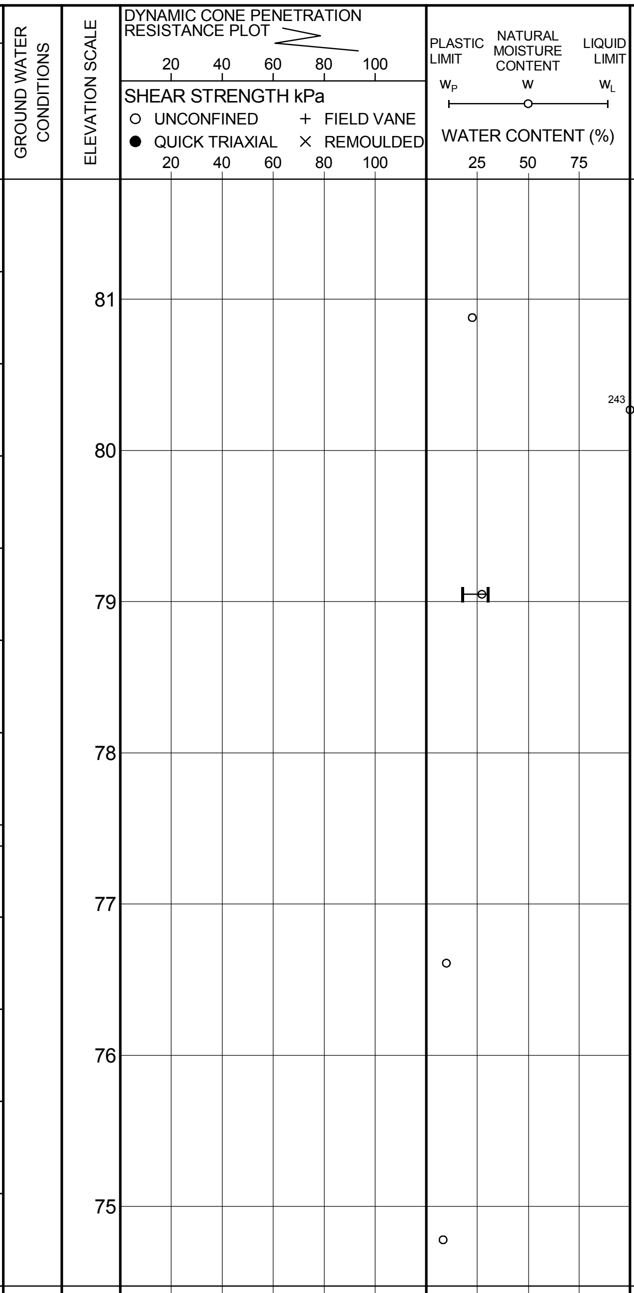
RECORD OF BOREHOLE No 15-1

SHEET 1 OF 1

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
LOCATION N 4975936.6; E 407704.1 MTM NAD ZONE 9 (LAT. 44.915994; LONG. -75.196662)
ORIGINATED BY HEC
DIST Eastern HWY 31
BOREHOLE TYPE Portable Drill, AW/BW Casing
COMPILED BY JM
DATUM Geodetic
DATE December 14, 2015
CHECKED BY SAT

SOIL PROFILE		SAMPLES			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
81.8	GROUND SURFACE								
0.0	TOPSOIL Dark brown Moist	1	SS	4					
81.2 0.6	Organic clayey silt, some sand, trace gravel, contains roots (ALLUVIUM) Very stiff Dark brown Moist to wet	2	SS	10					
80.6 1.2	PEAT Loose Dark brown to black Wet	3	SS	6	ORG = 56.1				
80.1 1.7	Silty SAND, trace organic matter, contains cobbles Compact Grey to grey-brown Wet	4	SS	15					
79.4 2.4	SILTY CLAY Very stiff Grey Wet	5	SS	18					
78.8 3.1	Sandy SILT, some gravel and clay, contains cobbles and boulders (TILL) Dense Grey-brown Wet	6	SS	48					
		7	SS	38					
77.4 4.4	Silty SAND, some gravel and clay, contains cobbles and boulders (TILL) Compact Wet	8	SS	17					
		9	SS	18		10	41	38	11
		10	SS	22					
		11	SS	19					
		12	SS	20		21	35	34	10
74.5 7.3	END OF BOREHOLE								



+³, ×³: Numbers refer to Sensitivity   ○³% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IM\GINT\PHASE 1750\1211210099-1750.GPJ GAL-GTA.GDT 11/15/18 JM

RECORD OF BOREHOLE No 15-2

SHEET 1 OF 1

METRIC

PROJECT 12-1121-0099-1750

G.W.P. 4415-01-01

LOCATION N 4975981.8; E 407686.7 MTM NAD ZONE 9 (LAT. 44.916403; LONG. -75.196872)

ORIGINATED BY HEC

DIST Eastern HWY 31

BOREHOLE TYPE Portable Drill, AW/BW Casing

COMPILED BY JM

DATUM Geodetic

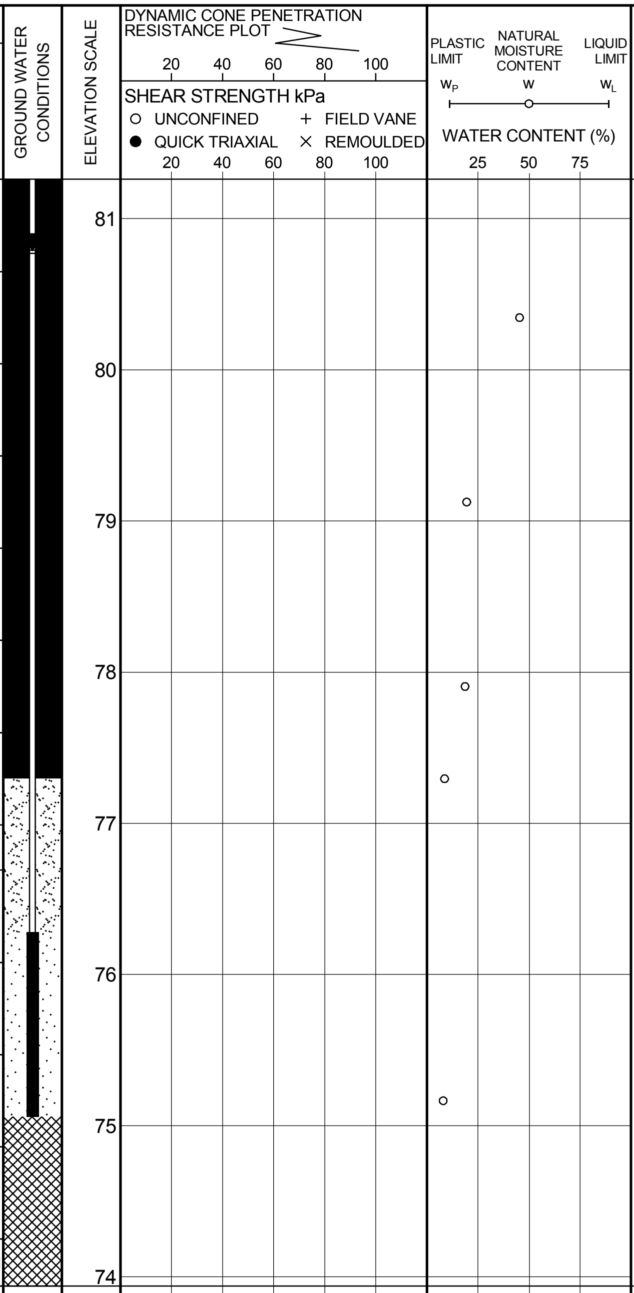
DATE December 15, 2015

CHECKED BY SAT

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
81.3	GROUND SURFACE								
0.0	TOPSOIL Grey Moist	1	SS	4					
80.7 0.6	Organic clayey silt, trace sand and gravel, contains roots (ALLUVIUM) Stiff to very stiff Dark brown Moist to wet	2	SS	3					
		3	SS	6					
79.5 1.8	Silty SAND, trace gravel and clay Compact Grey Wet	4	SS	14		8	45	42	5
		5	SS	11					
78.3 3.1	Sandy SILT, some gravel and clay (TILL) Compact to very dense Grey Wet	6	SS	14		27	23	26	24
		7	SS	57		12	37	38	13
77.0 4.3	COBBLES and BOULDERS								
76.7 4.6	Silty SAND, some gravel and clay, contains cobbles and boulders (TILL) Dense to compact Grey Wet	8	SS	33					
		9	SS	42					
		10	SS	36		19	37	33	11
		11	SS	26					
		12	SS	26/0.31					
74.0 7.3	END OF BOREHOLE								

NOTES:

1. Water level in well screen at a depth of 0.5 m below ground surface (Elev. 80.8 m), measured on April 1, 2016.



+ ³, × ³: Numbers refer to Sensitivity

○ ³% STRAIN AT FAILURE

PROJECT 12-1121-0099-1750	**RECORD OF BOREHOLE No 15-3**	SHEET 1 OF 5	**METRIC**
G.W.P. 4415-01-01	LOCATION N 4976011.7; E 407653.6 MTM NAD ZONE 9 (LAT. 44.916677; LONG. -75.197286)		ORIGINATED BY HEC
DIST Eastern HWY 31	BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core		COMPILED BY JM
DATUM Geodetic	DATE November 24-25, 2015		CHECKED BY SAT

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
87.7	GROUND SURFACE							
0.0	ASPHALTIC CONCRETE							
87.5 0.2	Sand and gravel, trace silt (FILL) Compact Brown Moist							
		1	SS	21				
86.5 1.2	Sandy silt, some gravel, trace clay (FILL) Compact Grey-brown Moist							
		2	SS	15				
85.4 2.3	Sandy silt/silty sand, some gravel to gravelly, trace clay (FILL) Loose to dense Grey-brown Moist							
		3	SS	8				
		4	SS	11	25	32	34	9
		5	SS	9				
		6	SS	12				
		7	SS	26	66	20	16	3
		8	SS	31				
		9	SS	32	48	25	19	8
		10	SS	18				
79.4 8.3	CLAY Very stiff Dark grey Wet							
		11	SS	11				
78.8 8.9	Silty SAND to Sandy SILT, some gravel, trace to some clay, occasional gravel layer, contains cobbles and boulders (TILL) Very dense to very loose Grey Wet							
		12	SS	60	29	31	29	11

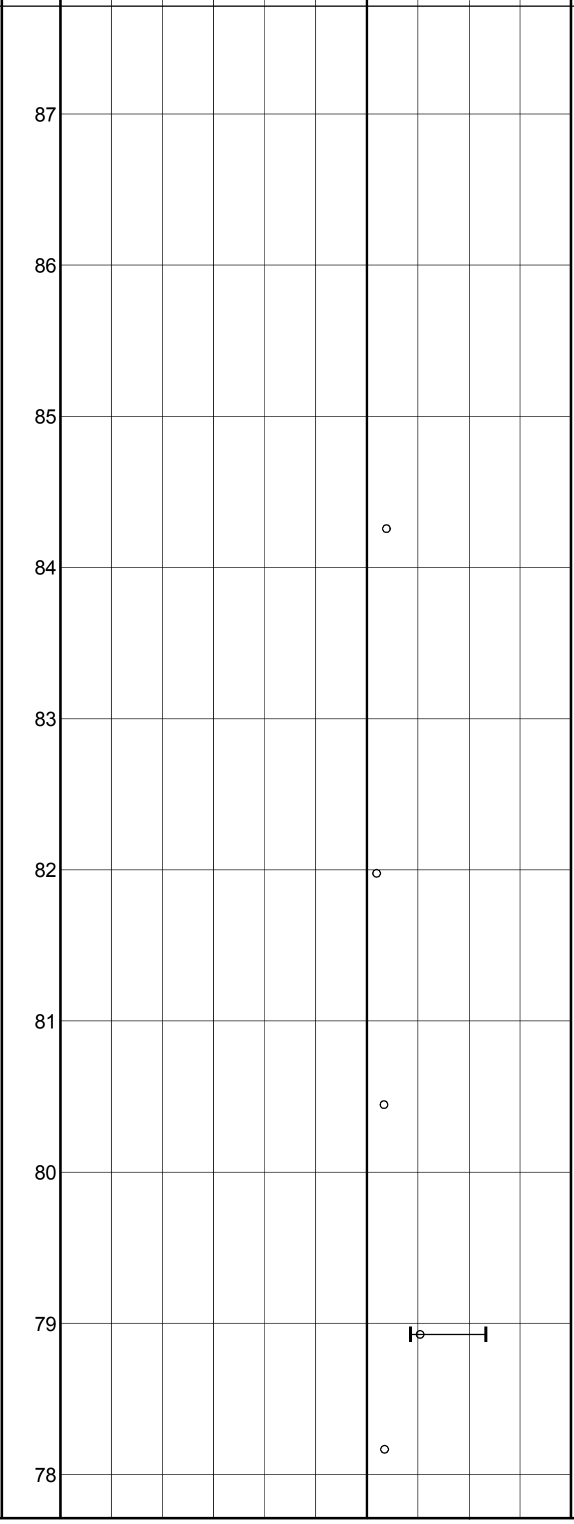
STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 25 50 75



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$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

PROJECT 12-1121-0099-1750
RECORD OF BOREHOLE No 15-3 SHEET 2 OF 5 **METRIC**
G.W.P. 4415-01-01
LOCATION N 4976011.7; E 407653.6 MTM NAD ZONE 9 (LAT. 44.916677; LONG. -75.197286)
ORIGINATED BY HEC
DIST Eastern HWY 31
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core
COMPILED BY JM
DATUM Geodetic
DATE November 24-25, 2015
CHECKED BY SAT

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	NUMBER	TYPE	"N" VALUES						
	Silty SAND to Sandy SILT, some gravel, trace to some clay, occasional gravel layer, contains cobbles and boulders (TILL) Very dense to very loose Grey Wet	13	SS	38						
						77				
		14	SS	27						
						76				
		15	SS	7				○H		22 32 37 9
						75				
						74				
		16	SS	4						
						73				
		17	SS	4				○		21 36 35 8
						72				
						71				
		18	SS	4						
						70				
		19	SS	3						
						69				
						68				

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$+^3, \times^3$: Numbers refer to Sensitivity
$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-3

SHEET 3 OF 5

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
DIST Eastern HWY 31
DATUM Geodetic
LOCATION N 4976011.7; E 407653.6 MTM NAD ZONE 9 (LAT. 44.916677; LONG. -75.197286)
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core
DATE November 24-25, 2015
ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	Silty SAND to Sandy SILT, some gravel, trace to some clay, occasional gravel layer, contains cobbles and boulders (TILL) Very dense to very loose Grey Wet	20	SS	2	23	34	34	9
		21	SS	8				
		22	SS	7				
		23	SS	23				
		24	SS	7				
60.2	SILTY CLAY Very stiff Dark grey Wet							
60.0 27.7	Silty SAND, some gravel and clay, contains cobbles and boulders (TILL) Loose Grey Wet	25	SS	7	20	33	32	15

GROUND WATER CONDITIONS

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

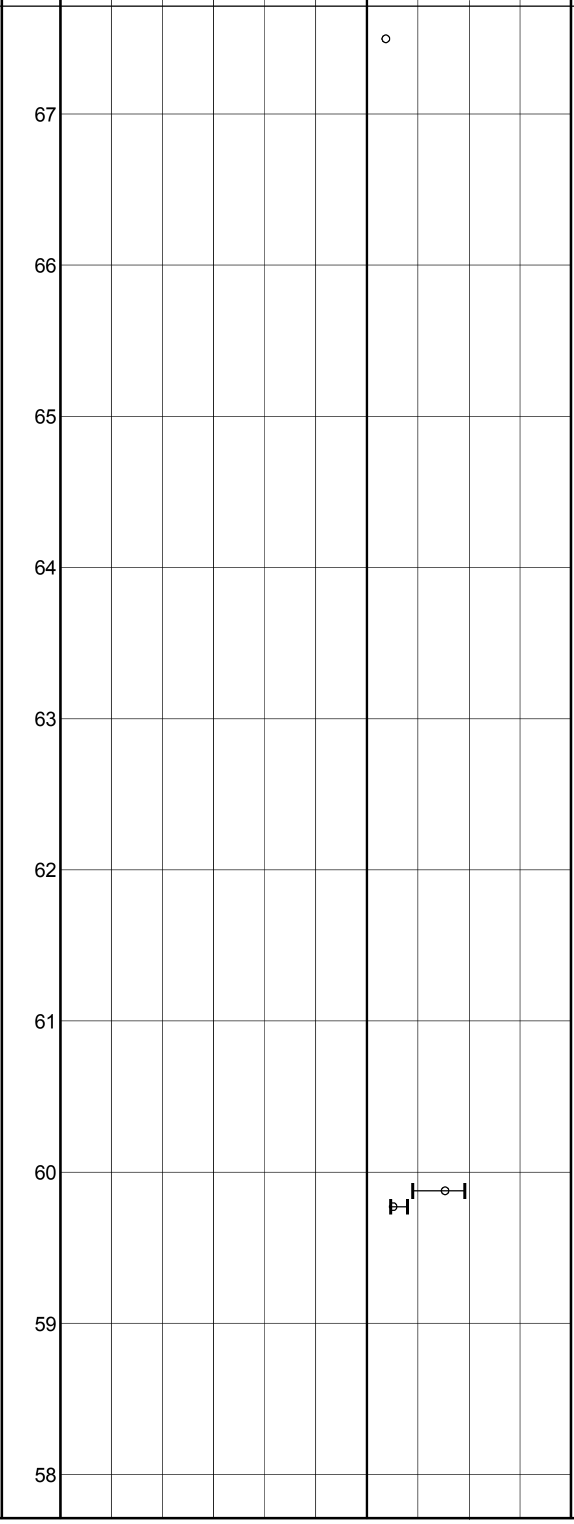
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-3

SHEET 4 OF 5

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
DIST Eastern HWY 31
DATUM Geodetic

LOCATION N 4976011.7; E 407653.6 MTM NAD ZONE 9 (LAT. 44.916677; LONG. -75.197286)
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core
DATE November 24-25, 2015

ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
57.5														
30.2	SAND, some silt, trace gravel Compact Black to grey-brown Wet						57							
			26	SS	35				○		8	75	14	3
56.5 31.2	Limestone (BEDROCK) Bedrock cored from depths of 31.2 m to 34.2 m For bedrock coring details refer to Record of Drillhole 15-3						56							
			1	RC	REC 94%						RQD = 73%			
							55							
			2	RC	REC 100%						RQD = 98%			
							54							
53.5 34.2	END OF BOREHOLE													

+^{3}, ×^{3}: Numbers refer to Sensitivity ○$^{3\%}$ STRAIN AT FAILURE

PROJECT: 12-1121-0099-1750

RECORD OF DRILLHOLE: 15-3

SHEET 5 OF 5

LOCATION: N 4976011.7 ;E 407653.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 24-25, 2015

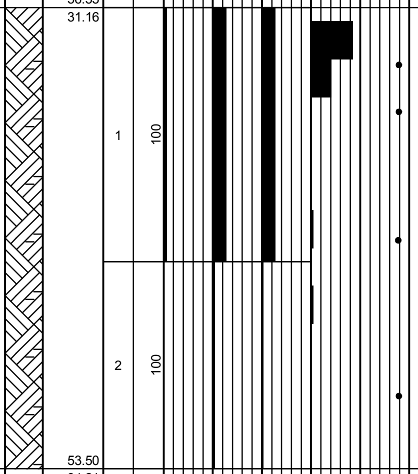
DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	FEATURES
		Continued from Record of Borehole 15-3		56.55						
	Rotary Drill HQ Core	Limestone (BEDROCK), with occasional medium thick interbeds of black shale Fresh Medium bedded Dark grey Fine grained Slightly porous		31.16						
							BD,PL,SM,Cl < 1mm	1	4	
							BD,PL,SM,Cl < 1mm	1	4	
32					1	100				
		- Broken core from 31.5 m to 31.9 m								
							BD,PL,SM	1	1	
33										
					2	100				
							BD,PL,SM	1	1	
34				53.50 34.21						
		END OF DRILLHOLE								
35										
36										
37										
38										
39										
40										
41										



DEPTH SCALE

1 : 50

GOLDER

LOGGED: HEC

CHECKED: SAT

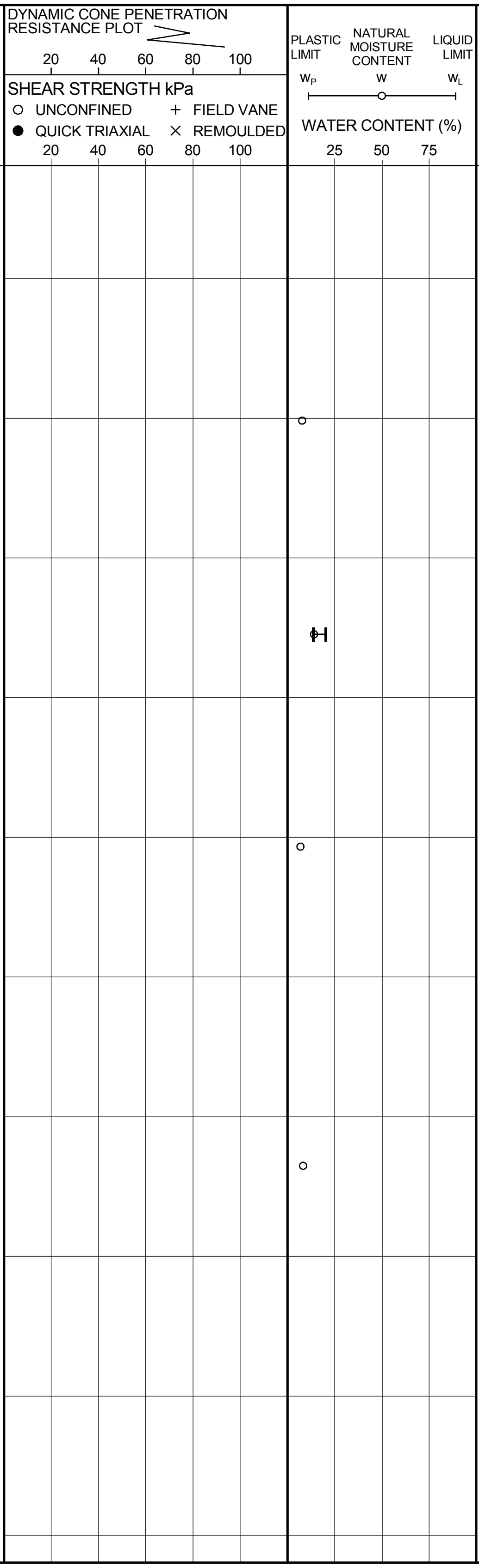
RECORD OF BOREHOLE No 15-4

SHEET 1 OF 5

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
DIST Eastern HWY 31
DATUM Geodetic
LOCATION N 4976053.3; E 407638.9 MTM NAD ZONE 9 (LAT. 44.917053; LONG. -75.197463)
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core
DATE December 1-4, 2015
ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

SOIL PROFILE ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
81.8	GROUND SURFACE								
0.0	Crushed stone, trace sand and silt (FILL) Compact Grey Moist								
					81				
		1	SS	21					
80.6 1.2	Silty sand, some gravel, contains organic matter and cobbles (FILL) Compact to loose Grey-brown to brown Wet								
		2	SS	13	80				
		3	SS	5					
					79				
78.9 2.9	Sandy SILT to Silty SAND, trace to some gravel and clay, contains cobbles and boulders (TILL) Compact to very loose Grey Wet								
		4	SS	12		5	23	48	24
					78				
		5	SS	17					
		6	SS	17	77	25	40	27	8
		7	SS	9					
					76				
		8	SS	8					
					75				
		9	SS	8		18	37	35	10
		10	SS	6	74				
		11	SS	19					
					73				
		12	SS	3					
					72				



UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-4

SHEET 2 OF 5

METRIC

PROJECT 12-1121-0099-1750

G.W.P. 4415-01-01

DIST Eastern HWY 31

DATUM Geodetic

LOCATION N 4976053.3; E 407638.9 MTM NAD ZONE 9 (LAT. 44.917053; LONG. -75.197463)

BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core

DATE December 1-4, 2015

ORIGINATED BY HEC

COMPILED BY JM

CHECKED BY SAT

SOIL PROFILE		SAMPLES									
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---											
	Sandy SILT to Silty SAND, trace to some gravel and clay, contains cobbles and boulders (TILL) Compact to very loose Grey Wet	13	SS	2					18	40	34	8
					71							
		14	TO	PH								
					70							
		15	SS	2								
					69							
					68							
		16	SS	4								
					67							
		17	SS	2								
					66							
					65							
		18	SS	3					16	35	38	11
					64							
		19	SS	3								
					63							
					62							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-4

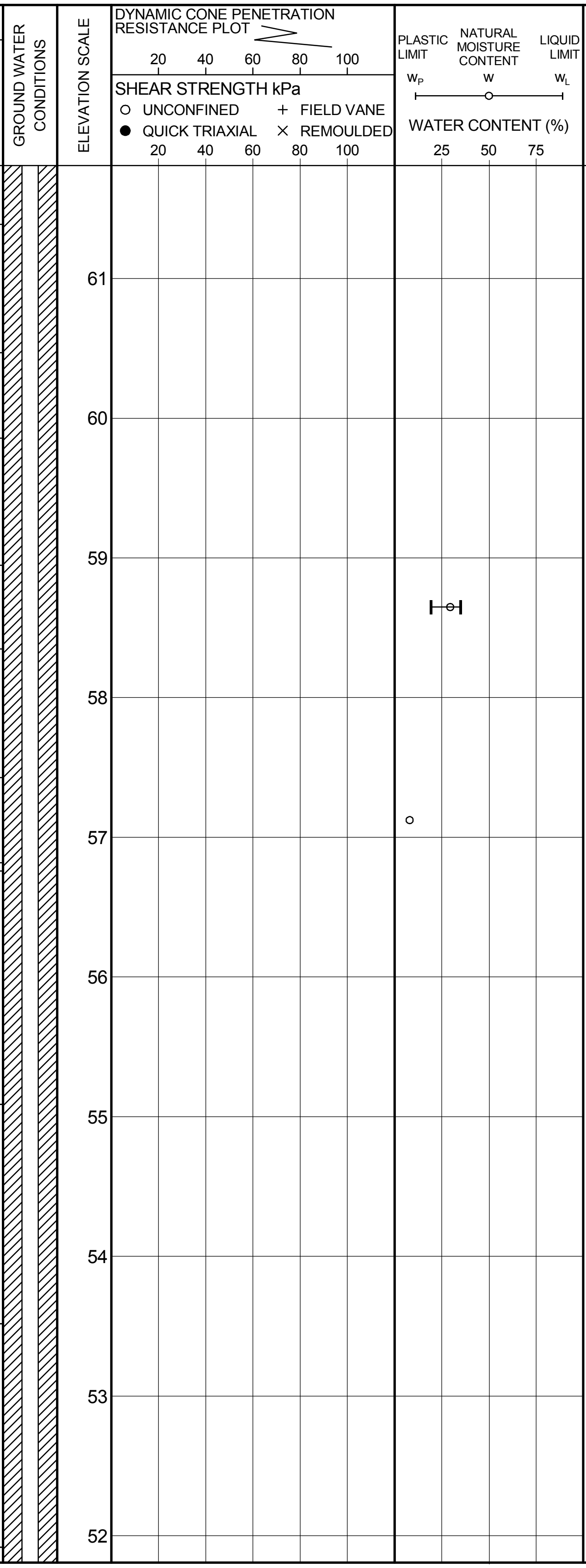
SHEET 3 OF 5

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
DIST Eastern HWY 31
DATUM Geodetic
LOCATION N 4976053.3; E 407638.9 MTM NAD ZONE 9 (LAT. 44.917053; LONG. -75.197463)
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core
DATE December 1-4, 2015
ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

--- *CONTINUED FROM PREVIOUS PAGE* ---

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	Sandy SILT to Silty SAND, trace to some gravel and clay, contains cobbles and boulders (TILL) Compact to very loose Grey Wet	20	SS	4	
		21	SS	2	
58.9 22.9	SILTY CLAY Very stiff Grey Wet	22	SS	5	
57.4 24.4	SAND, some silt, trace clay Compact Grey Wet	23	SS	17	
56.8 25.1	Limestone (BEDROCK)	1	RC	REC 100%	RQD = 47%
		2	RC	REC 100%	RQD = 100%
		3	RC	REC 100%	RQD = 100%
52.4 29.4		4	RC		RQD = 100%



UNIT WEIGHT γ kN/m³

GR SA SI CL

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IM\GINT\PHASE 1750\1211210099-1750.GPJ GAL-GTA.GDT 11/15/18 JM

RECORD OF BOREHOLE No 15-4

SHEET 4 OF 5

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
DIST Eastern HWY 31
DATUM Geodetic
LOCATION N 4976053.3; E 407638.9 MTM NAD ZONE 9 (LAT. 44.917053; LONG. -75.197463)
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core
DATE December 1-4, 2015
ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION *--- CONTINUED FROM PREVIOUS PAGE ---*	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Dolostone (BEDROCK) Bedrock cored from depths of 25.1 m to 30.1 m	▨	4	RC	REC 100%	▨					RQD = 100%
51.2 30.6	For bedrock coring details refer to Record of Drillhole 15-4 END OF BOREHOLE NOTES: 1. Bentonite-cement grouted 60 mm Diam. PVC casing installed between 0.0 m and 30.6 m depths.										

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

PROJECT: 12-1121-0099-1750

RECORD OF DRILLHOLE: 15-4

SHEET 5 OF 5

LOCATION: N 4976053.3 ;E 407638.9

DRILLING DATE: December 1-4, 2015

DATUM: Geodetic

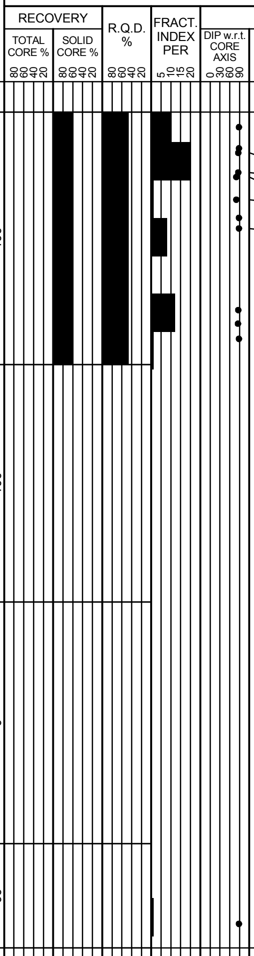
DRILL RIG: CME 750

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: CCC Drilling

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja
		Continued from Record of Borehole 15-4	56.76					
26	Rotary Drill HQ Core	Limestone (BEDROCK), with thin interbeds of black shale Fresh Thinly to medium bedded Dark grey Fine grained Slightly porous Medium strong - Broken core from 25.2 m to 25.3 m - Broken core from 25.3 m to 25.5 m - Broken core from 25.8 m to 25.8 m - Broken core from 26.4 m to 26.5 m	25.05	1	100	BD,CU,RO BD,PL,RO BD,PL,RO BD,PL,RO JN,PL,RO JN,PL,RO BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM	3 1.5 1.5 1.5 1.5 1.5 1 1 1 1 1	1 1 1 1 1 1 1 1 1 1 1
27 28				2	100			
29				3	0			
30		Dolostone (BEDROCK), with interbeds of thin to medium thick argillaceous limestone Fresh Medium bedded Light green to dark grey Fine grained Non-porous Medium strong to strong	52.40 29.41	4	96	BD,PL,SM	1	1
31 32 33 34 35		END OF DRILLHOLE	51.23 30.58					



HYDRAULIC CONDUCTIVITY K, cm/sec: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WEATHERING INDEX: W1 W2 W3 W4 W5 W6

FEATURES

SYMBOLIC LOG

DEPTH SCALE

1 : 50

LOGGED: HEC

CHECKED: SAT

RECORD OF BOREHOLE No 15-4-1

SHEET 1 OF 3

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
DIST Eastern HWY 31
DATUM Geodetic
LOCATION N 4976053.3; E 407638.9 MTM NAD ZONE 9 (LAT. 44.917053; LONG. -75.197463)
BOREHOLE TYPE Dynamic Cone
DATE December 4, 2015
ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

SOIL PROFILE

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES
81.8	GROUND SURFACE				
0.0	Crushed stone, trace sand and silt (FILL) Compact Grey Moist				
80.6 1.2	Silty sand, some gravel, contains organic matter and cobbles (FILL) Compact to loose Grey-brown to brown Wet				
78.9 2.9	Sandy SILT to Silty SAND, trace to some gravel and clay, contains cobbles and boulders (TILL) Compact to very loose Grey Wet				

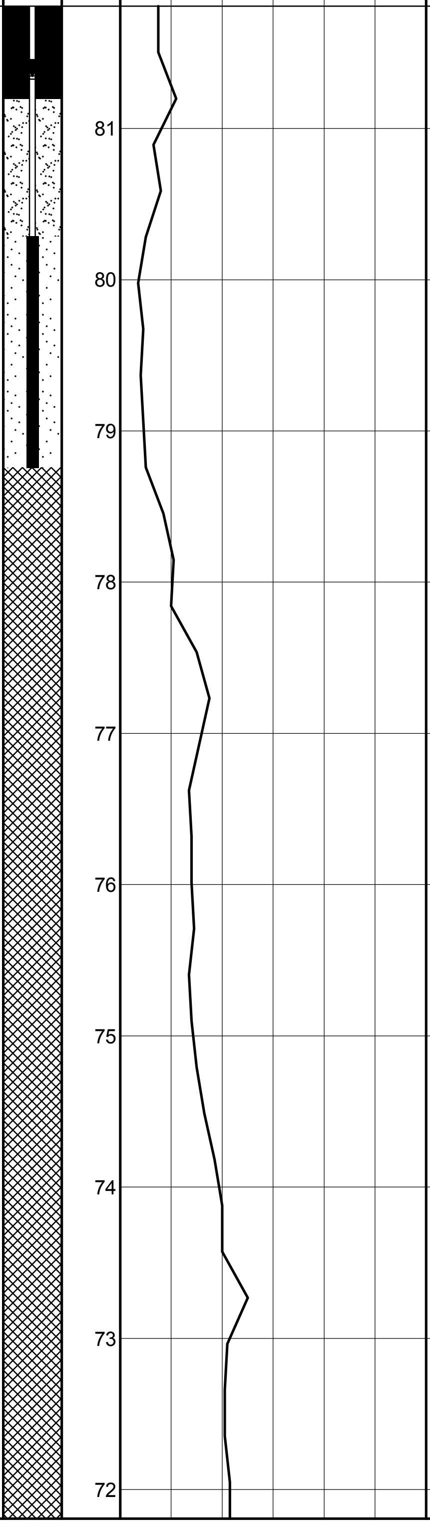
GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

PLASTIC LIMIT W_P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W_L
WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL



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+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

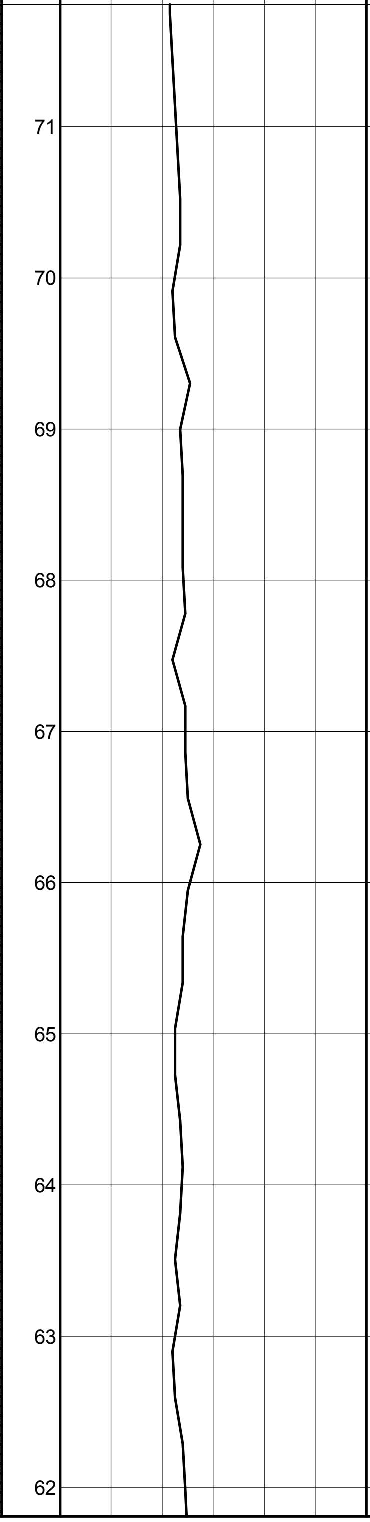
RECORD OF BOREHOLE No 15-4-1

SHEET 2 OF 3

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
LOCATION N 4976053.3; E 407638.9 MTM NAD ZONE 9 (LAT. 44.917053; LONG. -75.197463)
ORIGINATED BY HEC
DIST Eastern HWY 31
BOREHOLE TYPE Dynamic Cone
COMPILED BY JM
DATUM Geodetic
DATE December 4, 2015
CHECKED BY SAT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Sandy SILT to Silty SAND, trace to some gravel and clay, contains cobbles and boulders (TILL) Compact to very loose Grey Wet						71 70 69 68 67 66 65 64 63 62				



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+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IM\GINT\PHASE 1750\1211210099-1750.GPJ GAL-GTA.GDT 11/15/18 JM

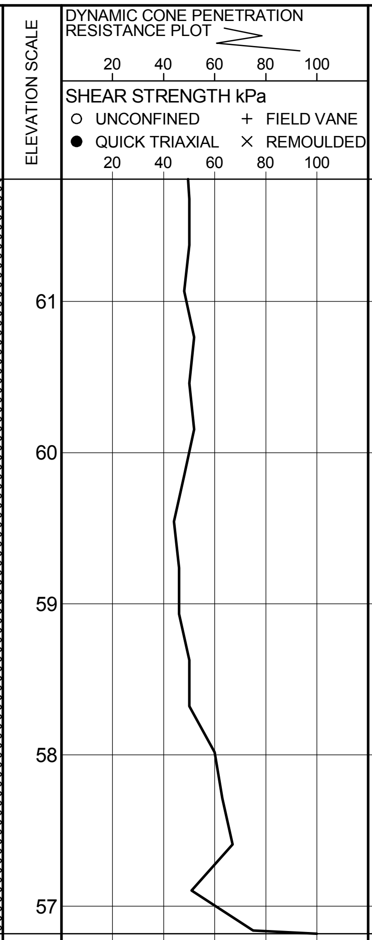
RECORD OF BOREHOLE No 15-4-1

SHEET 3 OF 3

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
DIST Eastern HWY 31
DATUM Geodetic
LOCATION N 4976053.3; E 407638.9 MTM NAD ZONE 9 (LAT. 44.917053; LONG. -75.197463)
BOREHOLE TYPE Dynamic Cone
DATE December 4, 2015
ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

SOIL PROFILE			SAMPLES									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Sandy SILT to Silty SAND, trace to some gravel and clay, contains cobbles and boulders (TILL) Compact to very loose Grey Wet										
58.9 22.9	SILTY CLAY Very stiff Grey Wet										
57.4 24.4	SAND, some silt, trace clay Compact Grey Wet										
56.8 25.0	END OF BOREHOLE DCPT REFUSAL NOTES: 1. Stratigraphy inferred from Record of Borehole 15-4. 2. Water level in well screen at a depth of 0.5 m below ground surface (Elev. 81.3 m), measured on April 1, 2016.										



+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IM\GINT\PHASE 1750\1211210099-1750.GPJ GAL-GTA.GDT 11/15/18 JM

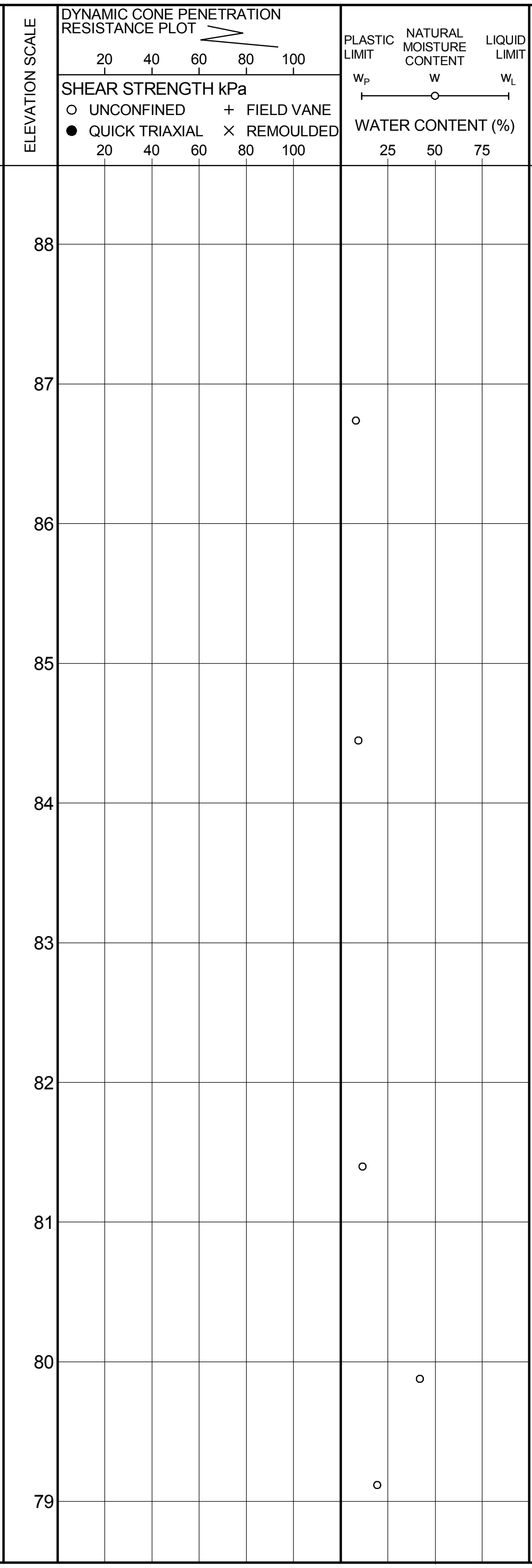
RECORD OF BOREHOLE No 15-5

SHEET 1 OF 5

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
DIST Eastern HWY 31
DATUM Geodetic
LOCATION N 4976093.0; E 407614.7 MTM NAD ZONE 9 (LAT. 44.917414; LONG. -75.197762)
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core
DATE November 26-30, 2015
ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
88.6	GROUND SURFACE									
0.0	ASPHALTIC CONCRETE	■								
88.4 0.2	Sand and gravel (FILL) Compact Brown Moist	▨								
			1	SS	29					
87.2 1.4	Silty sand/sandy silt, some gravel and clay (FILL) Loose to compact Brown to grey-brown Moist	▨	2	SS	4					
			3	SS	8					
			4	SS	27					
			5	SS	33		22	35	32	11
			6	SS	18					
			7	SS	13					
			8	SS	14					
			9	SS	10		15	31	37	17
			10	SS	12					
80.1 8.5	Organic CLAYEY SILT, contains roots (ALLUVIUM) Dark brown Wet		11	SS	4					
79.7 8.9	Silty SAND, trace gravel, contains shells and organic matter Compact Brown Wet		12	SS	14					
78.7										



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-5

SHEET 2 OF 5

METRIC

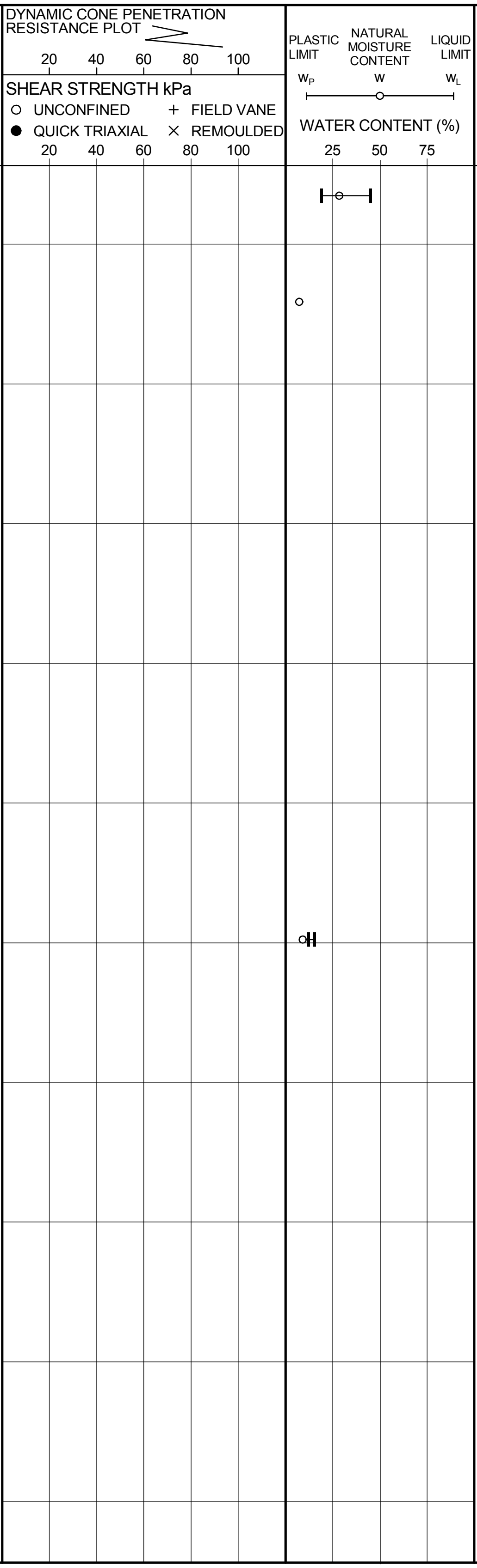
PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
DIST Eastern HWY 31
DATUM Geodetic

LOCATION N 4976093.0; E 407614.7 MTM NAD ZONE 9 (LAT. 44.917414; LONG. -75.197762)
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core
DATE November 26-30, 2015

ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
10.1	SILTY CLAY, trace gravel Grey Wet	13	SS	31				
	Silty SAND to Sandy SILT, some gravel and clay, contains cobbles and boulders (TILL) Dense to very loose Grey Wet	14	SS	21	17	38	35	10
		15	SS	10				
		16	SS	2				
		17	SS	5	20	34	35	11
		18	SS	10				
		19	SS	12				



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$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-5

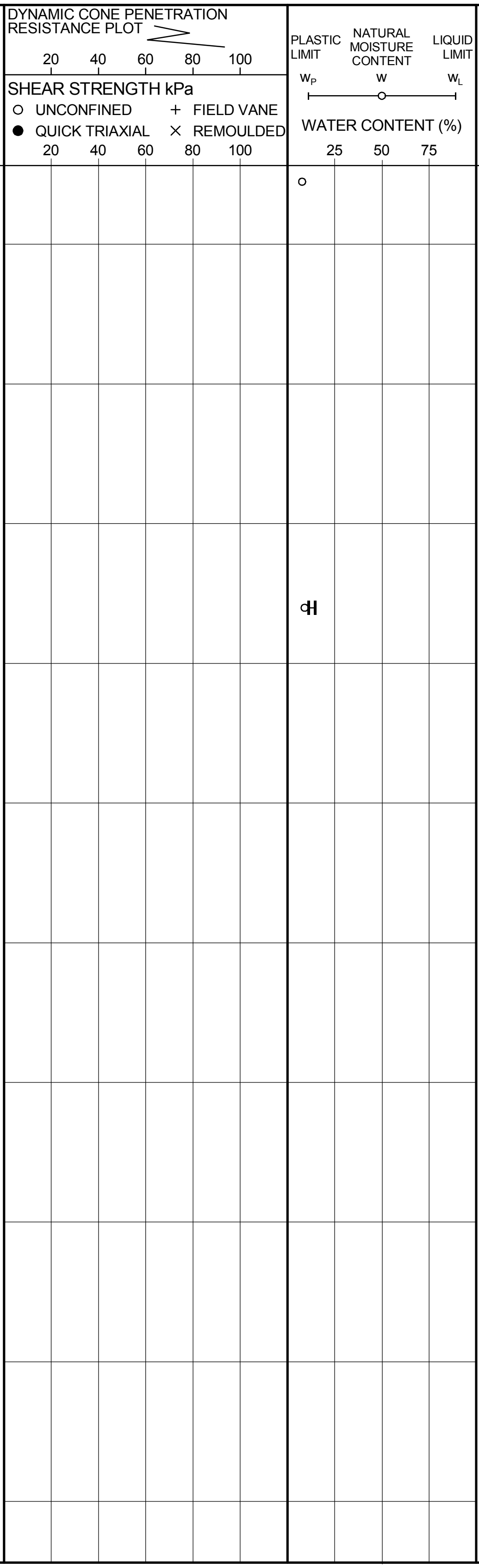
SHEET 3 OF 5

METRIC

PROJECT 12-1121-0099-1750
G.W.P. 4415-01-01
DIST Eastern HWY 31
DATUM Geodetic
LOCATION N 4976093.0; E 407614.7 MTM NAD ZONE 9 (LAT. 44.917414; LONG. -75.197762)
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core
DATE November 26-30, 2015
ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

--- CONTINUED FROM PREVIOUS PAGE ---

SOIL PROFILE ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	Silty SAND to Sandy SILT, some gravel and clay, contains cobbles and boulders (TILL) Dense to very loose Grey Wet	20	SS	6				28	34	19	9
						68					
		21	SS	10							
						67					
						66					
		22	SS	2				19	36	35	10
						65					
						64					
		23	SS	1							
						63					
						62					
						61					
		24	SS	6							
						60					
						59					



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$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-5

SHEET 4 OF 5

METRIC

PROJECT	12-1121-0099-1750		
G.W.P.	4415-01-01	LOCATION	N 4976093.0; E 407614.7 MTM NAD ZONE 9 (LAT. 44.917414; LONG. -75.197762)
ORIGINATED BY	HEC		
DIST	Eastern	HWY	31
BOREHOLE TYPE	Power Auger 200 mm Diam. (Hollow Stem), Wash Boring, Rotary Drill HQ Core	COMPILED BY	JM
DATUM	Geodetic	DATE	November 26-30, 2015
CHECKED BY	SAT		

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Silty SAND to Sandy SILT, some gravel and clay, contains cobbles and boulders (TILL) Dense to very loose Grey Wet										
			25	SS	15		58				
			26	RC	DD		57				
			27	RC	DD		56				
55.9 32.8	Limestone (BEDROCK) Bedrock cored from depths of 32.8 m to 36.1 m For bedrock coring details refer to Record of Drillhole 15-5		1	RC	REC 100%						RQD = 85%
			2	RC	REC 100%		55 54				RQD = 85%
			3	RC	REC 100%		53				RQD = 98%
52.5 36.1	END OF BOREHOLE										

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

PROJECT: 12-1121-0099-1750

RECORD OF DRILLHOLE: 15-5

SHEET 5 OF 5

LOCATION: N 4976093.0 ;E 407614.7

DRILLING DATE: November 26-30, 2015

DATUM: Geodetic

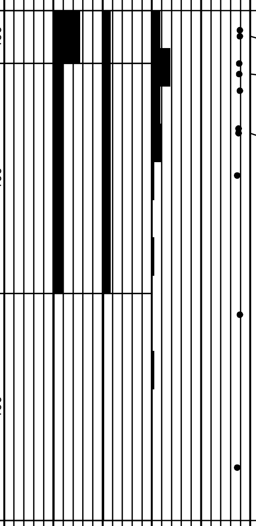
INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC Drilling

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN
		Continued from Record of Borehole 15-5	55.81		
33–36	Rotary Drill HQ Core	Limestone (BEDROCK), with medium thick interbeds of black fissile shale Fresh Medium to thickly bedded Dark grey Fine to medium grained Slightly porous Medium strong - Broken core from 32.8 m to 32.9 m - Broken core from 33.2 m to 33.3 m - Broken core from 33.5 m to 33.6 m	32.75	1 2 3	100 100 100
		END OF DRILLHOLE	52.44 36.12		



TYPE AND SURFACE DESCRIPTION	Jr	Ja
BD,PL,SM,Cl <1 mm	1	4
BD,PL,SM,Cl <1 mm	1	4
BD,PL,SM	1	1
BD,PL,SM	1	1
BD,PL,SM	1	1
BD,PL,SM	1	1
BD,CU,SM	2	1
BD,CU,RO	3	1
BD,PL,SM	1	1
BD,PL,SM	1	1

HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WEATHERING INDEX W1 W2 W3 W4 W5 W6

FEATURES

DEPTH SCALE
1 : 50

GOLDER

LOGGED: HEC
CHECKED: SAT

GOLDER

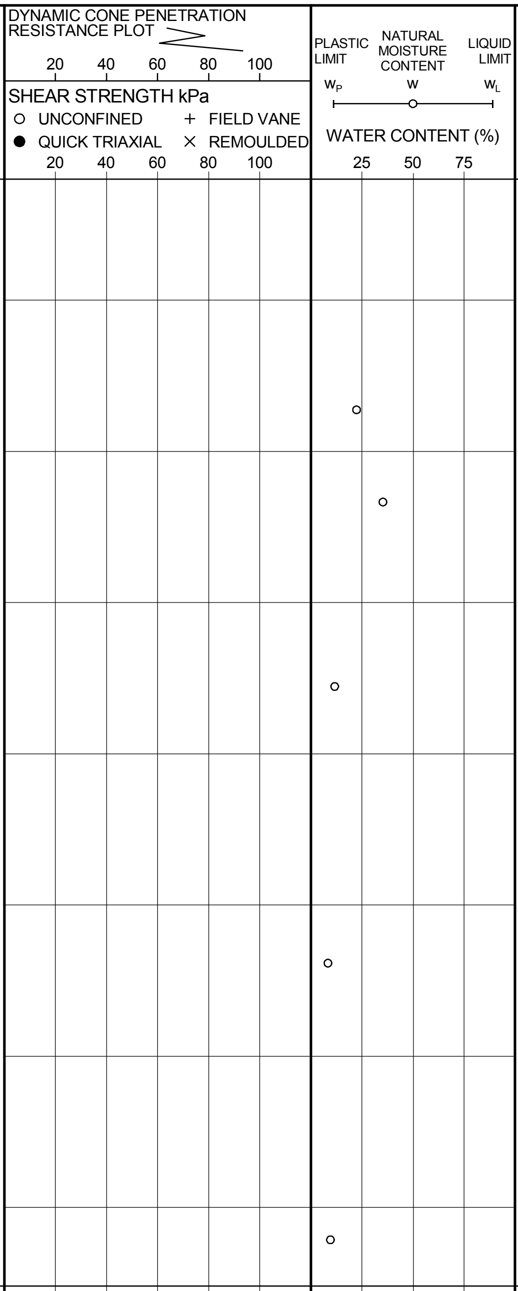
RECORD OF BOREHOLE No 15-6

SHEET 1 OF 1

METRIC

PROJECT 12-1121-0099-1750			
G.W.P. 4415-01-01	LOCATION N 4976137.3; E 407615.7 MTM NAD ZONE 9 (LAT. 44.917813; LONG. -75.197740)	ORIGINATED BY HEC	
DIST Eastern	HWY 31	BOREHOLE TYPE Portable Drill, AW/BW Casing	COMPILED BY JM
DATUM Geodetic	DATE December 16, 2015	CHECKED BY SAT	

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
81.8	GROUND SURFACE										
0.0	TOPSOIL Brown Moist	1	SS	8							
81.2 0.6	Layered organic clayey silt and sandy silt, trace gravel, contains roots (ALLUVIUM) Very stiff Brown, grey and black	2	SS	13		81					
		3	SS	16			ORG = 7.7				
		4	SS	12		80					
79.4	SAND, trace silt Brown Wet										
79.3 2.5	Sandy SILT to Silty SAND, some clay, trace to some gravel, contains cobbles and boulders (TILL) Compact to dense Grey Wet	5	SS	25		79					
		6	SS	28				8	32	40	20
		7	SS	45		78					
		8	SS	49		77					
		9	SS	23							
		10	SS	18		76					
		11	SS	16		75					
		12	SS	17				14	38	37	11
74.5 7.3	END OF BOREHOLE										



REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-7

SHEET 1 OF 1

METRIC

PROJECT 12-1121-0099-1750

G.W.P. 4415-01-01

DIST Eastern HWY 31

DATUM Geodetic

LOCATION N 4976181.6; E 407590.4 MTM NAD ZONE 9 (LAT. 44.918215; LONG. -75.198051)

BOREHOLE TYPE Portable Drill, AW/BW Casing

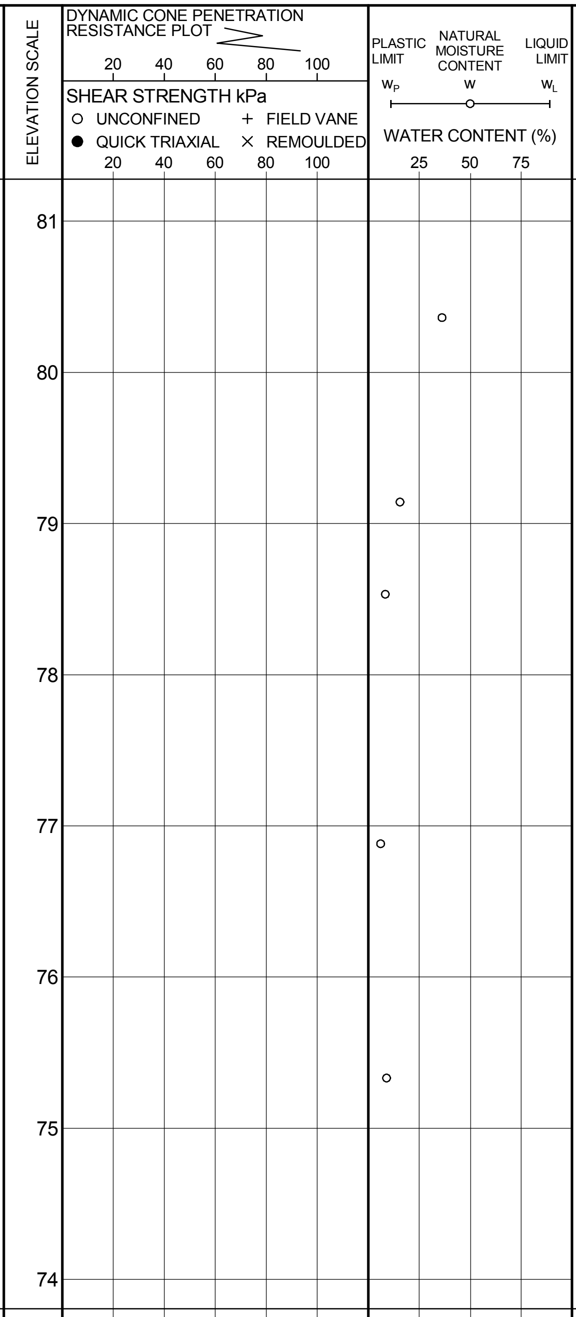
DATE December 15-16, 2015

ORIGINATED BY HEC

COMPILED BY JM

CHECKED BY SAT

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
81.3	GROUND SURFACE								
0.0	TOPSOIL Brown Moist	1	SS	7					
80.7 0.6	Organic clayey silt (ALLUVIUM) Very stiff Dark brown Moist to wet	2	SS	9	ORG = 11.2				
80.2 1.1	Silty SAND, trace to some gravel, trace clay Compact Grey Wet	3	SS	11					
79.5 1.8	SAND, trace silt Compact Grey Wet	4	SS	32					
79.2 2.1	Sandy SILT to Silty SAND, some gravel to gravelly, trace clay, contains cobbles and boulders (TILL) Very dense to dense Grey Wet	5	SS	63		12	34	42	12
		6	SS	53					
		7	SS	30					
		8	SS	86/0.25		47	27	20	6
76.8 4.5	GRAVEL, COBBLES and BOULDERS								
76.2 5.1	Silty SAND, some gravel, trace clay, contains cobbles and boulders (TILL) Compact Grey Wet	9	SS	13					
		10	SS	11		15	40	36	9
		11	SS	12					
		12	SS	10					
73.8 7.5	END OF BOREHOLE								



$+^3, \times^3$: Numbers refer to Sensitivity

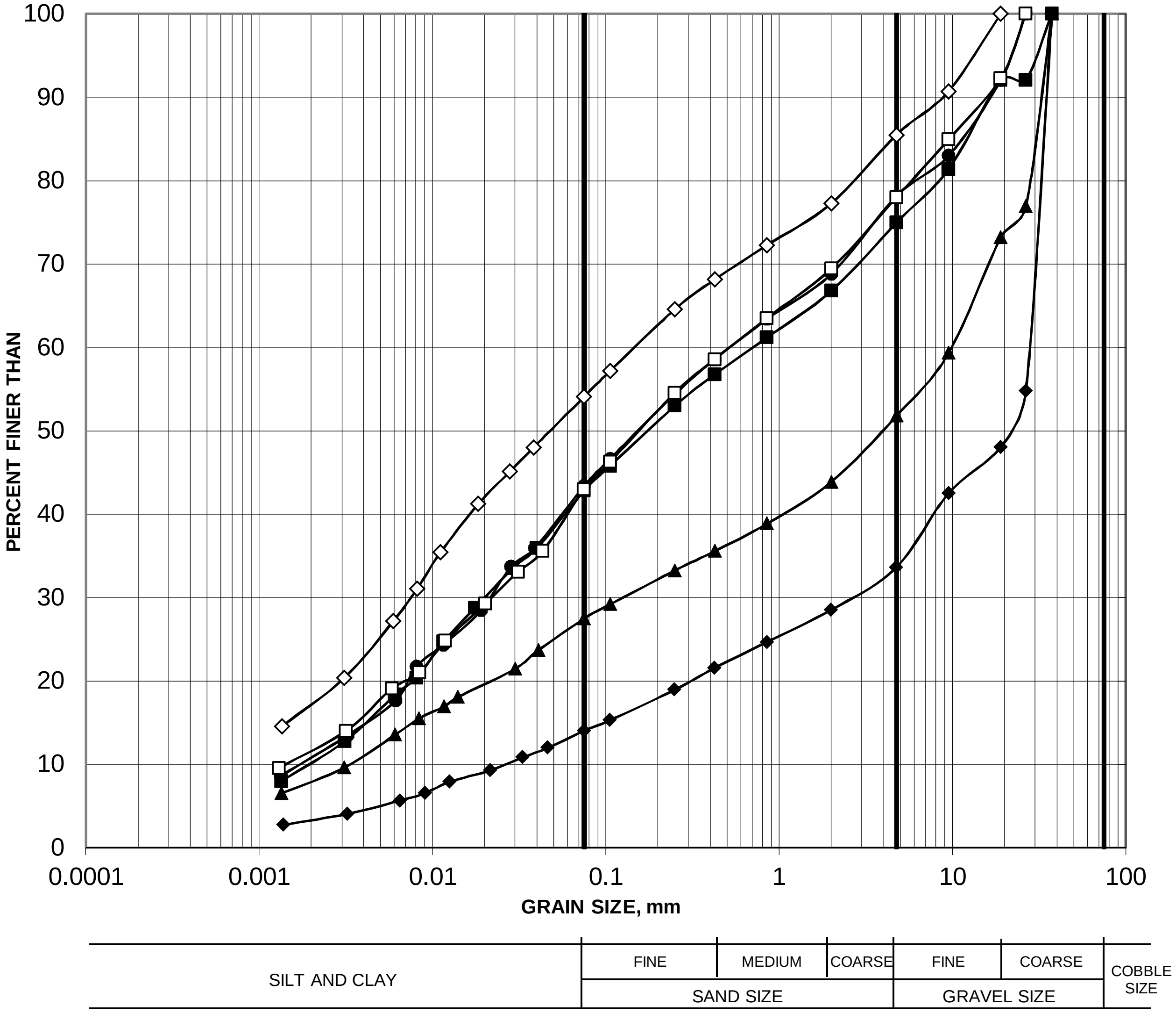
$\bigcirc^{3\%}$ STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1

EMBANKMENT FILL



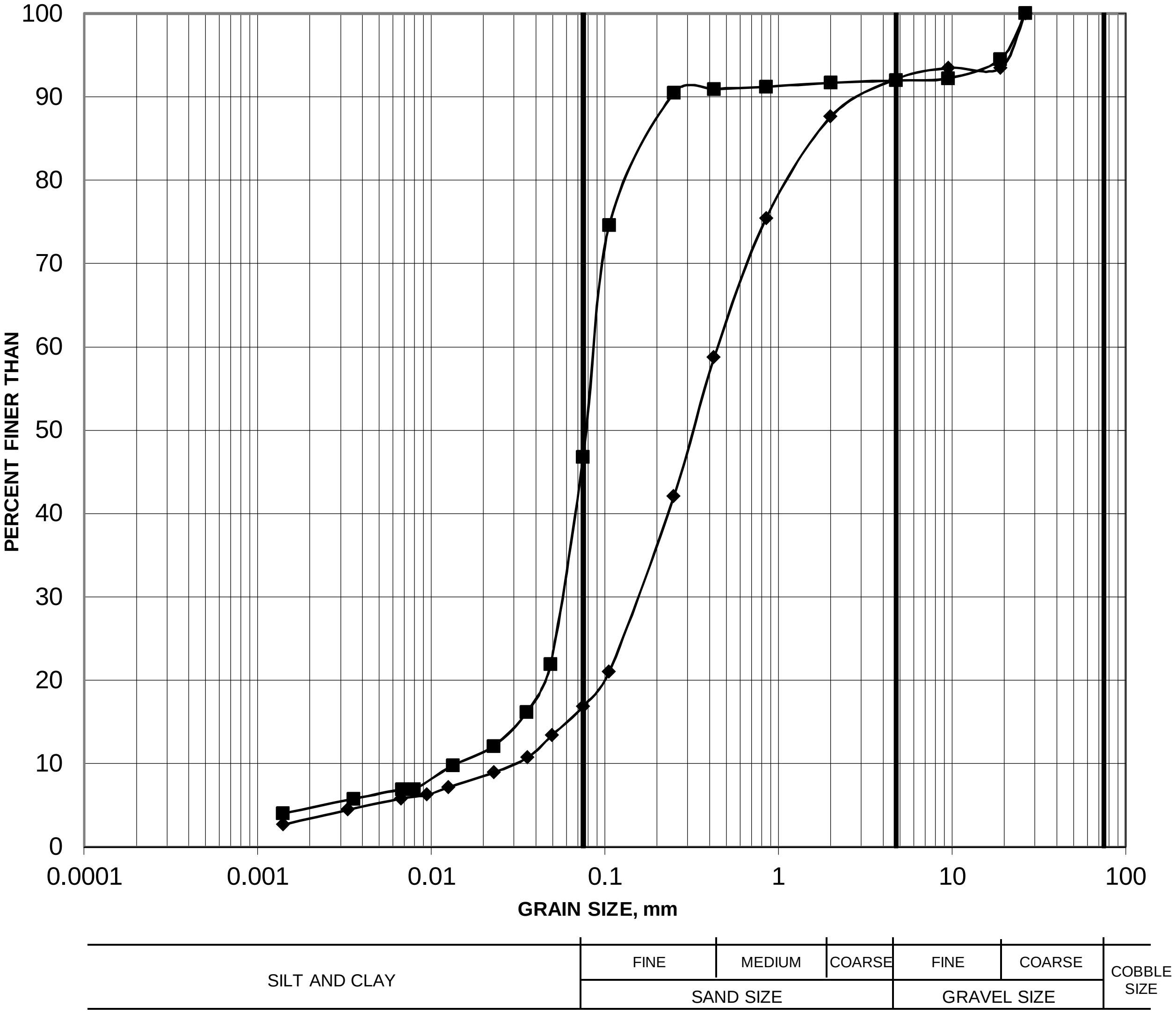
Borehole	Sample	Depth (m)
■ 15-3	4	3.15-3.76
◆ 15-3	7	5.44-6.05
▲ 15-3	9	6.96-7.57
● 15-5	2	1.52-2.13
□ 15-5	5	3.81-4.42
◇ 15-5	9	6.86-7.47

Created by: MI
Checked by: CNM

Project: <u>12-1121-0099/ 1750</u> **Golder Associates**

GRAIN SIZE DISTRIBUTION	FIGURE B2

Silty SAND to SAND, some silt

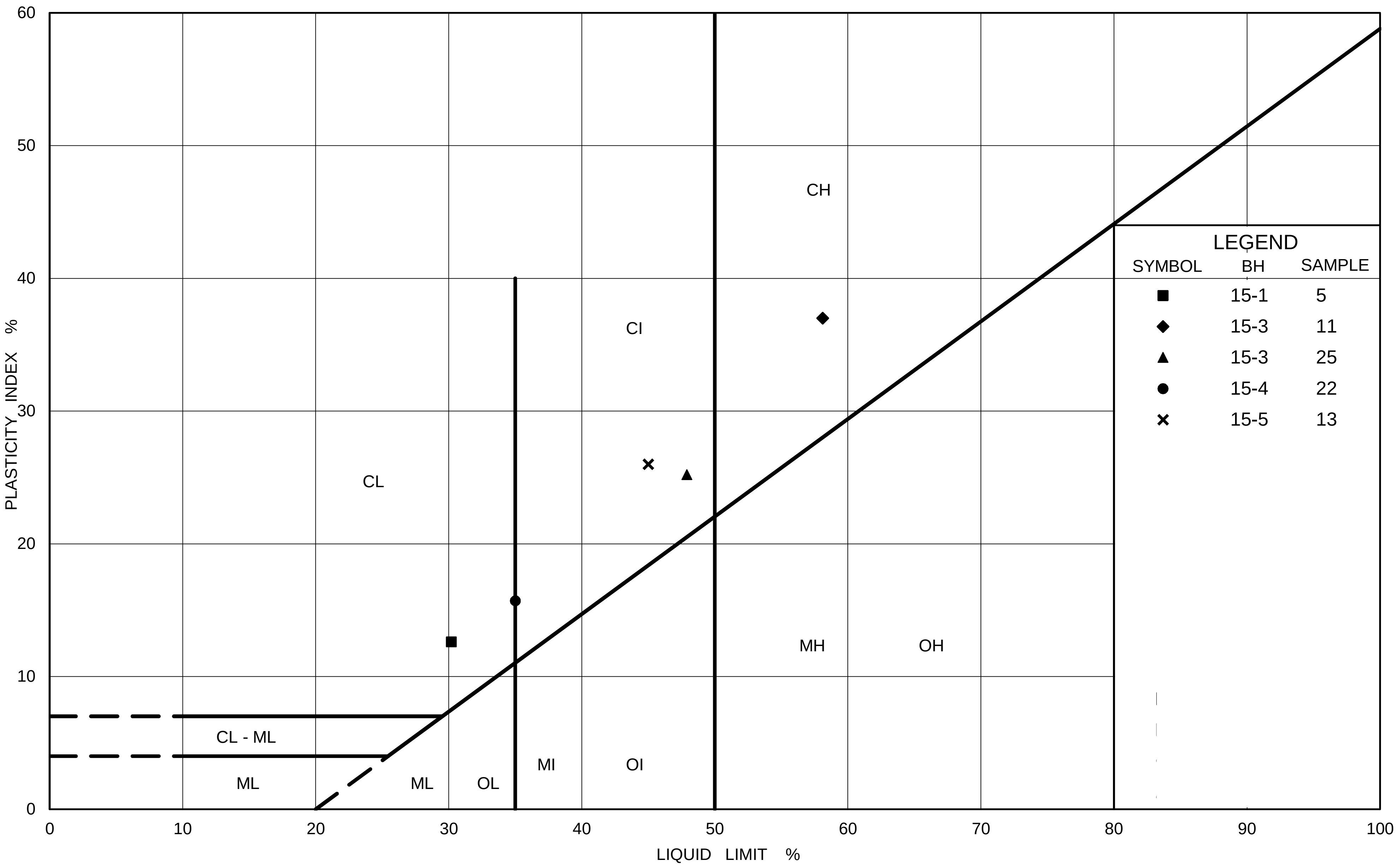


Borehole	Sample	Depth (m)
■ 15-2	4	1.83-2.44
◆ 15-3	26	30.59-30.79

Project: 12-1121-0099/ 1750

Golder Associates

Created by: MI
Checked by: CNM



Oct 75, FF-S-21

Ministry of Transportation

Ontario

PLASTICITY CHART
SILTY CLAY to CLAY

FIG No. B3

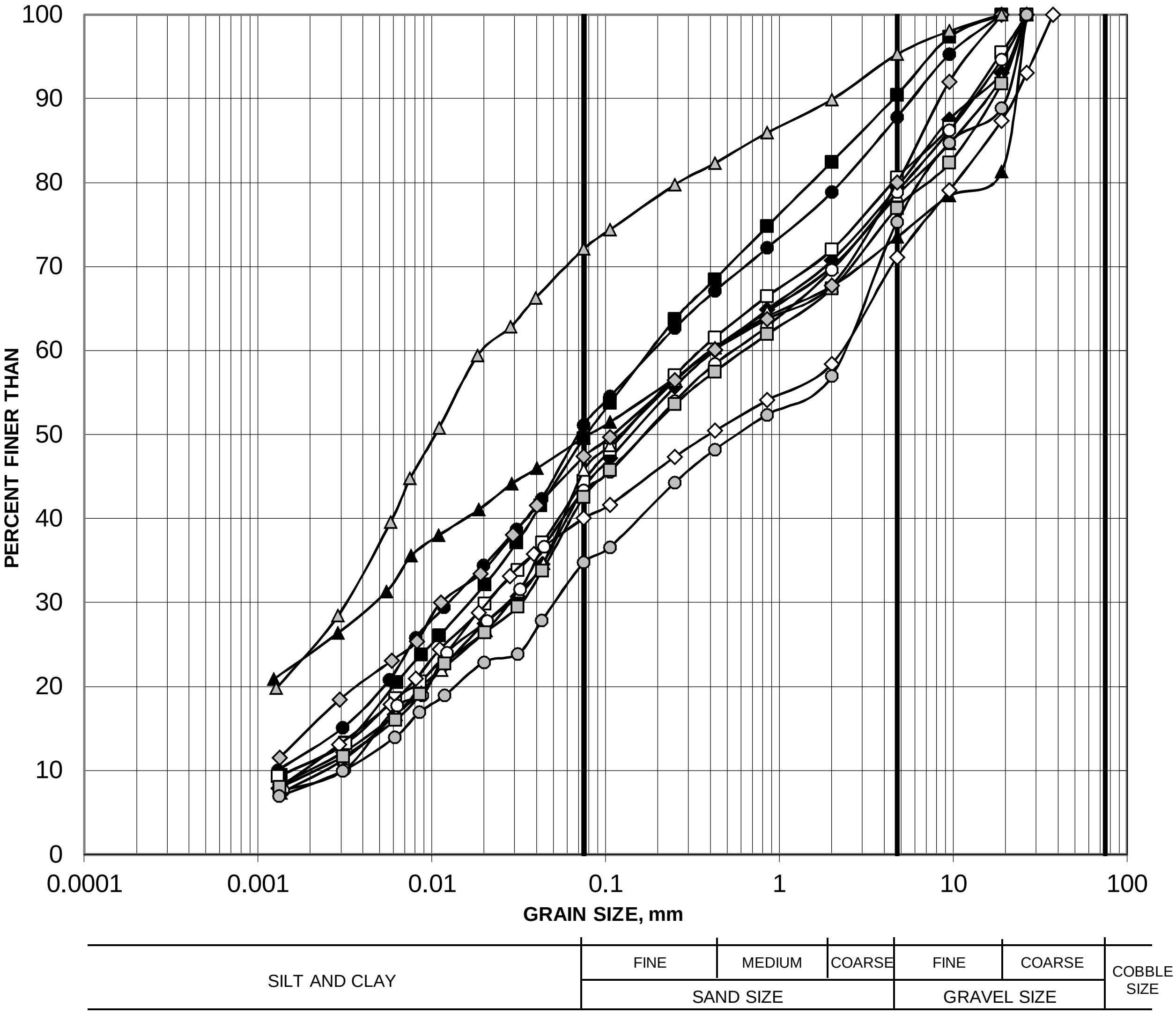
Project No. 12-1121-0099/1750

Compiled By : MI Checked By : CNM

GRAIN SIZE DISTRIBUTION

FIGURE B4a

Silty SAND to Sandy SILT (TILL)



Borehole	Sample	Depth (m)
—■— 15-1	9	4.88-5.49
—◆— 15-1	12	6.71-7.32
—▲— 15-2	6	3.05-3.66
—●— 15-2	7	3.66-4.27
—□— 15-2	10	5.79-6.40
—◇— 15-3	12	9.25-9.85
—△— 15-3	15	12.30-12.91
—○— 15-3	17	15.34-15.95
—▣— 15-3	20	19.92-20.53
—◈— 15-3	25A	27-74-28.15
—◬— 15-4	4	3.05-3.66
—◉— 15-4	6	4.57-5.18

Created by: MI
Checked by: CNM

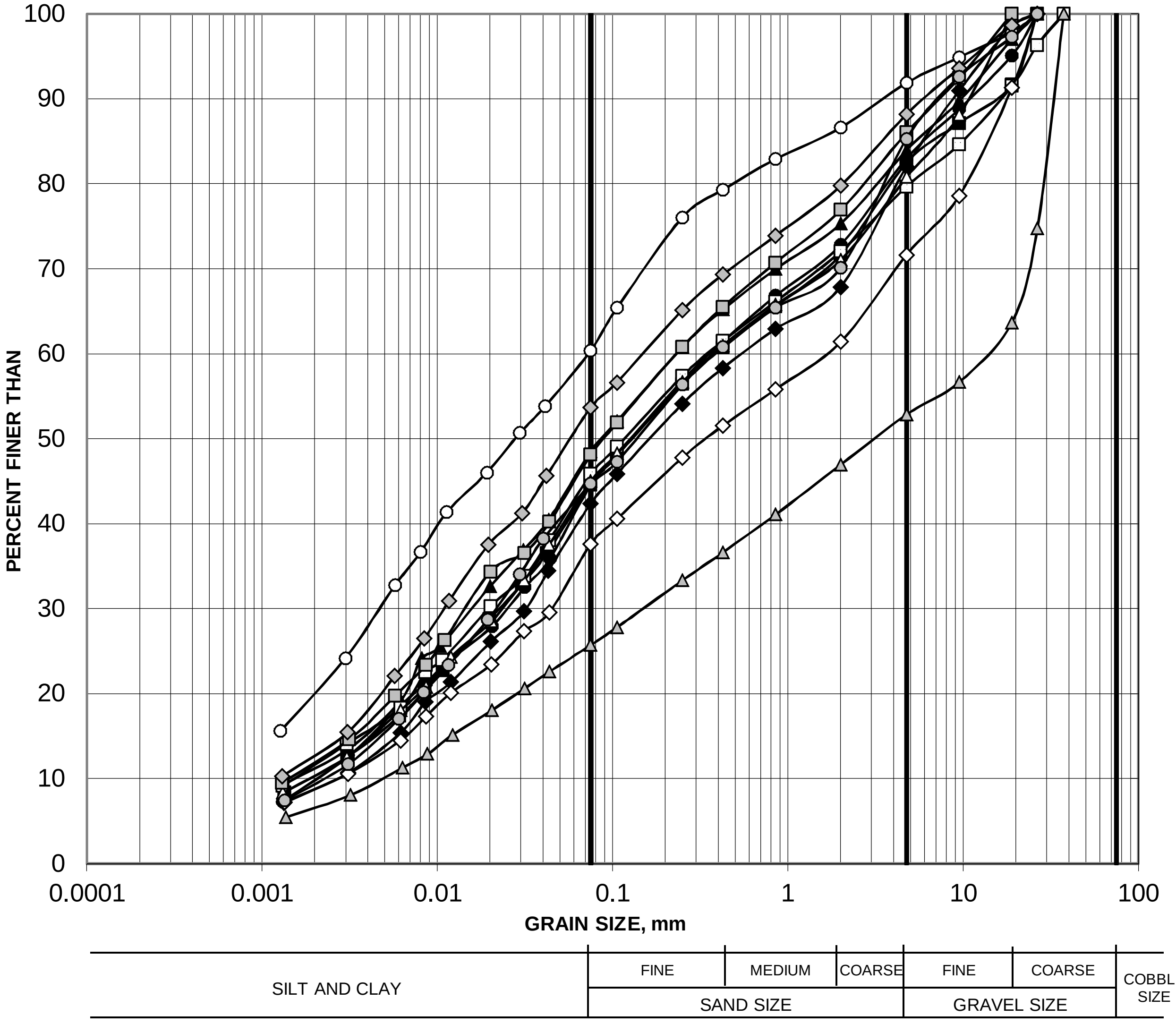
Project: 12-1121-0099/ 1750

Golder Associates

GRAIN SIZE DISTRIBUTION

FIGURE B4b

Silty SAND to Sandy SILT (TILL)



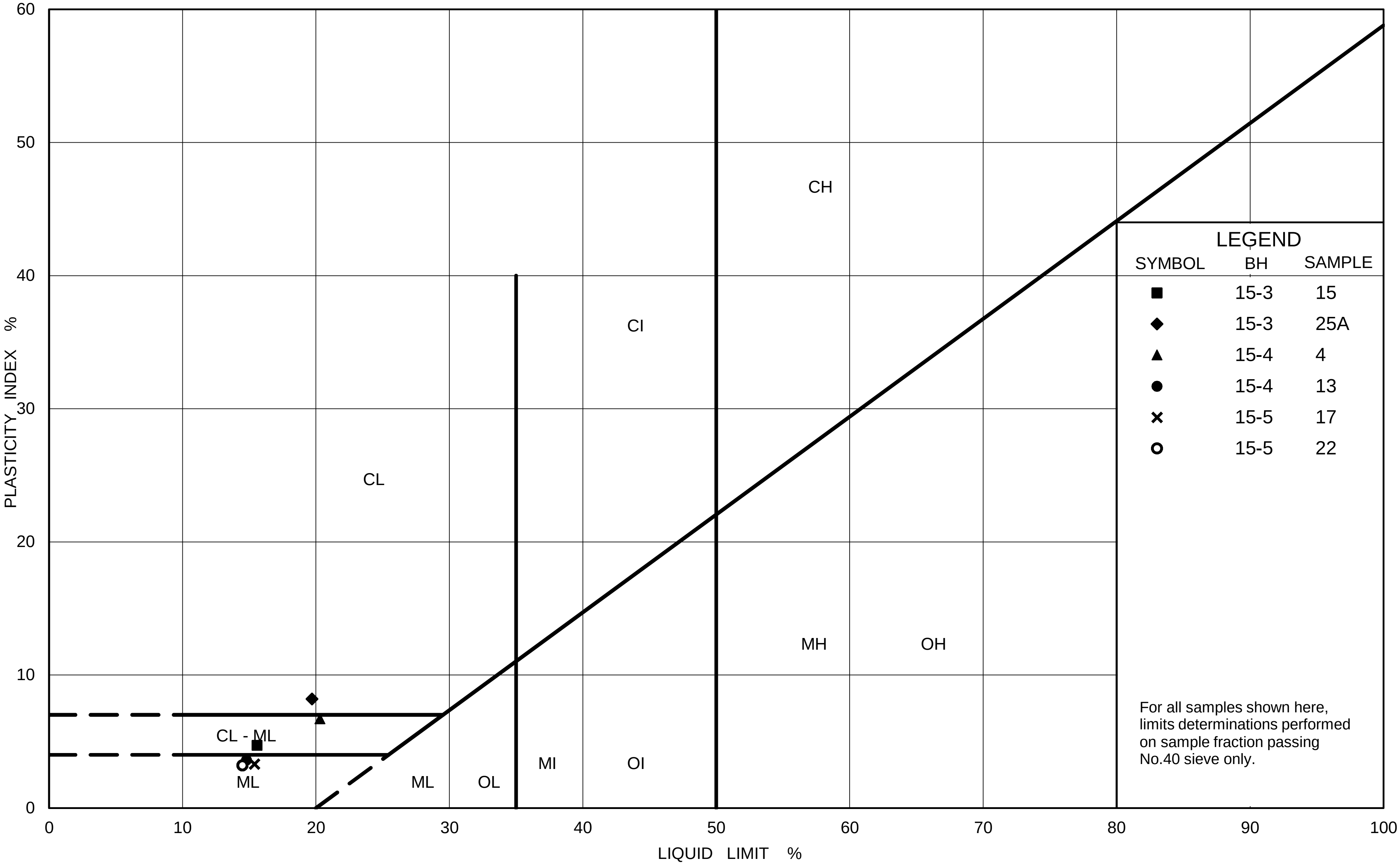
Borehole	Sample	Depth (m)
■ 15-4	9	6.86-7.47
◆ 15-4	13	9.91-10.52
▲ 15-4	18	16.77-17.38
● 15-5	14	10.67-11.28
□ 15-5	17	15.24-15.85
◇ 15-5	20	19.82-20.43
△ 15-5	22	22.87-23.48
○ 15-6	6	3.05-3.66
▨ 15-6	12	6.71-7.32
◈ 15-7	5	2.44-3.05
◭ 15-7	8	4.27-4.52
◍ 15-7	10	5.64-6.25

Created by: MI
Checked by: CNM

Project: 12-1121-0099/ 1750

Golder Associates

Oct 75, FF-S-21



For all samples shown here, limits determinations performed on sample fraction passing No.40 sieve only.

Ministry of Transportation

Ontario

PLASTICITY CHART
Silty SAND to Sandy SILT (TILL)

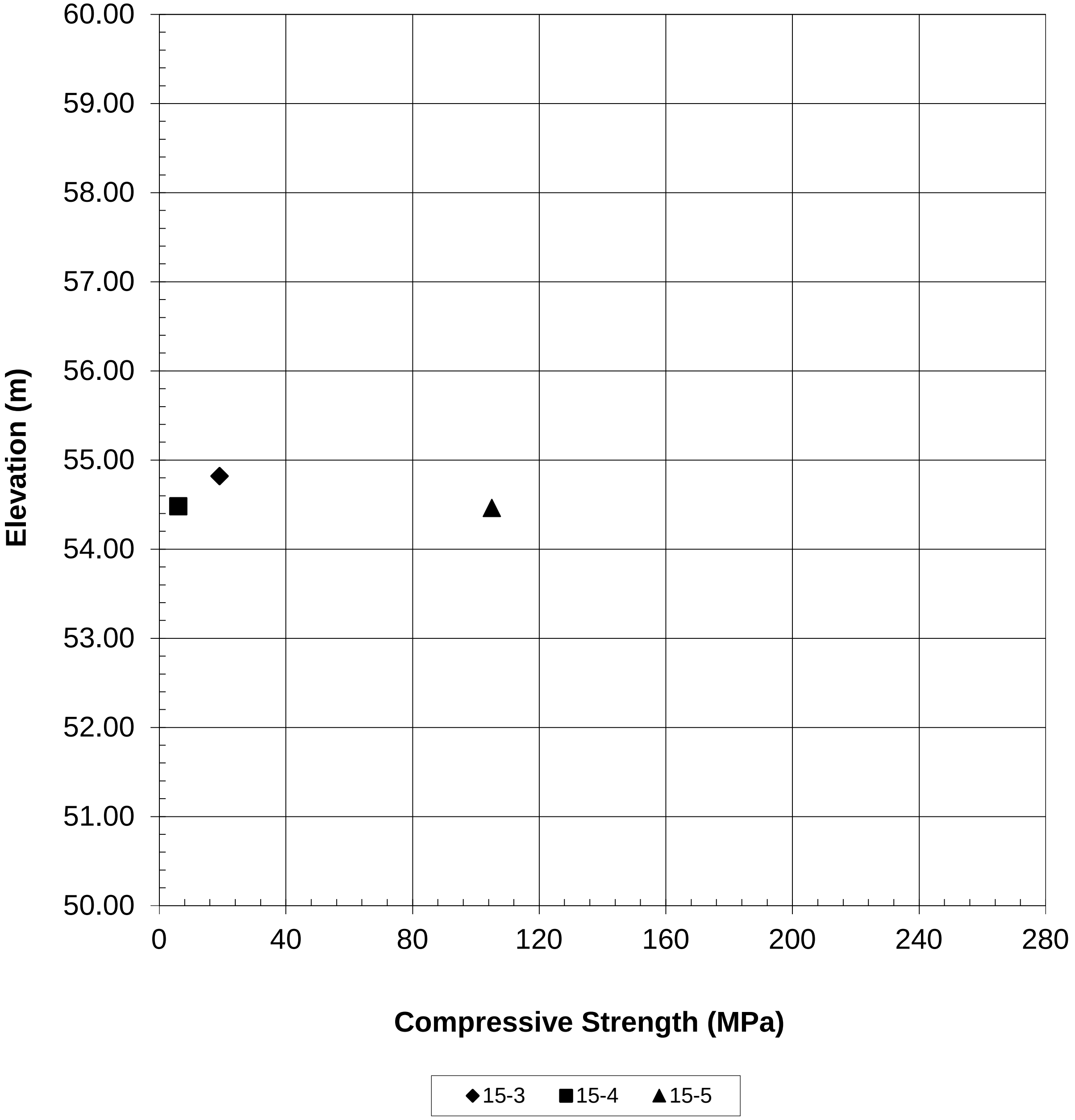
FIG No. B5

Project No. 12-1121-0099/1750

Compiled By : MI Checked By : CNM

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH POINT LOAD TESTING

FIGURE B6



Project: 12-1121-0099/ 1750

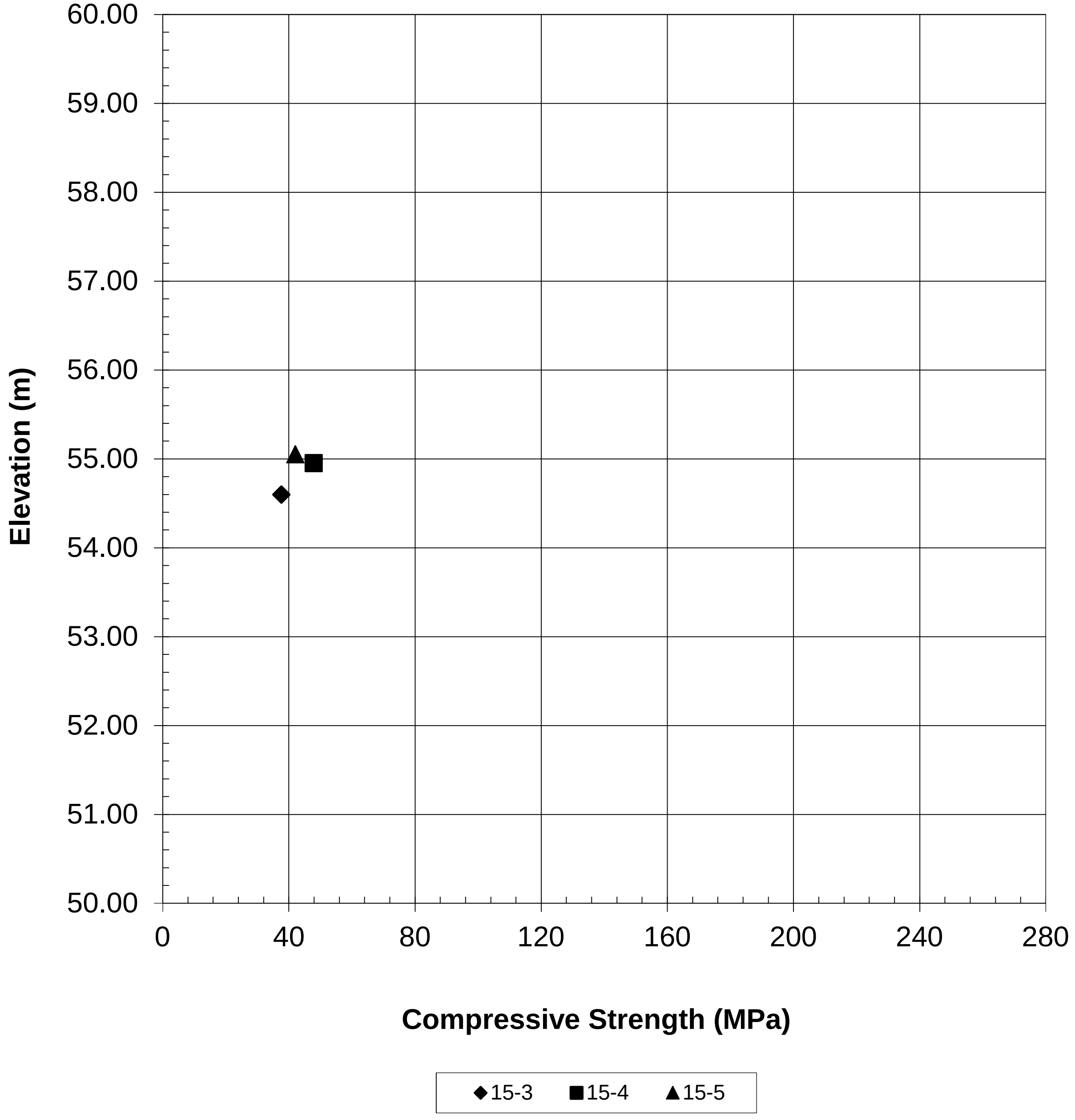
Golder Associates

Created by: SAT

Checked by: FJH

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH UNCONFINED COMPRESSION TESTS

FIGURE B7



Created by: SAT

Checked by: FJH

Project: 12-1121-0099/ 1750

Golder Associates

APPENDIX C

Seismic Cone Penetration Testing Report (ConeTec Investigations Ltd., 2016)

PRESENTATION OF SITE INVESTIGATION RESULTS

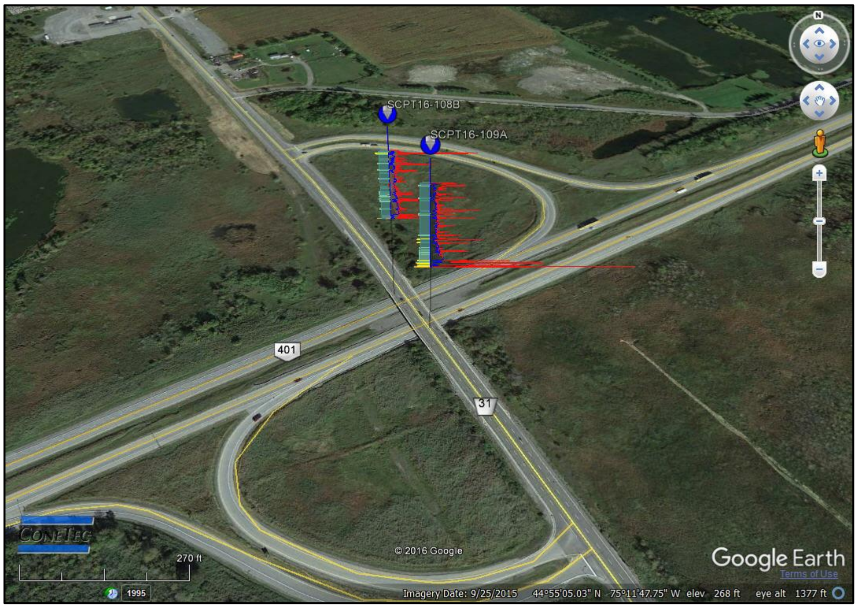
Highway 31 and Highway 401, ON

Prepared for:

Golder Associates Ltd.

ConeTec Job No: 16-05047

Project Start Date: 20-December-2016
Project End Date: 20-December-2016
Report Date: 23-December-2016



Prepared by:

ConeTec Investigations Ltd.
9033 Leslie Street, Unit 15
Richmond Hill, ON L4B 4K3

Tel: (905) 886-2663
Fax: (905) 886-2664
Toll Free: (800) 504-1116

Email: conetecon@conetec.com
www.conetec.com
www.conetecdataservices.com

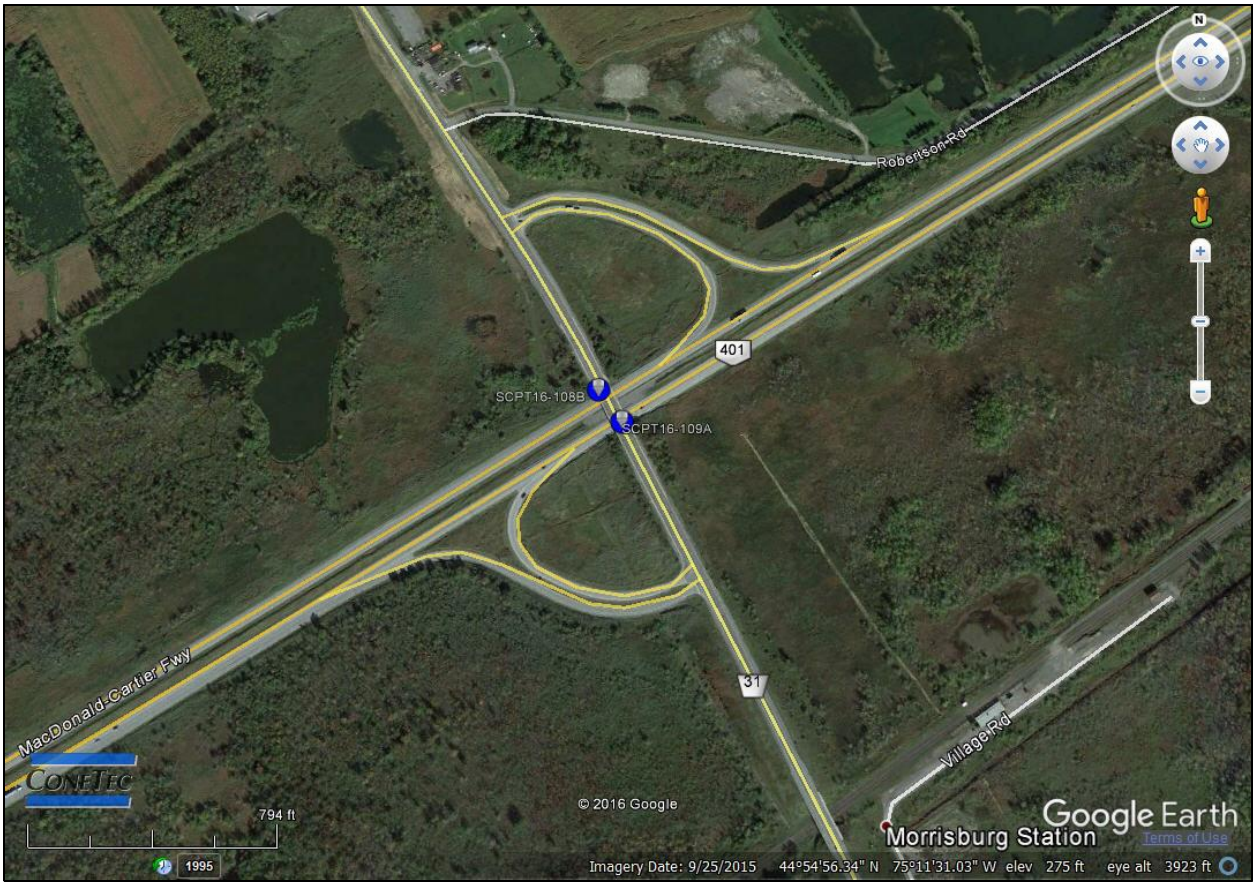
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates Ltd. at Highway 31 and 401 near Morrisburg, ON. The program consisted of 2 seismic cone penetration tests (SCPT).

Project Information

Project	
Client	Golder Associates Ltd.
Project	Highway 31 and Highway 401 near Morrisburg, ON
ConeTec project number	16-05047

A map from Google earth including the SCPT test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C3)	30 ton rig cylinder	SCPT

Coordinates		
Test Type	Collection Method	EPSG Number
SCPT	Consumer-grade GPS	32618

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Depth recording interval	5.0 cm
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Alternate range CPT plots, Seismic (Vs) plots, Advanced plots with undrained shear strength (Su-Nkt) and Overconsolidation Ratio (OCR) are included in the release folder.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
322:T1500F15U500	322	15	225	1500	15	500
Cone 322 was used for all the SCPT soundings.						

Interpretation Tables	
Additional information	The Soil Behaviour Type (SBT) classification chart (Robertson et al., 1986) was used to classify the soil for this project. A detailed set of CPT interpretations were generated and are provided in Excel format files in the release folder. The CPT interpretations are based on values of corrected tip (q_t), sleeve friction (f_s) and pore pressure.

Limitations

This report has been prepared for the exclusive use of Golder Associates Ltd. (Client) for the project titled “Highway 31 and Highway 401”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first Appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

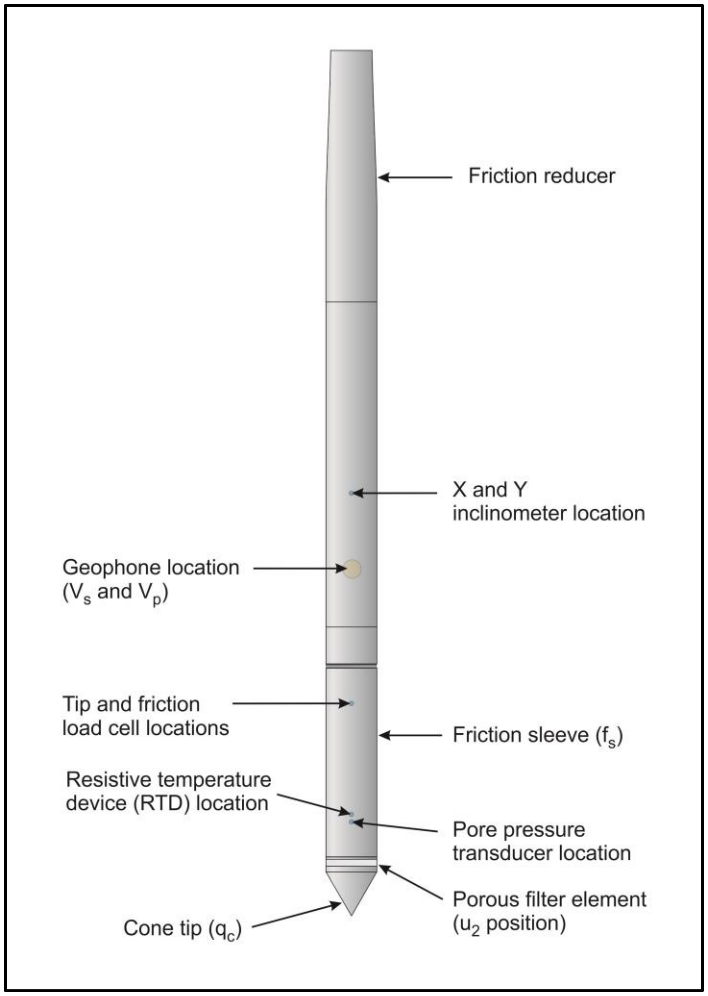


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerine under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is also included in the data release folder.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

Shear wave velocity testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave (Vp) velocity is also determined.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

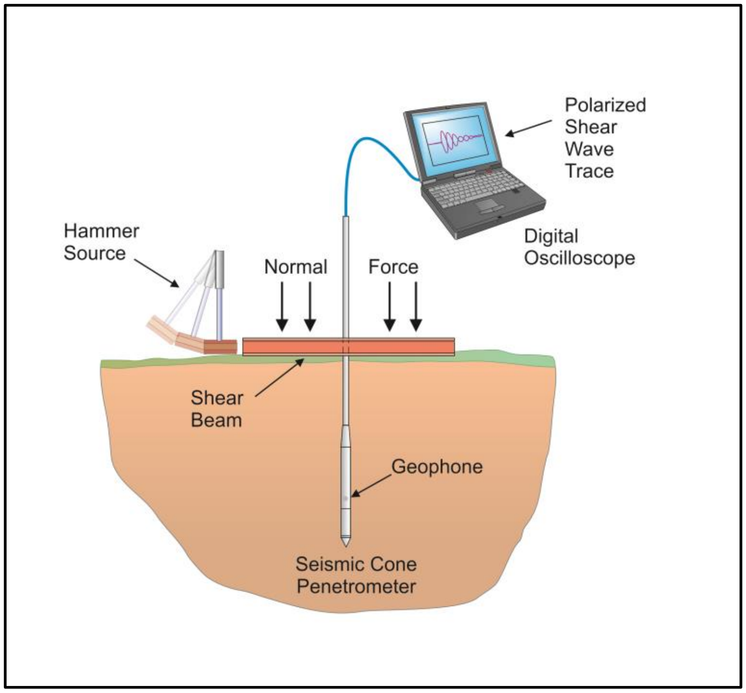


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Multiple wave traces are recorded for quality control purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et.al. (1986).

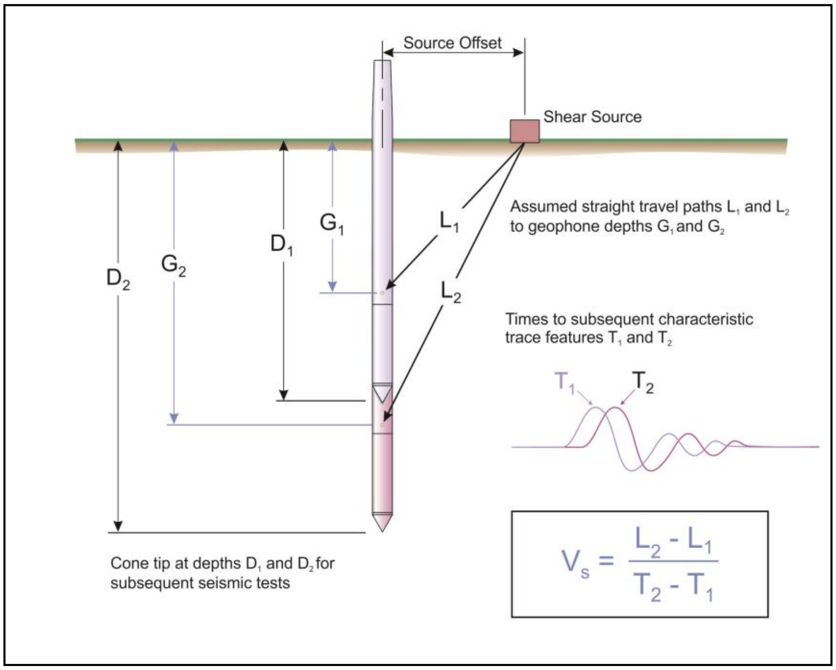


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 30 meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in Crow et al., 2012.

$$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ (30m)}{\sum(layer\ traveltimes)}$$

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

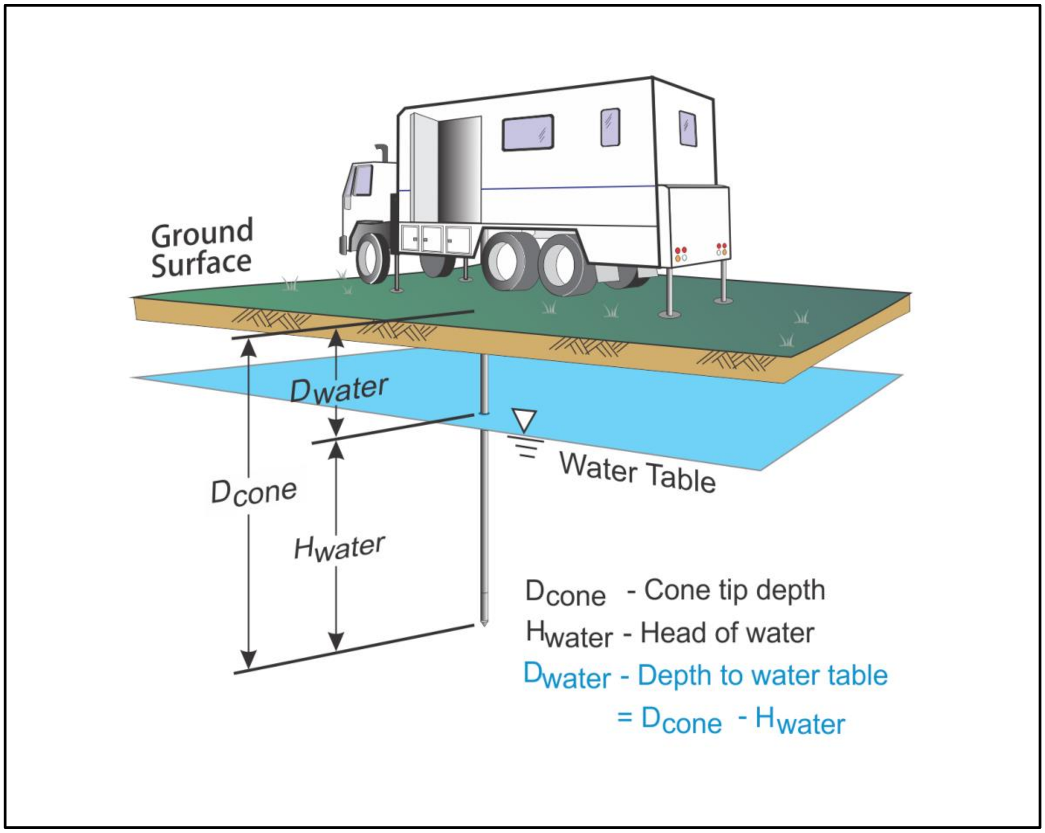


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

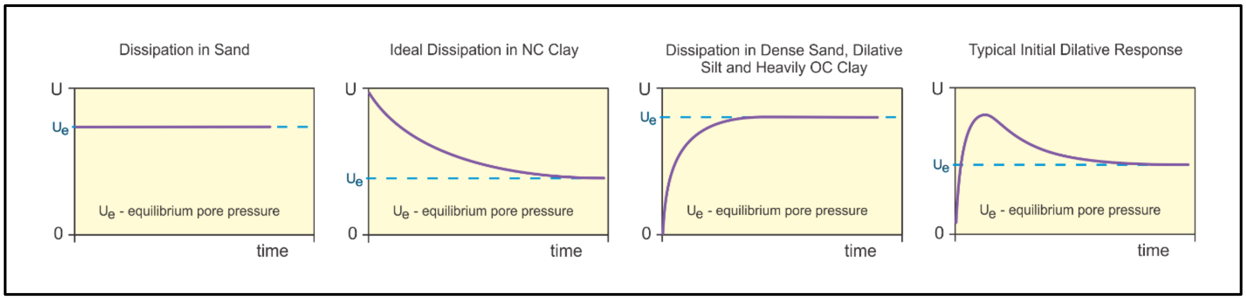


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T*	is the dimensionless time factor (Table Time Factor)
a	is the radius of the cone
I_r	is the rigidity index
t	is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

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Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Alternate Range Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with Undrained Shear Strength (Su-Nkt) and Overconsolidation Ratio (OCR)
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No: 16-05047
Client: Golder Associates Ltd.
Project: Highway 31 and Highway 401 near Morrisburg, ON
Start Date: 20-Dec-2016
End Date: 20-Dec-2016

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting (m)	Refer to Notation Number
SCPT16-108	16-05047_SP108	20-Dec-2016	322:T1500F15U500	0.9	21.80	4973768	484389	3
SCPT16-109A	16-05047_SP109A	20-Dec-2016	322:T1500F15U500	0.9	25.55	4973732	484413	

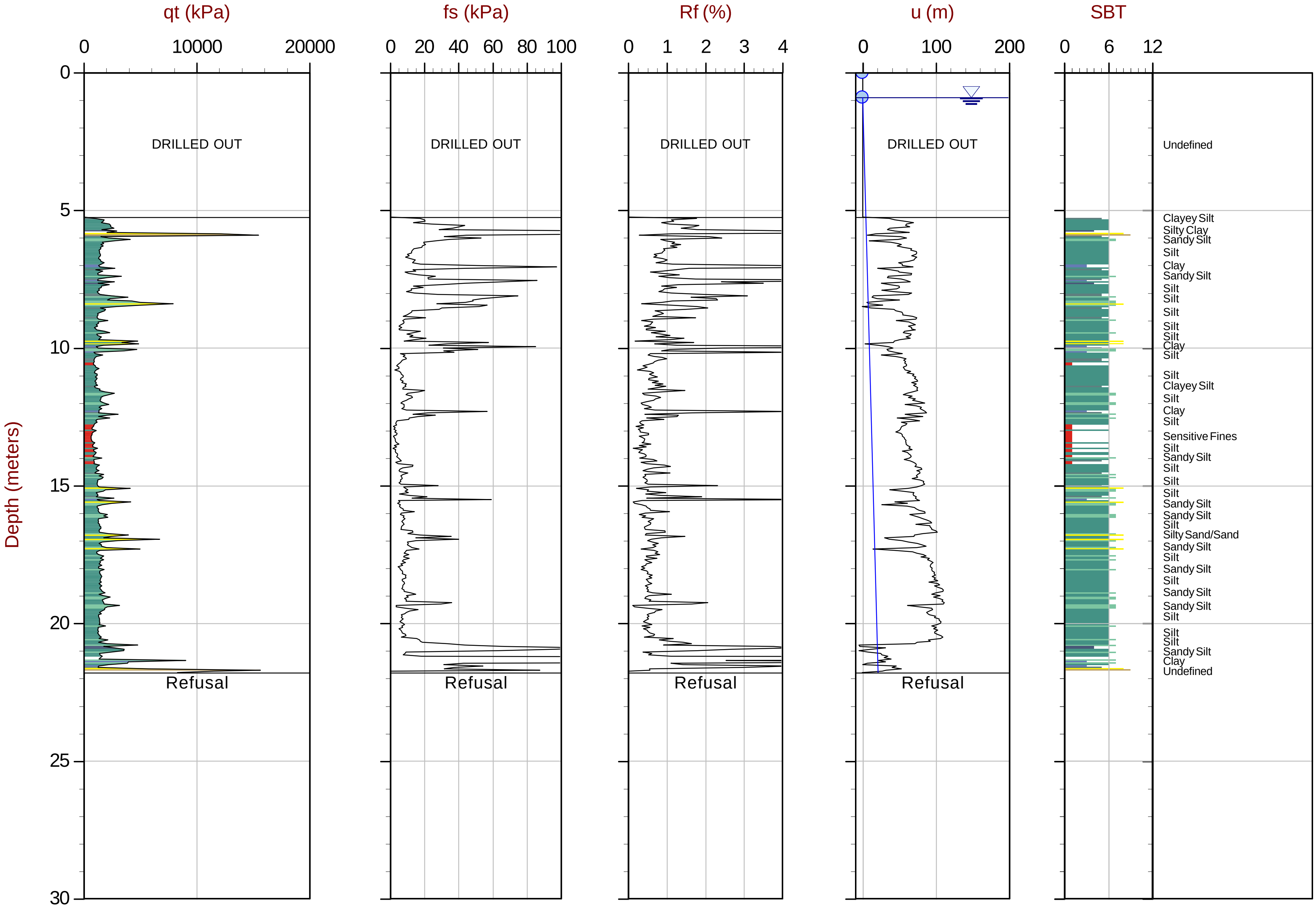
1. The assumed phreatic surface was based on pore pressure dissipation tests unless otherwise noted.
   Hydrostatic conditions were assumed for the calculated geotechnical parameters.
2. The coordinates were obtained using consumer-grade GPS device in datum: WGS84/UTM Zone 18 North.
3. The assumed phreatic surface was based on equilibrium achieved from nearby CPT sounding.

CONETEC

Golder Associates

Job No: 16-05047
Date: 12:20:16 13:33
Site: Hwy 31 and 401 Westbound

Sounding: SCPT16-108B
Cone: 322:T1500F15U500



Max Depth: 21.800 m / 71.52 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 16-05047_SP108B.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM Zone 18 N: 4973768m E: 484389m
Page No: 1 of 1

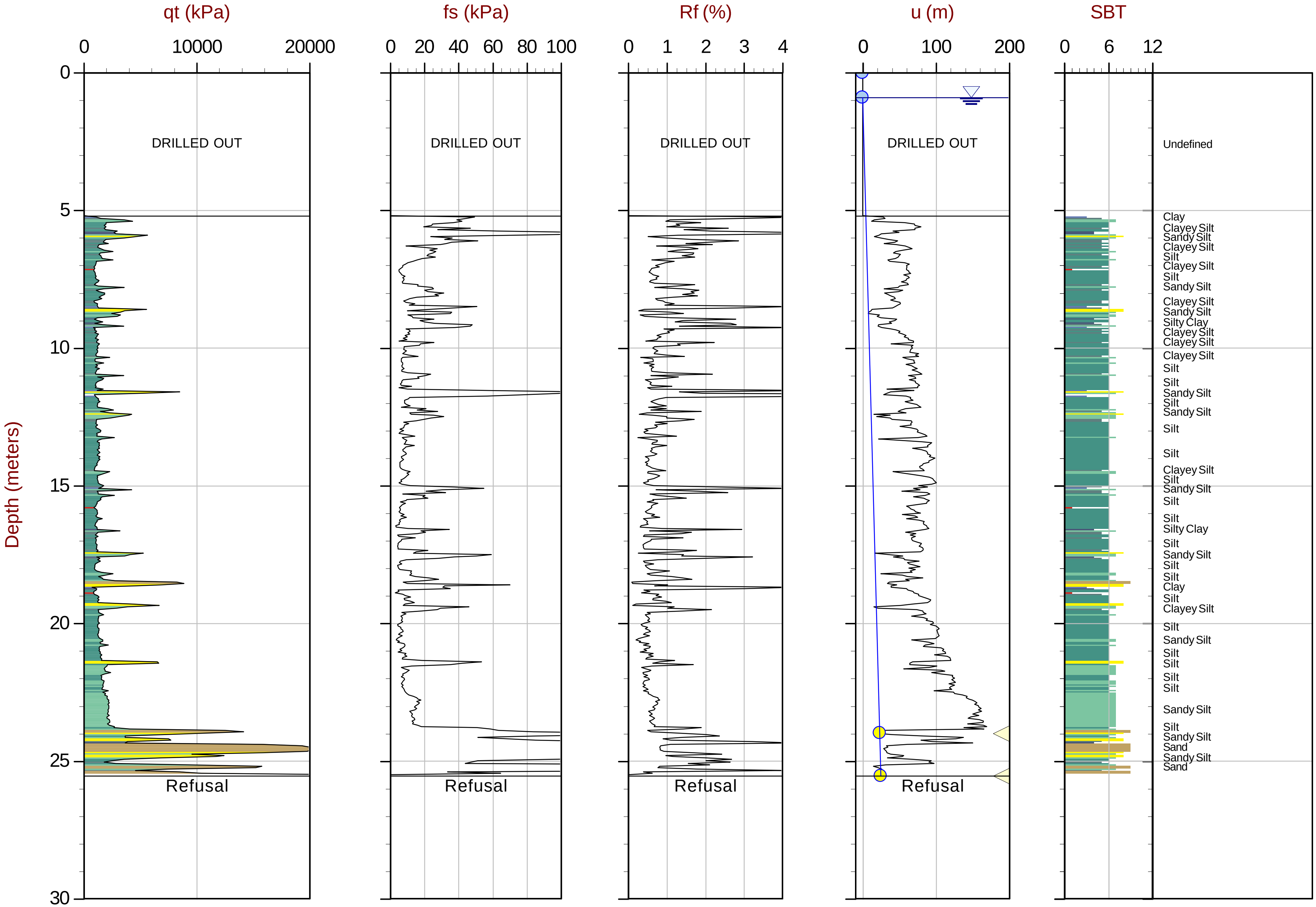
Overplot Item:
● Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line

CONETEC

Golder Associates

Job No: 16-05047
Date: 12:20:16 10:50
Site: Hwy 31 and 401 Eastbound

Sounding: SCPT16-109A
Cone: 322:T1500F15U500



Max Depth: 25.550 m / 83.82 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 16-05047_SP109A.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM Zone 18 N: 4973732m E: 484413m
Page No: 1 of 1

Overplot Item:
- ● Assumed Ueq
- ● Ueq
- ◁ Dissipation, equilibrium achieved
- ◁ Dissipation, equilibrium not achieved
- — Hydrostatic Line

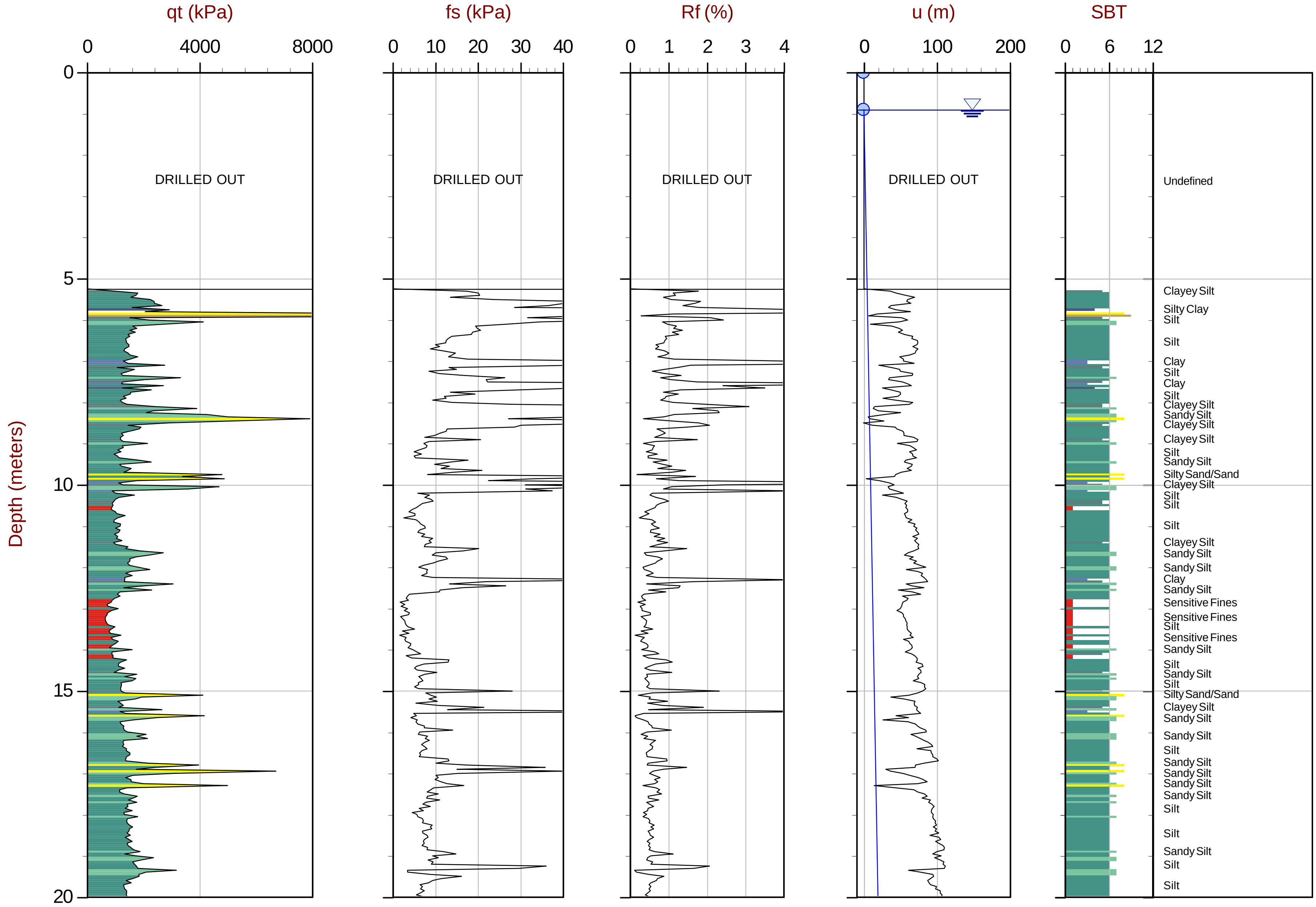
Alternate Range Cone Penetration Test Plots

CONETEC

Golder Associates

Job No: 16-05047
Date: 12:20:16 13:33
Site: Hwy 31 and 401 Westbound

Sounding: SCPT16-108B
Cone: 322:T1500F15U500



Max Depth: 21.800 m / 71.52 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 16-05047_SP108B.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM Zone 18 N: 4973768m E: 484389m
Page No: 1 of 2

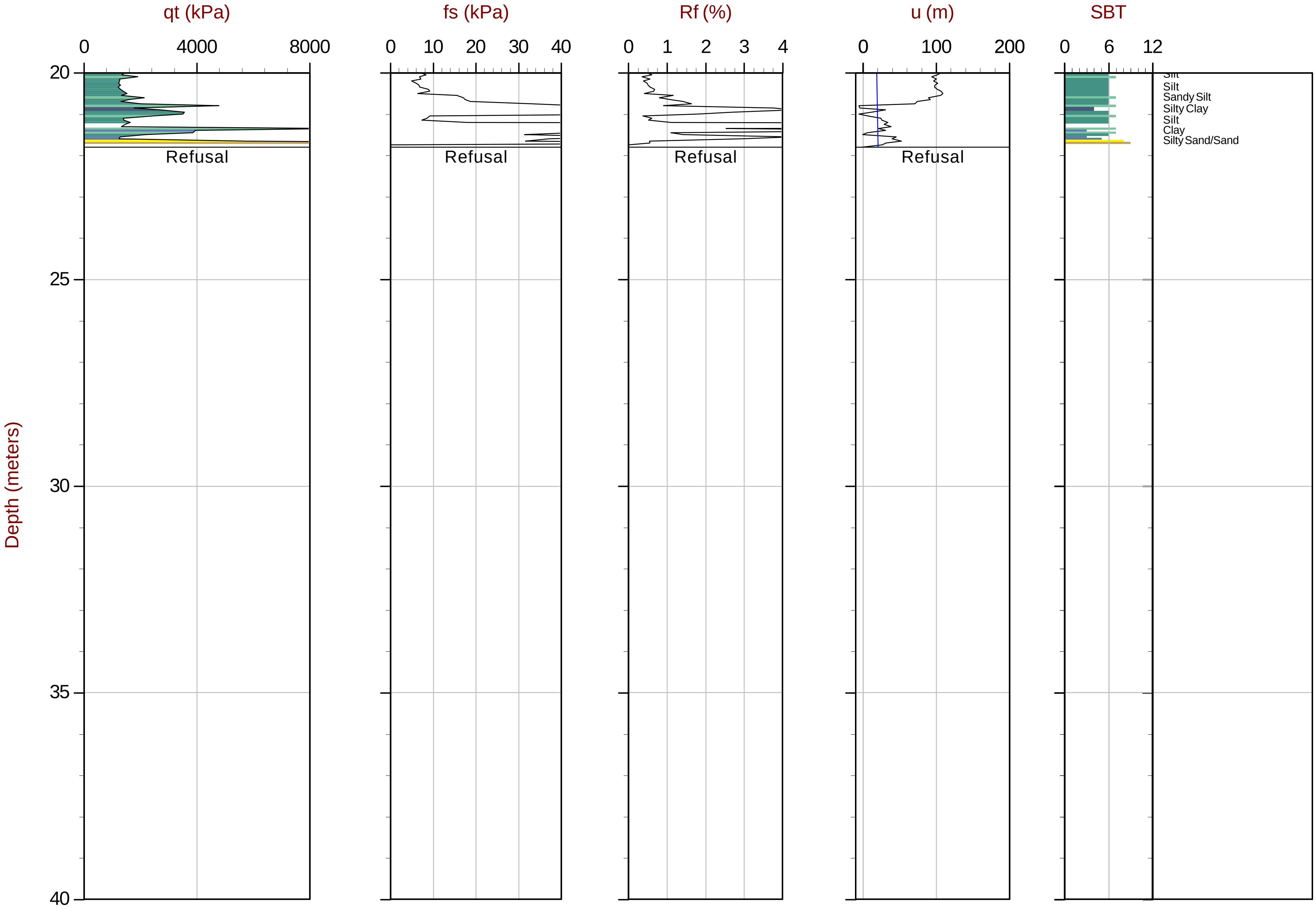
Overplot Item:
○ Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line

CONETEC

Golder Associates

Job No: 16-05047
Date: 12:20:16 13:33
Site: Hwy 31 and 401 Westbound

Sounding: SCPT16-108B
Cone: 322:T1500F15U500



Max Depth: 21.800 m / 71.52 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 16-05047_SP108B.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM Zone 18 N: 4973768m E: 484389m
Page No: 2 of 2

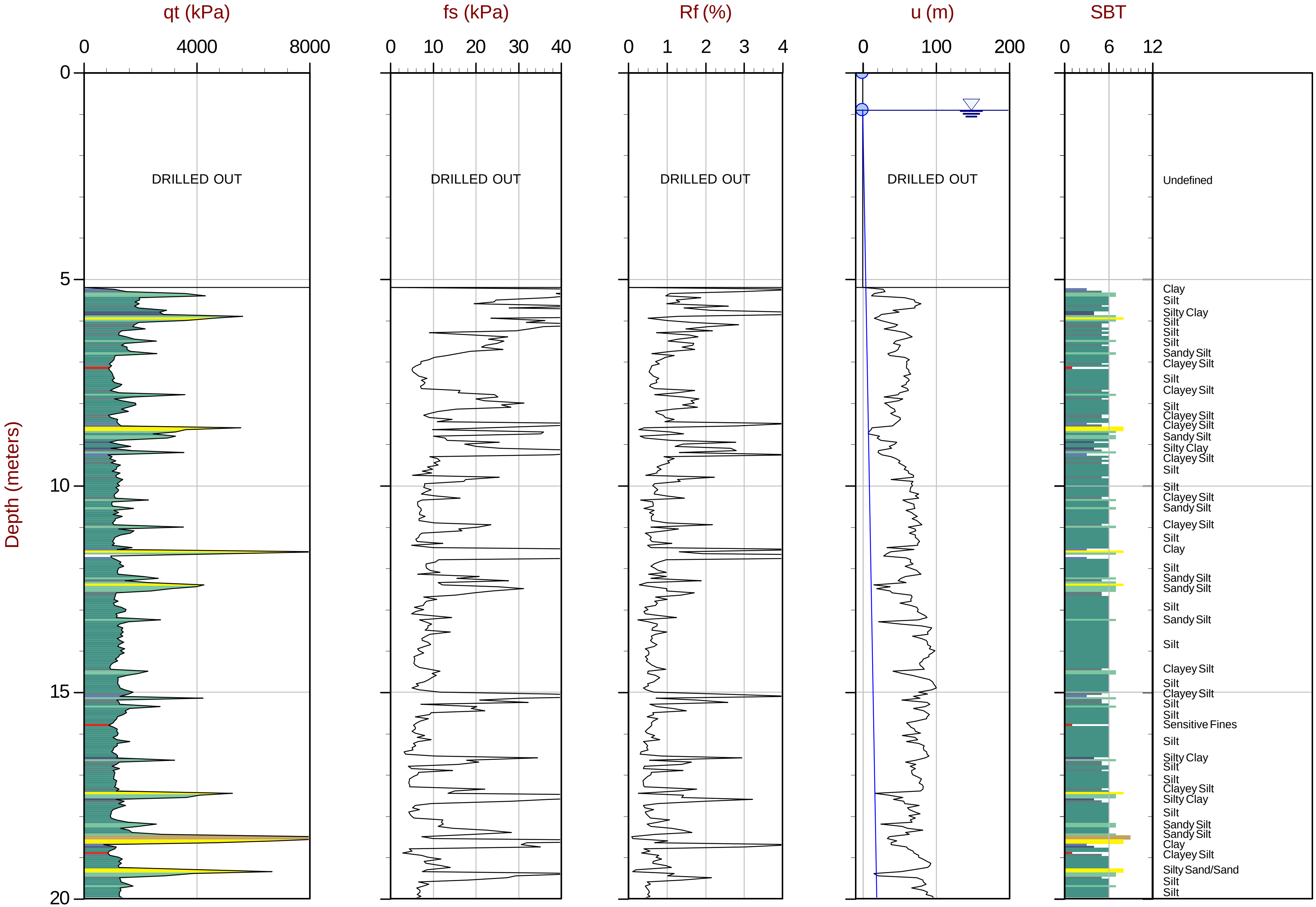
Overplot Item:
● Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line

CONETEC

Golder Associates

Job No: 16-05047
Date: 12:20:16 10:50
Site: Hwy 31 and 401 Eastbound

Sounding: SCPT16-109A
Cone: 322:T1500F15U500



Max Depth: 25.550 m / 83.82 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 16-05047_SP109A.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM Zone 18 N: 4973732m E: 484413m
Page No: 1 of 2

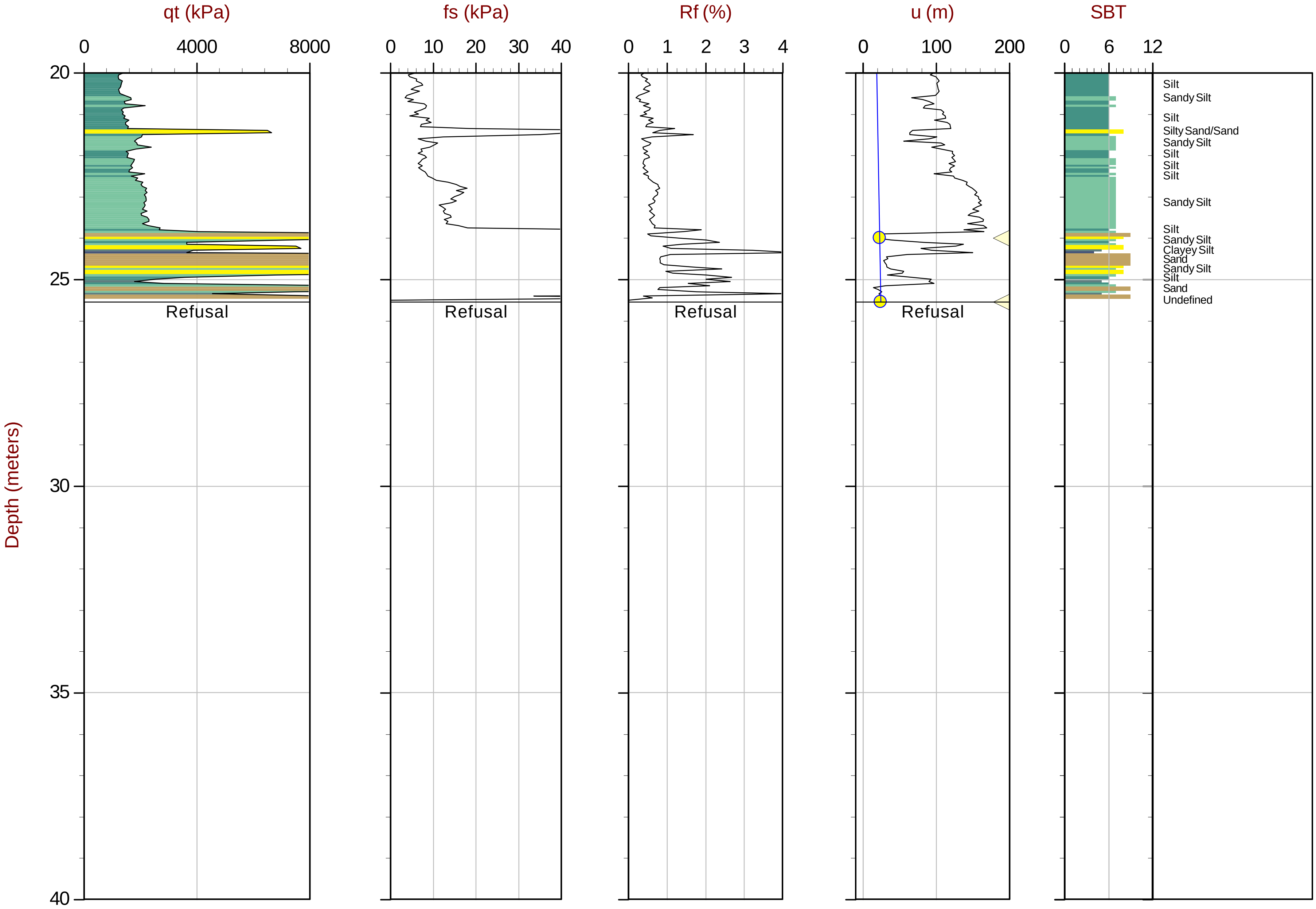
Overplot Item: ○ Assumed Ueq ● Ueq ◁ Dissipation, equilibrium achieved ◁ Dissipation, equilibrium not achieved — Hydrostatic Line

CONETEC

Golder Associates

Job No: 16-05047
Date: 12:20:16 10:50
Site: Hwy 31 and 401 Eastbound

Sounding: SCPT16-109A
Cone: 322:T1500F15U500



Max Depth: 25.550 m / 83.82 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 16-05047_SP109A.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM Zone 18 N: 4973732m E: 484413m
Page No: 2 of 2

Overplot Item:
○ Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line

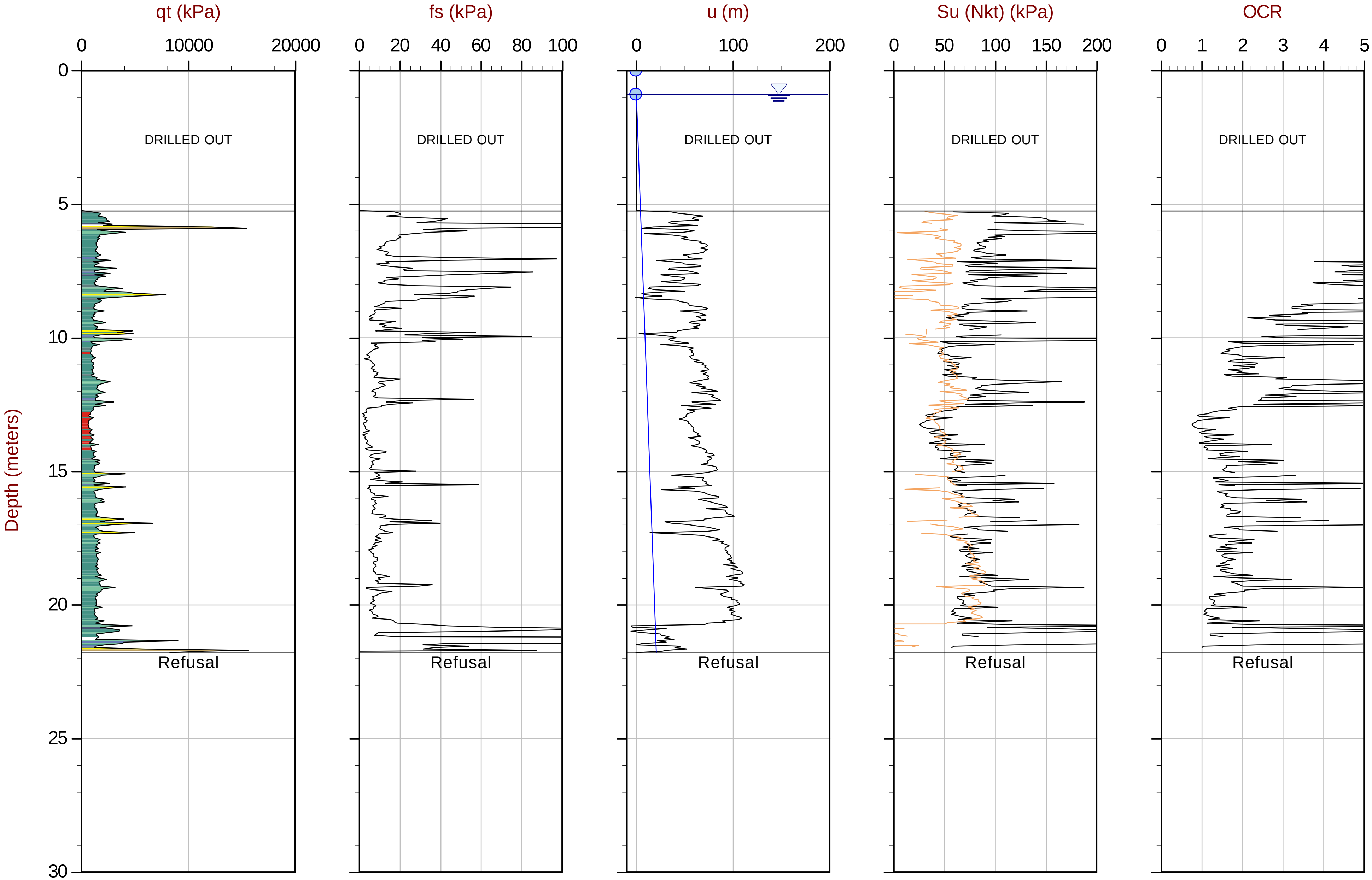
Advanced Cone Penetration Test Plots with Undrained Shear Strength (Su-Nkt) and Overconsolidation Ratio (OCR)

CONETEC

Golder Associates

Job No: 16-05047
Date: 12:20:16 13:33
Site: Hwy 31 and 401 Westbound

Sounding: SCPT16-108B
Cone: 322:T1500F15U500



Max Depth: 21.800 m / 71.52 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 16-05047_SP108B.COR
Unit Wt: SBT Zones
Su Nkt/Ndu: 15.0 / 10.0

SBT: Robertson and Campanella, 1986
Coords: UTM Zone 18 N: 4973768m E: 484389m
Page No: 1 of 1

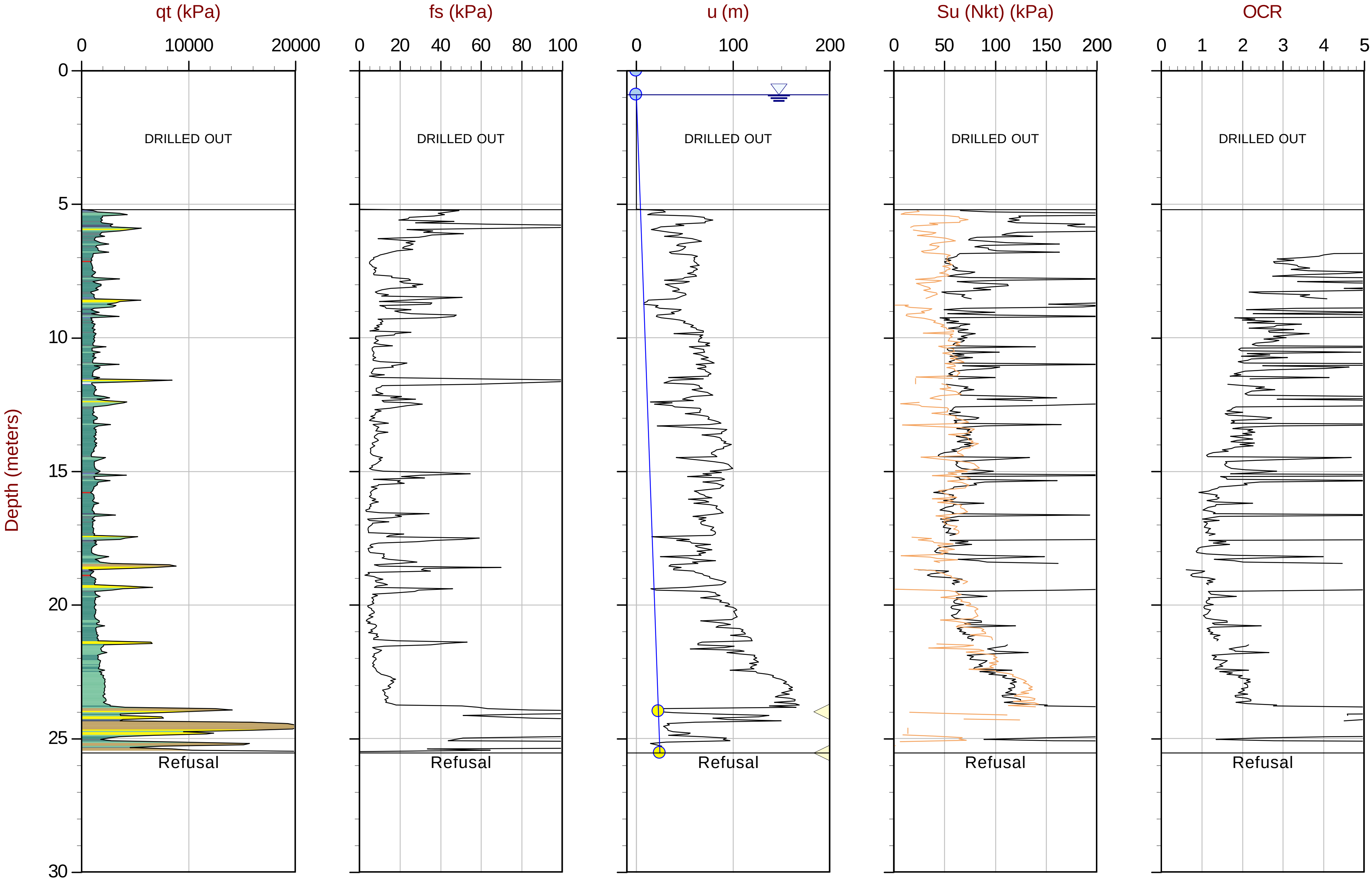
Overplot Item:
● Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line

CONETEC

Golder Associates

Job No: 16-05047
Date: 12:20:16 10:50
Site: Hwy 31 and 401 Eastbound

Sounding: SCPT16-109A
Cone: 322:T1500F15U500



Max Depth: 25.550 m / 83.82 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 16-05047_SP109A.COR
Unit Wt: SBT Zones
Su Nkt/Ndu: 15.0 / 10.0

SBT: Robertson and Campanella, 1986
Coords: UTM Zone 18 N: 4973732m E: 484413m
Page No: 1 of 1

Overplot Item:
- ● Assumed Ueq
- ● Ueq
- ◁ Dissipation, equilibrium achieved
- ◁ Dissipation, equilibrium not achieved
- — Hydrostatic Line

Seismic Cone Penetration Test Plots

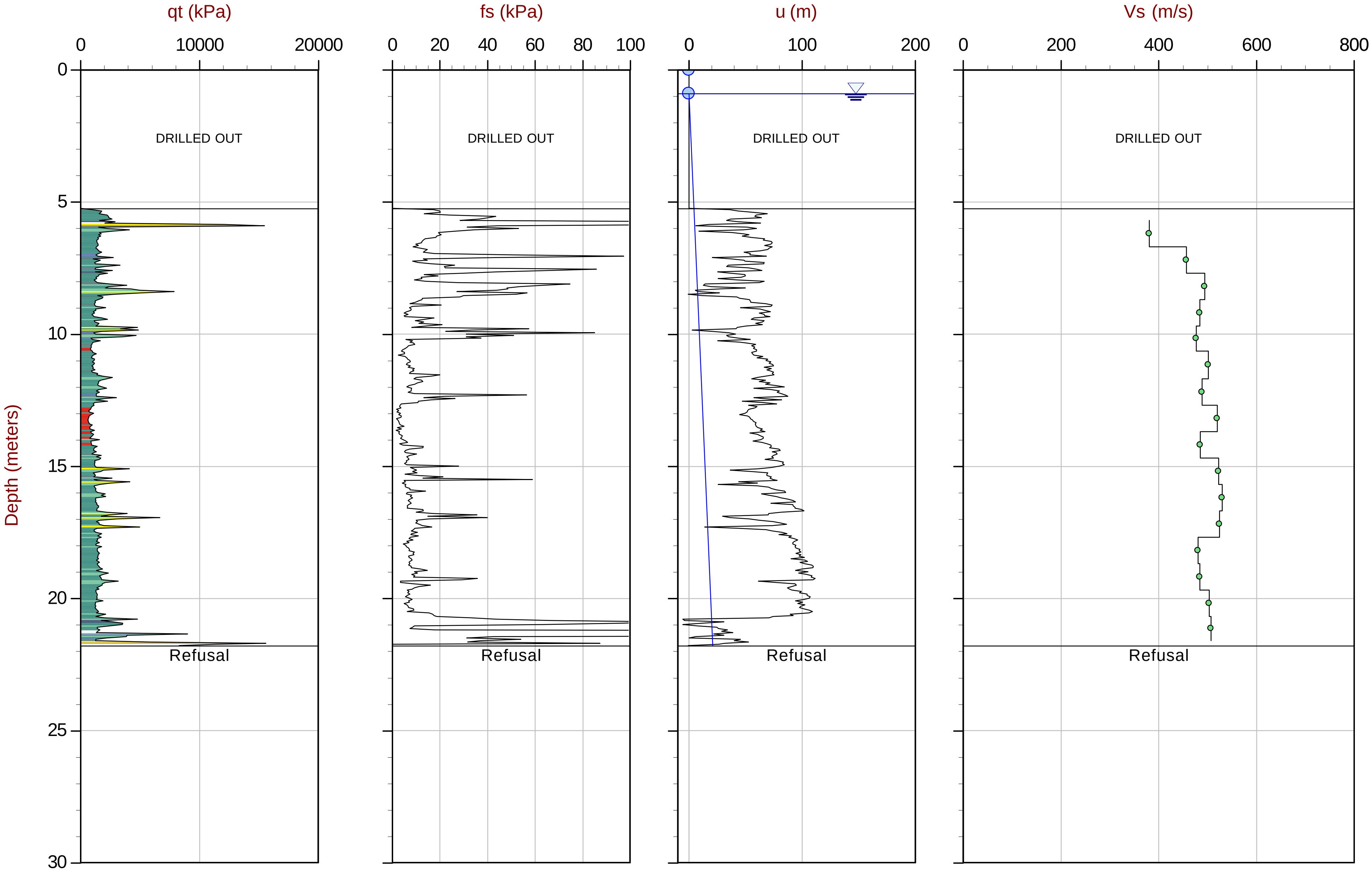
ConeTec

CONETEC

Golder Associates

Job No: 16-05047
Date: 12:20:16 13:33
Site: Hwy 31 and 401 Westbound

Sounding: SCPT16-108B
Cone: 322:T1500F15U500



Max Depth: 21.800 m / 71.52 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 16-05047_SP108B.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM Zone 18 N: 4973768m E: 484389m
Page No: 1 of 1

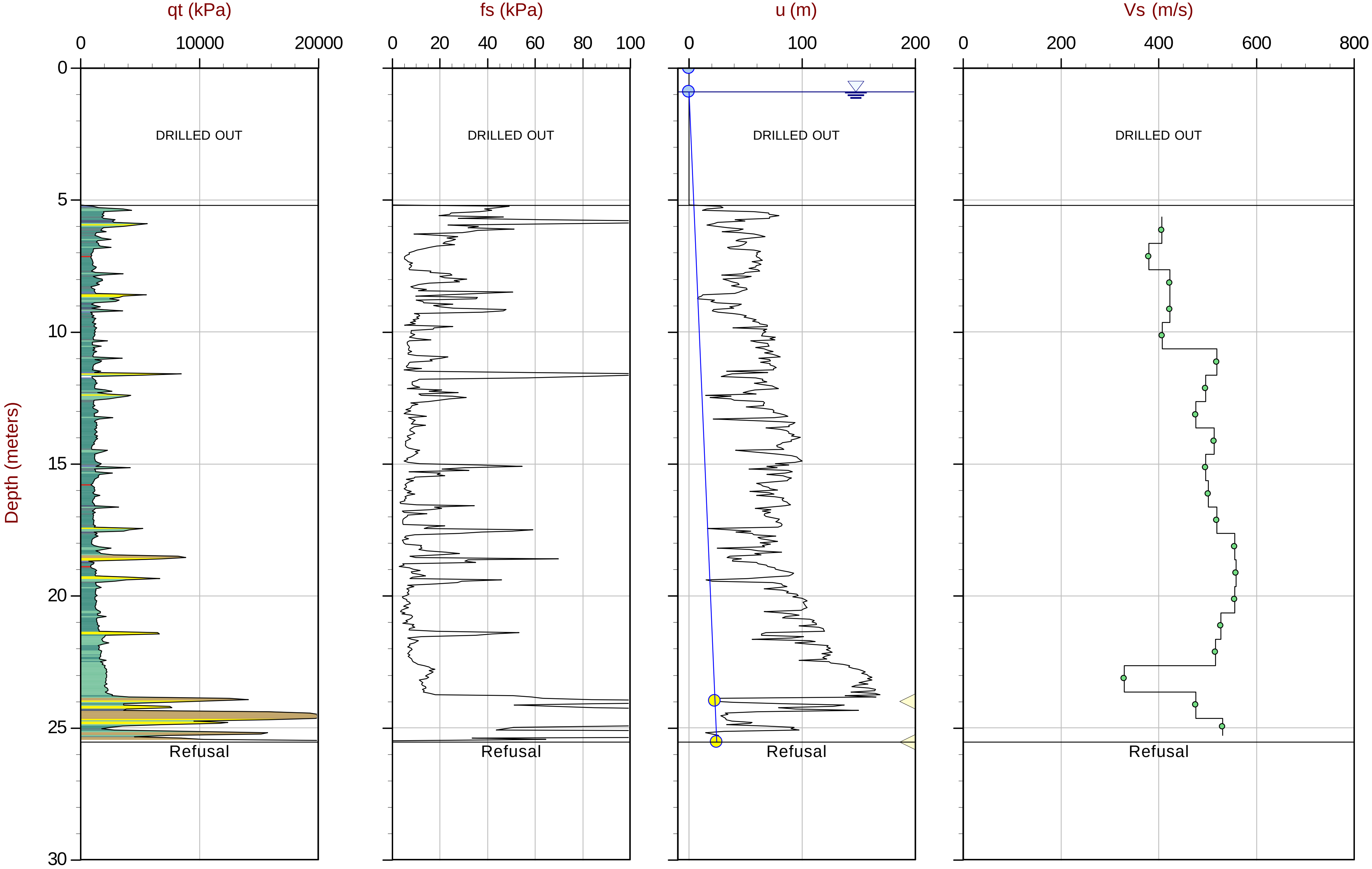
Overplot Item:
● Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line

CONETEC

Golder Associates

Job No: 16-05047
Date: 12:20:16 10:50
Site: Hwy 31 and 401 Eastbound

Sounding: SCPT16-109A
Cone: 322:T1500F15U500



Max Depth: 25.550 m / 83.82 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 16-05047_SP109A.COR
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986
Coords: UTM Zone 18 N: 4973732m E: 484413m
Page No: 1 of 1

Overplot Item:
● Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line

Seismic Cone Penetration Test Tabular Results

Job No: 16-05047
Client: Golder Associates Ltd.
Project: Highway 31 and Highway 401
Sounding ID: SCPT16-108B
Date: 20-Dec-2016

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
5.90	5.70	5.73			
6.90	6.70	6.72	1.00	2.61	381
7.90	7.70	7.72	1.00	2.18	457
8.90	8.70	8.72	1.00	2.02	495
9.90	9.70	9.72	1.00	2.06	485
10.85	10.65	10.66	0.95	1.99	477
11.90	11.70	11.71	1.05	2.09	502
12.90	12.70	12.71	1.00	2.04	489
13.90	13.70	13.71	1.00	1.92	520
14.90	14.70	14.71	1.00	2.06	486
15.90	15.70	15.71	1.00	1.91	523
16.90	16.70	16.71	1.00	1.89	530
17.90	17.70	17.71	1.00	1.90	525
18.90	18.70	18.71	1.00	2.08	481
19.90	19.70	19.71	1.00	2.06	485
20.90	20.70	20.71	1.00	1.98	504
21.80	21.60	21.61	0.90	1.77	508

CONETEC

Job No: 16-05047
Client: Golder Associates Ltd.
Project: Highway 31 and Highway 401
Sounding ID: SCPT16-109A
Date: 20-Dec-2016

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

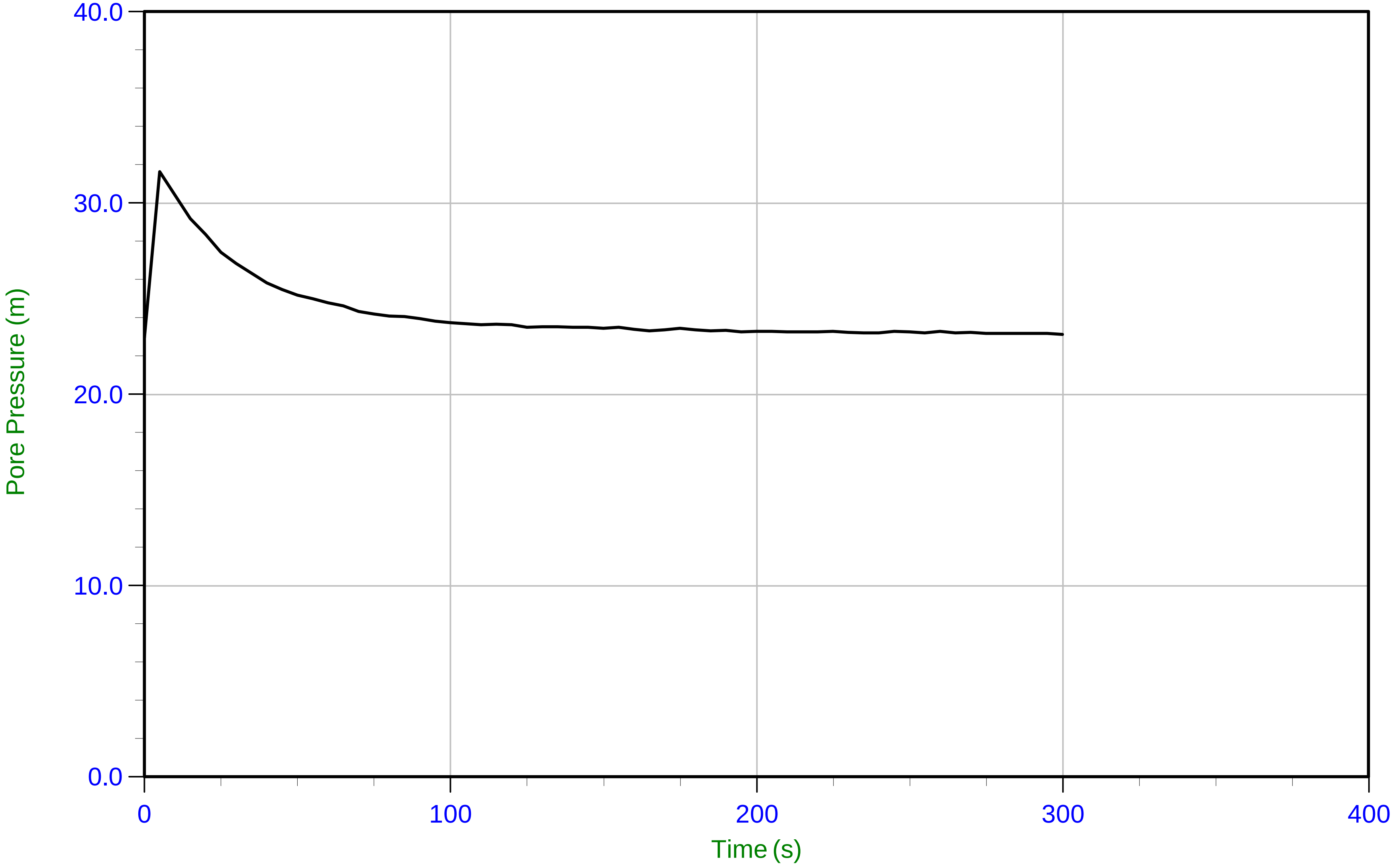
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
5.85	5.65	5.68			
6.85	6.65	6.67	1.00	2.45	407
7.85	7.65	7.67	1.00	2.62	380
8.85	8.65	8.67	1.00	2.36	423
9.85	9.65	9.67	1.00	2.36	423
10.85	10.65	10.66	1.00	2.45	408
11.85	11.65	11.66	1.00	1.92	519
12.85	12.65	12.66	1.00	2.01	497
13.85	13.65	13.66	1.00	2.10	476
14.85	14.65	14.66	1.00	1.94	514
15.85	15.65	15.66	1.00	2.01	497
16.85	16.65	16.66	1.00	1.99	502
17.85	17.65	17.66	1.00	1.92	519
18.85	18.65	18.66	1.00	1.80	556
19.85	19.65	19.66	1.00	1.79	559
20.85	20.65	20.66	1.00	1.80	556
21.85	21.65	21.66	1.00	1.89	528
22.85	22.65	22.66	1.00	1.93	517
23.85	23.65	23.66	1.00	3.03	330
24.85	24.65	24.66	1.00	2.10	476
25.50	25.30	25.31	0.65	1.22	531

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No: 16-05047
Client: Golder Associates Ltd.
Project: Highway 31 and Highway 401 near Morrisburg, ON
Start Date: 20-Dec-2016
End Date: 20-Dec-2016

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)
SCPT16-109A	16-05047_SP109A	15	300	24.00	23.1	0.9
SCPT16-109A	16-05047_SP109A	15	300	25.55	24.5	1.0



Trace Summary:

Filename: 16-05047_SP109A.PPF
Depth: 24.000 m / 78.739 ft
Duration: 300.0 s

U Min: 22.9 m
U Max: 31.6 m

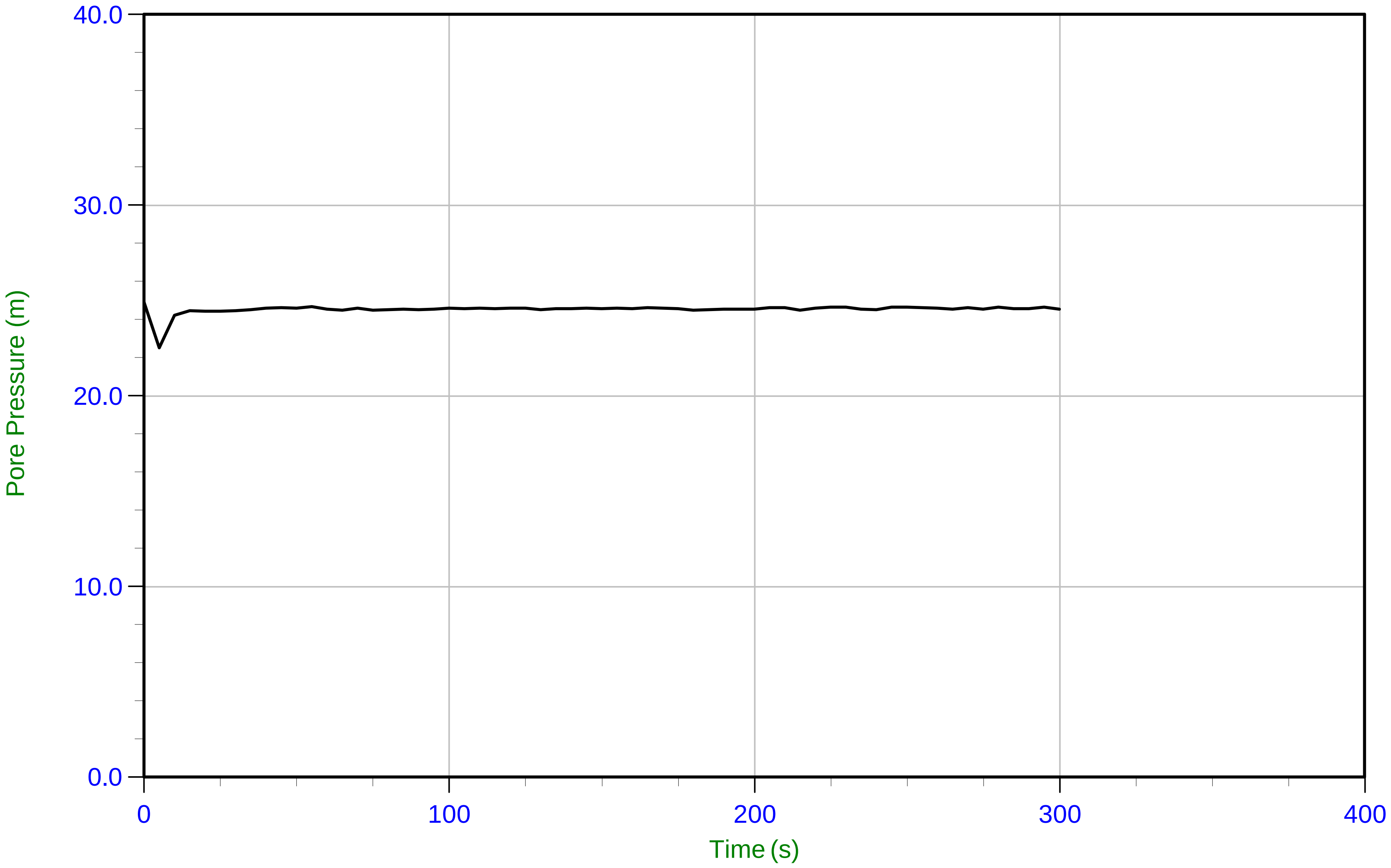
WT: 0.904 m / 2.966 ft
Ueq: 23.1 m

CONETEC

Golder Associates

Job No: 16-05047
Date: 12/20/2016 10:50
Site: Hwy 31 and 401 Eastbound

Sounding: SCPT16-109A
Cone: 322:T1500F15U500
Cone Area: 15 sq cm



Trace Summary:

Filename: 16-05047_SP109A.PPF
Depth: 25.550 m / 83.824 ft
Duration: 300.0 s

U Min: 22.5 m
U Max: 24.9 m

WT: 1.031 m / 3.383 ft
Ueq: 24.5 m

APPENDIX D

Seismic Cone Penetration Testing Report
(ConeTec Investigations Ltd., 2017)

PRESENTATION OF SITE INVESTIGATION RESULTS

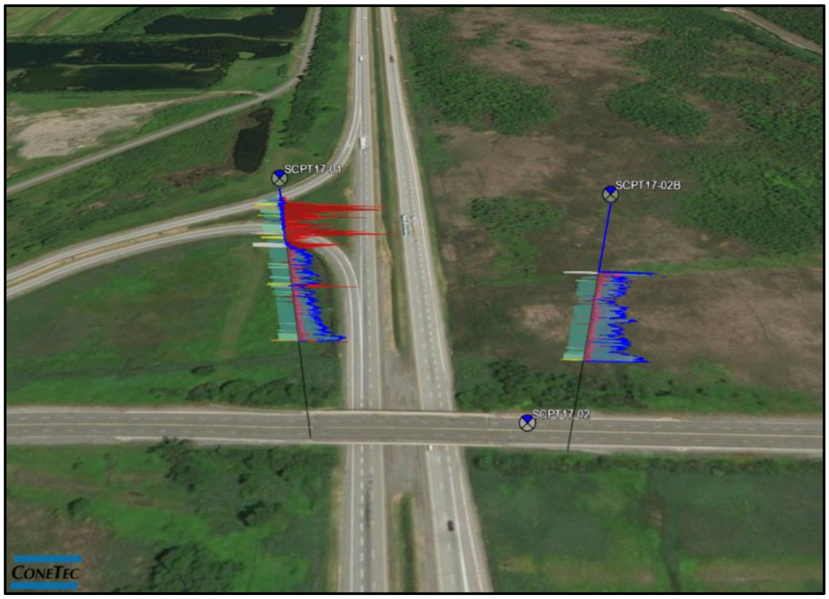
Hwy 31 and 401

Prepared for:

Thurber Engineering Ltd.

ConeTec Job No: 17-05022

Project Start Date: 24-May-2017
Project End Date: 24-May-2017
Report Date: 26-May-2017



Prepared by:

ConeTec Investigations Ltd.
9033 Leslie Street, Unit 15
Richmond Hill, ON L4B 4K3

Tel: (905) 886-2663
Fax: (905) 886-2664
Toll Free: (800) 504-1116

Email: conetecON@conetec.com
www.conetec.com
www.conetecdataservices.com

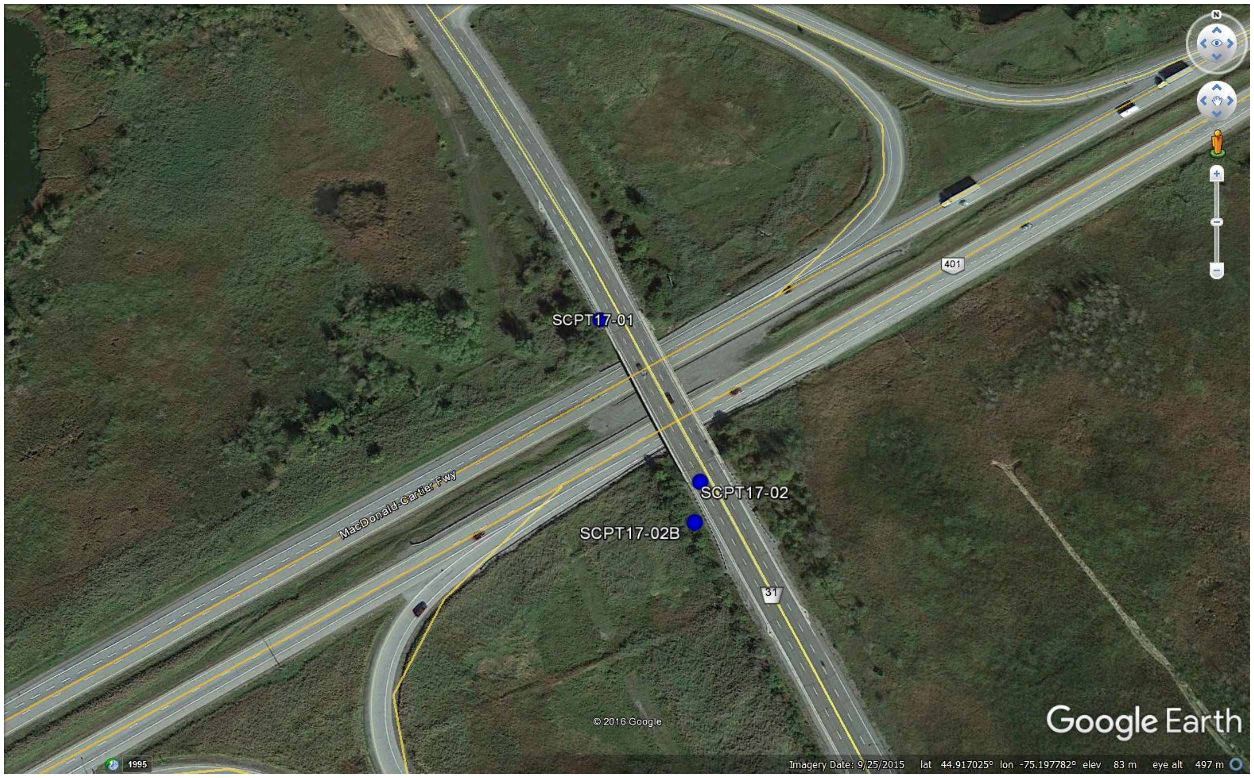
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Thurber Engineering Ltd. at Highway 31 and 401 near Morrisburg, ON . The program consisted of three seismic cone penetration tests (SCPT).

Project Information

Project	
Client	Thurber Engineering Ltd.
Project	Hwy 31 and 401
ConeTec project number	17-05022

A map from Google earth including the SCPT test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C3)	30 ton rig cylinder	SCPT

Coordinates			
Test Type	Collection Method	EPSG Number	Comments
SCPT	Consumer grade GPS	32618	Elevations were provided by the client.

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Depth recording interval	2.5 cm
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Alternate range, seismic and advanced CPT plots are included in the data release package.
Additional comments	Soundings SCPT17-02 and SCPT17-02B had the pore pressure filter in the u1 position.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
379:T1500F15U500	379	15	225	1500	15	500
Cone 379 was used for all CPT soundings.						

Calculated Parameters	
Additional information	The Soil Behaviour Type (SBT) classification chart (Robertson et al., 1986) was used to classify the soil for this project. A detailed set of calculated CPT parameters were generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip (q_t), sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behavior type zones and the assumed equilibrium pore pressure profile. Corrected tip values for soundings SCPT17-02 and SCPT17-02B will likely be a little higher than if the pore pressure were measured at the u2 location behind the tip. Soils were classified as either drained or undrained based on the Soil Behaviour Type (SBT) classification chart (Robertson et al., 1986). Calculations for both drained and undrained parameters were included for materials that classified as sandy silt (zone 7). Undrained parameters were included for materials that classified as undefined (zone 0).

Limitations

This report has been prepared for the exclusive use of Thurber Engineering Ltd. (Client) for the project titled “Hwy 31 and 401”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first Appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

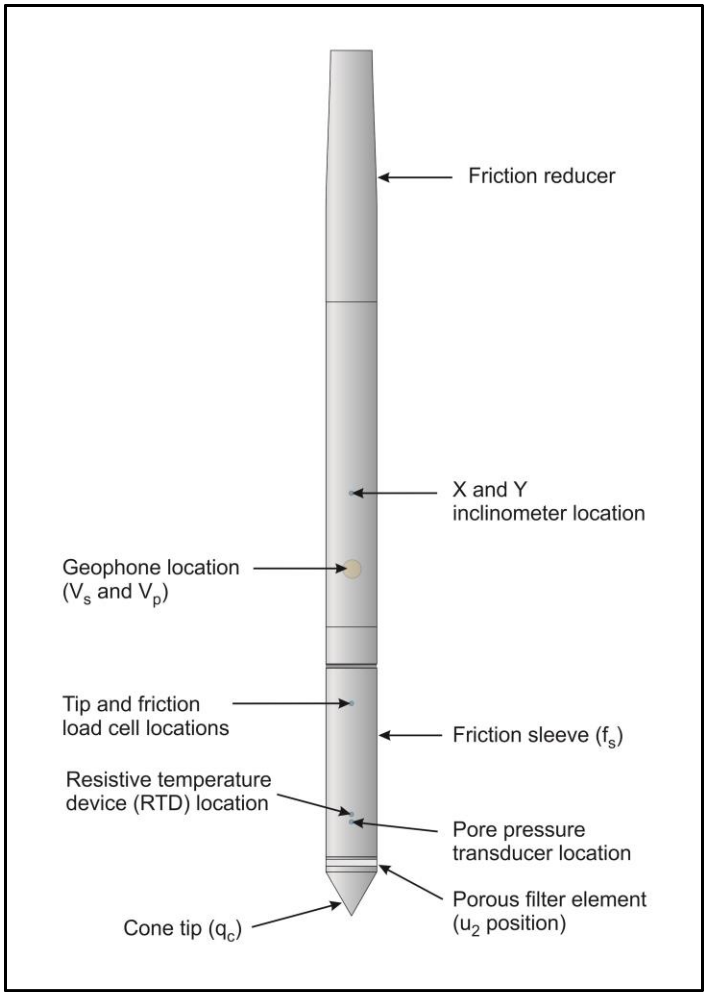


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerine under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is also included in the data release folder.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

Shear wave velocity testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave (Vp) velocity is also determined.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

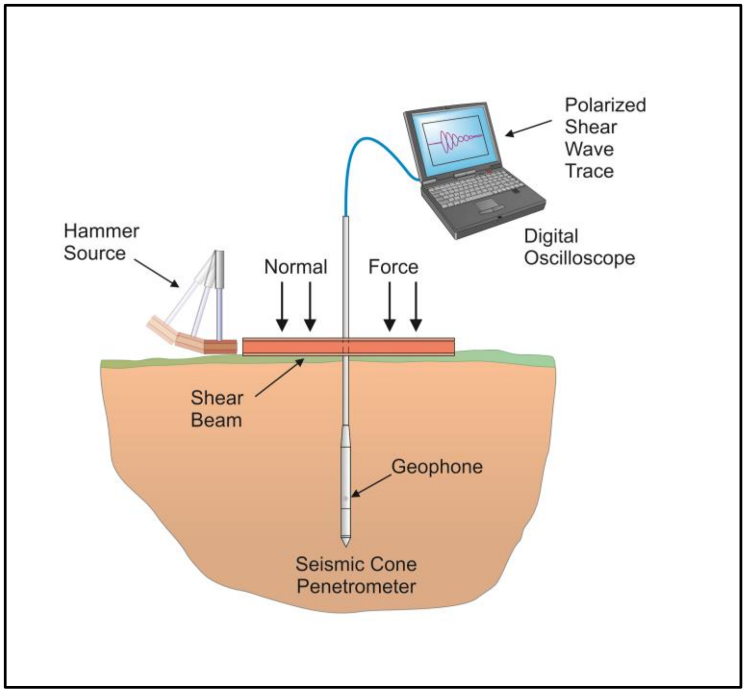


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Multiple wave traces are recorded for quality control purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et.al. (1986).

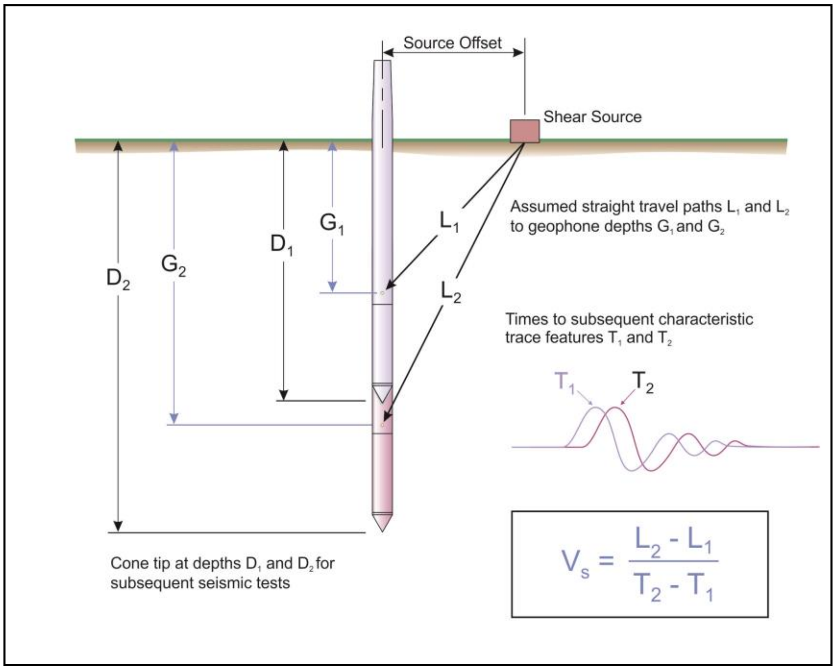


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 30 meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in Crow et al., 2012.

$$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ (30m)}{\sum(layer\ traveltimes)}$$

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

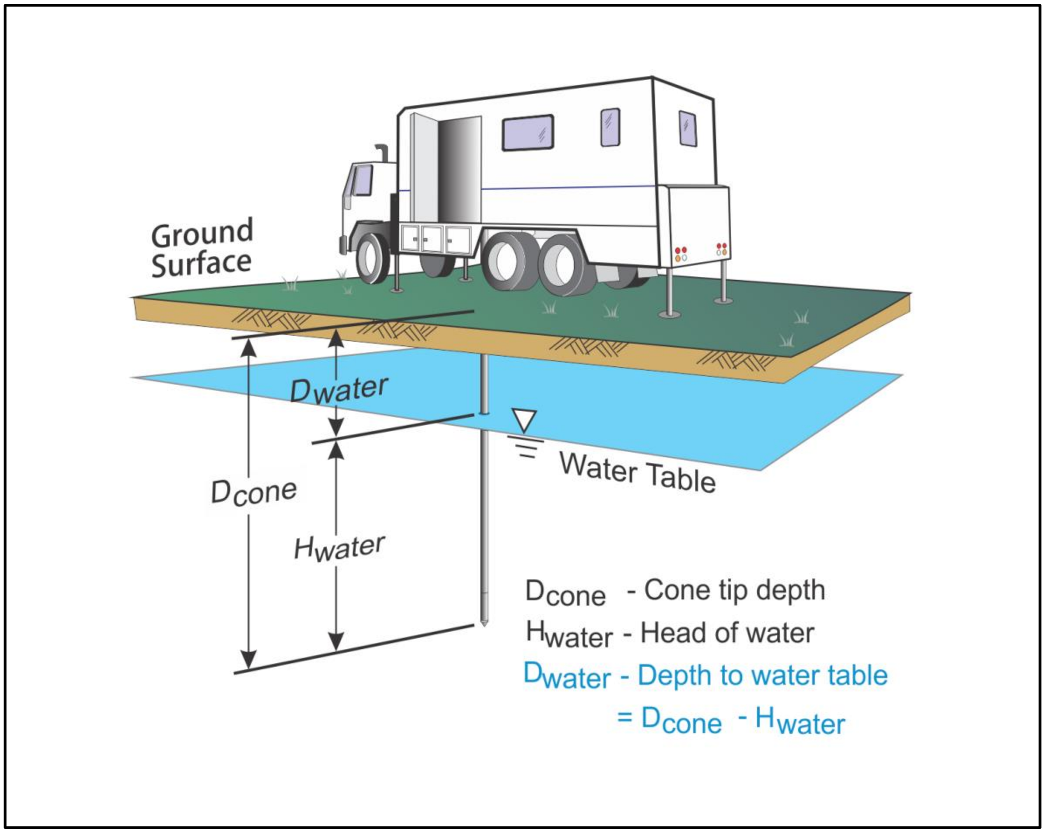


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

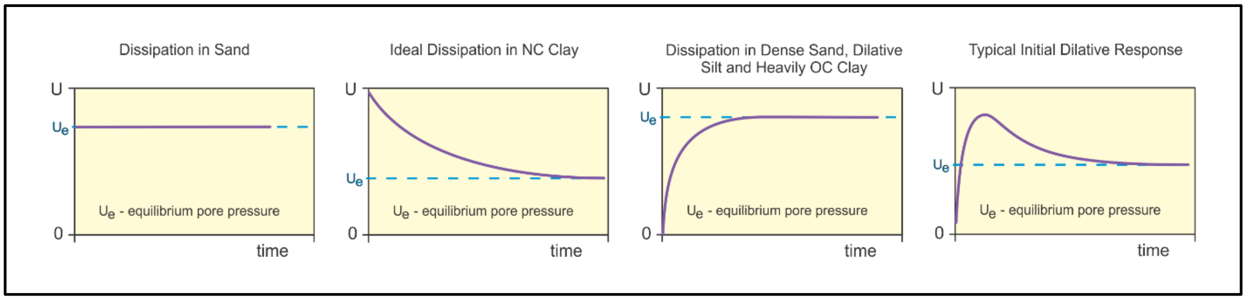


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T*	is the dimensionless time factor (Table Time Factor)
a	is the radius of the cone
I_r	is the rigidity index
t	is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

Burns, S.E. and Mayne, P.W., 1998, “Monotonic and dilatory pore pressure decay during piezocone tests”, Canadian Geotechnical Journal 26 (4): 1063-1073.

Burns, S.E. and Mayne, P.W., 2002, “Analytical cavity expansion-critical state model cone dissipation in fine-grained soils”, Soils & Foundations, Vol. 42(2): 131-137.

Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, “National shear wave measurement guidelines for Canadian seismic site assessment”, GeoManitoba 2012, Sept 30 to Oct 2, Winnipeg, Manitoba.

Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “Cone Penetration Testing in Geotechnical Practice”, Blackie Academic and Professional.

Mayne, P.W., 2013, “Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests”, Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420.

Mayne, P.W., 2014, “Interpretation of geotechnical parameters from seismic piezocone tests”, CPT’14 Keynote Address, Las Vegas, NV, May 2014.

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Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158.

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Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

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Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Standard Cone Penetration Test – Alternate Range
- Advanced Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

CONETEC

CONETEC

Job No: 17-05022
Client: Thurber Engineering Ltd.
Project: Hwy 31 and 401
Start Date: 24-May-2017
End Date: 24-May-2017

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting (m)	Elevation[3] (m)	Refer to Notation Number
SCPT17-01	17-05022_SP01	24-May-2017	379:T1500F15U500	6.9	29.700	4973782	484378	88.3	
SCPT17-02	17-05022_SP02	24-May-2017	379:T1500F15U500	6.9	11.625	4973707	484418	87.6	4, 5
SCPT17-02B	17-05022_SP02B	24-May-2017	379:T1500F15U500	6.9	30.175	4973690	484415	87.5	4, 5

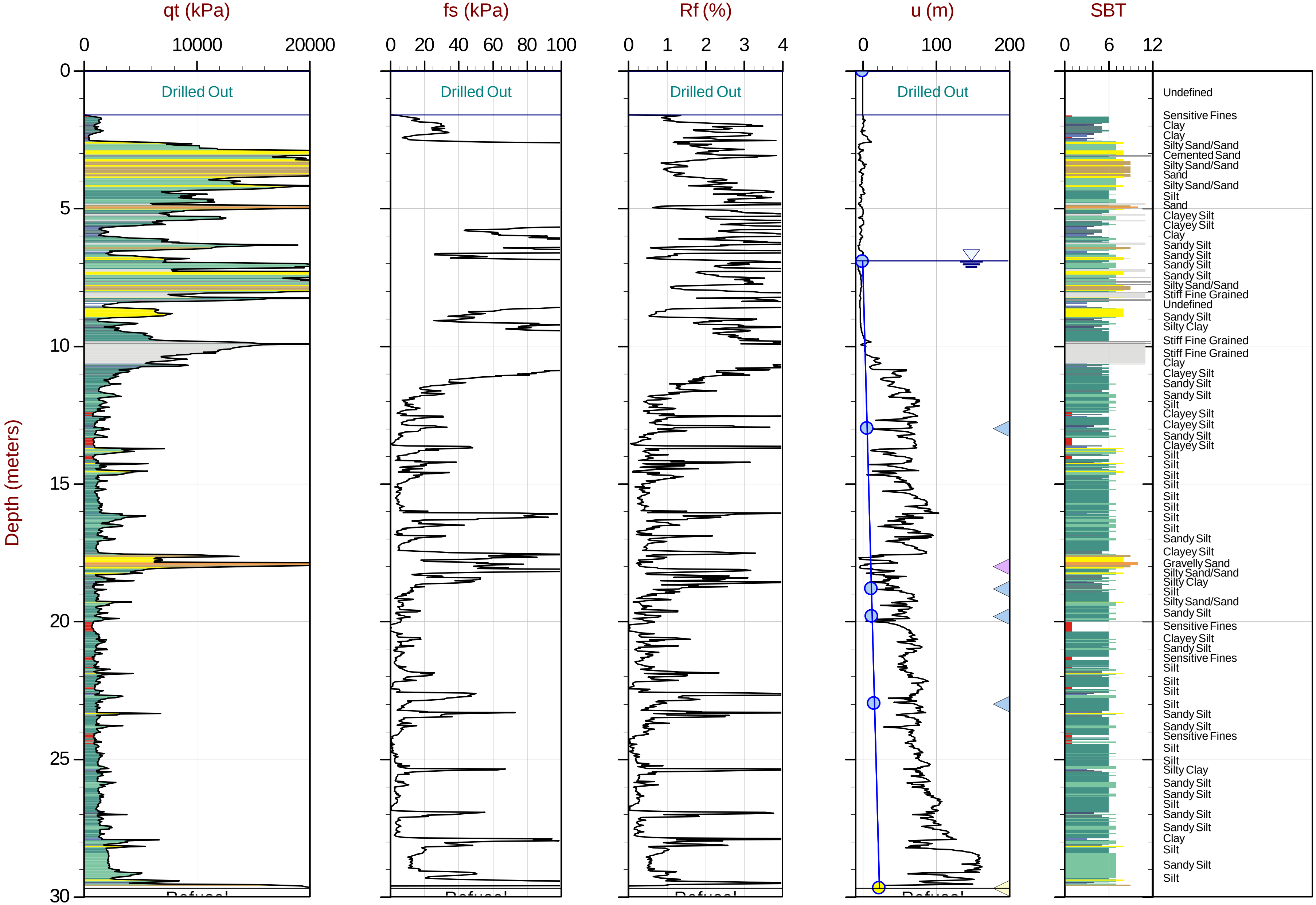
1. The assumed phreatic surface was based on pore pressure dissipation tests, unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 18 North.
3. Elevations are referenced to the existing ground surface at the time of testing. Elevations were provided by the client.
4. The assumed phreatic surface was based on a nearby sounding.
5. The pore pressure filter was located in the u1 position.

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 09:28
Site: Hwy 31 and 401

Sounding: SCPT17-01
Cone: 379:T1500F15U500



Max Depth: 29.700 m / 97.44 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP01.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973782m E: 484378m Elev: 88.3m
Sheet No: 1 of 1

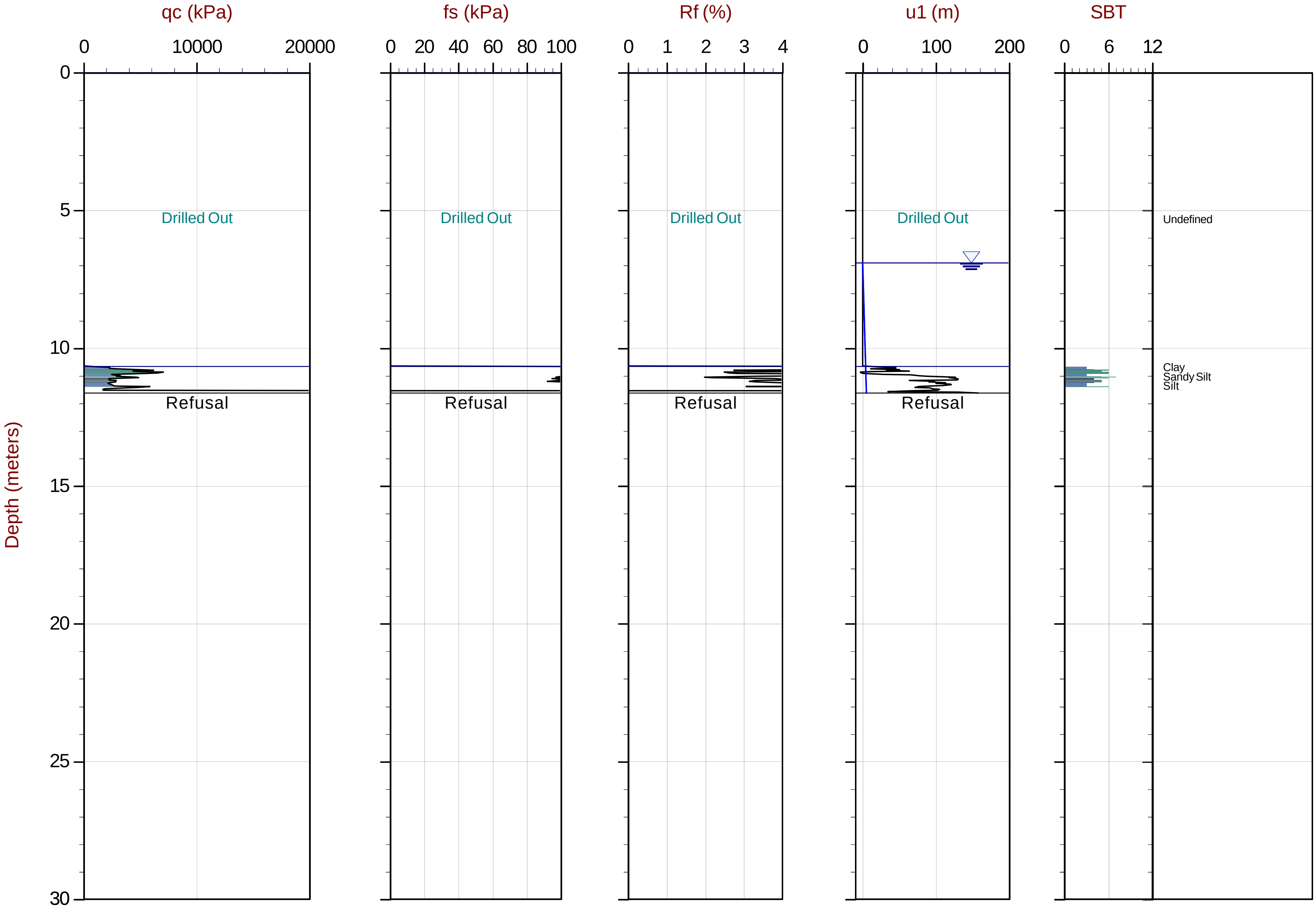
Overplot Item:
● Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line
◁ Dissipation, equilibrium assumed

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 15:43
Site: Hwy 31 and 401

Sounding: SCPT17-02
Cone: 379:T1500F15U500



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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973707m E: 484418m Elev: 87.6m
Sheet No: 1 of 1

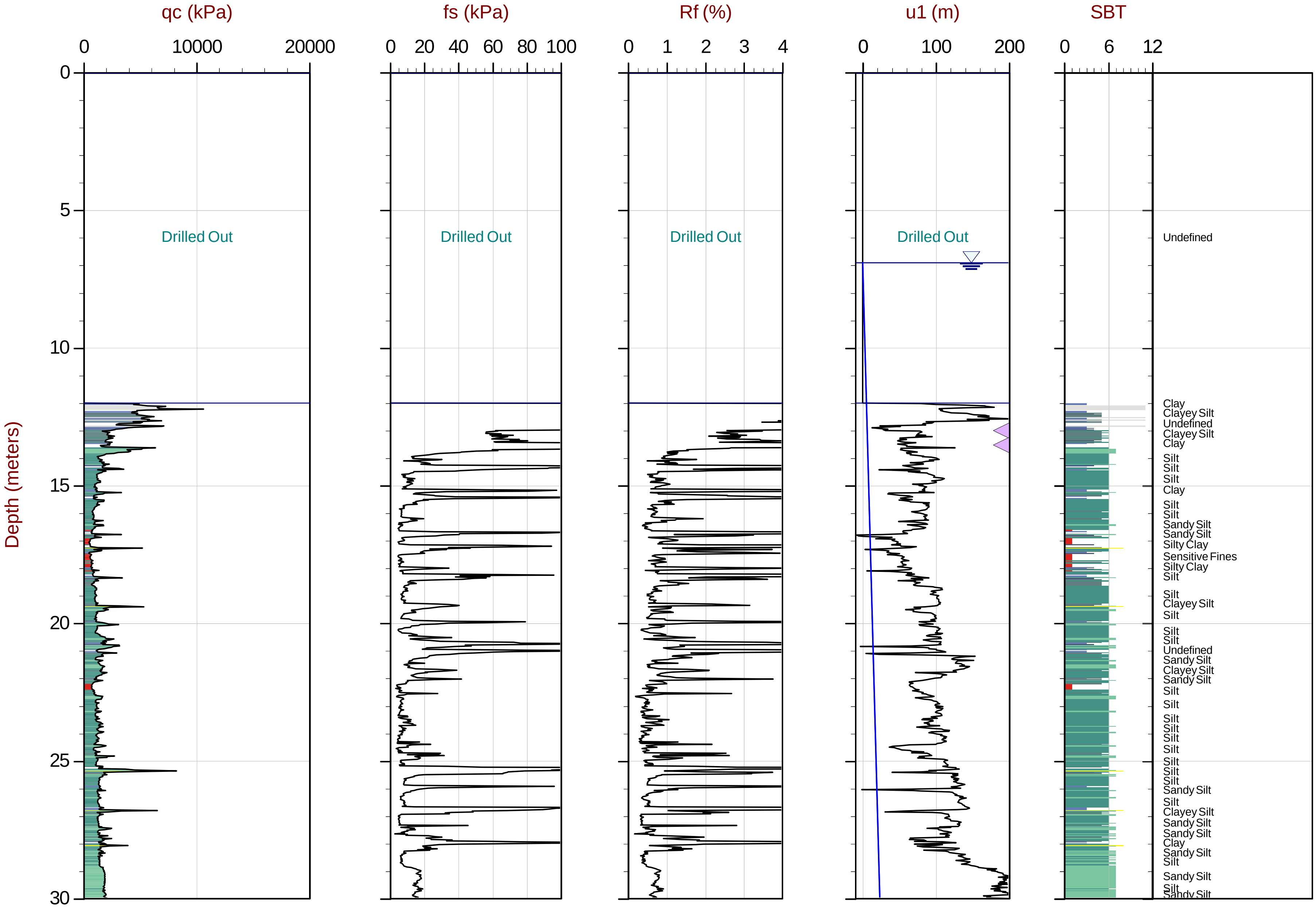
Overplot Item:
○ Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line
◁ Dissipation, equilibrium assumed

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 17:13
Site: Hwy 31 and 401

Sounding: SCPT17-02B
Cone: 379:T1500F15U500



Max Depth: 30.175 m / 99.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02B.COR
Unit Wt: SBT (R&C1986)

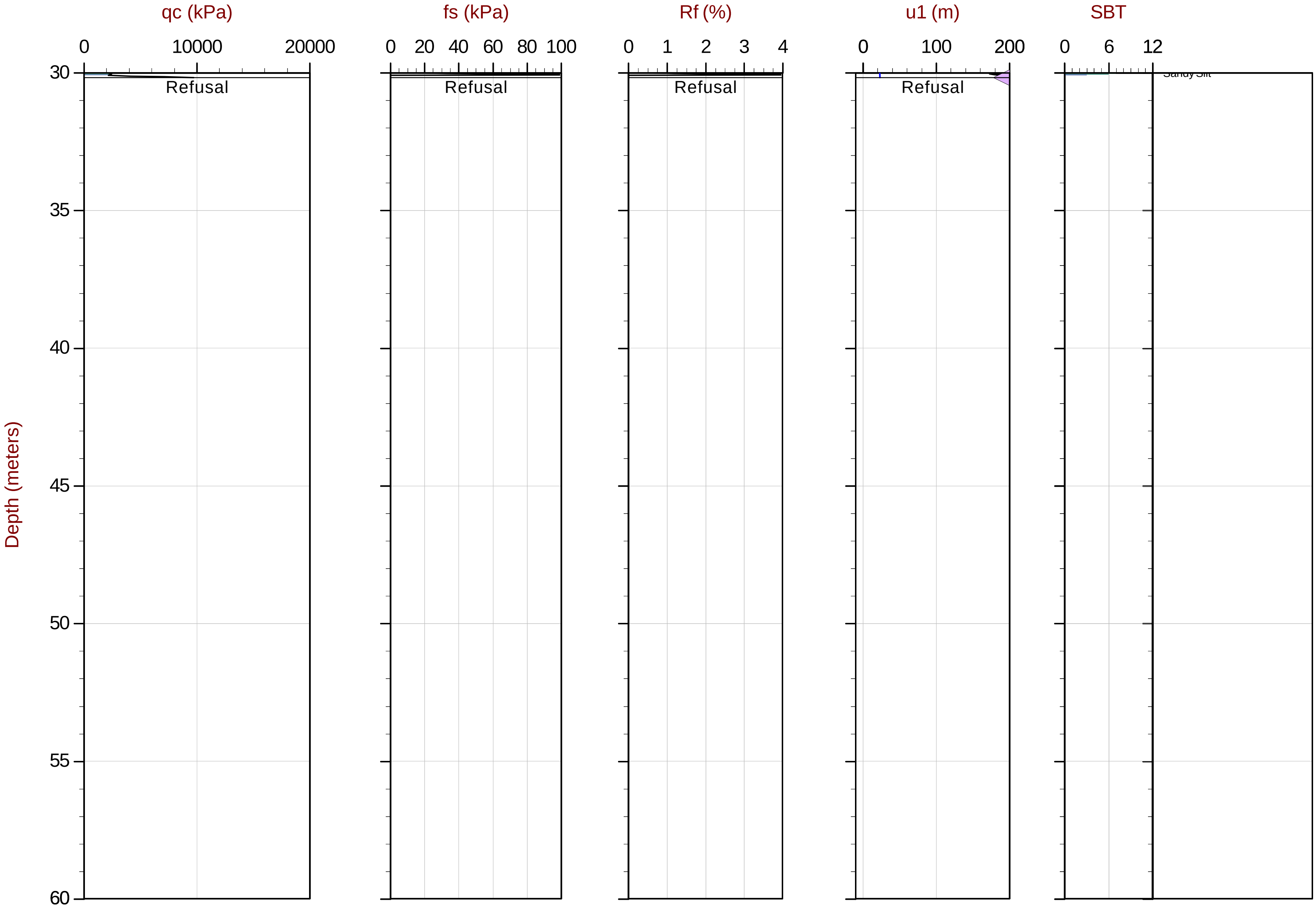
SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973690m E: 484415m Elev: 87.5m
Sheet No: 1 of 2

Overplot Item:
○ Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line
◁ Dissipation, equilibrium assumed

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 17:13
Site: Hwy 31 and 401

Sounding: SCPT17-02B
Cone: 379:T1500F15U500



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Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02B.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973690m E: 484415m Elev: 87.5m
Sheet No: 2 of 2

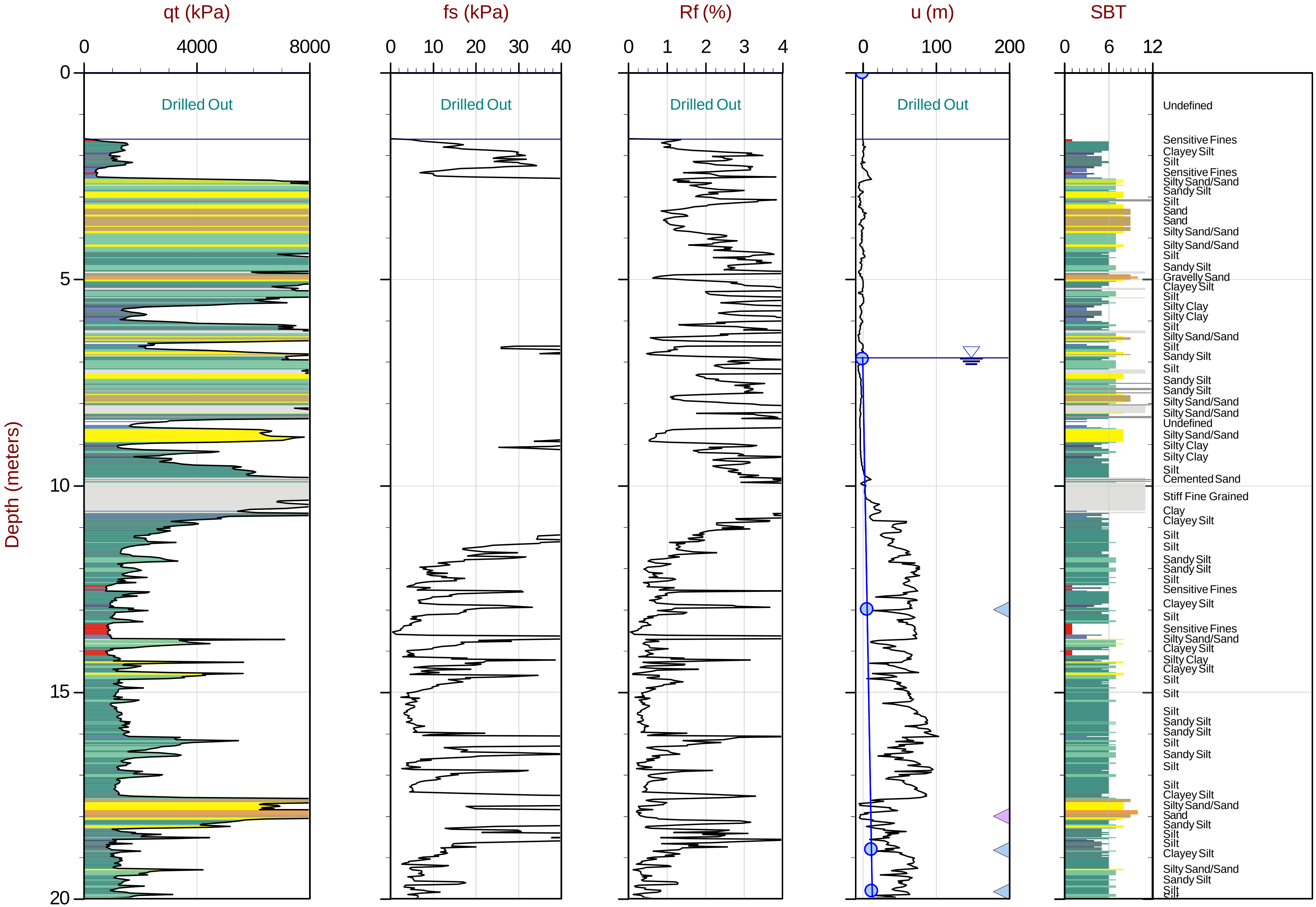
Overplot Item:			
	○ Assumed Ueq	◁ Dissipation, equilibrium achieved	— Hydrostatic Line
	● Ueq	◁ Dissipation, equilibrium not achieved	◁ Dissipation, equilibrium assumed

Standard Cone Penetration Test Plots – Alternate Range

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 09:28
Site: Hwy 31 and 401

Sounding: SCPT17-01
Cone: 379:T1500F15U500



Max Depth: 29.700 m / 97.44 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP01.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973782m E: 484378m Elev: 88.3m
Sheet No: 1 of 2

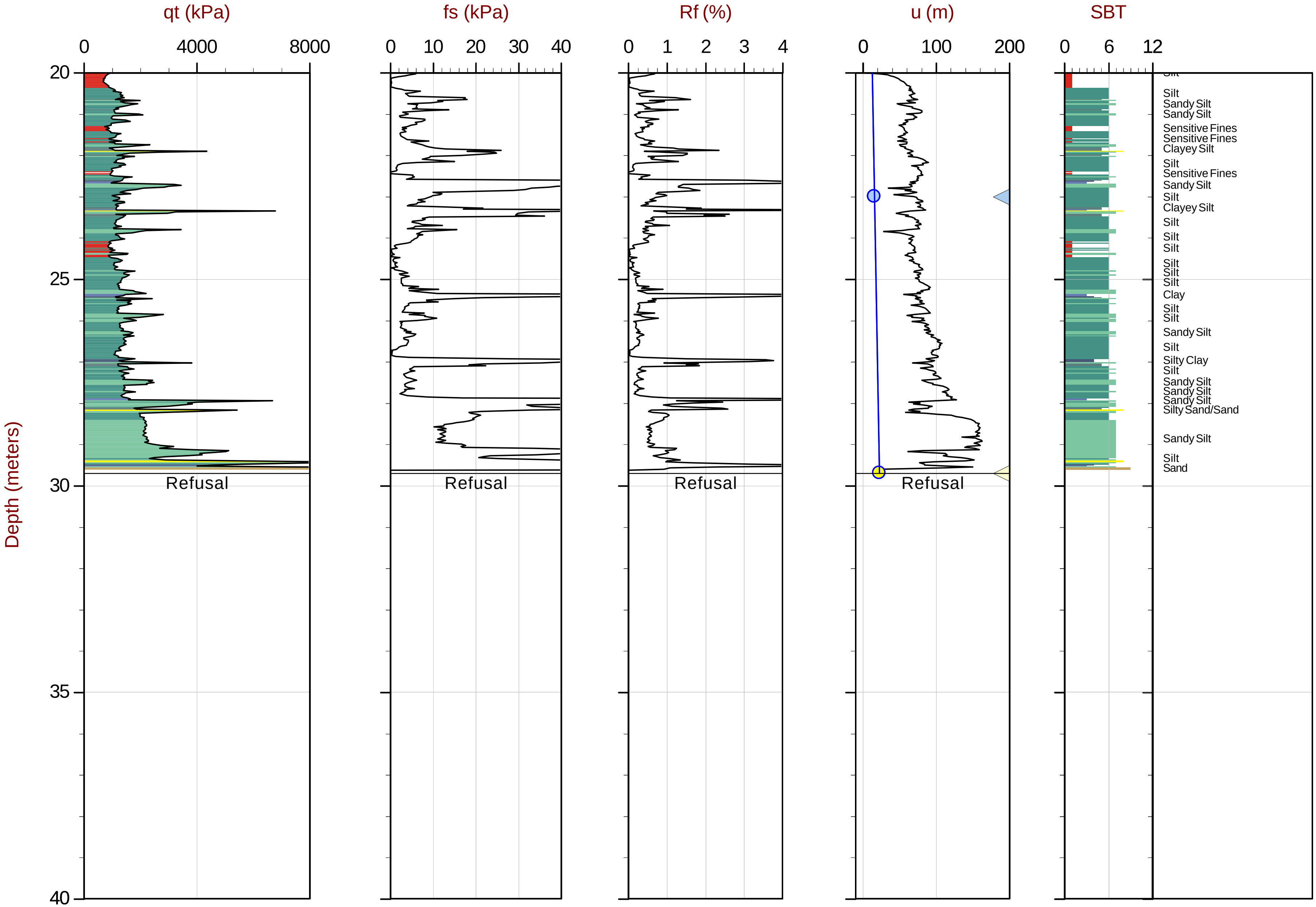
Overplot Item:			
	● Assumed Ueq	◁ Dissipation, equilibrium achieved	— Hydrostatic Line
	● Ueq	◁ Dissipation, equilibrium not achieved	◁ Dissipation, equilibrium assumed

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 09:28
Site: Hwy 31 and 401

Sounding: SCPT17-01
Cone: 379:T1500F15U500



Max Depth: 29.700 m / 97.44 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP01.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973782m E: 484378m Elev: 88.3m
Sheet No: 2 of 2

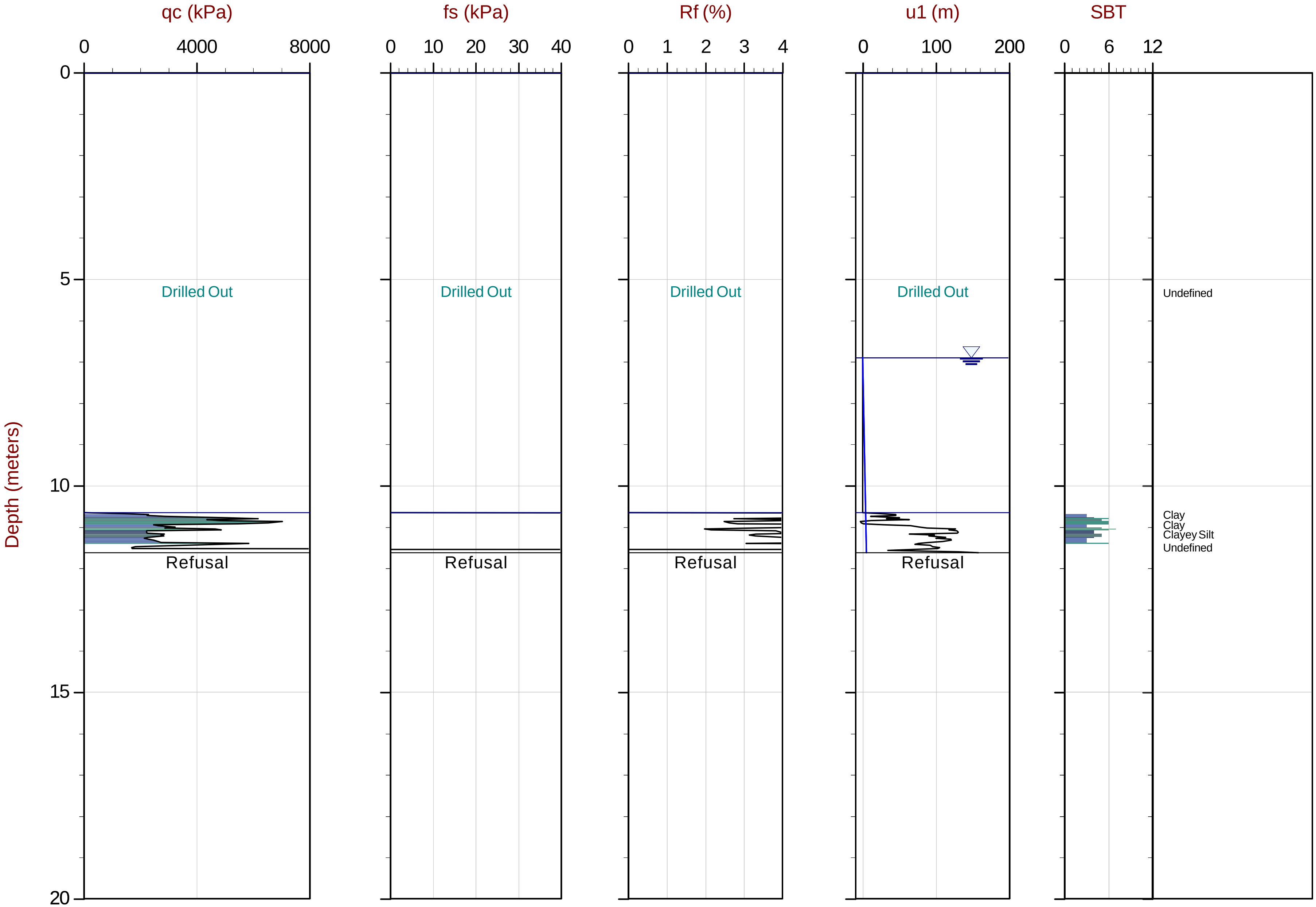
Overplot Item:
○ Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line
◁ Dissipation, equilibrium assumed

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 15:43
Site: Hwy 31 and 401

Sounding: SCPT17-02
Cone: 379:T1500F15U500



Max Depth: 11.625 m / 38.14 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973707m E: 484418m Elev: 87.6m
Sheet No: 1 of 1

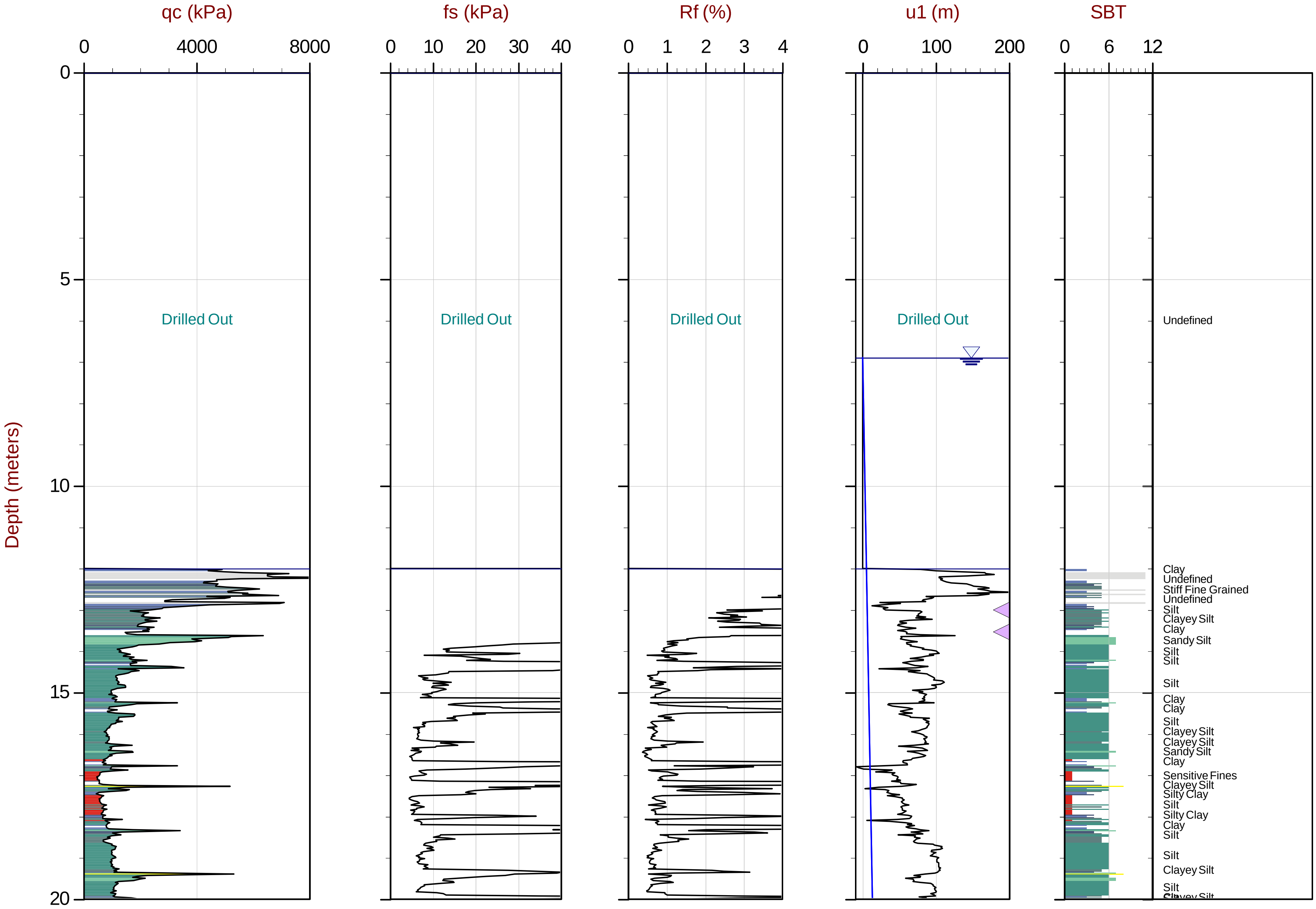
Overplot Item:	○ Assumed Ueq	◁ Dissipation, equilibrium achieved	— Hydrostatic Line
	○ Ueq	◁ Dissipation, equilibrium not achieved	◁ Dissipation, equilibrium assumed

ConeTec

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 17:13
Site: Hwy 31 and 401

Sounding: SCPT17-02B
Cone: 379:T1500F15U500



Max Depth: 30.175 m / 99.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02B.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973690m E: 484415m Elev: 87.5m
Sheet No: 1 of 2

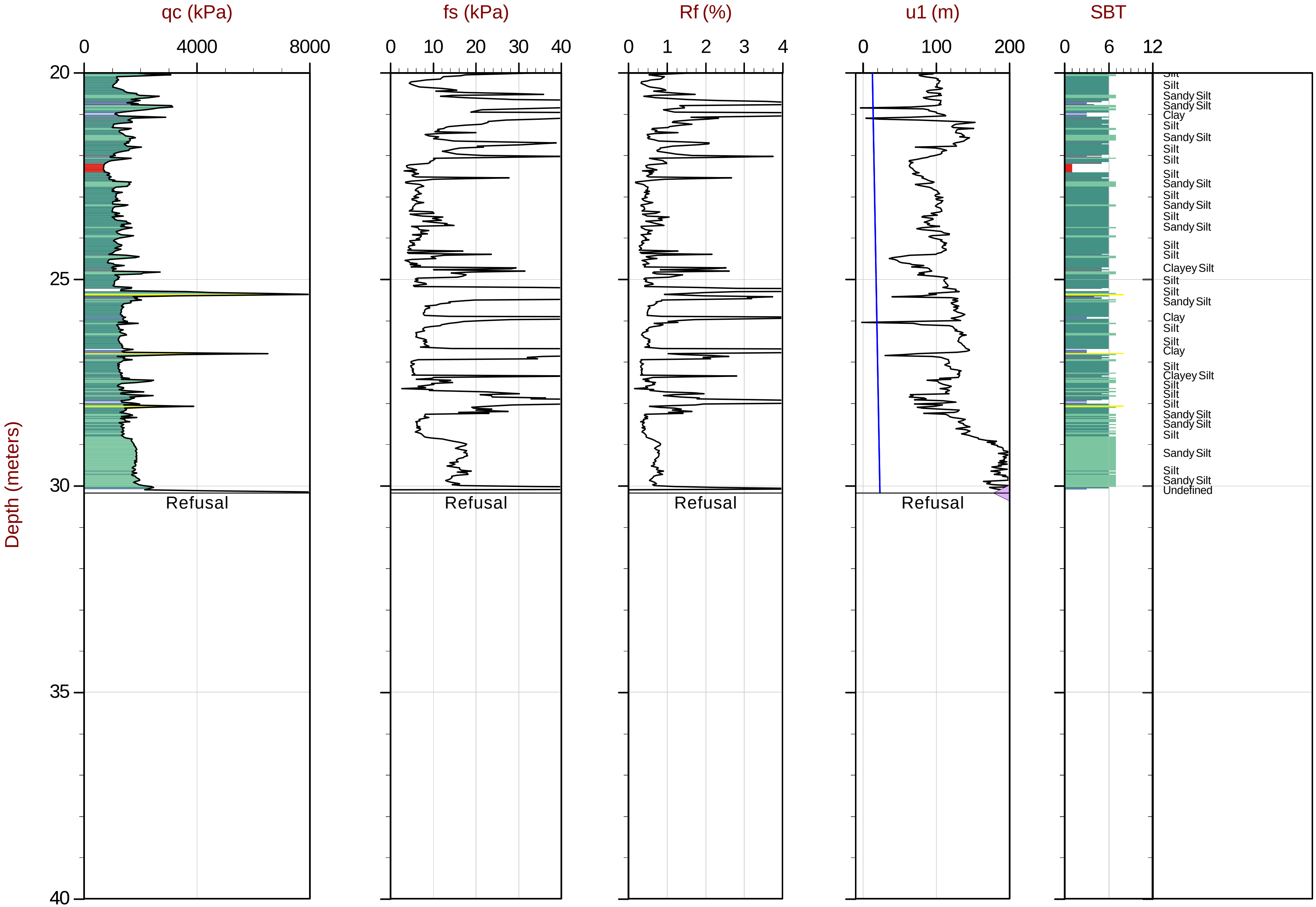
Overplot Item:
- ○ Assumed Ueq
- ● Ueq
- ◁ Dissipation, equilibrium achieved
- ◁ Dissipation, equilibrium not achieved
- — Hydrostatic Line
- ◁ Dissipation, equilibrium assumed

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 17:13
Site: Hwy 31 and 401

Sounding: SCPT17-02B
Cone: 379:T1500F15U500



Max Depth: 30.175 m / 99.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02B.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973690m E: 484415m Elev: 87.5m
Sheet No: 2 of 2

Overplot Item:

○ Assumed Ueq	◁ Dissipation, equilibrium achieved	— Hydrostatic Line
● Ueq	◁ Dissipation, equilibrium not achieved	◁ Dissipation, equilibrium assumed

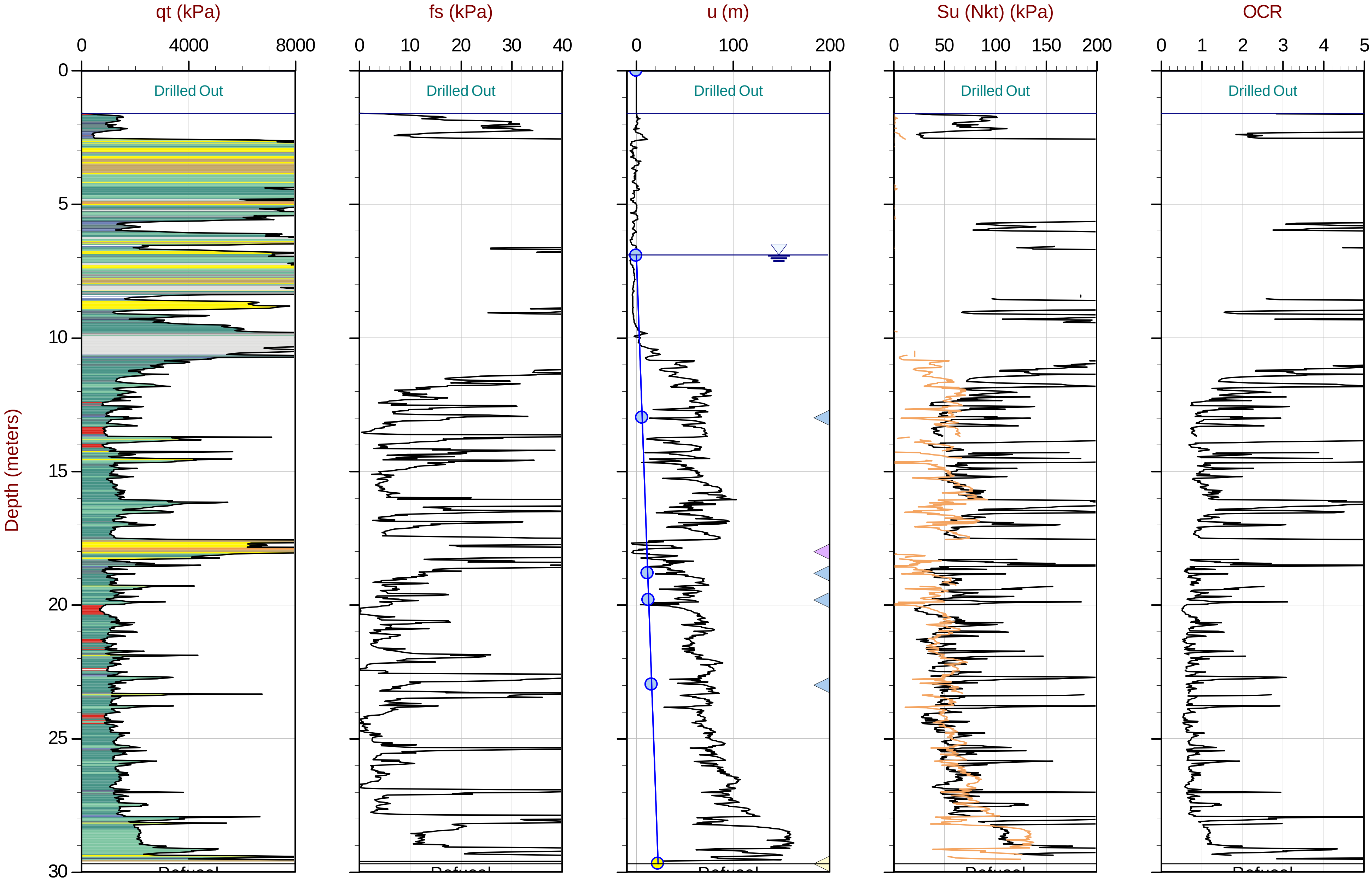
Advanced Cone Penetration Test Plots

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 09:28
Site: Hwy 31 and 401

Sounding: SCPT17-01
Cone: 379:T1500F15U500



Max Depth: 29.700 m / 97.44 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP01.COR
Unit Wt: SBT (R&C1986)
Su Nkt/Ndu: 15.0 / 10.0

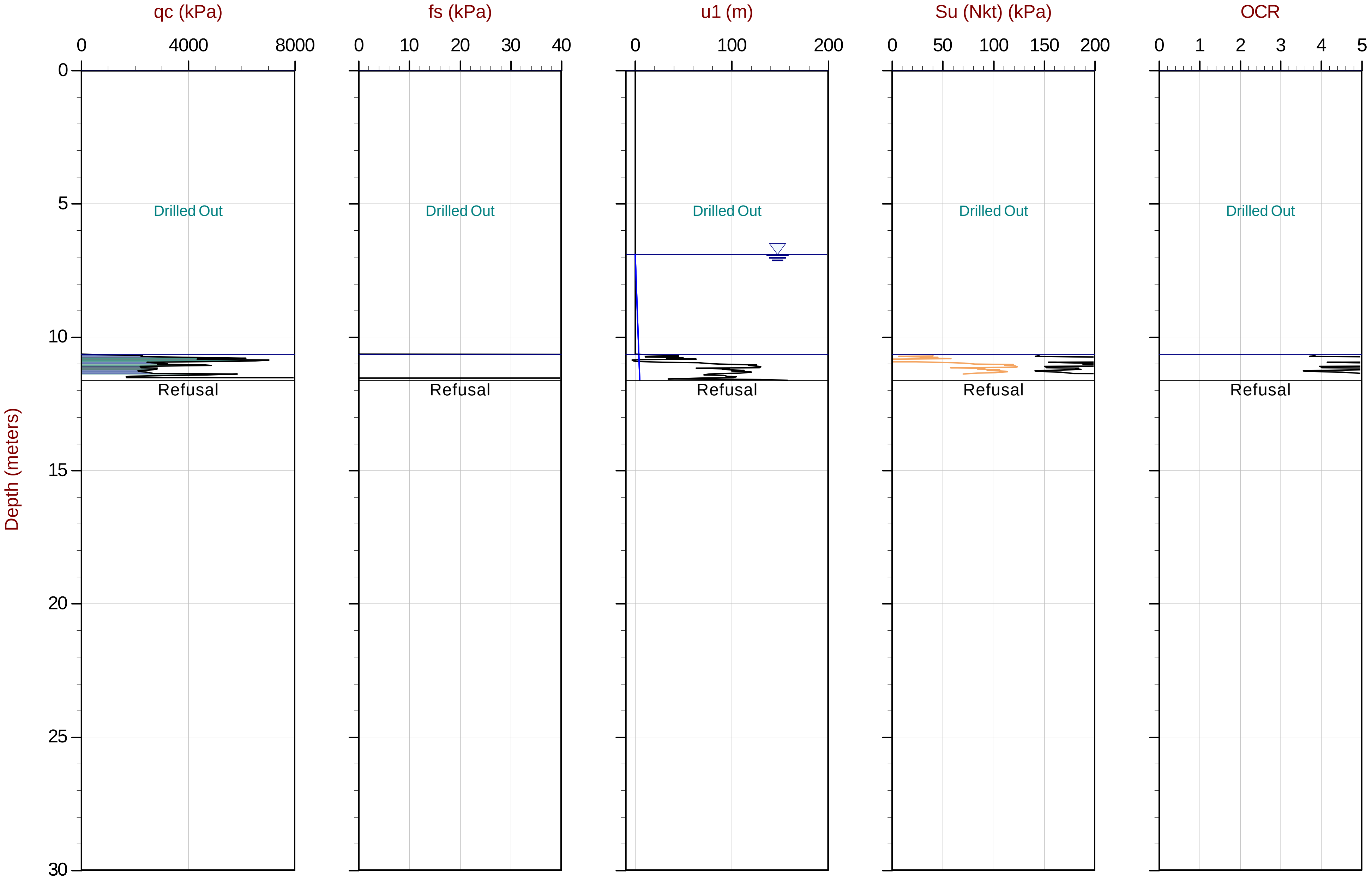
SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973782m E: 484378m Elev: 88.3m
Sheet No: 1 of 1

Overplot Item:
● Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line
◁ Dissipation, equilibrium assumed
— Su (Ndu)

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 15:43
Site: Hwy 31 and 401

Sounding: SCPT17-02
Cone: 379:T1500F15U500



Max Depth: 11.625 m / 38.14 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02.COR
Unit Wt: SBT (R&C1986)
Su Nkt/Ndu: 15.0 / 10.0

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973707m E: 484418m Elev: 87.6m
Sheet No: 1 of 1

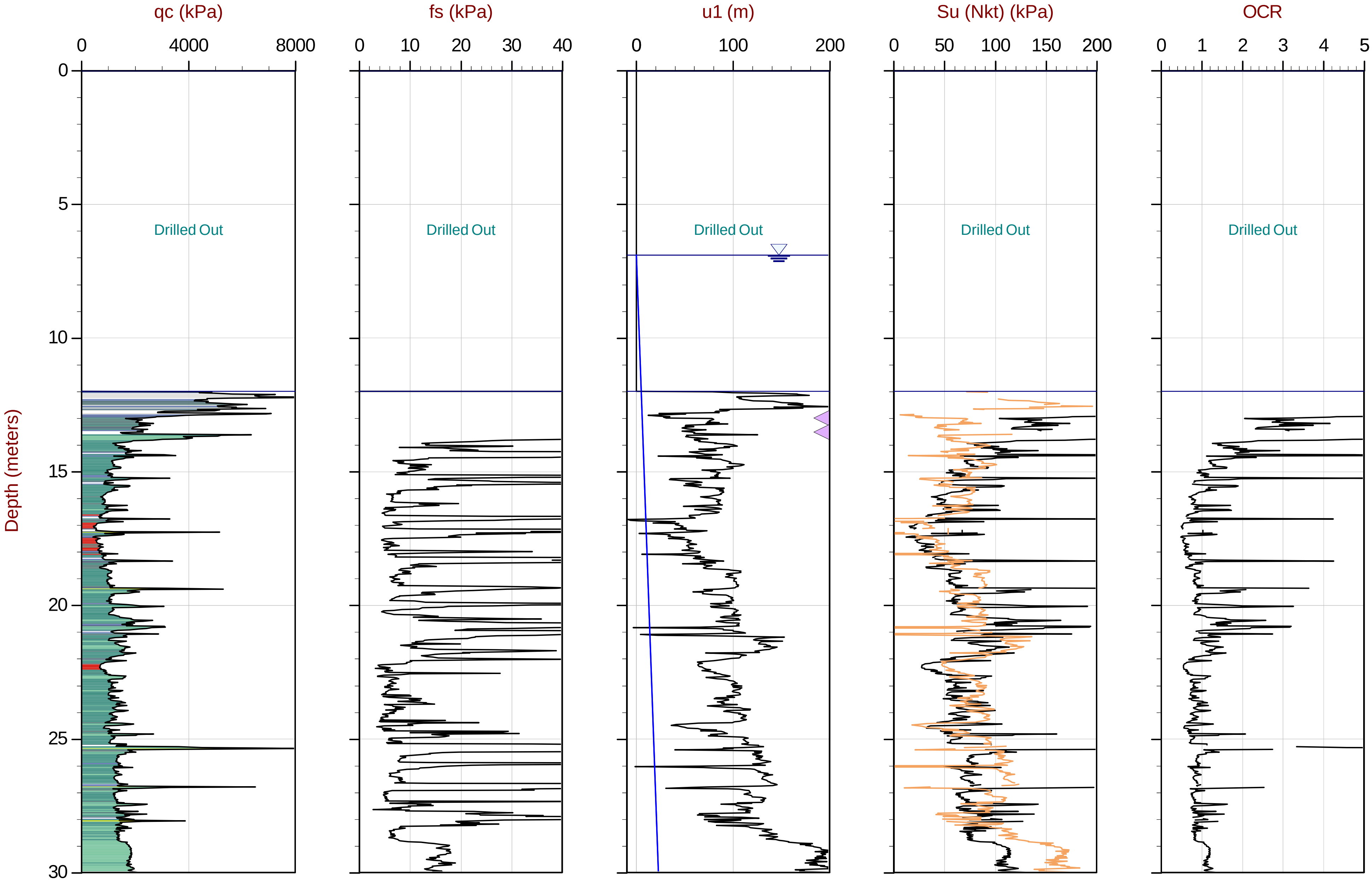
Overplot Item:
○ Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line
◁ Dissipation, equilibrium assumed
— Su (Ndu)

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 17:13
Site: Hwy 31 and 401

Sounding: SCPT17-02B
Cone: 379:T1500F15U500



Max Depth: 30.175 m / 99.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02B.COR
Unit Wt: SBT (R&C1986)
Su Nkt/Ndu: 15.0 / 10.0

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973690m E: 484415m Elev: 87.5m
Sheet No: 1 of 2

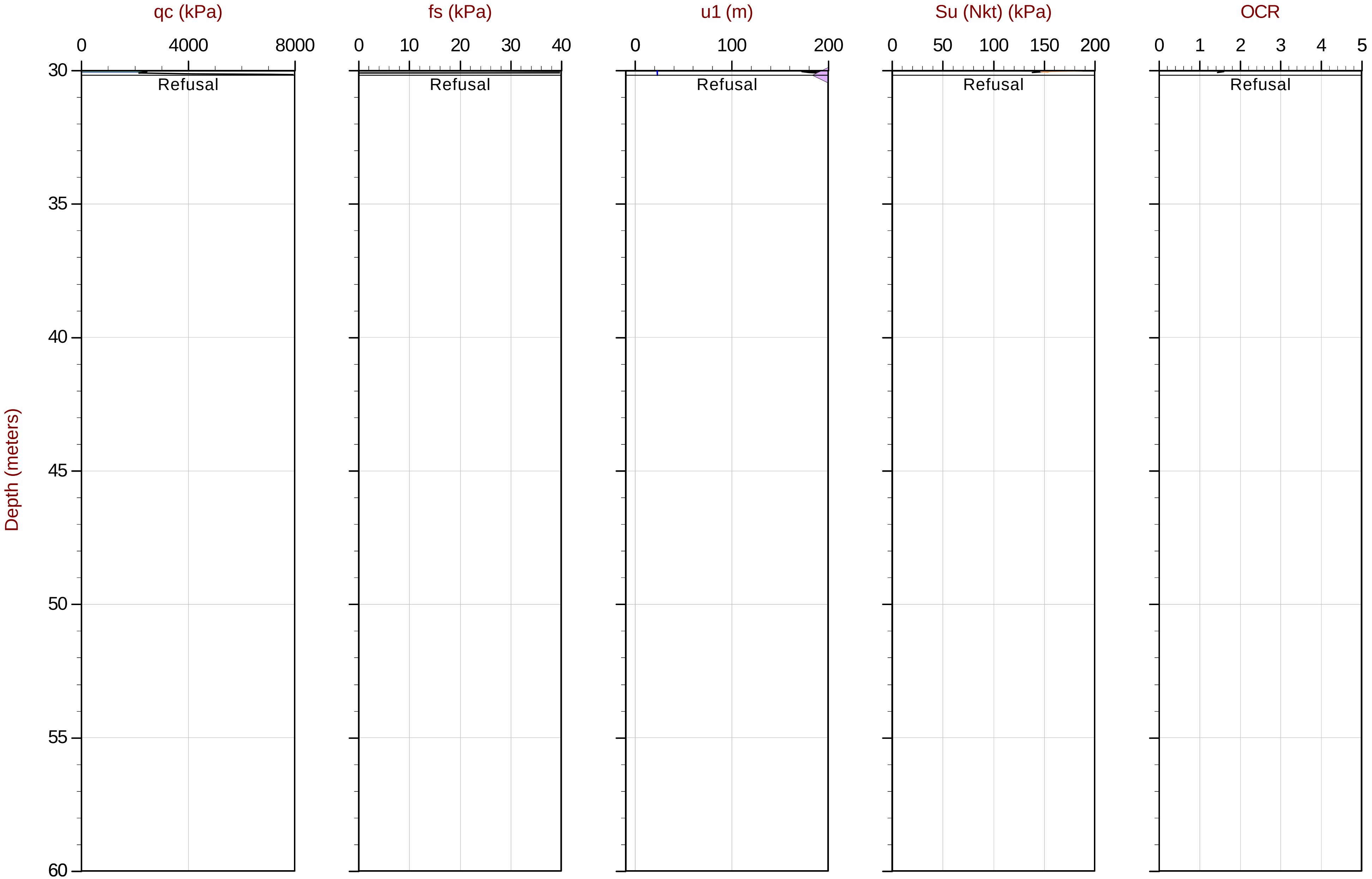
Overplot Item:
- ● Assumed Ueq
- ● Ueq
- ◁ Dissipation, equilibrium achieved
- ◁ Dissipation, equilibrium not achieved
- — Hydrostatic Line
- ◁ Dissipation, equilibrium assumed
- — Su (Ndu)

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 17:13
Site: Hwy 31 and 401

Sounding: SCPT17-02B
Cone: 379:T1500F15U500



Max Depth: 30.175 m / 99.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02B.COR
Unit Wt: SBT (R&C1986)
Su Nkt/Ndu: 15.0 / 10.0

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973690m E: 484415m Elev: 87.5m
Sheet No: 2 of 2

Overplot Item:
- ○ Assumed Ueq
- ○ Ueq
- ◁ Dissipation, equilibrium achieved
- ◁ Dissipation, equilibrium not achieved
- — Hydrostatic Line
- ◁ Dissipation, equilibrium assumed
- — Su (Ndu)

Seismic Cone Penetration Test Tabular Results

Job No: 17-05022
Client: Thurber Engineering Ltd.
Project: Hwy 31 and 401
Sounding ID: SCPT17-01
Date: 24-May-2017

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.85	1.65	1.74			
2.85	2.65	2.71	0.97	3.13	309
3.85	3.65	3.69	0.98	2.68	367
4.85	4.65	4.68	0.99	2.87	345
5.85	5.65	5.68	0.99	2.55	391
6.85	6.65	6.67	1.00	2.65	376
7.85	7.65	7.67	1.00	2.10	476
8.82	8.62	8.64	0.97	1.96	493
10.85	10.65	10.66	2.03	4.85	418
11.85	11.65	11.66	1.00	1.85	540
12.85	12.65	12.66	1.00	1.83	547
13.85	13.65	13.66	1.00	1.93	517
14.85	14.65	14.66	1.00	1.94	515
16.05	15.85	15.86	1.20	2.27	529
16.85	16.65	16.66	0.80	1.52	525
17.85	17.65	17.66	1.00	1.82	549
18.82	18.62	18.63	0.97	1.71	566
19.82	19.62	19.63	1.00	1.97	507
20.85	20.65	20.66	1.03	2.00	514
21.85	21.65	21.66	1.00	2.03	492
22.85	22.65	22.66	1.00	2.08	481
23.85	23.65	23.66	1.00	2.03	493
24.85	24.65	24.66	1.00	1.93	518
25.85	25.65	25.66	1.00	1.83	547
26.85	26.65	26.66	1.00	1.83	547
27.85	27.65	27.66	1.00	1.93	517
28.85	28.65	28.66	1.00	1.80	557

CONETEC

Job No:	17-05022
Client:	Thurber Engineering Ltd.
Project:	Hwy 31 and 401
Sounding ID:	SCPT17-02B
Date:	24-May-2017

Seismic Source:	Beam
Source Offset (m):	0.55
Source Depth (m):	0.00
Geophone Offset (m):	0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
13.00	12.80	12.81			
13.93	13.73	13.74	0.93	1.73	538
14.93	14.73	14.74	1.00	2.07	482
15.93	15.73	15.74	1.00	2.29	437
16.93	16.73	16.74	1.00	2.46	405
17.93	17.73	17.74	1.00	2.34	428
18.93	18.73	18.74	1.00	2.06	486
19.93	19.73	19.74	1.00	2.03	492
20.93	20.73	20.74	1.00	1.96	511
21.93	21.73	21.74	1.00	1.90	525
22.93	22.73	22.74	1.00	2.00	501
23.93	23.73	23.74	1.00	1.83	547
24.93	24.73	24.74	1.00	1.60	625
25.93	25.73	25.74	1.00	1.60	625
26.90	26.70	26.71	0.97	1.79	541
27.93	27.73	27.74	1.03	1.94	530
28.93	28.73	28.74	1.00	1.87	536
29.93	29.73	29.74	1.00	2.11	474

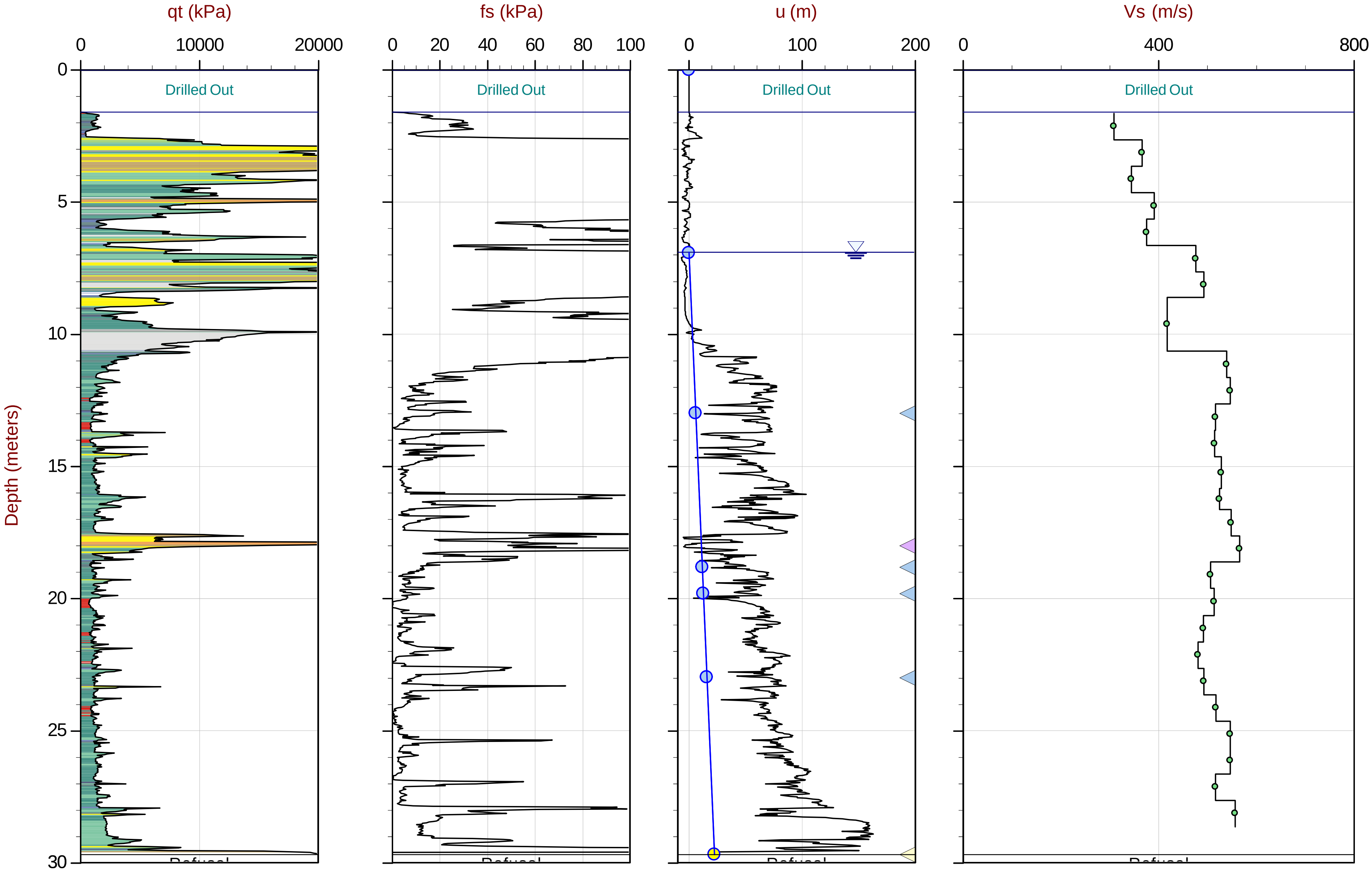
Seismic Cone Penetration Test Plots

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 09:28
Site: Hwy 31 and 401

Sounding: SCPT17-01
Cone: 379:T1500F15U500



Max Depth: 29.700 m / 97.44 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP01.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973782m E: 484378m Elev: 88.3m
Sheet No: 1 of 1

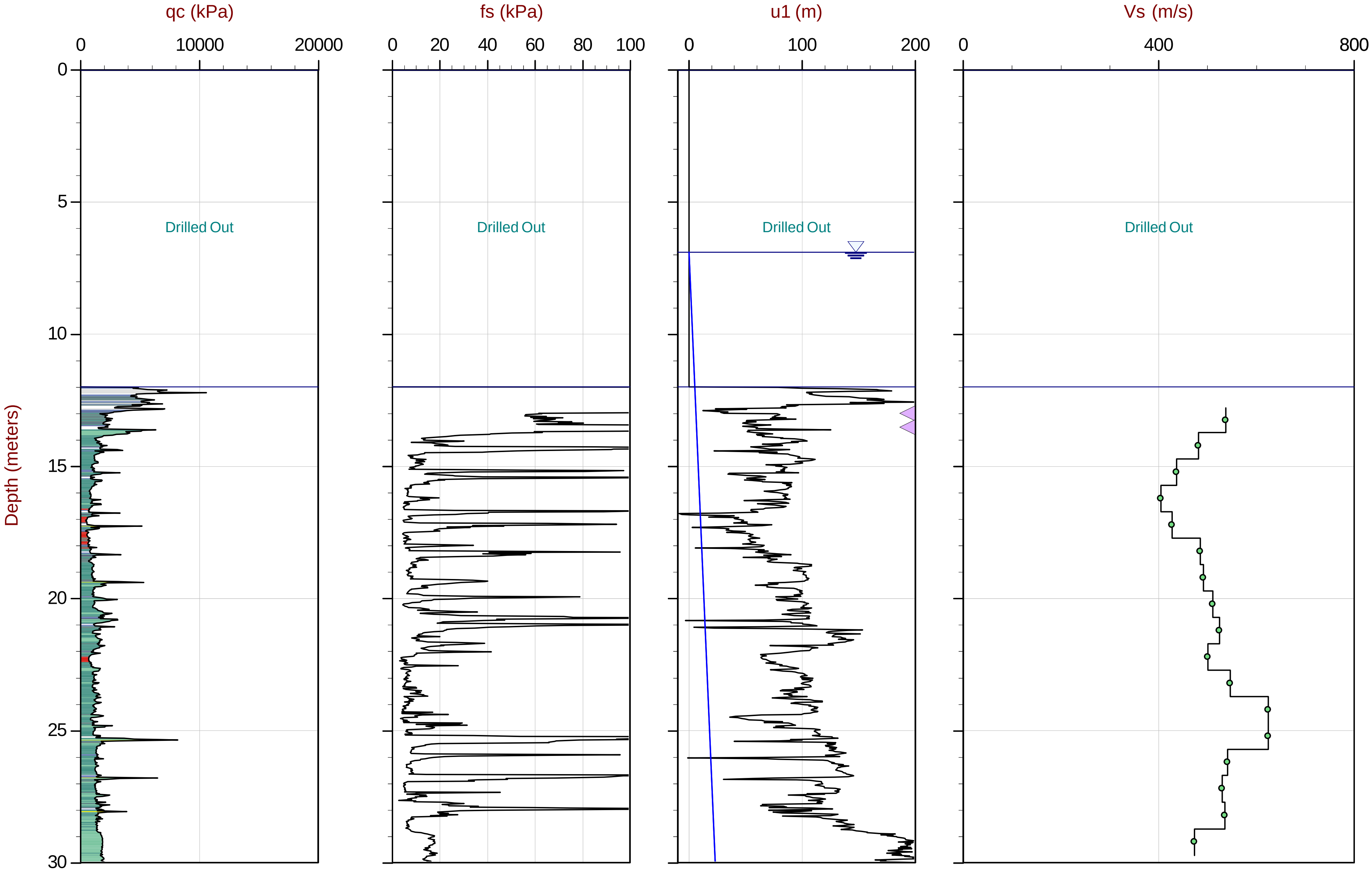
Overplot Item:
○ Assumed Ueq
● Ueq
◁ Dissipation, equilibrium achieved
◁ Dissipation, equilibrium not achieved
— Hydrostatic Line
◁ Dissipation, equilibrium assumed

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 17:13
Site: Hwy 31 and 401

Sounding: SCPT17-02B
Cone: 379:T1500F15U500



Max Depth: 30.175 m / 99.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02B.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973690m E: 484415m Elev: 87.5m
Sheet No: 1 of 2

Overplot Item:

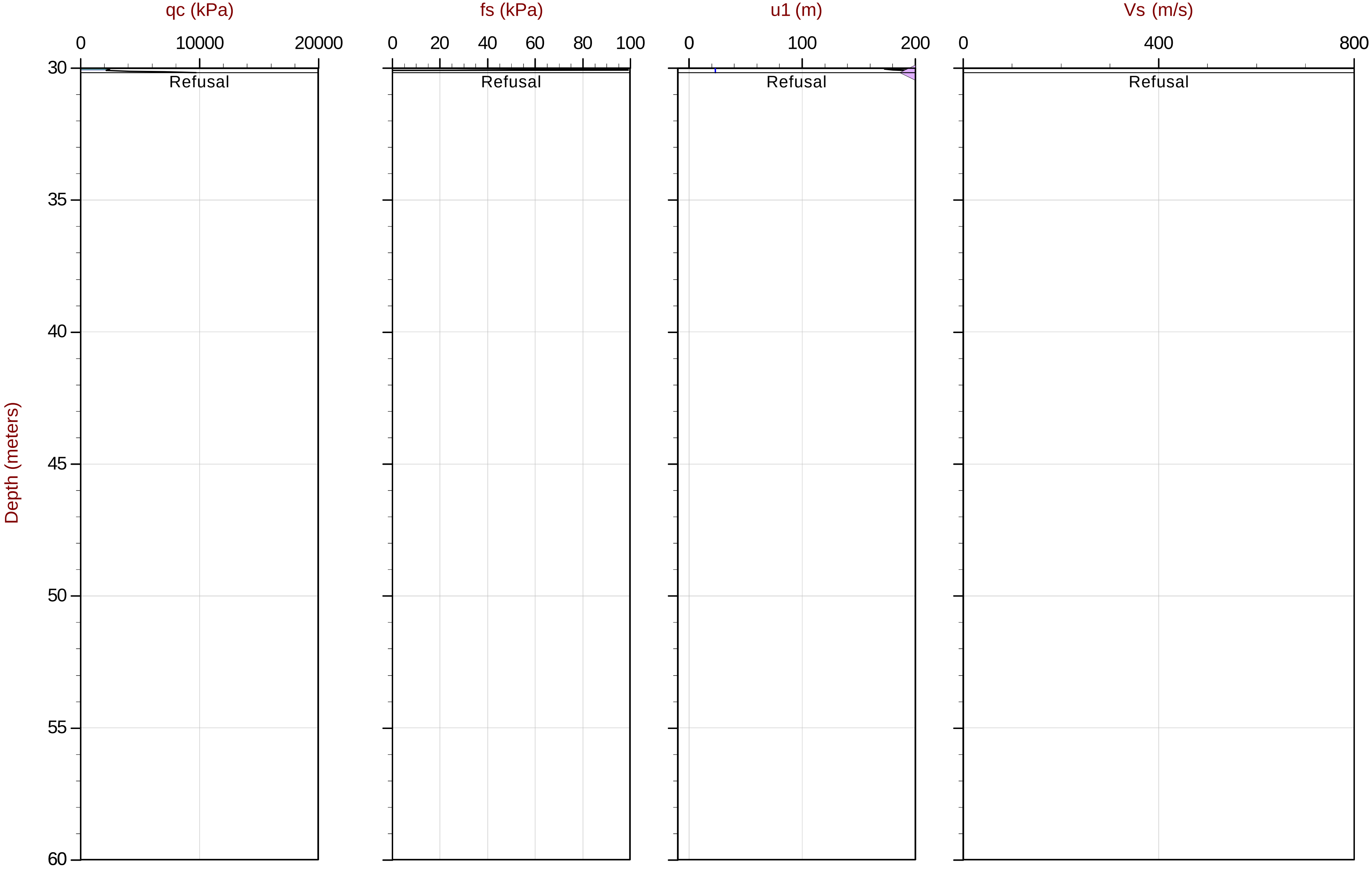
● Assumed Ueq	◁ Dissipation, equilibrium achieved	— Hydrostatic Line
● Ueq	◁ Dissipation, equilibrium not achieved	◁ Dissipation, equilibrium assumed

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 2017-05-24 17:13
Site: Hwy 31 and 401

Sounding: SCPT17-02B
Cone: 379:T1500F15U500



Max Depth: 30.175 m / 99.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05022_SP02B.COR
Unit Wt: SBT (R&C1986)

SBT: Robertson and Campanella, 1986
Coords: UTM 18N N: 4973690m E: 484415m Elev: 87.5m
Sheet No: 2 of 2

Overplot Item:
- ● Assumed Ueq
- ● Ueq
- ◁ Dissipation, equilibrium achieved
- ◁ Dissipation, equilibrium not achieved
- — Hydrostatic Line
- ◁ Dissipation, equilibrium assumed

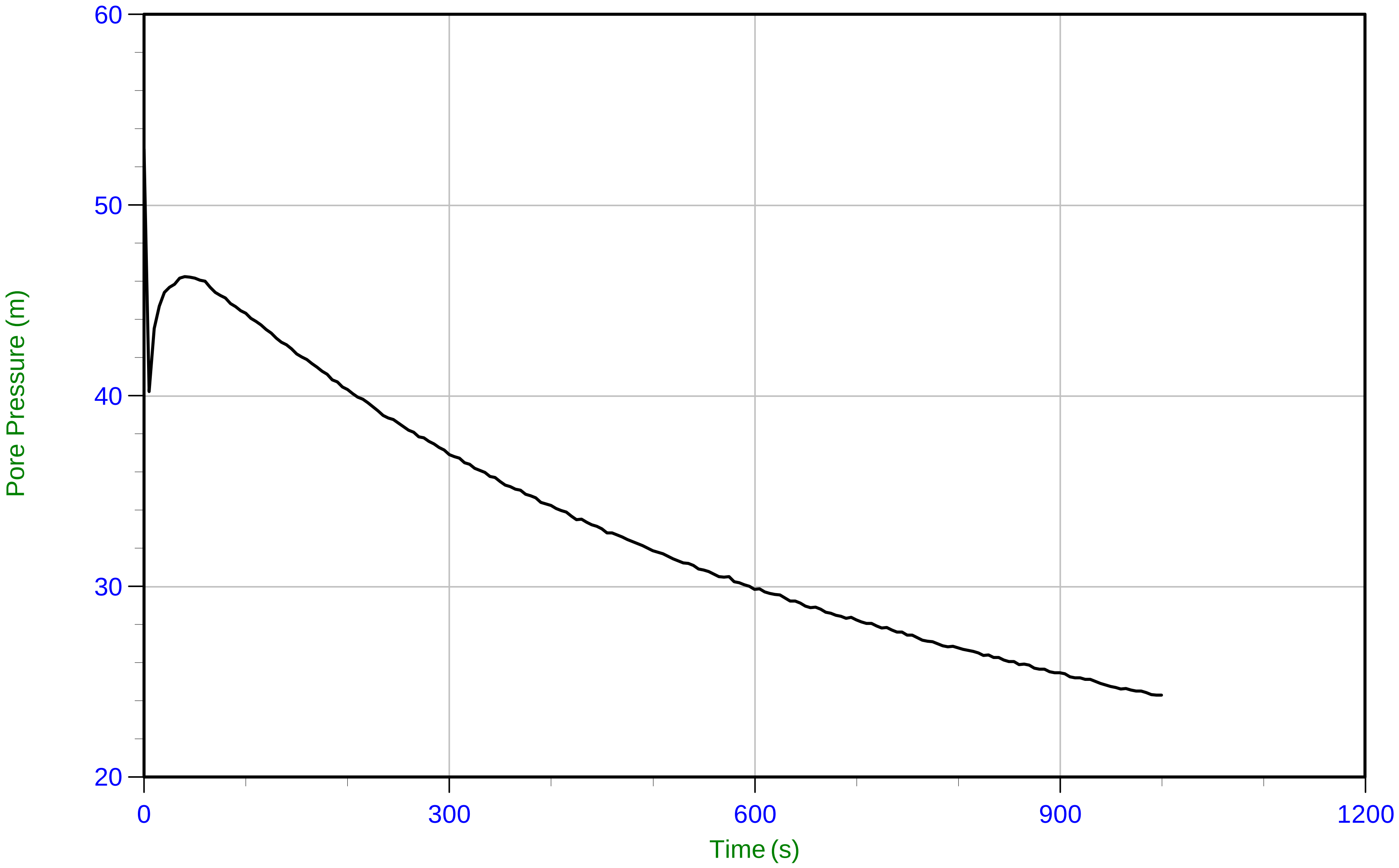
Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No: 17-05022
Client: Thurber Engineering Ltd.
Project: Hwy 31 and 401
Start Date: 24-May-2017
End Date: 24-May-2017

CPTu PORE PRESSURE DISSIPATION SUMMARY

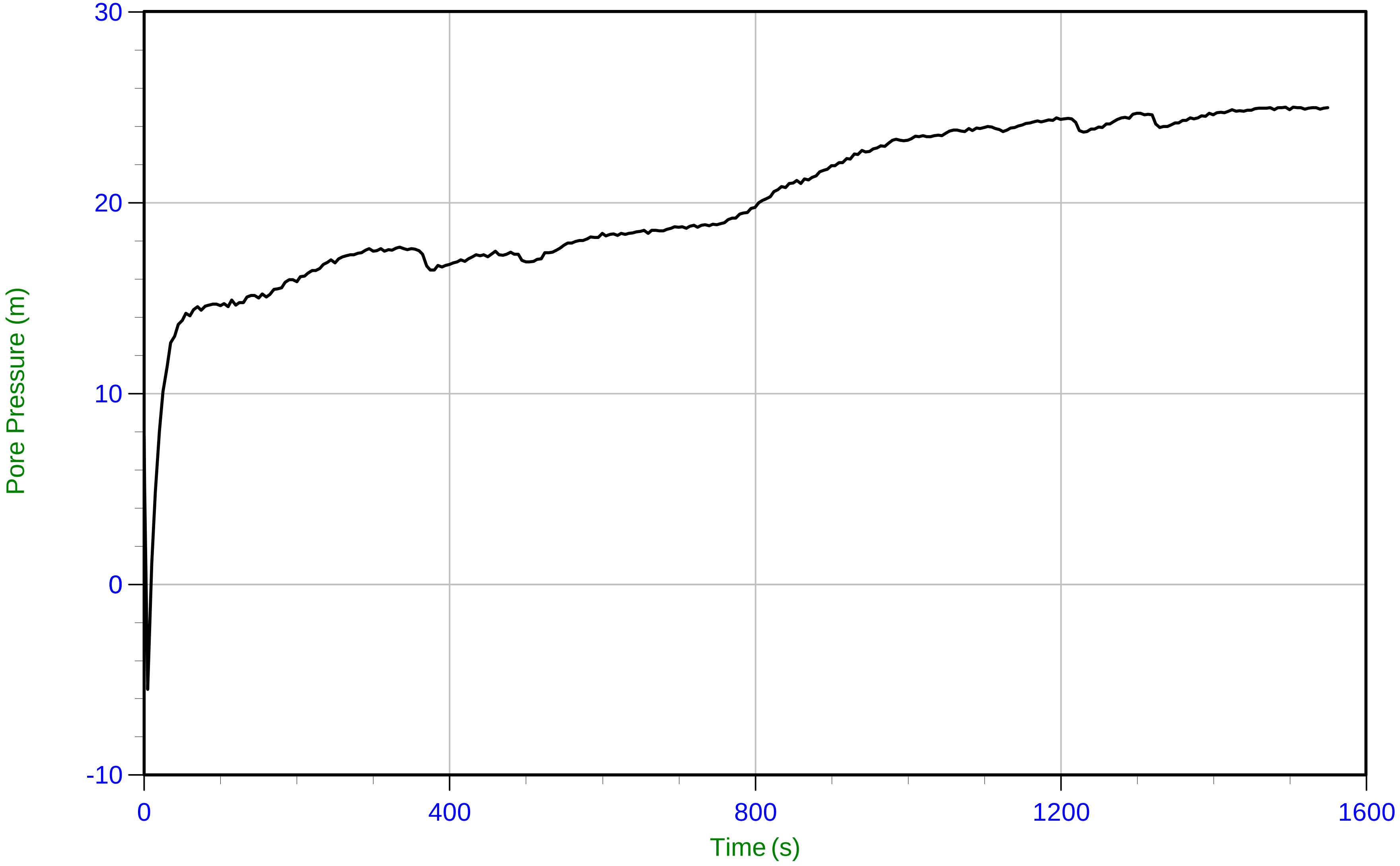
Sounding ID	File Name	Cone Area (cm^2)	Pore Pressure Filter Location	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)	t_{50} [a] (s)	Assumed Rigidity Index (I_r)	c_h [b] (cm^2/min)	Refer to Notation
SCPT17-01	17-05022_SP01	15	u2	1000	13.000	Not Achieved		6.9	627	100	1.1	
SCPT17-01	17-05022_SP01	15	u2	1550	18.000	Not Achieved						c
SCPT17-01	17-05022_SP01	15	u2	2300	18.825	Not Achieved		6.9	1753	100	0.4	
SCPT17-01	17-05022_SP01	15	u2	1200	19.825	Not Achieved		6.9	1053	100	0.7	
SCPT17-01	17-05022_SP01	15	u2	3800	23.000	Not Achieved		6.9	3007	100	0.2	
SCPT17-01	17-05022_SP01	15	u2	300	29.700	22.8	6.9					
SCPT17-02B	17-05022_SP02B	15	u1	800	13.000	Not Achieved						
SCPT17-02B	17-05022_SP02B	15	u1	2550	13.525	Not Achieved						
SCPT17-02B	17-05022_SP02B	15	u1	610	30.175	Not Achieved						

a. Time is relative to where umax occurred
b. Houlsby and Teh, 1991
c. This dissipation has potentially reached an equilibrium pore pressure of 25.0 m.



Trace Summary:				
	Filename: 17-05022_SP01.PPF	U Min: 24.3 m	WT: 6.936 m / 22.756 ft	T(50): 626.9 s
	Depth: 13.000 m / 42.650 ft	U Max: 52.9 m	Ueq: 6.1 m	Ir: 100
	Duration: 1000.0 s		U(50): 29.50 m	Ch: 1.1 sq cm/min

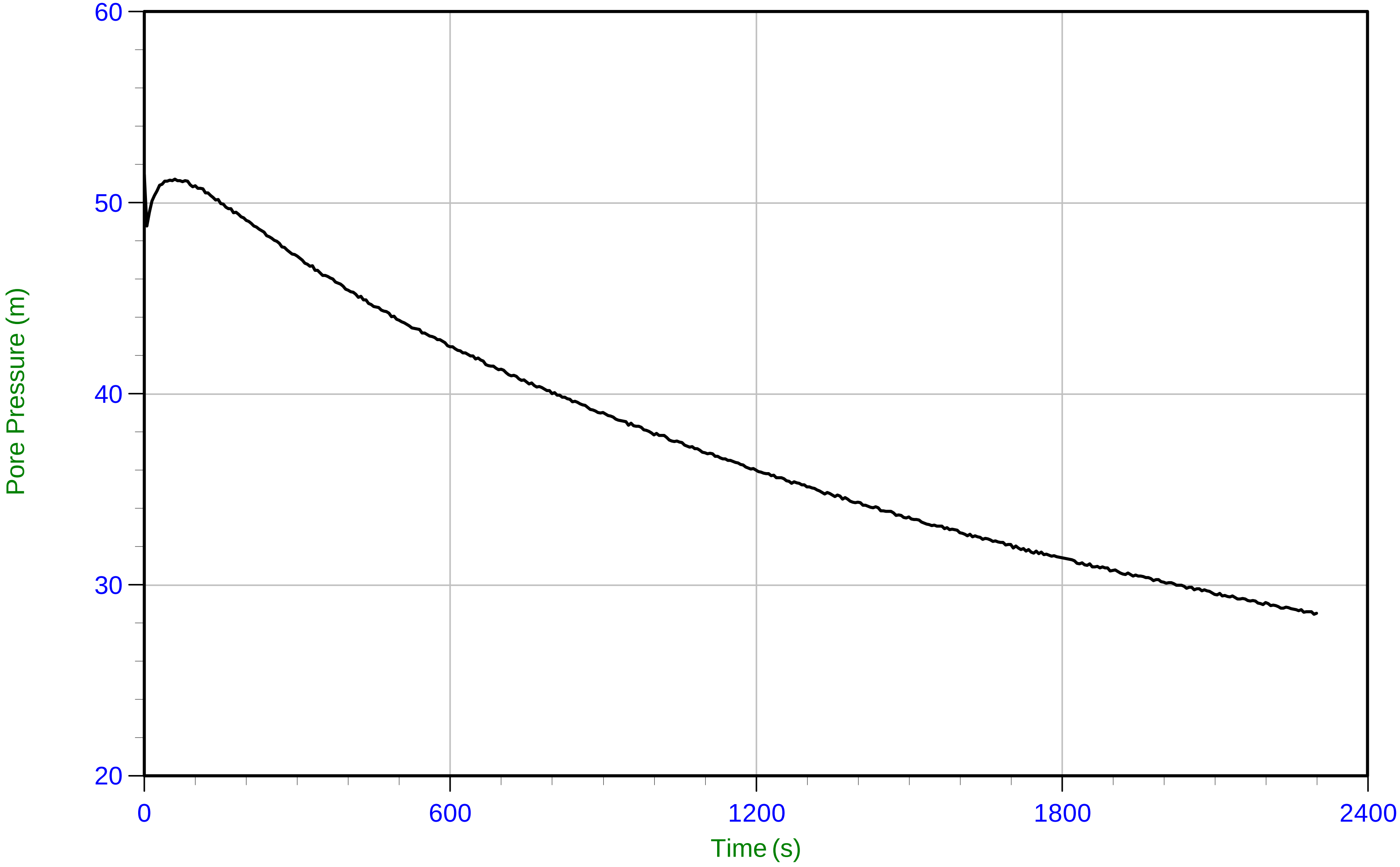
Thurber Engineering	Job No: 17-05022 Date: 05/24/2017 09:28 Site: Hwy 31 and 401	Sounding: SCPT17-01 Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 17-05022_SP01.PPF
Depth: 18.000 m / 59.054 ft
Duration: 1550.0 s

U Min: -5.5 m
U Max: 25.0 m



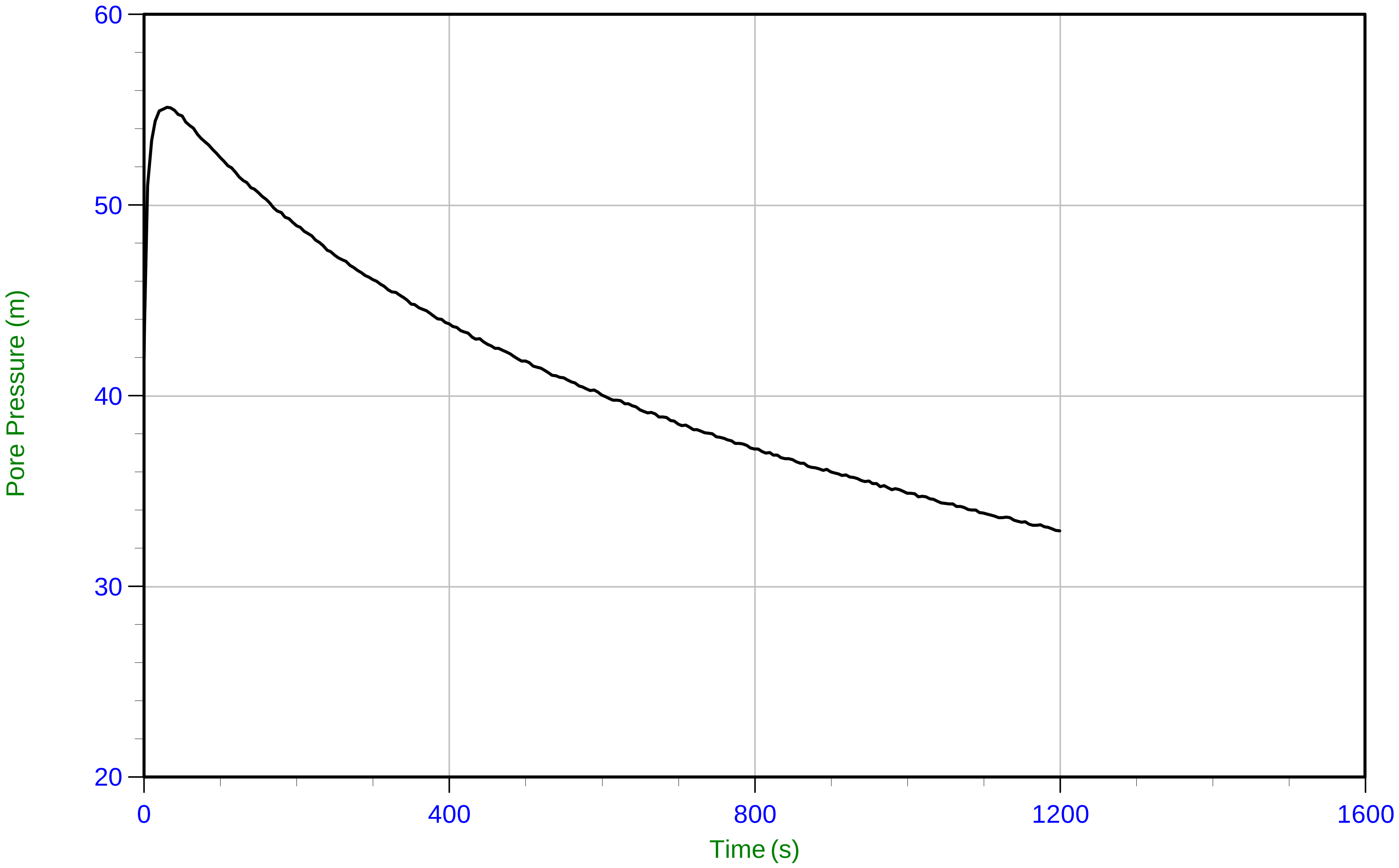
Trace Summary:

Filename: 17-05022_SP01.PPF
Depth: 18.825 m / 61.761 ft
Duration: 2300.0 s

U Min: 28.5 m
U Max: 51.5 m

WT: 6.936 m / 22.756 ft
Ueq: 11.9 m
U(50): 31.68 m

T(50): 1753.4 s
Ir: 100
Ch: 0.4 sq cm/min



Trace Summary:

Filename: 17-05022_SP01.PPF
Depth: 19.825 m / 65.042 ft
Duration: 1200.0 s

U Min: 32.9 m
U Max: 55.1 m

WT: 6.936 m / 22.756 ft
Ueq: 12.9 m
U(50): 34.02 m

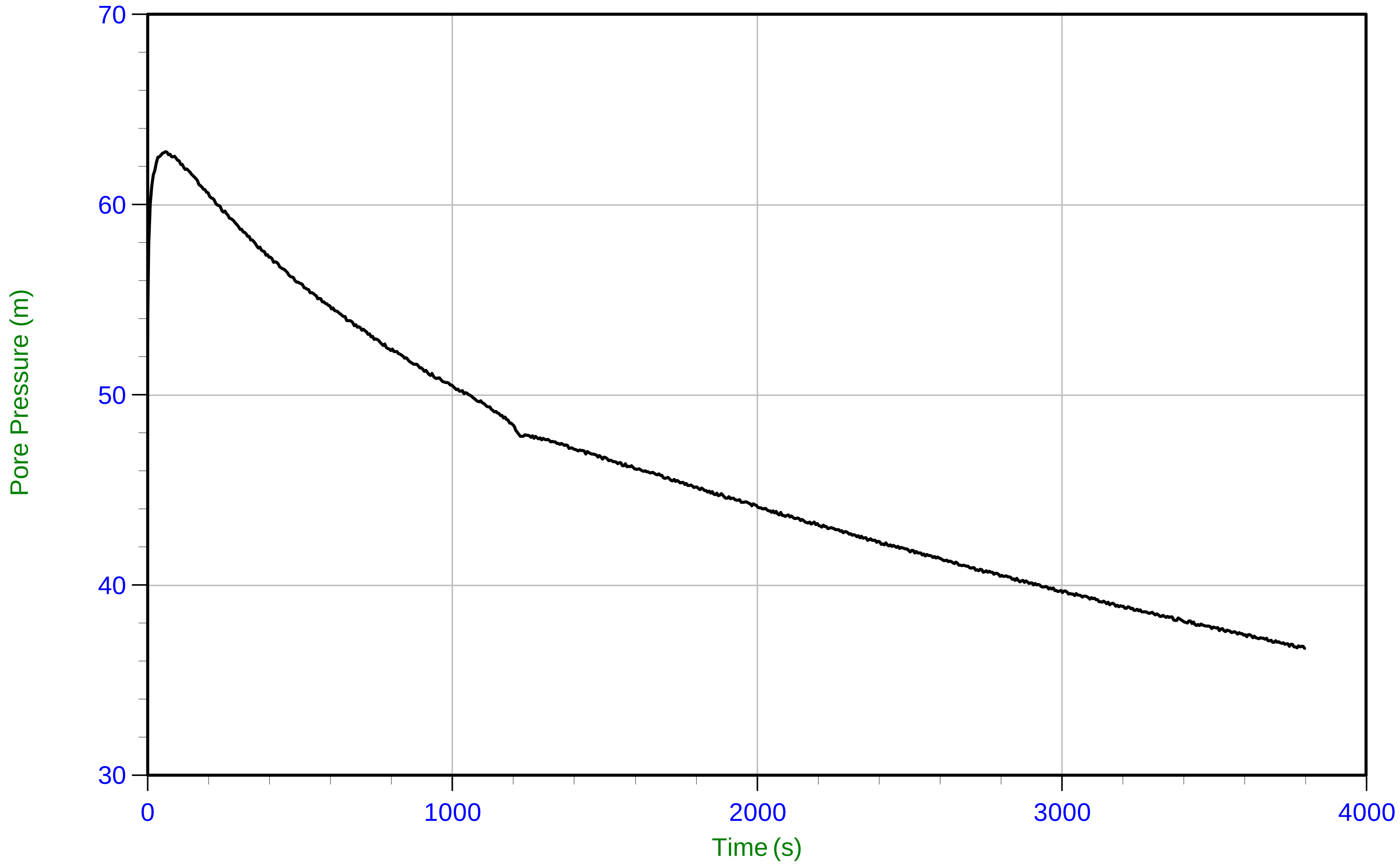
T(50): 1053.1 s
Ir: 100
Ch: 0.7 sq cm/min

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 05/24/2017 09:28
Site: Hwy 31 and 401

Sounding: SCPT17-01
Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 17-05022_SP01.PPF
Depth: 23.000 m / 75.458 ft
Duration: 3800.0 s

U Min: 36.7 m
U Max: 62.8 m

WT: 6.936 m / 22.756 ft
Ueq: 16.1 m
U(50): 39.42 m

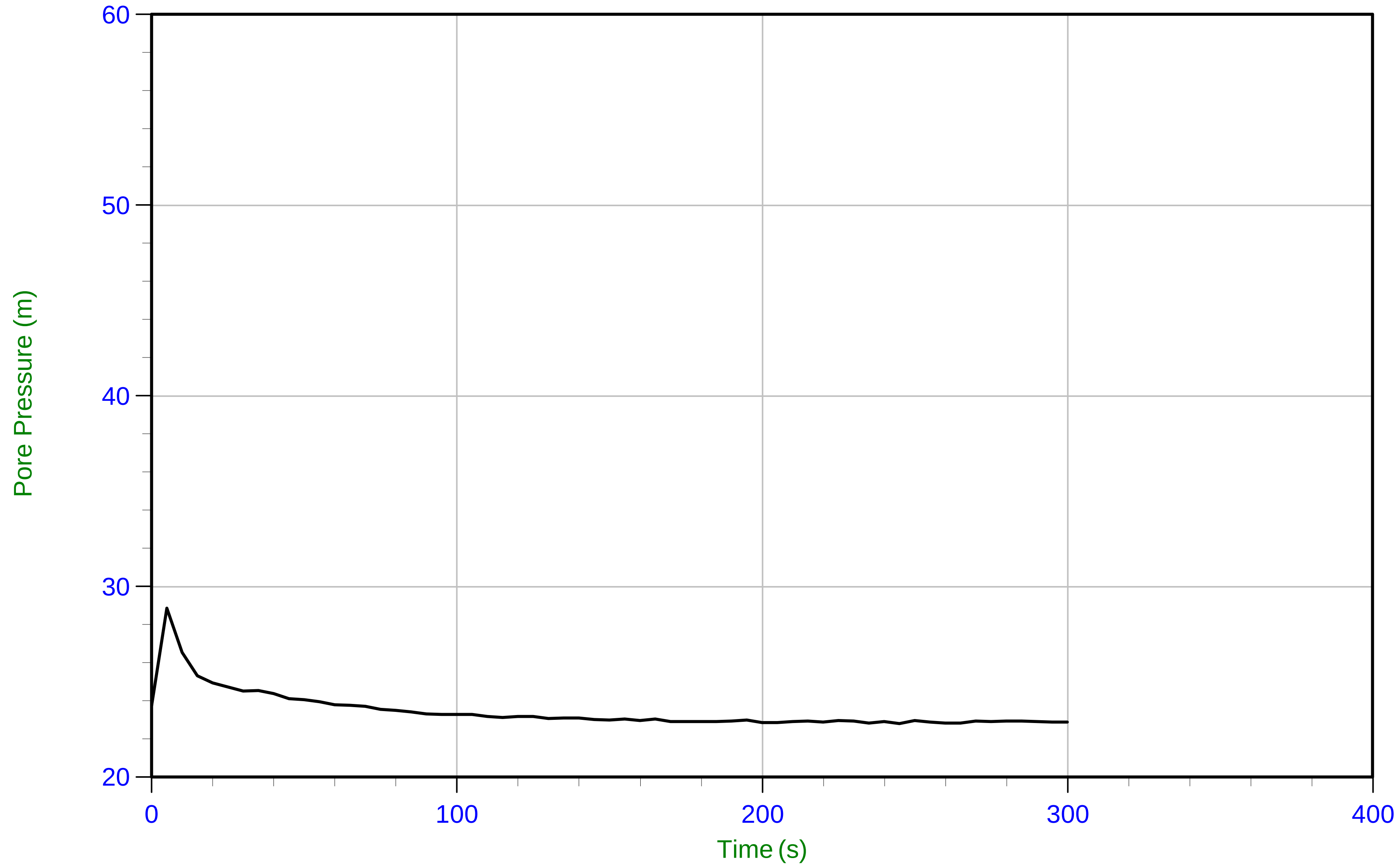
T(50): 3006.8 s
Ir: 100
Ch: 0.2 sq cm/min

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 05/24/2017 09:28
Site: Hwy 31 and 401

Sounding: SCPT17-01
Cone: 379:T1500F15U500 Area=15 cm²

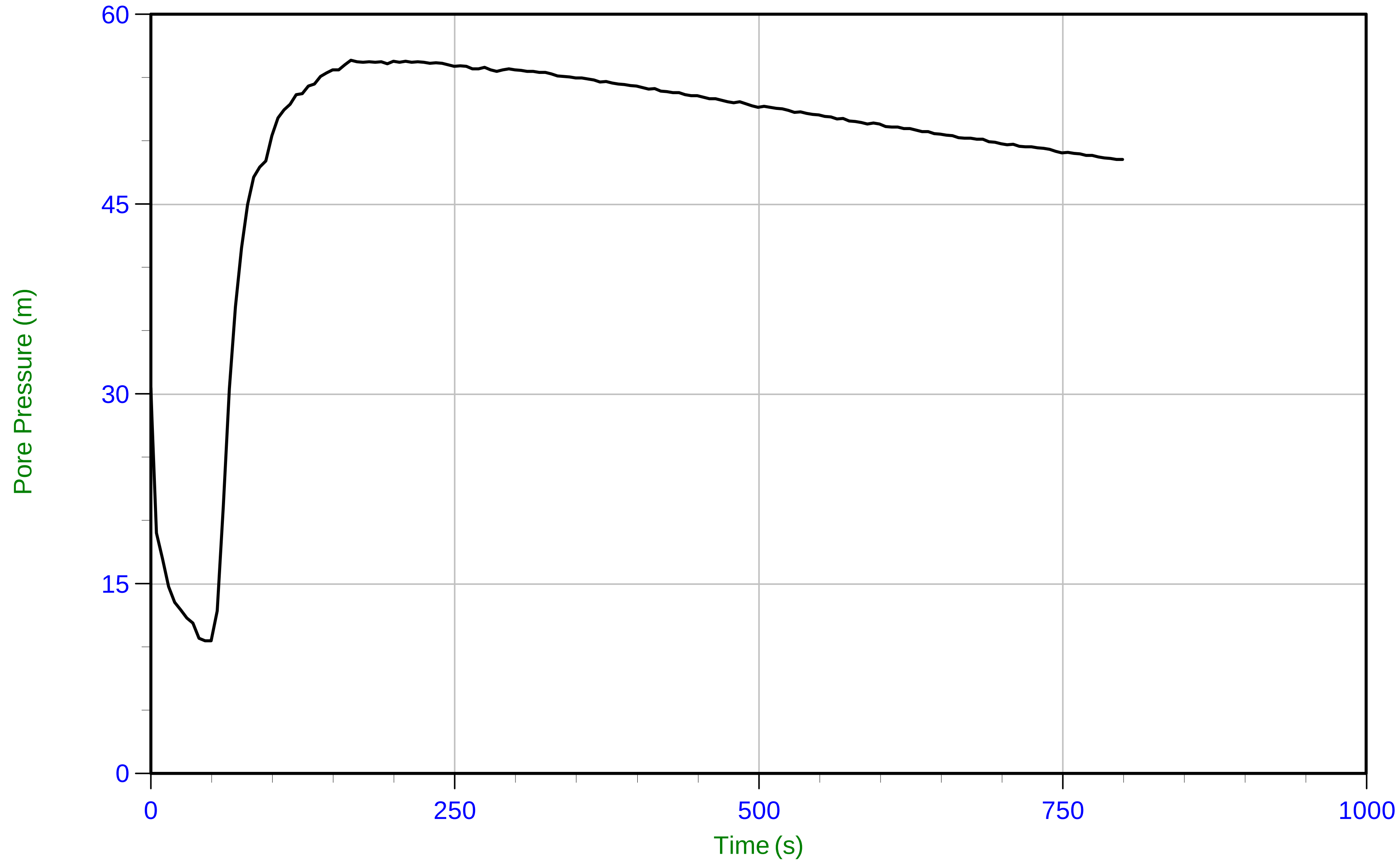


Trace Summary:

Filename: 17-05022_SP01.PPF
Depth: 29.700 m / 97.440 ft
Duration: 300.0 s

U Min: 22.8 m
U Max: 28.9 m

WT: 6.936 m / 22.756 ft
Ueq: 22.8 m



Trace Summary:

Filename: 17-05022_SP02B.PPF
Depth: 13.000 m / 42.650 ft
Duration: 800.0 s

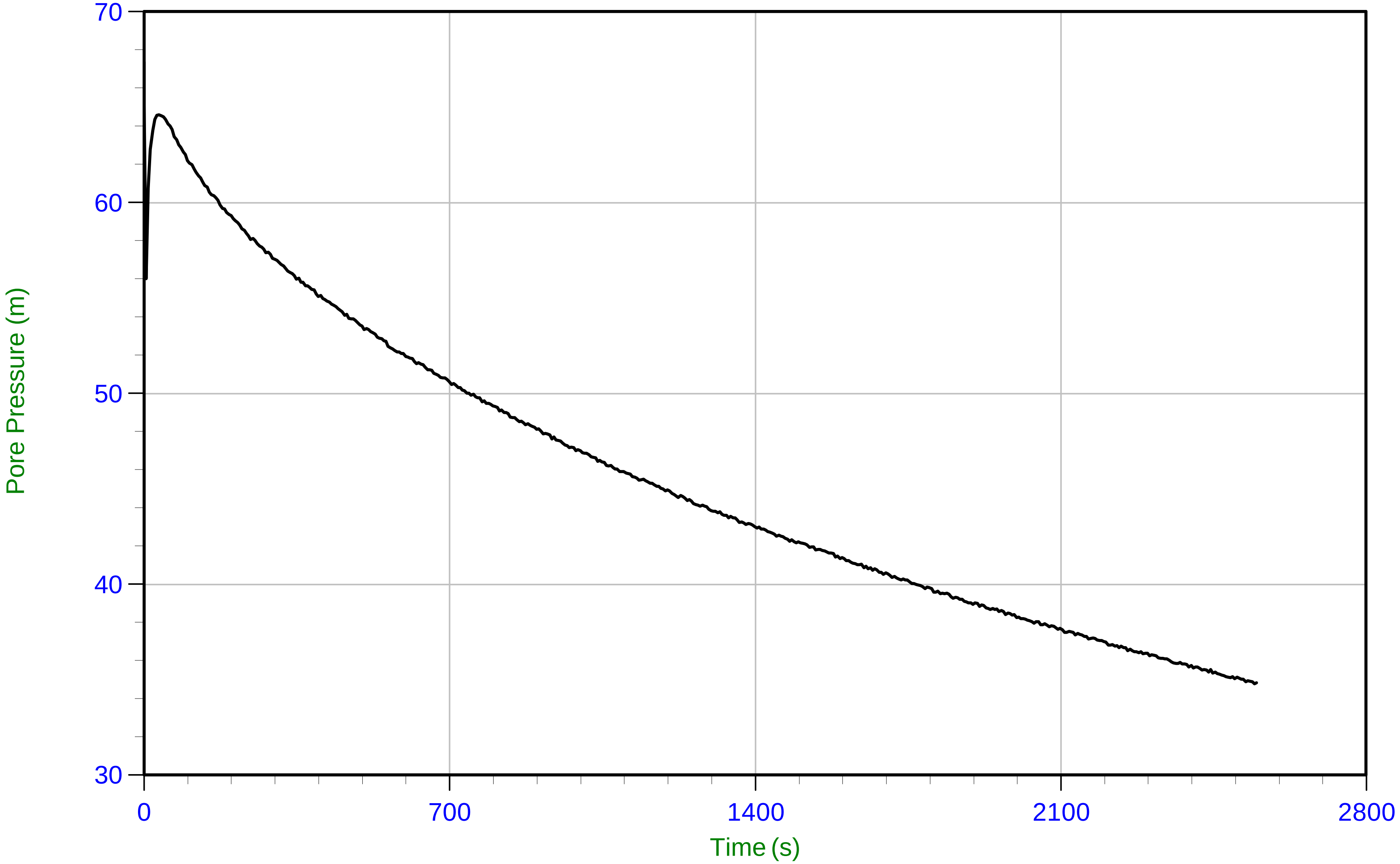
U Min: 10.5 m
U Max: 56.4 m

CONETEC

Thurber Engineering

Job No: 17-05022
Date: 05/24/2017 17:13
Site: Hwy 31 and 401

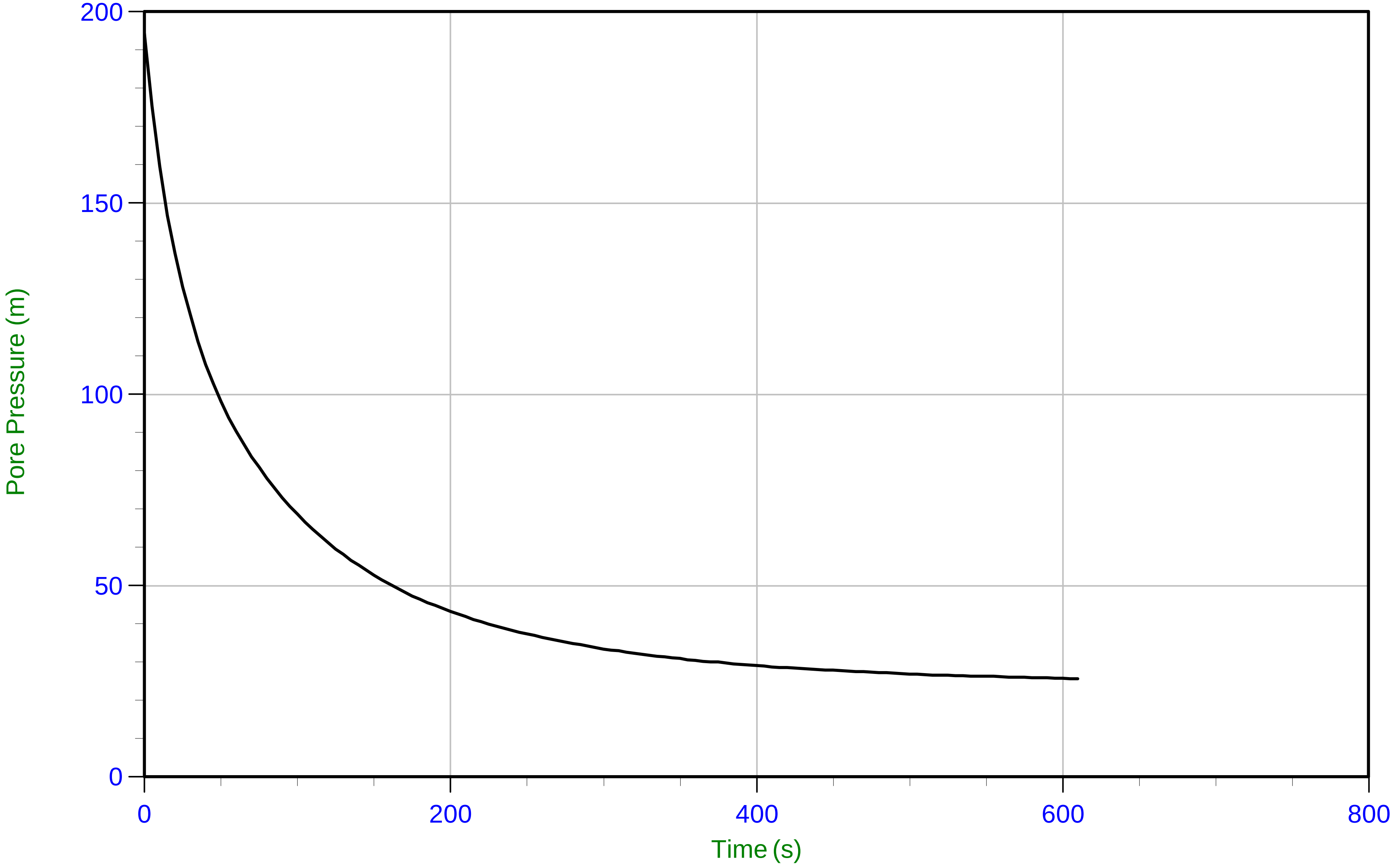
Sounding: SCPT17-02B
Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 17-05022_SP02B.PPF
Depth: 13.525 m / 44.373 ft
Duration: 2550.0 s

U Min: 34.8 m
U Max: 67.4 m



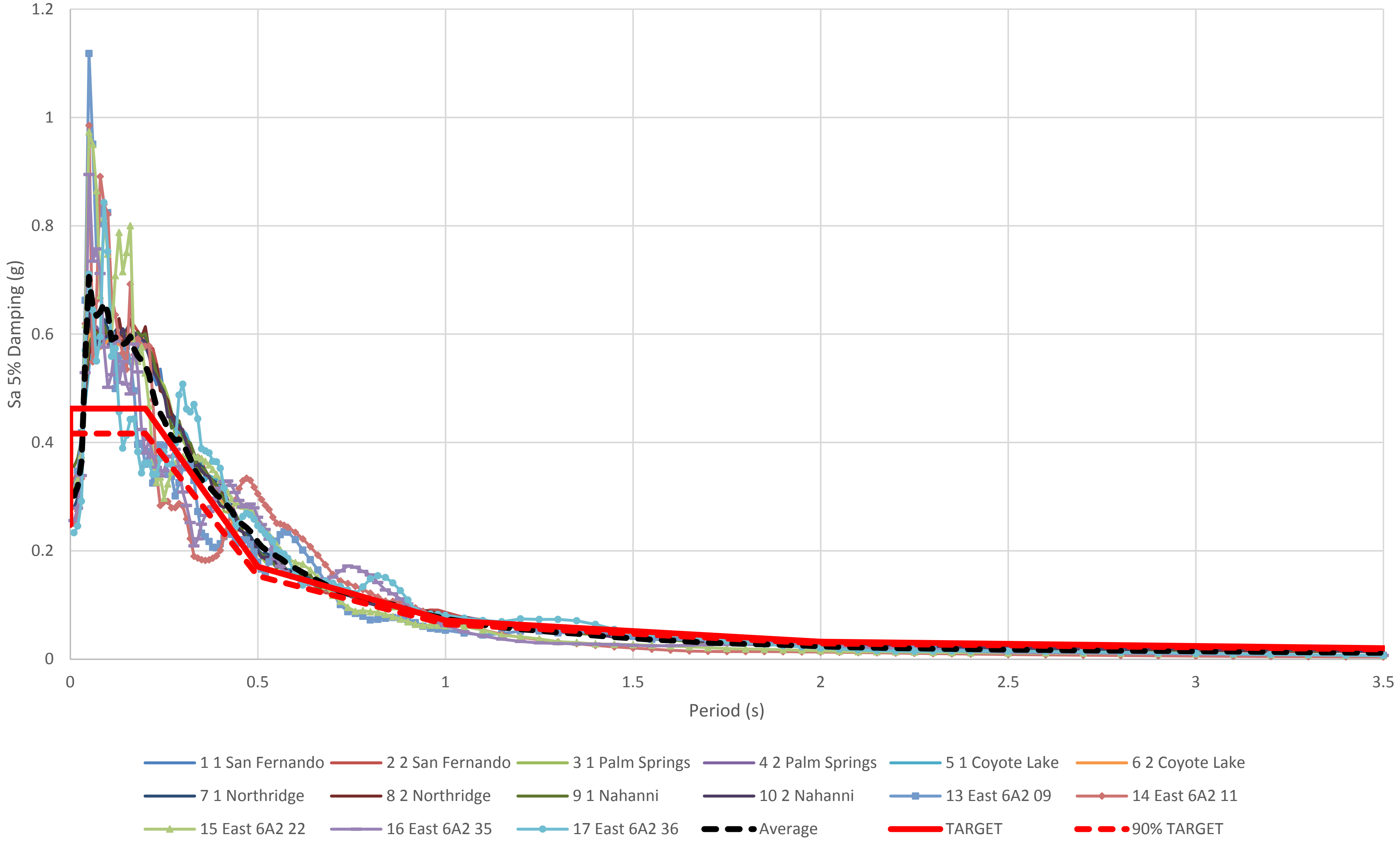
Trace Summary:

Filename: 17-05022_SP02B.PPF
Depth: 30.175 m / 98.998 ft
Duration: 610.0 s

U Min: 25.7 m
U Max: 194.3 m

APPENDIX E

Seismic Analysis Results



CLIENT
WSP Canada Group Limited

PROJECT
County Road 31 Underpass Replacement
Site No. 31-204
Highway 401, Morrisburg, Ontario

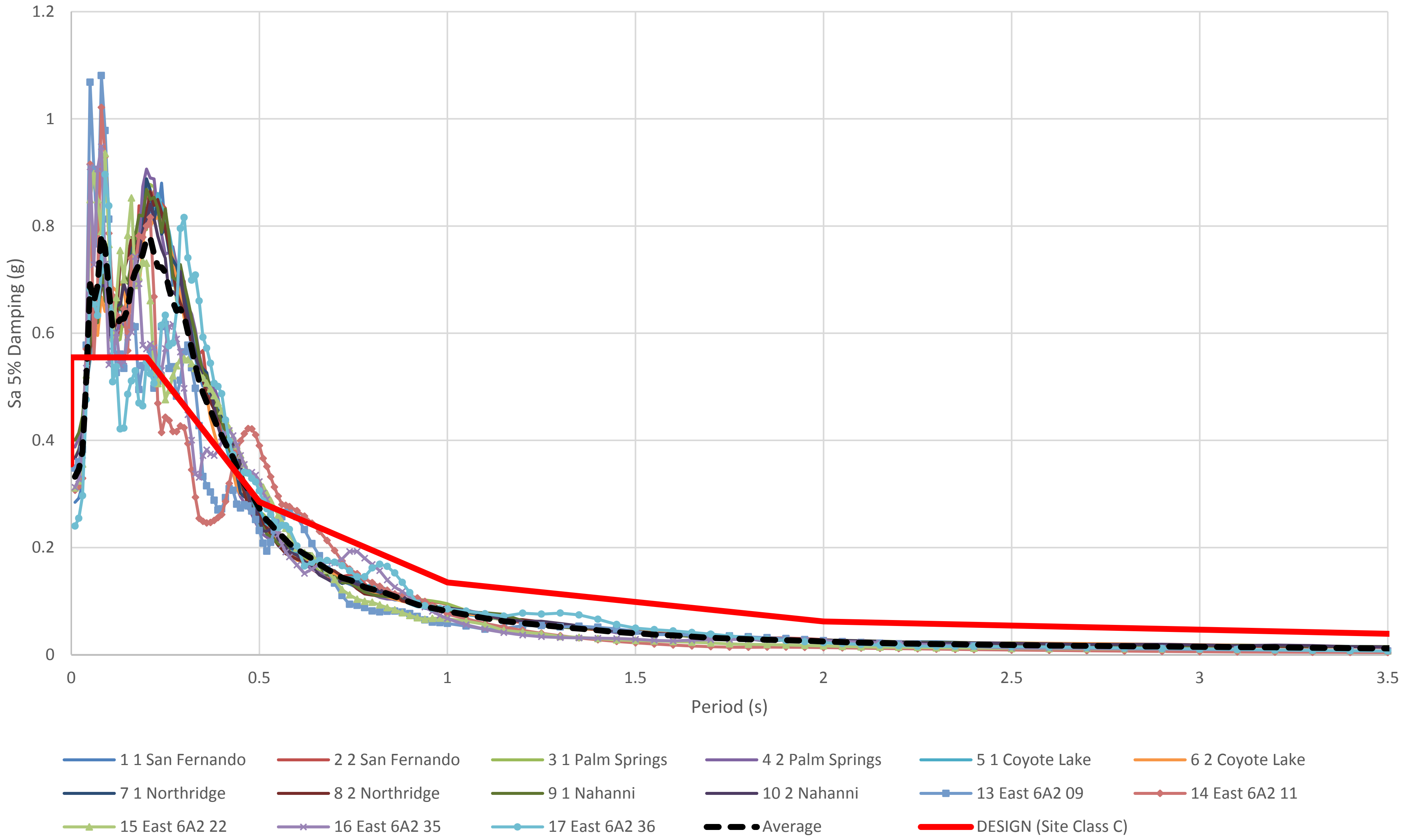
CONSULTANT

GOLDER

YYYY-MM-DD	2017/2/21
PREPARED	SG
DESIGN	SG
REVIEW	MJK
APPROVED	

TITLE
Site Class B Scaled Input Response Spectra

PROJECT No.	Phase	Rev.	Figure
12-1121-0099	**1750**	**1**	**E1**



CLIENT
WSP Canada Group Limited

PROJECT
County Road 31 Underpass Replacement
Site No. 31-204
Highway 401, Morrisburg, Ontario

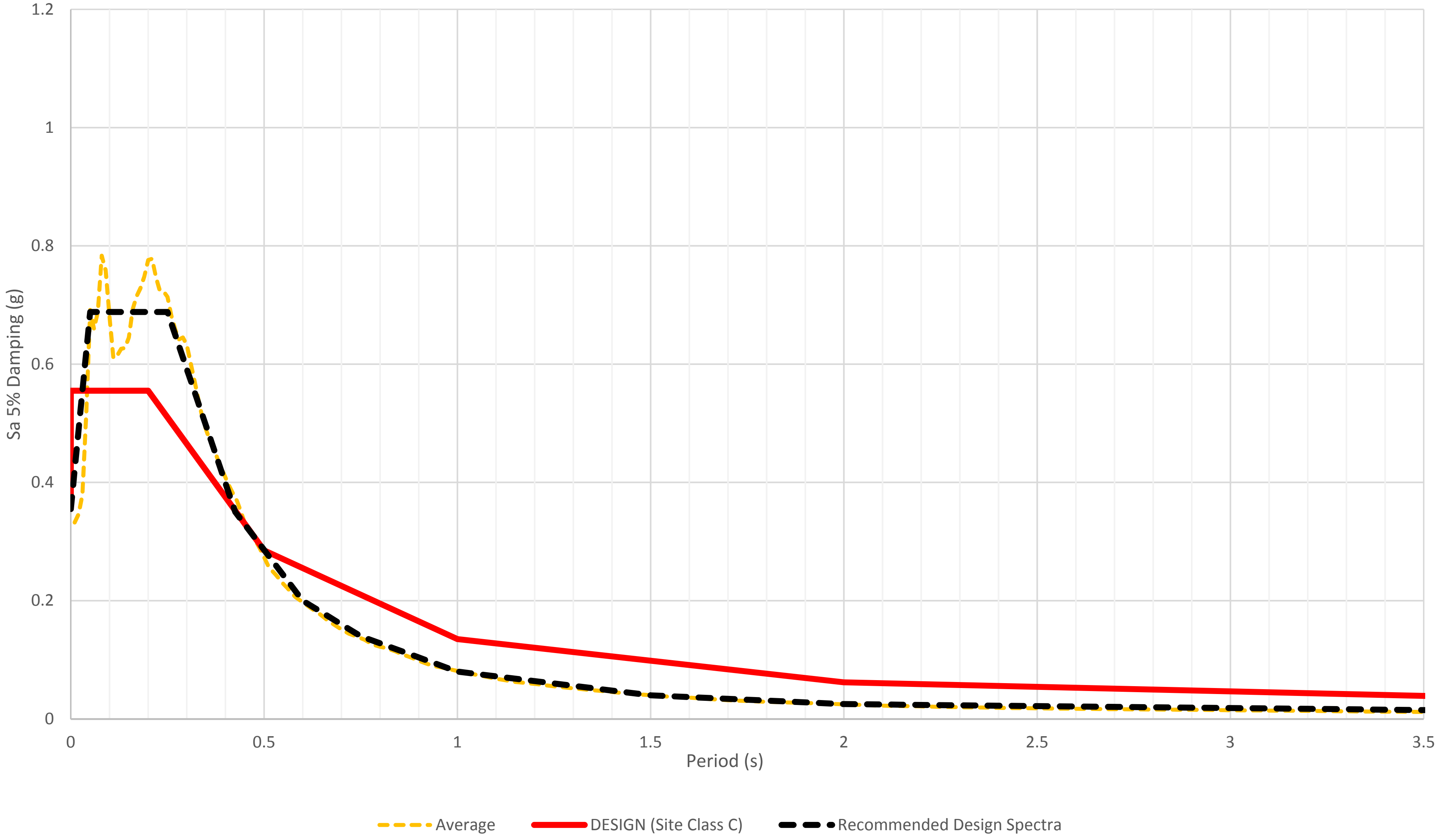
CONSULTANT

GOLDER

YYYY-MM-DD	2017/2/21
PREPARED	SG
DESIGN	SG
REVIEW	MJK
APPROVED	

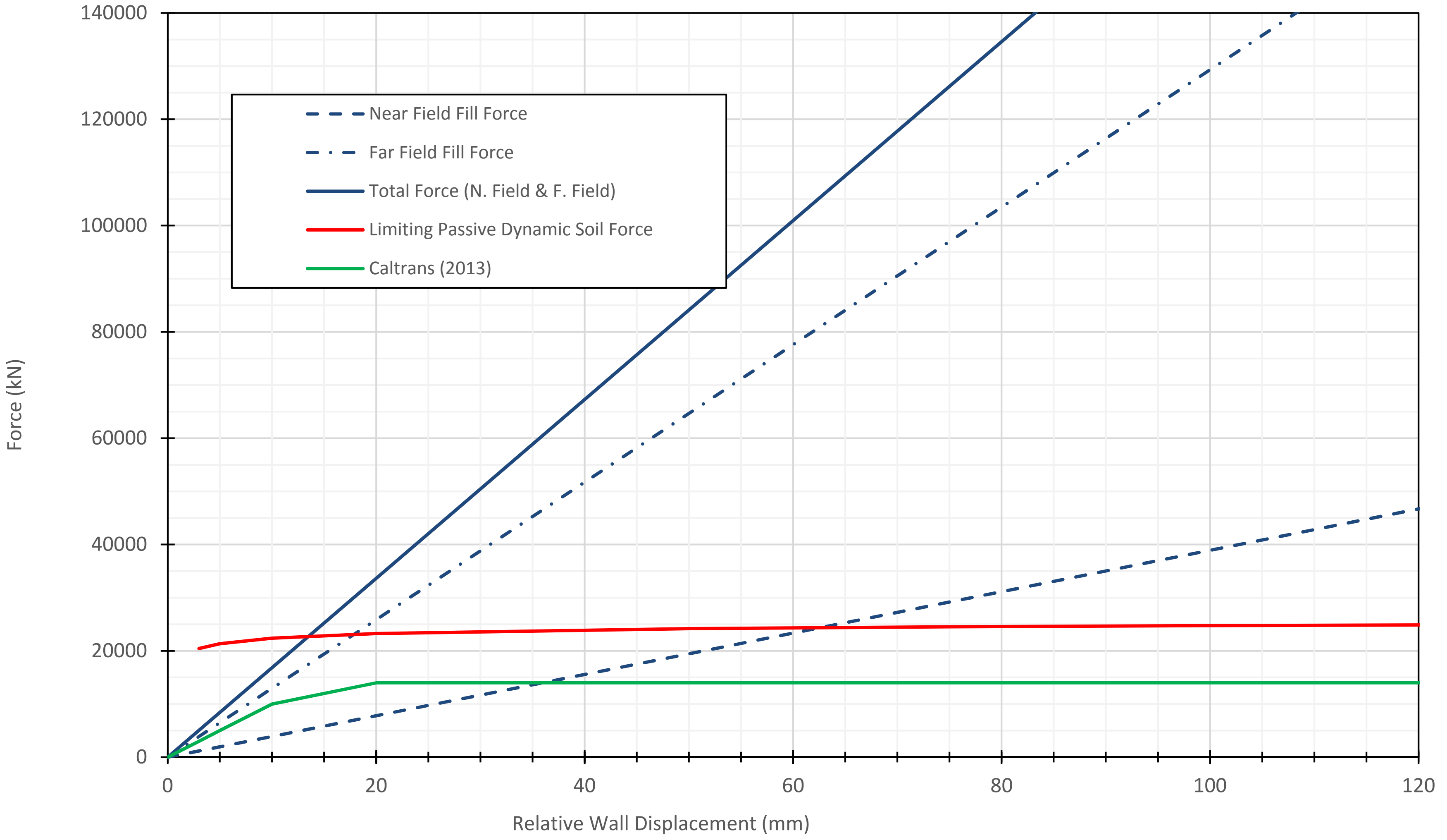
TITLE
Shake 2000 Output: Ground Surface Response Spectra

PROJECT No.	Phase	Rev.	Figure
12-1121-0099	**1750**	**1**	**E2**



CLIENT WSP Canada Group Limited		PROJECT County Road 31 Underpass Replacement Site No. 31-204 Highway 401, Morrisburg, Ontario	
CONSULTANT GOLDER	YYYY-MM-DD	2017/2/21	TITLE **Recommended Site-Specific Design Spectra**
	PREPARED	SG	
	DESIGN	SG	
	REVIEW	MJK	
	APPROVED		

PROJECT No.	Phase	Rev.	Figure
12-1121-0099	**1750**	**1**	**E3**



CLIENT
WSP Canada Group Limited

CONSULTANT
GOLDER

YYYY-MM-DD	2017/2/21
PREPARED	SG
DESIGN	SG
REVIEW	MJK
APPROVED	

PROJECT
County Road 31 Underpass Replacement
Site No. 31-204
Highway 401, Morrisburg, Ontario

TITLE
Passive Abutment Force vs. Relative Wall Displacement

PROJECT No.	Phase	Rev.	Figure
12-1121-0099	1750	1	E4

APPENDIX F

P-y Curves

P-y CURVES

Integral Abutments - Static Loading

Site 31-204, Highway 401 / County Road 31

Figure F1

SUMMARY OF P-y CURVES FOR A H-Pile 310x110 - Static Loading Condition - Abutment Locations - Integral Abutment Option (3 m of CSP)

Description	Loose Sand in CSP												Existing Embankment Fill			
Depth (z) *	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m	
Elevation	Elev. 83.5 m		Elev. 83.0 m		Elev. 82.5 m		Elev. 82.0 m		Elev. 81.5 m		Elev. 81.0 m		Elev. 80.5 m		Elev. 80.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	2.428	0.003	7.368	0.004	14.820	0.005	24.784	0.006	37.259	0.007	51.921	0.001	53.759	0.001	71.630
	0.004	4.681	0.006	14.203	0.008	28.567	0.010	47.772	0.012	71.818	0.014	100.080	0.002	103.623	0.002	138.071
	0.006	6.629	0.009	20.114	0.012	40.456	0.014	67.654	0.017	101.708	0.020	141.733	0.003	146.750	0.003	195.536
	0.008	8.215	0.011	24.926	0.015	50.133	0.019	83.836	0.023	126.036	0.027	175.635	0.004	181.852	0.004	242.307
	0.009	9.443	0.014	28.651	0.019	57.627	0.024	96.368	0.029	144.876	0.034	201.888	0.005	209.035	0.005	278.526
	0.011	10.357	0.017	31.427	0.023	63.209	0.029	105.704	0.035	158.911	0.041	221.447	0.006	229.286	0.006	305.510
	0.013	11.019	0.020	33.436	0.027	67.250	0.034	112.460	0.041	169.068	0.047	235.602	0.007	243.941	0.008	325.037
	0.015	11.489	0.023	34.859	0.031	70.113	0.039	117.248	0.046	176.266	0.054	245.632	0.008	254.327	0.009	338.875
	0.017	11.816	0.026	35.853	0.035	72.111	0.043	120.590	0.052	181.290	0.061	252.633	0.009	261.576	0.010	348.534
	0.019	12.042	0.029	36.540	0.038	73.492	0.048	122.899	0.058	184.761	0.068	257.470	0.009	266.584	0.011	355.207
	0.021	12.197	0.032	37.010	0.042	74.439	0.053	124.482	0.064	187.142	0.074	260.787	0.010	270.019	0.012	359.784
	0.023	12.303	0.034	37.332	0.046	75.085	0.058	125.563	0.070	188.766	0.081	263.051	0.011	272.362	0.013	362.907
	0.025	12.375	0.037	37.550	0.050	75.524	0.063	126.298	0.075	189.871	0.088	264.590	0.012	273.956	0.014	365.030
	0.026	12.424	0.040	37.698	0.054	75.822	0.068	126.796	0.081	190.620	0.095	265.635	0.013	275.038	0.015	366.472
	0.028	12.457	0.043	37.799	0.058	76.024	0.072	127.134	0.087	191.128	0.101	266.343	0.014	275.771	0.016	367.448
	0.030	12.480	0.046	37.867	0.062	76.161	0.077	127.363	0.093	191.472	0.108	266.822	0.015	276.267	0.017	368.108

Description	Compact Sandy Silt to Silty Sand (Till)										Very Loose to Loose Silt (Till)									
Depth (z) *	z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0m		z= 11.0 m		z= 12.0 m	
Elevation	Elev. 79.5 m		Elev. 79.0 m		Elev. 78.5 m		Elev. 78.0 m		Elev. 77.0 m		Elev. 76.0 m		Elev. 75.0 m		Elev. 74.0 m		Elev. 73.0 m		Elev. 72.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	65.495	0.002	76.806	0.002	84.735	0.002	92.664	0.002	116.452	0.002	132.311	0.002	148.170	0.002	164.029	0.002	179.887	0.002	195.746
	0.003	126.245	0.004	148.047	0.004	163.332	0.004	178.616	0.004	224.469	0.004	255.037	0.004	285.606	0.003	316.174	0.003	346.743	0.003	377.311
	0.005	178.788	0.005	209.664	0.005	231.310	0.005	252.955	0.005	317.892	0.005	361.183	0.005	404.474	0.005	447.765	0.005	491.056	0.005	534.347
	0.007	221.552	0.007	259.814	0.007	286.637	0.007	313.460	0.007	393.929	0.007	447.575	0.007	501.221	0.007	554.866	0.007	608.512	0.007	662.158
	0.009	254.669	0.009	298.651	0.009	329.483	0.009	360.316	0.009	452.813	0.009	514.478	0.009	576.142	0.009	637.807	0.009	699.472	0.009	761.137
	0.010	279.341	0.011	327.584	0.011	361.403	0.011	395.223	0.011	496.681	0.011	564.320	0.011	631.958	0.010	699.597	0.010	767.236	0.010	834.875
	0.012	297.197	0.013	348.522	0.013	384.504	0.013	420.485	0.012	528.428	0.012	600.390	0.012	672.352	0.012	744.314	0.012	816.276	0.012	888.238
	0.014	309.849	0.015	363.360	0.014	400.873	0.014	438.386	0.014	550.924	0.014	625.950	0.014	700.975	0.014	776.001	0.014	851.027	0.014	926.053
	0.016	318.681	0.016	373.717	0.016	412.300	0.016	450.882	0.016	566.628	0.016	643.792	0.016	720.957	0.016	798.121	0.016	875.285	0.016	952.450
	0.017	324.782	0.018	380.872	0.018	420.193	0.018	459.514	0.018	577.477	0.018	656.118	0.018	734.760	0.017	813.402	0.017	892.043	0.017	970.685
	0.019	328.967	0.020	385.779	0.020	425.607	0.020	465.434	0.020	584.916	0.019	664.571	0.019	744.226	0.019	823.881	0.019	903.536	0.019	983.191
	0.021	331.822	0.022	389.128	0.022	429.301	0.022	469.474	0.021	589.994	0.021	670.340	0.021	750.686	0.021	831.032	0.021	911.379	0.021	991.725
	0.023	333.764	0.024	391.405	0.023	431.813	0.023	472.222	0.023	593.446	0.023	674.263	0.023	755.079	0.023	835.896	0.023	916.712	0.023	997.529
	0.024	335.082	0.025	392.950	0.025	433.518	0.025	474.086	0.025	595.789	0.025	676.925	0.025	758.060	0.024	839.196	0.024	920.331	0.024	1001.467
	0.026	335.974	0.027	393.997	0.027	434.673	0.027	475.349	0.027	597.376	0.026	678.728	0.026	760.080	0.026	841.431	0.026	922.783	0.026	1004.135
	0.028	336.578	0.029	394.706	0.029	435.455	0.029	476.204	0.028	598.450	0.028	679.948	0.028	761.446	0.028	842.944	0.028	924.442	0.028	1005.940

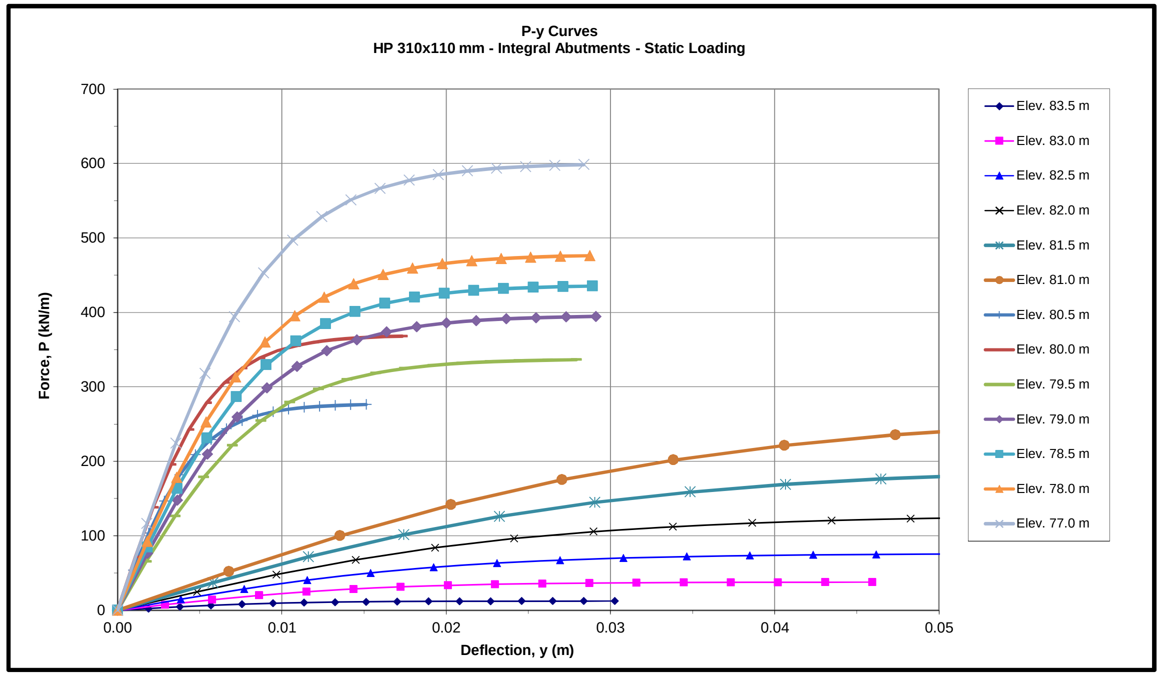
Description	Very Loose to Loose Silt (Till)															
Depth (z) *	z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m	
Elevation	Elev. 71.0 m		Elev. 70.0 m		Elev. 69.0 m		Elev. 68.0 m		Elev. 67.0 m		Elev. 66.0 m		Elev. 65.0 m		Elev. 64.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0	0	0.000	0.000	0.000	0.000	0.000	0.000	0	0	0.000	0.000
	0.002	211.605	0.002	227.463	0.00173	243.3219	0.002	259.181	0.002	211.605	0.002	227.463	0.00173	243.3219	0.002	259.181
	0.003	407.880	0.003	438.448	0.00346	469.0165	0.003	499.585	0.003	407.880	0.003	438.448	0.00346	469.01654	0.003	499.585
	0.005	577.638	0.005	620.929	0.00519	664.2196	0.005	707.511	0.005	577.638	0.005	620.929	0.00519	664.21957	0.005	707.511
	0.007	715.804	0.007	769.450	0.00692	823.0958	0.007	876.742	0.007	715.804	0.007	769.450	0.00692	823.09575	0.007	876.742
	0.009	822.802	0.009	884.466	0.00865	946.131	0.009	1007.796	0.009	822.802	0.009	884.466	0.00865	946.13104	0.009	1007.796
	0.010	902.513	0.010	970.152	0.01038	1037.791	0.010	1105.430	0.010	902.513	0.010	970.152	0.01038	1037.7909	0.010	1105.430
	0.012	960.201	0.012	1032.163	0.01211	1104.125	0.012	1176.087	0.012	960.201	0.012	1032.163	0.01211	1104.1249	0.012	1176.087
	0.014	1001.078	0.014	1076.104	0.01384	1151.13	0.014	1226.155	0.014	1001.078	0.014	1076.104	0.01384	1151.1297	0.014	1226.155
	0.016	1029.614	0.016	1106.778	0.01556	1183.943	0.016	1261.107	0.016	1029.614	0.016	1106.778	0.01556	1183.9427	0.016	1261.107
	0.017	1049.327	0.017	1127.968	0.01729	1206.61	0.017	1285.252	0.017	1049.327	0.017	1127.968	0.01729	1206.6099	0.017	1285.252
	0.019	1062.846	0.019	1142.500	0.01902	1222.155	0.019	1301.810	0.019	1062.846	0.019	1142.500	0.01902	1222.1553	0.019	1301.810
	0.021	1072.071	0.021	1152.417	0.02075	1232.764	0.021	1313.110	0.021	1072.071	0.021	1152.417	0.02075	1232.7636	0.021	1313.110
	0.023	1078.345	0.023	1159.162	0.02248	1239.978	0.022	1320.795	0.023	1078.345	0.023	1159.162	0.02248	1239.9781	0.022	1320.795
	0.024	1082.602	0.024	1163.738	0.02421	1244.873	0.024	1326.009	0.024	1082.602	0.024	1163.738	0.02421	1244.8734	0.024	1326.009
	0.026	1085.486	0.026	1166.838	0.02594	1248.19	0.026	1329.541	0.026	1085.486	0.026	1166.838	0.02594	1248.1897	0.026	1329.541
	0.028	1087.438	0.028	1168.936	0.02767	1250.434	0.028	1331.932	0.028	1087.438	0.028	1168.936	0.02767	1250.4339	0.028	1331.932

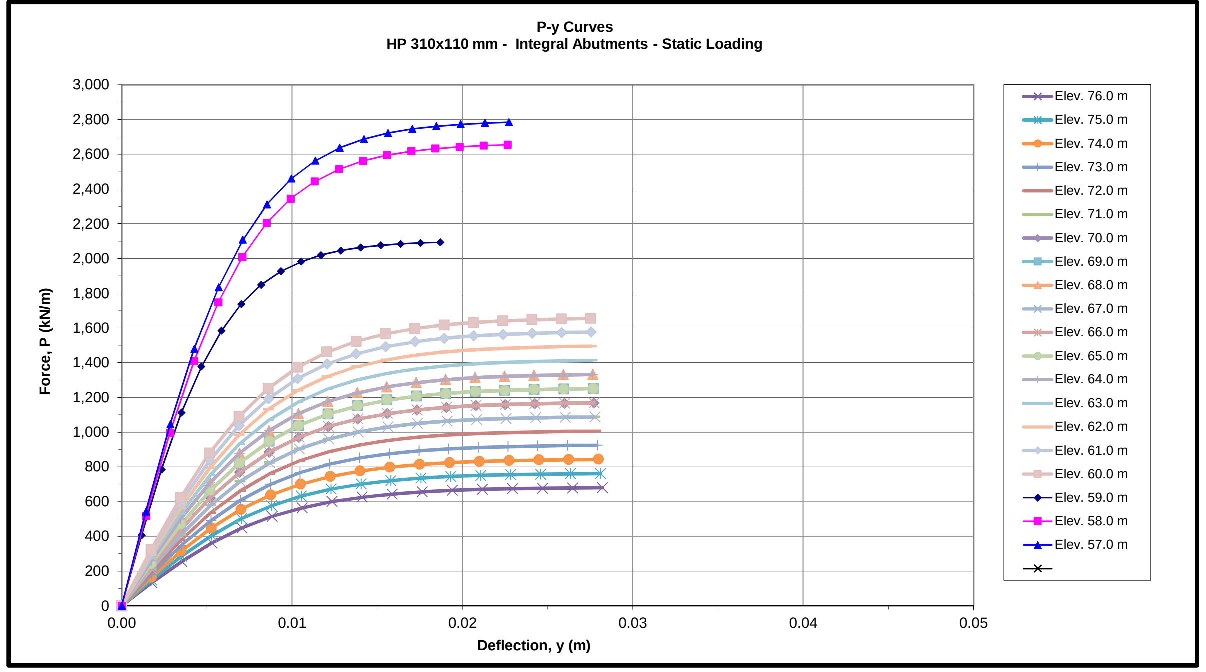
Description									Compact Sand/Very Stiff Silty Clay					
Depth (z) *	z= 21.0 m		z= 22.0 m		z= 23.0 m		z= 24.0 m		z= 25.0 m		z= 26.0 m		z= 27.0 m	
Elevation	Elev. 63.0 m		Elev. 62.0 m		Elev. 61.0 m		Elev. 60.0 m		Elev. 59.0 m		Elev. 58.0 m		Elev. 57.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	275.039	0.002	290.898	0.002	306.757	0.002	321.822	0.001	407.337	0.001	516.595	0.001	541.872
	0.003	530.154	0.003	560.722	0.003	591.291	0.003	620.331	0.002	785.165	0.003	995.765	0.003	1044.488
	0.005	750.802	0.005	794.093	0.005	837.384	0.005	878.510	0.004	1111.948	0.004	1410.200	0.004	1479.200
	0.007	930.387	0.007	984.033	0.007	1037.679	0.007	1088.643	0.005	1377.918	0.006	1747.508	0.006	1833.013
	0.009	1069.461	0.009	1131.125	0.009	1192.790	0.009	1251.372	0.006	1583.887	0.007	2008.724	0.007	2107.010
	0.010	1173.069	0.010	1240.707	0.010	1308.346	0.010	1372.603	0.007	1737.332	0.008	2203.326	0.009	2311.134
	0.012	1248.049	0.012	1320.011	0.012	1391.973	0.012	1460.338	0.008	1848.380	0.010	2344.159	0.010	2458.858
	0.014	1301.181	0.014	1376.207	0.014	1451.233	0.014	1522.507	0.009	1927.069	0.011	2443.955	0.011	2563.537
	0.016	1338.271	0.016	1415.436	0.015	1492.600	0.015	1565.906	0.011	1982.000	0.013	2513.620	0.013	2636.610
	0.017	1363.893	0.017	1442.535	0.017	1521.177	0.017	1595.886	0.012	2019.946	0.014	2561.744	0.014	2687.089
	0.019	1381.465	0.019	1461.120	0.019	1540.775	0.019	1616.447	0.013	2045.971	0.016	2594.749	0.016	2721.709
	0.021	1393.456	0.021	1473.802	0.021	1554.149	0.021	1630.478	0.014	2063.730	0.017	2617.271	0.017	2745.333
	0.022	1401.611	0.022	1482.428	0.022	1563.244	0.022	1640.020	0.015	2075.807	0.018	2632.588	0.018	2761.400
	0.024	1407.144	0.024	1488.280	0.024	1569.415	0.024	1646.494	0.016	2084.002	0.020	2642.981	0.020	2772.301
	0.026	1410.893	0.026	1492.245	0.026	1573.596	0.026	1650.880	0.018	2089.554	0.021	2650.022	0.021	2779.687
	0.028	1413.430	0.028	1494.928	0.028	1576.426	0.028	1653.849	0.019	2093.311	0.023	2654.787	0.023	2784.684

NOTES: * Depth (z) is measured to be positive below the underside of the abutments (Elevation 84.0 m).

The P-y curves have been generated based on the following assumptions:

1. P-y curves are generated for vertical piles (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves have been generated for strong axis of H-pile 310x110.





Date: August 2018
Project No: 12-1121-0099 Ph. 1750

Prepared By: ARV
Checked By: KJB

GOLDER

P-y CURVES

Pier - Static Loading

Site 31-204, Highway 401 / County Road 31

Figure F2

SUMMARY OF P-y CURVES FOR A H-Pile 310x110 - Pier Location - Static Loading Condition

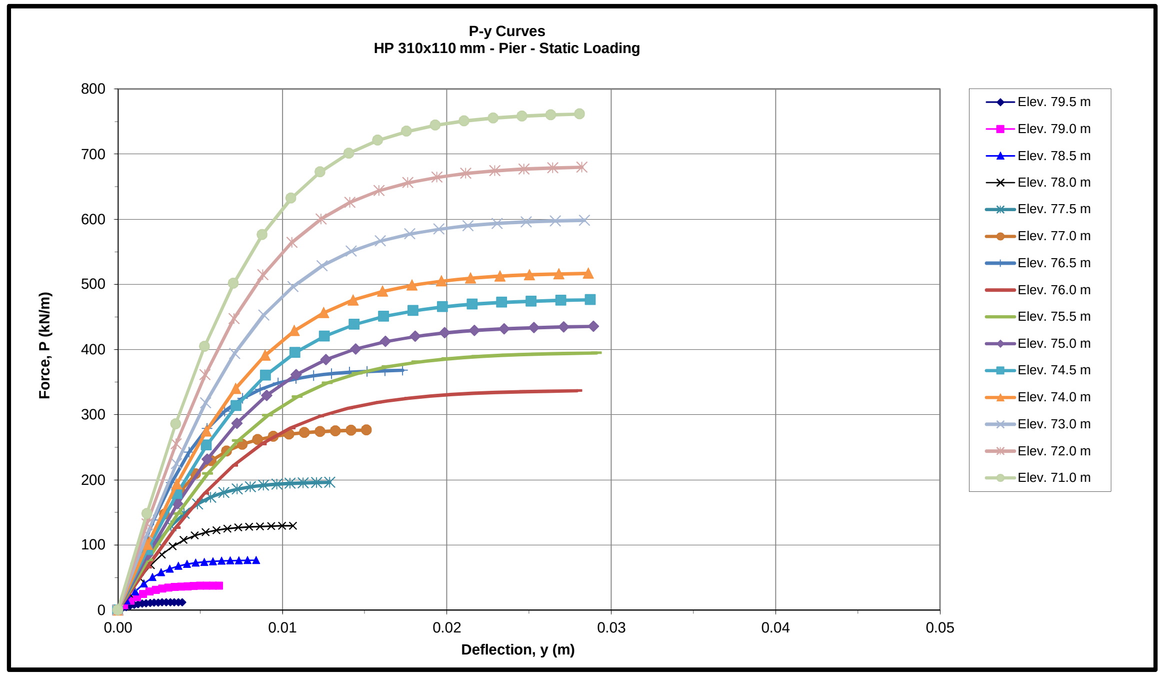
Description	**Compact Silty Sand to Sandy Silt (Till)**														**Very Loose to Loose Sand and Silt (Till)**																	
Depth (z) *	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0m		z= 11.0 m		z= 12.0 m	
Elevation	Elev. 79.5 m		Elev. 79.0 m		Elev. 78.5 m		Elev. 78.0 m		Elev. 77.5 m		Elev. 77.0 m		Elev. 76.5 m		Elev. 76.0 m		Elev. 75.5 m		Elev. 75.0 m		Elev. 74.5 m		Elev. 74.0 m		Elev. 73.0 m		Elev. 72.0 m		Elev. 71.0 m		Elev. 70.0 m		Elev. 69.0 m		Elev. 68.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	2.315	0.000	7.288	0.001	14.918	0.001	25.207	0.001	38.154	0.001	53.759	0.001	71.630	0.002	65.495	0.002	76.806	0.002	84.735	0.002	92.664	0.002	100.594	0.002	116.452	0.002	132.311	0.002	148.170	0.002	164.029	0.002	179.887	0.002	195.746
	0.000	4.462	0.001	14.047	0.001	28.756	0.001	48.588	0.002	73.544	0.002	103.623	0.002	138.071	0.003	126.245	0.004	148.047	0.004	163.332	0.004	178.616	0.004	193.900	0.004	224.469	0.004	255.037	0.004	285.606	0.003	316.174	0.003	346.743	0.003	377.311
	0.001	6.319	0.001	19.893	0.002	40.724	0.002	68.810	0.002	104.152	0.003	146.750	0.003	195.536	0.005	178.788	0.005	209.664	0.005	231.310	0.005	252.955	0.005	274.601	0.005	317.892	0.005	361.183	0.005	404.474	0.005	447.765	0.005	491.056	0.005	534.347
	0.001	7.830	0.002	24.652	0.002	50.465	0.003	85.269	0.003	129.065	0.004	181.852	0.004	242.307	0.007	221.552	0.007	259.814	0.007	286.637	0.007	313.460	0.007	340.283	0.007	393.929	0.007	447.575	0.007	501.221	0.007	554.866	0.007	608.512	0.007	662.158
	0.001	9.001	0.002	28.337	0.003	58.008	0.003	98.015	0.004	148.357	0.005	209.035	0.005	278.526	0.009	254.669	0.009	298.651	0.009	329.483	0.009	360.316	0.009	391.148	0.009	452.813	0.009	514.478	0.009	576.142	0.009	637.807	0.009	699.472	0.009	761.137
	0.001	9.873	0.002	31.082	0.003	63.628	0.004	107.511	0.005	162.730	0.006	229.286	0.006	305.510	0.010	279.341	0.011	327.584	0.011	361.403	0.011	395.223	0.011	429.042	0.011	496.681	0.011	564.320	0.011	631.958	0.010	699.597	0.010	767.236	0.010	834.875
	0.002	10.504	0.003	33.069	0.004	67.695	0.005	114.382	0.006	173.131	0.007	243.941	0.008	325.037	0.012	297.197	0.013	348.522	0.013	384.504	0.013	420.485	0.013	456.466	0.012	528.428	0.012	600.390	0.012	672.352	0.012	744.314	0.012	816.276	0.012	888.238
	0.002	10.951	0.003	34.476	0.004	70.577	0.005	119.252	0.006	180.502	0.008	254.327	0.009	338.875	0.014	309.849	0.015	363.360	0.014	400.873	0.014	438.386	0.014	475.898	0.014	550.924	0.014	625.950	0.014	700.975	0.014	776.001	0.014	851.027	0.014	926.053
	0.002	11.263	0.003	35.459	0.005	72.589	0.006	122.651	0.007	185.647	0.009	261.576	0.010	348.534	0.016	318.681	0.016	373.717	0.016	412.300	0.016	450.882	0.016	489.464	0.016	566.628	0.016	643.792	0.016	720.957	0.016	798.121	0.016	875.285	0.016	952.450
	0.002	11.479	0.004	36.138	0.005	73.978	0.007	124.999	0.008	189.201	0.009	266.584	0.011	355.207	0.017	324.782	0.018	380.872	0.018	420.193	0.018	459.514	0.018	498.835	0.018	577.477	0.018	656.118	0.018	734.760	0.017	813.402	0.017	892.043	0.017	970.685
	0.003	11.627	0.004	36.604	0.006	74.931	0.007	126.610	0.009	191.639	0.010	270.019	0.012	359.784	0.019	328.967	0.020	385.779	0.020	425.607	0.020	465.434	0.020	505.262	0.020	584.916	0.019	664.571	0.019	744.226	0.019	823.881	0.019	903.536	0.019	983.191
	0.003	11.727	0.005	36.921	0.006	75.582	0.008	127.709	0.010	193.302	0.011	272.362	0.013	362.907	0.021	331.822	0.022	389.128	0.022	429.301	0.022	469.474	0.021	509.647	0.021	589.994	0.021	670.340	0.021	750.686	0.021	831.032	0.021	911.379	0.021	991.725
	0.003	11.796	0.005	37.137	0.007	76.024	0.009	128.456	0.010	194.434	0.012	273.956	0.014	365.030	0.023	333.764	0.024	391.405	0.023	431.813	0.023	472.222	0.023	512.630	0.023	593.446	0.023	674.263	0.023	755.079	0.023	835.896	0.023	916.712	0.023	997.529
	0.003	11.843	0.005	37.284	0.007	76.324	0.009	128.963	0.011	195.201	0.013	275.038	0.015	366.472	0.024	335.082	0.025	392.950	0.025	433.518	0.025	474.086	0.025	514.654	0.025	595.789	0.025	676.925	0.025	758.060	0.024	839.196	0.024	920.331	0.024	1001.467
	0.004	11.874	0.006	37.383	0.008	76.528	0.010	129.307	0.012	195.721	0.014	275.771	0.016	367.448	0.026	335.974	0.027	393.997	0.027	434.673	0.027	475.349	0.027	516.025	0.027	597.376	0.026	678.728	0.026	760.080	0.026	841.431	0.026	922.783	0.026	1004.135
	0.004	11.896	0.006	37.451	0.008	76.665	0.011	129.539	0.013	196.073	0.015	276.267	0.017	368.108	0.028	336.578	0.029	394.706	0.029	435.455	0.029	476.204	0.029	516.953	0.028	598.450	0.028	679.948	0.028	761.446	0.028	842.944	0.028	924.442	0.028	1005.940

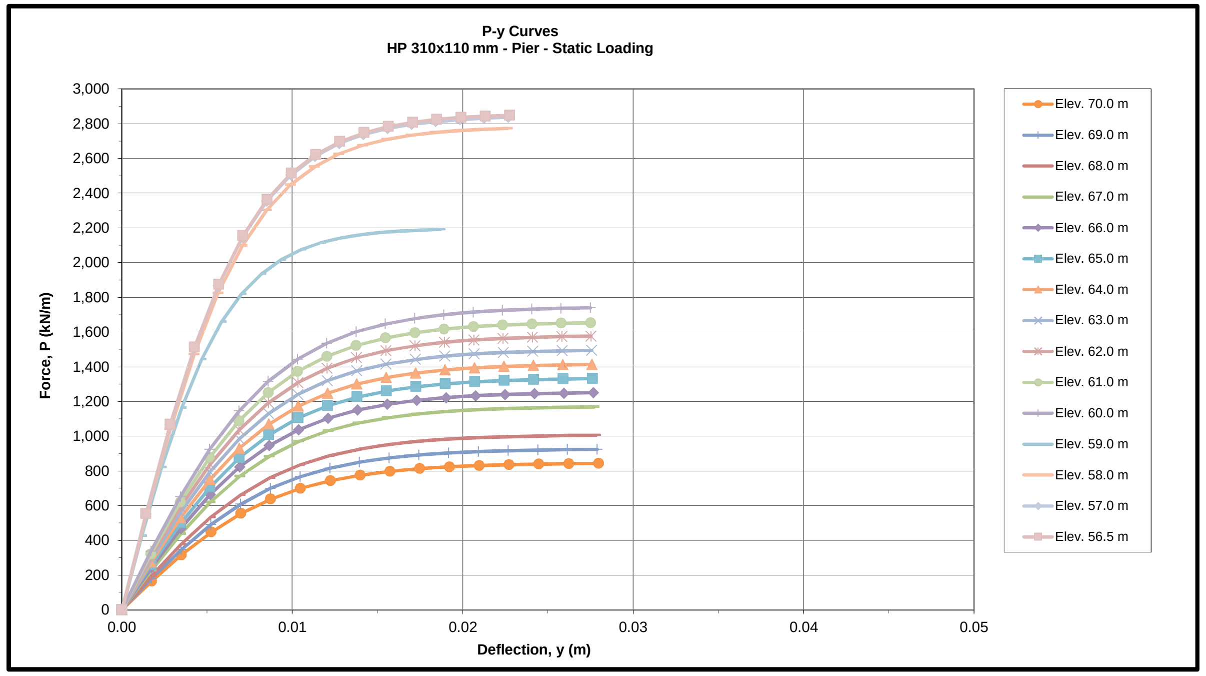
Description	**Very Loose to Loose Silt and Sand (Till)**																**Compact Sand/Very Stiff Silty Clay**							
Depth (z) *	z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m		z= 21.0 m		z= 22.0 m		z= 23.0 m		z= 23.5 m	
Elevation	Elev. 67.0 m		Elev. 66.0 m		Elev. 65.0 m		Elev. 64.0 m		Elev. 63.0 m		Elev. 62.0 m		Elev. 61.0 m		Elev. 60.0 m		Elev. 59.0 m		Elev. 58.0 m		Elev. 57.0 m		Elev. 56.5 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0	0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	227.463	0.002	243.322	0.00173	259.1806	0.002	275.039	0.002	290.898	0.002	306.757	0.002	321.822	0.002	338.474	0.001	426.422	0.001	539.615	0.001	552.000	0.001	554.257
	0.003	438.448	0.003	469.017	0.00345	499.585	0.003	530.154	0.003	560.722	0.003	591.291	0.003	620.331	0.003	652.427	0.002	821.953	0.003	1040.138	0.003	1064.012	0.003	1068.362
	0.005	620.929	0.005	664.220	0.00518	707.5106	0.005	750.802	0.005	794.093	0.005	837.384	0.005	878.510	0.005	923.966	0.004	1164.047	0.004	1473.039	0.004	1506.850	0.004	1513.010
	0.007	769.450	0.007	823.096	0.00691	876.7416	0.007	930.387	0.007	984.033	0.007	1037.679	0.007	1088.643	0.007	1144.971	0.005	1442.478	0.006	1825.379	0.006	1867.276	0.006	1874.911
	0.009	884.466	0.009	946.131	0.00863	1007.796	0.009	1069.461	0.009	1131.125	0.009	1192.790	0.009	1251.372	0.009	1316.120	0.006	1658.098	0.007	2098.234	0.007	2146.394	0.007	2155.170
	0.010	970.152	0.010	1037.791	0.01036	1105.43	0.010	1173.069	0.010	1240.707	0.010	1308.346	0.010	1372.603	0.010	1443.624	0.007	1818.732	0.008	2301.508	0.009	2354.334	0.009	2363.960
	0.012	1032.163	0.012	1104.125	0.01209	1176.087	0.012	1248.049	0.012	1320.011	0.012	1391.973	0.012	1460.338	0.012	1535.898	0.008	1934.983	0.010	2448.617	0.010	2504.820	0.010	2515.060
	0.014	1076.104	0.014	1151.130	0.01382	1226.155	0.014	1301.181	0.014	1376.207	0.014	1451.233	0.014	1522.507	0.014	1601.284	0.009	2017.359	0.011	2552.860	0.011	2611.455	0.011	2622.132
	0.016	1106.778	0.016	1183.943	0.01554	1261.107	0.016	1338.271	0.016	1415.436	0.015	1492.600	0.015	1565.906	0.015	1646.929	0.011	2074.864	0.013	2625.629	0.013	2685.894	0.013	2696.876
	0.017	1127.968	0.017	1206.610	0.01727	1285.252	0.017	1363.893	0.017	1442.535	0.017	1521.177	0.017	1595.886	0.017	1678.460	0.012	2114.588	0.014	2675.898	0.014	2737.317	0.014	2748.509
	0.019	1142.500	0.019	1222.155	0.019	1301.81	0.019	1381.465	0.019	1461.120	0.019	1540.775	0.019	1616.447	0.019	1700.084	0.013	2141.831	0.016	2710.373	0.016	2772.583	0.016	2783.919
	0.021	1152.417	0.021	1232.764	0.02072	1313.11	0.021	1393.456	0.021	1473.802	0.021	1554.149	0.021	1630.478	0.021	1714.841	0.014	2160.422	0.017	2733.899	0.017	2796.649	0.017	2808.084
	0.023	1159.162	0.022	1239.978	0.02245	1320.795	0.022	1401.611	0.022	1482.428	0.022	1563.244	0.022	1640.020	0.022	1724.877	0.015	2173.066	0.018	2749.899	0.018	2813.016	0.018	2824.517
	0.024	1163.738	0.024	1244.873	0.02418	1326.009	0.024	1407.144	0.024	1488.280	0.024	1569.415	0.024	1646.494	0.024	1731.687	0.016	2181.645	0.020	2760.755	0.020	2824.122	0.020	2835.668
	0.026	1166.838	0.026	1248.190	0.0259	1329.541	0.026	1410.893	0.026	1492.245	0.026	1573.596	0.026	1650.880	0.026	1736.300	0.018	2187.457	0.021	2768.109	0.021	2831.645	0.021	2843.222
	0.028	1168.936	0.028	1250.434	0.02763	1331.932	0.028	1413.430	0.028	1494.928	0.028	1576.426	0.028	1653.849	0.027	1739.422	0.019	2191.390	0.023	2773.086	0.023	2836.736	0.023	2848.334

NOTES: * Depth (z) is measured to be positive below the underside of the pier pile cap (Elevation 80 m).

The P-y curves have been generated based on the following assumptions:

1. P-y curves are generated for vertical piles (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves have been generated for strong axis of H-pile 310x110.





Date: May 2018
Project No: 12-1121-0099 Ph. 1750

Prepared By: ARV
Checked By: KJB

P-y CURVES

Abutments - Cyclic Loading

Site 31-204, Highway 401 / County Road 31

Figure F3

SUMMARY OF P-y CURVES FOR A H-Pile 310x110 - Cyclic Loading Condition - Integral Abutments (3 m of CSP)

Description	**Loose Sand in CSP**											
Depth (z) *	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m	
Elevation	Elev. 83.5 m		Elev. 83.0 m		Elev. 82.5 m		Elev. 82.0 m		Elev. 81.5 m		Elev. 81.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0
	0.002	1.821307	0.003	5.5263479	0.004	11.115124	0.005	18.5876348	0.006	27.943881	0.007	38.940601
	0.004	3.51067	0.006	10.652344	0.008	21.425021	0.010	35.828703	0.012	53.863388	0.014	75.06018
	0.006	4.971799	0.009	15.085812	0.012	30.34204	0.014	50.7404828	0.017	76.281143	0.020	106.29996
	0.008	6.161014	0.011	18.694221	0.015	37.599621	0.019	62.8772138	0.023	94.527	0.027	131.72609
	0.009	7.081955	0.014	21.488609	0.019	43.219964	0.024	72.2760188	0.029	108.65678	0.034	151.41634
	0.011	7.768045	0.017	23.570397	0.023	47.407056	0.029	79.2780225	0.035	119.1833	0.041	166.08535
	0.013	8.264566	0.020	25.076979	0.027	50.43724	0.034	84.345345	0.041	126.8013	0.047	176.70126
	0.015	8.616405	0.023	26.144557	0.031	52.584455	0.039	87.9360975	0.046	132.19949	0.054	184.22379
	0.017	8.862016	0.026	26.889807	0.035	54.083375	0.043	90.4427175	0.052	135.96784	0.061	189.47509
	0.019	9.031684	0.029	27.404627	0.038	55.118829	0.048	92.1742875	0.058	138.57101	0.068	193.10269
	0.021	9.148044	0.032	27.757696	0.042	55.828955	0.053	93.3618225	0.064	140.3563	0.074	195.59054
	0.023	9.227449	0.034	27.998632	0.046	56.313549	0.058	94.1721975	0.070	141.57459	0.081	197.28826
	0.025	9.281451	0.037	28.16249	0.050	56.643116	0.063	94.7233275	0.075	142.40313	0.088	198.44286
	0.026	9.318093	0.040	28.273671	0.054	56.866734	0.068	95.097285	0.081	142.96532	0.095	199.22628
	0.028	9.342916	0.043	28.348991	0.058	57.018225	0.072	95.35062	0.087	143.34617	0.101	199.75701
	0.030	9.359714	0.046	28.399962	0.062	57.120744	0.077	95.522055	0.093	143.60391	0.108	200.11617

Description	**Existing Embankment Fill**				**Compact Sandy Silt to Silty Sand (Till)**									
Depth (z) *	z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m	
Elevation	Elev. 80.5 m		Elev. 80.0 m		Elev. 79.5 m		Elev. 79.0 m		Elev. 78.5 m		Elev. 78.0 m		Elev. 77.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0
	0.001	40.319018	0.001	53.72266	0.002	49.121088	0.002	57.604323	0.002	63.551328	0.002	69.498333	0.002	87.339345
	0.002	77.71716	0.002	103.55342	0.003	94.6836375	0.004	111.03554	0.004	122.49873	0.004	133.96191	0.004	168.35147
	0.003	110.06276	0.003	146.65199	0.005	134.090633	0.005	157.24815	0.005	173.48227	0.005	189.71639	0.005	238.41875
	0.004	136.38893	0.004	181.73001	0.007	166.164075	0.007	194.86069	0.007	214.97789	0.007	235.09508	0.007	295.44667
	0.005	156.77617	0.005	208.89478	0.009	191.002065	0.009	223.98821	0.009	247.11251	0.009	270.23679	0.009	339.60966
	0.006	171.96443	0.006	229.13222	0.010	209.506088	0.011	245.68789	0.011	271.05242	0.011	296.41697	0.011	372.51059
	0.007	182.95612	0.008	243.77799	0.012	222.897383	0.013	261.39187	0.013	288.37767	0.013	315.36347	0.012	396.32087
	0.008	190.74493	0.009	254.15612	0.014	232.386578	0.015	272.51984	0.014	300.65449	0.014	328.78913	0.014	413.19305
	0.009	196.18211	0.010	261.40083	0.016	239.01075	0.016	280.28802	0.016	309.22464	0.016	338.16126	0.016	424.97111
	0.009	199.93813	0.011	266.40549	0.017	243.58674	0.018	285.65429	0.018	315.14491	0.018	344.63554	0.018	433.10741
	0.010	202.51404	0.012	269.83774	0.019	246.725003	0.020	289.33453	0.020	319.20509	0.020	349.07567	0.020	438.68737
	0.011	204.27185	0.013	272.17992	0.021	248.866568	0.022	291.84594	0.022	321.97579	0.022	352.10563	0.021	442.49516
	0.012	205.46733	0.014	273.77282	0.023	250.32303	0.024	293.55393	0.023	323.86011	0.023	354.16628	0.023	445.08481
	0.013	206.27848	0.015	274.85363	0.024	251.31126	0.025	294.71284	0.025	325.13865	0.025	355.56447	0.025	446.84193
	0.014	206.828	0.016	275.58583	0.026	251.980748	0.027	295.49794	0.027	326.00481	0.027	356.51168	0.027	448.0323
	0.015	207.19988	0.017	276.08133	0.028	252.433808	0.029	296.02925	0.029	326.59097	0.029	357.15269	0.028	448.83786

Description	**Very Loose to Loose Silt (Till)**									
Depth (z) *	z= 8.0 m		z= 9.0 m		z= 10.0m		z= 11.0 m		z= 12.0 m	
Elevation	Elev. 76.0 m		Elev. 75.0 m		Elev. 74.0 m		Elev. 73.0 m		Elev. 72.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0	0.000	0	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	99.233355	0.002	111.127365	0.002	123.021	0.002	134.915	0.002	146.809
	0.004	191.277833	0.004	214.2042	0.003	237.131	0.003	260.057	0.003	282.983
	0.005	270.886988	0.005	303.35523	0.005	335.823	0.005	368.292	0.005	400.760
	0.007	335.681063	0.007	375.915458	0.007	416.150	0.007	456.384	0.007	496.619
	0.009	385.858238	0.009	432.106815	0.009	478.355	0.009	524.604	0.009	570.853
	0.011	423.239663	0.011	473.96874	0.010	524.698	0.010	575.427	0.010	626.156
	0.012	450.29247	0.012	504.264068	0.012	558.236	0.012	612.207	0.012	666.179
	0.014	469.462328	0.014	525.731603	0.014	582.001	0.014	638.270	0.014	694.539
	0.016	482.844345	0.016	540.717585	0.016	598.591	0.016	656.464	0.016	714.337
	0.018	492.088658	0.018	551.06991	0.017	610.051	0.017	669.032	0.017	728.014
	0.019	498.428505	0.019	558.169643	0.019	617.911	0.019	677.652	0.019	737.393
	0.021	502.75485	0.021	563.014538	0.021	623.274	0.021	683.534	0.021	743.794
	0.023	505.69716	0.023	566.309505	0.023	626.922	0.023	687.534	0.023	748.147
	0.025	507.69357	0.025	568.545203	0.024	629.397	0.024	690.248	0.024	751.100
	0.026	509.046045	0.026	570.059783	0.026	631.074	0.026	692.087	0.026	753.101
	0.028	509.961308	0.028	571.084755	0.028	632.208	0.028	693.332	0.028	754.455

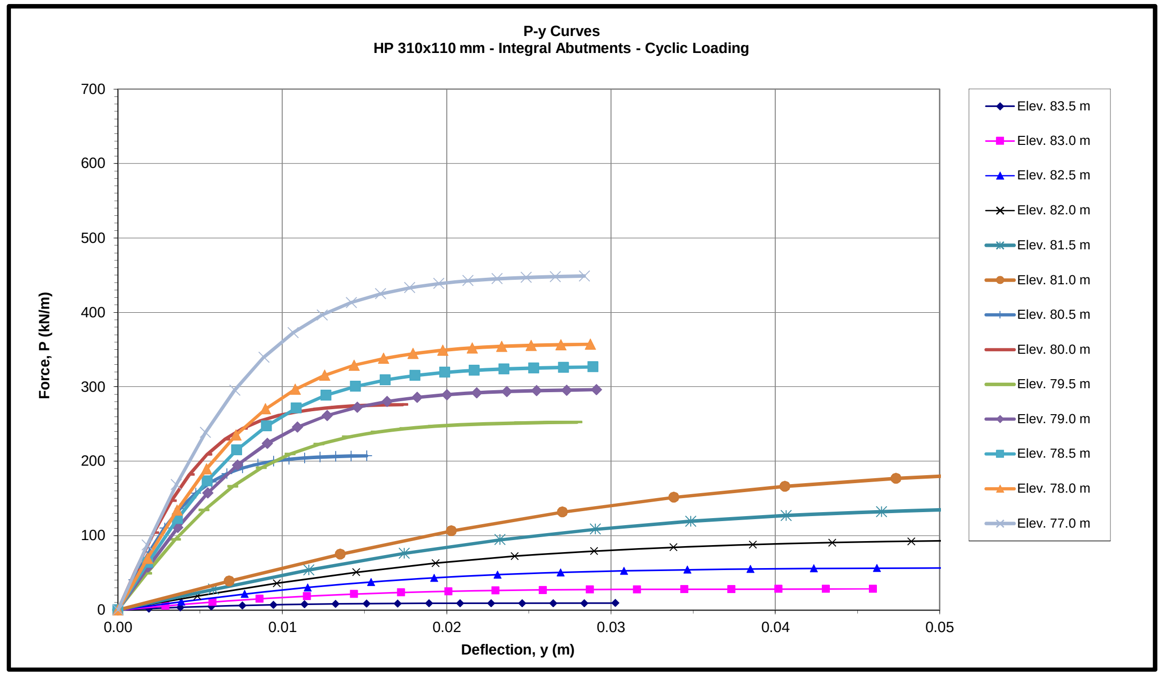
Description	**Very Loose to Loose Silt (Till)**											
Depth (z) *	z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m		z= 17.0 m		z= 18.0 m	
Elevation	Elev. 71.0 m		Elev. 70.0 m		Elev. 69.0 m		Elev. 68.0 m		Elev. 67.0 m		Elev. 66.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0	0.000	0	0	0	0.000	0	0.000	0	0.000	0
	0.002	158.7034	0.002	170.59742	0.00173	182.49143	0.002	194.385435	0.002	158.70341	0.002	170.59742
	0.003	305.9097	0.003	328.83604	0.00346	351.76241	0.003	374.688773	0.003	305.90967	0.003	328.83604
	0.005	433.2282	0.005	465.69644	0.00519	498.16468	0.005	530.63292	0.005	433.22819	0.005	465.69644
	0.007	536.853	0.007	577.08742	0.00692	617.32181	0.007	657.556208	0.007	536.85303	0.007	577.08742
	0.009	617.1011	0.009	663.3497	0.00865	709.59828	0.009	755.84685	0.009	617.10113	0.009	663.3497
	0.010	676.8851	0.010	727.61413	0.01038	778.34318	0.010	829.072275	0.010	676.88505	0.010	727.61413
	0.012	720.1505	0.012	774.1221	0.01211	828.09368	0.012	882.06525	0.012	720.15047	0.012	774.1221
	0.014	750.8087	0.014	807.078	0.01384	863.34728	0.014	919.61655	0.014	750.80873	0.014	807.078
	0.016	772.2105	0.016	830.08373	0.01556	887.95703	0.016	945.83025	0.016	772.2105	0.016	830.08373
	0.017	786.9949	0.017	845.97615	0.01729	904.95743	0.017	963.938625	0.017	786.99488	0.017	845.97615
	0.019	797.1342	0.019	856.8753	0.01902	916.61648	0.019	976.357575	0.019	797.1342	0.019	856.8753
	0.021	804.0533	0.021	864.31298	0.02075	924.5727	0.021	984.83235	0.021	804.05333	0.021	864.31298
	0.023	808.7589	0.023	869.37128	0.02248	929.98358	0.022	990.59595	0.023	808.7589	0.023	869.37128
	0.024	811.9517	0.024	872.80343	0.02421	933.65505	0.024	994.506675	0.024	811.95173	0.024	872.80343
	0.026	814.1148	0.026	875.1285	0.02594	936.14228	0.026	997.155975	0.026	814.1148	0.026	875.1285
	0.028	815.5786	0.028	876.702	0.02767	937.82543	0.028	998.94885	0.028	815.57858	0.028	876.702

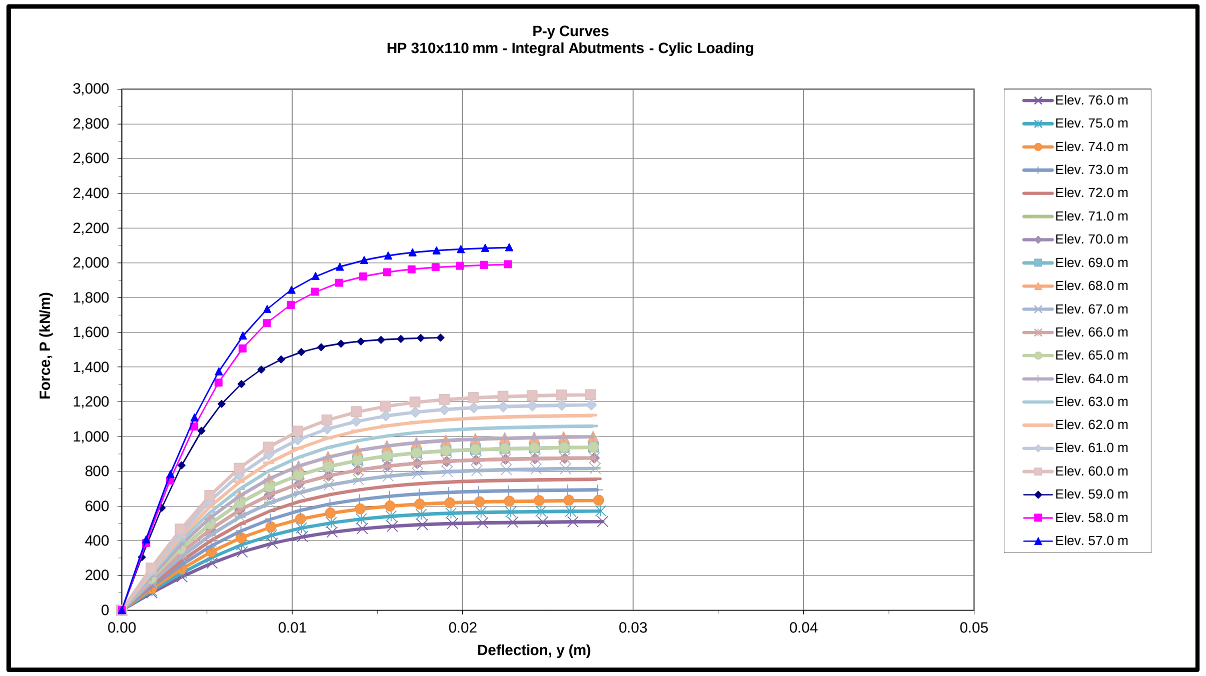
Description	**Very Loose to Loose Silt (Till)**												**Compact Sand/Very Stiff Silty Clay**					
Depth (z) *	z= 19.0 m		z= 20.0 m		z= 21.0 m		z= 22.0 m		z= 23.0 m		z= 24.0 m		z= 25.0 m		z= 26.0 m		z= 27.0 m	
Elevation	Elev. 65.0 m		Elev. 64.0 m		Elev. 63.0 m		Elev. 62.0 m		Elev. 61.0 m		Elev. 60.0 m		Elev. 59.0 m		Elev. 58.0 m		Elev. 57.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	0
	0.00173	182.49143	0.002	194.38544	0.002	206.279445	0.002	218.17346	0.002	230.06747	0.002	241.36678	0.001	305.50298	0.001	387.446145	0.001	406.403753
	0.00346	351.76241	0.003	374.68877	0.003	397.61514	0.003	420.54151	0.003	443.46788	0.003	465.24793	0.002	588.87402	0.003	746.824058	0.003	783.365925
	0.00519	498.16468	0.005	530.63292	0.005	563.101163	0.005	595.5694	0.005	628.03764	0.005	658.88247	0.004	833.9613	0.004	1057.64963	0.004	1109.400
	0.00692	617.32181	0.007	657.55621	0.007	697.790603	0.007	738.02499	0.007	778.2594	0.007	816.48203	0.005	1033.4384	0.006	1310.63123	0.006	1374.7599
	0.00865	709.59828	0.009	755.84685	0.009	802.09545	0.009	848.34405	0.009	894.59258	0.009	938.52878	0.006	1187.9155	0.007	1506.54263	0.007	1580.25728
	0.01038	778.34318	0.010	829.07228	0.010	879.801375	0.010	930.53048	0.010	981.2595	0.010	1029.4522	0.007	1302.9991	0.008	1652.4945	0.009	1733.3505
	0.01211	828.09368	0.012	882.06525	0.012	936.0369	0.012	990.00848	0.012	1043.9801	0.012	1095.2531	0.008	1386.2847	0.010	1758.11933	0.010	1844.1435
	0.01384	863.34728	0.014	919.61655	0.014	975.885825	0.014	1032.1551	0.014	1088.4244	0.014	1141.8803	0.009	1445.3017	0.011	1832.96595	0.011	1922.65238
	0.01556	887.95703	0.016	945.83025	0.016	1003.70348	0.016	1061.5767	0.015	1119.4499	0.015	1174.4295	0.011	1486.5	0.013	1885.2147	0.013	1977.45758
	0.01729	904.95743	0.017	963.93863	0.017	1022.9199	0.017	1081.9012	0.017	1140.8824	0.017	1196.9146	0.012	1514.9598	0.014	1921.30808	0.014	2015.31705
	0.01902	916.61648	0.019	976.35758	0.019	1036.09875	0.019	1095.8399	0.019	1155.581	0.019	1212.3351	0.013	1534.4779	0.016	1946.06138	0.016	2041.28145
	0.02075	924.5727	0.021	984.83235	0.021	1045.09208	0.021	1105.3517	0.021	1165.6115	0.021	1222.8581	0.014	1547.7971	0.017	1962.9531	0.017	2058.99975
	0.02248	929.98358	0.022	990.59595	0.022	1051.20833	0.022	1111.8206	0.022	1172.433	0.022	1230.0148	0.015	1556.8554	0.018	1974.44108	0.018	2071.04978
	0.02421	933.65505	0.024	994.50668	0.024	1055.3583	0.024	1116.2099	0.024	1177.0616	0.024	1234.8707	0.016	1563.0016	0.020	1982.23583	0.020	2079.22598
	0.02594	936.14228	0.026	997.15598	0.026	1058.16975	0.026	1119.1835	0.026	1180.1972	0.026	1238.1603	0.018	1567.1654	0.021	1987.51643	0.021	2084.76495
	0.02767	937.82543	0.028	998.94885	0.028	1060.07235	0.028	1121.1958	0.028	1182.3192	0.028	1240.3865	0.019	1569.9831	0.023	1991.08995	0.023	2088.5133

NOTES: * Depth (z) is measured to be positive below the underside of the abutments (Elevation 84.0 m).

The P-y curves have been generated based on the following assumptions:

1. P-y curves are generated for vertical piles (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves have been generated for strong axis of H-pile 310x110.





Date: August 2018
Project No: 12-1121-0099 Ph. 1750

Prepared By: ARV
Checked By: KJB

P-y CURVES

Pier - Cyclic Loading

Site 31-204, Highway 401 / County Road 31

Figure F4

SUMMARY OF P-y CURVES FOR A H-Pile 310x110 - Pier Location - Cyclic Loading Condition

Description	**Compact Silty Sand to Sandy Silt (Till)**													
Depth (z) *	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m	
Elevation	Elev. 79.5 m		Elev. 79.0 m		Elev. 78.5 m		Elev. 78.0 m		Elev. 77.5 m		Elev. 77.0 m		Elev. 76.5 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	1.736	0.000	5.393	0.001	11.189	0.001	18.905	0.001	28.615	0.001	40.319	0.001	53.723
	0.000	3.346	0.001	10.395	0.001	21.567	0.001	36.441	0.002	55.158	0.002	77.717	0.002	103.553
	0.001	4.739	0.001	14.721	0.002	30.543	0.002	51.608	0.002	78.114	0.003	110.063	0.003	146.652
	0.001	5.873	0.002	18.242	0.002	37.849	0.003	63.952	0.003	96.799	0.004	136.389	0.004	181.730
	0.001	6.750	0.002	20.969	0.003	43.506	0.003	73.511	0.004	111.268	0.005	156.776	0.005	208.895
	0.001	7.404	0.002	23.001	0.003	47.721	0.004	80.633	0.005	122.047	0.006	171.964	0.006	229.132
	0.002	7.878	0.003	24.471	0.004	50.771	0.005	85.787	0.006	129.848	0.007	182.956	0.008	243.778
	0.002	8.213	0.003	25.513	0.004	52.933	0.005	89.439	0.006	135.376	0.008	190.745	0.009	254.156
	0.002	8.447	0.003	26.240	0.005	54.441	0.006	91.988	0.007	139.235	0.009	196.182	0.010	261.401
	0.002	8.609	0.004	26.742	0.005	55.484	0.007	93.750	0.008	141.901	0.009	199.938	0.011	266.405
	0.003	8.720	0.004	27.087	0.006	56.199	0.007	94.957	0.009	143.729	0.010	202.514	0.012	269.838
	0.003	8.796	0.005	27.322	0.006	56.686	0.008	95.782	0.010	144.977	0.011	204.272	0.013	272.180
	0.003	8.847	0.005	27.482	0.007	57.018	0.009	96.342	0.010	145.825	0.012	205.467	0.014	273.773
	0.003	8.882	0.005	27.590	0.007	57.243	0.009	96.723	0.011	146.401	0.013	206.278	0.015	274.854
	0.004	8.906	0.006	27.664	0.008	57.396	0.010	96.980	0.012	146.791	0.014	206.828	0.016	275.586
	0.004	8.922	0.006	27.713	0.008	57.499	0.011	97.155	0.013	147.055	0.015	207.200	0.017	276.081

Description	**Very Loose to Loose Sand and Silt (Till)**																					
Depth (z) *	z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0m		z= 11.0 m		z= 12.0 m	
Elevation	Elev. 76.0 m		Elev. 75.5 m		Elev. 75.0 m		Elev. 74.5 m		Elev. 74.0 m		Elev. 73.0 m		Elev. 72.0 m		Elev. 71.0 m		Elev. 70.0 m		Elev. 69.0 m		Elev. 68.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	49.121	0.002	57.604	0.002	63.551	0.002	69.498	0.002	75.445	0.002	87.339	0.002	99.233	0.002	111.127	0.002	123.021	0.002	134.915	0.002	146.809
	0.003	94.684	0.004	111.036	0.004	122.499	0.004	133.962	0.004	145.425	0.004	168.351	0.004	191.278	0.004	214.204	0.003	237.131	0.003	260.057	0.003	282.983
	0.005	134.091	0.005	157.248	0.005	173.482	0.005	189.716	0.005	205.951	0.005	238.419	0.005	270.887	0.005	303.355	0.005	335.823	0.005	368.292	0.005	400.760
	0.007	166.164	0.007	194.861	0.007	214.978	0.007	235.095	0.007	255.212	0.007	295.447	0.007	335.681	0.007	375.915	0.007	416.150	0.007	456.384	0.007	496.619
	0.009	191.002	0.009	223.988	0.009	247.113	0.009	270.237	0.009	293.361	0.009	339.610	0.009	385.858	0.009	432.107	0.009	478.355	0.009	524.604	0.009	570.853
	0.010	209.506	0.011	245.688	0.011	271.052	0.011	296.417	0.011	321.782	0.011	372.511	0.011	423.240	0.011	473.969	0.010	524.698	0.010	575.427	0.010	626.156
	0.012	222.897	0.013	261.392	0.013	288.378	0.013	315.363	0.013	342.349	0.012	396.321	0.012	450.292	0.012	504.264	0.012	558.236	0.012	612.207	0.012	666.179
	0.014	232.387	0.015	272.520	0.014	300.654	0.014	328.789	0.014	356.924	0.014	413.193	0.014	469.462	0.014	525.732	0.014	582.001	0.014	638.270	0.014	694.539
	0.016	239.011	0.016	280.288	0.016	309.225	0.016	338.161	0.016	367.098	0.016	424.971	0.016	482.844	0.016	540.718	0.016	598.591	0.016	656.464	0.016	714.337
	0.017	243.587	0.018	285.654	0.018	315.145	0.018	344.636	0.018	374.126	0.018	433.107	0.018	492.089	0.018	551.070	0.017	610.051	0.017	669.032	0.017	728.014
	0.019	246.725	0.020	289.335	0.020	319.205	0.020	349.076	0.020	378.946	0.020	438.687	0.019	498.429	0.019	558.170	0.019	617.911	0.019	677.652	0.019	737.393
	0.021	248.867	0.022	291.846	0.022	321.976	0.022	352.106	0.021	382.235	0.021	442.495	0.021	502.755	0.021	563.015	0.021	623.274	0.021	683.534	0.021	743.794
	0.023	250.323	0.024	293.554	0.023	323.860	0.023	354.166	0.023	384.472	0.023	445.085	0.023	505.697	0.023	566.310	0.023	626.922	0.023	687.534	0.023	748.147
	0.024	251.311	0.025	294.713	0.025	325.139	0.025	355.564	0.025	385.990	0.025	446.842	0.025	507.694	0.025	568.545	0.024	629.397	0.024	690.248	0.024	751.100
	0.026	251.981	0.027	295.498	0.027	326.005	0.027	356.512	0.027	387.019	0.027	448.032	0.026	509.046	0.026	570.060	0.026	631.074	0.026	692.087	0.026	753.101
	0.028	252.434	0.029	296.029	0.029	326.591	0.029	357.153	0.029	387.714	0.028	448.838	0.028	509.961	0.028	571.085	0.028	632.208	0.028	693.332	0.028	754.455

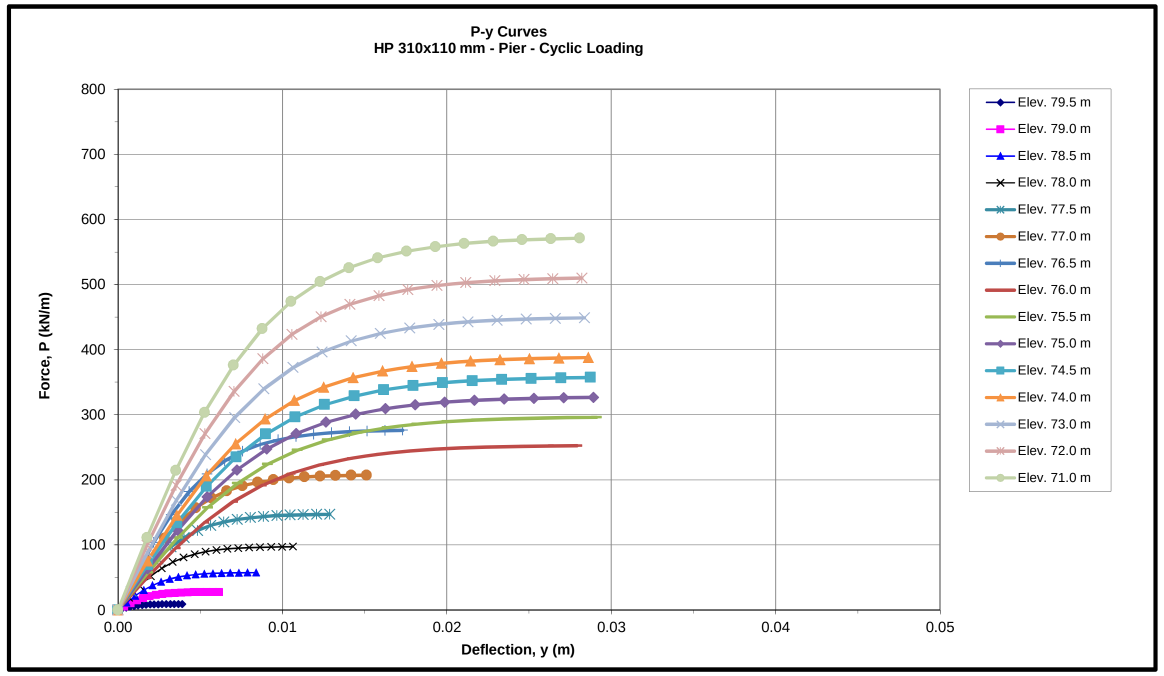
Description	**Very Loose to Loose Silt and Sand (Till)**															
Depth (z) *	z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m	
Elevation	Elev. 67.0 m		Elev. 66.0 m		Elev. 65.0 m		Elev. 64.0 m		Elev. 63.0 m		Elev. 62.0 m		Elev. 61.0 m		Elev. 60.0 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	170.597	0.002	182.491	0.00173	194.385	0.002	206.279	0.002	218.173	0.002	230.067	0.002	241.367	0.002	253.855
	0.003	328.836	0.003	351.762	0.00345	374.689	0.003	397.615	0.003	420.542	0.003	443.468	0.003	465.248	0.003	489.321
	0.005	465.696	0.005	498.165	0.00518	530.633	0.005	563.101	0.005	595.569	0.005	628.038	0.005	658.882	0.005	692.974
	0.007	577.087	0.007	617.322	0.00691	657.556	0.007	697.791	0.007	738.025	0.007	778.259	0.007	816.482	0.007	858.728
	0.009	663.350	0.009	709.598	0.00863	755.847	0.009	802.095	0.009	848.344	0.009	894.593	0.009	938.529	0.009	987.090
	0.010	727.614	0.010	778.343	0.01036	829.072	0.010	879.801	0.010	930.530	0.010	981.260	0.010	1029.452	0.010	1082.718
	0.012	774.122	0.012	828.094	0.01209	882.065	0.012	936.037	0.012	990.008	0.012	1043.980	0.012	1095.253	0.012	1151.923
	0.014	807.078	0.014	863.347	0.01382	919.617	0.014	975.886	0.014	1032.155	0.014	1088.424	0.014	1141.880	0.014	1200.963
	0.016	830.084	0.016	887.957	0.01554	945.830	0.016	1003.703	0.016	1061.577	0.015	1119.450	0.015	1174.430	0.015	1235.196
	0.017	845.976	0.017	904.957	0.01727	963.939	0.017	1022.920	0.017	1081.901	0.017	1140.882	0.017	1196.915	0.017	1258.845
	0.019	856.875	0.019	916.616	0.019	976.358	0.019	1036.099	0.019	1095.840	0.019	1155.581	0.019	1212.335	0.019	1275.063
	0.021	864.313	0.021	924.573	0.02072	984.832	0.021	1045.092	0.021	1105.352	0.021	1165.611	0.021	1222.858	0.021	1286.131
	0.023	869.371	0.022	929.984	0.02245	990.596	0.022	1051.208	0.022	1111.821	0.022	1172.433	0.022	1230.015	0.022	1293.658
	0.024	872.803	0.024	933.655	0.02418	994.507	0.024	1055.358	0.024	1116.210	0.024	1177.062	0.024	1234.871	0.024	1298.765
	0.026	875.129	0.026	936.142	0.0259	997.156	0.026	1058.170	0.026	1119.183	0.026	1180.197	0.026	1238.160	0.026	1302.225
	0.028	876.702	0.028	937.825	0.02763	998.949	0.028	1060.072	0.028	1121.196	0.028	1182.319	0.028	1240.387	0.027	1304.566

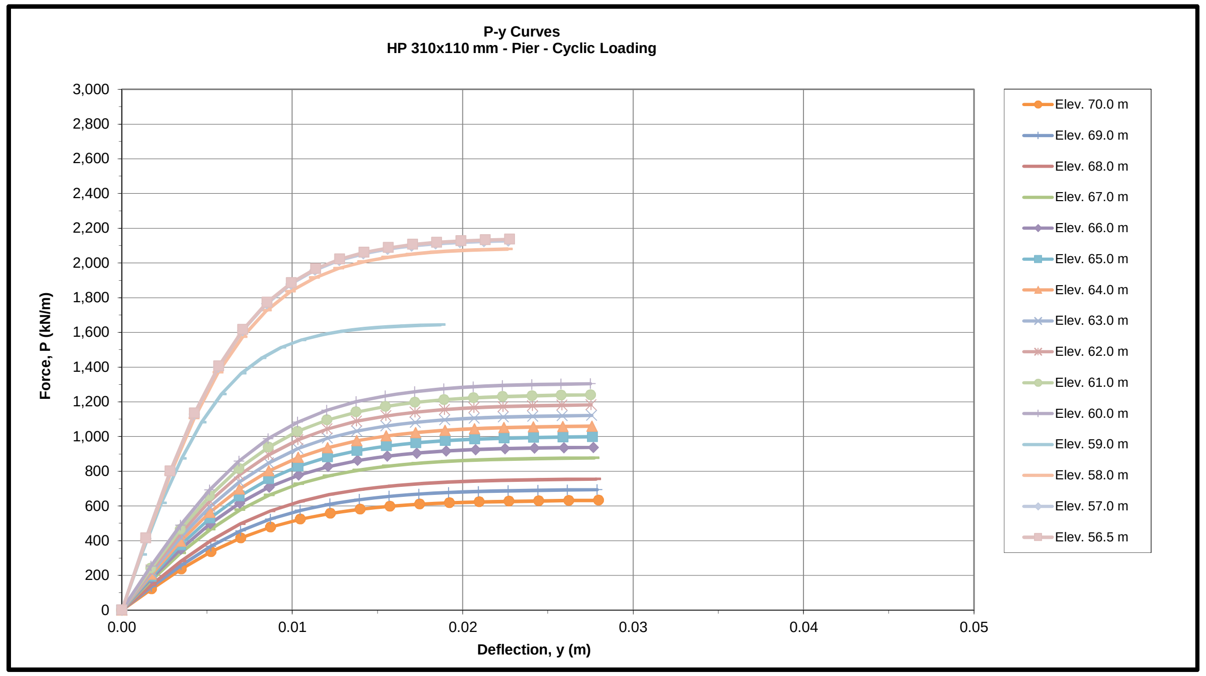
Description	**Compact Sand/Very Stiff Silty Clay**							
Depth (z) *	z= 21.0 m		z= 22.0 m		z= 23.0 m		z= 23.5 m	
Elevation	Elev. 59.0 m		Elev. 58.0 m		Elev. 57.0 m		Elev. 56.5 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.001	319.817	0.001	404.711	0.001	414.000	0.001	415.693
	0.002	616.465	0.003	780.103	0.003	798.009	0.003	801.271
	0.004	873.035	0.004	1104.779	0.004	1130.137	0.004	1134.758
	0.005	1081.859	0.006	1369.034	0.006	1400.457	0.006	1406.183
	0.006	1243.573	0.007	1573.676	0.007	1609.796	0.007	1616.377
	0.007	1364.049	0.008	1726.131	0.009	1765.751	0.009	1772.970
	0.008	1451.237	0.010	1836.463	0.010	1878.615	0.010	1886.295
	0.009	1513.019	0.011	1914.645	0.011	1958.591	0.011	1966.599
	0.011	1556.148	0.013	1969.222	0.013	2014.421	0.013	2022.657
	0.012	1585.941	0.014	2006.923	0.014	2052.988	0.014	2061.381
	0.013	1606.373	0.016	2032.780	0.016	2079.438	0.016	2087.939
	0.014	1620.317	0.017	2050.424	0.017	2097.487	0.017	2106.063
	0.015	1629.799	0.018	2062.424	0.018	2109.762	0.018	2118.388
	0.016	1636.234	0.020	2070.566	0.020	2118.091	0.020	2126.751
	0.018	1640.592	0.021	2076.082	0.021	2123.734	0.021	2132.417
	0.019	1643.542	0.023	2079.815	0.023	2127.552	0.023	2136.251

NOTES: * Depth (z) is measured to be positive below the underside of the pier pile cap (Elevation 80 m).

The P-y curves have been generated based on the following assumptions:

1. P-y curves are generated for vertical piles (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves have been generated for strong axis of H-pile 310x110.





Date: May 2018
Project No: 12-1121-0099 Ph. 1750

Prepared By: ARV
Checked By: KJB

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APPENDIX G

Non-Standard Special Provisions

Vibration Monitoring During Pile Driving

Obstructions During Piling

Deep Foundations - Item No.

Special Provision

Amendment to OPSS 903, April 2016

Vibration Monitoring During Piling

This special provision describes requirements for vibration monitoring during pile installation works.

Definitions

Foundation Engineering Specialist (FES): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the piling work at a minimum of two (2) projects of similar scope to the contract. The FES shall be retained by the Contract Administrator to ensure general conformance with the contract documents and shall issue certificate(s) of conformance.

Submission Requirements

The Contractor shall submit details of the vibration monitoring plan to the FES for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Qualifications of vibrations monitoring specialist;
  Proposed instrumentation;
- Proposed location of instruments;
- Proposed frequency of readings; and,
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the FES.

Monitoring

The FES shall take readings during driving of each pile. The readings should be taken and recorded during the entire length of driving and during seating of the pile on the bedrock (if applicable). As a minimum, one vibration monitoring point shall be installed on the nearest existing abutment wall to the pile driving activities.

The pile(s) furthest from the monitored structure or utility should be driven first to assess the vibration level at the existing structures. If necessary, the contractor must alter the pile driving procedures for the remaining piles. The revised procedure shall be submitted to the Contract Administrator for approval prior to driving the remaining piles.

The measured vibrations shall not exceed 100 mm/s (peak particle velocity).

If it is not practical to drive the piles furthest from the existing structure first due to space constraints, the piles nearest the existing structure may be driven first but the measured vibrations in that case shall not exceed 50 mm/s.

The results shall be submitted to the Contract Administrator after each pile has been driven and prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next piles with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations are within acceptable levels. The above process must be repeated for each pile.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

Deep Foundations - Item No.

Special Provision

Amendment to OPSS 903, April 2016

Obstructions During Piling

This special provision describes requirements for pile installation through obstructions and natural cobbles and boulders.

Definitions

Foundation Engineering Specialist (FES): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the piling work at a minimum of two (2) projects of similar scope to the contract. The FES shall be retained by the Contract Administrator to ensure general conformance with the contract documents and shall issue certificate(s) of conformance.

Submission Requirements

The Contractor shall submit details for advancing the piles though obstructions, cobles and boulders. The submittals shall satisfy the specifications and at a minimum contain specific information on their approach to advancing the piles in the event such conditions are encountered.

Pile Driving Through Obstacles, Cobbles, Boulders

The soils at the site are glacially-derived and are known to contain cobbles and boulders within the till deposits. The embankment fills at the site may also contain some obstructions. The Contractor is advised that appropriate equipment and construction procedures will be required to penetrate or remove obstructions, such as cobbles and boulders, to permit installation of deep foundations.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

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