



## REPORT

# Foundation Investigation and Design Strandherd Drive and CNR Overpass Bridges Site No. 3-549, Highway 416 Ottawa, Ontario

*W.P. 4133-01-01 & 4134-01-01*

Submitted to:

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**PART A**

Foundation Investigation Report  
Strandherd Drive and CNR Overpass Bridges  
Site No. 3-549, Highway 416  
Ottawa, Ontario  
W.P. 4133-01-01 & 4134-01-01

## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by WSP Canada Group Limited (WSP) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering input required for the seismic evaluation of several existing structures included as part of the Mega 6 Rehabilitation of 21 Structures project. This report presents the results of the detailed foundation investigation conducted for the seismic evaluation of the Highway 416 overpass bridges at McKenna Casey Drive (formerly known, and referenced herein, as Strandherd Drive) and the Canadian National Rail (CNR) tracks (Site 3-549) in Ottawa, Ontario.

At the preliminary design stage of the project, a geotechnical desktop study based on the available GEOCREs information and original structural design drawings was carried out. The existing subsurface information was provided in the following three reports contained in the GEOCREs Report No. 31G5-162:

- Report prepared by MTO Engineering Materials Office, Foundation Design Section “*Foundation Investigation Report for Hwy 416, Strandherd Drive/CNR Overpass, Structures #21A and #21B, W.P. 128-87-05/06, Site 3-549, District 9, Ottawa*”, 1989 (GEOCREs No. 31G5-162);
- Report prepared by MTO Engineering Materials Office, Foundation Design Section “*Foundation Investigation Report for Hwy 416 Fill Area between Strandherd Drive Overpass (Structure #21) and Jock River Overpass (Structure #22), W.P. 128-87-00, District 9, Ottawa*”, 1989 (GEOCREs No. 31G5-162); and,
- Report prepared by MTO Engineering Materials Office, Foundation Design Section “*Foundation Investigation Report for Hwy 416/Strandherd Rive and C.N.R., W.P. 128-87-05/06, District 9, Ottawa*”, 1989 (GEOCREs No. 31G5-162).

The purpose of the current foundation investigation was to assess the subsurface conditions and provide additional subsurface information required for the detailed seismic assessment of the structures. A total of four boreholes and eight Seismic Cone Penetration Tests (SCPTs) were put down at the site as part of the current study. The field work and foundation design work were carried out in accordance with Golder’s change proposal to WSP, dated November 10, 2017.

## 2.0 SITE DESCRIPTION AND GEOLOGY

### 2.1 General

The twin bridges, which carry a total of four lanes of traffic over the former Strandherd Drive and CNR tracks, are located on Highway 416 about 2.0 km south of Fallowfield Road in Ottawa, Ontario.

Each bridge consists of a fifteen-span, pre-cast concrete deck supported predominantly on HP 310x110 piles driven to bedrock. The northernmost piers and north abutments are supported on spread footings. The structures are relatively straight, aligned approximately north-south, about 384 m long at their centrelines and about 14 m wide.

The 1991 structural design drawings indicate that the natural ground surface prior to construction of the bridge was at about Elevation 91 m at the south abutments to about Elevation 93 m at Pier N13/S13, and increased to the north to about Elevation 99 m at the north abutments. The results of the current field investigation indicate that natural ground surface below the bridge is currently between about Elevation 92 and 93 m. In the area of the bridge, the embankments range from about 7 m in height (at the south abutment) to 4 m in height (at the north abutment) above the ground surface. The bridge pavement surface ranges from about Elevation 96.5 m, at the south abutments, to Elevation 103.5 m, at the north abutments.

Based on the 1991 structural design drawings and observations during the current field investigation, the embankment side slopes are oriented at about 2 horizontal to 1 vertical (i.e., 2H:1V). Based on visual observation at the time of planning and carrying out the site investigation, the existing embankment side slopes appear to be performing satisfactorily.

## 2.2 Regional Geological Conditions

The site is located within the physiographic region known as the Ottawa Valley Clay Flats adjacent to the Ottawa River, as delineated in *The Physiography of Southern Ontario*<sup>1</sup>.

The Ottawa Valley Clay Plain region is characterized by relatively thick deposits of sensitive marine clay, silty clay and silt that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thinner, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock.<sup>1</sup>

## 3.0 INVESTIGATION PROCEDURES

### 3.1 Current Investigation (2018)

The current subsurface investigation for the overpass bridges was carried out between April 30 and May 16, 2018 at which time four boreholes (numbered 18-01, 18-02, 18-05 and 18-08) and eight Seismic Cone Penetration Test holes (numbered SCPT-18-03 to SCPT-18-10, inclusive) were advanced at the locations shown on Drawings 1 and 2 and outlined below.

- Boreholes 18-01 and 18-02 were located near the north and south abutments of the bridges, respectively. Borehole 18-01 was put down within the passing lane of the of the southbound structure. Borehole 18-02 was put down within the median of the south abutment, adjacent to the passing lane of the southbound structure. The boreholes were advanced using 200 mm inside diameter continuous-flight hollow-stem augers and/or wash boring using NW casing with a truck-mounted drill rig, supplied and operated by CCC Geotechnical & Environmental Drilling Ltd. (CCC) of Ottawa, Ontario. The boreholes were advanced through the overburden to depths of about 20.8 and 14.8 m below the existing ground surface (Elevations 82.7 to 80.9 m). At the bedrock surface, the boreholes were then cored about 5.5 m into the bedrock using HQ-size coring equipment.
- Boreholes 18-05 and 18-08 were advanced from ground surface below the existing bridges at selected pier locations. Borehole 18-05 was put down adjacent to pier N11/S11 and Borehole 18-08 was advanced between Piers N7/S7 and Piers N8/S8. The boreholes were put down with a track-mounted drill rig, also supplied and operated by CCC. The boreholes were advanced to depths of about 10.5 and 12.2 m (Elevations 82.5 to 80.20 m) below the existing ground surface in the overburden, to effective refusal.
- Eight seismic cone penetration tests (denoted as SCPT18-03 to SCPT18-10) were carried out adjacent to selected piers along the bridges. The portable SCPT equipment was supplied and operated by ConeTec Investigations Ltd. (ConeTec) of Richmond Hill, Ontario, and included a 15 cm<sup>2</sup> tip base area probe with an equal end area friction sleeve and tip and sleeve capacities of 1,500 bar and 15 bar, respectively. SCPT18-03 to SCPT18-06 were pre-drilled and cased to about 1.5 metres below ground surface to allow for penetration of the upper granular fill present at the site. The pre-drilling (where required) and advancement of the SCPT probe was carried out by CCC. The SCPTs were advanced to probe refusal at depths ranging from 4.77 to 11.37 m below the existing ground surface (Elevations 91.9 to 93.5 m).

<sup>1</sup> 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.

Soil samples in the boreholes were obtained at vertical intervals of about 0.6 to 1.5 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Water was maintained in the hollow-stem augers during drilling and testing to minimize the potential for sample disturbance.

The ground water levels at SCPT locations were inferred based on pore water pressure measurements and dissipation tests taken during advancement. The summary and plots of the pore pressure dissipation tests are included in Conetec report provided in Appendix C.

At Boreholes 18-01 and 18-02, a 64 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 Vertical Seismic Profile (VSP) testing to support the selection of a seismic Site Class for the site and site-specific ground-response analyses. VSP testing was carried out on June 25, 2018 to measure the shear wave velocity of the soil and bedrock at regular intervals of depth. Further discussion on the VSP testing methodology and results is provided in Appendix G.

Shear wave velocity testing was also carried out as part of the seismic cone penetration testing. A built-in geophone within the cone penetration probe recorded seismic wave traces from a surface source as the SCPTs were advanced. Measurements were recorded at roughly one-meter intervals. 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 the VSP casings. The site conditions were substantially restored following completion of work.

Test Hole Number	Test Hole Location	Northing <sup>1</sup> (m)	Easting <sup>1</sup> (m)	Ground Surface Elevation <sup>2</sup> (m)
BH18-01	North Abutment	5013055.7	360897.3	103.5
BH18-02	South Abutment	5012694.5	361054.6	95.7
BH/SCPT18-03	Adjacent to Pier N13	5013014.7	360932.7	93.5
BH/SCPT18-04	Between Piers N12/S12	5012990.4	360937.4	93.4
BH/SCPT18-05	Between Piers N11/S11	5012966.7	360946.9	93.1
BH/SCPT18-06	Adjacent to Pier S10	5012941.5	360949.5	92.9
BH/SCPT18-07	Adjacent to Pier S9	5012909.3	360963.5	92.3
BH/SCPT18-08	Between Piers N7 and S8	5012883.4	360982.3	92.4
BH/SCPT18-09	Adjacent to Pier N6	5012852.4	361000.9	92.4
BH/SCPT18-10	Adjacent to Pier S3	5012774.1	361022.1	91.9

<sup>1</sup> Northing and Easting coordinates shown are relative to the MTM NAD83 (Zone 9) coordinate system.

<sup>2</sup> Ground surface elevations shown are relative to Geodetic Datum.

The field work was supervised by members of Golder's technical and engineering staff, who located the boreholes, 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 and unconfined compressive strength tests were carried out on selected rock core samples at the Golder Ottawa laboratory. All laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

Following completion of the field work, the borehole and SCPT locations were 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 table above and are shown on Drawings 1 and 2.

### 3.2 Previous Investigations (1989 and 1991)

The original foundation investigations for the design of the existing bridges were carried out in 1989 and 1991. At that time, 31 boreholes were put down along the then-proposed bridge alignments. The investigation procedures and results were contained in the reports included as part of the GEOCRE Report No. 31G5-162 (see Section 1.0).

The boreholes were put down with a hollow-stem auger through the overburden to effective refusal. The boreholes within the southern portion of the then-proposed alignments and selected boreholes in the northern portion of the alignment (Boreholes 21A-8 through 21A-11, 21B-8 through 21B-11, 89-7, 89-7A, 89-87, 89-8A, 89-9, 89-9A, 89-10, and 16) were terminated upon auger refusal. The remaining boreholes, which were mostly put down within the northern portion of the site (Boreholes 21A-1 through 21A-7, and 21B-1 through 21B-7, 15, 31, and 32), were cored up to about 3.5 m into the underlying bedrock following auger refusal.

The approximate location and ground surface elevation at each borehole at the time of the 1989 and 1991 investigations were provided on the borehole records, included on the Record of Borehole sheets in Appendix E. The ground surface elevations were surveyed relative to Geodetic datum. The borehole locations in plan were surveyed relative to an unknown datum and were adjusted to be consistent with the MTM NAD83 (Zone 9) coordinate system based on prominent site features provided on the site plans included in the source GEOCRE reports. The approximate borehole locations and ground surface elevations at the borehole locations are shown on Drawings 1 and 2, and are summarized in the table below.

Test Hole Number	Test Hole Location	Northing <sup>1</sup> (m)	Easting <sup>1</sup> (m)	Ground Surface Elevation <sup>2</sup> (m)
BH21A-1	NBL, near Pier N8	5012906.7	360993.0	91.4
BH21A-2	NBL, between Piers N8 and N9	5012921.3	360986.9	91.4
BH21A-3	NBL, between Piers N8 and N9	5012921.9	360977.3	91.5
BH21A-4	NBL, between Piers N10 and N11	5012960.4	360970.8	91.8
BH21A-5	NBL, between Piers N11 and N12	5012982.4	360961.6	92.0
BH21A-6	NBL, near Pier N12	5012999.1	360954.5	92.3
BH21A-7	NBL, between Piers N12 and N13	5013013.8	360948.5	92.5
BH21A-8	NBL, near Pier N6	5012857.6	361008.8	91.3
BH21A-9	NBL, near Pier N7	5012885.2	360997.4	91.3
BH21A-10	NBL, between Piers N13 and N14	5013029.3	360937.4	94.4
BH21A-11	NBL, between Pier N14 and the North Abutment	5013053.6	360917.6	96.7
BH21B-1	SBL, near Pier S8	5012880.6	360962.7	91.3
BH21B-2	SBL, between Piers S8 and S9	5012895.3	360956.6	91.4
BH21B-4	SBL, near Pier S10	5012934.2	360940.4	91.8
BH21B-5	SBL, near Pier S11	5012956.3	360931.2	92.1
BH21B-6	SBL, between Piers S11 and S12	5012973.1	360924.2	92.3

Test Hole Number	Test Hole Location	Northing <sup>1</sup> (m)	Easting <sup>1</sup> (m)	Ground Surface Elevation <sup>2</sup> (m)
BH21B-7	SBL, between Piers S12 and S13	5012987.7	360918.1	92.5
BH21B-8	SBL, near Pier S6	5012837.4	360985.3	91.3
BH21B-9	SBL, near Pier S7	5012865.1	360973.7	91.3
BH21B-10	SBL, between Piers S13 and S14	5013013.6	360911.6	95.1
BH21B-11	SBL, between Pier S14 and the North Abutment	5013030.6	360899.6	96.4
BH89-7	SBL, near South Abutment	5012686.7	361047.9	91.3
BH89-7A	SBL, south of South Abutment	5012640.1	361067.1	91.3
BH89-8	NBL, near Pier N1	5012744.1	361056.0	91.3
BH89-8A	NBL, near South Abutment	5012716.6	361067.1	91.3
BH89-9	SBL, between Piers S3 and S4	5012779.0	361009.5	91.2
BH89-9A	SBL, between South Abutment and Pier S1	5012705.6	361040.1	91.2
BH89-10	NBL, near Pier N5	5012836.5	361017.6	91.3
15	NBL, near Pier N14	5013047.0	360930.0	97.0
16	NBL, near North Abutment	5013070.7	360920.2	100.0
31	SBL, near Pier S14	5013029.1	360905.5	95.0
32	SBL, near South Abutment	5013052.9	360895.7	98.8

<sup>1</sup> Northing and Easting coordinates shown are approximate and given relative to the MTM NAD83 (Zone 9) coordinate system.

<sup>2</sup> Ground surface elevations shown are relative to Geodetic Datum.

## 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 of the current geotechnical investigation 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 seismic cone penetration testing are provided in Appendix C, which includes the result of shear wave velocity tests and pore pressure dissipation tests. Photographs of the bedrock core from the current investigation are presented in Appendix D. The subsurface soil and groundwater conditions encountered in the boreholes put down as part of the original 1989 and 1991 investigations, and the results of related in situ and laboratory testing, are given on the Record of Borehole and Drillhole sheets contained in Appendix E and the geotechnical laboratory test result sheets contained in Appendix F.

The results of VSP testing conducted as part of the current investigation in Boreholes 18-01 and 18-02 are provided in Appendix G of this report.

The interpreted stratigraphic conditions along the centrelines of the existing bridges are shown on Drawings 1 and 2. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic section included on Drawings 1 and 2 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 subsurface conditions encountered at the site generally consist of a deposit of silty clay up to 11 m thick over glacial till overlying dolostone and sandstone 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 southbound Highway 416 pavement structure was penetrated at borehole 18-01. The pavement structure consists of 100 mm of asphaltic concrete overlying about 300 mm of Portland cement concrete. The pavement is underlain by about 5.0 m of embankment fill (Elevations 103.1 to 98.1). The embankment fill generally consists of a gravelly sand to sandy gravel with the Standard Penetration Test (SPT) “N” values ranging from 9 to 22 blows per 0.3 m of penetration indicating a loose to compact state of packing.

Borehole 18-02 was put down in the highway median between the south abutments. At the location of BH18-02 the embankment fill is about 4.4 m thick, extending down to about Elevation 91.3 m, and consists of a sand with trace to some silt, some gravel to gravelly. The SPT “N” values range from 16 to 31 blows per 0.3 m of penetration indicating a compact to dense state of packing. The fill is underlain by a layer of topsoil about 300 mm thick.

Boreholes 18-05 and 18-08 were put down below the existing bridges near the piers. Topsoil about 20 mm thick was encountered at both locations, and consisted of silt, clay, and sand containing organics, and trace amounts of gravel. The topsoil is underlain by about 0.9 m of granular fill consisting of gravelly silty sand. The SPT “N” values in the granular fill at both locations range from 8 to 24 blows per 0.3 m of penetration indicating a loose to compact state of packing. The granular fill is underlain by another thin topsoil layer that ranged from 100 to 300 mm thick.

Three grain size distribution tests were carried out on samples of the embankment and grade fill. The results of these tests are provided on Figure B1 in Appendix B. The measured water content of selected samples of the granular fill ranged from approximately 7 to 33 percent.

## 4.2 Silty Clay to Clay

A deposit of silty clay to clay containing silty sand seams was encountered beneath the existing grade fill under the bridges and the embankment fill at the south abutments. The silty clay to clay was also encountered at ground surface at the time of the 1989 and 1991 investigations, at the locations of the boreholes put down near the then-proposed pier locations. In general, the upper portion of the deposit has been weathered to a grey brown crust. The weathered crust extends to depths below the current ground surface ranging from about 3.0 m near the piers to about 6.0 metres at the south abutment (Elevations ranging from about 89 to 90 m).

South of the CNR line, the silty clay to clay deposit generally extends to depths ranging from about 7.7 to 9.5 m below the existing ground surface (Elevations ranging from about 83.3 to 87.0 m). The silty clay to clay north of the CNR line extends to depths ranging from about 7.1 to 3.7 m (Elevations ranging from about 86.5 to 94.3 m), generally getting thinner to the north.

The SPT “N” values in the weathered crust measured during the current investigation ranged from 1 to 8 blows per 0.3 m of penetration. The results of in situ vane testing carried out in the unweathered portion of the deposit measured undrained shear strength values ranging from about 12 to 70 kPa. In the upper 4 to 8 m of the unweathered portion (above about Elevation 86 m), the shear strength values generally ranged from about 15 to 30 kPa indicating a soft consistency.



The measured tip resistances within the deposit at the SCPT locations were generally in the range of 400 to 1,300 kPa, with localized “spikes” in the tip resistance of up to about 2,500 kPa which likely reflect interlayered silt and fine sand layers.

Three grain size distribution tests were carried out on samples of the silty clay deposit from the current investigation. The results of these tests are provided on Figure B2 in Appendix B. Several grain size distribution tests were carried out on samples of the silty clay to clay as part of the 1989 and 1991 investigations. Gradation envelopes representing the results of these tests are provided in Appendix F.

Atterberg limit tests carried out on samples of the silty clay to clay deposit obtained from the previous and current investigations gave plasticity index values ranging from about 20 to 52 percent and liquid limit values ranging from about 35 to 75 percent, indicating a medium to high plasticity soil. The measured water content ranged from approximately 30 to 50 percent in the weathered crust, and about 50 to 90 in the unweathered portion of the deposit.

Oedometer consolidation testing was carried out on four Shelby tube samples from the grey silty clay deposit at Boreholes 21A-1, 21B-6, and 21B-2 following the 1989 investigation. The results of this testing are summarized in the table below.

Borehole/ Sample Number	Sample Depth/Elevation (m)	$\sigma_p'$ (kPa)	$\sigma_o'$ (kPa)	$C_c$	$C_r$	$e_o$	OCR
21A-1 / 7	5.5 / 85.9	105	35	1.65	-	1.91	3.0
21A-6 / 4	3.3 / 89.0	68	18	1.62	-	2.32	3.8
21B-2 / 5	4.1 / 87.3	62	20	2.54	-	2.51	3.1
21B-6 / 2	2.6 / 89.7	68	15	2.12	-	2.31	4.5
89-1 / 6	8.0 / 83.2	102	58	1.09	-	0.70	1.8
89-2 / 7	9.4 / 81.9	102	71	1.74	-	1.89	1.4
89-8 / 3	3.3 / 88.0	95	30	1.75	-	2.19	3.2
89-8 / 6	7.9 / 83.4	105	58	0.86	-	1.49	1.8
89-10 / 3	3.2 / 88.1	85	25	0.91	-	1.71	3.4

$\sigma_o'$  - Initial effective stress

$C_c$  - Compression index

$e_o$  - Initial void ratio

$\sigma_p'$  - Apparent preconsolidation pressure

$C_r$  - Recompression index

OCR - Overconsolidation Ratio

### 4.3 Glacial Till

Glacial till was encountered beneath the silty clay to clay and the grade and embankment fill at the northern extent of the site. In general, the deposit consists of a heterogeneous mixture of silty clay to clayey silt to silt, sand and gravel to a more granular mixture of silt, sand, and gravel. The glacial till deposit also contains cobbles and boulders.

South of the CNR line, the glacial till was generally encountered at depths ranging from about 7.9 to 9.5 m below the current existing ground surface. The glacial till extends down to elevations ranging from about 79 to 80 m and, where fully penetrated, the glacial till has a thickness ranging from about 3.3 to 5.0 m.

North of the CNR line, the glacial till was encountered below the silty clay, except at Boreholes 15, 16, 32, and 21A-11 where it was encountered at the ground surface, and at 18-01 where it was encountered below the embankment fill. The glacial till was encountered at depths up to 2.1 to 6.2 m below the existing ground surface, generally getting shallower to the north. Where fully penetrated, the glacial till extends down to elevations ranging from about 78 to 83 m.

Discontinuous layers of sandy silt to sand with gravel were encountered within the glacial till deposit, generally north of the CNR line. The layers were generally encountered at depths greater than about 5 m and have thicknesses that ranges from about 1.0 to 5.0 m.

SPT “N” values in the glacial till ranged from 2 to greater than 100 blows per 0.3 m of penetration, but generally greater than 10 blows, indicating a compact to very dense state of packing. Diamond drilling techniques were required to penetrate portions of the deposit.

Atterberg limit tests carried out on samples of the cohesive portions of the deposit recovered during the 1989 investigation gave plasticity index values of about 2 to 23 percent and liquid limit values of about 15 to 37 percent, indicating a low plasticity soil. A summary of the Atterberg limits are provided in Appendix F. The measured water content of the deposit ranged from approximately 6 to 54 percent.

The results of grain size distribution testing carried out on seven samples of the glacial till deposit collected during the current investigation are provided on Figure B3 in Appendix B. Gradation envelopes representing the results of the grain size tests carried out on samples of the glacial till deposit recovered during the 1989 and 1991 investigations are provided in Appendix F.

#### 4.4 Refusal and Bedrock

At all locations where borehole drilling penetrated the glacial till, bedrock was encountered at elevations ranging from about 78 to 83 m. The bedrock was cored for lengths of 5.2 and 5.4 m at the abutments during the current investigation, and lengths ranging from about 1.3 to 3.5 m during the 1989 and 1991 investigations. Photographs of the bedrock recovered from the current investigation are provided in Appendix D.

The bedrock recovered from the 1989 and 1991 investigations was described as silty dolostone and dolomitic shale. During the current investigation, dolostone was encountered in borehole 18-02 at the south abutments, and about 3.4 m of sandstone was encountered overlying dolostone, interbedded with shale in borehole 18-01 at the north abutments.

The Rock Quality Designation (RQD) values measured on the bedrock samples collected as part of the current investigation ranged from about 57 to 100 percent, but were generally above 80 percent, indicating a good to excellent quality rock. The RQD values measured during the 1989 and 1991 investigations ranged from about 0 to 97 percent, but were generally greater than 40 percent, indicating a poor to excellent quality rock.

Laboratory unconfined compressive strength testing was carried out on selected specimens of the bedrock core recovered as part of the current investigation. The results of the testing are summarized in Appendix B and indicate values of 127 MPa (in the sandstone) and 187 MPa (in the dolostone).

#### 4.5 Groundwater Conditions

During the current investigation, ground water levels were inferred from the results of dissipation tests carried out in the granular layers during the SCPT advancement. The inferred ground water levels at the SCPT locations are presented in the table below:

Borehole	Ground Surface Elevation (m)	Water Level Depth <sup>1</sup> (m)	Water Level Elevation <sup>1</sup> (m)	Date
SCPT18-03	93.5	1.6	91.9	May 16, 2018
SCPT18-04	93.4	1.6	91.8	May 16, 2018
SCPT18-05	93.1	1.6	91.5	May 16, 2018

Borehole	Ground Surface Elevation (m)	Water Level Depth <sup>1</sup> (m)	Water Level Elevation <sup>1</sup> (m)	Date
SCPT18-06	92.9	1.7	91.2	May 15, 2018
SCPT18-07	92.3	1.8	90.5	May 14, 2018
SCPT18-08	92.4	1.9	90.5	May 15, 2018
SCPT18-09	92.4	1.9	90.5	May 15, 2018
SCPT18-10	91.9	1.8	90.1	May 15, 2018

<sup>1</sup> Ground water level inferred from SCPT data

Ground water levels were recorded in the open boreholes during the 1989 and 1991 investigations and were encountered generally at or up to 1.0 m below the existing ground surface (about Elevation 91 to 92 m) with the exception of the area around the north abutment where the groundwater was encountered at about 4.6 to 6.0 m depth (Elevations 92 to 94 m) at the time of drilling.

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 Investigation 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.

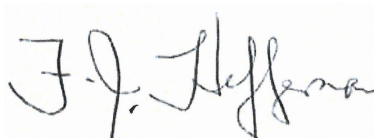
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**PART B**

Foundation Design Report  
Strandherd Drive and CNR Overpass Bridges  
Site No. 3-549, Highway 416  
Ottawa, Ontario  
W.P. 4133-01-01 & 4134-01-01

## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides foundation design recommendations for the seismic evaluation and structural rehabilitation of the Highway 416 overpass bridges at Strandherd Drive and the CNR tracks. The recommendations are based on interpretation of the factual data obtained from the test holes advanced during the current subsurface investigation, as well as that previously collected at the site by others and included in the GEOCRE information. The discussion and recommendations presented below are intended to provide the designers with sufficient information to carry out a seismic evaluation and structural rehabilitation of the structure as part of the design of rehabilitation measures that comply with the 2014 Canadian Highway Bridge Design Code (CHBDC, S6-14).

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.

### 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  $M_N$ ) 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 5<sup>th</sup> generation seismic hazard maps) that were made available for public use in December 2015.

#### 6.2.2 Seismic Site Classification

Vertical Seismic Profile (VSP) geophysical testing was carried out at the existing bridge location to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site (i.e.  $V_{s30}$ ). The shear wave velocities measured are presented in a technical memorandum included in Appendix G. In addition, the SCPT results and measured shear wave velocity of the soil profile were considered in the selection of the appropriate site class.

The shear wave velocity results indicate that at the north abutment, which is supported on spread footings founded directly on the glacial till, the  $V_{s30}$  was calculated to be about 950 m/s. At the south abutment, which is supported on piles end bearing on bedrock, the  $V_{s30}$  was calculated to be about 460 m/s. Using the SCPT results and extrapolating the VSP results in the deeper strata, the  $V_{s30}$  at the piers was calculated to range from about 300 to 510 m/s.

Table 4.1 of the CHBDC contains a requirement at locations where more than 3 m of soft silty clay with  $PI > 20\%$ , water content  $> 40\%$ , and undrained shear strengths of less than 25 kPa be classified as a Site Class of E. Based on the presence of greater than 3 m of such soft silty clay at many of the pier locations, it is considered that a Site Class E should be used at Piers N1/S1 to N13/S13.

Boreholes put down near the south abutments as part of the current investigation (Borehole 18-02) and previous investigations (Boreholes 89-7, 89-8A, and 89-9A) indicate that the undrained shear strengths are generally greater than 25 kPa in this area. Therefore, the Site Class at the south abutment can be assessed considering the average shear wave velocity of 460 m/s alone, resulting in a Site Class C.

Based on the results of the shear wave velocity measurements at the north end of the bridges, it is considered that a Site Class C designation is appropriate for the spread footings founded directly on the native glacial till (Piers N14/S14 and north abutments).

However, Table 4.1 of the CHBDC 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 presented below in Section 6.2.7, the soil at this site may be considered to be non-liquefiable for design.

### 6.2.3 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. The results of the subsequent liquefaction assessment indicated that the site soils may be considered to be non-liquefiable for design. As such, a site-specific ground response analysis is no longer strictly required by the requirements of Table 4.1 of the CHBDC, but 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 site-specific design spectra. 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 Target Spectrum

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the bridge, the following are the reference (Site Class C) peak seismic hazard values based on data obtained from Earthquakes Canada ([www.earthquakescanada.nrcan.gc.ca](http://www.earthquakescanada.nrcan.gc.ca)).

**Site Class C Spectral Values for Subject Site**

Parameter	Value at Given Probability of Exceedance in 50 Years		
	10% (475-year)	5% (975-year)	2% (2,475-year)
PGA	0.093 g	0.151 g	0.264 g
$T \leq 0.2$ s	0.149 g	0.238 g	0.413 g
$T = 0.5$ s	0.083 g	0.131 g	0.224 g
$T = 1.0$ s	0.043 g	0.066 g	0.112 g
$T = 2.0$ s	0.020 g	0.031 g	0.054 g
$T = 5.0$ s	0.005 g	0.008 g	0.014 g
$T \geq 10.0$ s	0.002 g	0.003 g	0.005 g

Based on the results of the VSP testing carried out at the site, the bedrock has an average shear wave velocity of 1,900 m/s, which corresponds to a Site Class A, as defined in Table 4.1 of the CHBDC. The reference peak seismic hazard values (Site Class C) were modified to the site-specific bedrock classification (Site Class A) using Table 4.2 to 4.8 in the Section 4.4.3.3 in the CHBDC. The site-specific, target firm-ground spectra are given in the table below.

#### Site Class A Spectral Values for Subject Site

Parameter	Value at Given Probability of Exceedance in 50 Years		
	10% (475-year)	5% (975-year)	2% (2,475-year)
PGA	0.084 g	0.136 g	0.238 g
T ≤ 0.2 s	0.103 g	0.164 g	0.285 g
T = 0.5 s	0.047 g	0.075 g	0.128 g
T = 1.0 s	0.025 g	0.038 g	0.064 g
T = 2.0 s	0.012 g	0.018 g	0.031 g
T = 5.0 s	0.003 g	0.005 g	0.009 g
T ≥ 10.0 s	0.001 g	0.002 g	0.003 g

### 6.2.5 Spectrum-Compatible Time Histories

To develop time histories compatible with the target firm-ground spectrum, a hazard deaggregation was first carried out to identify the primary contributors of earthquake magnitude and hypocentral distance for each of the 975-year and 2,475-year design earthquake events. A suite of representative seed time histories that matched the primary contributors were selected for each design earthquake. The time histories were then linearly scaled to match the Site Class A target spectra to represent the site-specific design firm-ground accelerations, for use in the site-specific ground response analyses.

Linear scaling of the time histories was selected as the preferred method to best match the target spectrum. Since code-based, target spectra have a plateau at periods less than 0.2 s, linearly scaling seed time histories that are selected to provide a suitable match to the target spectrum result in time histories that provide a more realistic representation of the earthquake. Spectrally matching time histories to fit both the lower period plateau and the higher period values of the target spectrum can result in significant modification of the seed time history, resulting in an earthquake record that is less representative of the recorded ground motions.

The period range of interest for selection and matching of time histories to the target spectra was taken as 0.075 s to 4.0 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. Further details on the selection of seed time histories and scaling to the target spectra are presented in the memorandum included in Appendix H.

A summary of the earthquake records used in the site-specific ground response analyses for each design earthquake are provided in the tables below. Plots of the Site Class A scaled spectral accelerations of the input time histories, along with the target Site Class A spectra, for each of the 2,475-year, 975-year, and 475-year design earthquakes are shown on Figures I1 to I3 in Appendix I.



**Summary of Input Time History Earthquake Events – 2,475-Year Design Earthquake**

Database	Event Name	Event Year	Station / Suite Name	Mag.	Dist. (km)	Scaling Method
PEER	Northridge	1994	LA - Wonderland Ave	6.7	15	Linear Scaling
PEER	Northridge	1994	Vasquez Rocks Park	6.7	23	Linear Scaling
PEER	Hector Mine	1999	Twenty-nine Palms	7.1	42	Linear Scaling
PEER	Norcia Italy	1979	Cascia	5.9	1	Linear Scaling
EST	Motion #31	-	East7a1 Suite	7.0	26	Linear Scaling
EST	Motion #33	-	East7a1 Suite	7.0	26	Linear Scaling
EST	Motion #37	-	East7a1 Suite	7.0	26	Linear Scaling
EST	Motion #39	-	East7a1 Suite	7.0	26	Linear Scaling
EST	Motion #40	-	East7a1 Suite	7.0	26	Linear Scaling
EST	Motion #01	-	East7a2 Suite	7.0	42	Linear Scaling
EST	Motion #26	-	East7a2 Suite	7.0	70	Linear Scaling

**Summary of Input Time History Earthquake Events – 975-Year and 475-Year Design Earthquakes**

Database	Event Name	Event Year	Station / Suite Name	Mag.	Dist. (km)	Scaling Method
PEER	Coyote Lake	1979	Gilroy Array #1	5.7	11	Linear Scaling
PEER	Morgan Hill	1984	Gilroy Array #1	6.2	15	Linear Scaling
PEER	Chi-Chi	1999	CHY102	6.2	39	Linear Scaling
PEER	Alum Rock	2007	Lick Observatory, Mt. Hamilton	5.5	14	Linear Scaling
PEER	Hector Mine	1999	Twentynine Palms	7.1	42	Linear Scaling
PEER	Sparks	2011	Luther Middle School	5.7	41	Linear Scaling
EST	Motion #02	-	East7a2 Suite	7.0	42	Linear Scaling
EST	Motion #10	-	East7a2 Suite	7.0	50	Linear Scaling
EST	Motion #26	-	East7a2 Suite	7.0	70	Linear Scaling
EST	Motion #30	-	East7a2 Suite	7.0	48	Linear Scaling
EST	Motion #40	-	East7a2 Suite	7.0	94	Linear Scaling

**6.2.6 One-Dimensional Ground Response Analyses**

One-dimensional ground response analyses were undertaken to assess the ground response at the site. Two stratigraphic profiles were selected for analysis, representative of the north and south bridge soil conditions, respectively. The ground response analyses were carried out for each of the two profiles, for each of the 2,475-year, 975-year and 475-year design earthquakes.

Based on the results of the field investigation and an average shear wave velocity of 1,900 m/s for the bedrock, representative index properties and shear wave velocity variations of the overburden soil and rock were developed for each design soil profile, and are summarized in the table below.



### Summary of Representative Stratigraphy and Material Properties

Soil Unit	$\gamma$ (kN/m <sup>3</sup> )	North Profile		South Profile	
		Depth (m)	$V_s$ (m/s)	Depth (m)	$V_s$ (m/s)
Silty Clay	16	0 – 5.0	116 – 203	0 – 7.5	110 – 200
Silty Clay / Clayey Silt	18	5.0 – 6.0	291 – 680	7.5 – 8.3	320 – 510
Glacial Till	20	6.0 – 11.5	1070 – 1080	8.3 – 10.5	610 – 990
Bedrock	23	> 11.5	1900	> 10.5	1900

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:

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

#### 6.2.6.1 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:

- Silty Clay: Vucetic and Dobry (1991) for  $I_p = 30\%$ ;
- Silty Clay to Clayey Silt: Vucetic and Dobry (1991) for  $I_p = 30\%$ ;
- Glacial Till: Seed and Idriss (1970) average curves for shear modulus and damping; and,
- Bedrock: Schnabel, 1973.

#### 6.2.6.2 Analysis Results

The ground response analyses were carried out to quantify the site-specific ground response to develop site-specific design spectra and as input to the updated liquefaction assessment, for each of the three design earthquakes.

The acceleration time histories at the ground surface were obtained from the site-specific ground response analyses. The averages of the time history response spectra at the ground surface for each of the North Profile, the South Profile, and the geomean of the North and South Profiles (to provide an average, representative response spectrum for the overall site) are shown on Figure I4 to I6 for each of the 2,475-year, 975-year, and 475-year design earthquakes. For comparison, these average modeled response spectra are plotted relative to the Site Class E design response spectra defined in the CHBDC, along with the recommended design spectra for each of the design earthquakes.

The Cyclic Stress Ratio (CSR) values with depth were also calculated as part of the site-specific ground response analyses and used in the updated liquefaction assessment described below.

## 6.2.7 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.

The liquefaction susceptibility of granular soils was evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required.

The methodology used to assess liquefaction potential at the site is consistent with the approach outlined in the CHBDC and by Idriss and Boulanger (2008). It 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 CRR values with depth were calculated using the existing (GEOCRE) information as well as the borehole and SCPT data collected as part of the 2018 investigation. However, the SCPTs were not able to penetrate very far into the glacial till deposit underlying the silty clay at the site and, therefore, did not provide subsurface information relevant to liquefaction assessment within the glacial till. The results of the initial preliminary liquefaction analyses, using the “simplified” method of estimating the CSR profile, indicated localized and discontinuous layers within the glacial till deposit that underlies the silty clay that are potentially liquefiable during the 2,475-year design earthquake.

The site-specific CSR values with depth for each design earthquake were then calculated as part of the ground-response analyses and used in the updated liquefaction assessment. As suggested by Table C4.4 in the CHBDC Commentary (after Youd and Perkins, 1978) glacial till generally has a low to very low potential for liquefaction. The updated liquefaction analyses were re-assessed to consider an increased CRR in the glacial till to account for aging effects of this Holocene-aged deposit. The CRR was increased by 20 to 30% based on work done by Leon et al (2006).

The results of the updated liquefaction assessment indicate that the site may be considered to be non-liquefiable for design under all three design earthquakes.

The susceptibility of the silty clay to clay deposit to cyclic mobility was assessed based on the methodology provided in Idriss and Boulanger (2008), in which the CRR for clay-like soil is calculated based on the undrained shear strength and approximate OCR of the soil. The CRR is equated with the CSR (for reference stress equal to 65% of peak shear stress) to calculate the factor of safety against cyclic softening that would be expected to result in greater than 3% shear strain. Based on the results of the analyses, the silty clay is not considered to be susceptible to cyclic softening.

## 6.3 Existing Foundations

Based on the 1991 structural design drawings (Cont. 91-46, WP No 128-87-05/06) the fifteen-span bridges are founded on a combination of deep foundations consisting of HP 310 x 110 piles driven to bedrock at the south abutments and Piers N1/S1 to N13/S13, and shallow spread footings founded on the glacial till at Piers N14/S14 and the north abutments.

The Pier N14/S14 footings are 1.5 m thick and roughly square in plan measuring 5.2 m by 5.2 m. The footings at the north abutments are 0.8 m thick and roughly rectangular in plan measuring 13.4 m by about 3.4 m.

### 6.3.1 Geotechnical Resistance

The 1991 structural design drawings indicate that the HP 310 x 110 piles supporting the south abutments and Piers N1/S1 to N13/S13 were driven to refusal on the limestone bedrock. The HP 310x110 piles may be considered to have a factored geotechnical resistance of 2,000 kN at Ultimate Limit States (ULS). SLS resistances do not apply to piles driven to the bedrock, because the SLS resistance for 25 mm of settlement will be greater than the factored axial geotechnical resistance at ULS.

Assuming that the piles at the south abutment were driven prior to placement of the embankment fill, the increase in effective stress resulting from embankment loads in the soft to stiff silty clay that underlies the site may have produced settlement that generated downdrag loads on the piles supporting the south abutments. These downdrag loads (i.e., negative skin friction) should be considered in the structural assessment. The unfactored downdrag load acting on a single HP 310x110 pile is estimated to be about 310 kN at the south abutment.

The structural capacity of the piles should be checked for the factored dead and downdrag loads in accordance with the Canadian Highway Bridge Design Code (CHBDC).

The silty clay and glacial till at the site will provide uplift resistance to the piles. The skin friction contributing to the uplift values on a single HP 310 x 110 pile were calculated using the  $\alpha$ -method based on measured/interpreted shear strength values in the cohesive deposits and the Nordlund method based on pile type/size and friction angle in the granular deposits. The factored uplift resistance on a single HP 310x110 pile at each pier location is summarized in the table below.

As outlined in Section C6.11.3.2 of the Commentary to the CHBDC, the group efficiency of driven piles in cohesionless soils are typically taken as 1.0. The group efficiency of piles in cohesive soil is often taken as the lesser of the sum of resistance of individual piles and the resistance of the group acting as a block. Based on the original foundation layout drawings, a typical foundation element consists of seven HP 310 x 110 piles (total pile circumference of approximately 8.6 m) within a 3.5 m square grid (total block circumference of about 12.3 m), therefore the group efficiency may be taken as 1.0.

#### Factored Uplift Resistance of a Single 310 x 110 Pile

Bridge Structure	Pier No.	Factored Uplift Resistance
NBL Structure	N1 to N11	125 kN
	N12	175 kN
	N13	240 kN
SBL Structure	S1 to S11	125 kN
	S12	175 kN
	S13	240 kN

The northernmost piers (Piers N14/S14) and north abutments are founded on spread footings bearing on the native glacial till. For the spread footings at Piers N14/S14, a factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kPa and a factored geotechnical resistance of 300 kPa at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for the structural assessment. For the spread footings on glacial till at the north

abutments, a factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kPa and a factored geotechnical resistance of 200 kPa at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for the structural assessment.

## 6.4 Lateral Earth Pressures

The lateral earth pressures acting on the abutment and wing walls 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.

It is understood that a significant portion of the south embankments were constructed with lightweight fill (slag) to reduce the potential post-construction settlements of the embankments. However, it is unclear what material was placed directly behind the abutments for drainage. The GEOCRETS foundation design report recommends Granular A backfill. Based on the information available, abutment backfill consisting of either Granular A or Granular B may be considered for calculation of lateral earth pressures in the structural assessment.

The static lateral earth pressures may be calculated using the lateral earth pressure coefficients provided in the table below.

	Granular A	Granular B
Soil Unit Weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Active, $K_a$	0.27	0.33
At rest, $K_o$	0.43	0.50
Passive, $K_p$	3.70	3.00

Where the abutment walls do not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where the abutment walls allow lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure(s). The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the CHBDC.

The seismic active earth pressure acting on the north and south abutment walls can be calculated using the seismic active earth pressure coefficients ( $K_{AE}$ ) provided in the table below.

Wall Type	Coefficient Value ( $K_{AE}$ )	
	Granular A	Granular B
Yielding	0.35	0.42
Non-yielding	0.44	0.53

In accordance with Sections 4.6.5 and C.4.6.5 of the 2014 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 equal to the site adjusted PGA estimated at the ground surface (Site Class C at both the north and south abutments). For structures which allow lateral yielding,  $k_h$  is taken as 0.5 times site adjusted PGA estimated at the ground surface.

It should be noted that the seismic earth pressure coefficients provided in the table above were calculated considering 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.

The  $K_{AE}$  value for a yielding wall is applicable provided that the wall can move up to  $250k_h$  mm, where  $k_h$  is the site adjusted PGA estimated at the ground surface. At the north abutments (Site Class C), the ground surface PGA is 0.26 g. At the south abutments (Site Class E), the site adjusted ground surface PGA is 0.32 g. This corresponds to displacements of up to approximately 65 mm and 80 mm at the north and south abutments, respectively.

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:

$$\sigma_h(z) = K_A \gamma z + (K_{AE} - K_A) \gamma (H-z) \text{ for yielding walls;}$$

$$\sigma_h(z) = K_0 \gamma z + (K_{AE} - K_A) \gamma (H-z) \text{ for non-yielding walls; and,}$$

Where:  $\sigma_h(z)$  is the lateral earth pressure at depth 'z' (kPa);

$K$  is the static active earth pressure coefficient ( $K_A$ ) or the static at-rest earth pressure coefficient ( $K_0$ );

$K_{AE}$  is the seismic earth pressure coefficient;

$\gamma$  is the unit weight of the backfill soil ( $\text{kN/m}^3$ ), use  $20 \text{ kN/m}^3$ ;

$z$  is the depth below the top of the wall (m); and,

$H$  is the total height of the wall (m).

## 6.5 Lateral Soil-Structure Interaction Springs

The foundation lateral soil-structure interaction springs required for the dynamic analysis of the pile supported south abutments and Piers N1/S1 to N13/S13, as well as the spread footing supported Piers N14/S14 and north abutments were computed based on the available subsurface information on the soil layers surrounding the foundations and the pile/footing dimensions.

### 6.5.1 P-y Curves for Deep Foundations

The lateral response of the existing pile foundations can be analyzed considering the soil-structure interaction between the pile(s) and the surrounding soils or bedrock using the load transfer method. From a geotechnical perspective, the lateral load-displacement behaviour of the soils and bedrock can be modeled using P-y curves as described in Section C6.11.2.2.1 of the Commentary to the CHBDC.

P-y curves representing the non-linear response of the soils and bedrock under lateral loading from the pile foundations have been generated using the commercially available program LPILE (version 2015.8.08) produced by ENSOFT Inc. Given that the structural analysis is to consider the dynamic loading on the structure, the P-y curves have been calculated considering the cyclical loading option for the lateral soil models. The family of P-y curves were calculated at 0.5 m to 1.0 m depth increments for a single HP310x110 steel pile. Due to the

variability in soil stratigraphy along the length of the bridge, two representative soil stratigraphies were considered: the south abutments and Piers N1/S1 to N10/S10; and, Piers N11/S11 to N13/S13. Tabular and graphical presentations of the P-y curves are included on the Figures in Appendix J.

The P-y curves are for a single pile and do not include any effects of group action. Group action for lateral loading must be considered when the pile spacing in a group in the direction of loading is less than seven (7) pile diameters. These 'Group Effects' can be incorporated into the analysis using a method that modifies the single pile P-y curve(s) by a reduction factor termed a 'P-Multiplier'. Generalized P-Multipliers (or reduction factors) for a range of pile spacings and loading directions are provided in Section C6.11.3.4 of the Commentary to CHBDC.

### 6.5.2 Lateral Resistance of Pile Caps

The soil adjacent to the pile caps at Piers N1/S1 to N13/S13 will also provide some lateral resistance (in addition to that of the piles represented by the P-y curves). Based on the assumed construction methodology and backfill present adjacent to these pier pile caps, the resistance to lateral loading can be derived in accordance with the recommendations provided in Section C.4.6.4 of the CHBDC Commentary describing near-field lateral springs (Caltrans, 2013) using the following relationship:

$$K = (K_i)(w)(h)/1.7$$

Where: K is the soil stiffness (kN/mm, up to a maximum passive pressure of 240 kilopascals);

$K_i$  is the initial soil stiffness (taken as 14.4 kN/mm per metre width, as outlined in Caltrans, 2013);

w is the effective width of the pile cap (metres); and,

h is the embedded height of pile cap (metres).

The relationship provided above is applicable provided that the width of backfill surrounding the pile cap is at least three times the embedded height of the pile cap. Caution should be used in relying on such lateral resistance within the frost zone as frost-susceptible soils experience material reductions in strength and stiffness during periods of thaw. If the existing backfill around the pile cap is removed and replaced with well compacted, non-frost-susceptible, free-draining engineered granular fill (such as Granular A or Granular B Type I or II), the initial embankment fill stiffness ( $K_i$ ) may be taken as 28.7 kN/mm per metre width, as outlined in Caltrans (2013).

### 6.5.3 Compliance Springs for Shallow Foundations

Shallow foundation compliance springs were developed for the dynamic analysis of Piers N14/S14 and the north abutments. Upper and lower bound spring values are provided considering that the stress-strain relationship of soil and bedrock is highly non-linear and the stiffness values of the foundation system depend on the stress and strain levels that the subsurface soils will be subjected to under the applied loadings.

The foundation compliance springs were computed as described in Section C4.6.4 of the CHBDC using the methodology to determine the load-deflection characteristics of shallow bearing foundations outlined in the FEMA document titled "*Prestandard and Commentary for the Seismic Rehabilitation of Buildings*" (FEMA-356, dated November 2000). The methodology considers the footing dimensions, the depth of embedment below the ground surface, and the engineering properties of the soil or bedrock on which the footing is founded. The dimensions and details of Piers N14/S14 and the north abutments were based on those included in the contract drawings.

The analysis was carried out using shear moduli estimated from in-situ testing results presented in the available GEOCRE information and engineering judgement. An average shear wave velocity of 560 m/s was estimated for the glacial till supporting the spread footings at Piers N14/S14 and the north abutments based on the shear wave velocity measurements at borehole 18-01. The effects of the proximity of the nearby bedrock layer underlying the glacial till were considered using the formulae provided by Gazetas (1993), as recommended in the Commentary to the CHBDC.

The shear modulus values used to calculate the upper and lower bound compliance springs were taken as 50 percent and 20 percent of the estimated maximum shear modulus values, as recommended in Section C4.6.4 of the CHBDC.

	Bound Limit	Horizontal Translation X-Direction (kN/m)	Horizontal Translation Y-Direction (kN/m)	Vertical Translation Z-Direction (kN/m)	Rotation about X-Axis (kNm/rad)	Rotation about Y-Axis (kNm/rad)	Torsion about Z-Axis (kNm/rad)
Piers N14/S14	Upper (0.5·G <sub>max</sub> )	1.79E+07	1.79E+07	1.76E+07	7.02E+07	6.97E+07	1.32E+08
	Lower (0.2·G <sub>max</sub> )	7.16E+06	7.16E+06	7.02E+06	2.81E+07	2.79E+07	5.30E+07
North Abutments	Upper (0.5·G <sub>max</sub> )	1.83E+07	2.05E+07	1.79E+07	5.16E+07	3.01E+08	3.90E+08
	Lower (0.2·G <sub>max</sub> )	7.31E+06	8.21E+06	7.17E+06	2.07E+07	1.21E+08	1.56E+08

**Notes:** 1) The X-Axis is transverse to the bridge alignment (towards project west), Y-Axis is along the bridge alignment (towards project north), and Z-Axis is vertical downward.  
 2) The forces and translations are positive if they follow the positive direction of the corresponding axis. Moments and rotations are positive if they act in the clockwise direction when viewed from the origin in the positive direction of the pertinent axis.

The compliance springs have been computed for longitudinal, transverse, vertical, and rotational, modes of excitation. The compliance springs were computed at the base centroids of the footings in plan (from the top view) and the springs are applicable for foundation response at the computed locations. These spring stiffness values may need to be adjusted to suit the structural model, if the locations of load application are different from the computed locations.

The computed foundation compliance springs are summarized in the table above. The springs were calculated relative to axes oriented along the overall bridge alignment. In the table, the X-axis is defined as transverse to the bridge alignment (positive towards project west), the Y-axis is defined as along the bridge alignment (positive towards project north), and the Z-axis is defined as vertical downward.



## 7.0 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.

### Golder Associates Ltd.



Matt Kennedy, P.Eng.  
*Senior Geotechnical Engineer*



Michael Snow, P.Eng.  
*Principal, Senior Geotechnical Engineer*



Fintan Heffernan, P.Eng.  
*Designated MTO Foundations Contact*

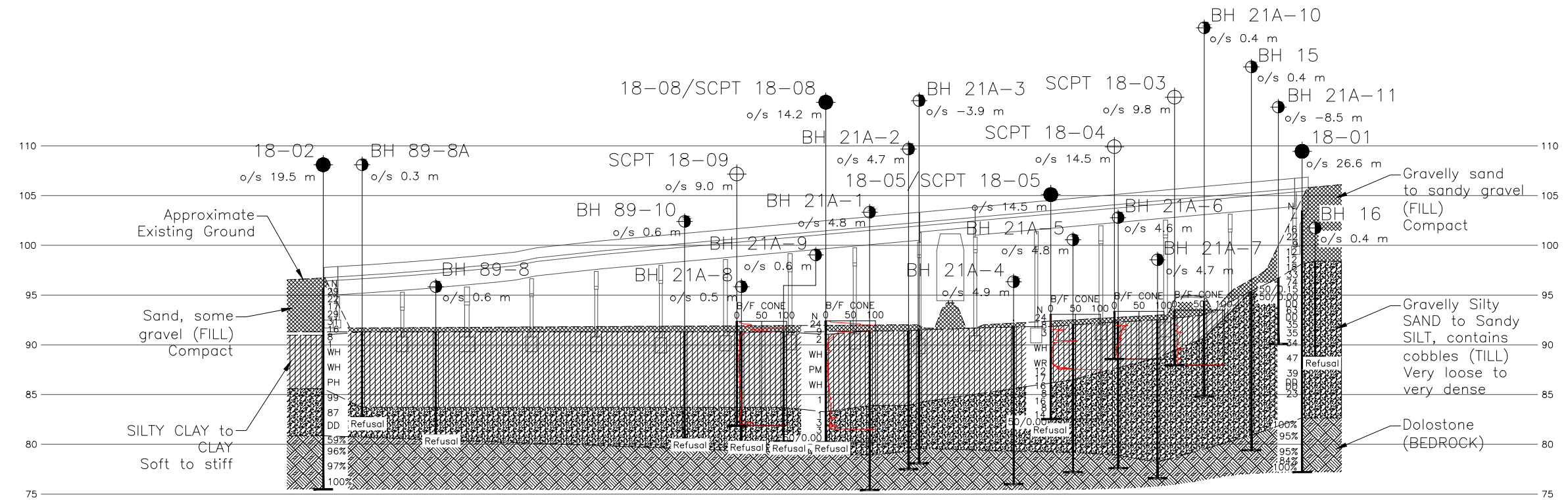
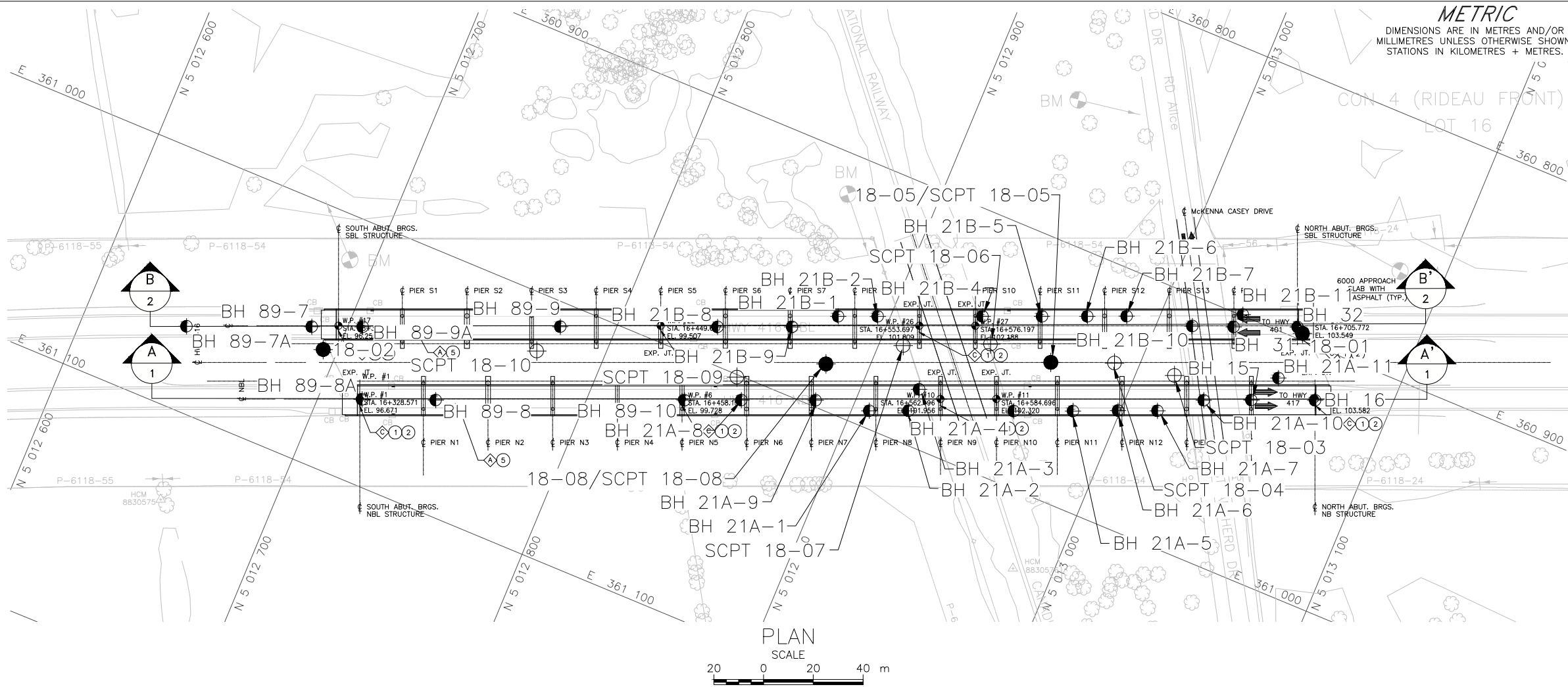


MJK/MSS/FJH/mvrd

[https://golderassociates.sharepoint.com/sites/21451g/08\\_reports/phase 1231 - strandherd cnr bridges/1417217-1231 rpt-001 final strandherd cnr site 3-549 march 2019.docx](https://golderassociates.sharepoint.com/sites/21451g/08_reports/phase%201231-strandherd%20cnr%20bridges/1417217-1231_rpt-001_final_strandherd_cnr_site_3-549_march_2019.docx)

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#### NOTES

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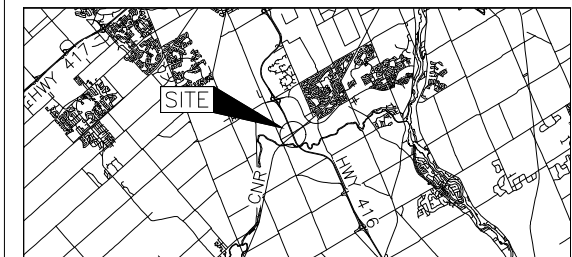
#### REFERENCE

CONT No.

WP No. 4133-01-01 &  
4134-01-01

HIGHWAY 416 OVERPASS  
BRIDGES AT STRANDHERD DRIVE  
AND CNR

BOREHOLE LOCATIONS AND SOIL STRATA



KEY PLAN  
SCALE  
4 0 4 6 km

#### LEGEND

- Borehole - Current Investigation
- ⊕ Dynamic Cone Penetration Test - Current Investigation
- Borehole - Previous Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated  
(Std. Pen. Test, 475 j/blow)
- 100% Total Core Recovery (REC)

#### BOREHOLE CO-ORDINATES (MTM ZONE 9)

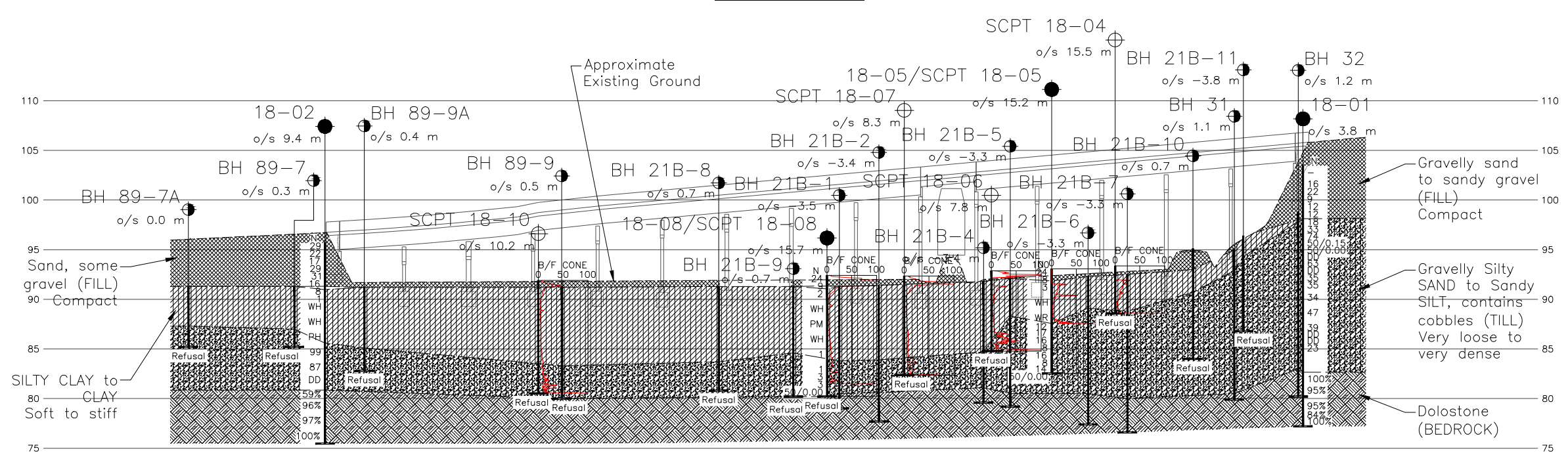
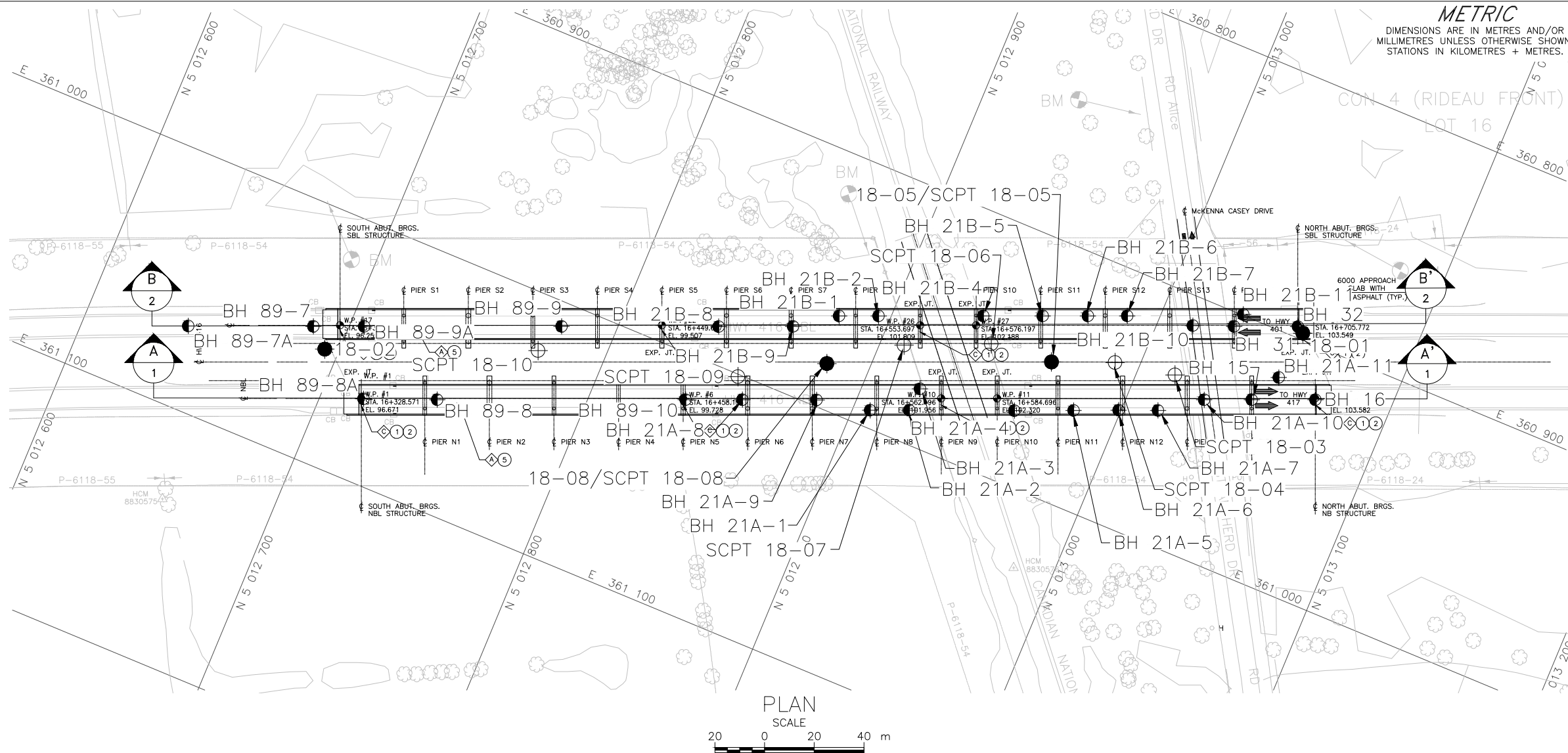
No.	ELEVATION	NORTHING	EASTING
18-01	103.5	5013055.7	360897.3
18-02	95.7	5012694.5	361054.6
18-05/SCPT 18-05	93.1	5012966.7	360946.9
18-08/SCPT 18-08	92.4	5012883.4	360982.3
SCPT 18-03	93.5	5013014.7	360932.7
SCPT 18-04	93.4	5012990.4	360937.4
SCPT 18-06	92.9	5012941.5	360949.5
SCPT 18-07	92.3	5012909.3	360963.5
SCPT 18-09	92.4	5012852.4	361000.9
SCPT 18-10	91.9	5012774.1	361022.1
BH 15	97.0	5013047.0	360930.0
BH 16	100.0	5013070.7	360920.2
BH 21A-1	91.4	5012906.7	360993.0
BH 21A-2	91.4	5012921.3	360986.9
BH 21A-3	91.5	5012921.9	360977.3
BH 21A-4	91.8	5012960.4	360970.8
BH 21A-5	92.0	5012982.4	360961.6
BH 21A-6	92.3	5012991.1	360954.5
BH 21A-7	92.5	5013013.8	360948.5
BH 21A-8	91.0	5012857.6	361008.8
BH 21A-9	91.3	5012885.2	360997.4
BH 21A-10	94.4	5013029.3	360937.4
BH 21A-11	96.7	5013053.6	360917.6
BH 21B-1	91.3	5012880.6	360962.7
BH 21B-2	91.4	5012895.3	360956.6
BH 21B-4	91.8	5012934.2	360940.4
BH 21B-5	92.1	5012956.3	360931.2
BH 21B-6	92.3	5012973.1	360924.2
BH 21B-7	92.5	5012987.7	360918.1
BH 21B-8	91.3	5012837.4	360985.3
BH 21B-9	91.3	5012865.1	360973.7
BH 21B-10	95.1	5013013.6	360911.6
BH 21B-11	96.4	5013030.6	360899.6
BH 31	95.0	5013029.1	360905.5
BH 32	98.8	5013052.9	360895.7
BH 89-7	91.3	5012686.7	361047.9
BH 89-7A	91.3	5012640.1	361067.1
BH 89-8	91.3	5012744.1	361056.0
BH 89-8A	91.3	5012716.6	361067.1
BH 89-9	91.2	5012779.0	361009.5
BH 89-9A	91.2	5012705.6	361040.1
BH 89-10	91.3	5012836.5	361017.6

#### REFERENCE

Base plans provided in digital format by WSP, drawing file no. 3415003-180-001\_STRANDHERD\_GA.dwg, received JULY 26, 2018.

NO.	DATE	BY	REVISION
1	10/3/2018	FJH	1
Geocres No. 3165-299			
HWY. 416		PROJECT NO. 1417217	
SUBM'D. MJK		DATE: 10/3/2018	
DRAWN: JM		SITE: 3-549	
CHKD. MJK		APPD. FJH	
DWG. 1		DIST. EASTERN	



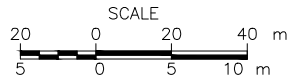


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#### PROFILE B-B' ALONG HWY 416 SBL



CONT No.

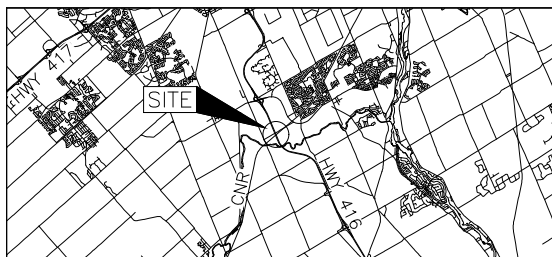
WP No. 4133-01-01 &  
4134-01-01

HIGHWAY 416 OVERPASS  
BRIDGES AT STRANDHERD DRIVE  
AND CNR

BOREHOLE LOCATIONS AND SOIL STRATA



SHEET



KEY PLAN  
SCALE  
4 0 4 6 km

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- ⊕ Dynamic Cone Penetration Test - Current Investigation
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SCPT 18-03	93.5	5013014.7	360932.7
SCPT 18-04	93.4	5012990.4	360937.4
SCPT 18-06	92.9	5012941.5	360949.5
SCPT 18-07	92.3	5012909.3	360963.5
SCPT 18-09	92.4	5012852.4	361000.9
SCPT 18-10	91.9	5012774.1	361022.1
BH 15	97.0	5013047.0	360930.0
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BH 21A-3	91.5	5012921.9	360977.3
BH 21A-4	91.8	5012960.4	360970.8
BH 21A-5	92.0	5012982.4	360961.6
BH 21A-6	92.3	5012999.1	360954.5
BH 21A-7	92.5	5013013.8	360948.5
BH 21A-8	91.0	5012857.6	361008.8
BH 21A-9	91.3	5012885.2	360997.4
BH 21A-10	94.4	5013029.3	360937.4
BH 21A-11	96.7	5013052.6	360917.6
BH 21B-1	91.3	5012880.6	360962.7
BH 21B-2	91.4	5012895.3	360956.6
BH 21B-4	91.8	5012934.2	360940.4
BH 21B-5	92.1	5012956.3	360931.2
BH 21B-6	92.3	5012973.1	360924.2
BH 21B-7	92.5	5012987.7	360918.1
BH 21B-8	91.3	5012837.4	360985.3
BH 21B-9	91.3	5012865.1	360973.7
BH 21B-10	95.1	5013013.6	360911.6
BH 21B-11	96.4	5013030.6	360899.6
BH 31	95.0	5013029.1	360905.5
BH 32	98.8	5013052.9	360895.7
BH 89-7	91.3	5012686.7	361047.9
BH 89-7A	91.3	5012640.1	361067.1
BH 89-8	91.3	5012744.1	361056.0
BH 89-8A	91.3	5012716.6	361067.1
BH 89-9	91.2	5012779.0	361009.5
BH 89-9A	91.2	5012705.6	361040.1
BH 89-10	91.3	5012836.5	361017.6

#### REFERENCE

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NO.	DATE	BY	REVISION
1	2018-07-26	J.M.	Initial Issue
2	2018-08-01	F.J.H.	Revised Borehole Locations
3	2018-08-01	M.J.K.	Final Design Configuration

Geocres No. 3165-299

HWY. 416	PROJECT NO. 1417217	DIST. EASTERN
SUBM'D. MJK	CHKD. MJK	DATE: 10/3/2018
DRAWN: JM	CHKD. FJH	APPD. FJH
		SITE: 3-549
		DWG. 2

**APPENDIX A**

**Borehole and Drillhole Records  
(Golder, 2018)**





## LIST OF SYMBOLS

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

### I. GENERAL

$\pi$	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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

<b>(a)</b>	<b>Index Properties</b>
$\rho(\gamma)$	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
$\gamma'$	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

### (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)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_{\alpha}$	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
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	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

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

Notes: 1  
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



## 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, 60° 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<sup>2</sup> 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

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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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 <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	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 With (cohesive)	Sand and Gravel 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	

<b>PROJECT</b> 1417217-1231		<b>RECORD OF BOREHOLE No 18-01</b>		SHEET 1 OF 4		<b>METRIC</b>	
<b>G.W.P.</b> 4133-01-01 & 4134-01-01		<b>LOCATION</b> N 5013055.7; E 360897.3 NAD MTM ZONE 9 (LAT. 45.255220; LONG. -75.785300)		<b>ORIGINATED BY</b> RI			
<b>DIST</b> Eastern HWY 416		<b>BOREHOLE TYPE</b> Power Auger, 200 mm Diam. (Hollow Stem)/Wash boring, NW Casing/Rotary Drill, HOD		<b>COMPILED BY</b> ZS			
<b>DATUM</b> CGVD28		<b>DATE</b> April 30, 2018		<b>CHECKED BY</b> WAM			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × REMOULDED					W <sub>P</sub>	W	W <sub>L</sub>		GR	SA	SI	CL
103.5	GROUND SURFACE							20	40	60	80	100								
0.0	ASPHALTIC CONCRETE																			
0.1	PORTLAND CEMENT CONCRETE																			
103.1	(SP/GP) Gravelly sand to sandy gravel, angular (FILL) Compact to loose Grey to dark grey Moist to wet		1	GRAB	-															
0.4																				
			2	SS	16															
			3	SS	22									○				47 42 7 7		
			4	SS	9															
			5	SS	12									○						
			6	SS	12															
98.2	(SM/ML) Gravelly Silty SAND to Sandy SILT, trace to some clay, contains cobbles (TILL) Compact to very dense Grey brown Moist		7	SS	18															
5.3																				
					8	SS	33									○			22 33 37 8	
					9	SS	74									○				
			10	SS	50/0.15															
			11	SS	50/0.06															
			12	RC	DD															
			13	SS	63									○			26 27 42 5			

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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



PROJECT		1417217-1231		RECORD OF BOREHOLE No 18-01		SHEET 3 OF 4		METRIC								
G.W.P.		4133-01-01 & 4134-01-01		LOCATION		N 5013055.7; E 360897.3 NAD MTM ZONE 9 (LAT. 45.255220; LONG. -75.785300)		ORIGINATED BY								
DIST		Eastern HWY 416		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)/Wash boring, NW Casing/Rotary Drill, HOD		COMPILED BY								
DATUM		CGVD28		DATE		April 30, 2018		CHECKED BY								
								WAM								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
82.7	(SM) Gravelly Silty SAND, contains cobbles and boulders (TILL) Dense to compact Grey Wet															
20.8	Possible Bedrock															
82.4																
21.1	Sandstone (BEDROCK)															
	Bedrock cored from depths 21.1 m to 26.3 m		1	RC	REC 100%											RQD = 100%
	For bedrock coring detail refer to Record of Drillhole 18-01															
			2	RC	REC 100%											RQD = 95%
			3	RC	REC 100%											RQD = 95%
79.0																
24.6	Dolostone (BEDROCK)															
			4	RC	REC 96%											RQD = 84%
			5	RC	REC 100%											RQD = 100%
77.2																
26.3	END OF BOREHOLE															

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SHEET 4 OF 4

DATUM: CGVD28

DRILLING CONTRACTOR: CCC Drilling

[illegible]

LOGGED: RI  
CHECKED: MJK

1 : 50



+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

PROJECT 1417217-1231		<b>RECORD OF BOREHOLE No 18-02</b>		SHEET 2 OF 4		<b>METRIC</b>								
G.W.P. 4133-01-01 & 4134-01-01		LOCATION N 5012694.5; E 361054.6 NAD MTM ZONE 9 (LAT. 45.251960; LONG. -75.783300)		ORIGINATED BY DG										
DIST Eastern HWY 416		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Wash boring, NW Casing/Rotary Drill, HCCM		COMPILED BY ZS										
DATUM CGVD28		DATE May 14, 2018		CHECKED BY WAM										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa		W <sub>p</sub>	W			W <sub>L</sub>
85.6 10.1	(GP) Sand GRAVEL, some silt, contains cobbles and boulders (TILL) Very dense Grey Wet	[Pattern]					20 40 60 80 100	20 40 60 80 100	25 50 75					
			12	SS	99									
			13	SS	87									
			14	RC	DD									
80.9 14.8	Dolostone (BEDROCK)	[Pattern]												
	Bedrock cored from depths 14.8 m to 20.2 m		1	RC	REC 88%									RQD = 59%
	For bedrock coring detail refer to Record of Drillhole 18-02													
			2	RC	REC 100%									RQD = 96%
			3	RC	REC 100%									RQD = 97%
			4	RC	REC 100%									RQD = 100%

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

SHEET 4 OF 4

DATUM: CGVD28

DRILLING CONTRACTOR: CCC Drilling

CHECKED: WAM

PROJECT		1417217-1231		RECORD OF BOREHOLE No 18-05				SHEET 1 OF 2		METRIC								
G.W.P.		4133-01-01 & 4134-01-01		LOCATION		N 5012966.7; E 360946.9 NAD MTM ZONE 9 (LAT. 45.254420; LONG. -75.784700)				ORIGINATED BY		RI						
DIST		Eastern HWY 416		BOREHOLE TYPE		Rotary Drill/Wash Boring, NQ Casing				COMPILED BY		ZS						
DATUM		CGVD28		DATE		May 16, 2018				CHECKED BY		WAM						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED								PLASTIC LIMIT w <sub>p</sub> NATURAL MOISTURE CONTENT w LIQUID LIMIT w <sub>L</sub> WATER CONTENT (%)		
93.1	GROUND SURFACE																	
0.0	(SM) Gravelly silty sand (TOPSOIL)																	
0.2	Brown Moist		1	SS	24													
92.3	(SM) Gravelly silty sand, contains cobbles and boulders (FILL)																	
0.8	Compact Brown Moist																	
92.0	(SP) Sand, trace silt and gravel (FILL)		2	SS	8													
1.2	Loose Brown Wet																	
	(CL) Silty clay, trace sand (TOPSOIL)		3	SS	3													
	Black Moist																	
	(CI/CH) SILTY CLAY to CLAY, trace sand, highly fissured, contains silty sand seams (WEATHERED CRUST) Very stiff to stiff Grey-brown Moist																	
90.1	(CI/CH) SILTY CLAY to CLAY, contains silty sand seams Very soft to stiff Grey		4	SS	WH													
3.1																		
			5	SS	WR													
87.6	(SP/GP) SAND and GRAVEL, some silt, contains cobbles (TILL) Compact to loose Grey Wet		6	SS	12													
5.5																		
			7	SS	17													
			8	SS	16													
			9	SS	8													
84.9	(SM) Gravelly Silty SAND, trace to some clay, contains cobbles (TILL) Compact to loose Grey Wet		10	SS	16													
8.2																		
			11	SS	8													

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE



PROJECT		1417217-1231		RECORD OF BOREHOLE No 18-05		SHEET 2 OF 2		METRIC									
G.W.P.		4133-01-01 & 4134-01-01		LOCATION		N 5012966.7; E 360946.9 NAD MTM ZONE 9 (LAT. 45.254420; LONG. -75.784700)		ORIGINATED BY		RI							
DIST		Eastern HWY 416		BOREHOLE TYPE		Rotary Drill/Wash Boring, NQ Casing		COMPILED BY		ZS							
DATUM		CGVD28		DATE		May 16, 2018		CHECKED BY		WAM							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
82.6			12	SS	14		83										
10.5	END OF BOREHOLE		13	SS	50/0.06												

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PROJECT 1417217-1231			RECORD OF BOREHOLE No 18-08			SHEET 1 OF 2			METRIC								
G.W.P. 4133-01-01 & 4134-01-01			LOCATION N 5012883.4; E 360982.3 NAD MTM ZONE 9 (LAT. 45.253670; LONG. -75.784200)			ORIGINATED BY RI											
DIST Eastern HWY 416			BOREHOLE TYPE Rotary Drill/Wash Boring, NQ Casing			COMPILED BY ZS											
DATUM CGVD28			DATE May 16, 2018			CHECKED BY WAM											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
92.4	GROUND SURFACE							20	40	60	80	100					
0.0	(SM) Silty sand, trace gravel (TOPSOIL)																
0.2	Brown Moist		1	SS	24												
	(SP/SM) SAND, trace to some gravel and silt (FILL) Compact to loose Dark brown Moist to wet																
91.3			2	SS	9												8 78 11 8
1.1	(CL/ML) Silty clay to clayey silt (TOPSOIL)																
91.0	Dark brown Moist																
1.4	(CI/CH) SILTY CLAY to CLAY, trace sand, highly fissured, contains silty sand seams (WEATHERED CRUST); Very stiff to stiff Grey-brown Moist		3	SS	2												
89.4	(CI/CH) SILTY CLAY to CLAY, trace sand Firm to soft Grey Wet		4	SS	WH												
3.1																	
88.8	(CI/CH) SILTY CLAY to CLAY, trace sand, contains silty sand seams Firm to stiff Grey Wet																
5.6																	
86.8			5	SS	PM												0 0 41 59
86.8																	
85.6																	
84.8																	
84.0																	
83.3	(ML) Sandy SILT, some clay, trace to some gravel, contains silty clay seams (TILL) Very loose Grey Wet		8	SS	1												
9.2																	

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\SPATIAL\_IMMMM\_GROUP\MEGA6\_VARIOUSSTRUCTURES02\_DATAGINTY1417217.GPJ GAL-GTA.GDT 28/3/19

PROJECT <u>1417217-1231</u>		<b>RECORD OF BOREHOLE No 18-08</b>				SHEET 2 OF 2		<b>METRIC</b>	
G.W.P. <u>4133-01-01 &amp; 4134-01-01</u>		LOCATION <u>N 5012883.4; E 360982.3 NAD MTM ZONE 9 (LAT. 45.253670; LONG. -75.784200)</u>				ORIGINATED BY <u>RI</u>			
DIST <u>Eastern</u> HWY <u>416</u>		BOREHOLE TYPE <u>Rotary Drill/Wash Boring, NQ Casing</u>				COMPILED BY <u>ZS</u>			
DATUM <u>CGVD28</u>		DATE <u>May 16, 2018</u>				CHECKED BY <u>WAM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
	--- CONTINUED FROM PREVIOUS PAGE ---																
	(ML) Sandy SILT, some clay, trace to some gravel, contains silty clay seams (TILL) Very loose Grey Wet		9	SS	3												
			10	SS	3												
80.2																	
12.2	END OF BOREHOLE		11	SS	50/0.00												

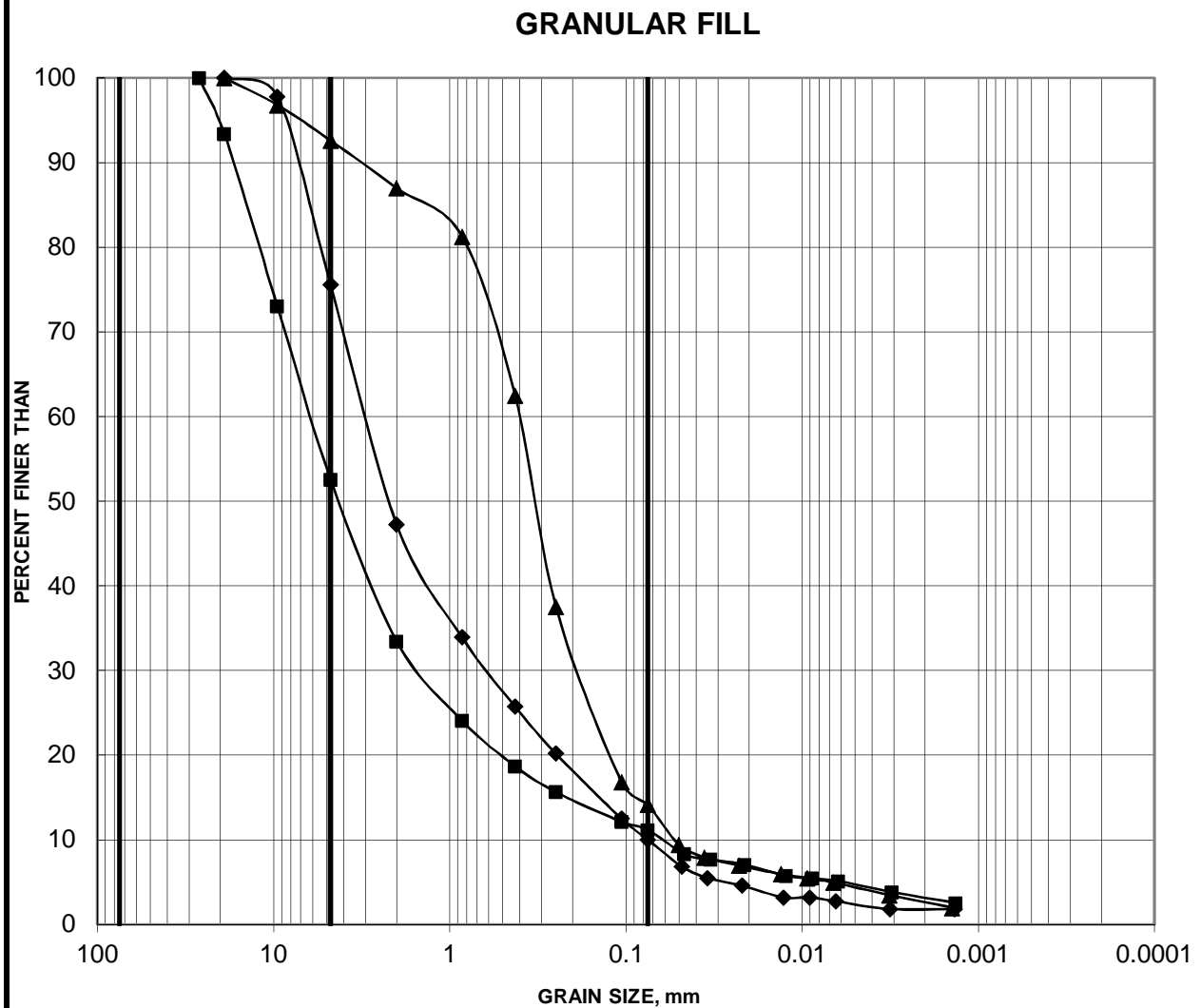
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**APPENDIX B**

**Laboratory Test Results  
(Golder, 2018)**

# GRAIN SIZE DISTRIBUTION

FIGURE B1

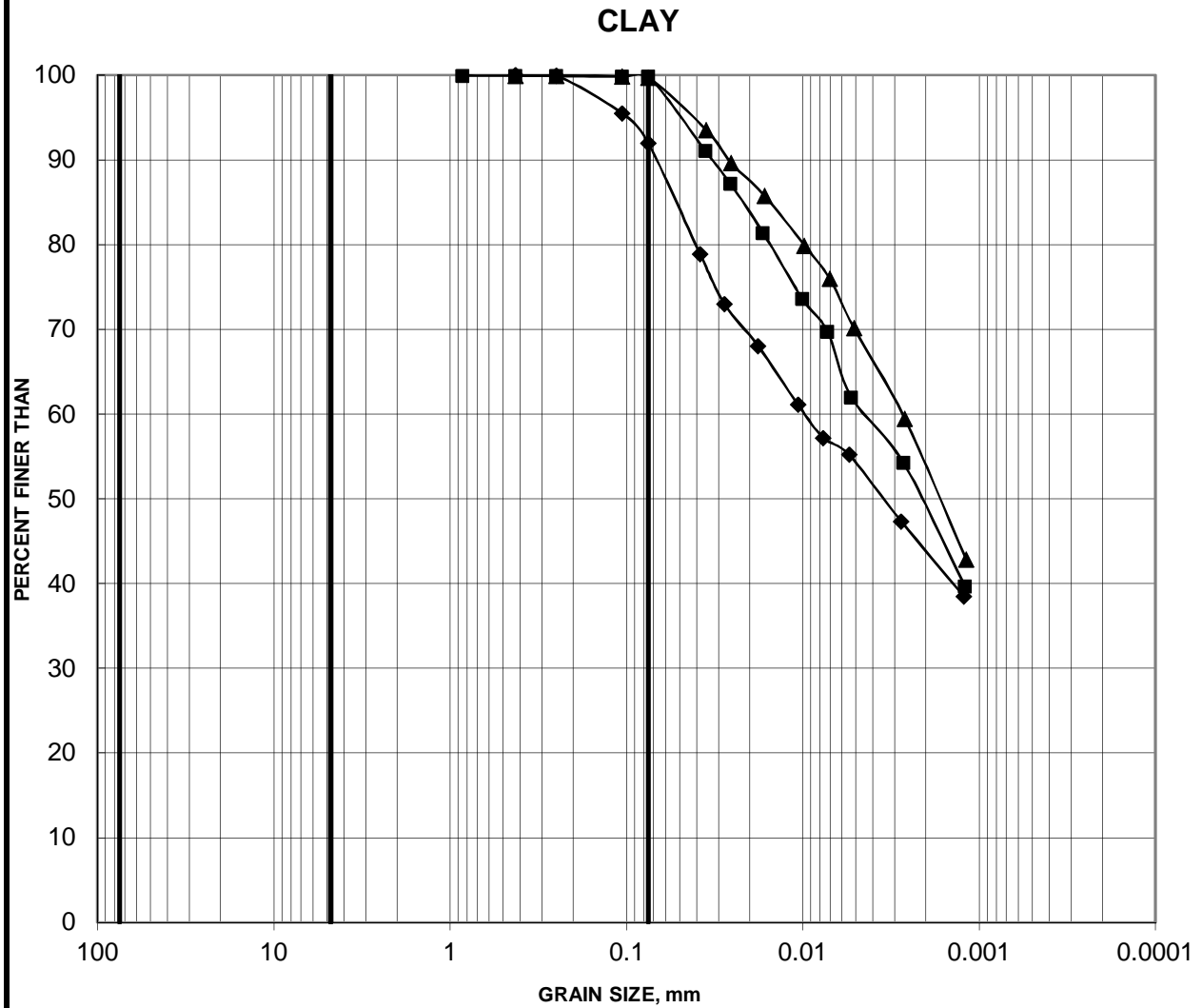


Cobble Size	coarse	fine	coarse	medium	fine	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
18-01	3	2.29-2.90
18-02	4	2.29-2.90
18-08	2A	0.76-1.07

# GRAIN SIZE DISTRIBUTION

FIGURE B2

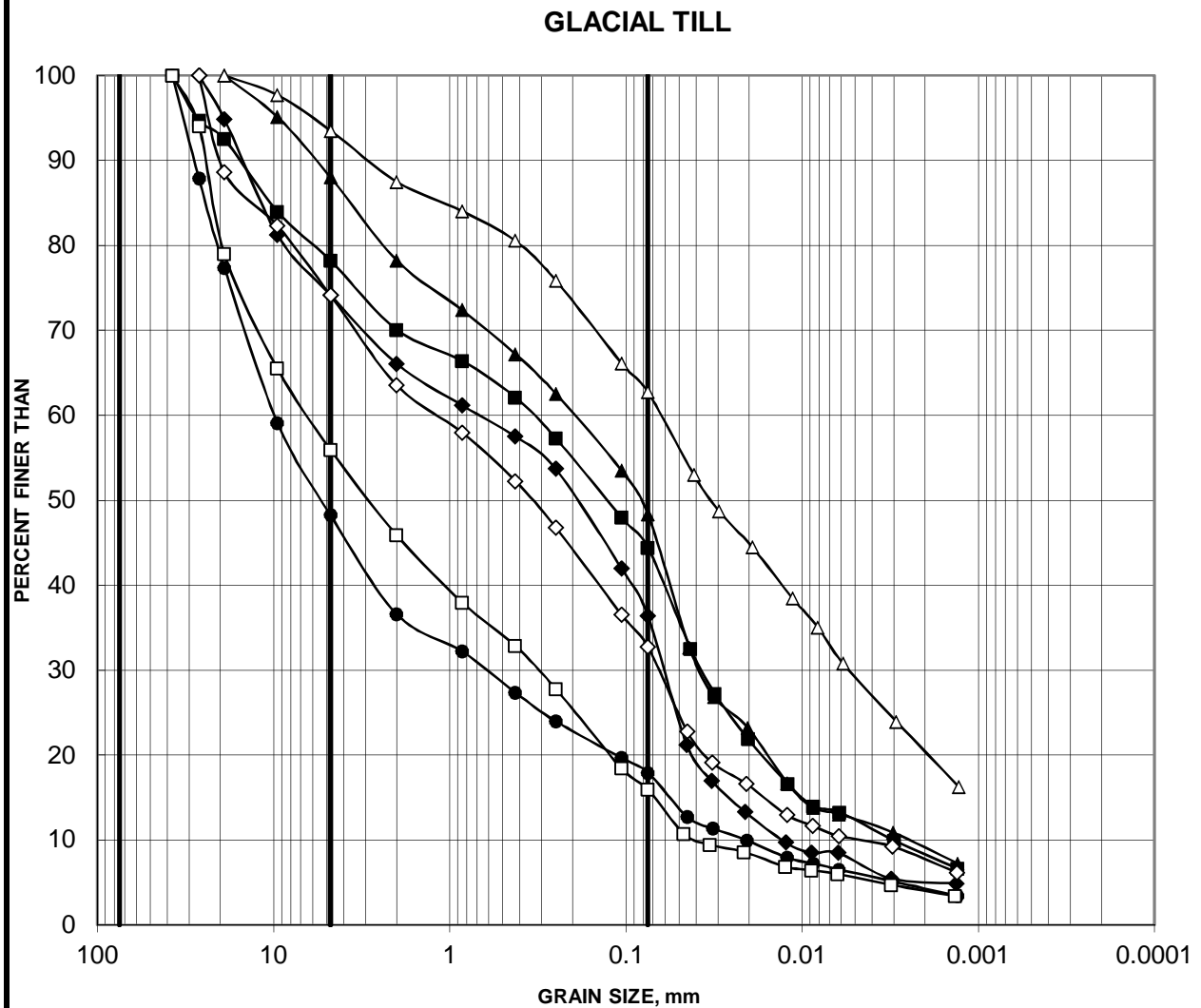


Cobble Size	coarse	fine	coarse	medium	fine	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
18-02	10	7.62-8.23
18-05	3	1.52-2.13
18-08	5	4.57-5.18

# GRAIN SIZE DISTRIBUTION

FIGURE B3



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
■ 18-01	8	6.10-6.71
◆ 18-01	13	9.54-10.15
▲ 18-01	17	12.95-13.56
● 18-02	13	12.19-12.80
□ 18-05	7	6.10-6.71
◇ 18-05	11	9.14-9.75
△ 18-08	9	9.91-10.52



**Golder Associates Ltd.**  
1931 Robertson Road  
Ottawa, Ontario  
K2H 5B7





## **UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE**

**Project:** MMM 4014 E 0015 Mega 6 Eastern Region  
- Strandherd Bridge

**Project No.:** 1417217/1231

**Date:** June 21, 2018


**Location(s):** See below

Bore Hole No.	Depth (m)	Date Tested	Core Size	Diameter (mm)	Density (kg/m <sup>3</sup> )	Compressive Strength (MPa)	Failure Mode
18-01	21.35-21.49	Jun 20/18	HQ	60.7	2538	127.1	
18-02	15.31-15.45	Jun 20/18	HQ	60.6	2736	186.8	

REMARKS : - Cores tested in vertical direction.  
- Cores tested in air-dry condition.  
- Specimen ends prepared with high-strength plaster, but un-restrained.  
- L/D ratio's between 2.0:1 and 2.5:1  
- Time to failure > 2 and < 15 minutes.

TESTING WAS CARRIED OUT IN GENERAL ACCORDANCE WITH ASTM D7012 - Method C

SIGNED:

  
C.N. Mangione P.Eng.

**APPENDIX C**

**Cone Penetration Testing Report  
(ConeTec Investigations Ltd., 2018)**

# PRESENTATION OF SITE INVESTIGATION RESULTS

## Hwy 416 and McKenna Casey Dr

*Prepared for:*

Golder Associates

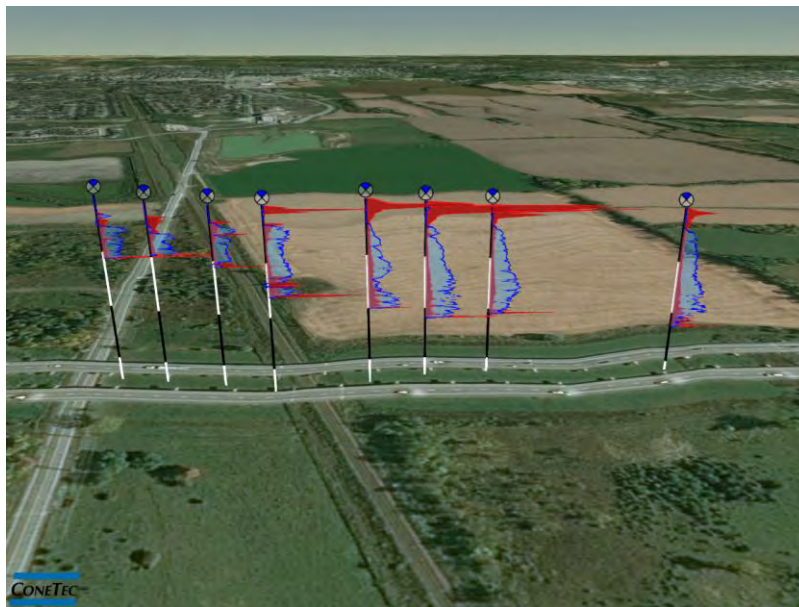
ConeTec Job No: 18-05030

Project Start Date: 14-May-2018

Project End Date: 16-May-2018

Report Date: 29-May-2018

Revision Date: 15-Jun-2018



*Prepared by:*

ConeTec Investigations Ltd.  
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[www.conetec.com](http://www.conetec.com)

[www.conetecdataservices.com](http://www.conetecdataservices.com)



## Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates near Hwy 416 and McKenna Casey Dr, Ontario. The program consisted eight seismic cone penetration tests (SCPT).

## Project Information

Project	
Client	Golder Associates
Project	Hwy 416 and McKenna Casey Dr
ConeTec project number	18-05030

An aerial overview from Google Earth including the CPT locations is presented below.



Rig Description	Deployment System	Test Type
Portable Track Rig	Portable	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.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Expanded range plots, advanced CPT plots with $I_c$ , $S_u$ and N160 (IC RW1998), Soil Behavior Type (SBT) scatter plots as well as seismic Vs plots have been included in the data release package.
Additional Comment	The dynamic pore pressure response at SCPT18-07 was subdued due to saturation loss.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm <sup>2</sup> )	Sleeve Area (cm <sup>2</sup> )	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
330:T1500F15U500	330	15	225	1500	15	500
Cone 330 was used for all CPT soundings.						

Calculated Geotechnical Parameter Tables	
Additional information	<p>The Normalized Soil Behaviour Type Chart based on <math>Q_{tn}</math> (SBT <math>Q_{tn}</math>) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip resistance (<math>q_t</math>) sleeve friction (<math>f_s</math>) and pore pressure (<math>u_2</math>). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile.</p> <p>Soils were classified as either drained or undrained based on the <math>Q_{tn}</math> Normalized Soil Behaviour Type Chart (Robertson, 2009). Both drained and undrained parameters were calculated for soils that classified as Sand Mixtures (zone 5).</p>

## Limitations

This report has been prepared for the exclusive use of Golder Associates (Client) for the project titled "Hwy 416 and McKenna Casey Dr". 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<sup>2</sup> and 15 cm<sup>2</sup> 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<sup>2</sup> penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm<sup>2</sup> 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<sub>2</sub>" 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.





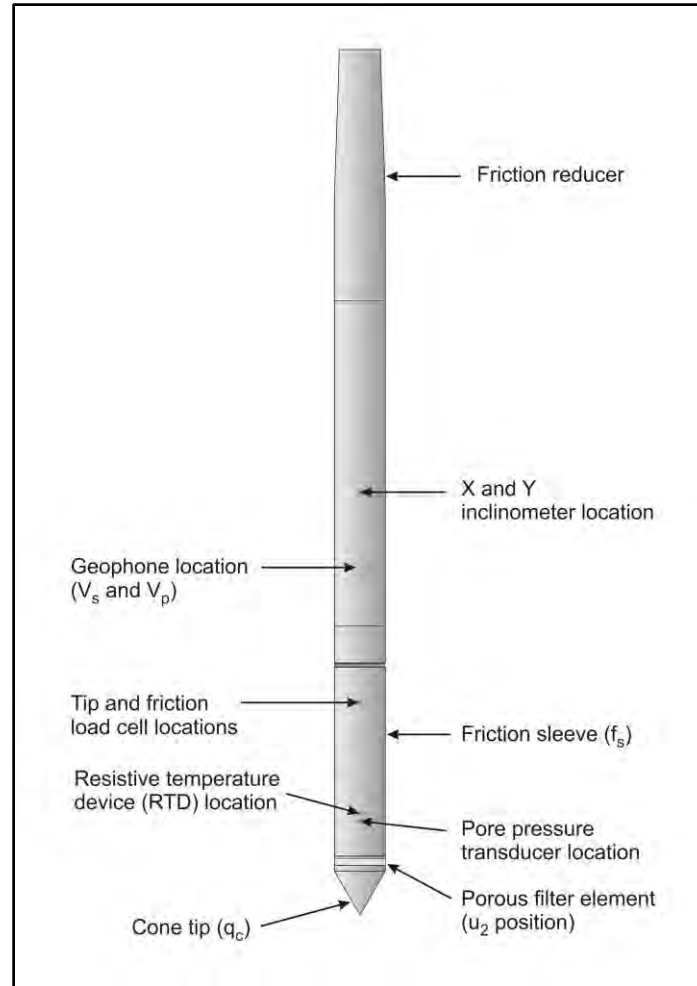


Figure CPTu. Piezocone Penetrometer (15 cm<sup>2</sup>)

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:

$$q_t = q_c + (1-a) \cdot u_2$$

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 ( $R_f$ ) 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 ( $V_p$ ) 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.

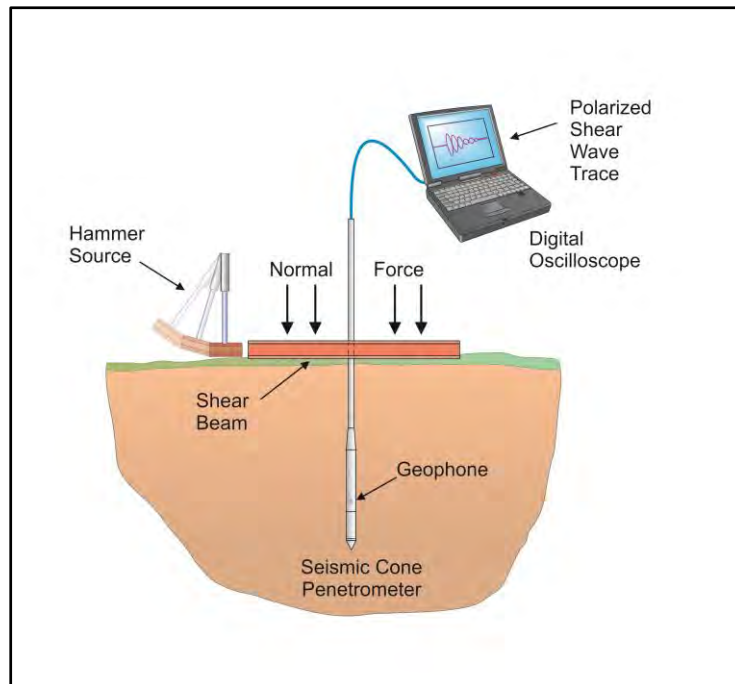


Figure SCPTu-1. Illustration of the SCPTu system

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).

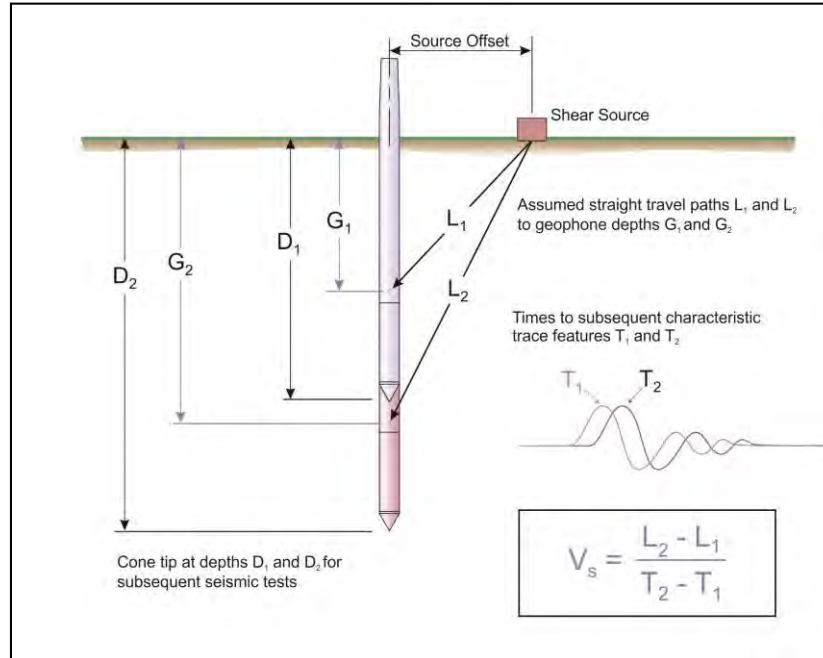


Figure SCPTu-2. Illustration of a seismic cone penetration test

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.

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

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 ( $t$ ).

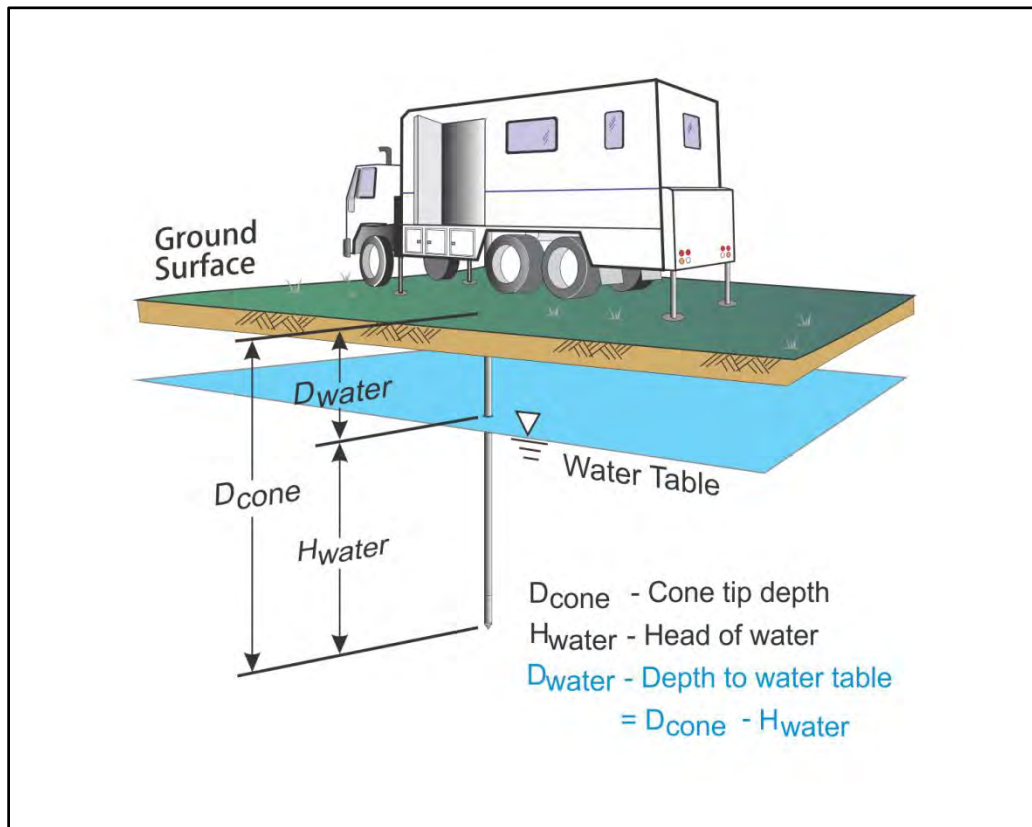


Figure PPD-1. Pore pressure dissipation test setup

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.

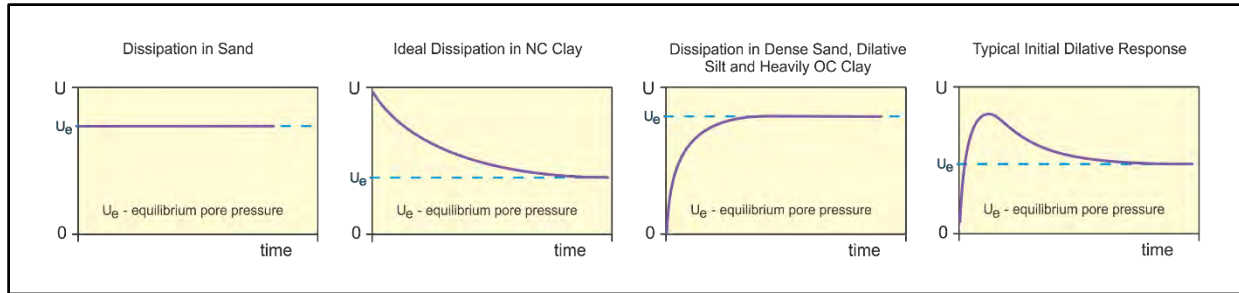


Figure PPD-2. Pore pressure dissipation curve examples

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.

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## REFERENCES

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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
- Advanced Cone Penetration Test Plots
- Soil Behavior Type (SBT) Scatter Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Wave Traces
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

## Cone Penetration Test Summary and Standard Cone Penetration Test Plots





Job No: 18-05030  
Client: Golder Associates  
Project: Hwy 416 and McKenna Casey Dr  
Start Date: 14-May-2018  
End Date: 16-May-2018

### ***CONE PENETRATION TEST SUMMARY***

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface <sup>1</sup> (m)	Final Depth (m)	Northing <sup>2</sup> (m)	Easting (m)	Refer to Notation Number
SCPT18-03	18-05030_SP03	16-May-2018	330:T1500F15U500	1.6	5.550	5011559	438416	
SCPT18-04	18-05030_SP04	16-May-2018	330:T1500F15U500	1.6	4.775	5011539	438422	
SCPT18-05	18-05030_SP05	16-May-2018	330:T1500F15U500	1.6	5.550	5011513	438429	
SCPT18-06	18-05030_SP06	15-May-2018	330:T1500F15U500	1.7	8.100	5011489	438432	
SCPT18-07	18-05030_SP07	14-May-2018	330:T1500F15U500	1.8	9.975	5011450	438451	
SCPT18-08	18-05030_SP08	15-May-2018	330:T1500F15U500	1.9	10.925	5011428	438467	
SCPT18-09	18-05030_SP09	15-May-2018	330:T1500F15U500	1.9	10.500	5011401	438482	
SCPT18-10	18-05030_SP10	15-May-2018	330:T1500F15U500	1.8	11.375	5011318	438505	

1. The assumed phreatic surface was based on dynamic pore pressure response. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were acquired using consumer grade GPS equipment in datum WGS1984/UTM Zone 18 North.



Golder

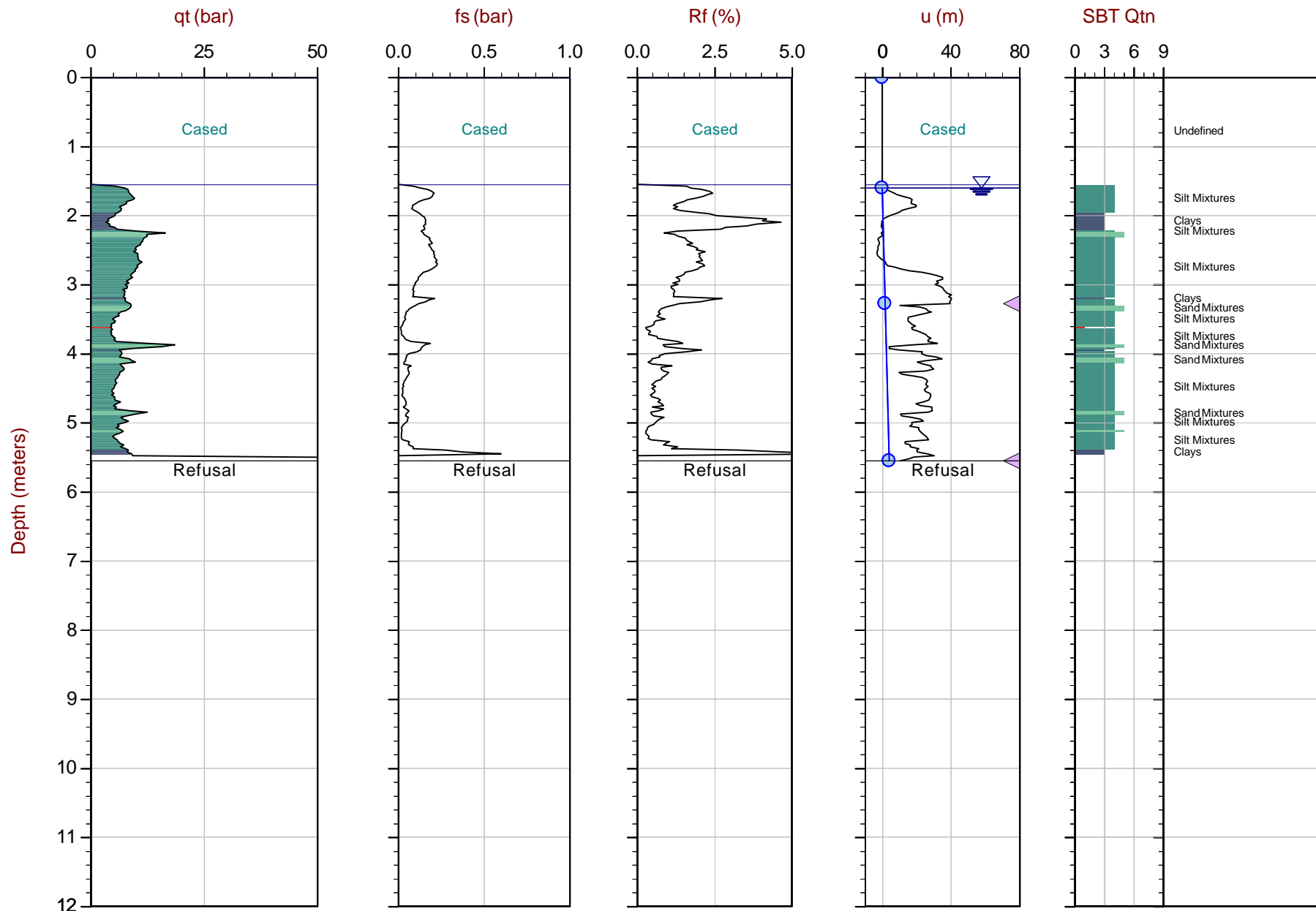
Job No: 18-05030

Date: 2018-05-16 11:08

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-03

Cone: 330:T1500F15U500



Max Depth: 5.550 m / 18.21 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP03.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N N: 5011559m E: 438416m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▼ Dissipation, Ueq not achieved    — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

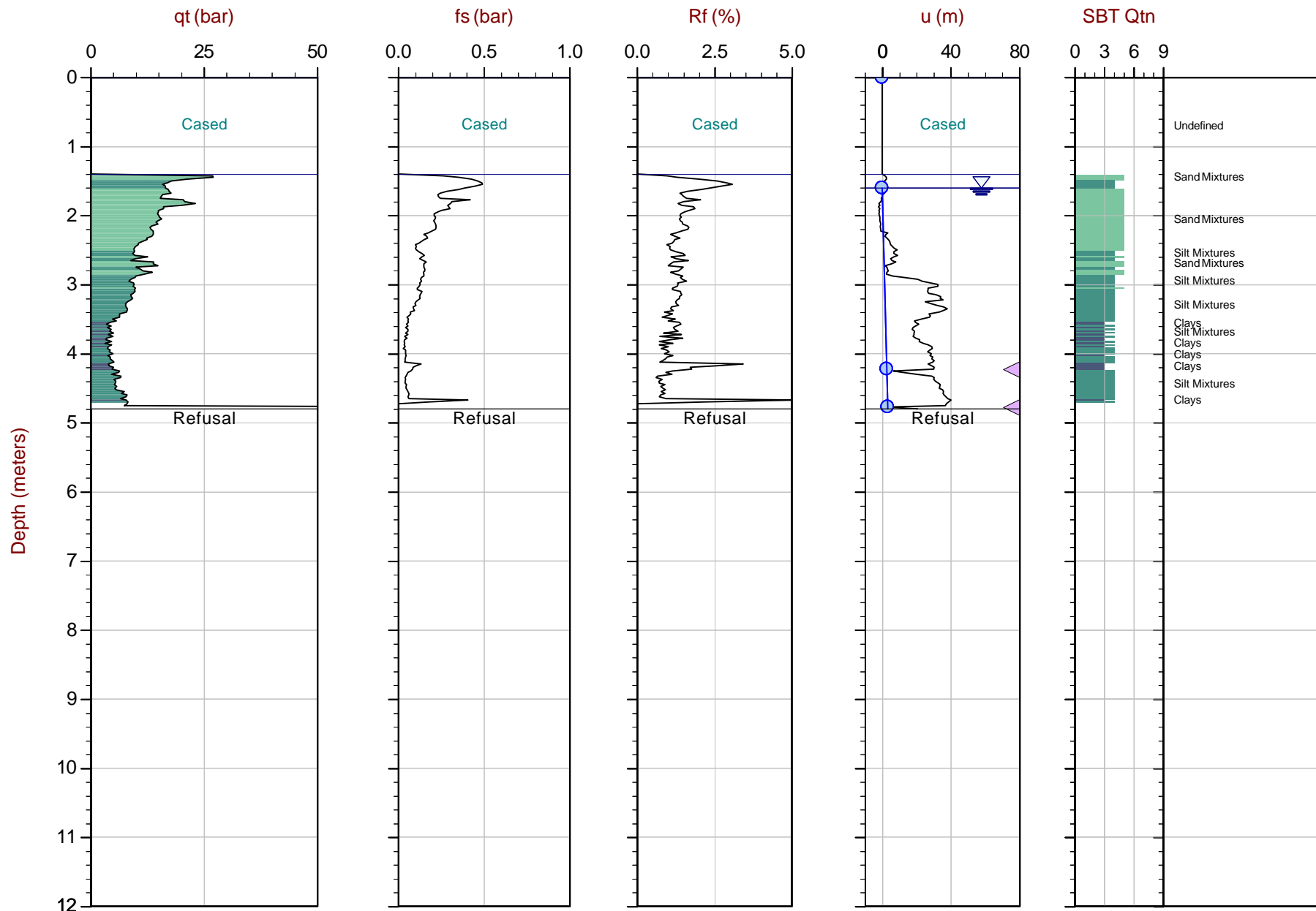
Job No: 18-05030

Date: 2018-05-16 09:32

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-04

Cone: 330:T1500F15U500



Max Depth: 4.800 m / 15.75 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP04.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N N: 5011539m E: 438422m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▼ Dissipation, Ueq not achieved    — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.





Golder

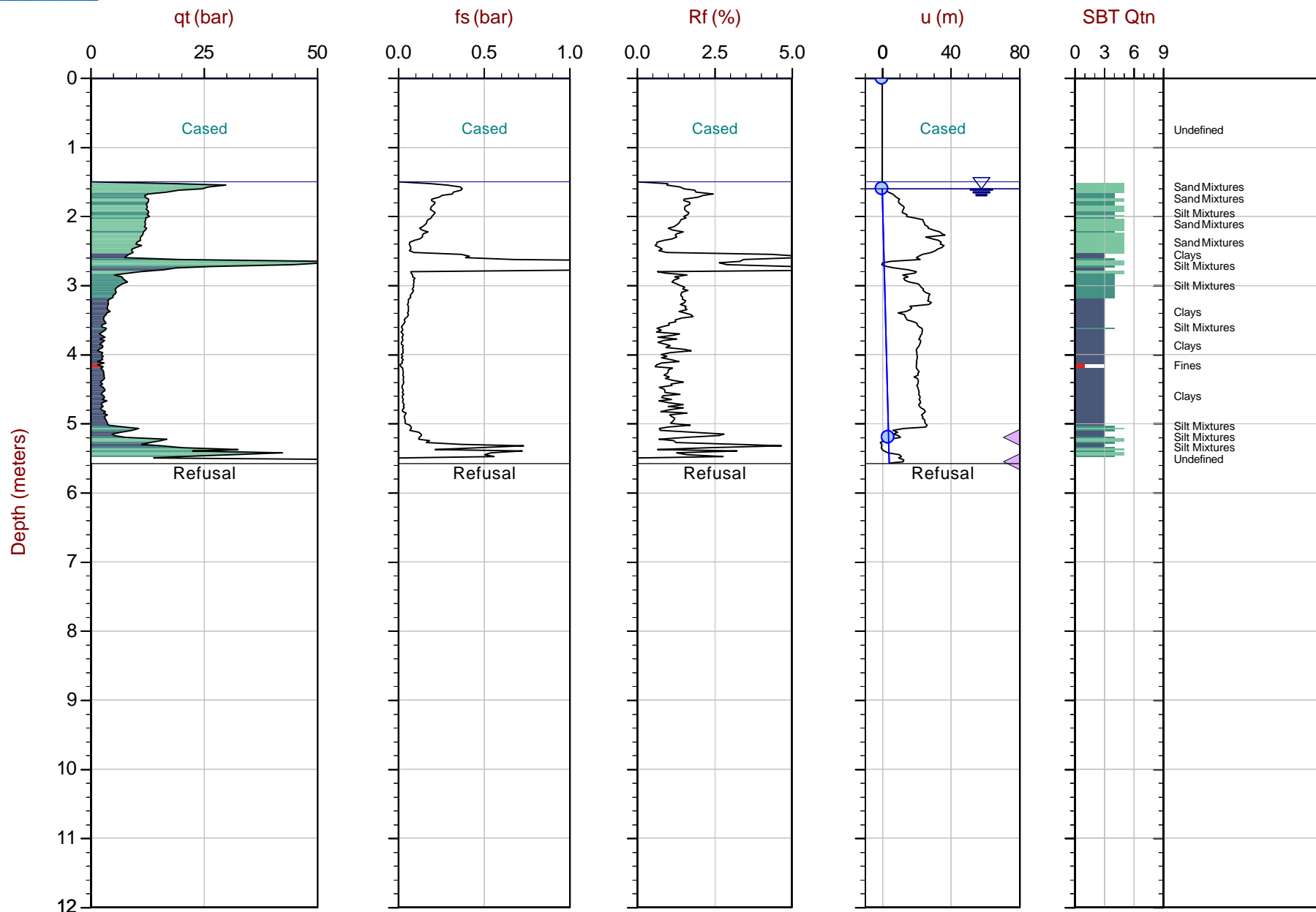
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Date: 2018-05-16 08:22

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-05

Cone: 330:T1500F15U500



Max Depth: 5.575 m / 18.29 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP05.COR

Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N N: 5011513m E: 438429m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▼ Dissipation, Ueq not achieved    — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

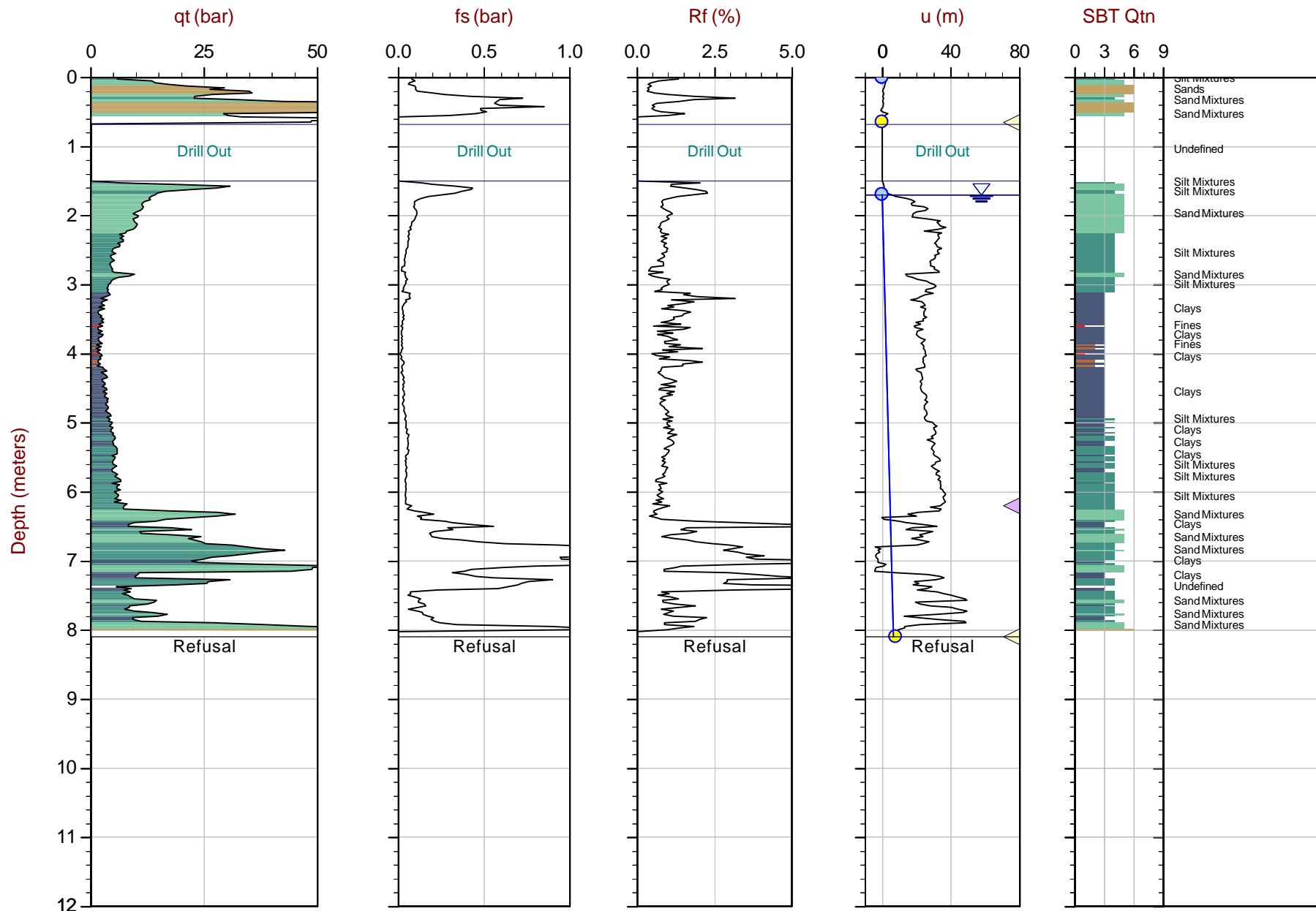
Job No: 18-05030

Date: 2018-05-15 14:58

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-06

Cone: 330:T1500F15U500



Max Depth: 8.100 m / 26.57 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP06.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N:5011489m E:438432m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

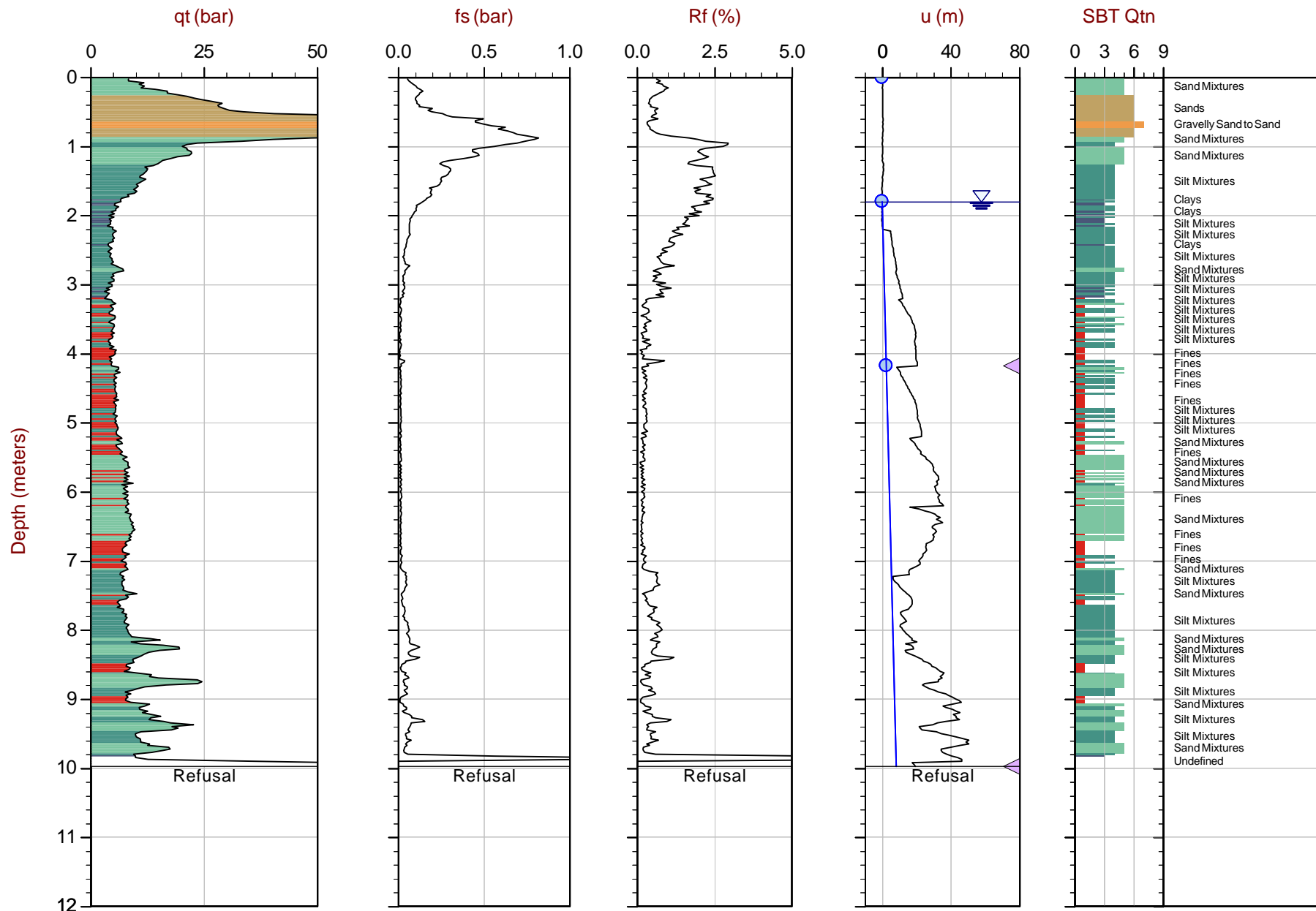
Job No: 18-05030

Date: 2018-05-14 14:54

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-07

Cone: 330:T1500F15U500



Max Depth: 9.975 m / 32.73 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP07.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N: 5011450mE: 438451m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

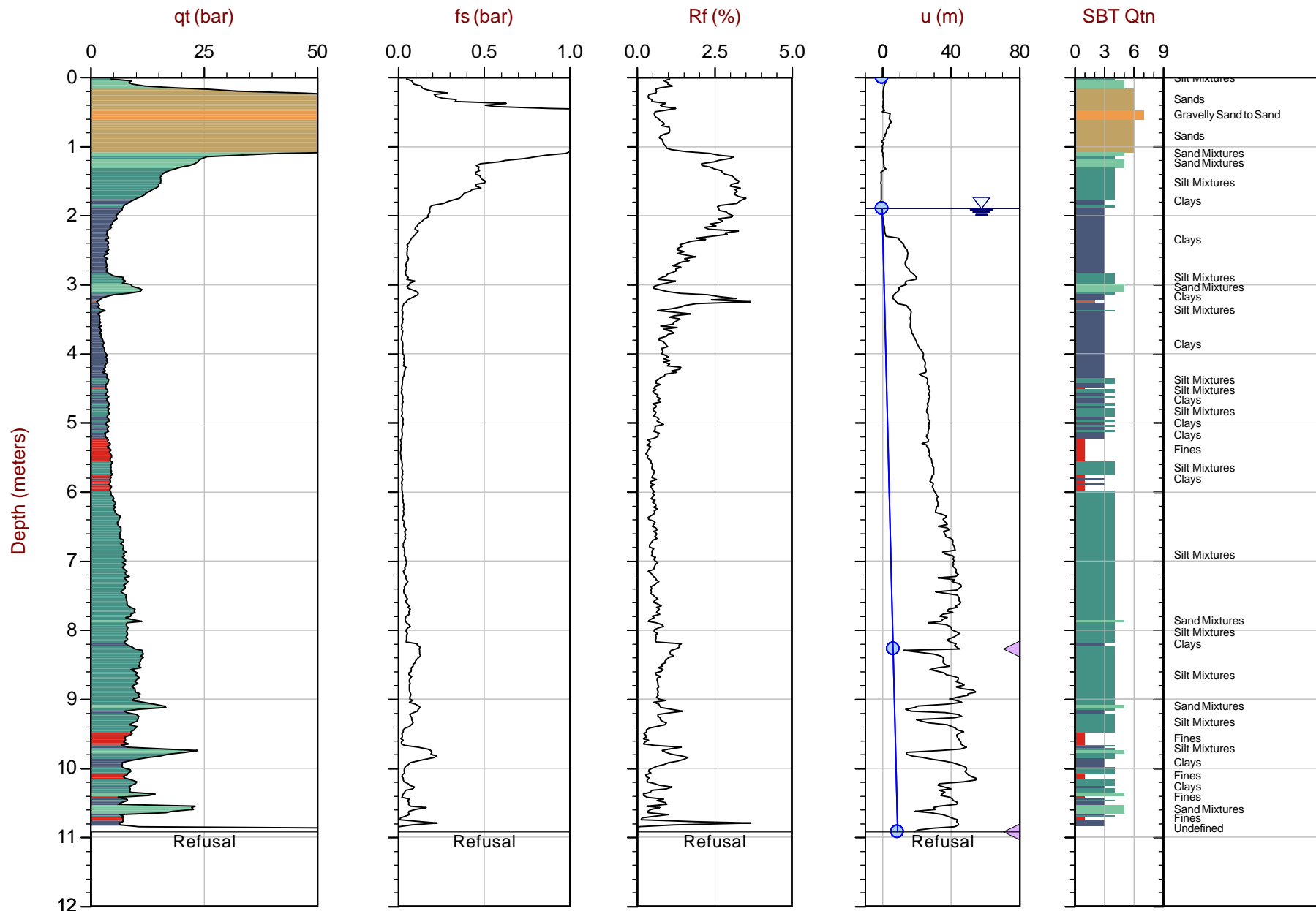
Job No: 18-05030

Date: 2018-05-15 10:18

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-08

Cone: 330:T1500F15U500



Max Depth: 10.925 m / 35.84 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP08.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N:5011428mE:438467m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

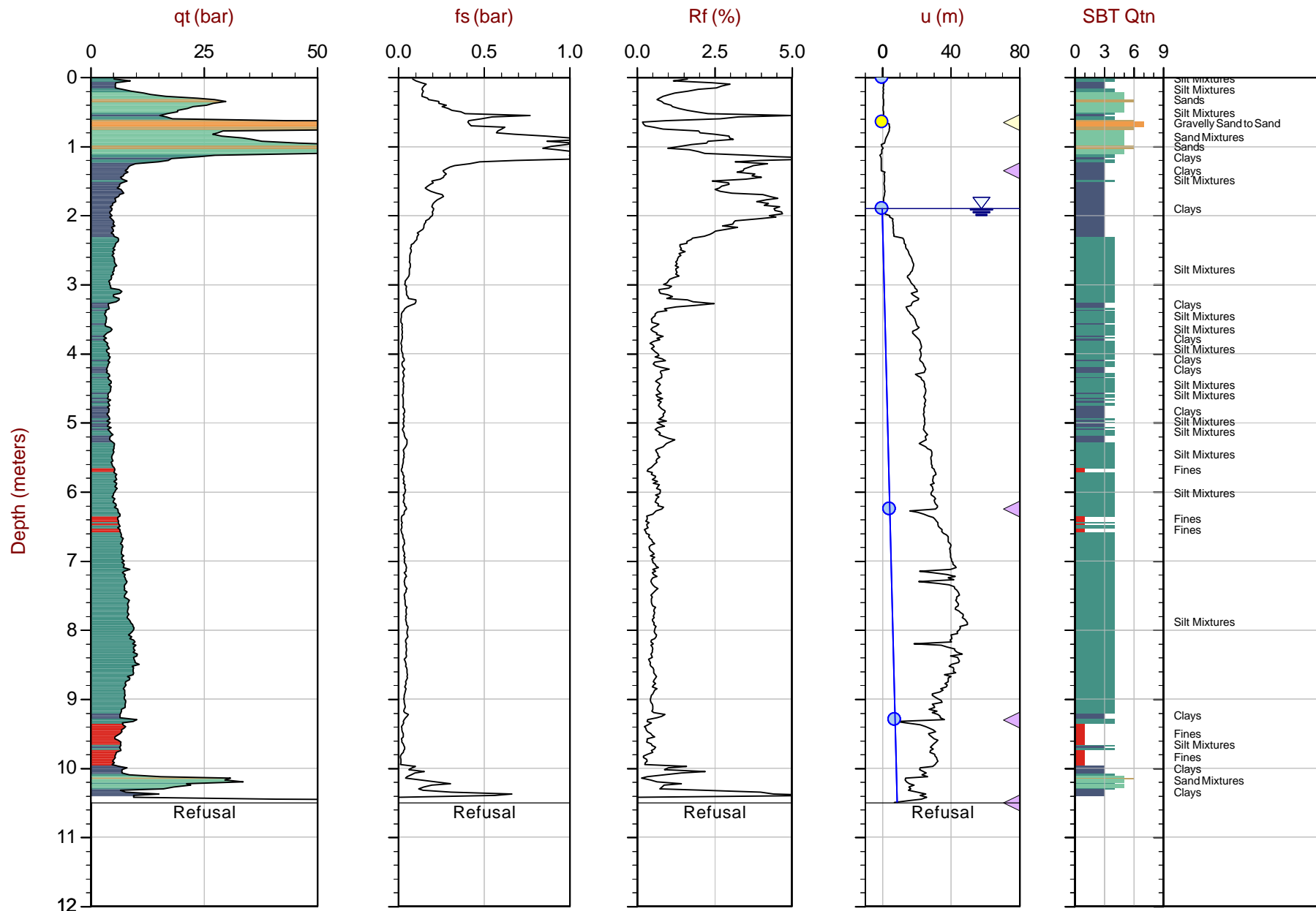
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Date: 2018-05-15 08:36

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-09

Cone: 330:T1500F15U500



Max Depth: 10.500 m / 34.45 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP09.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N:5011401mE:438482m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▼ Dissipation, Ueq not achieved    — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

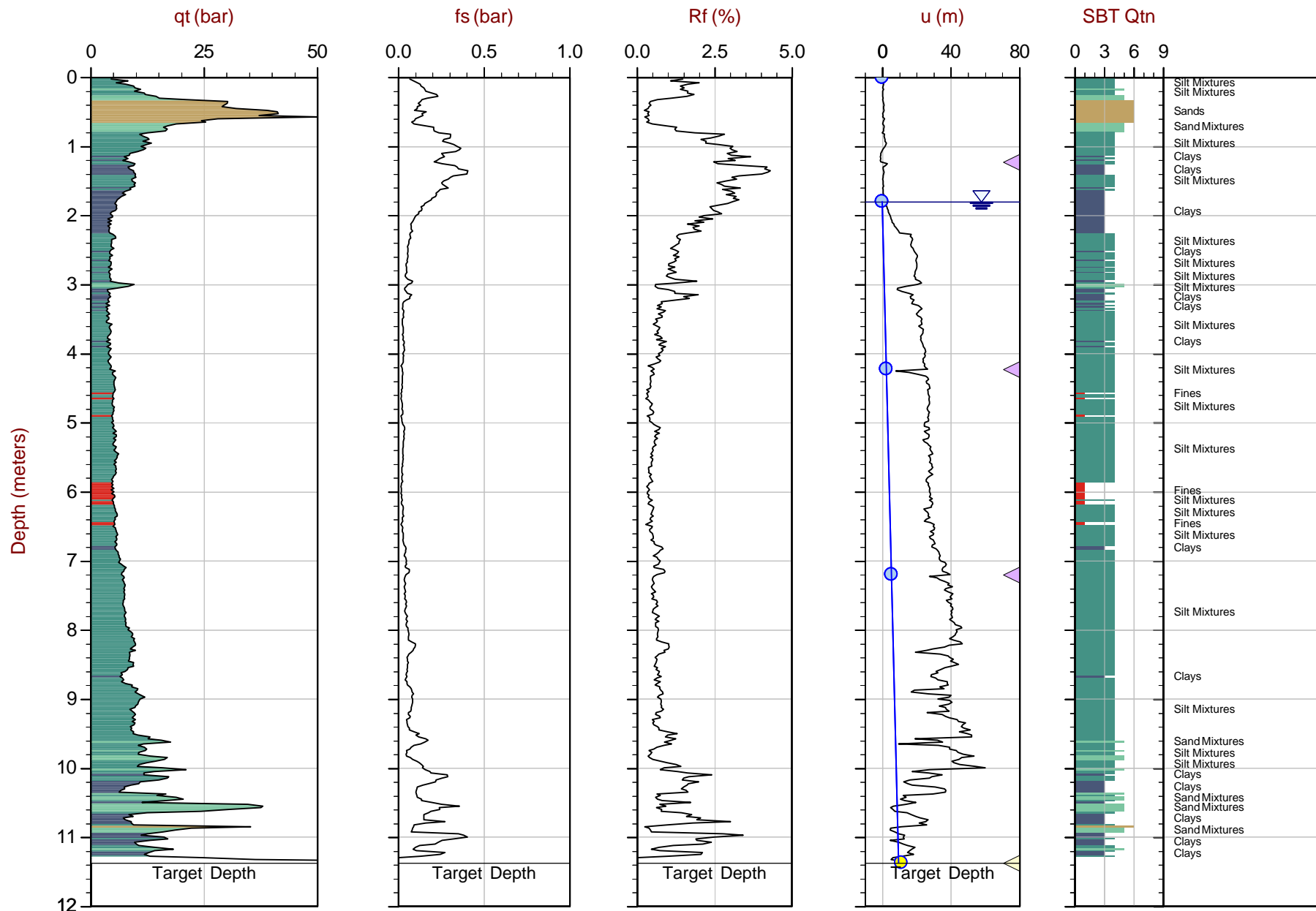
Job No: 18-05030

Date: 2018-05-15 12:01

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-10

Cone: 330:T1500F15U500



Max Depth: 11.375 m / 37.32 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP10.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N:5011318mE:438505m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

## Advanced Cone Penetration Test Plots



Golder

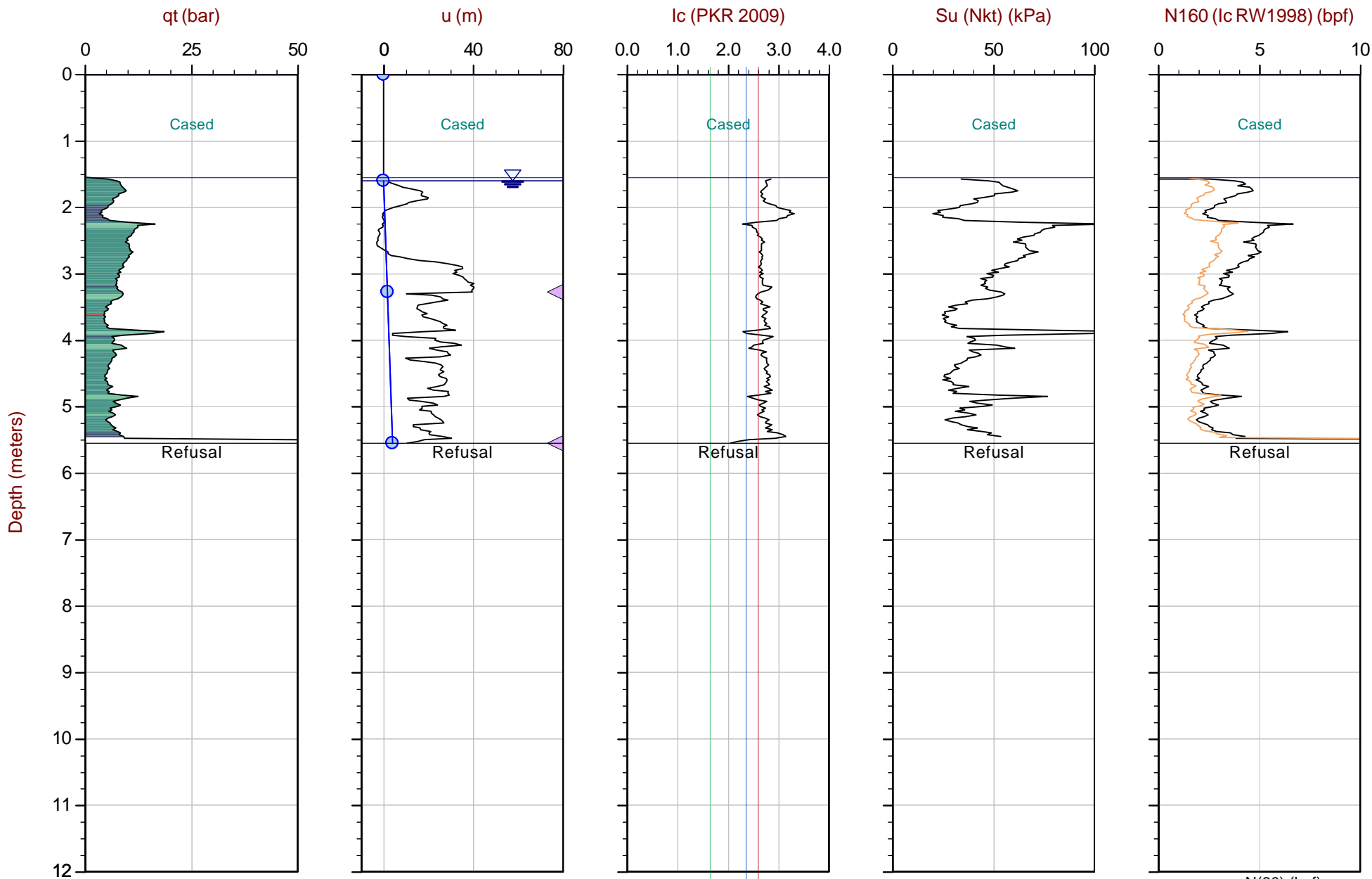
Job No: 18-05030

Date: 2018-05-16 11:08

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-03

Cone: 330:T1500F15U500



Max Depth: 5.550 m / 18.21 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP03.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM18N N: 5011559m E: 438416m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.





Golder

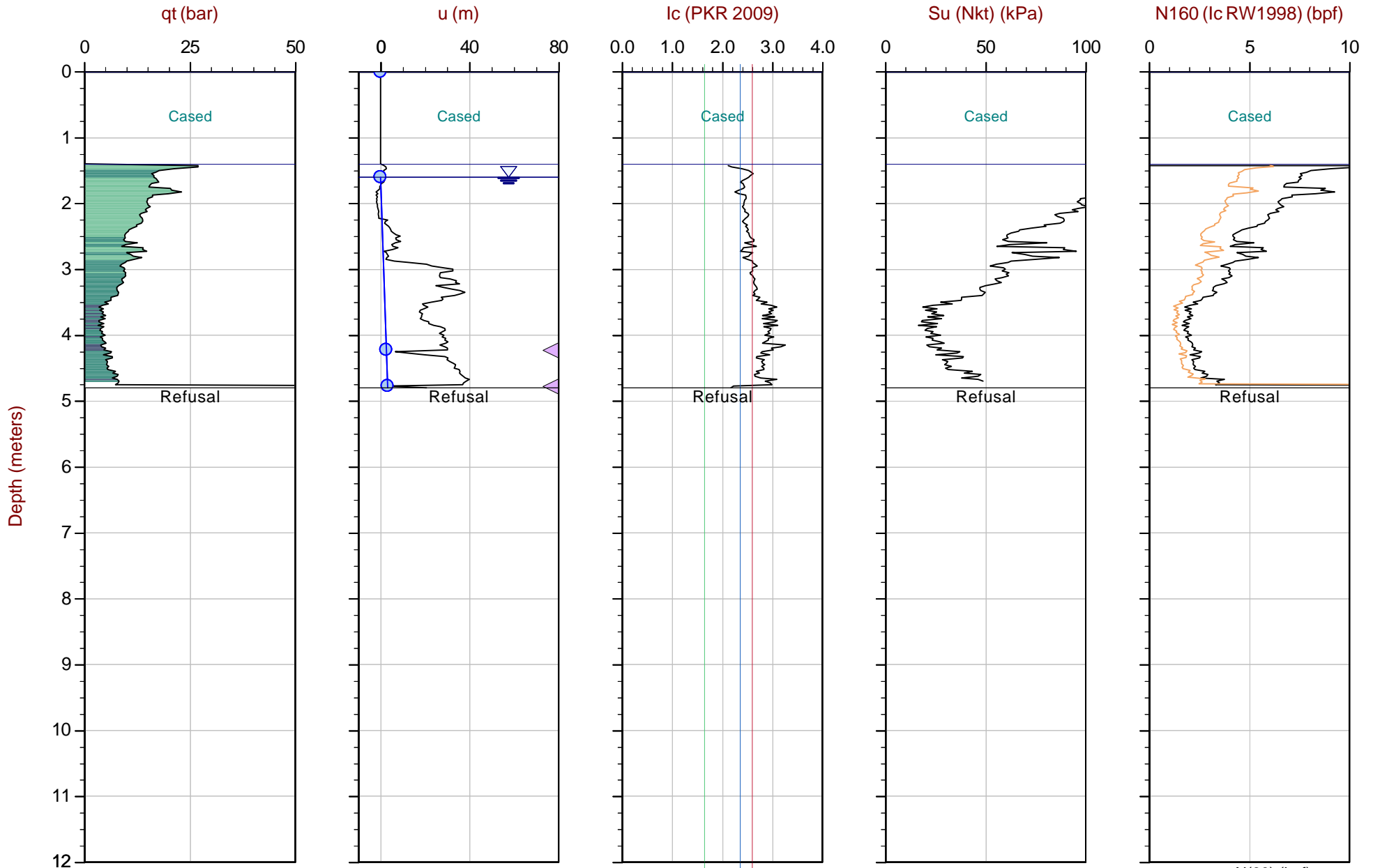
Job No: 18-05030

Date: 2018-05-16 09:32

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-04

Cone: 330:T1500F15U500



Max Depth: 4.800 m / 15.75 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP04.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM 18N N: 5011539m E: 438422m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

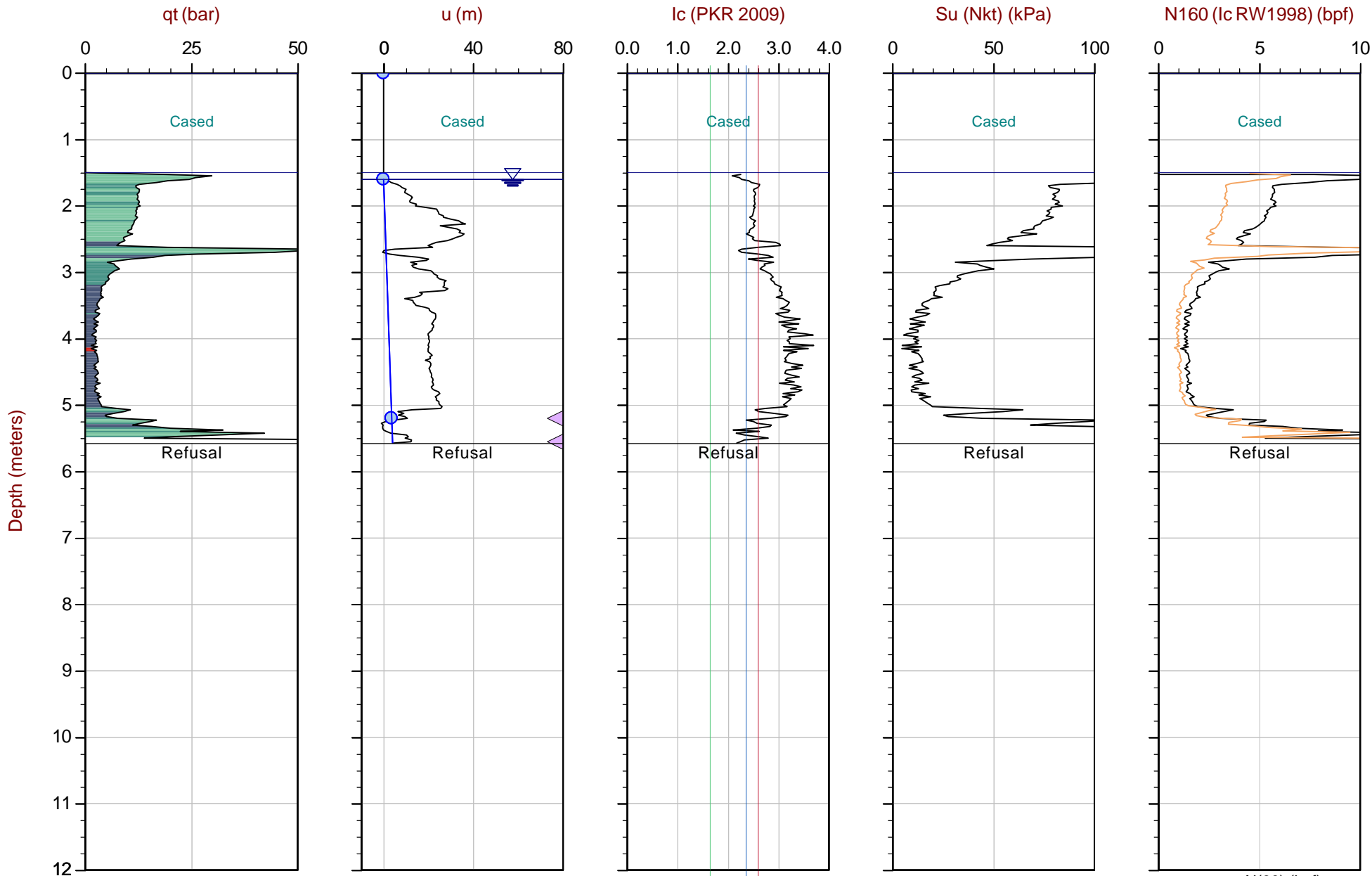
Job No: 18-05030

Date: 2018-05-16 08:22

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-05

Cone: 330:T1500F15U500



Max Depth: 5.575 m / 18.29 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP05.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM18N N: 5011513m E: 438429m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

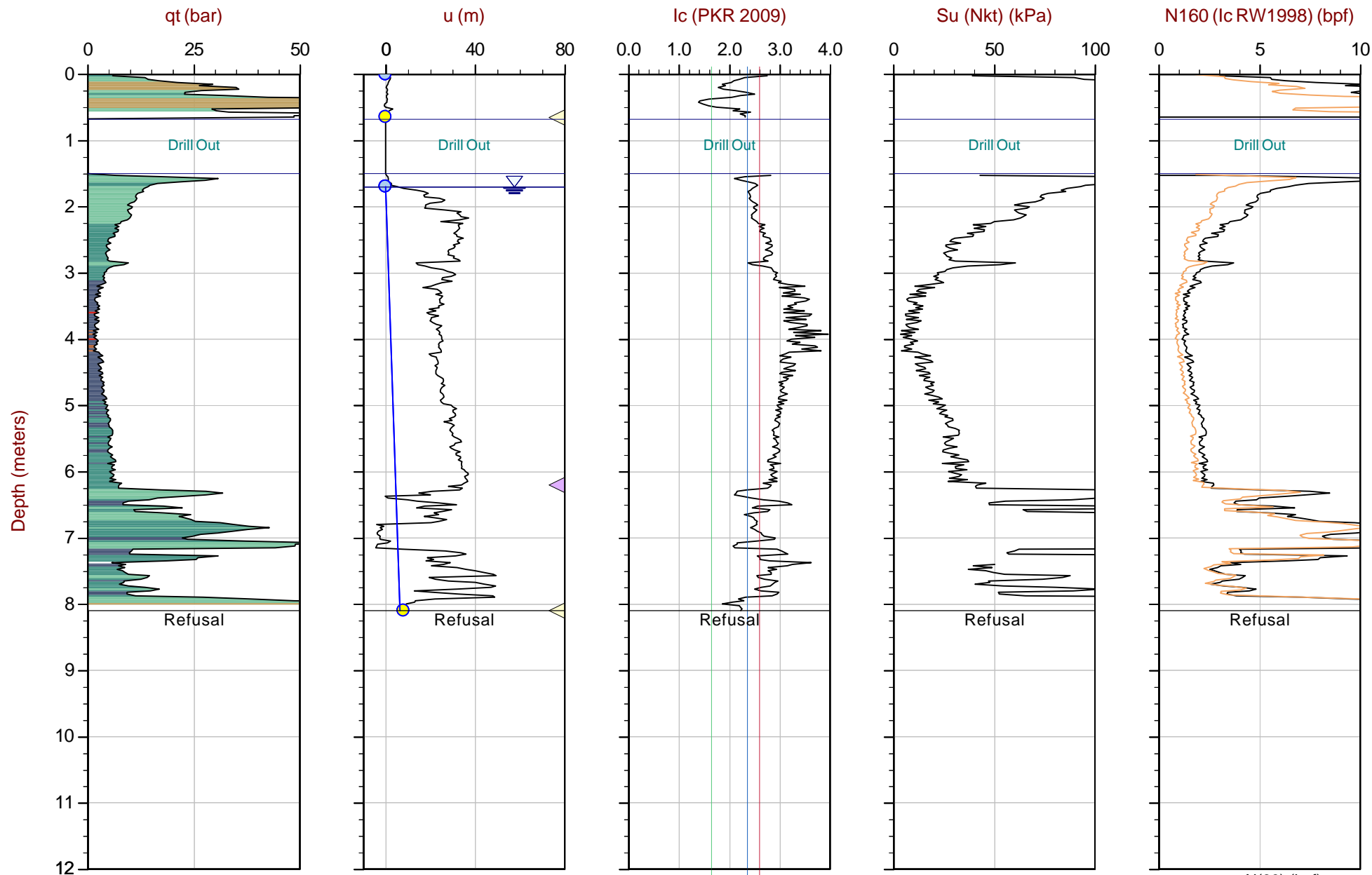
Job No: 18-05030

Date: 2018-05-15 14:58

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-06

Cone: 330:T1500F15U500



Max Depth: 8.100 m / 26.57 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP06.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM18N: 5011489m E: 438432m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

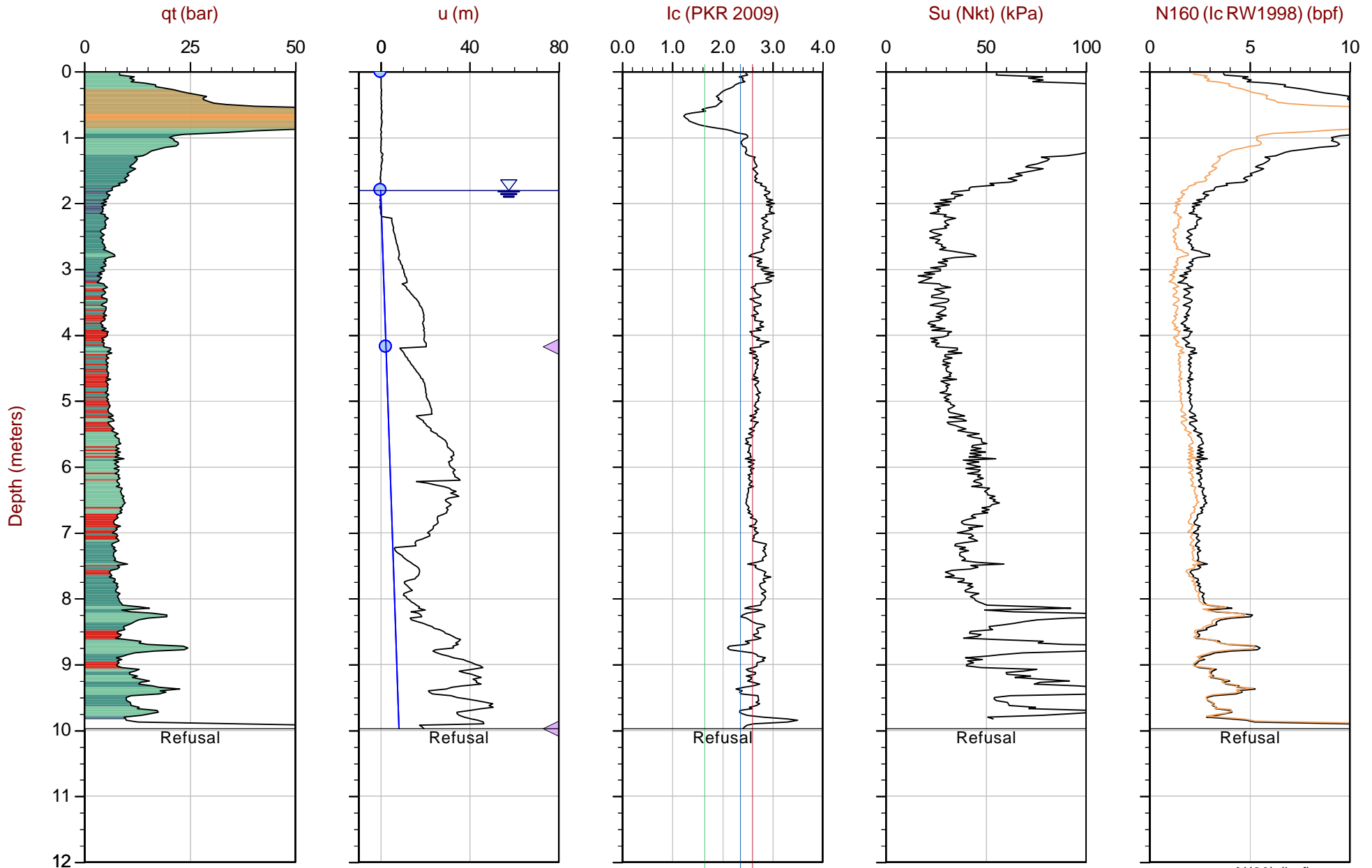
Job No: 18-05030

Date: 2018-05-14 14:54

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-07

Cone: 330:T1500F15U500



Max Depth: 9.975 m / 32.73 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP07.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM18N: 5011450m E: 438451m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

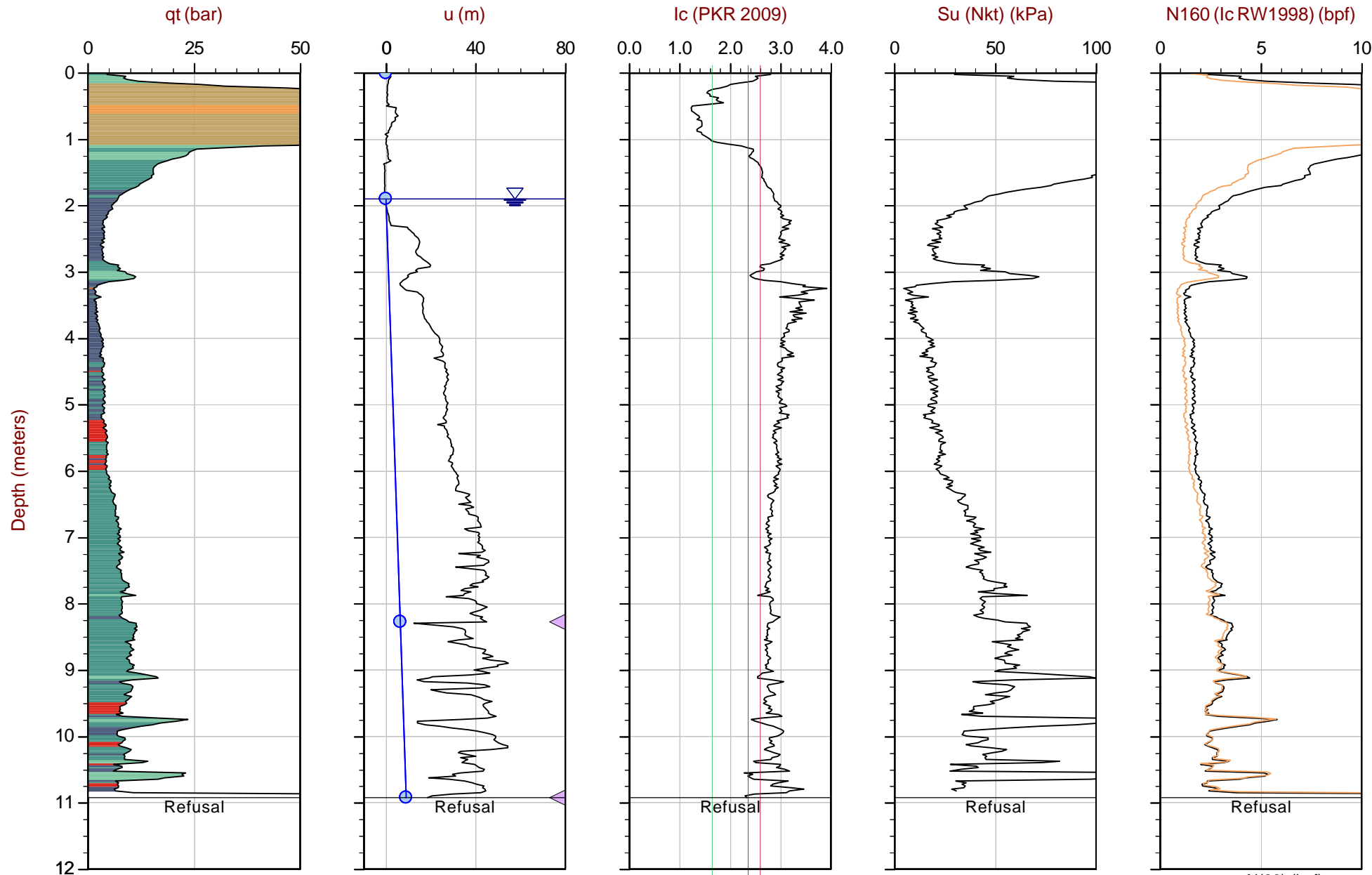
Job No: 18-05030

Date: 2018-05-15 10:18

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-08

Cone: 330:T1500F15U500



Max Depth: 10.925 m / 35.84 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP08.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM18N: 5011428mE: 438467m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

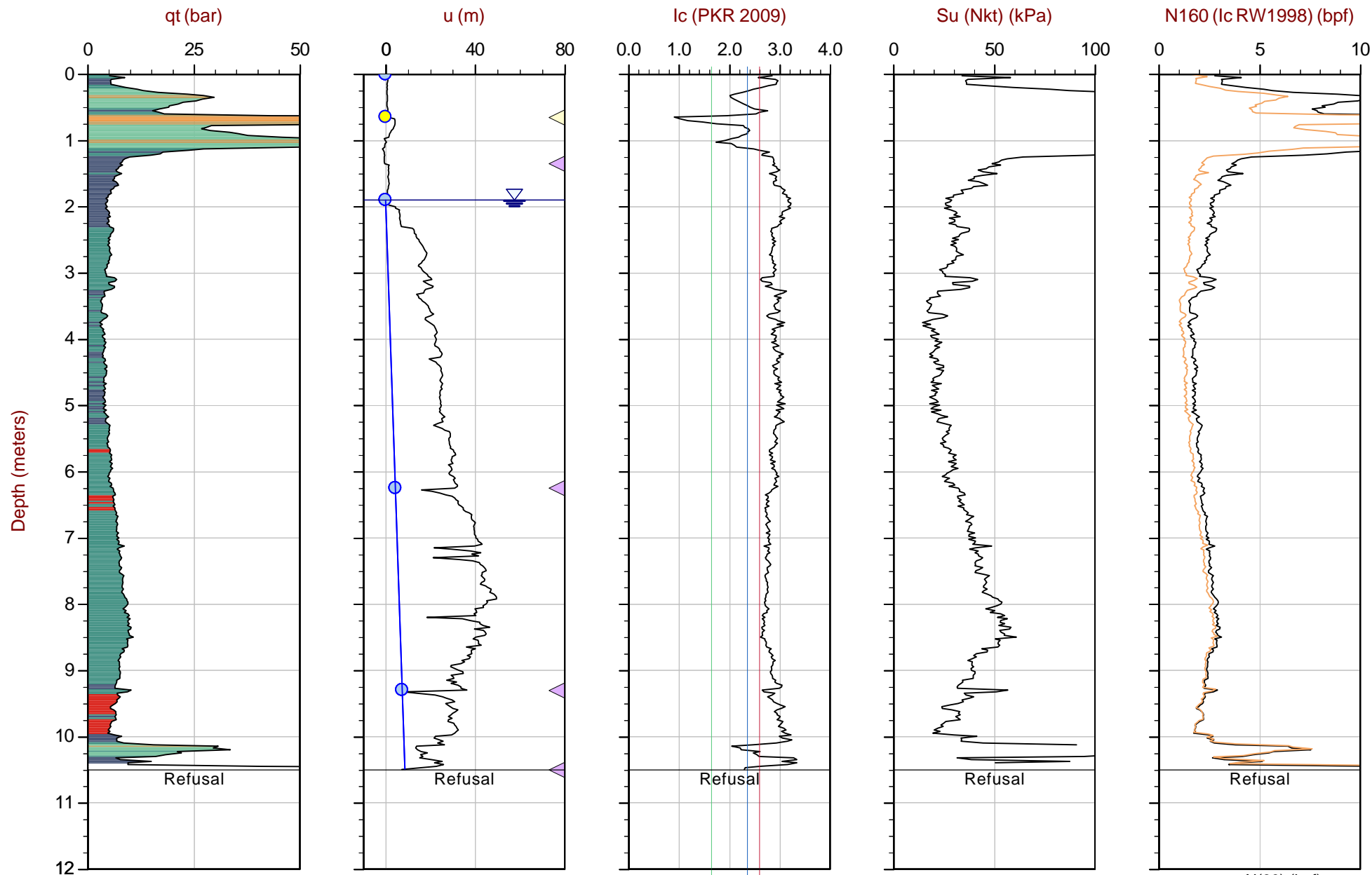
Job No: 18-05030

Date: 2018-05-15 08:36

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-09

Cone: 330:T1500F15U500



Max Depth: 10.500 m / 34.45 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP09.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM18N: 5011401mE: 438482m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

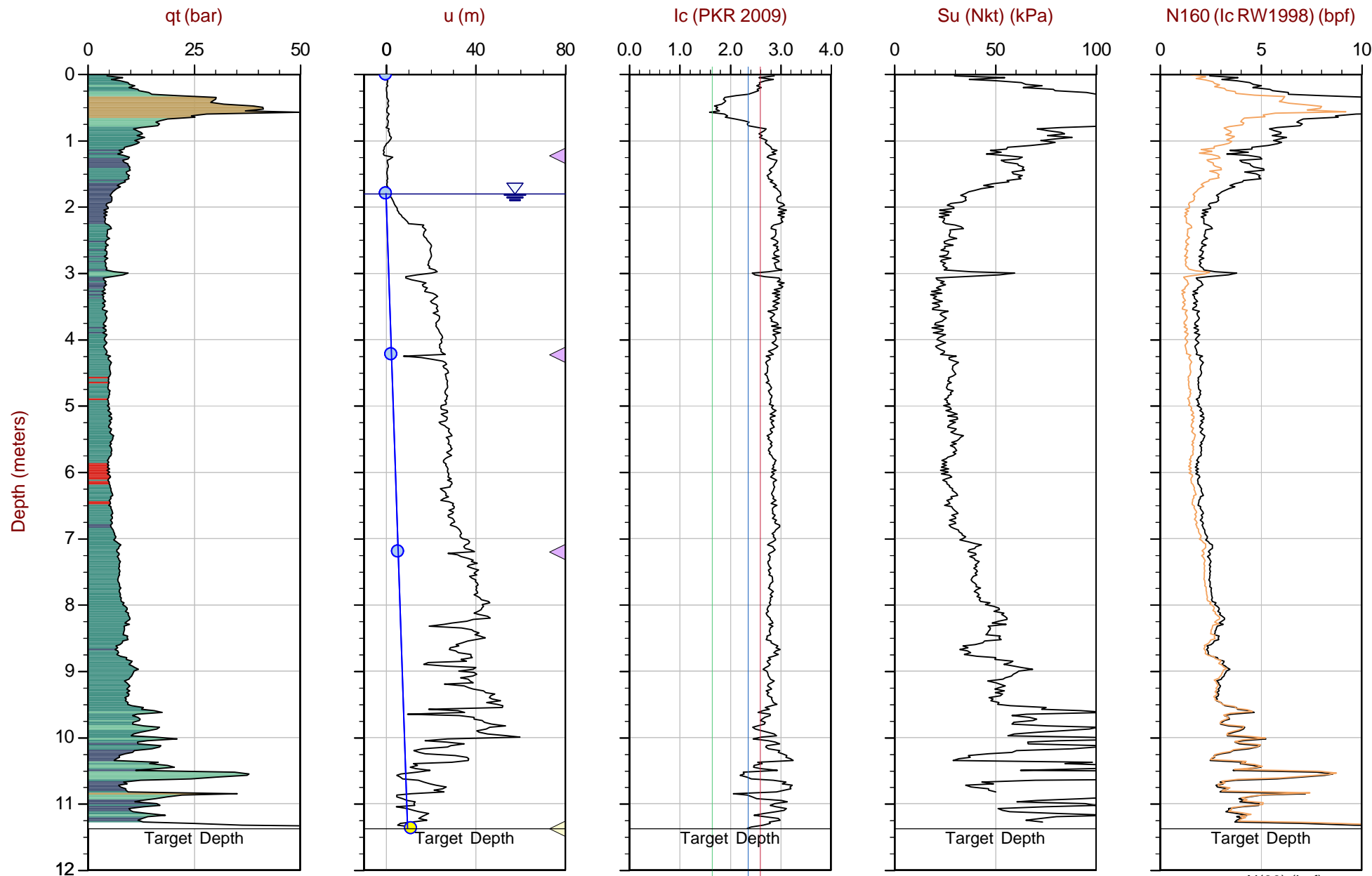
Job No: 18-05030

Date: 2018-05-15 12:01

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-10

Cone: 330:T1500F15U500



Max Depth: 11.375 m / 37.32 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 18-05030\_SP10.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM18N: 5011318mE: 438505m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

## Soil Behaviour Type (SBT) Scatter Plots





*Golder*

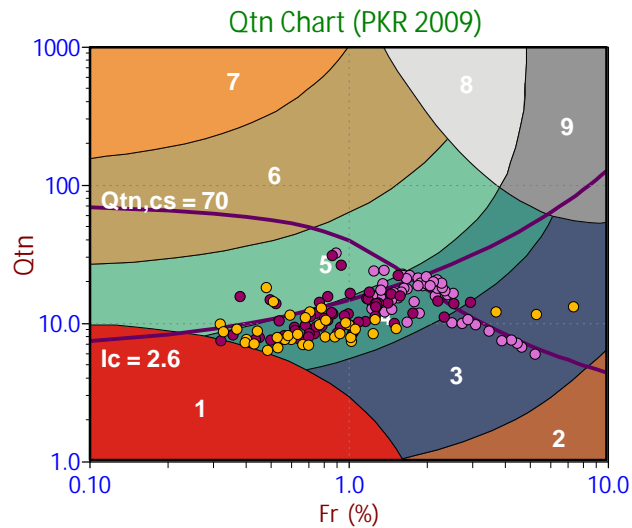
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Date: 2018-05-16 11:08

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-03

Cone: 330:T1500F15U500

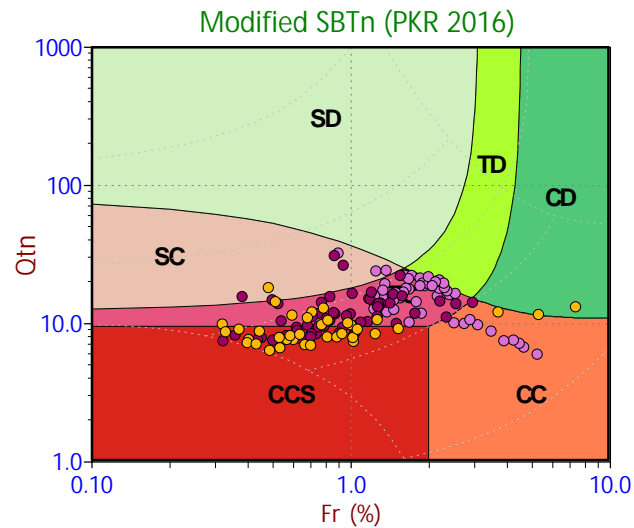


Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

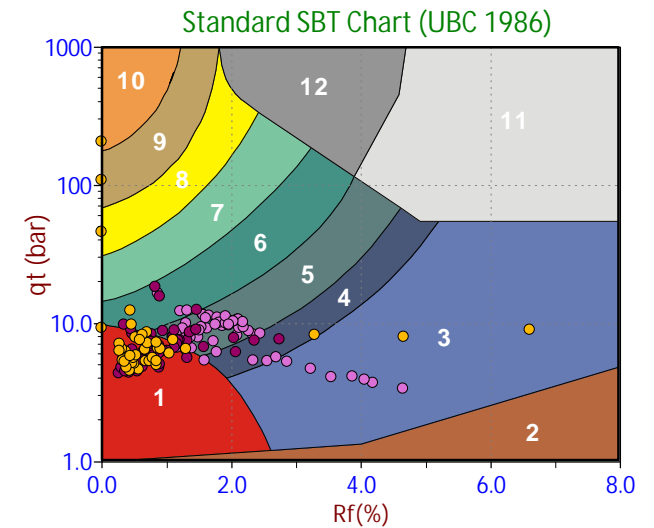
Legend

- Fines
- Fines
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained



Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)



Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand



*Golder*

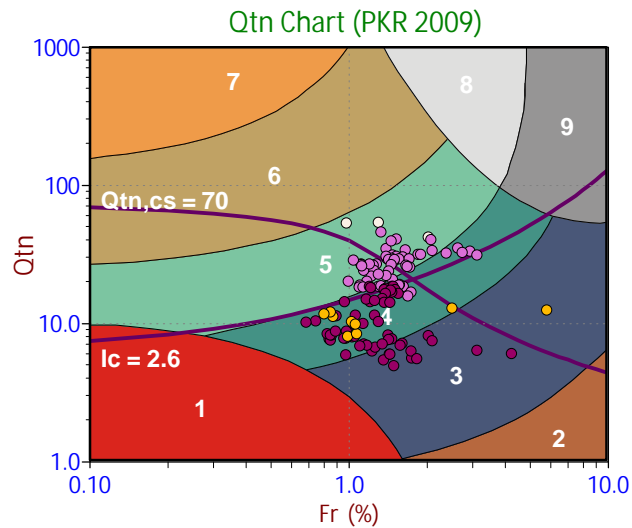
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Date: 2018-05-16 09:32

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Sounding: SCPT18-04

Cone: 330:T1500F15U500

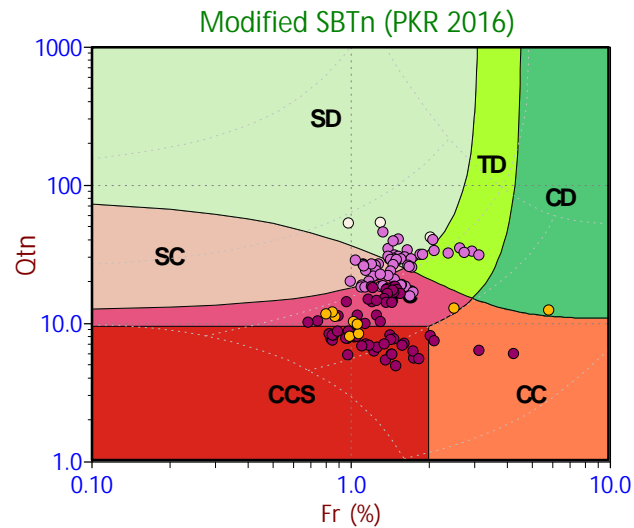


Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
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- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

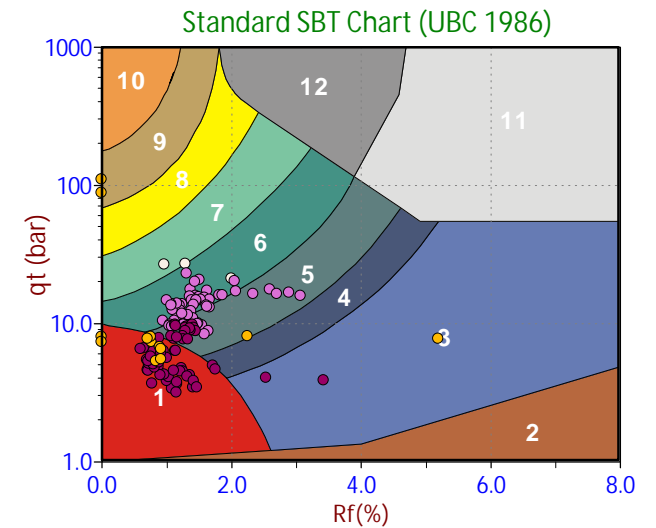
Legend

- Fines
- Fines
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained



Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)



Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand



*Golder*

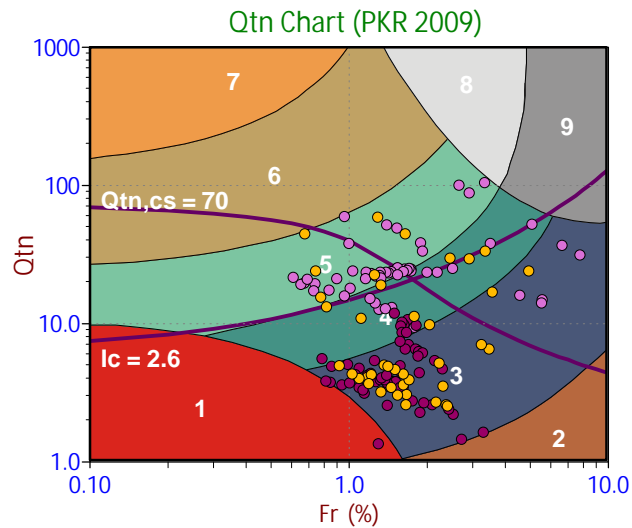
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Sounding: SCPT18-05

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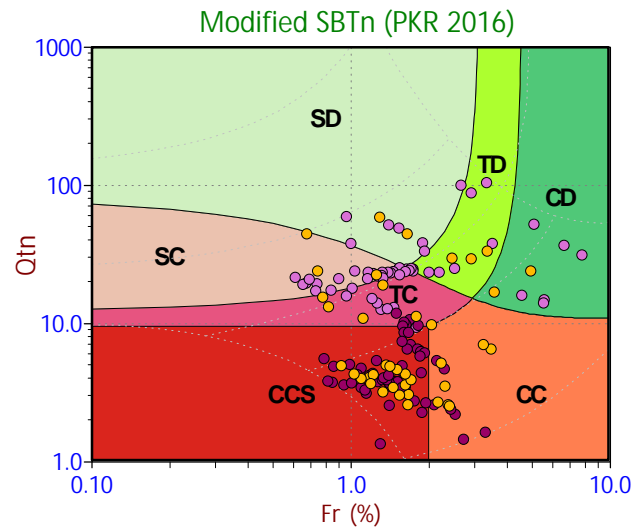


Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

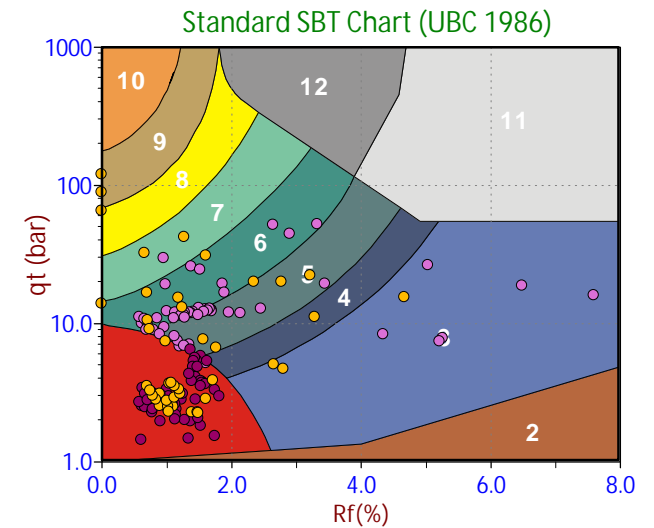
Legend

- Fines
- Fines
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained



Legend

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- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)



Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand



*Golder*

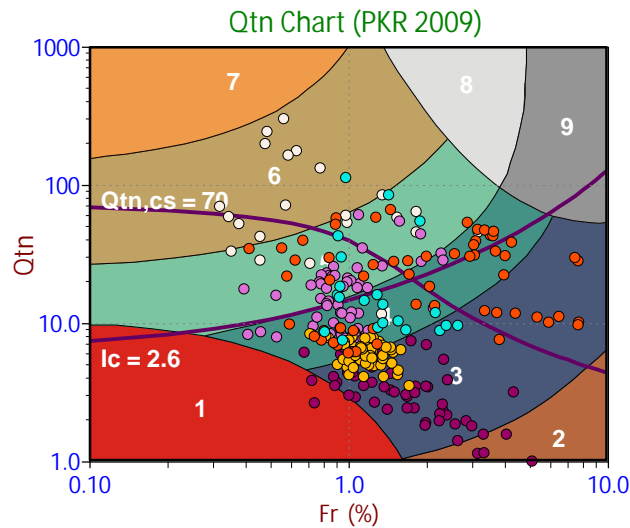
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Date: 2018-05-15 14:58

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-06

Cone: 330:T1500F15U500

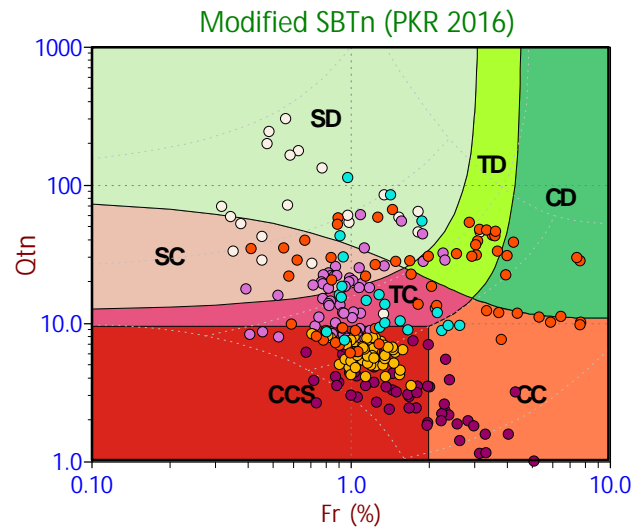


Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

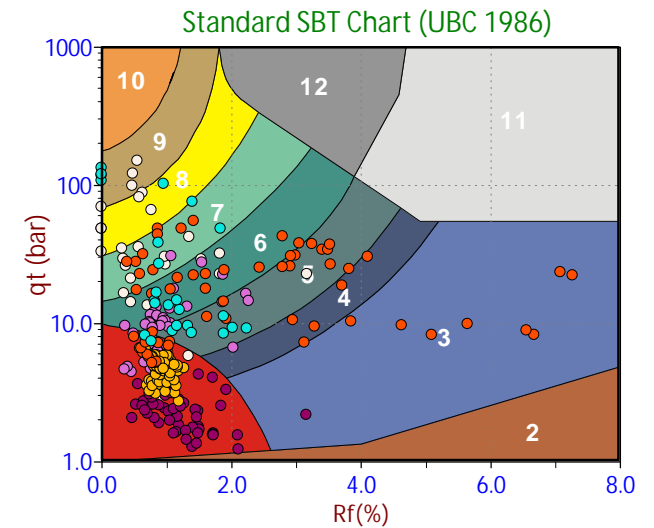
Legend

- Fines
- Fines
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained



Legend

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Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand



*Golder*

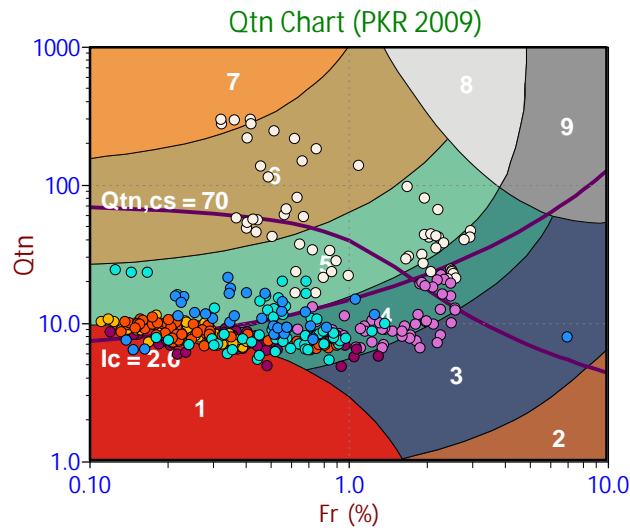
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Date: 2018-05-14 14:54

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-07

Cone: 330:T1500F15U500

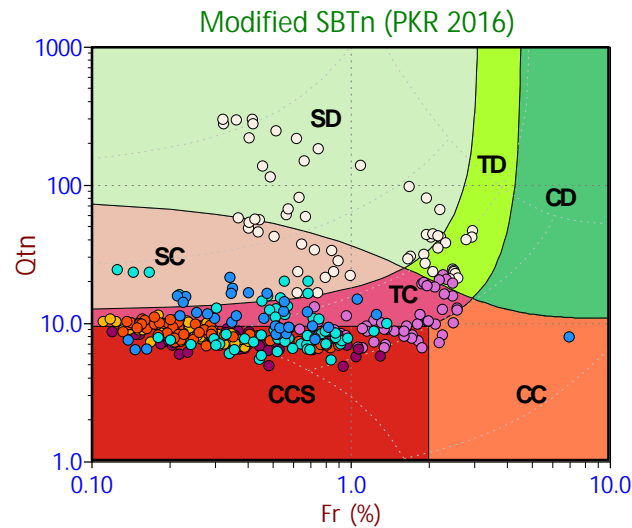


Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
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- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

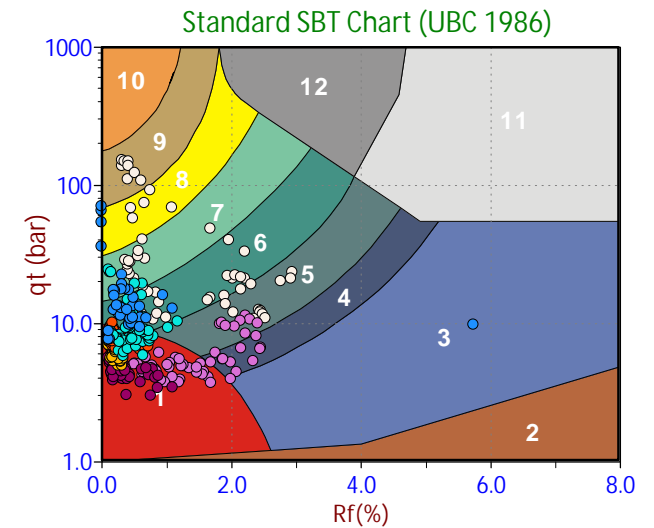
Legend

- Fines
- Fines
- Clays
- Silt Mixtures
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- Sands
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Legend

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- SD (Dil. sand like)



Legend

- Sensitive Fines
- Organic Soil
- Clay
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- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand



*Golder*

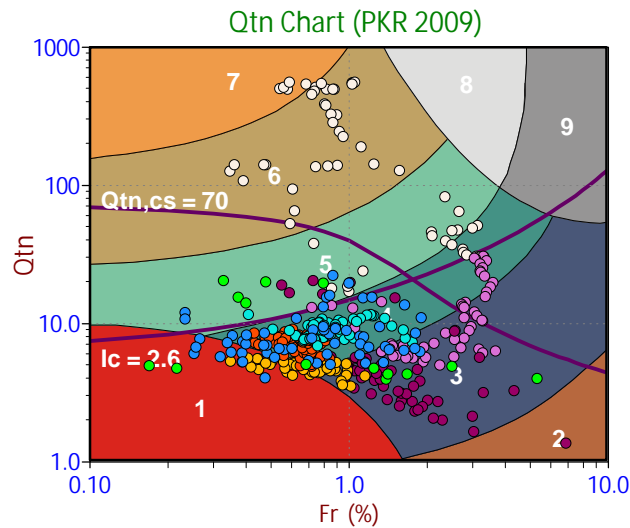
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Date: 2018-05-15 10:18

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-08

Cone: 330:T1500F15U500

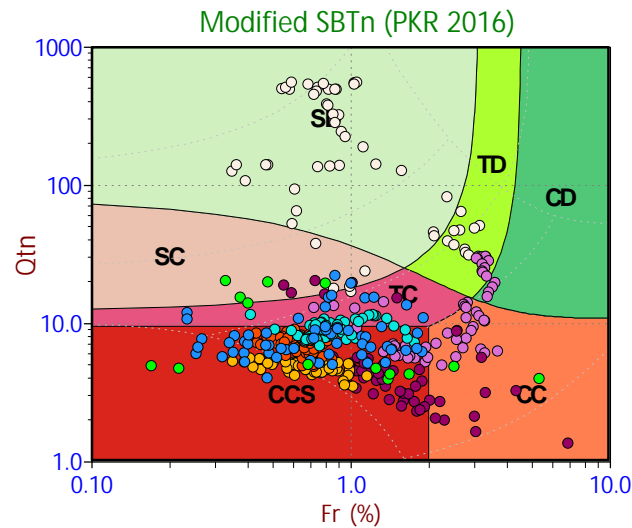


Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
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- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

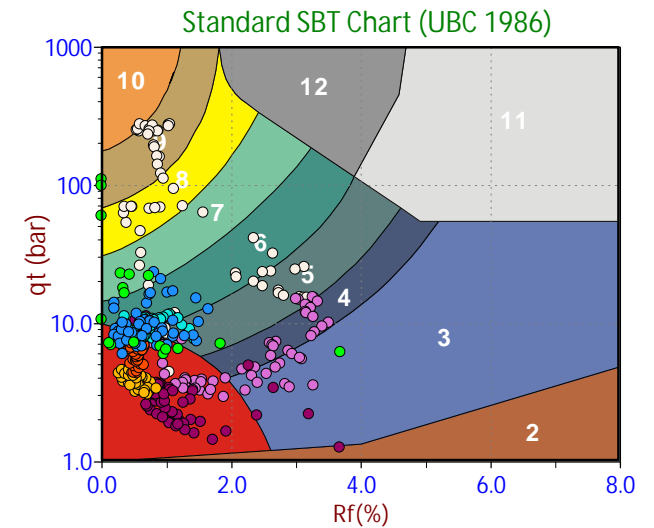
Legend

- Fines
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- Silt Mixtures
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- Gravelly Sand to Sand
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Legend

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Legend

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- Silt
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*Golder*

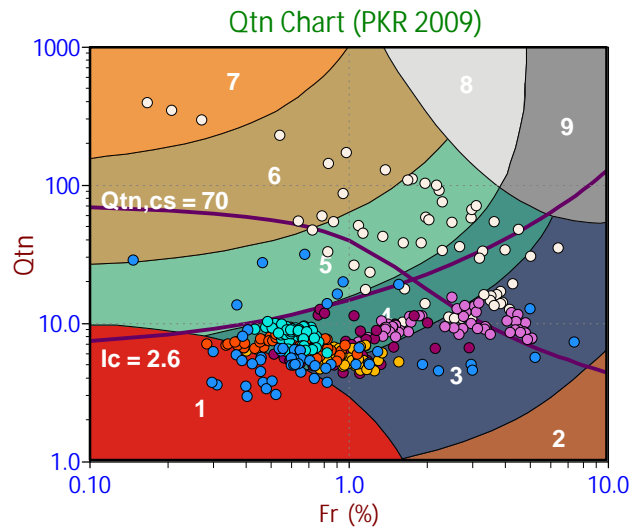
Job No: 18-05030

Date: 2018-05-15 08:36

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-09

Cone: 330:T1500F15U500

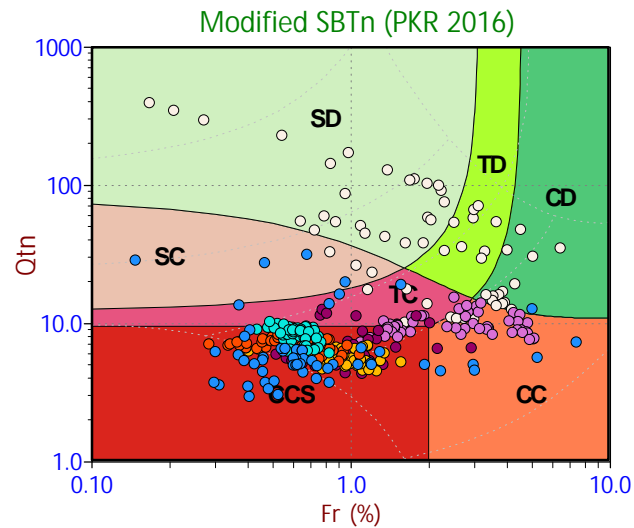


Depth Ranges

- >0.0 to 1.5 m
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- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

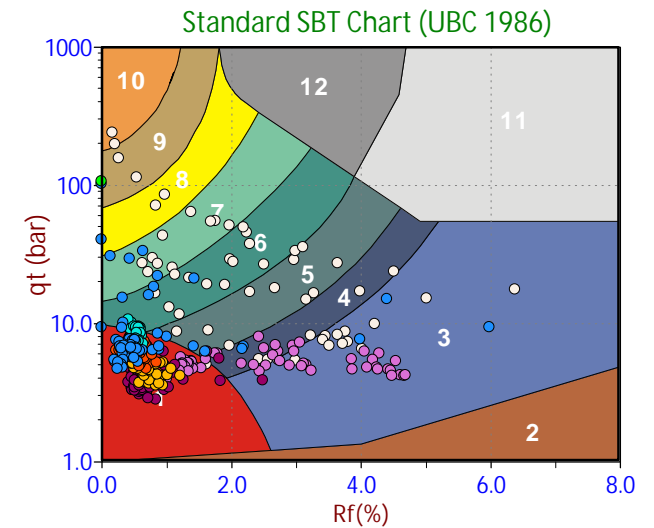
Legend

- Fines
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- Clays
- Silt Mixtures
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Legend

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- SD (Dil. sand like)



Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand



*Golder*

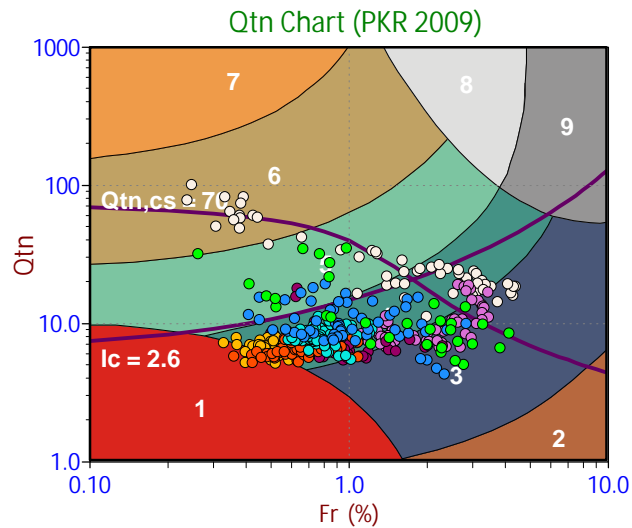
Job No: 18-05030

Date: 2018-05-15 12:01

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-10

Cone: 330:T1500F15U500

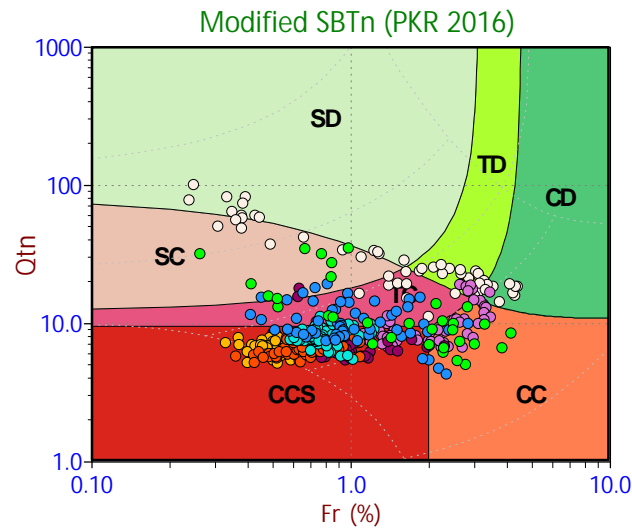


Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

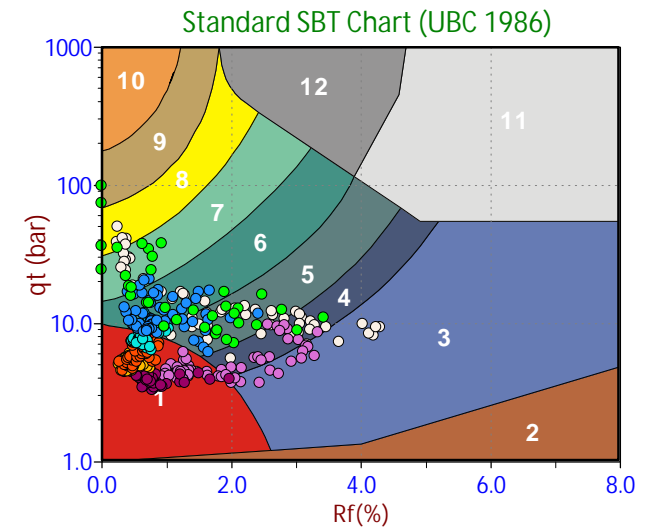
Legend

- Fines
- Fines
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained



Legend

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- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)



Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand



## Seismic Cone Penetration Test Tabular Results



Job No: 18-05030  
Client: Golder Associates  
Project: Hwy 416 and McKenna Casey Dr  
Sounding ID: SCPT18-03  
Date: 16-May-2018

Seismic Source: Beam  
Source Offset (m): 1.15  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
2.27	2.07	2.37			
3.28	3.08	3.29	0.92	5.65	163
4.25	4.05	4.21	0.92	8.08	114
5.25	5.05	5.18	0.97	8.22	118



Job No: 18-05030  
Client: Golder Associates  
Project: Hwy 416 and McKenna Casey Dr  
Sounding ID: SCPT18-04  
Date: 16-May-2018

Seismic Source: Beam  
Source Offset (m): 1.20  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
2.23	2.03	2.36			
3.23	3.03	3.26	0.90	5.46	165
4.22	4.02	4.20	0.94	6.46	145
4.80	4.60	4.75	0.56	5.62	99



Job No: 18-05030  
Client: Golder Associates  
Project: Hwy 416 and McKenna Casey Dr  
Sounding ID: SCPT18-05  
Date: 16-May-2018

Seismic Source: Beam  
Source Offset (m): 1.15  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
2.28	2.08	2.35			
3.27	3.07	3.26	0.91	5.82	156
4.28	4.08	4.23	0.96	9.52	101
5.20	5.00	5.12	0.89	11.14	80



Job No: 18-05030  
Client: Golder Associates  
Project: Hwy 416 and McKenna Casey Dr  
Sounding ID: SCPT18-06  
Date: 15-May-2018

Seismic Source: Beam  
Source Offset (m): 1.15  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
2.20	2.00	2.31			
3.20	3.00	3.21	0.91	7.13	127
4.20	4.00	4.16	0.95	9.21	103
5.22	5.02	5.15	0.99	11.47	86
6.20	6.00	6.11	0.96	7.87	122
7.20	7.00	7.09	0.98	4.32	228
8.10	7.90	7.98	0.89	2.85	312



Job No: 18-05030  
Client: Golder Associates  
Project: Hwy 416 and McKenna Casey Dr  
Sounding ID: SCPT18-07  
Date: 14-May-2018

Seismic Source: Beam  
Source Offset (m): 1.20  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### **SCPT<sub>u</sub> 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)
1.23	1.03	1.58			
2.20	2.00	2.33	0.75	6.61	114
3.20	3.00	3.23	0.90	7.03	128
4.17	3.97	4.15	0.92	10.39	88
5.20	5.00	5.14	0.99	12.56	79
6.20	6.00	6.12	0.98	10.91	90
7.20	7.00	7.10	0.98	7.23	136
8.18	7.98	8.07	0.97	5.79	167
9.20	9.00	9.08	1.01	4.67	216
9.97	9.77	9.84	0.76	1.93	397



Job No: 18-05030  
Client: Golder Associates  
Project: Hwy 416 and McKenna Casey Dr  
Sounding ID: SCPT18-08  
Date: 15-May-2018

Seismic Source: Beam  
Source Offset (m): 1.10  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> 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)
1.30	1.10	1.56			
2.30	2.10	2.37	0.82	7.44	110
3.30	3.10	3.29	0.92	7.80	118
4.28	4.08	4.23	0.94	9.64	97
5.28	5.08	5.20	0.97	10.80	90
6.28	6.08	6.18	0.98	10.79	91
7.28	7.08	7.16	0.99	9.43	105
8.28	8.08	8.15	0.99	6.56	151
9.28	9.08	9.15	0.99	5.75	173
10.30	10.10	10.16	1.01	4.55	223
10.93	10.73	10.79	0.63	2.07	302



Job No: 18-05030  
Client: Golder Associates  
Project: Hwy 416 and McKenna Casey Dr  
Sounding ID: SCPT18-09  
Date: 15-May-2018

Seismic Source: Beam  
Source Offset (m): 1.20  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### **SCPT<sub>u</sub> 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)
1.35	1.15	1.66			
2.30	2.10	2.42	0.76	5.94	127
3.30	3.10	3.32	0.91	8.90	102
4.30	4.10	4.27	0.95	7.70	123
5.30	5.10	5.24	0.97	9.13	106
6.30	6.10	6.22	0.98	9.22	106
7.30	7.10	7.20	0.98	8.85	111
8.28	8.08	8.17	0.97	7.12	136
9.30	9.10	9.18	1.01	6.72	150
10.32	10.12	10.19	1.01	5.84	173





Job No: 18-05030  
Client: Golder Associates  
Project: Hwy 416 and McKenna Casey Dr  
Sounding ID: SCPT18-10  
Date: 15-May-2018

Seismic Source: Beam  
Source Offset (m): 1.10  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### **SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>**

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.23	1.03	1.51			
2.23	2.03	2.31	0.80	5.75	139
3.23	3.03	3.22	0.91	7.04	130
4.23	4.03	4.18	0.95	8.21	116
5.23	5.03	5.15	0.97	8.31	117
6.22	6.02	6.12	0.97	10.78	90
7.20	7.00	7.09	0.97	9.50	102
8.25	8.05	8.12	1.04	6.40	162
9.18	8.98	9.05	0.92	5.26	175
10.22	10.02	10.08	1.03	4.77	216
11.20	11.00	11.05	0.97	3.11	314

## Seismic Cone Penetration Test Plots



Golder

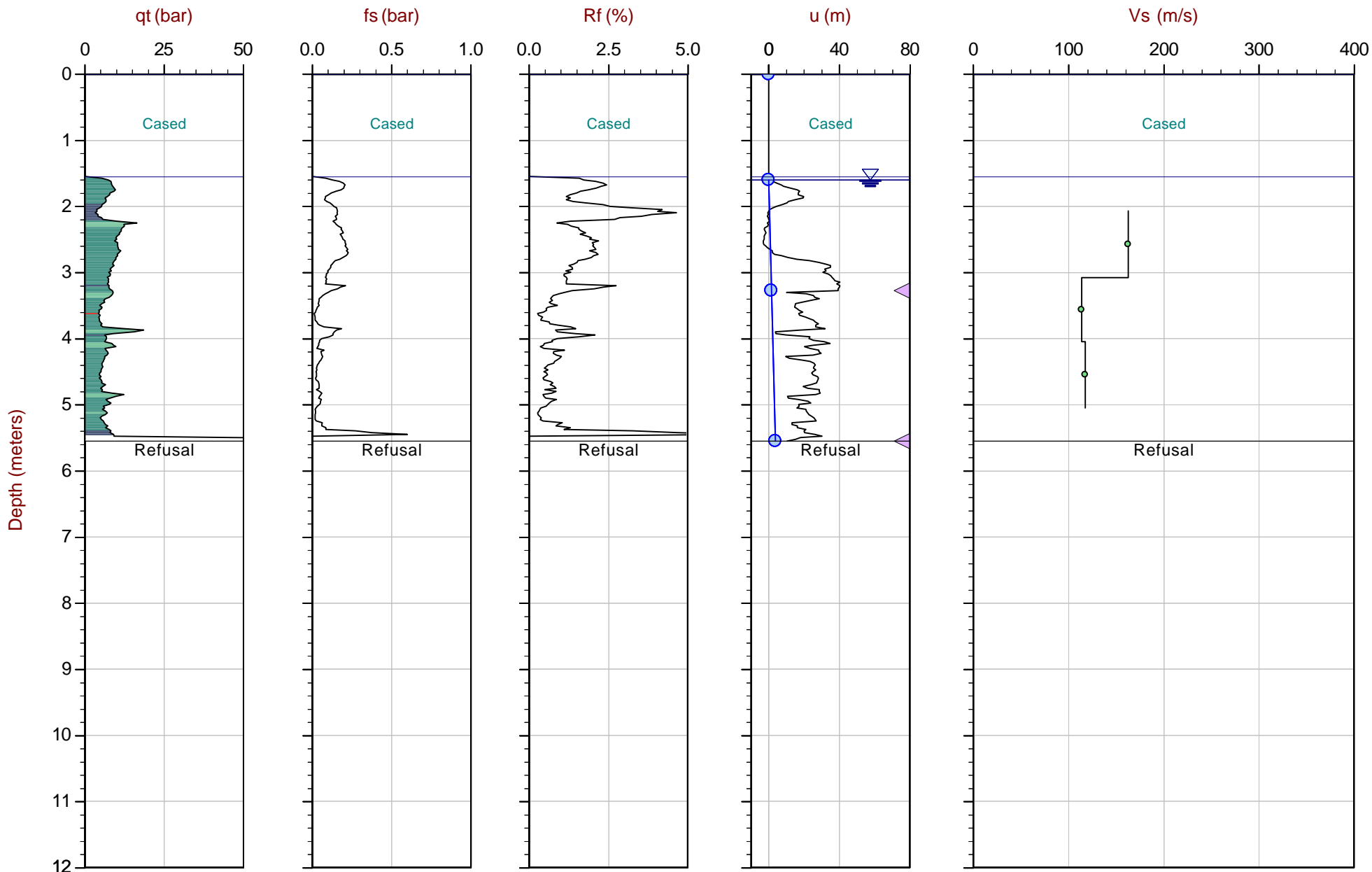
Job No: 18-05030

Date: 2018-05-16 11:08

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-03

Cone: 330:T1500F15U500



Max Depth: 5.550 m / 18.21 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP03.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N N: 5011559m E: 438416m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

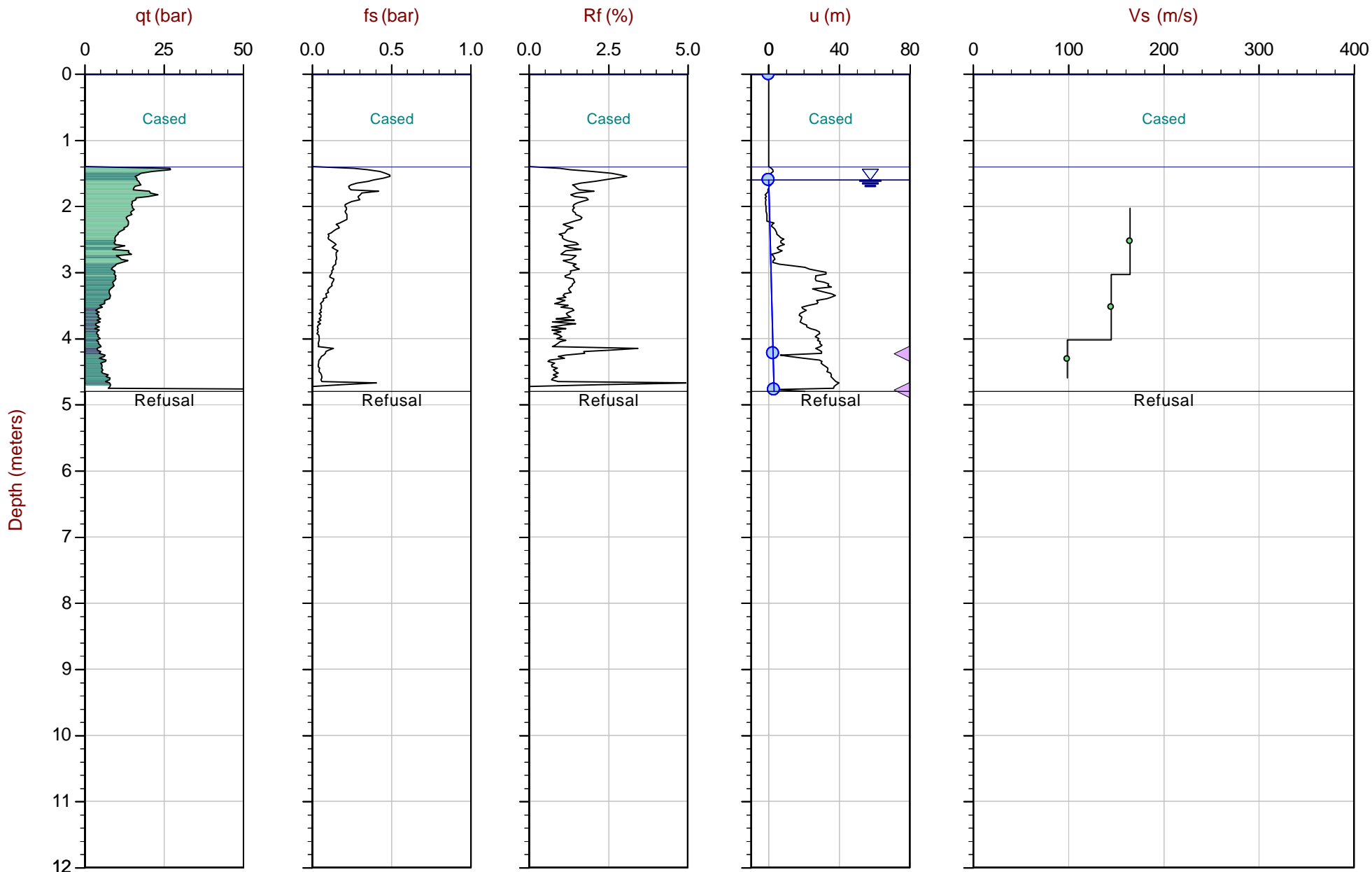
Job No: 18-05030

Date: 2018-05-16 09:32

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-04

Cone: 330:T1500F15U500



Max Depth: 4.800 m / 15.75 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP04.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N N: 5011539m E: 438422m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

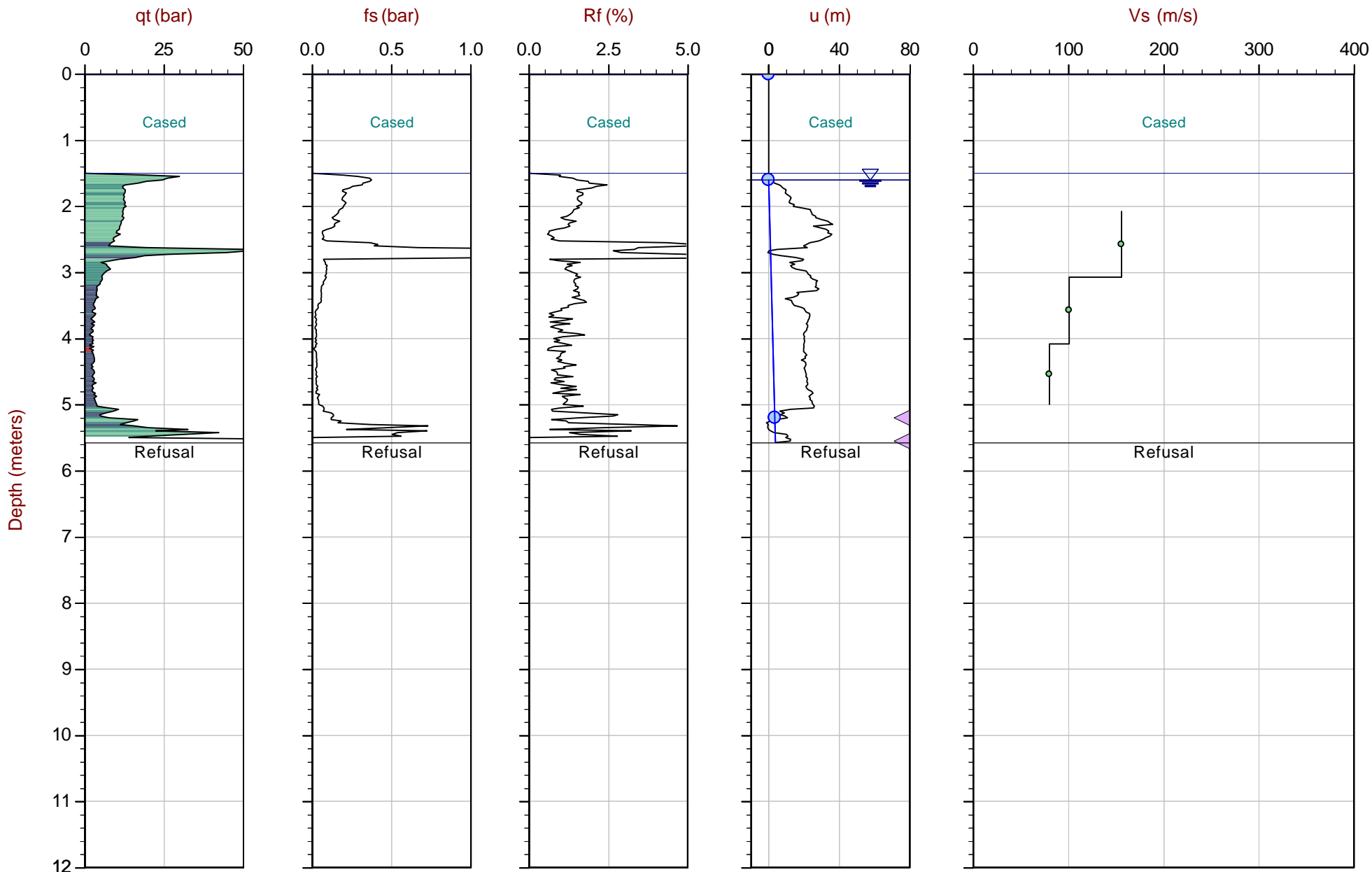
Job No: 18-05030

Date: 2018-05-16 08:22

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-05

Cone: 330:T1500F15U500



Max Depth: 5.575 m / 18.29 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP05.COR

Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N: 5011513mE: 438429m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Golder*

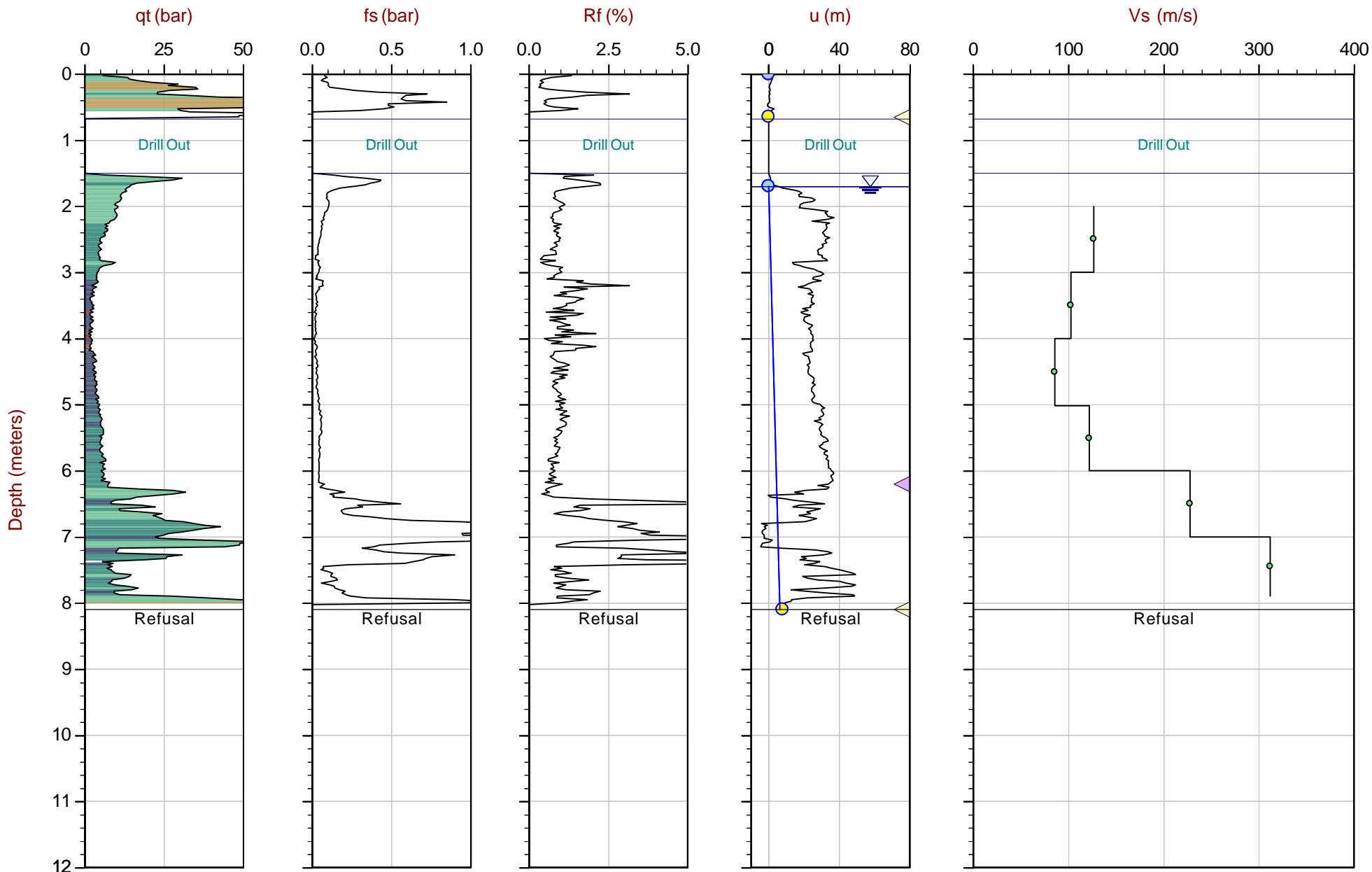
Job No: 18-05030

Date: 2018-05-15 14:58

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-06

Cone: 330:T1500F15U500



Max Depth: 8.100 m / 26.57 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP06.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N N: 5011489m E: 438432m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Golder

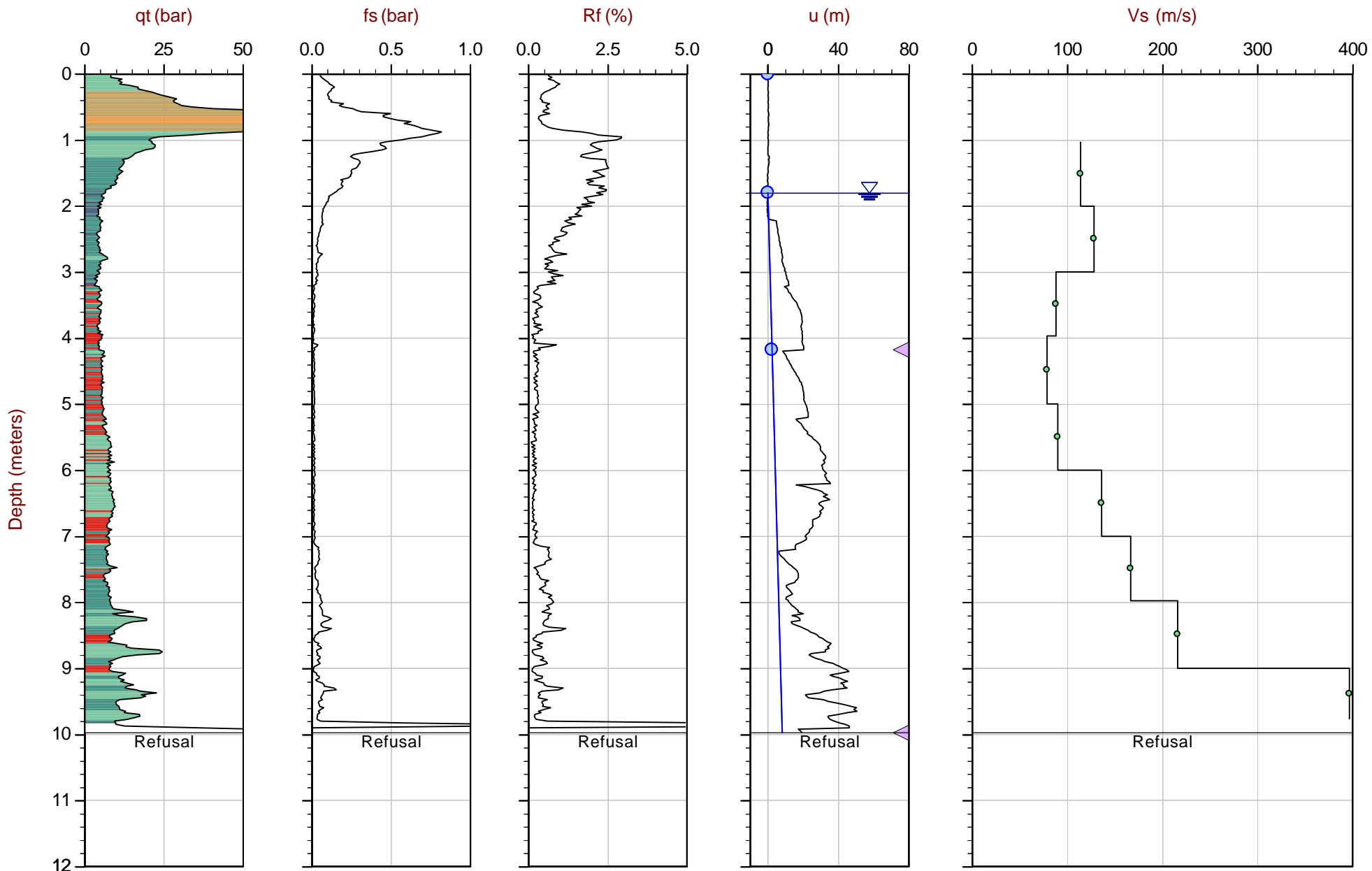
Job No: 18-05030

Date: 2018-05-14 14:54

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-07

Cone: 330:T1500F15U500



Max Depth: 9.975 m / 32.73 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 18-05030\_SP07.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N: 5011450mE: 438451m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Golder*

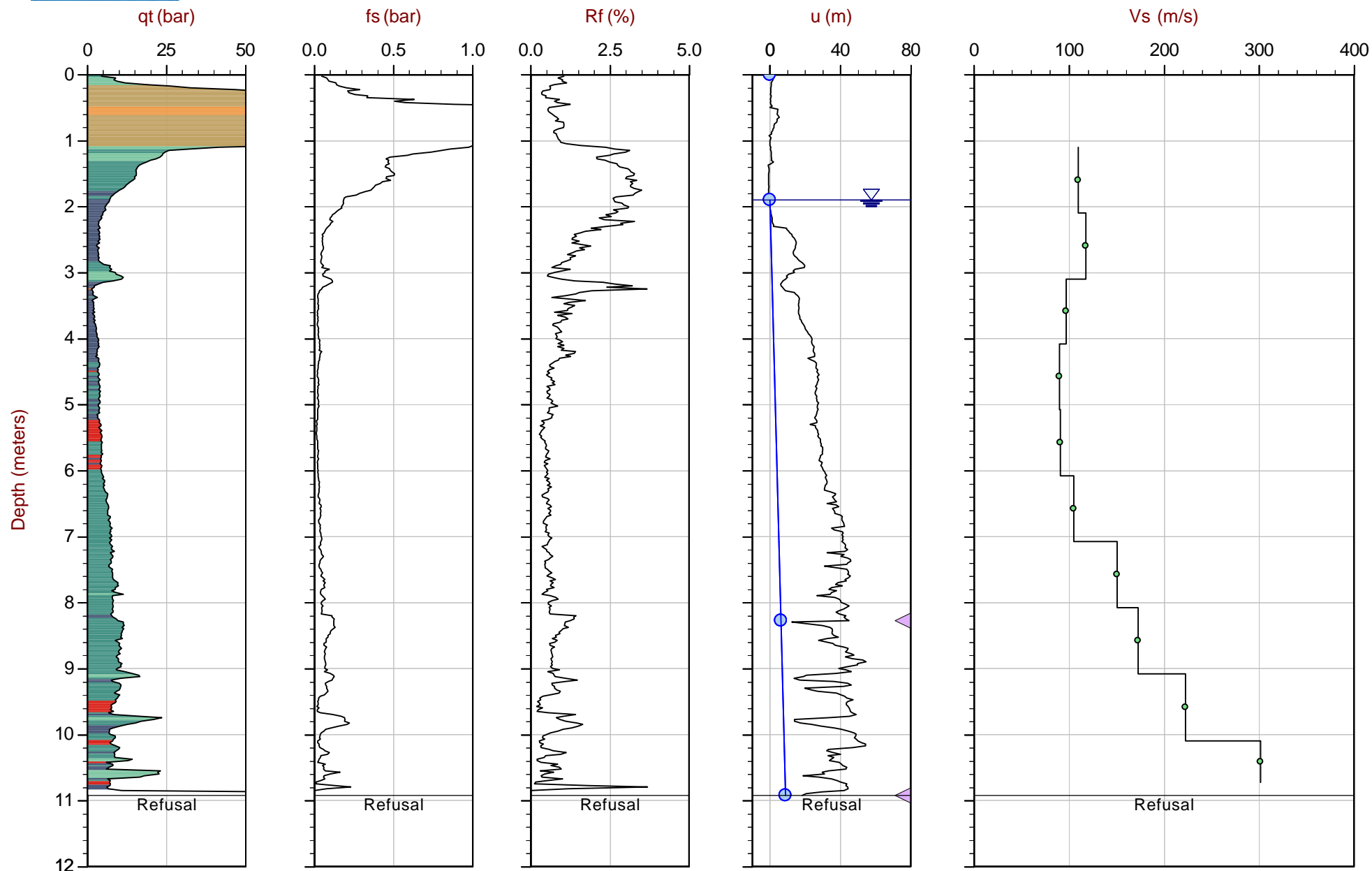
Job No: 18-05030

Date: 2018-05-15 10:18

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-08

Cone: 330:T1500F15U500



Max Depth: 10.925 m / 35.84 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP08.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N: 5011428mE: 438467m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▲ Dissipation, Ueq not achieved    — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.





**Golder**

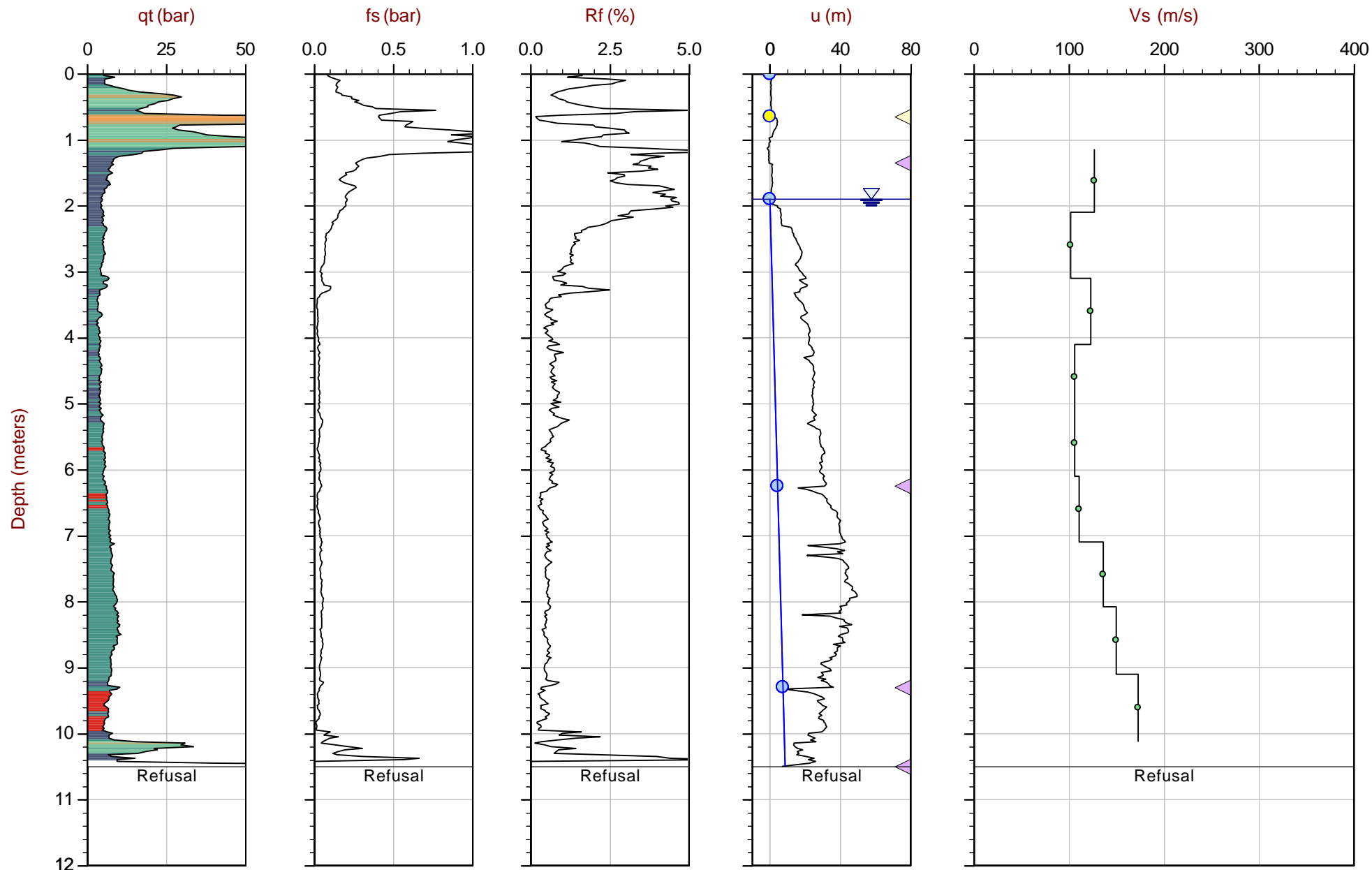
Job No: 18-05030

Date: 2018-05-15 08:36

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-09

Cone: 330:T1500F15U500



Max Depth: 10.500 m / 34.45 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP09.COR

Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 18N N: 5011401m E: 438482m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▼ Dissipation, Ueq not achieved    — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Golder*

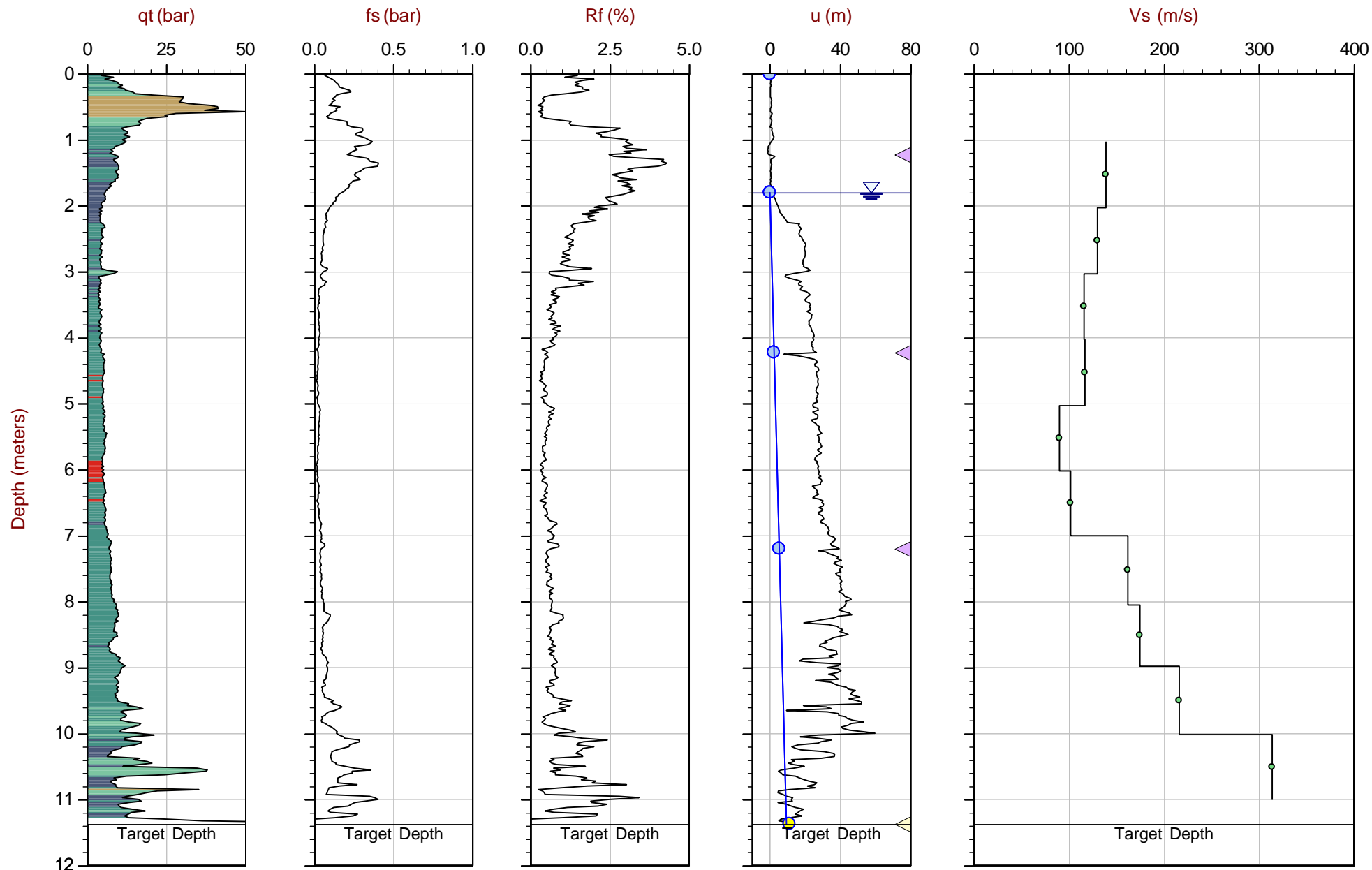
Job No: 18-05030

Date: 2018-05-15 12:01

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-10

Cone: 330:T1500F15U500



Max Depth: 11.375 m / 37.32 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-05030\_SP10.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM18N: 5011318m E: 438505m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▲ Dissipation, Ueq not achieved    — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

## Seismic Cone Penetration Wave Traces





Job No: 18-05030

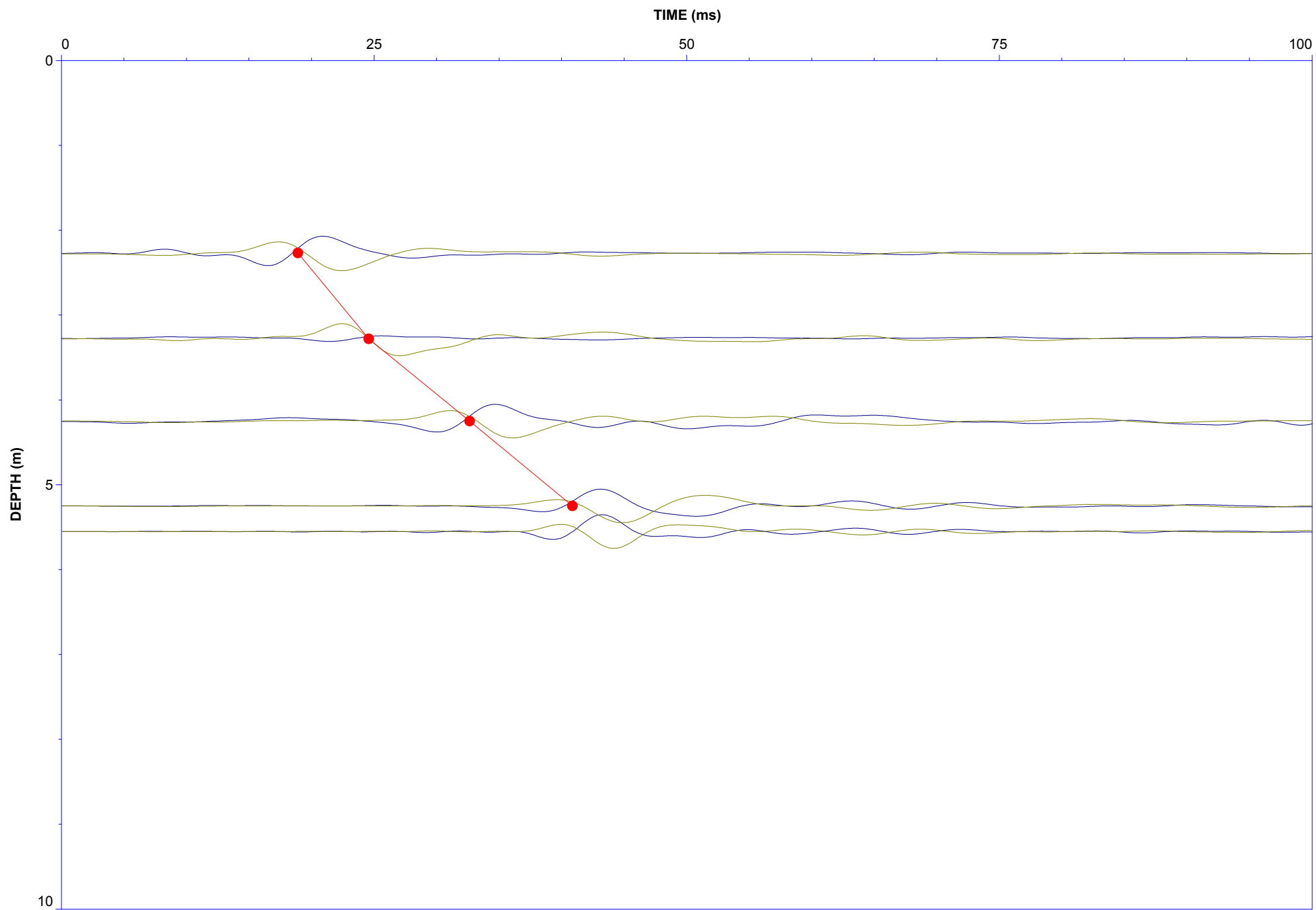
Client: Golder

Project Title: Hwy 416 & Mckenna Casey Dr

Filter: 0-200 Hz BP

Sounding ID: SCPT18-03

Date: 16-May-2018





Job No: 18-05030

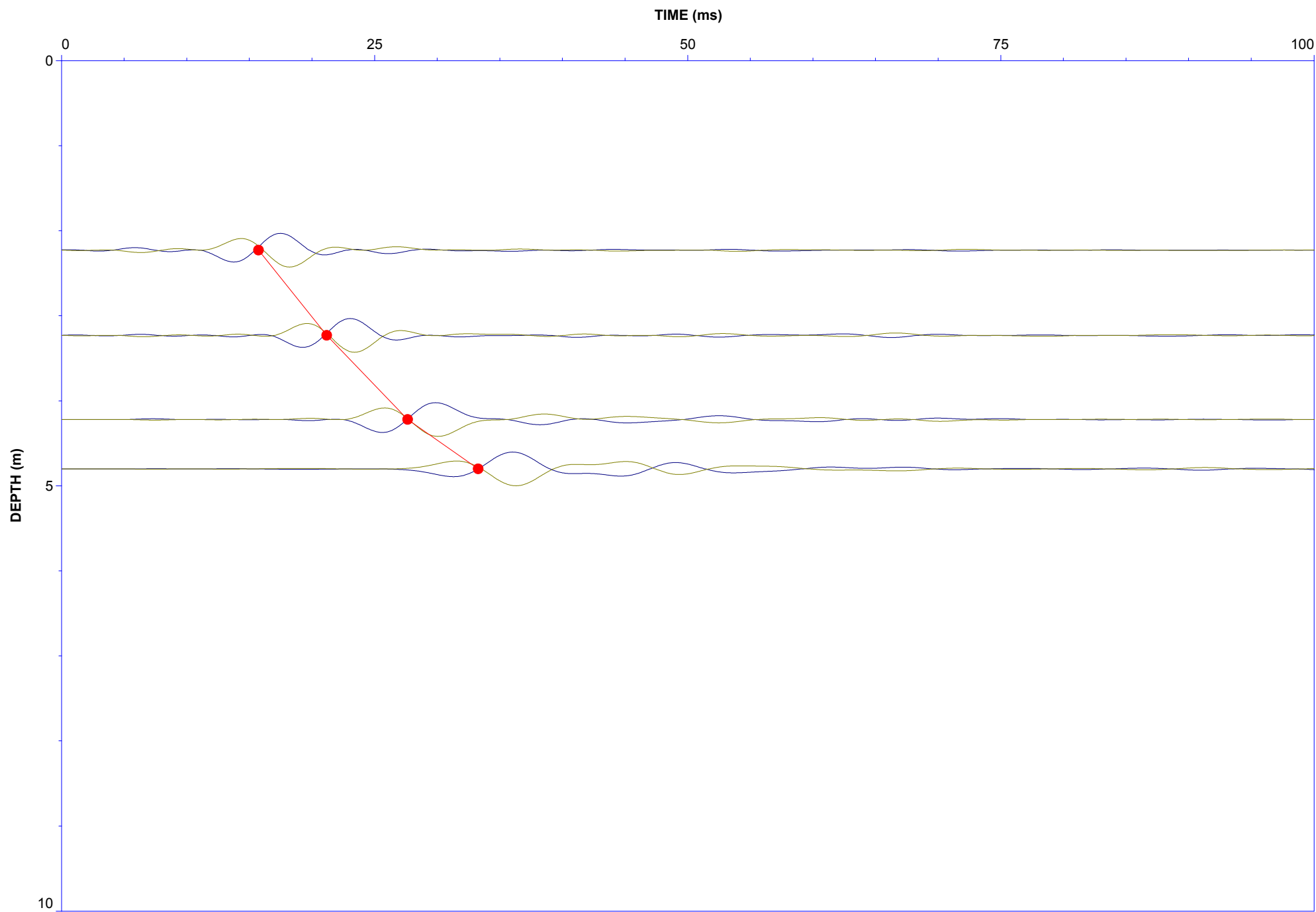
Client: Golder

Project Title: Hwy 416 & Mckenna Casey Dr

Filter: 0-200 Hz BP

Sounding ID: SCPT18-04

Date: 16-May-2018





Job No: 18-05030

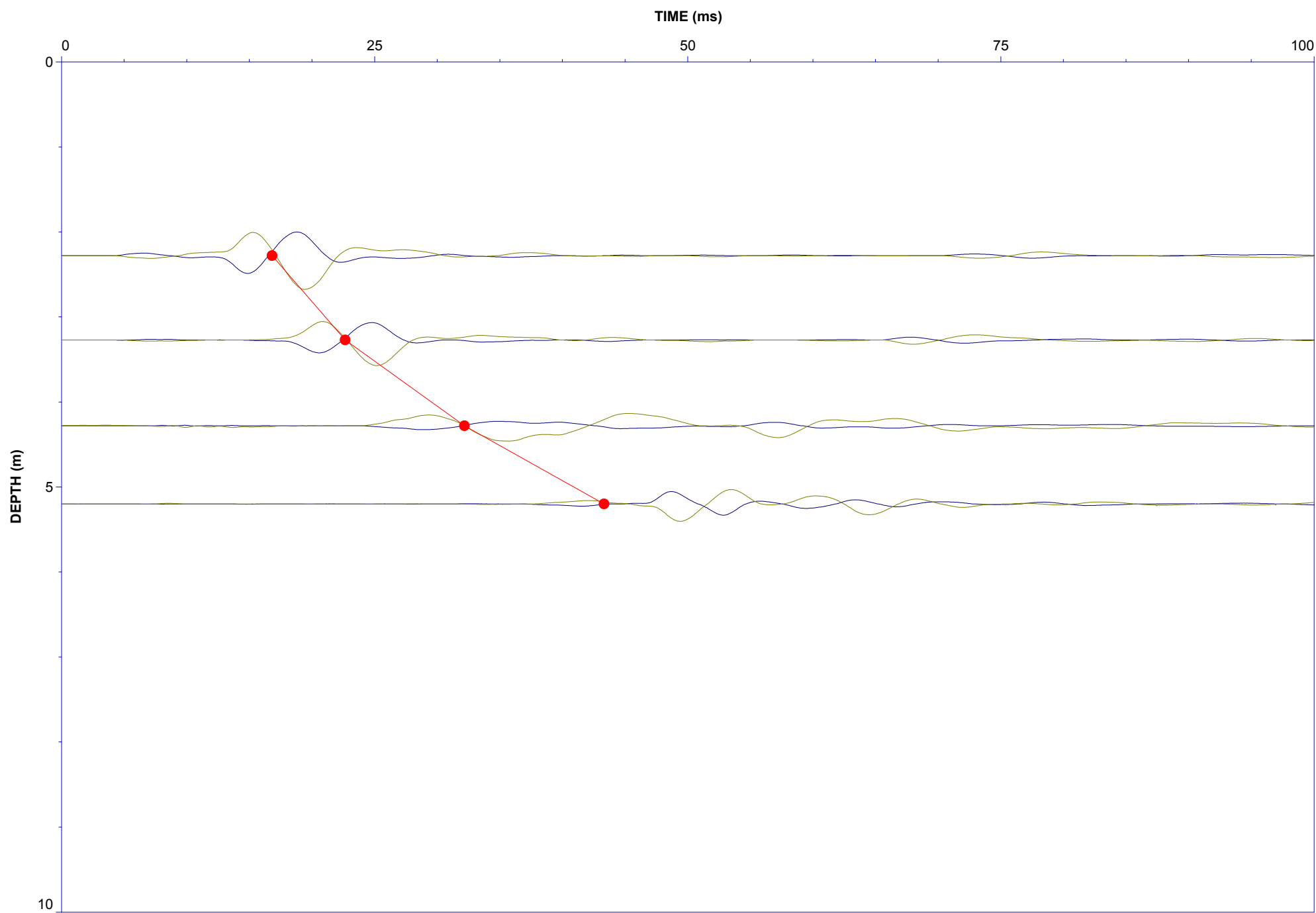
Client: Golder

Project Title: Hwy 416 & Mckenna Casey Dr

Filter: 0-200 Hz BP

Sounding ID: SCPT18-05

Date: 16-May-2018





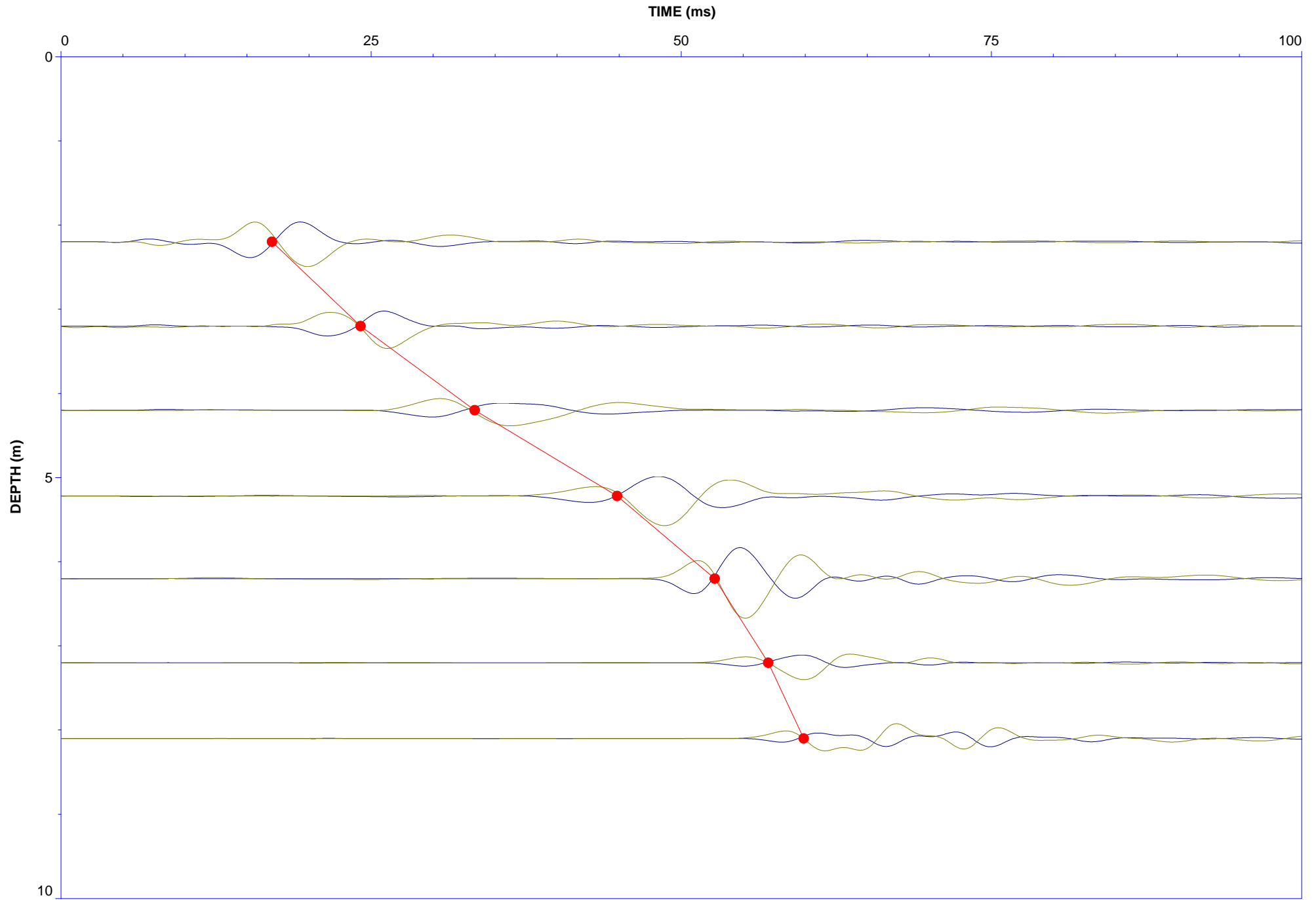
Job No: 18-05030

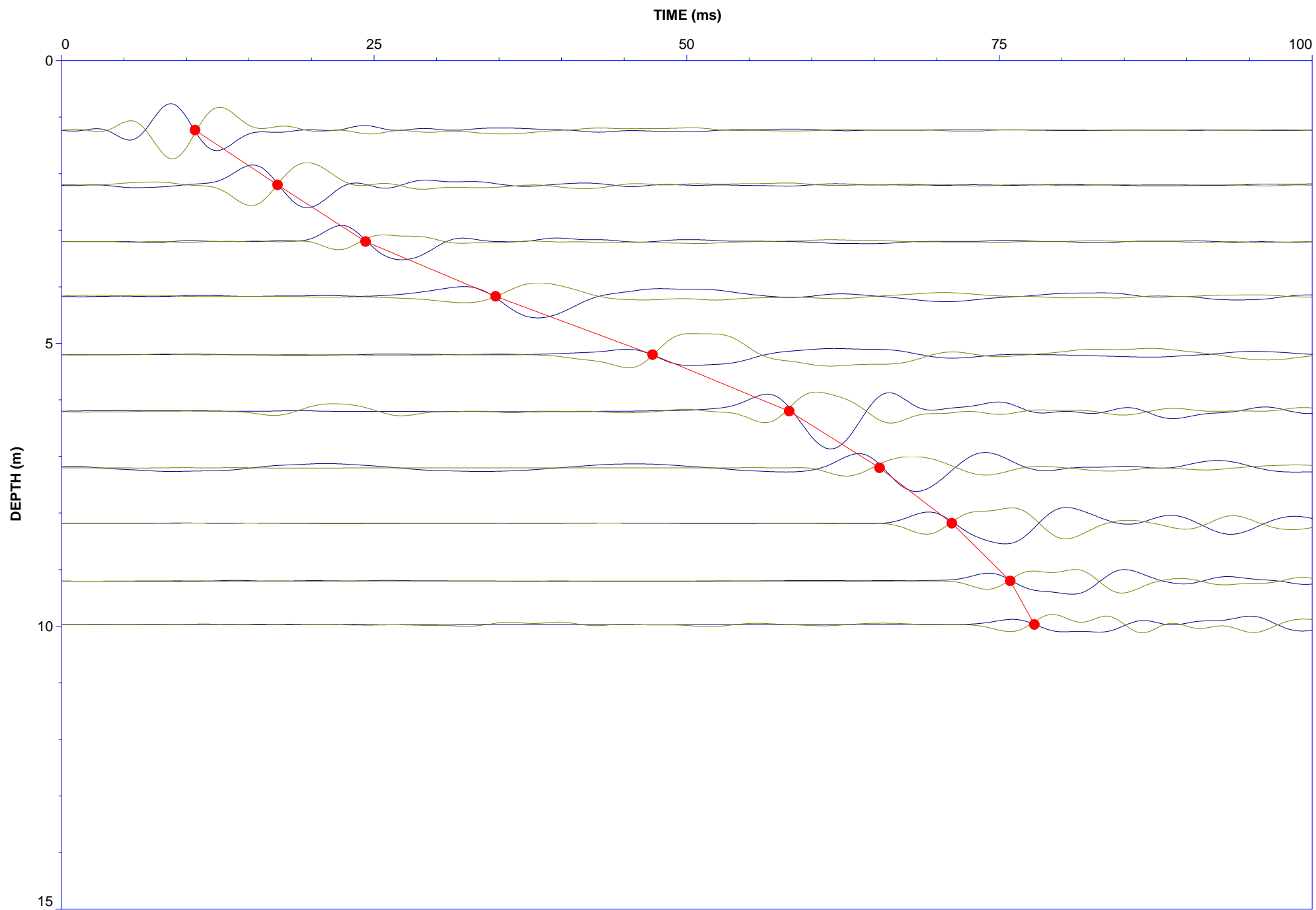
Client: Golder

Project Title: Hwy 416 and McKenna Casey Dr

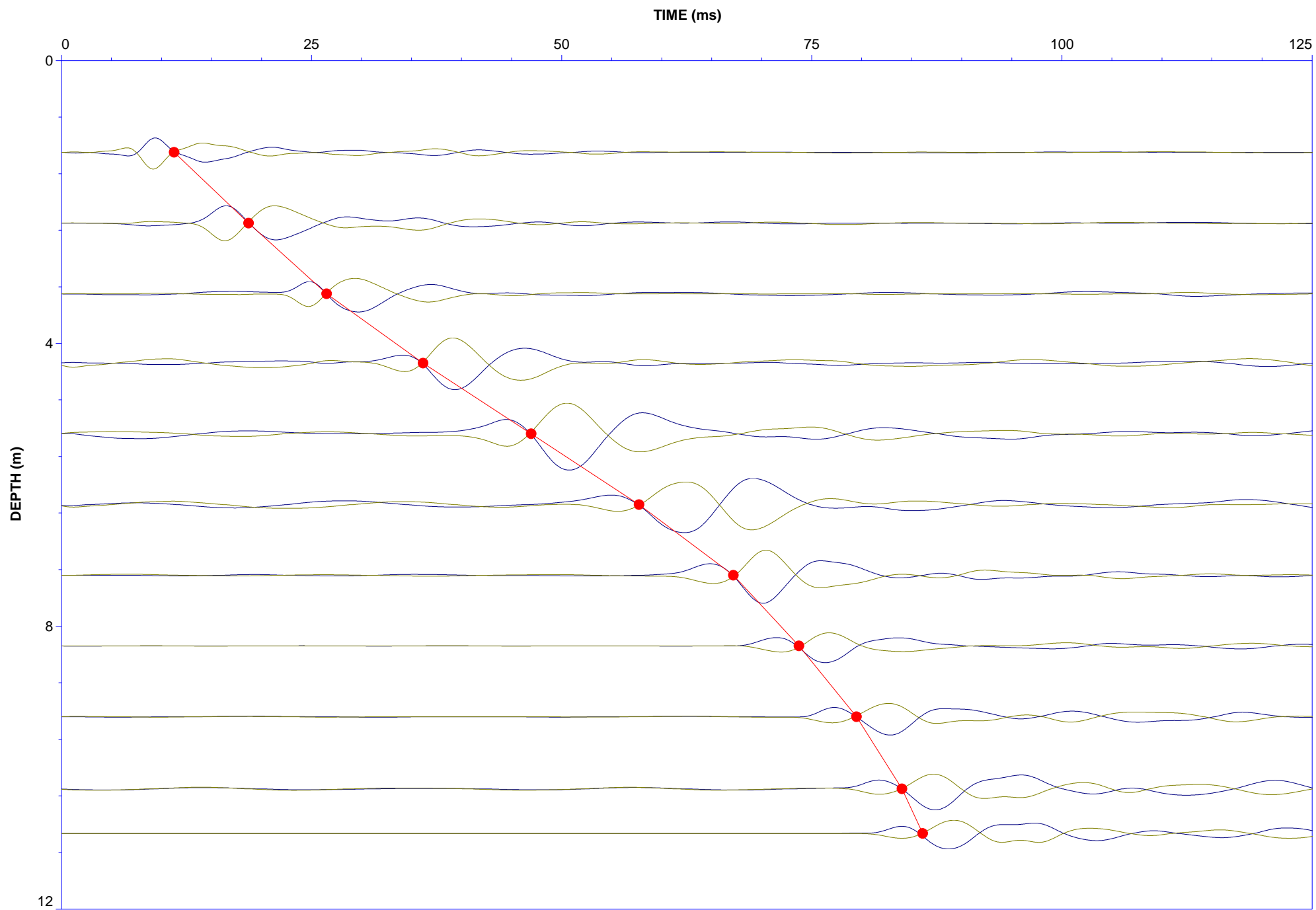
Hole: SCPT18-06

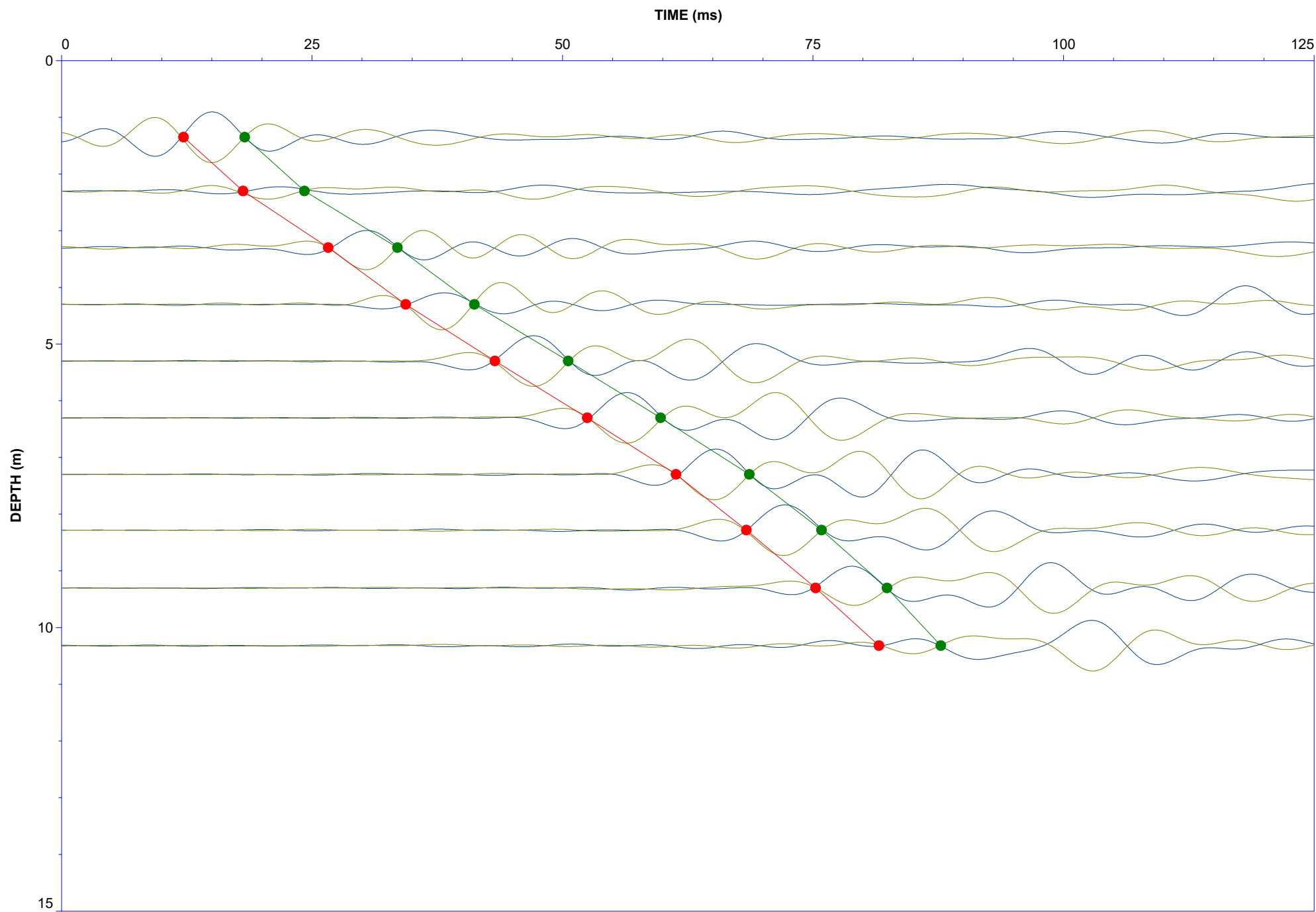
Date: 15-May-2018

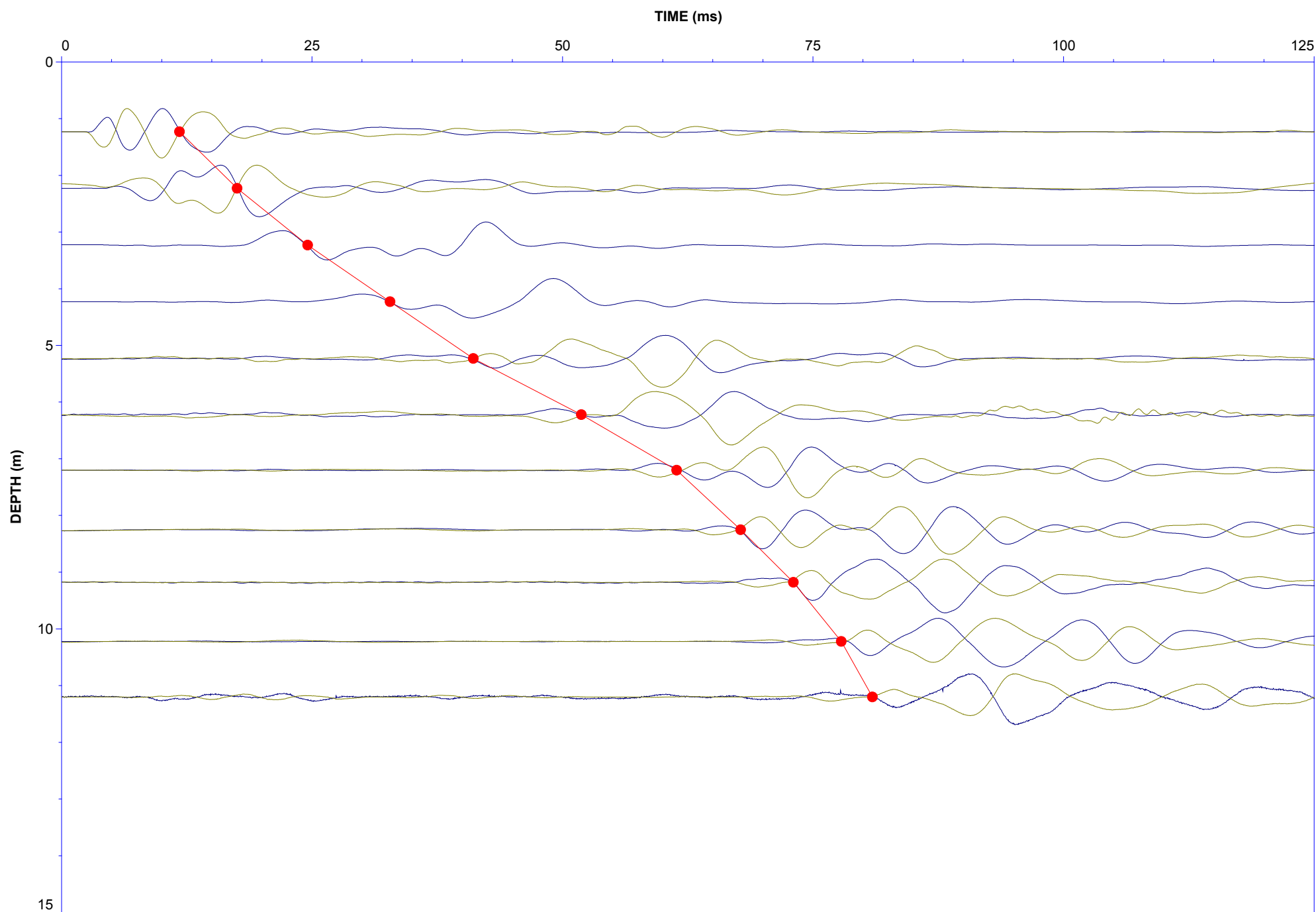












## Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



Job No: 18-05030  
 Client: Golder Associates  
 Project: Hwy 416 and McKenna Casey Dr  
 Start Date: 14-May-2018  
 End Date: 16-May-2018

### CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm <sup>2</sup> )	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U <sub>eq</sub> (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)	t <sub>50</sub> <sup>a</sup> (s)	Assumed Rigidity Index (I <sub>r</sub> )	c <sub>h</sub> <sup>b</sup> (cm <sup>2</sup> /min)
SCPT18-03	18-05030_SP03	15	900	3.275	Not achieved		1.6	182	100	3.9
SCPT18-03	18-05030_SP03	15	1200	5.550	Not achieved		1.6	298	100	2.4
SCPT18-04	18-05030_SP04	15	900	4.225	Not achieved		1.6	301	100	2.3
SCPT18-04	18-05030_SP04	15	900	4.775	Not achieved		1.6	524	100	1.3
SCPT18-05	18-05030_SP05	15	900	5.200	Not achieved		1.6	535	100	1.3
SCPT18-05	18-05030_SP05	15	400	5.550	Not achieved					
SCPT18-06	18-05030_SP06	15	120	0.650	0.0					
SCPT18-06	18-05030_SP06	15	1550	6.200	Not achieved					
SCPT18-06	18-05030_SP06	15	200	8.100	7.9					
SCPT18-07	18-05030_SP07	15	900	4.175	Not achieved		1.8	578	100	1.2
SCPT18-07	18-05030_SP07	15	400	9.975	Not achieved					
SCPT18-08	18-05030_SP08	15	900	8.275	Not achieved		1.9	114	100	6.2
SCPT18-08	18-05030_SP08	15	600	10.925	Not achieved		1.9	45	100	15.7
SCPT18-09	18-05030_SP09	15	245	0.650	0.0					
SCPT18-09	18-05030_SP09	15	110	1.350	Not achieved					
SCPT18-09	18-05030_SP09	15	900	6.250	Not achieved		1.9	373	100	1.9
SCPT18-09	18-05030_SP09	15	95	9.300	Not achieved		1.9	54	100	12.9
SCPT18-09	18-05030_SP09	15	350	10.500	Not achieved					
SCPT18-10	18-05030_SP10	15	115	1.225	Not achieved					
SCPT18-10	18-05030_SP10	15	900	4.225	Not achieved		1.8	452	100	1.6
SCPT18-10	18-05030_SP10	15	900	7.200	Not achieved		1.8	789	100	0.9
SCPT18-10	18-05030_SP10	15	200	11.375	11.2					

a. Time is relative to where umax occurred

b. Houlsby and Teh, 1991



*Golder*

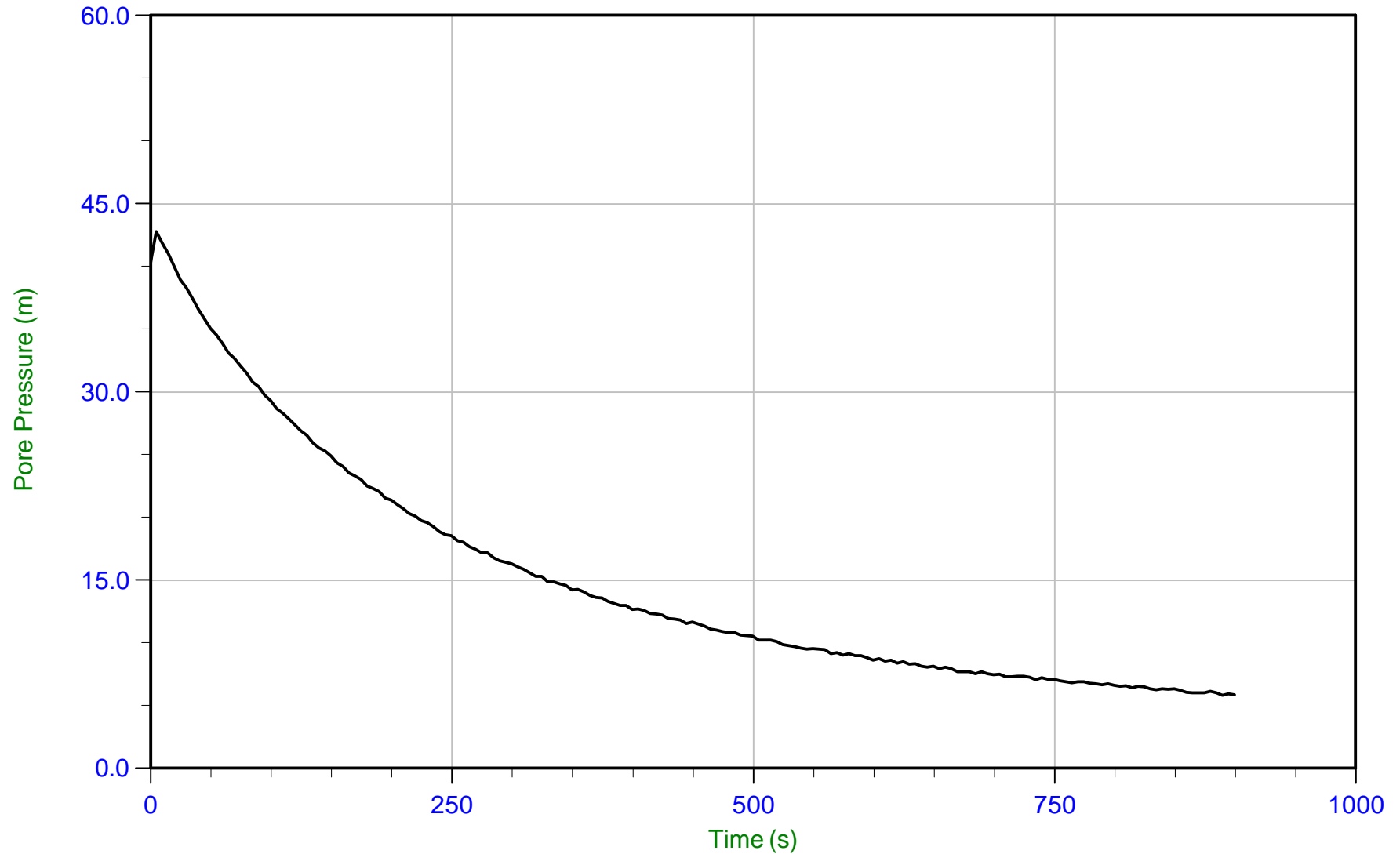
Job No: 18-05030

Date: 05/16/2018 11:08

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-03

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP03.PPF  
Depth: 3.275 m / 10.745 ft  
Duration: 900.0 s

U Min: 5.8 m  
U Max: 42.8 m

WT: 1.600 m / 5.249 ft  
Ueq: 1.7 m  
U(50): 22.22 m

T(50): 181.8 s  
Ir: 100  
Ch: 3.9 cm<sup>2</sup>/min



*Golder*

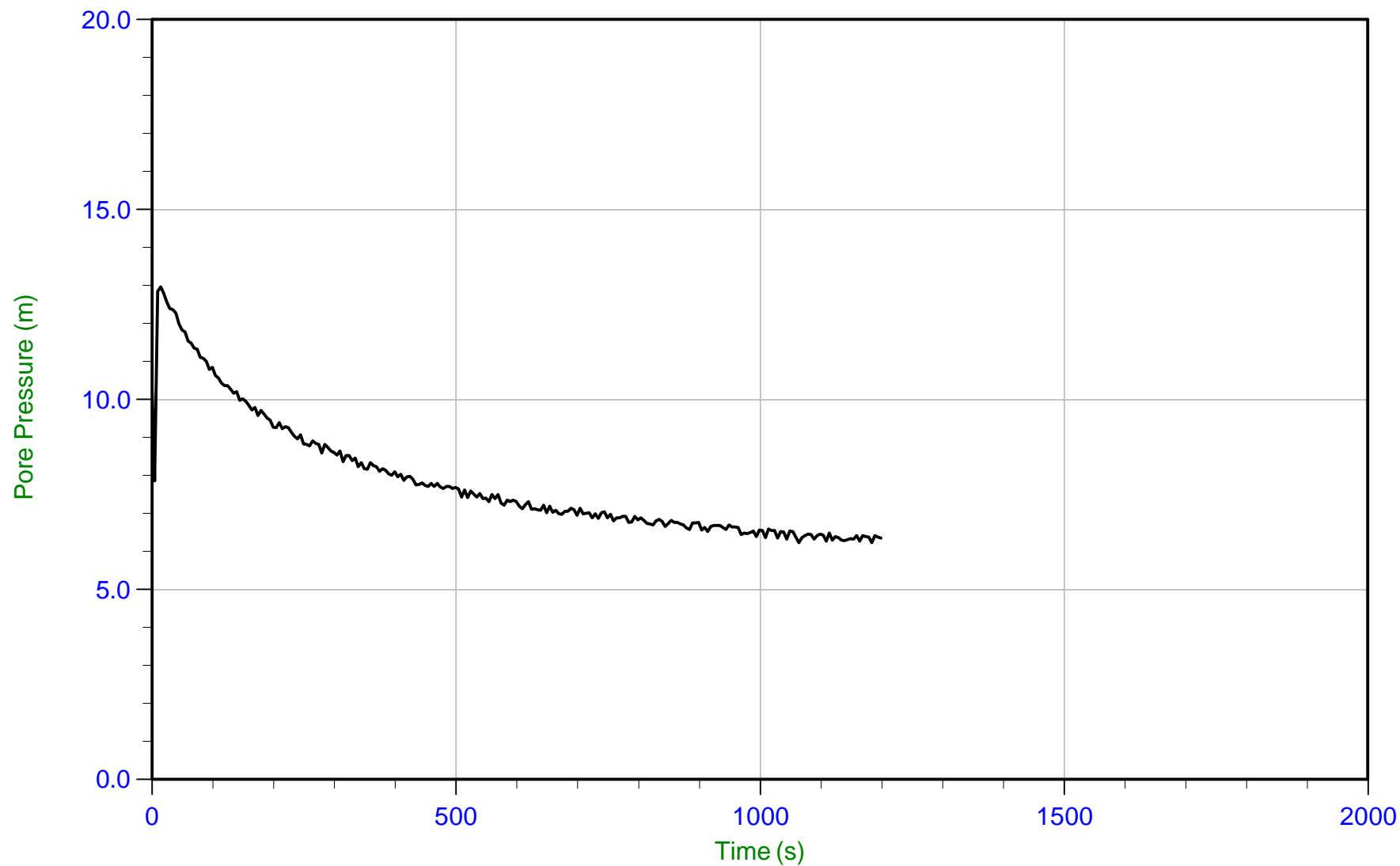
Job No: 18-05030

Date: 05/16/2018 11:08

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-03

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 18-05030\_SP03.PPF  
Depth: 5.550 m / 18.208 ft  
Duration: 1200.0 s

U Min: 6.2 m  
U Max: 13.0 m

WT: 1.600 m / 5.249 ft  
Ueq: 4.0 m  
U(50): 8.46 m

T(50): 298.4 s  
Ir: 100  
Ch: 2.4 cm<sup>2</sup>/min



*Golder*

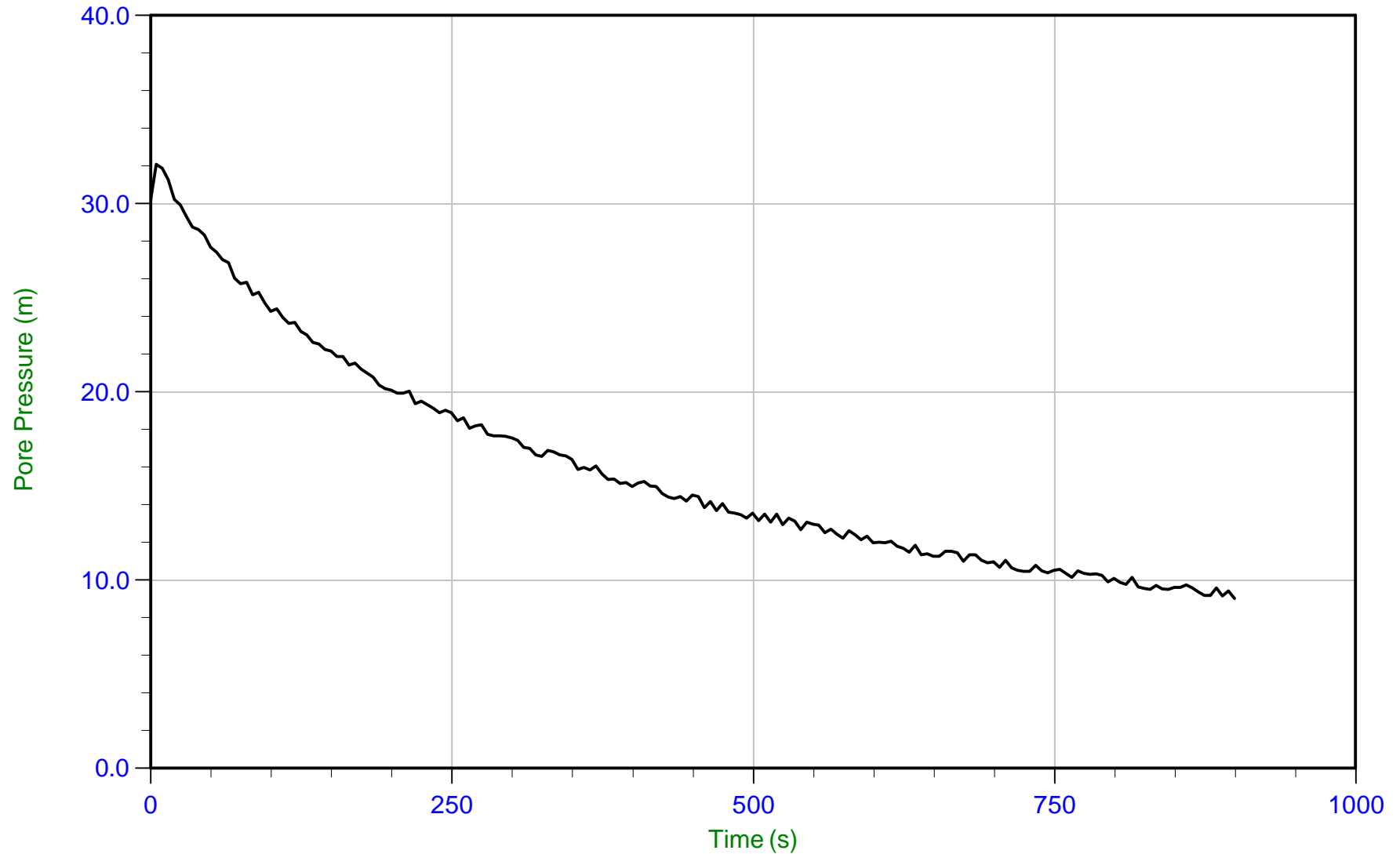
Job No: 18-05030

Date: 05/16/2018 09:32

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-04

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP04.PPF  
Depth: 4.225 m / 13.861 ft  
Duration: 900.0 s

U Min: 9.0 m  
U Max: 32.1 m

WT: 1.600 m / 5.249 ft  
Ueq: 2.6 m  
U(50): 17.36 m

T(50): 300.9 s  
Ir: 100  
Ch: 2.3 cm<sup>2</sup>/min





*Golder*

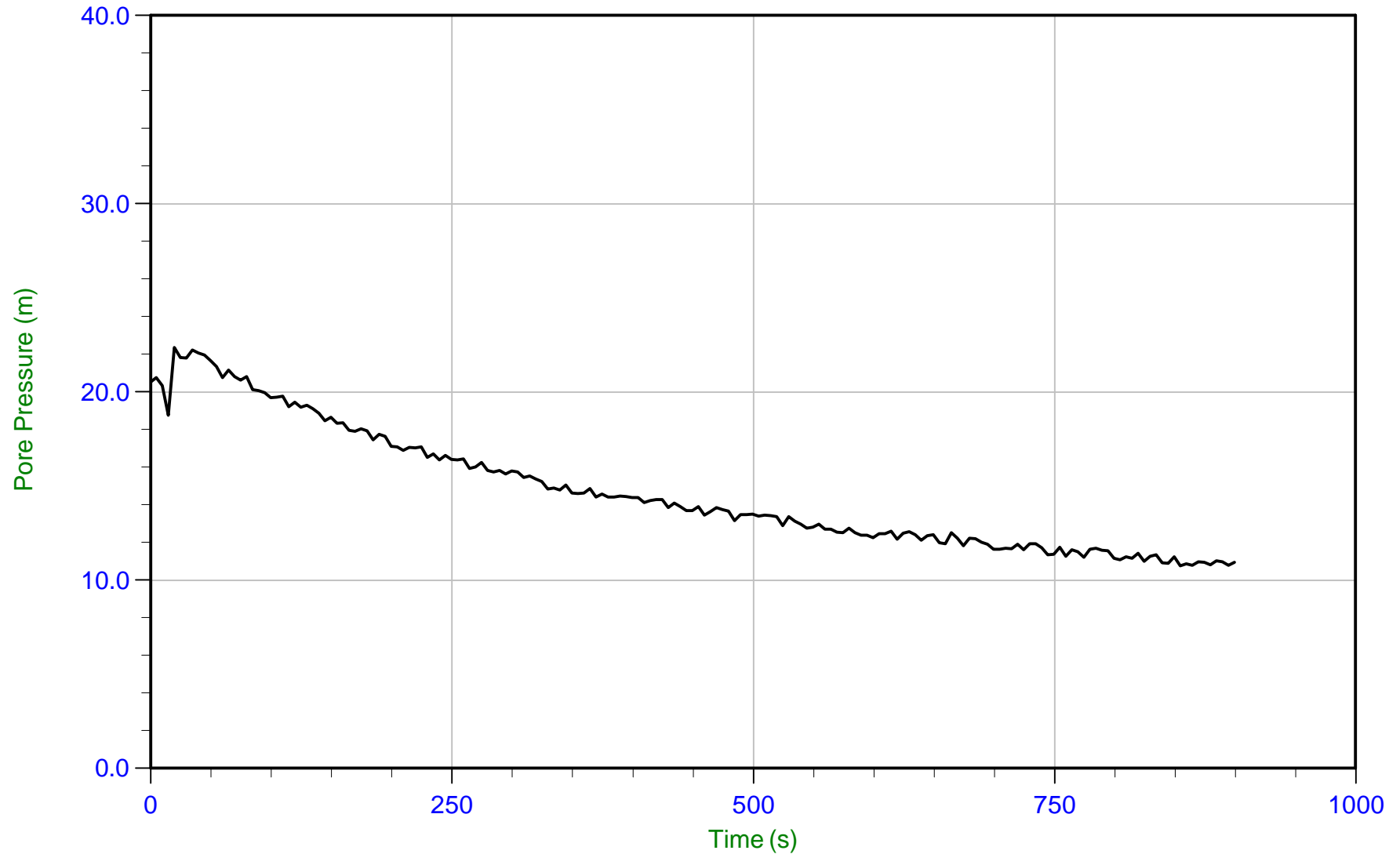
Job No: 18-05030

Date: 05/16/2018 09:32

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-04

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP04.PPF  
Depth: 4.775 m / 15.666 ft  
Duration: 900.0 s

U Min: 10.8 m  
U Max: 22.4 m

WT: 1.600 m / 5.249 ft  
Ueq: 3.2 m  
U(50): 12.77 m

T(50): 524.5 s  
Ir: 100  
Ch: 1.3 cm<sup>2</sup>/min



*Golder*

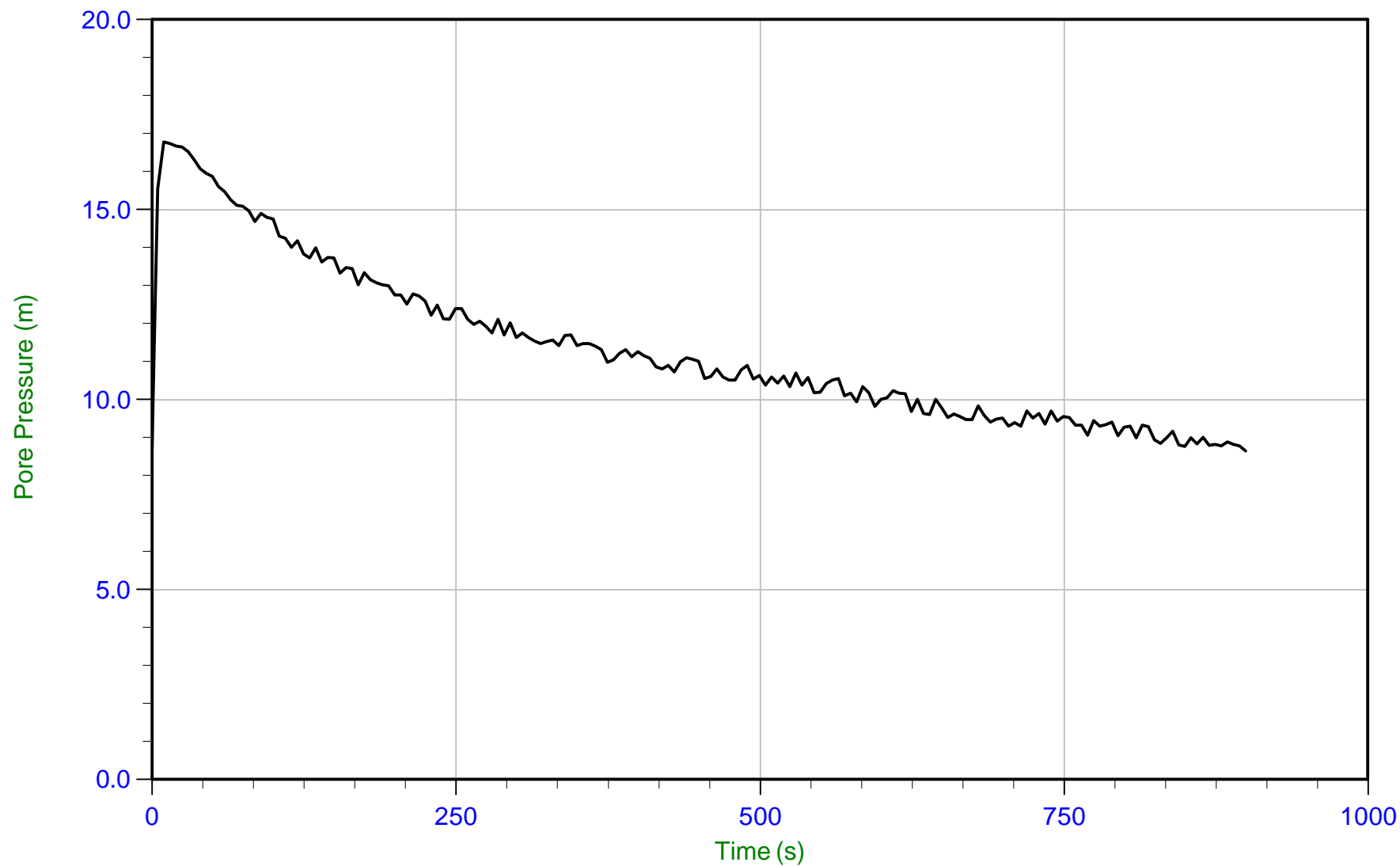
Job No: 18-05030

Date: 05/16/2018 08:22

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-05

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 18-05030\_SP05.PPF  
Depth: 5.200 m / 17.060 ft  
Duration: 900.0 s

U Min: 8.6 m  
U Max: 16.8 m

WT: 1.600 m / 5.249 ft  
Ueq: 3.6 m  
U(50): 10.19 m

T(50): 534.9 s  
Ir: 100  
Ch: 1.3 cm<sup>2</sup>/min



*Golder*

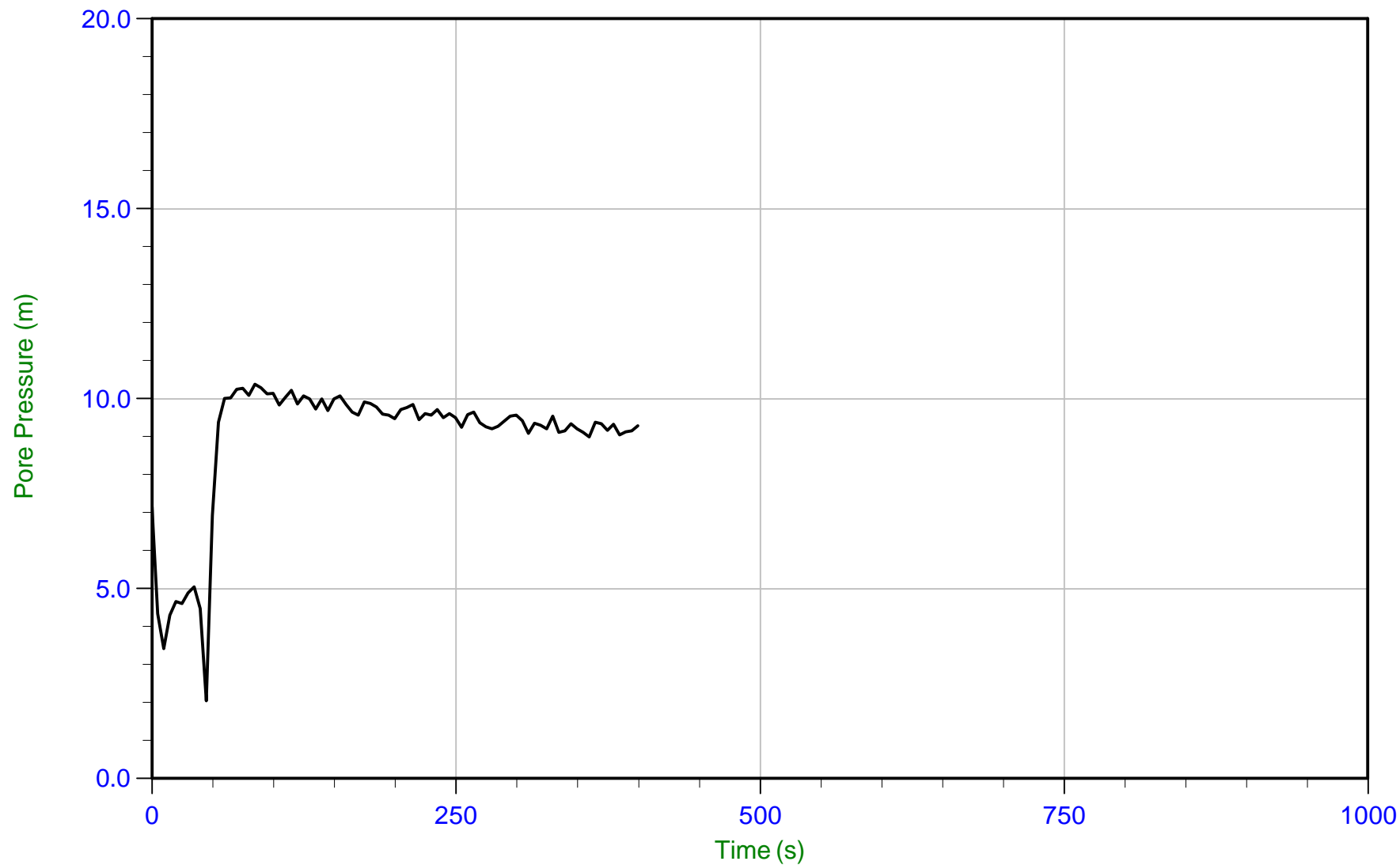
Job No: 18-05030

Date: 05/16/2018 08:22

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-05

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP05.PPF  
Depth: 5.550 m / 18.208 ft  
Duration: 400.0 s

U Min: 2.0 m  
U Max: 10.4 m



*Golder*

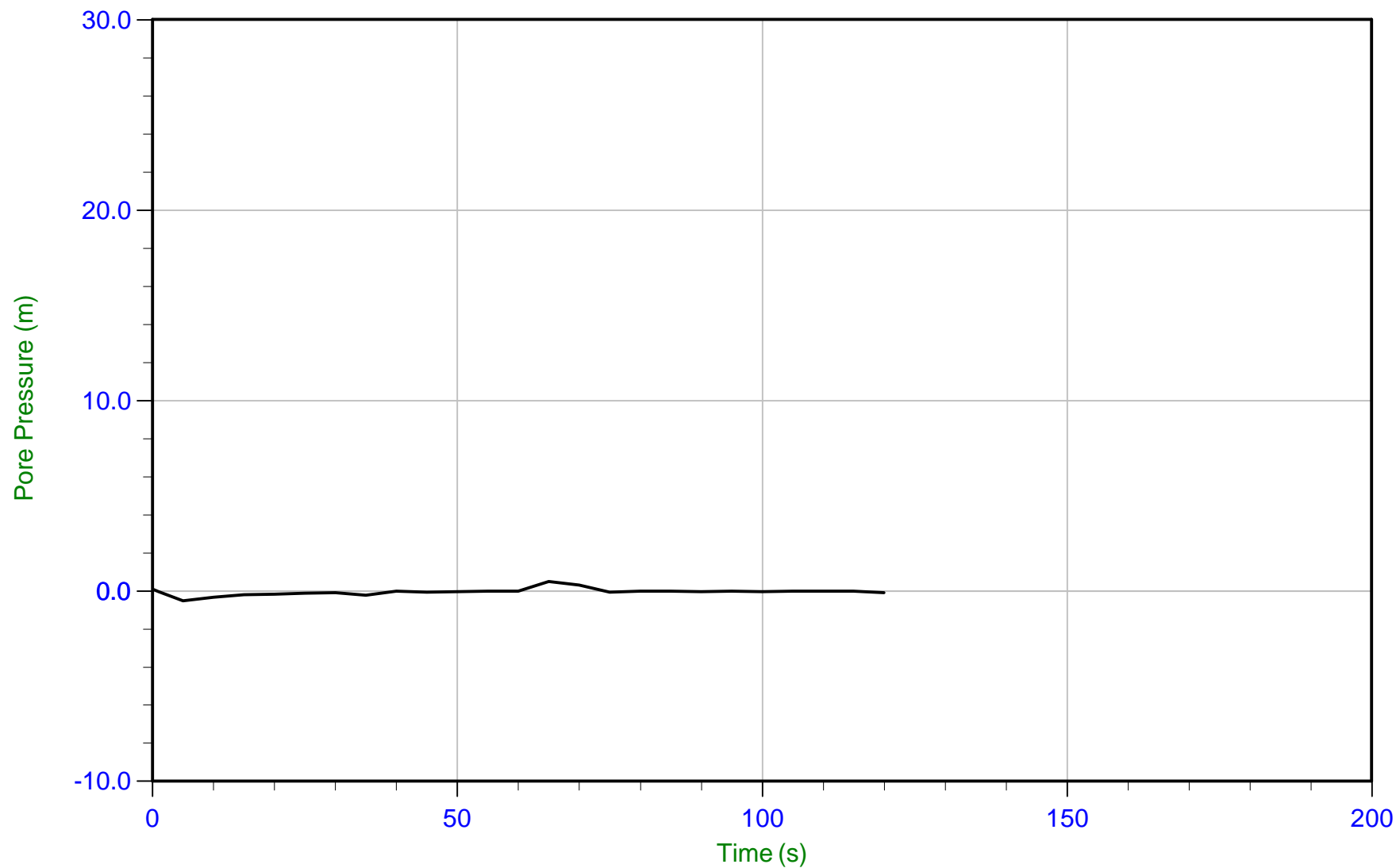
Job No: 18-05030

Date: 05/15/2018 14:58

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-06

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 18-05030\_SP06.PPF

Depth: 0.650 m / 2.133 ft

Duration: 120.0 s

U Min: -0.5 m

U Max: 0.5 m

WT: 0.650 m / 2.133 ft

Ueq: 0.0 m



*Golder*

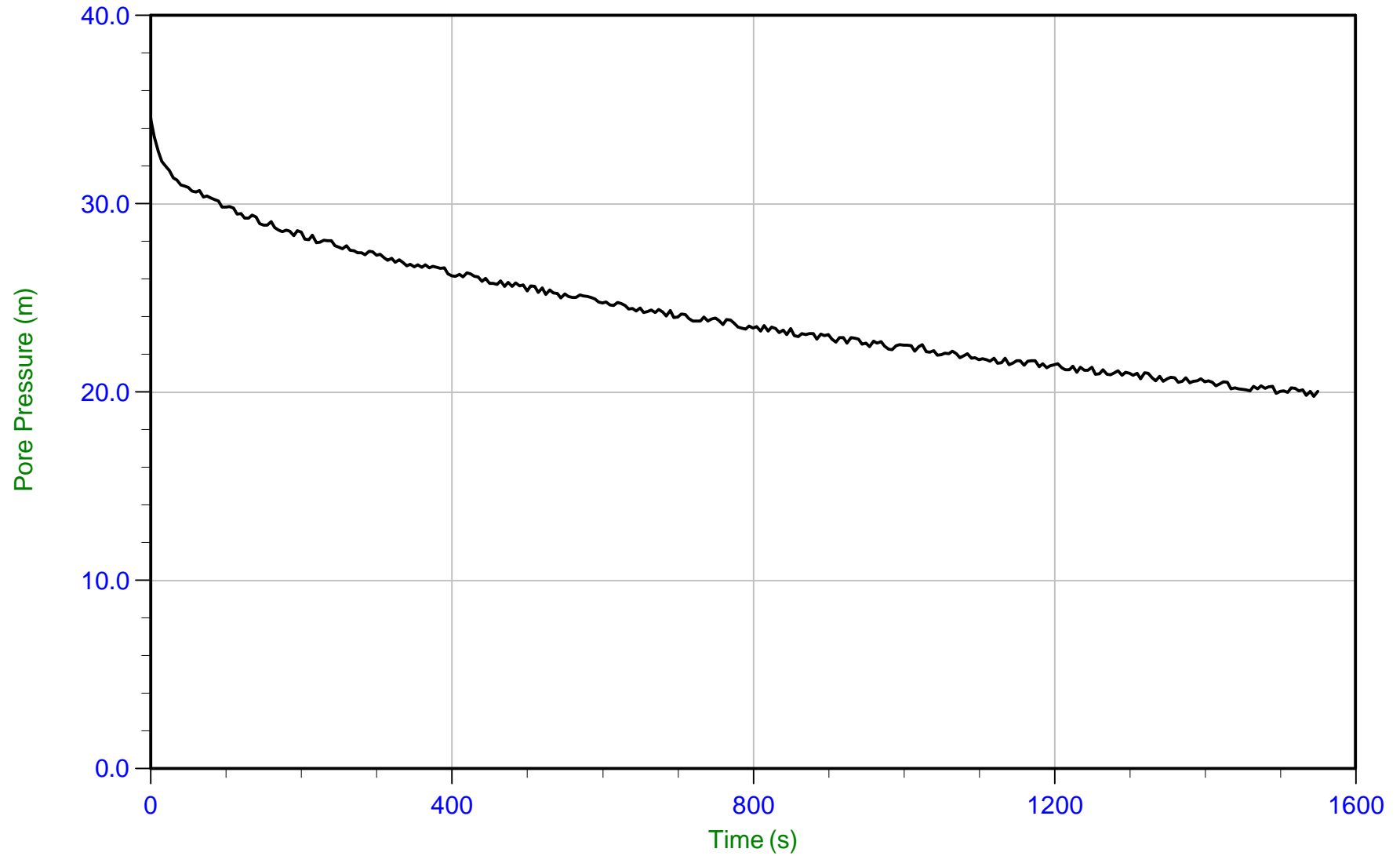
Job No: 18-05030

Date: 05/15/2018 14:58

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-06

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP06.PPF U Min: 19.8 m  
Depth: 6.200 m / 20.341 ft U Max: 34.6 m  
Duration: 1550.0 s



*Golder*

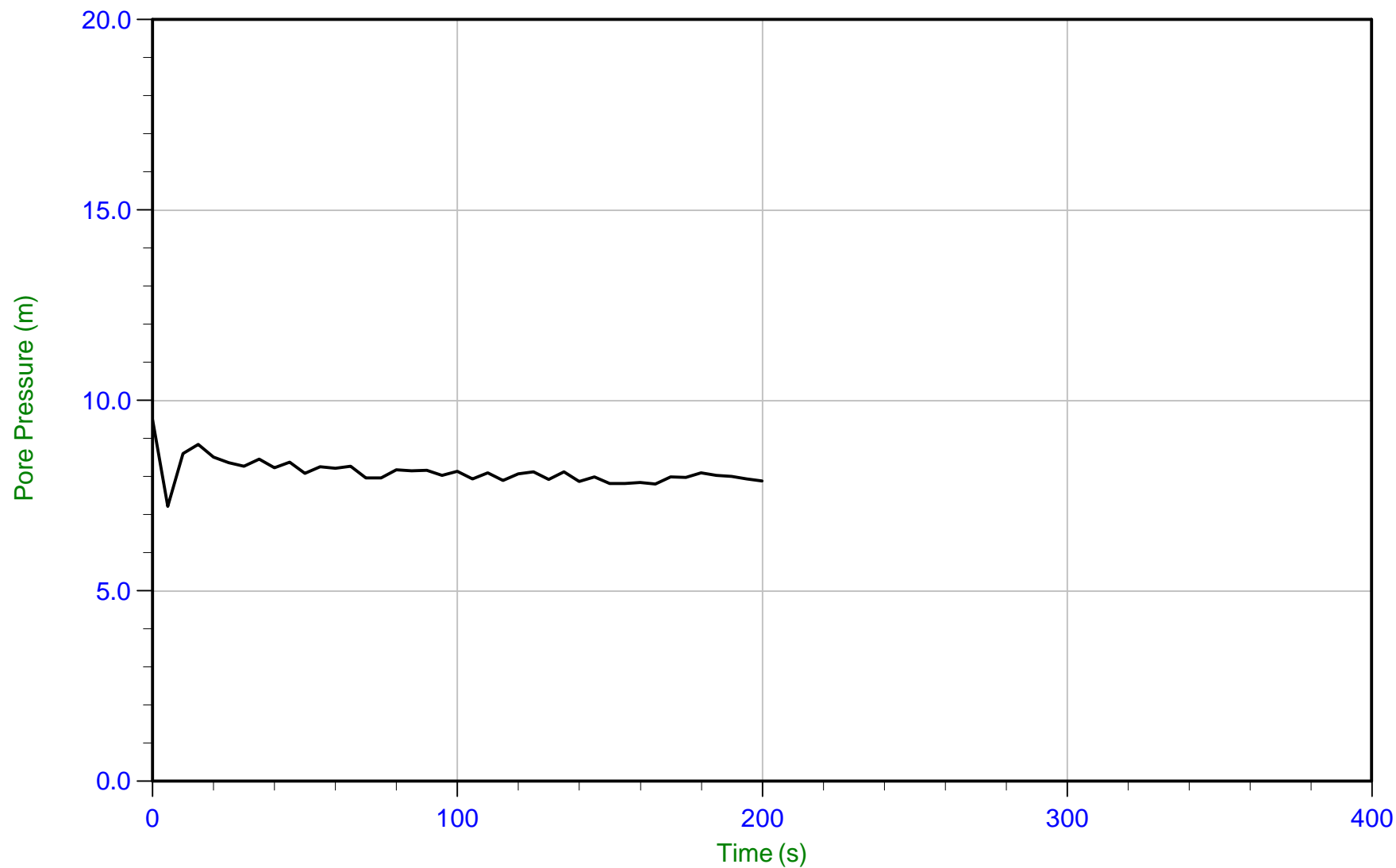
Job No: 18-05030

Date: 05/15/2018 14:58

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-06

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP06.PPF U Min: 7.2 m WT: 0.219 m / 0.718 ft  
Depth: 8.100 m / 26.574 ft U Max: 9.5 m Ueq: 7.9 m  
Duration: 200.0 s



*Golder*

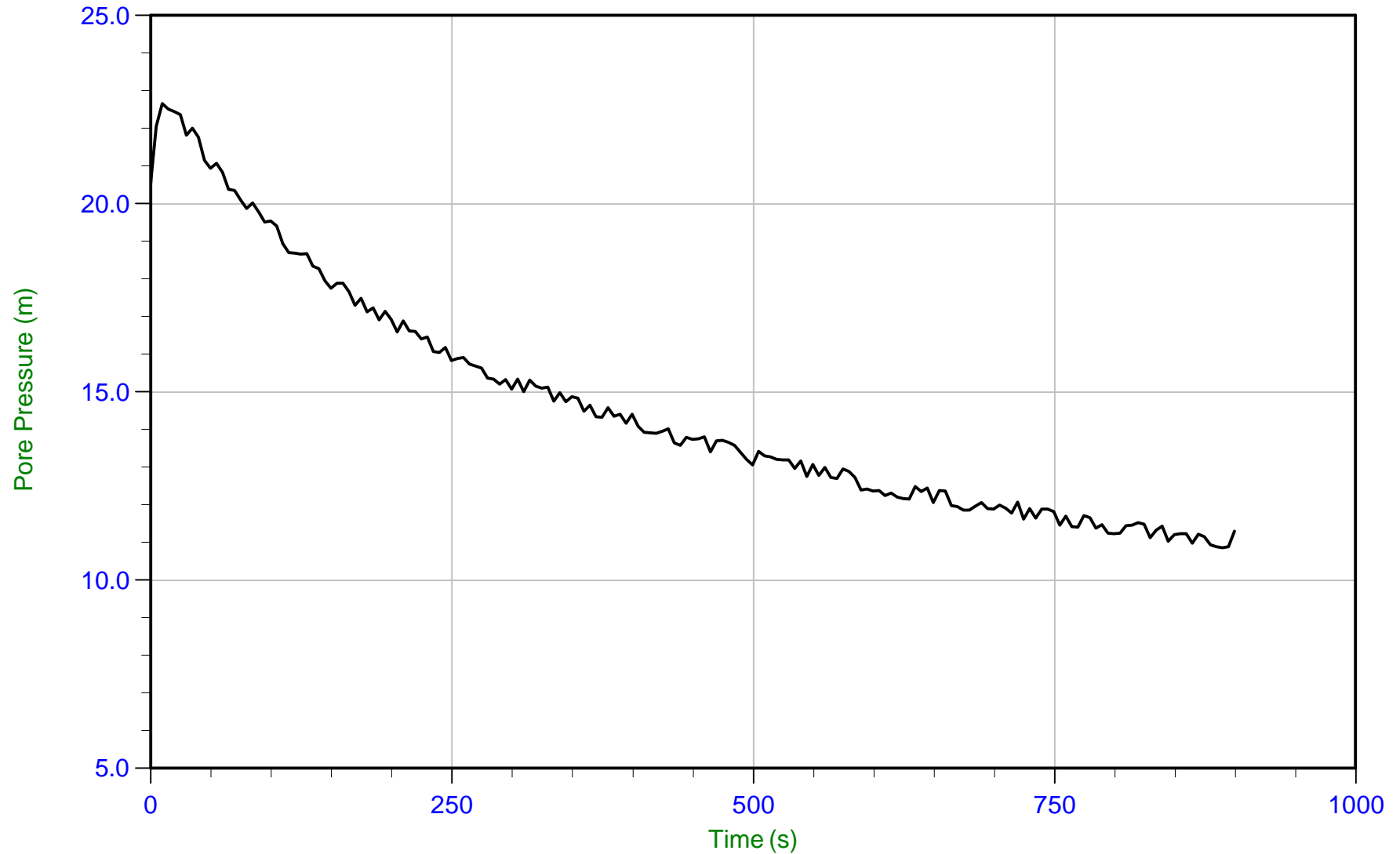
Job No: 18-05030

Date: 05/14/2018 14:54

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-07

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP07.PPF  
Depth: 4.175 m / 13.697 ft  
Duration: 900.0 s

U Min: 10.9 m  
U Max: 22.7 m

WT: 1.800 m / 5.905 ft  
Ueq: 2.4 m  
U(50): 12.52 m

T(50): 578.1 s  
Ir: 100  
Ch: 1.2 cm<sup>2</sup>/min



*Golder*

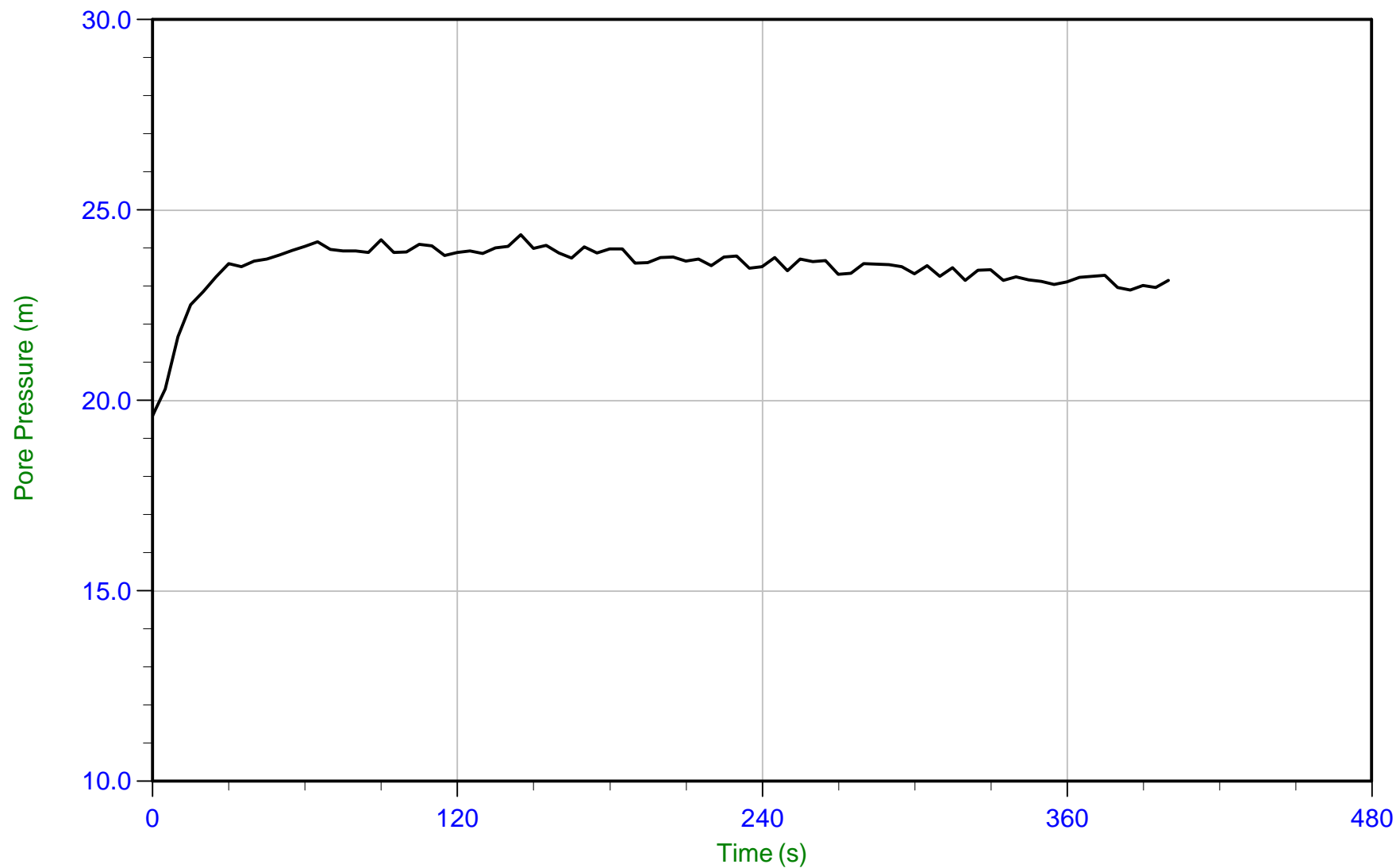
Job No: 18-05030

Date: 05/14/2018 14:54

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-07

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP07.PPF  
Depth: 9.975 m / 32.726 ft  
Duration: 400.0 s

U Min: 19.6 m  
U Max: 24.4 m





*Golder*

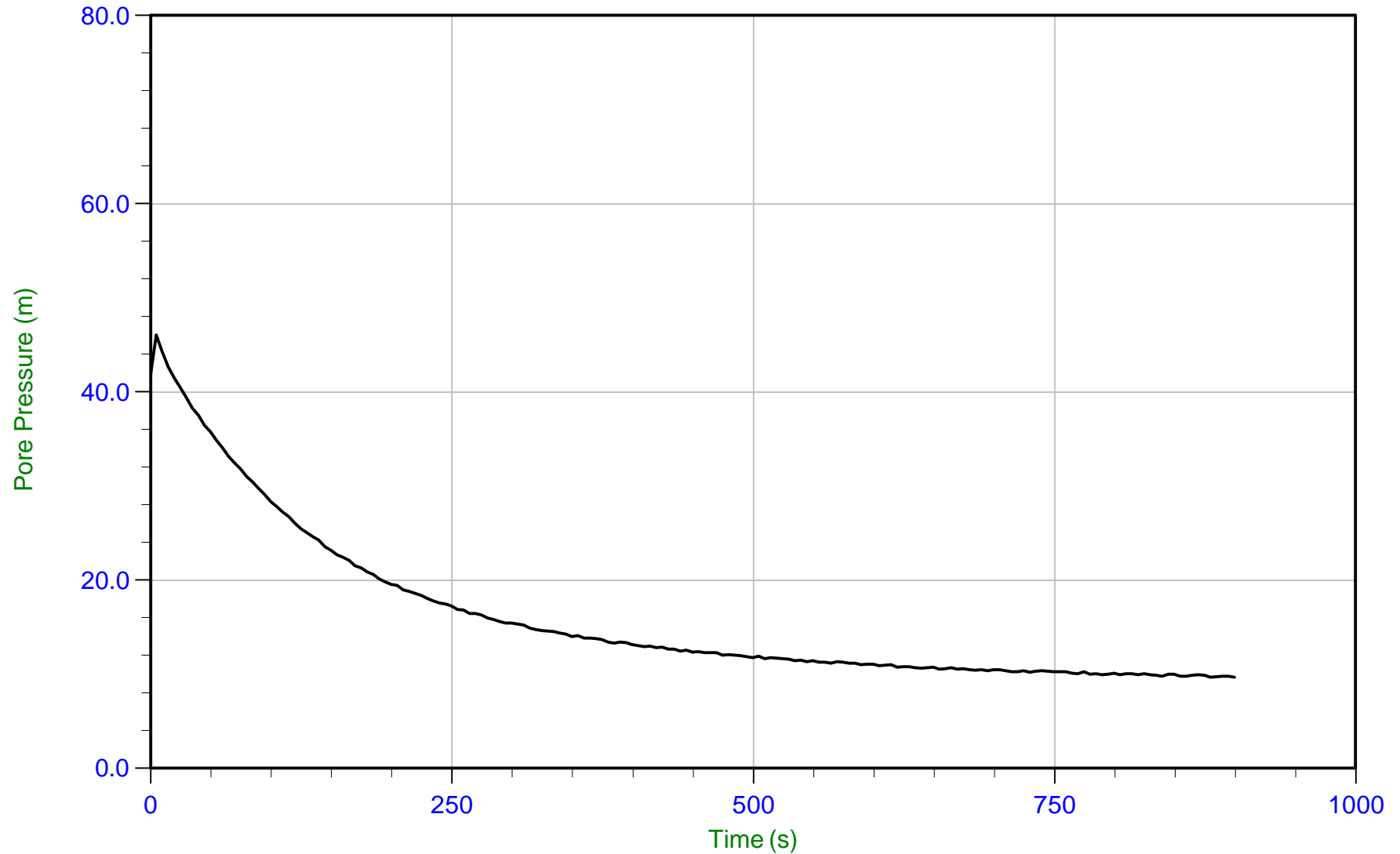
Job No: 18-05030

Date: 05/15/2018 10:18

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-08

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP08.PPF  
Depth: 8.275 m / 27.149 ft  
Duration: 900.0 s

U Min: 9.7 m  
U Max: 46.1 m

WT: 1.900 m / 6.234 ft  
Ueq: 6.4 m  
U(50): 26.23 m

T(50): 113.8 s  
Ir: 100  
Ch: 6.2 cm<sup>2</sup>/min



*Golder*

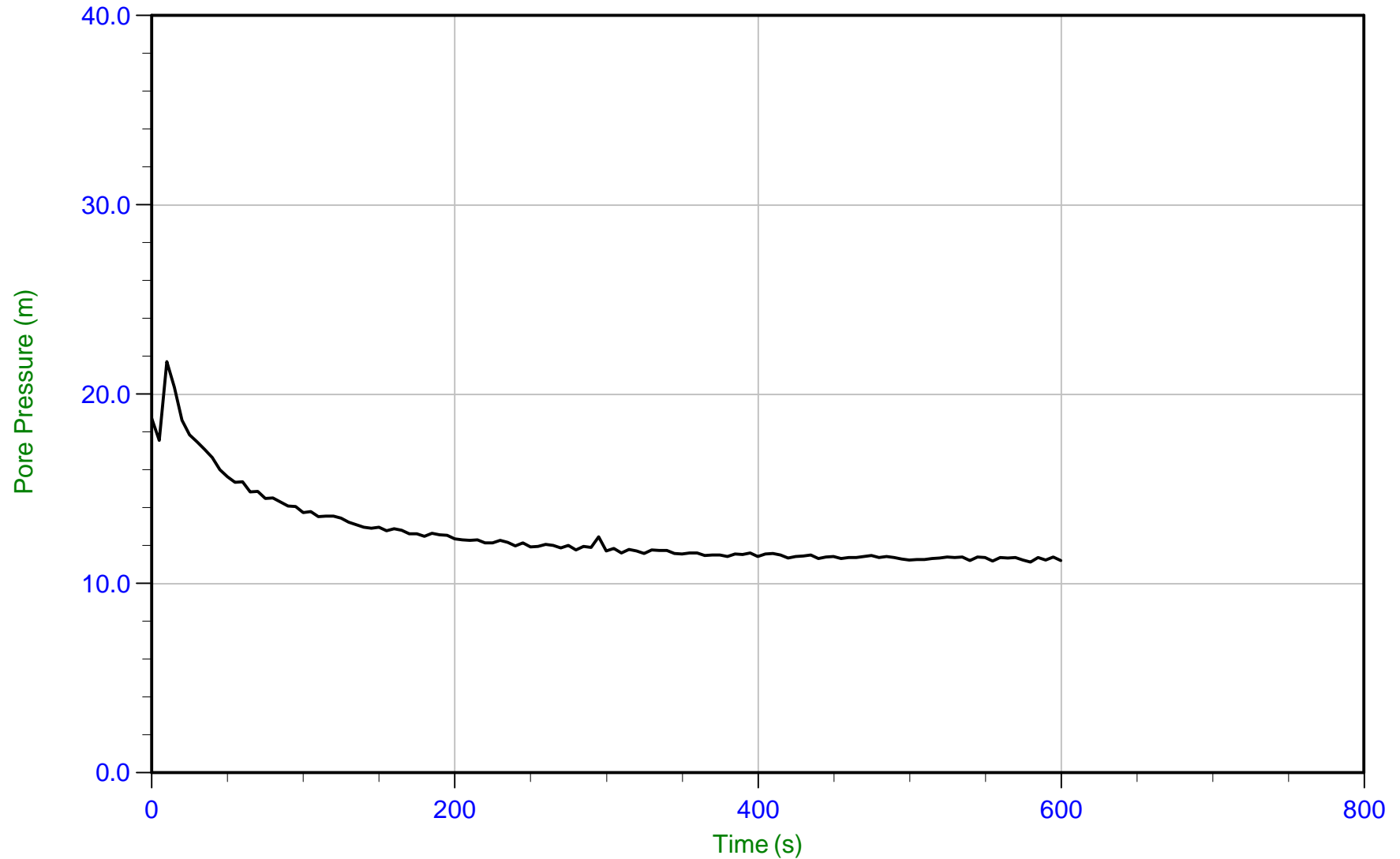
Job No: 18-05030

Date: 05/15/2018 10:18

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-08

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP08.PPF  
Depth: 10.925 m / 35.843 ft  
Duration: 600.0 s

U Min: 11.1 m  
U Max: 21.7 m

WT: 1.900 m / 6.234 ft  
Ueq: 9.0 m  
U(50): 15.37 m

T(50): 44.6 s  
Ir: 100  
Ch: 15.7 cm<sup>2</sup>/min



*Golder*

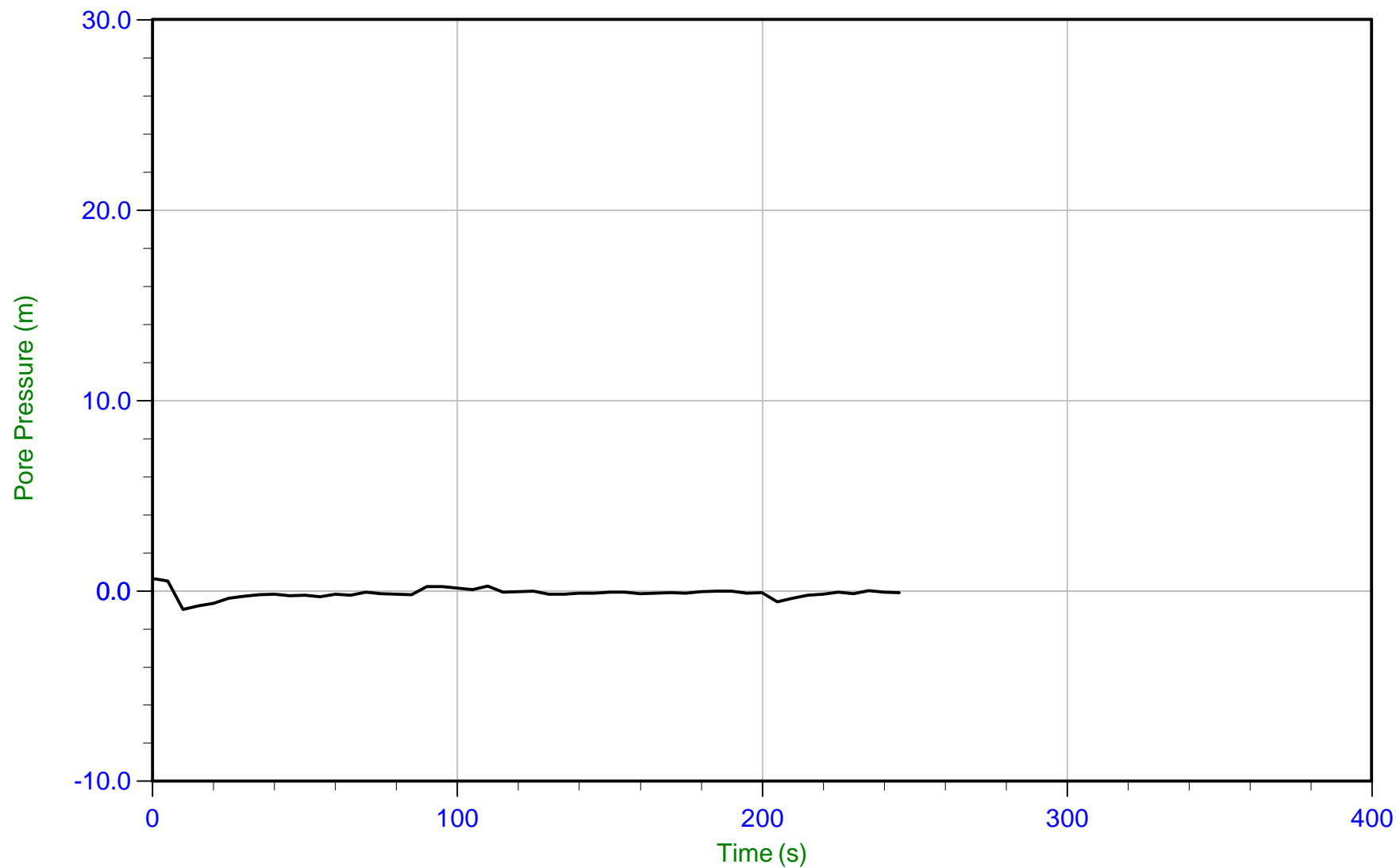
Job No: 18-05030

Date: 05/15/2018 08:36

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-09

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:	Filename: 18-05030_SP09.PPF	U Min: -1.0 m	WT: 0.650 m / 2.133 ft
	Depth: 0.650 m / 2.133 ft	U Max: 0.6 m	Ueq: 0.0 m
	Duration: 245.0 s		



*Golder*

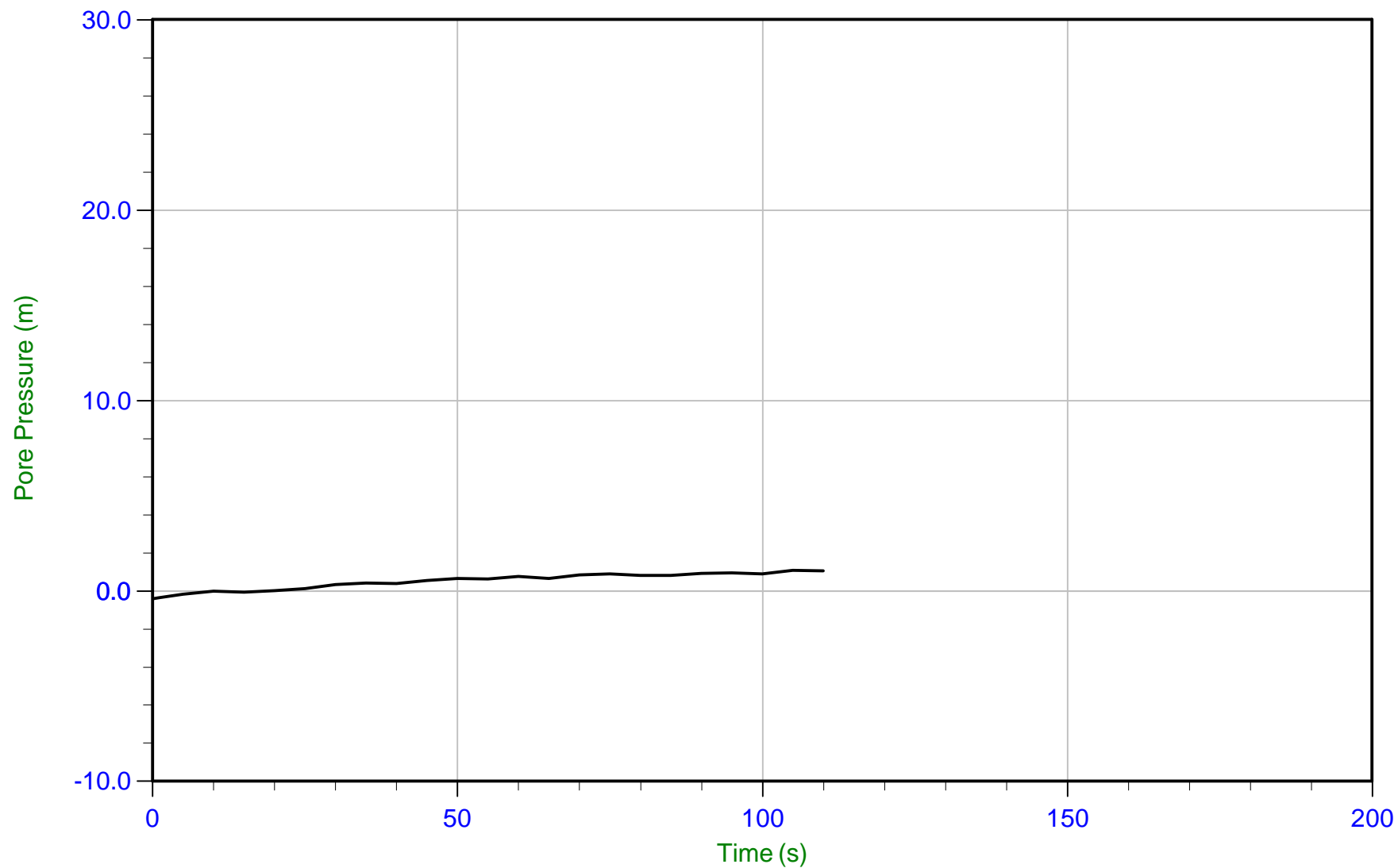
Job No: 18-05030

Date: 05/15/2018 08:36

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-09

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP09.PPF  
Depth: 1.350 m / 4.429 ft  
Duration: 110.0 s

U Min: -0.4 m  
U Max: 1.1 m



*Golder*

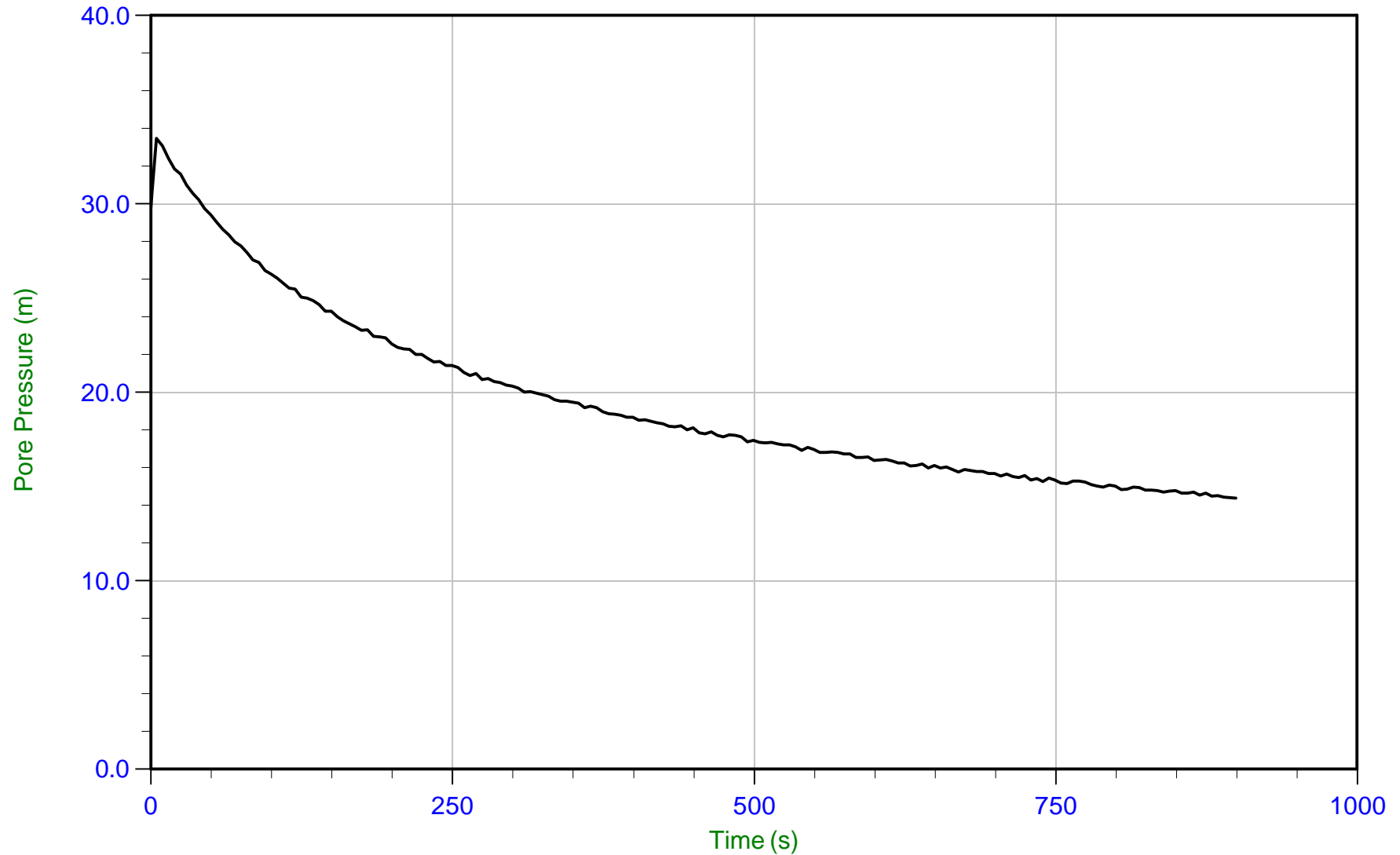
Job No: 18-05030

Date: 05/15/2018 08:36

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-09

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP09.PPF  
Depth: 6.250 m / 20.505 ft  
Duration: 900.0 s

U Min: 14.4 m  
U Max: 33.5 m

WT: 1.900 m / 6.234 ft  
Ueq: 4.3 m  
U(50): 18.92 m

T(50): 372.6 s  
Ir: 100  
Ch: 1.9 cm<sup>2</sup>/min



*Golder*

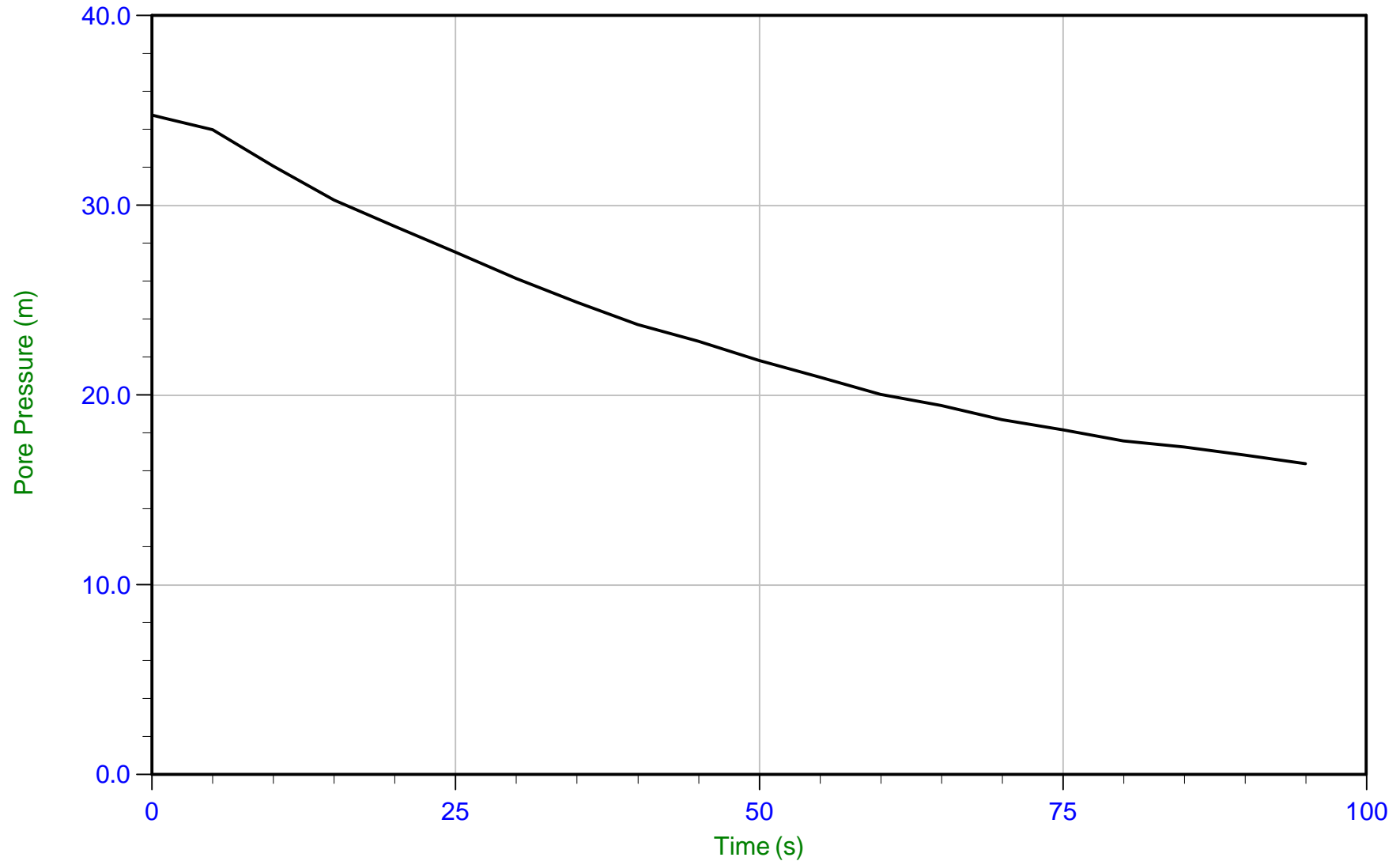
Job No: 18-05030

Date: 05/15/2018 08:36

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-09

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP09.PPF  
Depth: 9.300 m / 30.511 ft  
Duration: 95.0 s

U Min: 16.4 m  
U Max: 34.8 m

WT: 1.900 m / 6.234 ft  
Ueq: 7.4 m  
U(50): 21.08 m

T(50): 54.2 s  
Ir: 100  
Ch: 12.9 cm<sup>2</sup>/min



*Golder*

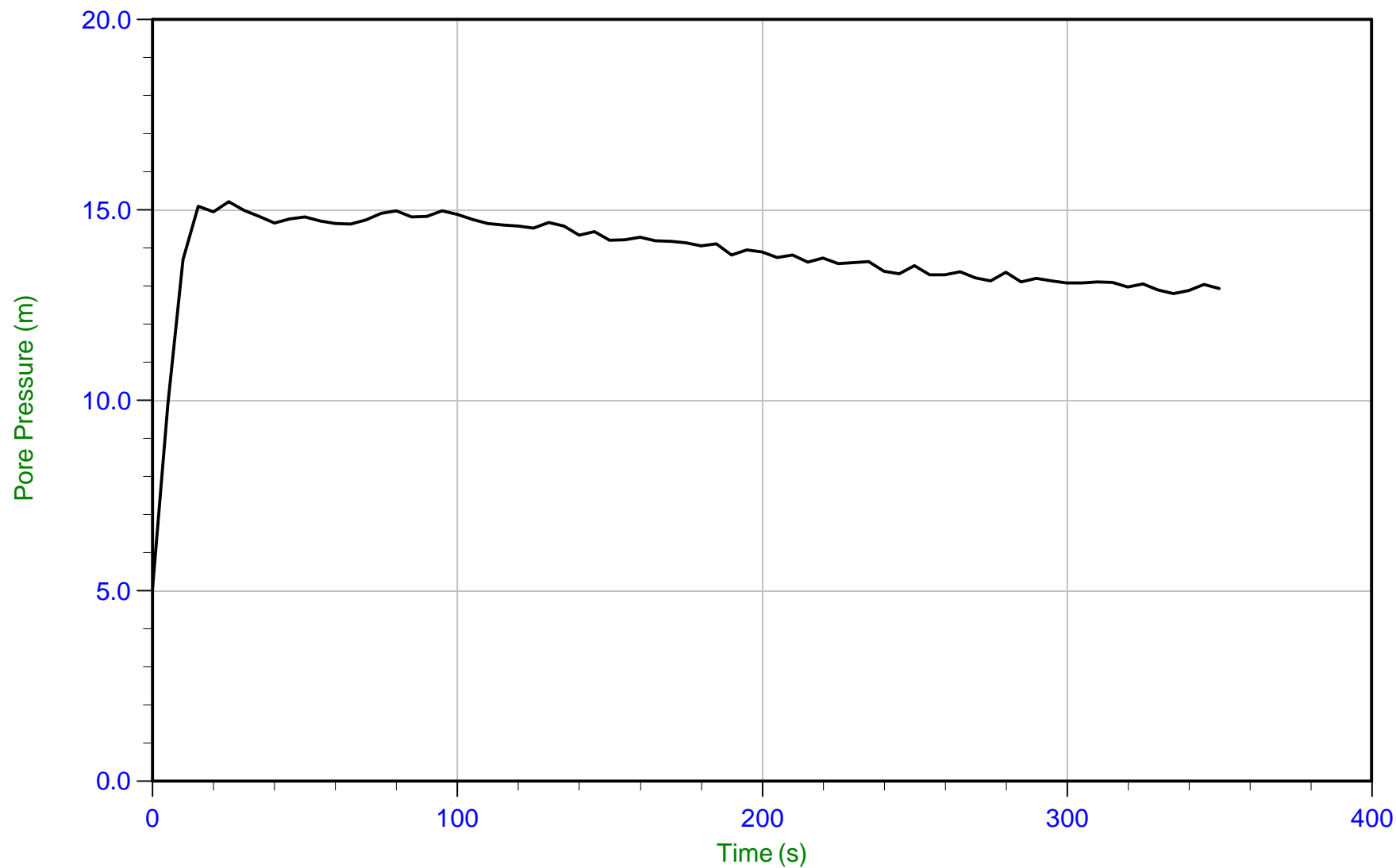
Job No: 18-05030

Date: 05/15/2018 08:36

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-09

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP09.PPF  
Depth: 10.500 m / 34.448 ft  
Duration: 350.0 s

U Min: 5.1 m  
U Max: 15.2 m



*Golder*

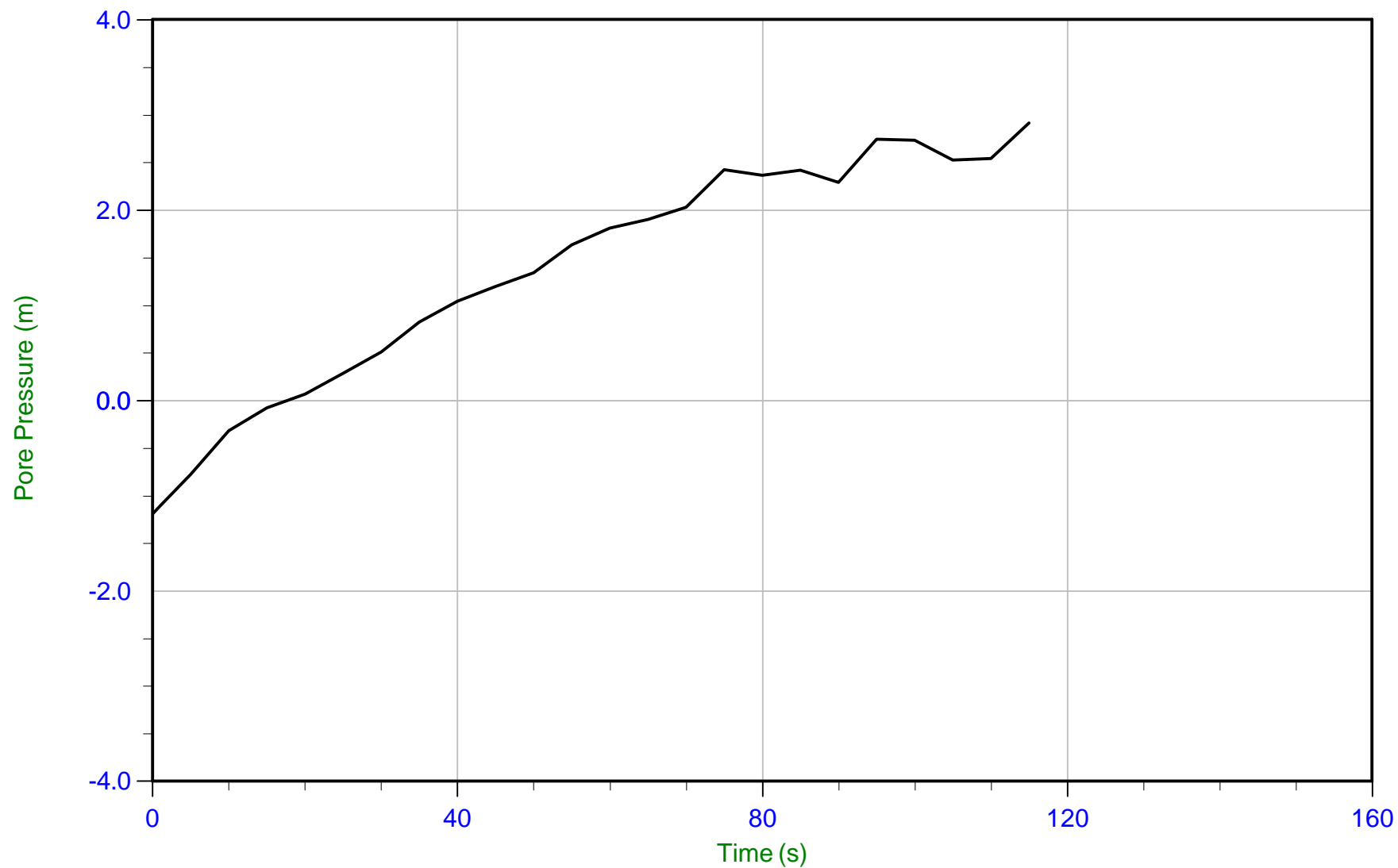
Job No: 18-05030

Date: 05/15/2018 12:01

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-10

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP10.PPF  
Depth: 1.225 m / 4.019 ft  
Duration: 115.0 s

U Min: -1.2 m  
U Max: 2.9 m





*Golder*

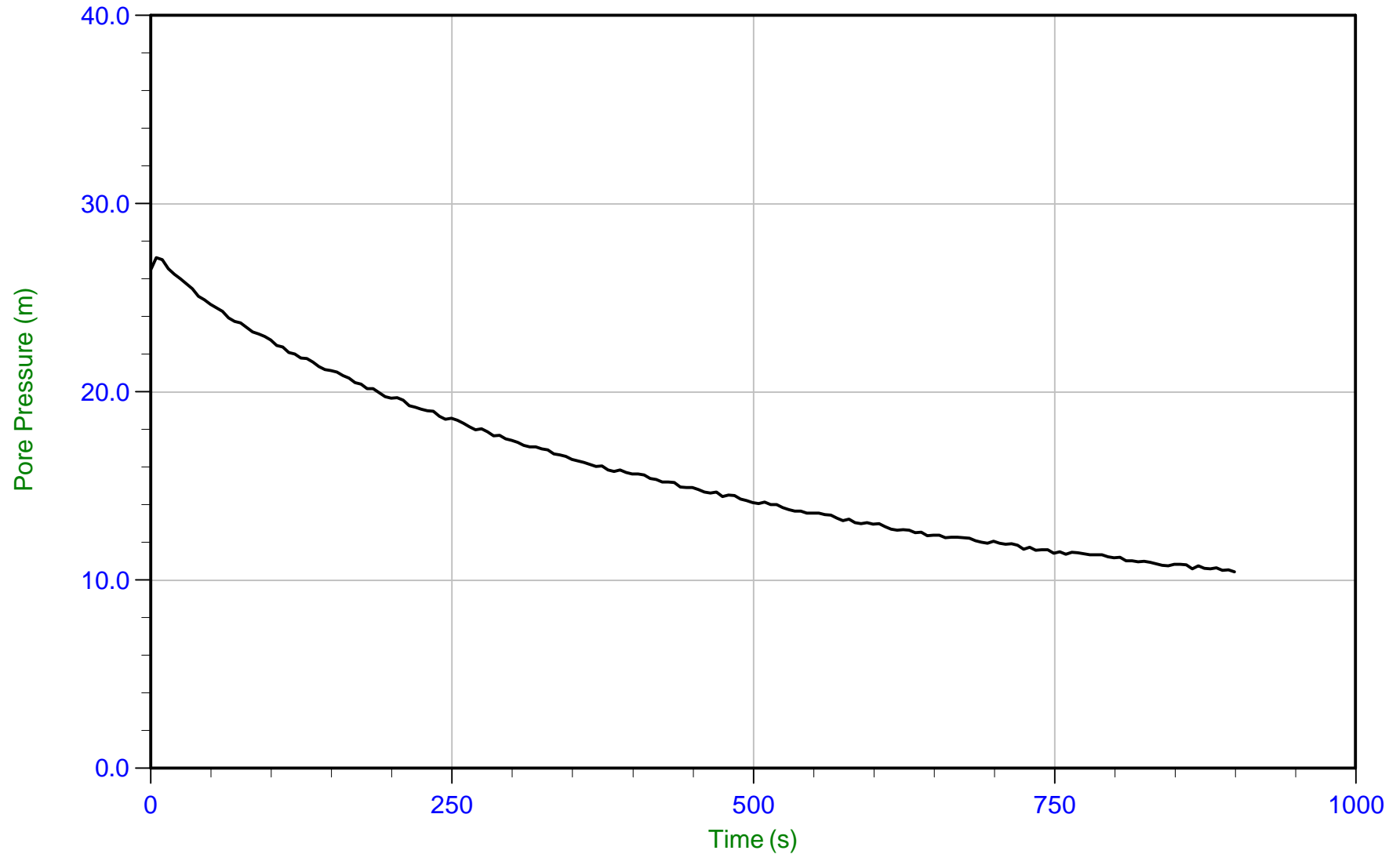
Job No: 18-05030

Date: 05/15/2018 12:01

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-10

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP10.PPF  
Depth: 4.225 m / 13.861 ft  
Duration: 900.0 s

U Min: 10.4 m  
U Max: 27.1 m

WT: 1.800 m / 5.905 ft  
Ueq: 2.4 m  
U(50): 14.77 m

T(50): 451.9 s  
Ir: 100  
Ch: 1.6 cm<sup>2</sup>/min



*Golder*

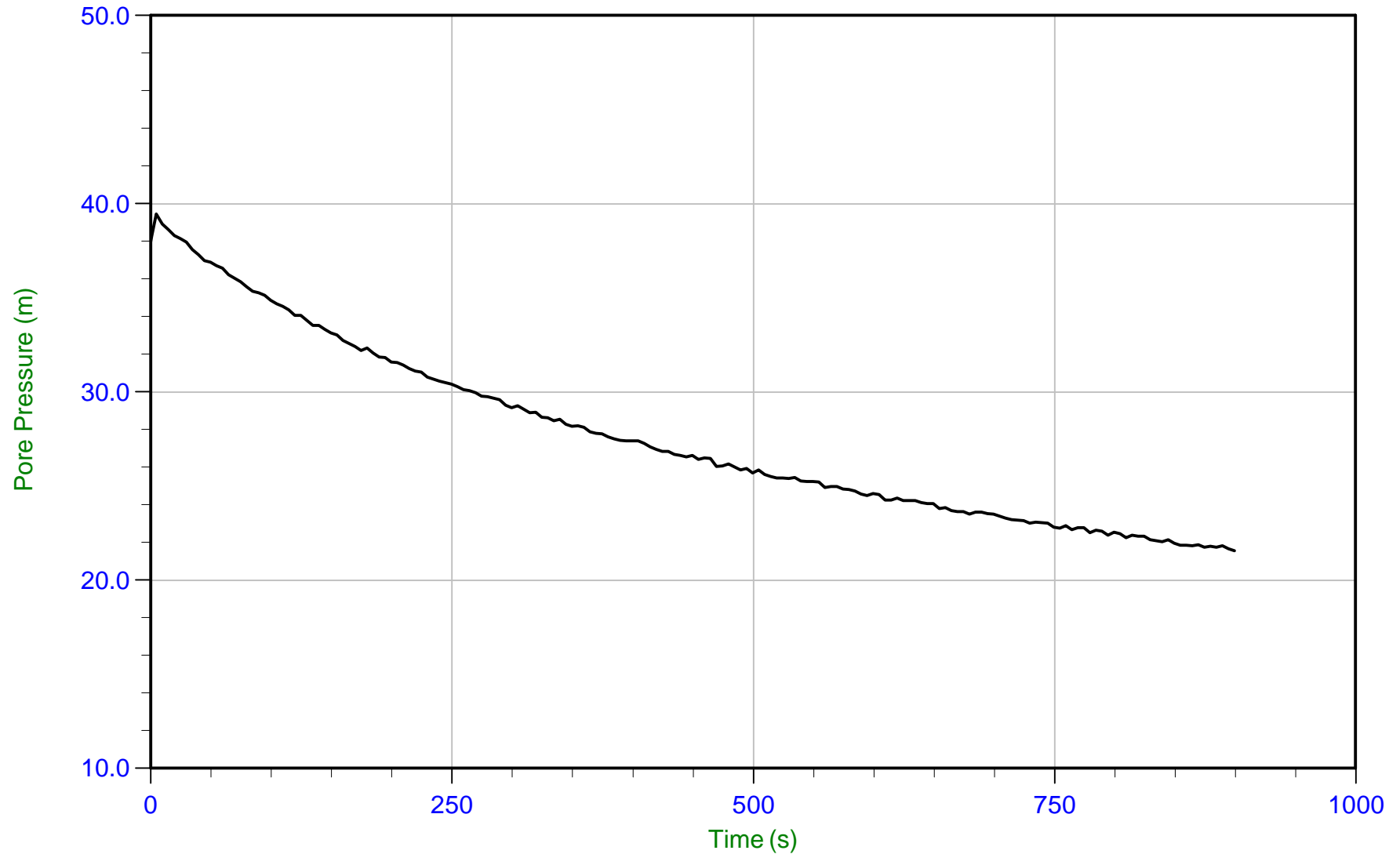
Job No: 18-05030

Date: 05/15/2018 12:01

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-10

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP10.PPF  
Depth: 7.200 m / 23.622 ft  
Duration: 900.0 s

U Min: 21.6 m  
U Max: 39.5 m

WT: 1.800 m / 5.905 ft  
Ueq: 5.4 m  
U(50): 22.43 m

T(50): 788.9 s  
Ir: 100  
Ch: 0.9 cm<sup>2</sup>/min



*Golder*

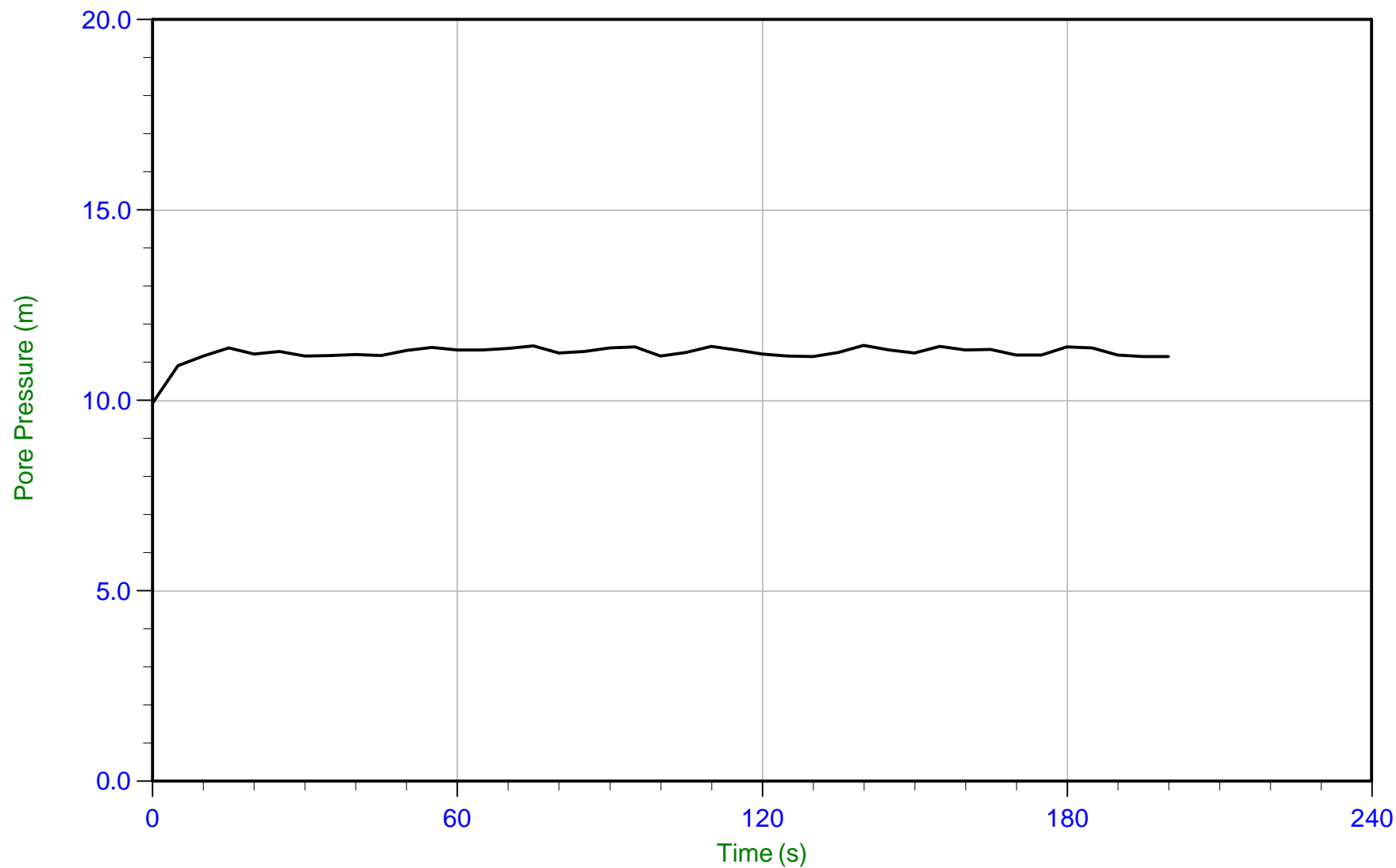
Job No: 18-05030

Date: 05/15/2018 12:01

Site: Hwy 416 and McKenna Casey Dr

Sounding: SCPT18-10

Cone: 330:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary: Filename: 18-05030\_SP10.PPF  
Depth: 11.375 m / 37.319 ft  
Duration: 200.0 s

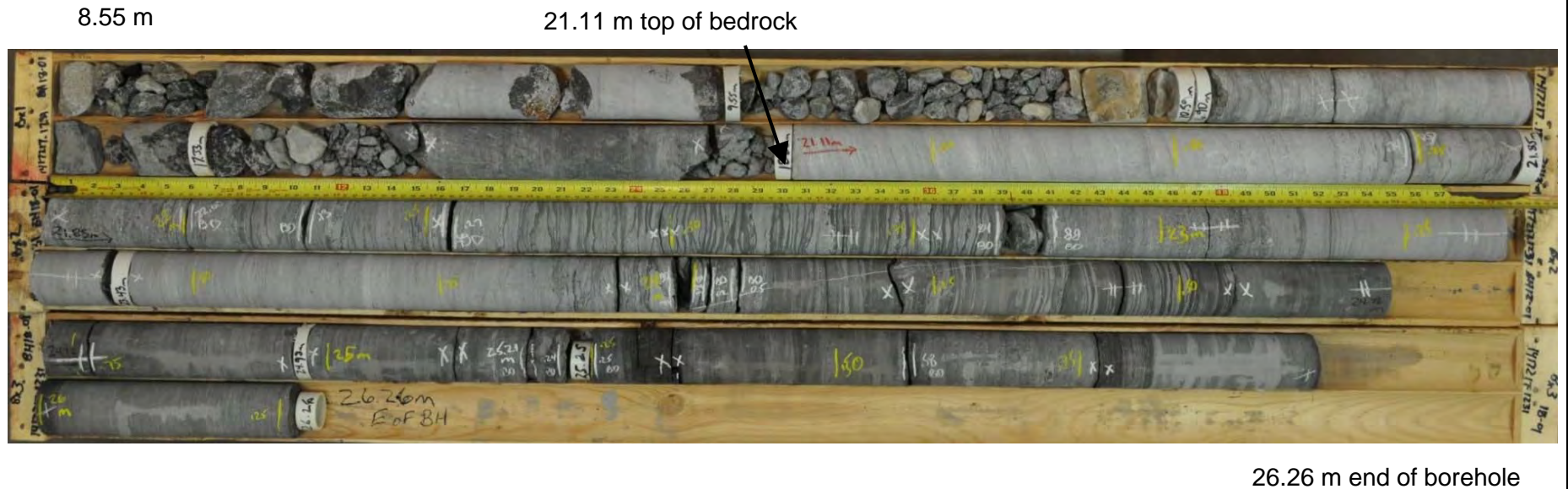
U Min: 9.9 m  
U Max: 11.5 m

WT: 0.194 m / 0.636 ft  
Ueq: 11.2 m

**APPENDIX D**

**Bedrock Core Photographs  
(Golder, 2018)**

**BH 18-01 (Dry)**  
**Cored Length of 8.55 to 26.26 metres**  
**Core Box 1 to 3 of 3**



Note: Materials in coreboxes from 8.55 to 21.11 metres is gravel and cobbles recovered from overburden.



**Geotechnical Investigation**  
**Hwy 416 Overpass Bridges at Strandherd Drive and CNR**  
**Ottawa, Ontario**

Project No.	1417217
Drawn:	MVRD
Date:	2018/06/06
Checked:	MJK
Review:	MSS

**Figure D1**

**BH 18-01 (Wet)**  
**Cored Length of 8.55 to 26.26 metres**  
**Core Box 1 to 3 of 3**

8.55 m

21.11 m top of bedrock



26.26 m end of borehole

Note: Materials in coreboxes from 8.55 to 21.11 metres is gravel and cobble recovered from overburden.



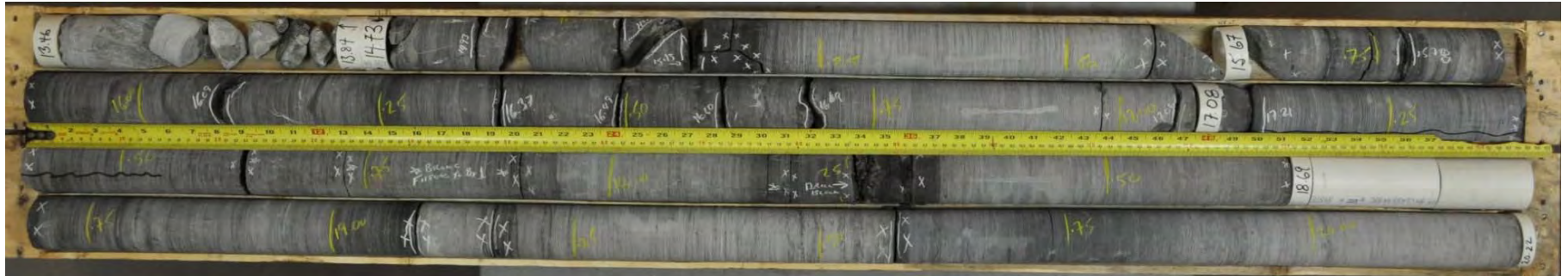
**Geotechnical Investigation**  
**Hwy 416 Overpass Bridges at Strandherd Drive and CNR**  
**Ottawa, Ontario**

Project No.	1417217
Drawn:	MVRD
Date:	2018/06/06
Checked:	MJK
Review:	MSS

**Figure D2**

**BH 18-02 (Dry)**  
**Cored Length of 14.73 to 20.22 metres**  
**Core Box 1 and 2 of 2**

14.73 m top of bedrock



20.22 m end of borehole

Note: Materials in coreboxes from 13.46 to 14.73 metres is gravel and cobbles recovered from overburden.



**Geotechnical Investigation**  
**Hwy 416 Overpass Bridges at Strandherd Drive and CNR**  
**Ottawa, Ontario**

Project No.	1417217
Drawn:	MVRD
Date:	2018/06/06
Checked:	MJK
Review:	MSS

**Figure D3**



**BH 18-02 (Wet)**  
**Cored Length of 14.73 to 20.22 metres**  
**Core Box 1 and 2 of 2**

14.73 m top of bedrock



20.22 m end of borehole

Note: Materials in coreboxes from 13.46 to 14.73 metres is gravel and cobbles recovered from overburden.



**Geotechnical Investigation**  
**Hwy 416 Overpass Bridges at Strandherd Drive and CNR**  
**Ottawa, Ontario**

Project No.	1417217
Drawn:	MVRD
Date:	2018/06/06
Checked:	MJK
Review:	MSS

**Figure D4**



**APPENDIX E**

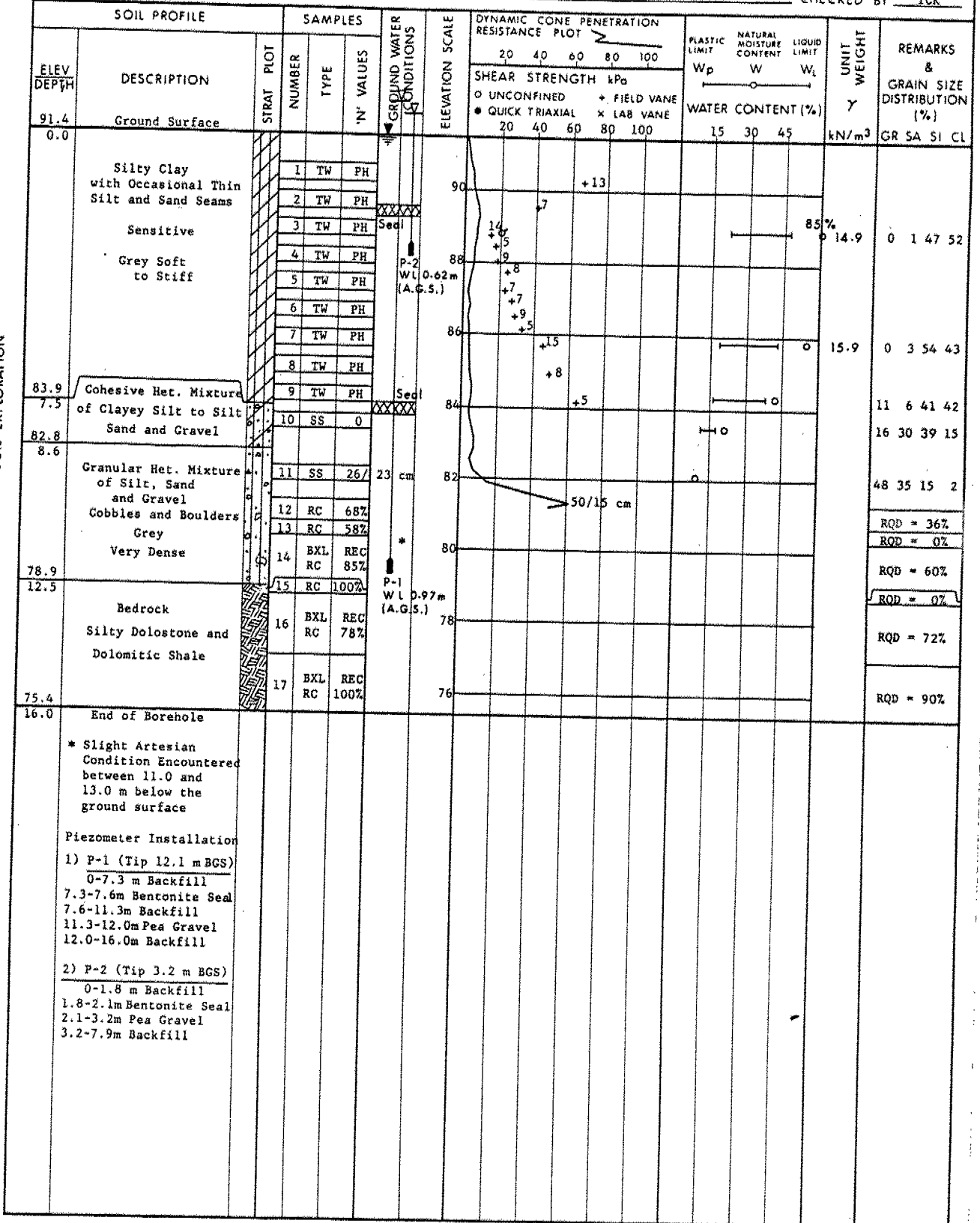
**Borehole and Drillhole Logs  
(1989 and 1991 Investigations)**

# RECORD OF BOREHOLE No 21A-1

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 683.1; E 360 968.9  
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring & Cone Test  
 DATUM Geodetic DATE 89 05 08 - 09  
 ORIGINATED BY TK  
 COMPILED BY AL  
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION



+3, x5: Numbers refer to  
Sensitivity  
20  
15  $\div$  5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 21A-2

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 697.7; E 360 962.8 ORIGINATED BY TCK  
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring & Cone Test COMPILED BY AL  
 DATUM Geodetic DATE 89 05 09 - 10 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION [%]
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER								
91.4	Ground Surface										
0.0	Silty Clay With Occasional Thin Silt and Sand Seams  Sensitive  Grey Soft to Stiff		1	TW	PH						0 4 56 40
			2	TW	PH						
			3	TW	PH						0 0 55 45
			4	TW	PH						
			5	TW	PH						2 2 66 30
83.9	Cohesive Het. Mixture of Clayey Silt to Silt		6	SS	0						15 25 49 11
7.5	Sand and Gravel		7	SS	110						
82.8	Granular Het. Mixture of Silt, Sand and Gravel		8	RC	-						
8.6	Cobbles and Boulders Grey		9	RC	100%						RQD = 75%
	Very Dense		10	RC	88%						RQD = 69%
79.3			11	RC	92%						RQD = 67%
12.1	Bedrock		12	RC	100%						RQD = 38%
77.5	Silty Dolostone and Dolomitic Shale		13	BXL RC	100% REC						RQD = 76%
13.9	End of Borehole										
	* Slight Artesian Condition Encountered Below 9.8 m G.S.										

+3, x5: Numbers refer to 20  
Sensitivity 15 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 21A-3

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 698.3; E 360 953.2  
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring & Cone Test  
 DATUM Geodetic DATE 89 05 10  
 ORIGINATED BY TCK  
 COMPILED BY AL  
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	'N' VALUES					
91.5	Ground Surface											
0.0	Silty Clay With Occasional Thin Silt and Sand Seams  Sensitive  Grey Soft to Stiff		1	TW	PH							
			2	TW	PH							
			3	TW	PH							
			4	TW	PH							
84.2	Cohesive Het. Mixture											3 5 56 36
7.3	Clayey Silt to Silt											
82.9	Sand and Gravel		5	SS	2/38 cm							2 29 48 21
8.6	Granular Het. Mixture of Silt, Sand and Gravel Cobbles and Boulders Grey Dense to Very Dense		6	SS	35							45 38 13 4
			7	SS	Bouncing *							
79.6			8	RC	REC 100%							RQD = 77%
11.9	Bedrock		9	RC	-							
78.1	Silty Dolostone and Dolomitic Shale		10	BXL RC	REC 100%							RQD = 76%
13.4	End of Borehole											
	* Slight Artesian Condition Encountered Below 10.7 m G.S.											

+3, +5: Numbers refer to  
Sensitivity  
20  
15 + 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 21A-4

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 736.8; E 360 946.7  
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, BXL Rock Coring & Cone Test  
 DATUM Geodetic DATE 89 05 04  
 ORIGINATED BY TS  
 COMPILED BY TCK  
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>		
91.8 0.0	Ground Surface													
	Silty Clay With Occasional Thin Silt and Sand Seams  Sensitive  Grey Soft to Stiff		1	SS	4									
			2	TW	PH									
			3	TW	PH									
			4	TW	PH									
85.6 6.2	Cohesive Het. Mixture of Clayey Silt to Silt Sand and Gravel		5	TW	PH									
			6	SS	12									
82.4 9.4	Granular Het. Mixture of Silt, Sand and Gravel Cobbles and Boulders Very Dense		7	SS	8									
			8	RC	58%									
			9	RC	32%									
			10	RC	27%									
79.3 12.5	Bedrock Silty Dolostone and Dolomitic Shale		11	RC	65%									
			12	BXL RC	REC 100%									
			13	BXL RC	REC 100%									
76.0 15.8	End of Borehole													

+3, x5: Numbers refer to  
Sensitivity  
20  
15  
10  
5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 21A-5

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 758.8; E 360 937.5  
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring & Cone Test  
 DATUM Geodetic DATE 89 05 05 - 06  
 ORIGINATED BY TCK  
 COMPILED BY AL  
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
92.0	Ground Surface													
0.0	Silty Clay With Occasional Thin Silt and Sand Seams Sensitive Grey Soft to Stiff		1	SS	3									
			2	TW	PH									
			3	TW	PH									
			4	TW	PH									
86.5														
5.5	Granular Het. Mixture of Silt, Sand and Gravel Cobbles and Boulders		5	SS	5									
			6	SS	10									
			7	SS	4									
81.9														
10.1	Sandy Silt Trace of Gravel Very Loose		8	SS	1									
79.9														
12.1			9	SS	34	8 cm								
79.2	Loose to Very Dense		10	RC	100%	REC								
12.8	Bedrock Silty Dolostone and Dolomitic Shale		12	RC	86%	REC								
			13	RC	100%	REC								
77.2			14	BXL	REC									
14.8	End of Borehole													

+3, x5: Numbers refer to Sensitivity  
 20  
 15 5 (%) STRAIN AT FAILURE  
 10

RECORD OF BOREHOLE No 21A-6

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 775.5; E 360 930.4  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring & Cone Test  
DATUM Geodetic DATE 89 05 05 - 06  
ORIGINATED BY TCK  
COMPILED BY AL  
CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
92.3 0.0	Ground Surface													
	Silty Clay With Occasional Thin Silt and Sand Seams		1	TW	PH		92							
	Sensitive		2	TW	PH		90							
	Grey Soft to Stiff		3	TW	PH		88							
			4	TW	PH		86							
87.7			5	TW	PH		84							
4.6	Cohesive Het. Mixture Clayey Silt to Silt Sand and Gravel		6	TW	PH		82							
86.5			7	TW	PH		80							
5.8	Granular Het. Mixture of Silt, Sand and Gravel		8	SS	5		78							
	Cobbles and Boulders		9	SS	8									
82.2			10	SS	7									
10.1	Sandy Silt Trace of Gravel Loose		11	SS	4									
80.6			12	SS	-									
11.7	Loose to Very Dense		13	BXL RC	REC 14%									
78.9			14	BXL RC	REC 100%									
13.4	Bedrock Silty Dolostone and Dolomitic Shale													
77.6														
14.7	End of Borehole													

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10  
5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 21A-7

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 790.2; E 360 924.4  
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring & Cone Test  
 DATUM Geodetic DATE 89 05 04 - 05  
 ORIGINATED BY TCK  
 COMPILED BY AL  
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			VALUES	20 40 60 80 100	Wp W WL	Wp W WL	Wp W WL		
92.5	Ground Surface												
0.0	Silty Clay with Occasional Thin Silt and Sand Seams Sensitive Grey Soft to Very Stiff		1	TW	PH								
			2	TW	PH								
			3	TW	PH								
88.7			4	TW	PH								
3.8	Cohesive Het. Mixture of Clayey Silt to Silt Sand and Gravel		5	TW	refusal								
87.3			6	SS	0								
5.2	Sand with Gravel Some Silt Grey Loose to Compact		7	SS	1								
			8	SS	7								
			9	SS	5								
			10	SS	13								
			11	SS	13								
82.4			12	SS	52	13 cm							
10.1	Granular Het. Mixture of Silt, Sand and Gravel Cobbles and Boulders Very Dense		13	BXL	REC								
			14	RC	38%								
			15	BXL	REC								
			16	RC	59%								
78.2													
14.3	Bedrock Silty Dolostone and Dolomitic Shale												
76.6													
15.9	End of Borehole												

+3, x5; Numbers refer to Sensitivity  
 20 15 10  
 5 (%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No 21A-8

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 634.0; E 360 984.7  
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger and Cone Test ORIGINATED BY TS  
 DATUM Geodetic DATE 89 05 09 COMPILED BY TCK  
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES								
91.3 0.0	Ground Surface												
	Silty Clay With Occasional Thin Silt and Sand Seams  Sensitive Grey Soft to Stiff		1	SS	5								
			2	TW	PH								
			3	TW	PH								
			4	TW	PH								
			5	TW	PH								
83.7 7.6	Cohesive Het. Mixture of Clayey Silt to Silt Sand and Gravel		6	SS	1							18.9	0 21 55 24
82.5 8.8	Granular Het. Mixture of Silt, Sand and Gravel Cobbles and Boulders Very Loose to V. Dense		7	SS	2								1 11 43 45
80.2 11.1	End of Borehole		8	SS	100/25								1 14 55 30
	* Slight Artesian Condition Encountered below 10.7 m G.S.												22 57 18 3

+3, x5; Numbers refer to  
Sensitivity

20  
15  
10  
5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 21A-9

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 661.6; E 360 973.3 ORIGINATED BY TS  
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger and Cone Test COMPILED BY TCK  
 DATUM Geodetic DATE 89 05 08 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W		
91.3	Ground Surface												
0.0	Silty Clay With Occasional Thin Silt and Sand Seams  Sensitive Gray Soft to Stiff		1	SS	4							19.2	4 14 58 24
			2	TW	PH								
			3	TW	PH								
			4	TW	PH								
83.7	Cohesive Het. Mixture of Clayey Silt to Silty		5	SS	4								0 1 40 59
7.6	Sand and Gravel												32 37 24 7
82.5	Granular Het. Mixture of Silt, Sand and Gravel		6	SS	16								22 46 26 6
8.8	Cobbles and Boulders Compact to Very Dense		7	SS	100/18 cm								
80.3	End of Borehole												
11.0													

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10  
5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 21A-10

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 805.7; E 360 913.3  
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger ORIGINATED BY TS  
 DATUM Geodetic DATE 89 05 06 COMPILED BY TCK  
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
94.4	Ground Surface																
0.0	Sand and Gravel																
93.0	Brown (Fill)		1	SS	11		94										
1.4	Silty Clay Brown With Grey		2	SS	10												
	Occasional Thin Silt and Sand Seams		3	SS	7		92										0 10 52 38
90.7																	
3.7	Cohesive Het. Mixture		4	SS	20		90										
	Clayey Silt to Silt		5	SS	6												7 14 35 44
88.8	Sand and Gravel																
5.6	Sand with Gravel		6	SS	70		88										41 47 8 4
87.3	Some Silt, Grey																
7.1	Very Dense																
	Granular Het. Mixture		7	SS	18		86										
	of Silt, Sand																
	and Gravel																
84.8	Grey, Compact																
			8	SS	11												
9.6	End of Borehole																6 32 52 10

+3, x5: Numbers refer to 20  
Sensitivity 15 ± 5 (%) STRAIN AT FAILURE  
10

## RECORD OF BOREHOLE No 21A-11

METRIC

W P 128-87-05/06

LOCATION Co-ords. N 5 012 830.0; E 360 893.5

ORIGINATED BY TS

DIST 9 HWY 416

BOREHOLE TYPE HS Auger

COMPILED BY TCK

DATUM Geodetic

DATE 89 05 06

CHECKED BY        TCK

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

+3, x5 : Numbers refer to Sensitivity

15-20 5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 21B-1

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 657.0; E 360 938.6  
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, EXL Rock Coring & Cone Test ORIGINATED BY TCK  
 DATUM Geodetic DATE 89 05 09 COMPILED BY AL  
 CHECKED BY TCK

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 x LAB VANE 20 40 60 80 100	PLASTIC LIMIT Wp NATURAL MOISTURE CONTENT W LIQUID LIMIT Wl WATER CONTENT (%) 15 30 45	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
91.3	Ground Surface									
0.0			1	TW	PH					
	Silty Clay		2	TW	PH					
	With Occasional Thin Silt and Sand Seams		3	TW	PH					
	Sensitive		4	TW	PH					
	Grey Soft to Firm		5	TW	PH					
83.8			6	TW	PH				15.3	0 0 52 48
7.5	Cohesive Het. Mixture of Clayey Silt to Silt Sand and Gravel		7	SS	6					
82.7			8	SS	Refusal					
8.6	Granular Het. Mixture of Silt, Sand and Gravel Cobbles and Boulders Loose to Very Dense		9	EXL RC	REC 100%					
80.5										
10.8	Bedrock									
79.0	Silty Dolostone and Dolomitic Shale									
12.3	End of Borehole									RQD = 87%

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 21B-2

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 671.7; E 360 932.5 ORIGINATED BY TCK  
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring & Cone Test COMPILED BY AL  
 DATUM Geodetic DATE 89 05 08 - 09 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

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 x LAB VANE 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	WATER CONTENT (%) 15 30 45	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES									
91.4 0.0	Ground Surface													
	Silty Clay With Occasional Thin Silt and Sand Seams Sensitive Grey Soft to Stiff		1	TW	PH									
			2	TW	PH									
			3	TW	PH									
			5	TW	PH									
			6	TW	PH									
			7	TW	PH									
			8	TW	PH									
83.9 7.5		Granular Mat. Mixture of Silt, Sand and Gravel Cobbles and Boulders Loose to Very Dense		9	TW	PH								
			10	TW	PH									
			11	SS	4									
80.6 10.8	Bedrock Silty Dolostone and Dolomitic Shale		12	SS	Refusal *									
			13	BXL RC	REC 100%									
			14	BXL RC	REC 98%									
77.7 13.7	End of Borehole													
	* Slight Artesian Condition Encountered Below 10.7 m B.G.													

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 21B-4

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 710.6; E 360 916.3  
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, Washboring, BXL Rock Coring & Cone Test  
 DATUM Geodetic DATE 89 05 03 - 04  
 ORIGINATED BY TS  
 COMPILED BY TCK  
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE			VALUES	20 40 60 80 100					
91.8	Ground Surface											
0.0	Silty Clay With Occasional Thin Silt and Sand Seams  Sensitive  Grey Soft to Firm	1	SS	4		+12					15.2	0 1 42 57
		2	TW	PH		+12 +5						
		3	TW	PM								
		4	SS	2		+11 +10						
84.7		5	TW	PH								1 11 33 55
7.1	Granular Het. Mixture of Silt, Sand and Gravel Cobbles and Boulders  Loose	6	SS	1		+12						
		7	SS	5								25 29 36 10
81.1		8	SS	Bouncing								
10.7	Bedrock Silty Dolostone and Dolomitic Shale	9	BXL RC	REC 98%								RQD = 55%
79.6												
12.2	End of Borehole											

+3, x5: Numbers refer to  
Sensitivity

20  
15 +5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 21B-5

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 732.7; E 360 907.1  
 DIST 9 HWY 416 BOREHOLE TYPE HS Auger, BW Casing, BXL Rock Coring & Cone Test  
 DATUM Geodetic DATE 89 05 04  
 ORIGINATED BY TS  
 COMPILED BY TCK  
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

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 × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 15 30 45	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
92.1	Ground Surface										
0.0	Silty Clay With Occasional Thin Silt and Sand Seams Sensitive Grey Soft to Stiff		1	SS	5						
88.3			2	TW	PH						
3.8	Granular Het. Mixture of Silt, Sand and Gravel Cobbles and Boulders Loose to Very Dense		3	TW	PH						
			4	SS	7						
			5	SS	12						
			6	SS	13						
			7	SS	20						
			8	RC	58%						
81.7											
10.4	Sandy Silt Trace Gravel		9	SS	1						
80.5											
11.6	Bedrock Silty Dolostone and Dolomitic Shale		10	BXL RC	REC 90%						
79.2											
12.9	End of Borehole										

+3, x5: Numbers refer to Sensitivity  
 20  
 15 + 5 (%) STRAIN AT FAILURE  
 10



# RECORD OF BOREHOLE No 21B-6

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 749.5; E 360 900.1 ORIGINATED BY TS  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger, Washboring, BXL Rock Coring COMPILED BY TCK  
 DATUM Geodetic DATE 89 05 03 - 06 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20 40 60 80 100	15 30 45	W <sub>p</sub>	W	W <sub>L</sub>		
92.3	Ground Surface												
0.0	Silty Clay With Occasional Thin Silt and Sand Seams Sensitive Grey Soft to Stiff		1	SS	6	92							0 15 57 28
89.1			2	TW	PM	90	+11 +10 +6 +15					15.2	0 2 49 49
3.2	Granular Het. Mixture of Silt, Sand and Gravel Cobbles and Boulders Compact to Very Dense		3	SS	11	88							RQD = 0%
			4	SS	30								
			5	RC	REC 40%	86							
			6	SS	48								
			7	SS	69	84							RQD = 0%
			8	RC	31%								23 35 33 9
82.2			9	SS	14	82							6 37 47 10
10.1	Sandy Silt Trace Gravel Loose		10	SS	6								RQD = 0%
80.5						80							RQD = 69%
11.8	Bedrock Silty Dolostone and Dolomitic Shale		11	RC	81%								RQD = 0%
			12	BXL RC	100%								
			13	BXL RC	31%	78							
77.4													RQD = 0%
14.9	End of Borehole												

+3, x5: Numbers refer to 20  
Sensitivity 15 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 21B-7

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 764.1; E 360 894.0 ORIGINATED BY TS  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger, Washboring, BXL Rock Coring & Cone Test COMPILED BY TCK  
 DATUM Geodetic DATE 89 05 02 - 03 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20 40 60 80 100	20 40 60 80 100					
92.5	Ground Surface												
0.0	Silty Clay With Occasional Thin Silt and Sand Seams Sensitive Grey Stiff		1	SS	3								0 1 59 40
			2	TW	PH								
88.8			3	SS	16								
3.7	Granular Het. Mixture of Silt, Sand and Gravel Cobbles and Boulders Compact to Very Dense		4	SS	17								37 38 18 7
			5	SS	16								
			6	RC	REC 67%								RQD = 0%
			7	SS	10								
			8	RC	REC 96%								RQD = 75%
83.9			9	SS	50								
8.6	Cohesive Het. Mixture of Silty Clay, Sand and Gravel		10	SS	4								12 32 41 15
82.4													
10.1	Sandy Silt Trace of Clay and Gravel		11	SS	1								1 9 80 10
80.0			12	SS	25								
12.5	Bedrock Silty Dolostone and Dolomitic Shale		13	RC	62%								RQD = 0%
			14	BXL RC	REC 100%								RQD = 79%
			15	BXL RC	REC 100%								RQD = 86%
76.6													
15.9	End of Borehole												

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 21B-8

METRIC

W P 128-87-05/06

LOCATION Co-ords. N 5 012 613.8; E 360 961.2

ORIGINATED BY JS

DIST 9 HWY 416

BOREHOLE TYPE H.S. Auger & Cone Test

COMPILED BY TCK

DATUM Geodetic

DATE 89 05 08

CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

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 x LAB VANE 20 40 60 80 100	PLASTIC LIMIT Wp	NATURAL MOISTURE CONTENT W	LIQUID LIMIT WL	WATER CONTENT (%) 15 30 45	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES										
91.3	Ground Surface														
0.0	Silty Clay With Occasional Thin Silt and Sand Seams  Sensitive  Grey Soft to Firm		1	SS	3		90	8						17.7	0 2 43 55
			2	TW	PH			+7							
			3	TW	PH		88	+3							
			4	TW	PH			+7							
			5	TW	PH		86	+8							
83.7			6	TW	PH			+6							
7.6	Cohesive Het. Mixture of Silty Clay, Sand and Gravel  Soft		7	SS	3		84	+5							0 15 46 39
								+7							
								+4							
								+5							
								+4							
80.8	End of Borehole At Refusal (Probable Boulders)						82								
10.5															

+3, x5 : Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 21B-9

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 641.5; E 360 949.6  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger & Cone Test ORIGINATED BY TS  
 DATUM Geodetic DATE 89 05 08 COMPILED BY TCK  
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W		
91.3 0.0	Ground Surface												
	Silty Clay With Occasional Thin Silt and Sand Seams  Sensitive Grey Soft to Stiff		1	SS	6								
			2	TW	PH								
			3	TW	PH								
			4	TW	PH								
			5	TW	PH								
84.6 6.7	Cohesive Het. Mixture of Silty Clay Sand and Gravel  Soft to Stiff		6	SS	4								
			7	SS	9								
			8	SS	8	25 cm							
80.2 11.1	End of Borehole  * Sampler Bouncing												

+3, x5: Numbers refer to  
Sensitivity  
20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10

RECORD OF BOREHOLE No 21B-10

METRIC

W P 128-87-05/06 LOCATION Co-ords. N 5 012 790.0; E 360 887.5  
DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger & Cone Test ORIGINATED BY TS  
DATUM Geodetic DATE 89 05 05 COMPILED BY TCK  
CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

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 x LAB VANE 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 15 30 45	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
95.1	Ground Surface										
0.0	Sand and Gravel (Fill)		1	SS	40						
93.0			2	SS	13						
2.1	Silty Clay with Occasional thin Silt and Sand Seams		3	SS	14						
90.7	Stiff to Very Stiff		4	SS	18						
4.4	Cohesive Het. Mixture of Silty Clay, Sand and Gravel		5	SS	10						
89.9			6	SS	2						
5.2	Granular Het. Mixture of Silt, Sand and Gravel, Cobbles and Boulders		7	SS	13						
	Compact		8	SS	24						
			9	SS	27						
85.0			10	SS	30						
10.1	Sandy Silt, Dense										
84.0											
11.1	End of Borehole		11	SS	30						

# RECORD OF BOREHOLE No 21B-11

METRIC

W P 128-87-05/06

LOCATION Co-ords. N 5 012 807.0; E 360 875.5

ORIGINATED BY TS

DIST 9 HWY 416

BOREHOLE TYPE H.S. Auger

COMPILED BY TCK

DATUM Geodetic

DATE 89 05 05

CHECKED BY TCK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	SHEAR STRENGTH kPa						WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE							
96.4	Ground Surface														
0.0	Silty Clay with Sand and Gravel Firm to Stiff Brown Grey		1	SS	7										
94.3			2	SS	20										
2.1			3	SS	60										
	Heterogeneous mixture of Silt, Sand and Gravel (Glacial Till) Compact to Very Dense		4	SS	60										
			5	SS	58										
			6	SS	45										
	SANDY SILT														
86.8			7	SS	20										
9.6	End of Borehole														

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

**ROCK CORE DESCRIPTION**  
**WP 128-87-05/06**

1../3

CORE RECOVERY					CORE DESCRIPTION	
BH - RC # #	DEPTH (m)	CR* (%)	RQD* (%)	DEPTH (m)	DESCRIPTION	
21 A1	12	10.06-10.69	68	36	10.06-12.50	OVERBURDEN - rock fragments up to 53 cm.
	13	10.69-11.00	58	0	12.50-15.98	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; close spaced fractures.
	14	10.00-12.57	85	60		
	15	12.57-12.62	100	0		
	16	12.62-14.38	78	72		
	17	14.38-15.98	100	90		
21 A2	8	9.50- 9.65	100	0	9.50-12.09	OVERBURDEN - rock fragments up to 65 cm.
	9	9.65-10.41	100	75	12.09-13.92	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; close to very close spaced fractures.
	10	10.41-11.02	88	69		
	11	11.02-11.99	92	67		
	12	11.99-12.40	100	38		
	13	12.40-13.92	100	76		
21 A3	8	10.69-11.68	100	77	10.69-11.99	OVERBURDEN - rock fragments up to 51 cm.
	9	11.68-11.76	100	0	11.99-13.36	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; close spaced fractures.
	10	11.76-13.36	100	76		
21 A4	8	9.45-10.06	58	23	9.45-12.50	OVERBURDEN - rock fragments up to 15 cm.
	9	10.06-11.20	32	20	12.50-15.82	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; very close to moderately wide spaced fractures.
	10	11.20-11.99	27	0		
	11	11.99-12.67	65	0		
	12	12.67-14.22	100	72		
	13	14.22-15.82	100	70		

\*CR = CORE RECOVERY (NOTE: Depths are approximated in zones of poor core recovery.)

\*RQD = ROCK QUALITY DESIGNATION

Logged by: S. A. Senior, Soils and Aggregates Section.

**ROCK CORE DESCRIPTION**  
**WP 128-87-05/06**

2../3

CORE RECOVERY					CORE DESCRIPTION	
BH - RC #    #		DEPTH (m)	CR* (%)	RQD* (%)	DEPTH (m)	DESCRIPTION
21 A5	10	12.65-12.80	100	0	12.65-12.80	OVERBURDEN - rock fragments up to 8 cm.
	11	12.80-12.98	100	0		
	12	12.98-13.54	86	23	12.80-14.78	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered to slightly weathered, some fractures clay coated; very close to close spaced fractures.
	13	13.54-13.72	100	71		
	14	13.72-14.78	93	24		
21 A6	13	12.27-13.36	14	0	12.27-13.36	OVERBURDEN - rock fragments up to 3 cm, weathered, rounded.
	14	13.36-14.71	100	92	13.36-14.71	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; very close to moderately wide spaced fractures.
21 A7	13	11.58-13.23	38	0	11.58-14.20	OVERBURDEN - rock fragments up to 9 cm.
	14	13.23-13.54	58	0		
	15	13.54-14.53	59	0	14.20-15.93	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; close to moderately wide spaced fractures.
	16	14.33-15.93	100	65		
21 B1	9	10.80-12.32	100	87	10.80-12.32	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; close to moderately wide spaced fractures.
21 B2	13	10.80-12.19	100	77	10.80-13.69	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; close to moderately spaced fractures.
	14	12.19-13.69	98	84		

\*CR = CORE RECOVERY (NOTE: Depths are approximated in zones of poor core recovery.)

\*RQD = ROCK QUALITY DESIGNATION

Logged by: S. A. Senior, Soils and Aggregates Section.



**ROCK CORE DESCRIPTION**  
**WP 128-87-05/06**

3../3

CORE RECOVERY					CORE DESCRIPTION	
BH #	RC #	DEPTH (m)	CR* (%)	RQD* (%)	DEPTH (m)	DESCRIPTION
21 B4	9	10.67-12.19	98	55	10.67-12.19	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; very close to close spaced fractures.
21 B5	8	9.45-10.06	58	42	9.45-11.63	OVERBURDEN - rock fragments up to 25 cm.
	10	11.38-12.90	90	42	11.63-12.90	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; very close to close spaced fractures.
21 B6	8	8.53- 9.14	31	0	8.53-11.84	OVERBURDEN - rock fragments up to 8 cm.
	-	not cored	-	-		
	11	11.86-12.32	81	0	11.84-14.94	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; very close to close spaced fractures.
	12	12.32-13.69	100	69		
	13	13.69-14.94	31	0		
21 B7	6	5.03- 5.94	67	0	5.03-12.50	OVERBURDEN - rock fragments up to 34 cm.
	-	not cored	-	-		
	8	6.71- 7.62	96	75	12.50-15.88	SILTY DOLOSTONE and DOLOMITIC SHALE, medium to light grey; fine grained; weak to medium strong rock; unweathered; close spaced fractures.
	-	not cored	-	-		
	13	12.50-12.83	62	0		
	14	12.83-14.35	100	79		
	15	14.35-15.80	100	86		

\*CR = CORE RECOVERY (NOTE: Depths are approximated in zones of poor core recovery.)

\*RQD = ROCK QUALITY DESIGNATION

Logged by: S. A. Senior, Soils and Aggregates Section.

# RECORD OF BOREHOLE No 89-6

METRIC

W P 128-87-00 LOCATION Co-ords: N 5 012 428.3; E 361 070.2 ORIGINATED BY TS  
 DIST 9 HWY 416 BOREHOLE TYPE H S Auger & Cone Test COMPILED BY TS  
 DATUM Geodetic DATE 89 05 10 CHECKED BY           

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
91.3 0.0	Ground Surface														
	Silty Clay														
	With occ. Sand Seams		1	SS	2		90							0 2 50 48	
	Trace Organics		2	TW	PH		88							18.5 0 10 65 25	
	Grey, Soft						86								
	to Firm														
84.3	Het. Mixture of		3	TW	PH		84								
83.8	Silt Sand & Gravel														
	(Glacial Till)														
7.5	Auger Refusal														
	(Probable Boulder)														
	End of Borehole														

OFFICE REPORT ON SOIL EXPLORATION



# RECORD OF BOREHOLE No\*89-7

METRIC

W P 128-87-00 LOCATION Co-ords: N 5 012 463.1; E 361 023.8 ORIGINATED BY TS  
DIST 9 HWY 416 BOREHOLE TYPE H S Auger & Cone Test COMPILED BY TS  
DATUM Geodetic DATE 89 05 10 CHECKED BY \_\_\_\_\_

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 x LAB VANE 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
91.3	Ground Surface										
0.0	Silty Clay With occ. Sand Seams, Trace Organics Grey, Firm to Stiff		1	SS	8		90				0 3 57 40
			2	SS	4						0 0 42 58
87.0			3	TW	PH		88				
4.3	Het. Mixture of Silt Sand and Gravel (Glacial Till) Very Dense		4	SS	60	15cm	86				
85.2											
6.1	Auger Refusal (Probable Boulder) End of Borehole										

OFFICE REPORT ON SOIL EXPLORATION

+<sup>3</sup>, x<sup>5</sup>: Numbers refer to  
Sensitivity

20  
15 ◇ 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 89-7A

METRIC

W P 128-87-00

LOCATION Co-ords: N 5 012 416.5; E 361 043.0

ORIGINATED BY TS

DIST 9 HWY 416

BOREHOLE TYPE H S Auger & Cone Test

COMPILED BY TS

DATUM Geodetic

DATE 89 05 10

CHECKED BY

## 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 x LAB VANE  
20 40 60 80 100

PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT  
W<sub>p</sub> W W<sub>L</sub>  
WATER CONTENT (%)

UNIT WEIGHT  
Y

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

ELEV DEPTH

DESCRIPTION

STRAT PLOT

NUMBER

TYPE

'N' VALUES

91.3  
0.0

Ground Surface

87.3  
4.0

Het. Mixture of Silt, Sand, & Gravel

85.2  
6.1

(Glacial Till)

\* Sampler Bouncing (Probable Boulder) End of Borehole

1 SS 2

2 SS 2

3 SS \*

Seal

Piezometer

Seal

90

88

86

5

+6

+3

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to Sensitivity

20  
15  $\div$  5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 89-8

METRIC

W P 128-87-00 LOCATION Co-ords: N 5 012 520.5; E 361 031.9 ORIGINATED BY TS  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger & Cone Test COMPILED BY TS  
 DATUM Geodetic DATE 89 05 10 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ, kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
91.3	Ground Surface														
0.0	Silty Clay With occ. Sand Seams, Trace Organics Grey, Firm to Stiff		1	SS	5		90	6						15.6	0 1 51 48
			2	SS	3		88	10							
			3	TW	PH		86	5						17.4	0 10 49 41
			4	TW	PH		84	5							
			5	TW	PH		82	5							
83.7	Het. Mixture of Silty Clay, Sand and Gravel (Glacial Till)		6	TW	PH			10							
7.6	Grey, Firm							5							
82.2	Het. Mixture of Silt, Sand and Gravel (Glacial Till) Compact		7	SS	10										
9.1	Auger Refusal (Probable Boulder) End of Borehole														
80.9															
10.4															

OFFICE REPORT ON SOIL EXPLORATION

+<sup>3</sup>, x<sup>5</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 89-8A

METRIC

W P 128-87-00

LOCATION Co-ords: N 5 012 493.0; E 361 043.0

DIST 9 HWY 416

BOREHOLE TYPE H S Auger

ORIGINATED BY TS

DATUM Geodetic

DATE 89 05 10

COMPILED BY TS

CHECKED BY

## 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 x LAB VANE  
20 40 60 80 100

PLASTIC LIMIT Wp NATURAL MOISTURE CONTENT W LIQUID LIMIT Wl  
WATER CONTENT (%)  
20 40 60

UNIT WEIGHT γ

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

ELEV DEPTH

DESCRIPTION

STRAT PLOT

NUMBER

TYPE

'N' VALUES

91.3

0.0

Ground Surface

Silty Clay

With occ. Sand Seams

Trace Organics

Grey, Firm

to Stiff

1

AS

--

2

SS

2

3

SS

1

4

TW

PH

5

SS

1

83.7

7.6

Het. Mixture of Silty Clay, Sand and Gravel (Glacial Till)

82.7

8.6

Grey, Soft to Firm

Auger Refusal

(Probable Boulder)

End of Borehole

6

SS

1

90

88

86

84

15.7

0 9 53 38

0 3 57 40

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 89-9

METRIC

W P 128-87-00 LOCATION Co-ords: N 5 012 555.4; E 360 985.4  
 DIST 9 HWY 416 BOREHOLE TYPE H S Auger & Cone Test ORIGINATED BY TS  
 DATUM Geodetic DATE 89 05 09-10 COMPILED BY TS  
 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60	W <sub>p</sub>	W	W <sub>L</sub>		
91.2 0.0	Ground Surface														
	Silty Clay		1	SS	2		90							17.2	0 3 54 43
	With occ. Sand		2	TW	PH										
	Seams, Trace		3	TW	PH		88								
	Organics		4	TW	PH										
	Grey, Soft to		5	TW	PH		86							17.7	0 9 54 37
	Stiff														
83.3 7.9	Met. Mixture of		6	SS	2		84								0 7 37 56
82.1	Silty, Clay, Sand &														
	Gravel (Glacial Till)														
9.1	Grey, Firm		7	SS	10		82								
	Met. Mixture of														
	Silt, Sand &														
	Gravel (Glacial Till)														
80.0	Compact		8	SS	24										
11.2	Auger Refusal													15 73 10 2	
	(Probable Boulder)														
	End of Borehole														

+3, +5: Numbers refer to  
Sensitivity

20  
15  
10

5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 89-9A

METRIC

W P 128-87-00

LOCATION Co-ords: N 5 012 482.0; E 361 016.0

ORIGINATED BY TS

DIST 9 HWY 416

BOREHOLE TYPE H S Auger & Cone Test

COMPILED BY TS

DATUM Geodetic

DATE 89 05 10

CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								20 40 60 80 100	20 40 60 80 100						
91.2	Ground Surface														
0.0															
	Silty Clay With occ. Sand Seams, Trace Organics Grey, Firm to Stiff		1	SS	2										
			2	SS	1										
85.1															
6.1	Het. Mixture of Silt, Sand and Gravel (Glacial Till)		3	SS	8										
	Loose to Dense		4	SS	30										
82.8															
8.4	Auger Refusal (Probable Boulder) End of Borehole														

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10



# RECORD OF BOREHOLE No 89-10

METRIC

W P 128-87-00

LOCATION Co-ords: N 5 012 612.9; E 360 993.5

DIST 9 HWY 416

BOREHOLE TYPE R S Auger

ORIGINATED BY TS

DATUM Geodetic

DATE 89 05 09

COMPILED BY TS

CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
91.3	Ground Surface																
0.0																	
	Silty Clay		1	SS	3		90									18.4	0 4 64 32
	With occ. Sand		2	TW	PH											17.4	
	Seams, Trace Organics		3	TW	PH		88									17.1	0 8 62 30
	Grey, Soft to Stiff		4	TW	PH		86									15.3	0 1 52 47
			5	TW	PH												
83.7	Het. Mixture of Silty Clay, Sand and Gravel (Glacial Till) Grey Firm		6	SS	3		84										1 6 47 46
7.6																	
82.8																	
8.5	Het. Mixture of Silt, Sand and Gravel (Glacial Till) Compact to Dense		7	SS	18		82										
80.5																	
10.8	End of Borehole		8	SS	30/3cm												27 42 25 6

+3, x5: Numbers refer to Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 15

1 OF 1

METRIC

W.P. 128-87-05/06 LOCATION Coords: N 5 012 823.4, E 360 905.9 ORIGINATED BY M.M.  
DIST 5 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring COMPILED BY M.M.  
DATUM Geodetic DATE 91 02 07 CHECKED BY B.I.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT 7 KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
95.4	Ground Surface													
0.0	Heterogeneous mixture of of Clay, Silt, Sand and Gravel (Glacial Till) Firm To Very Stiff		1	SS	10		95							4 11 55 30
92.5			2	SS	7		93							
2.9			3	SS	22									
			4	SS	29									
			5	SS	42		91							35 51 7 7
			6	SS	39									
			7	RC	REC	100%	89							RQD 96%
			8	SS	24		87							9 36 46 9
			9	RC	REC	84%	85							RQD 0%
			10	RC	REC	73%								RQD 51%
			11	RC	REC	38%	83							RQD 17%
80.9			12	RC	REC	100%	81							RQD 0%
14.5	Bedrock		13	RC	REC	98%								RQD 97%
79.4	Sandy Dolostone													
16.0	End of Borehole													

# RECORD OF BOREHOLE No 16

1 OF 1

METRIC

W.P. 128-87-05/05 LOCATION Coords: N 5 012 847.1, E 360 896.1 ORIGINATED BY M.M.  
DIST 6 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring COMPILED BY M.M.  
DATUM Geodetic DATE 91 02 07 CHECKED BY B.J.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
98.4	Ground Surface																
0.0			1	SS	0												
			2	SS	100	/15cm											
	Cloyey Silt with Sand and Gravel, Hard		3	SS	100	/25cm											
			4	SS	126												
			5	SS	120												
	Heterogeneous mixture of Silt, Sand and Gravel (Glacial Till)		6	SS	110												
	Trace Clay, Very Dense		7	SS	85												
			8	SS	85												
88.8			9	SS	100	/15cm											
9.6	End of Borehole																

RECORD OF BOREHOLE No 31

1 OF 1

METRIC

W.P. 128-87-05/06

LOCATION Coords: N 5 012 805.5, E 360 881.4

ORIGINATED BY M.M.

DIST 6 HWY 416

BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring

COMPILED BY M.M.

DATUM Geodetic

DATE 91 02 07

CHECKED BY B.I.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
95.0	Ground Surface													
0.0	Clayey Silt with Sand and Gravel		1	SS	7									
92.8	Trace Organics, Firm		2	SS	6									
2.1	Heterogeneous mixture of Silt, Sand and Gravel Trace Clay Dense to very Dense (Glacial Till)		3	SS	35									40 43 13 4
			4	SS	33									52 38 7 3
			5	SS	44									
			6	SS	28									
	Sandy Silt		7	SS	46									3 13 74 10
			8	SS	8									
	Some boulders		9	RC	REC 37%									RQD 23%
			10	SS	31									
			11	RC	REC 31%									RQD 0%
			12	RC	REC 38%									RQD 0%
80.5														
79.8	Bedrock Dolomitic Sandstone		13	RC	REC	83%								RQD 26%
15.1	End of Borehole													

RECORD OF BOREHOLE No 32

1 OF 1

METRIC

W.P. 128-87-05/06

LOCATION Coords: N 5 012 829.3, E 360 871.6

ORIGINATED BY M.M.

DIST 9 HWY 416

BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring

COMPILED BY M.M.

DATUM Geodetic

DATE 90 02 03

CHECKED BY B.L.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
98.8	Ground Surface																
0.0	Heterogeneous mixture of Silt, Sand and Gravel Trace Clay  Compact to Very Dense (Glacial Till)		1	SS	23		98										61 22 12 5
			2	SS	22	/8cm											
			3	SS	46		96										RQD 37%
	Boulders		4	RC	REC	53%											RQD 0%
	Quartz Sandstone		5	RC	REC	15%											RQD 0%
			6	RC	REC	45%											RQD 0%
			7	SS	18	/15cm											RQD 0%
			8	RC	REC	36%											
	Some Silt		9	SS	46		92									23.1	13 23 51 13
			10	SS	92												
			11	SS	126		90										56 33 9 2
			12	SS	32		88										
			13	RC	REC	38%											RQD 0%
			14	SS	52		86										
			15	SS	29		84										69 22 8 1
82.9			16	SS	100	/8cm											
15.8	Bedrock		17	RC	REC	78%	82										RQD 0%
	Sandy Dolostone		18	RC	REC	96%											RQD 40%
80.2																	
18.5	End of Borehole																

# **ROCK CORE DESCRIPTION** **WP 128-87-05/06**

Page 1 of 2

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
15	7	5.94-6.55	100	96	5.94-14.43	OVERBURDEN (boulder till).
	9	10.06-10.54	84	0	14.43-15.70	DOLOMITIC SANDSTONE, light grey to medium light grey; fine to coarse grained; medium strong; unweathered to slightly weathered; fractures wide spaced, flat, undulating, smooth to rough.
	10	10.54-12.04	73	51		
	11	12.19-13.72	38	17		
	12	14.02-14.43	100	0	15.70-15.95	SANDY DOLOSTONE, light grey to medium dark grey; fine crystalline; medium strong; unweathered to slightly weathered; fractures close to extremely close spaced, flat to near vertical, undulating to planar, smooth.
	13	14.43-15.95	98	97		
31	9	8.53-10.06	37	23	8.53-14.48	OVERBURDEN (boulder till).
	11	10.52-11.89	31	0	14.48-15.04	DOLOMITIC SANDSTONE, light grey to medium light grey; medium to coarse grained; medium strong; unweathered to slightly weathered; fractures close to very close spaced, flat to near vertical, planar to undulating, smooth to rough.
	12	12.65-14.17	38	0		
	13	14.17-15.09	83	26	15.04-15.09	DOLOSTONE, medium grey to medium dark grey; fine crystalline; medium strong; unweathered to slightly weathered; fractures close spaced, flat to near vertical, planar to undulating, smooth.

\*CR = CORE RECOVERY

\*RQD = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section

# **ROCK CORE DESCRIPTION** **WP 128-87-05/06**

Page 2 of 2

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
32	4	3.05-3.81	53	37	3.05-15.85	OVERBURDEN (boulder till).
	5	3.81-4.57	15	0	15.85-18.11	QUARTZ SANDSTONE, very light grey to light grey; fine to medium grained; medium strong; unweathered to slightly weathered; fractures very close to moderately close spaced, flat, planar to undulating, smooth.
	6	4.57-6.10	45	0		
	8	6.10-7.01	36	0		SANDY DOLOSTONE, light grey to medium dark grey; fine crystalline; medium strong; unweathered to slightly weathered; fractures extremely close to close spaced, flat, undulating to planar, smooth.
	13	12.19-12.80	38	0	18.11-18.59	
	17	15.85-17.37	78	0		
	18	17.37-18.59	96	40		

\*CR = CORE RECOVERY

\*RQD = ROCK QUALITY DESIGNATION

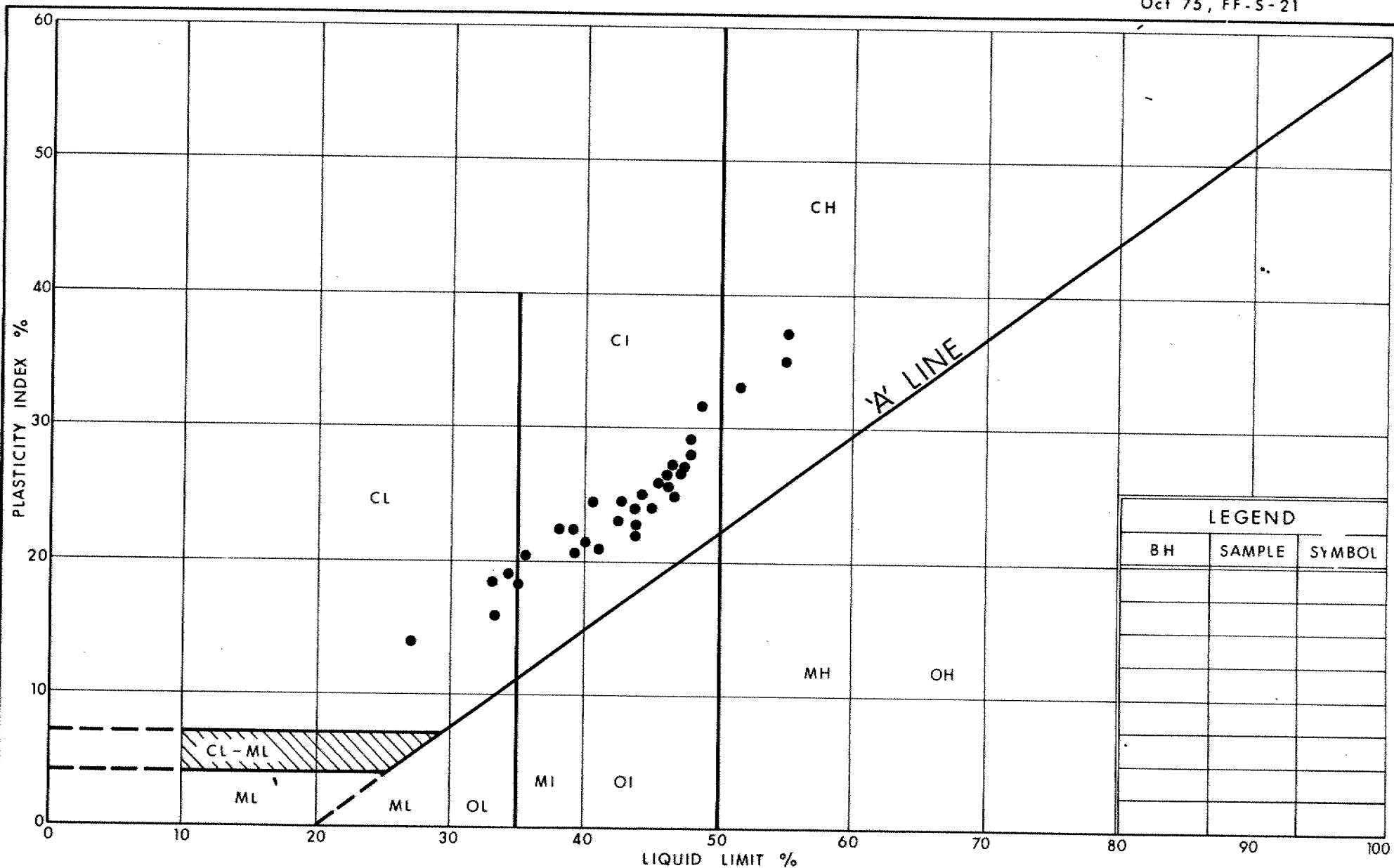
(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section

**APPENDIX F**

**Laboratory Test Results  
(1989 and 1991 Investigations)**





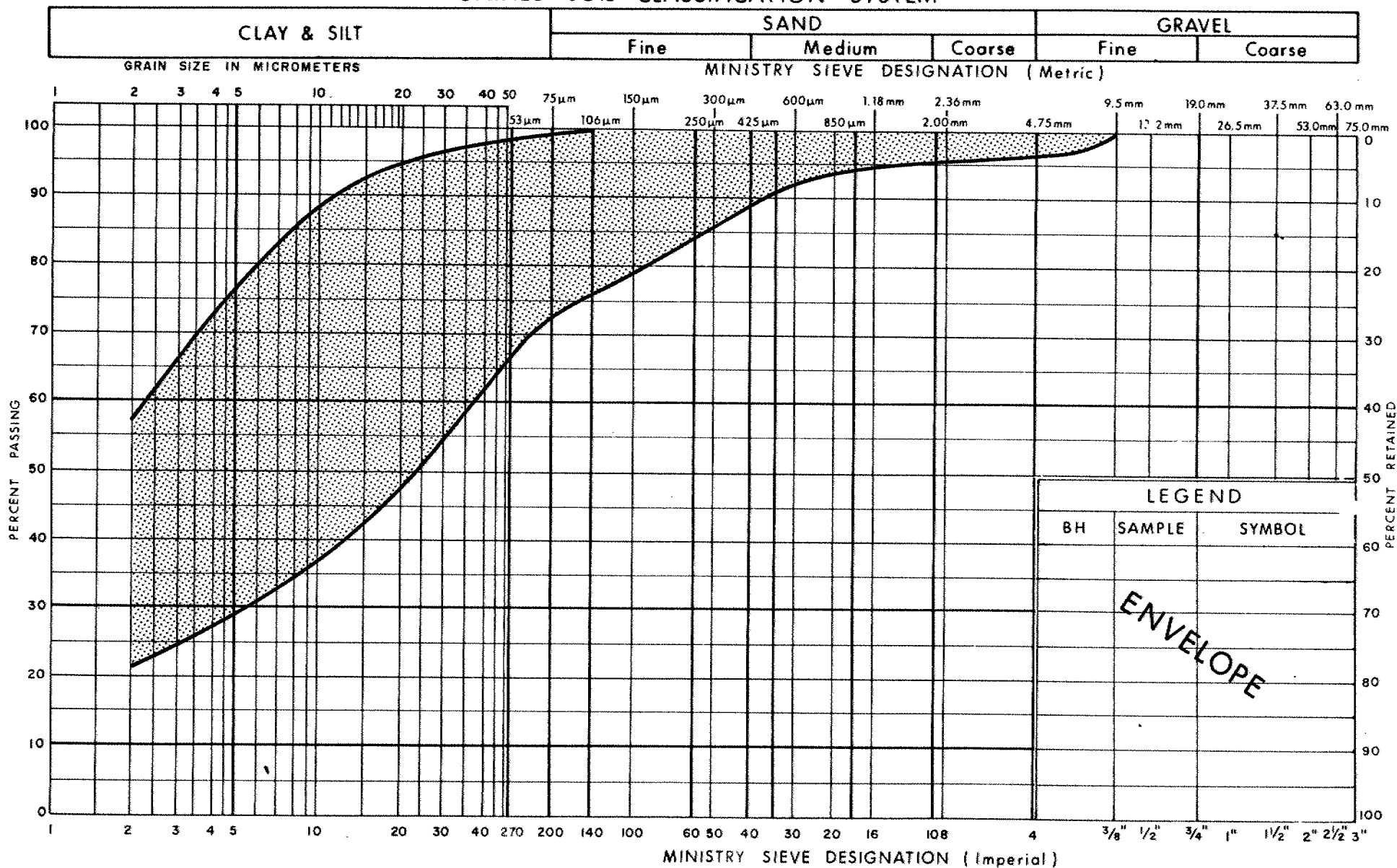
Ministry of  
Transportation

## PLASTICITY CHART SILTY CLAY

FIG No 1

W P 128-87-05/06

## UNIFIED SOIL CLASSIFICATION SYSTEM

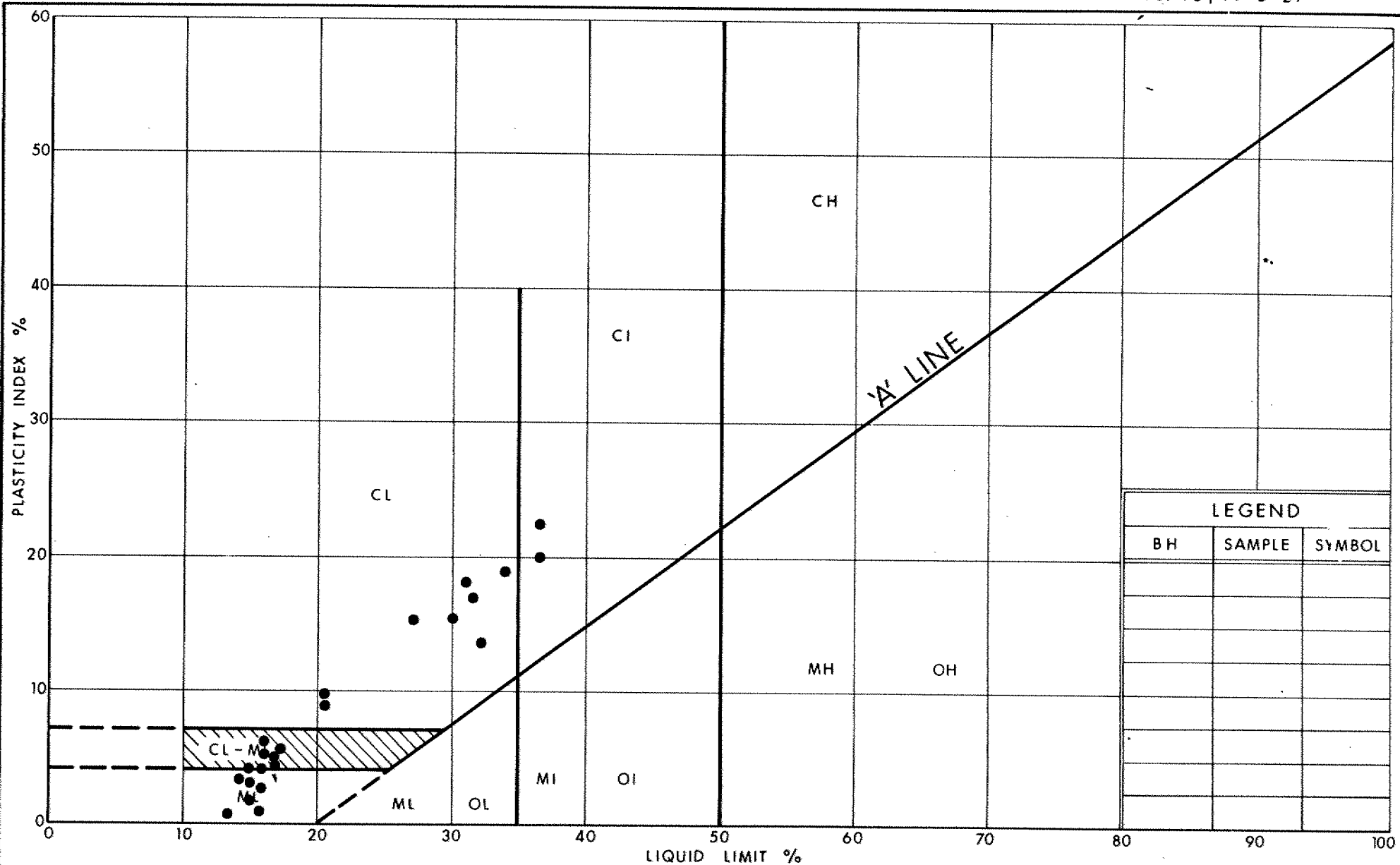


Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
SILTY CLAY

FIG No 2

W P 128-87-05/06



Ministry of  
Transportation

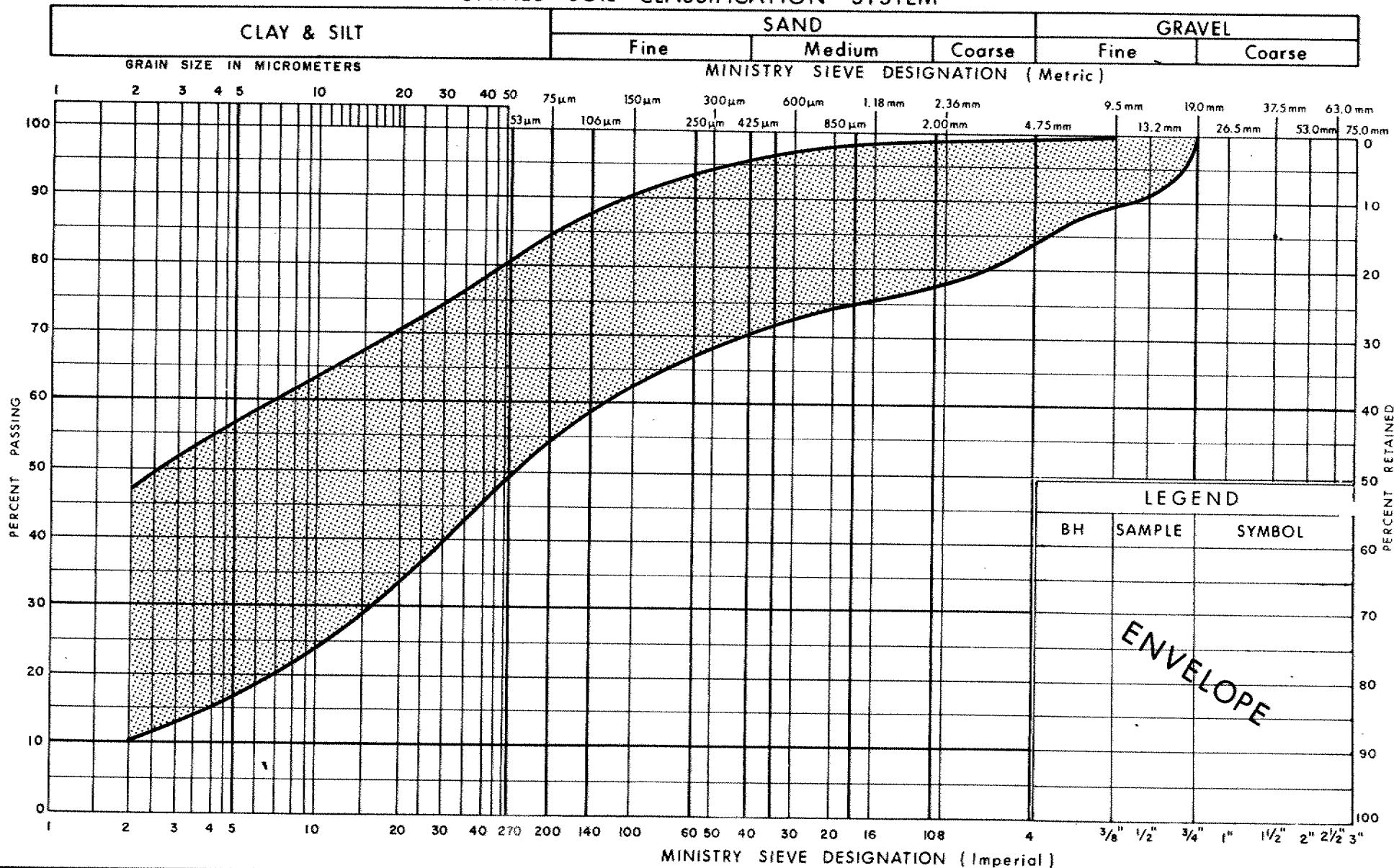
Ontario

# PLASTICITY CHART COHESIVE HET MIXTURE OF CLAYEY SILT TO SILT, SAND & GRAVEL

FIG No 3

W P 128-87-05/06

## UNIFIED SOIL CLASSIFICATION SYSTEM



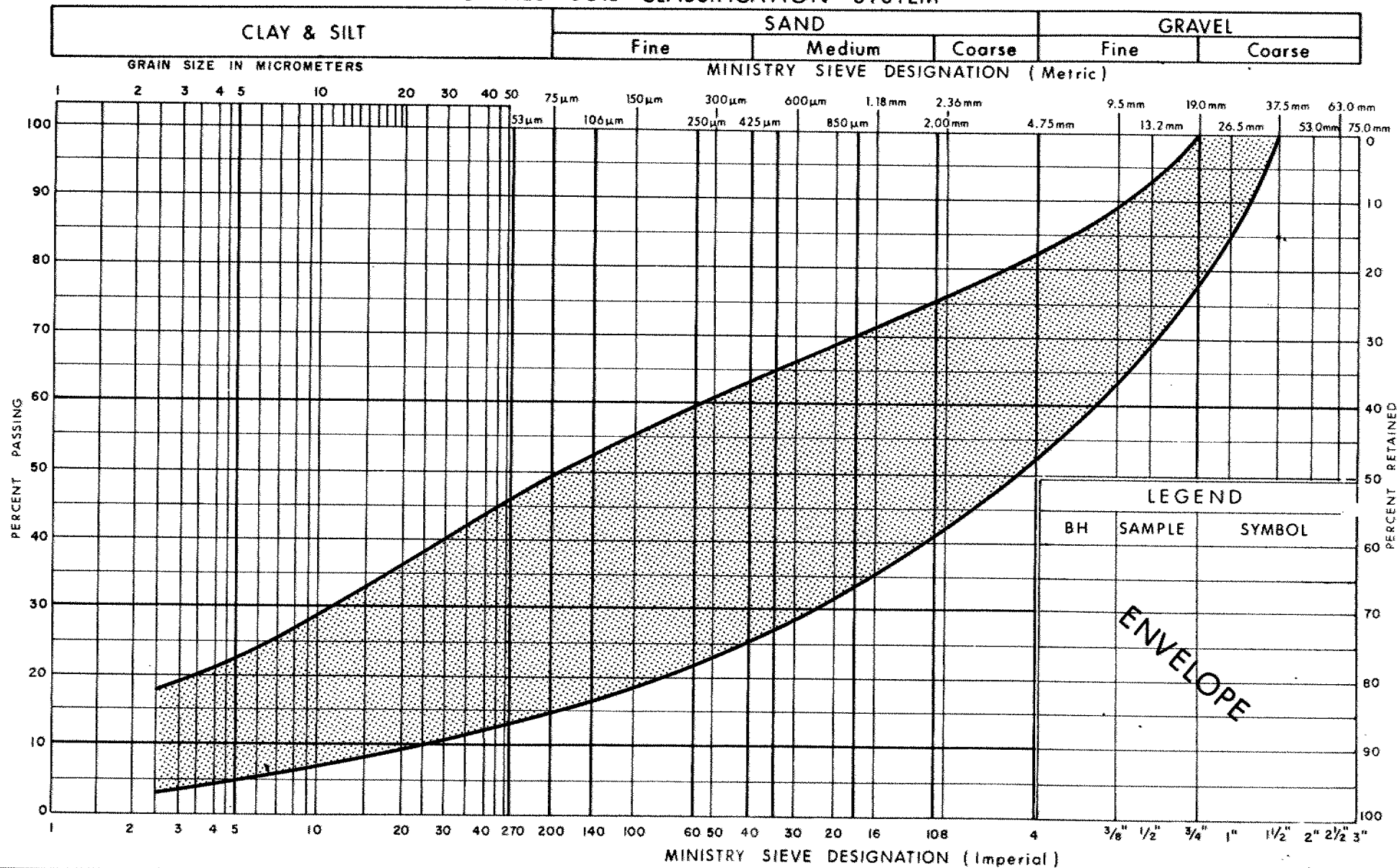
Ministry of  
Transportation

**GRAIN SIZE DISTRIBUTION**  
COHESIVE HET MIXTURE OF  
CLAYEY SILT TO SILT, SAND & GRAVEL

FIG No 4

W P 128-87-05/06

## UNIFIED SOIL CLASSIFICATION SYSTEM



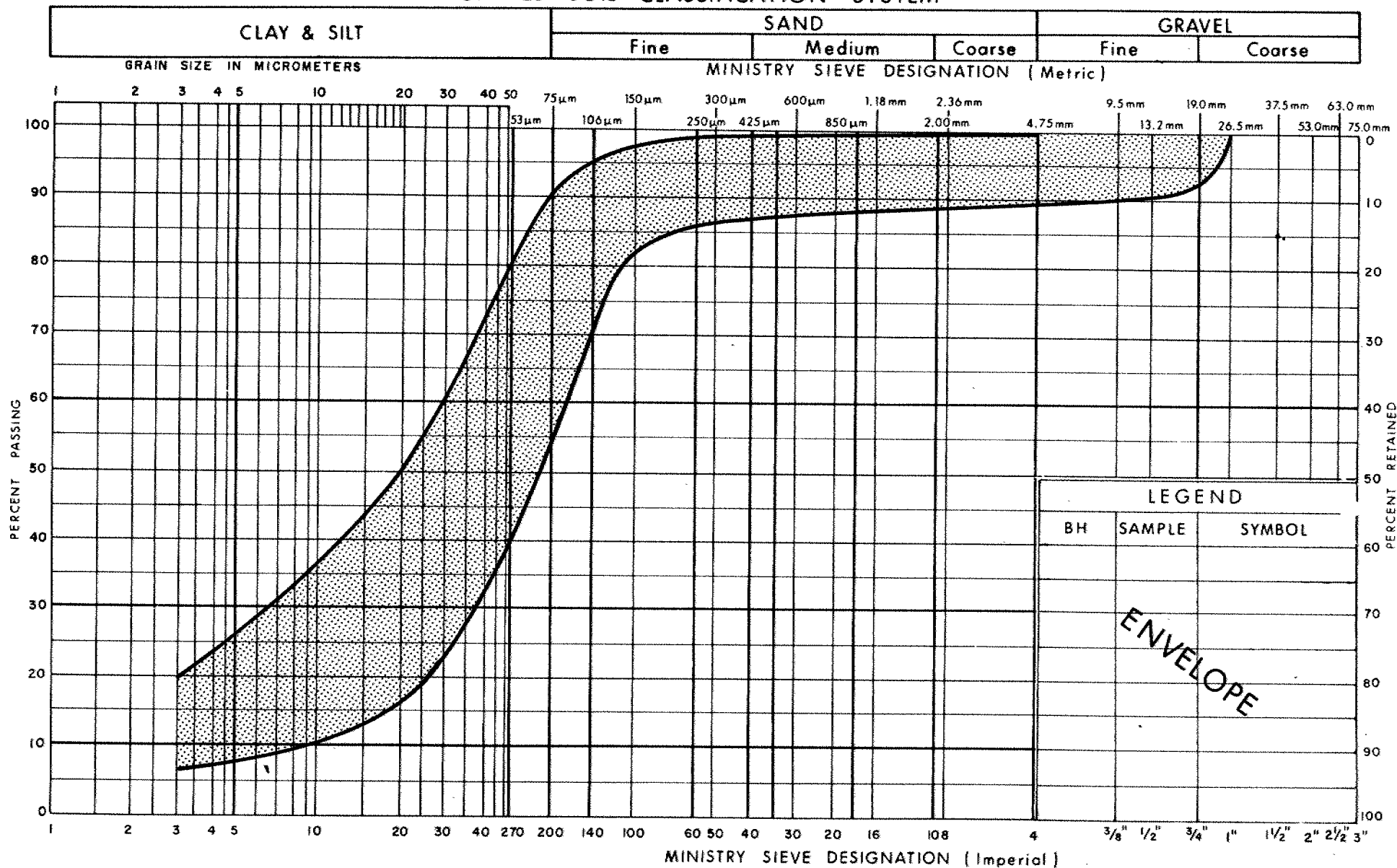
Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
GRANULAR HET MIXTURE OF  
SILT, SAND & GRAVEL

FIG No 5

W P 128-87-05/06

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

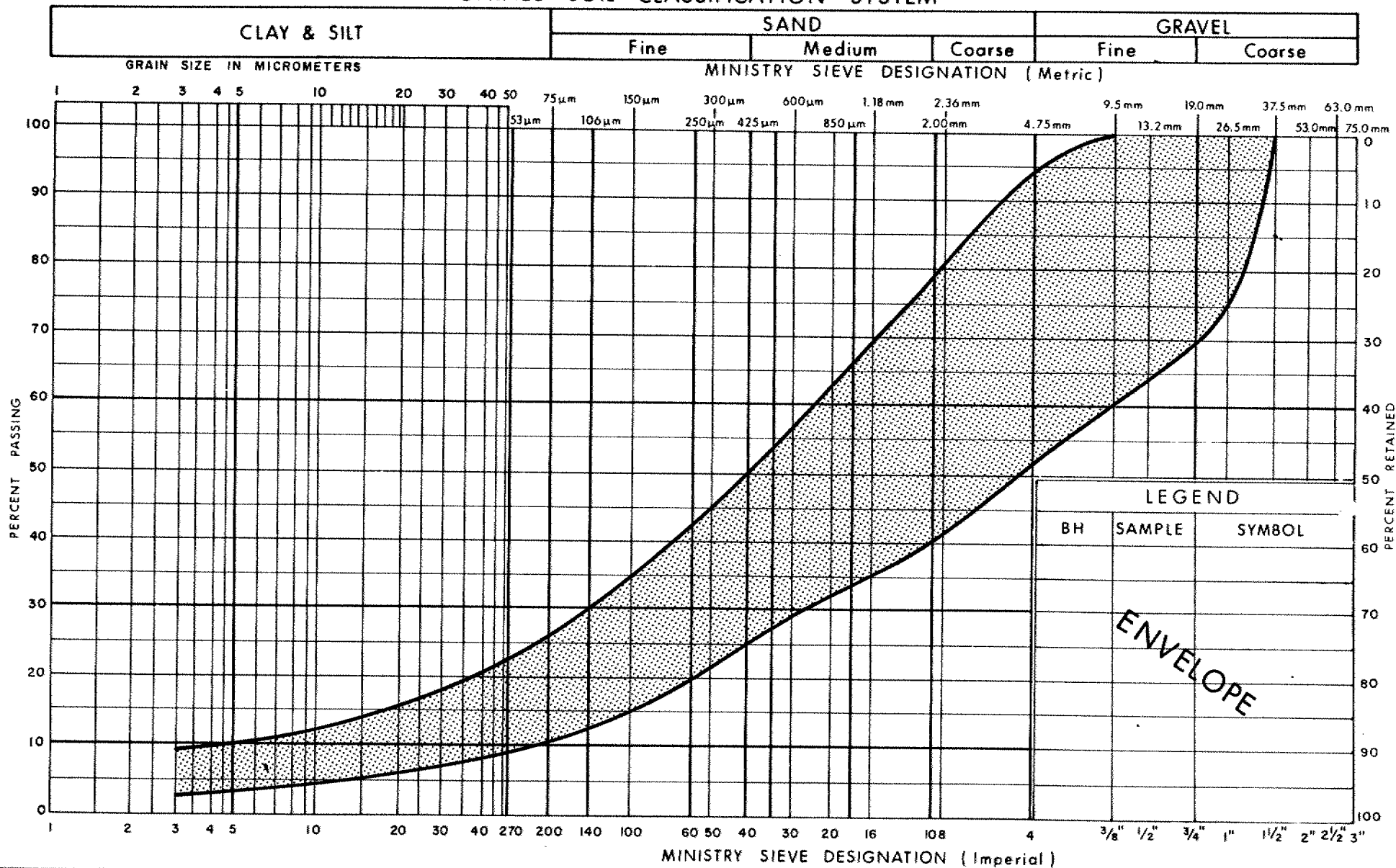
Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
SANDY SILT, SOME CLAY TRACE OF GRAVEL

FIG No 6

W P 128-87-05/06

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
SAND WITH GRAVEL, SOME SILT

FIG No 7

W P 128-87-05/06

# VOID RATIO - PRESSURE CURVES

23

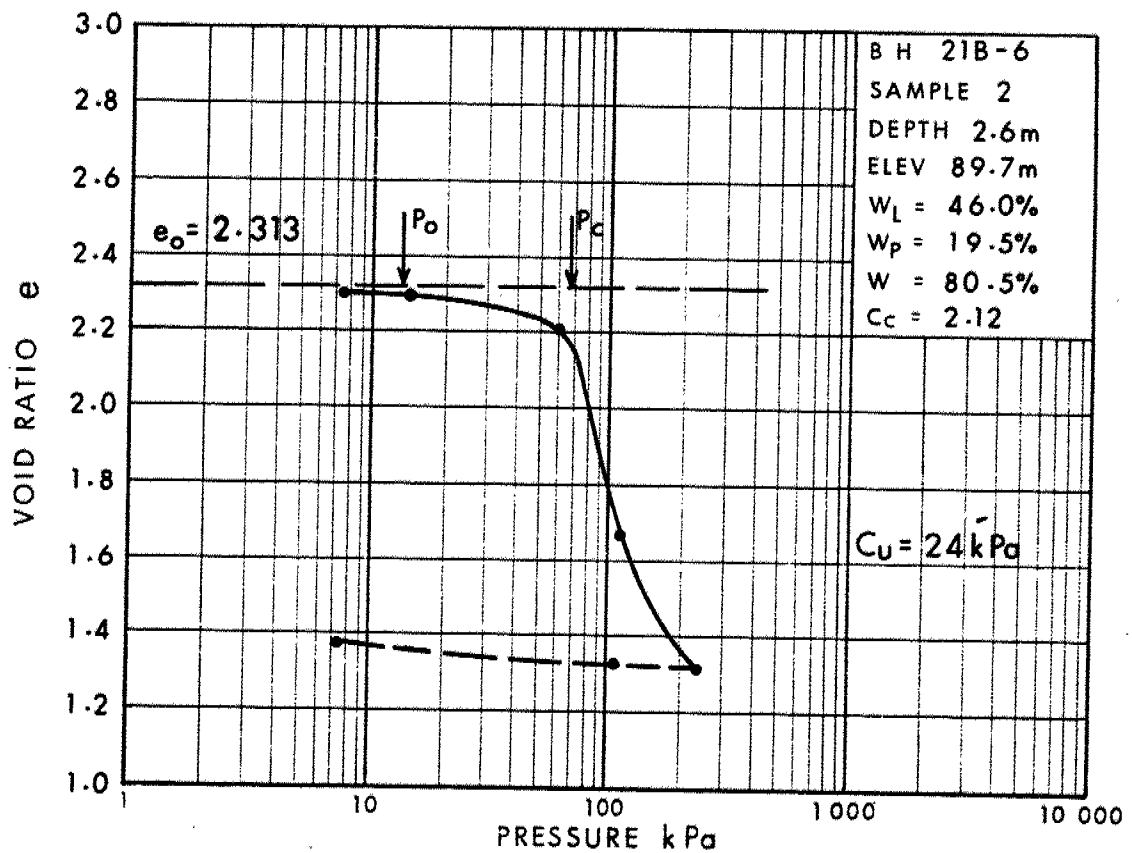
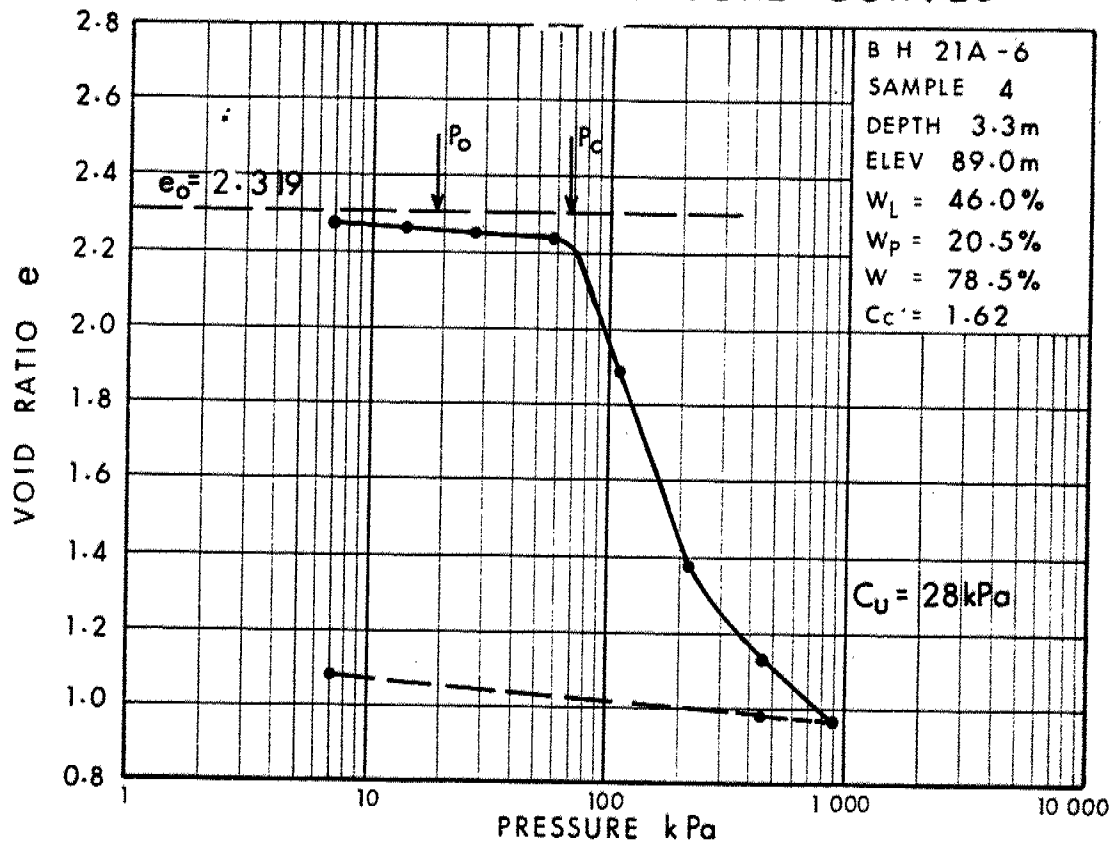


Fig 8

W P 128-87-05/06



## VOID RATIO - PRESSURE CURVES

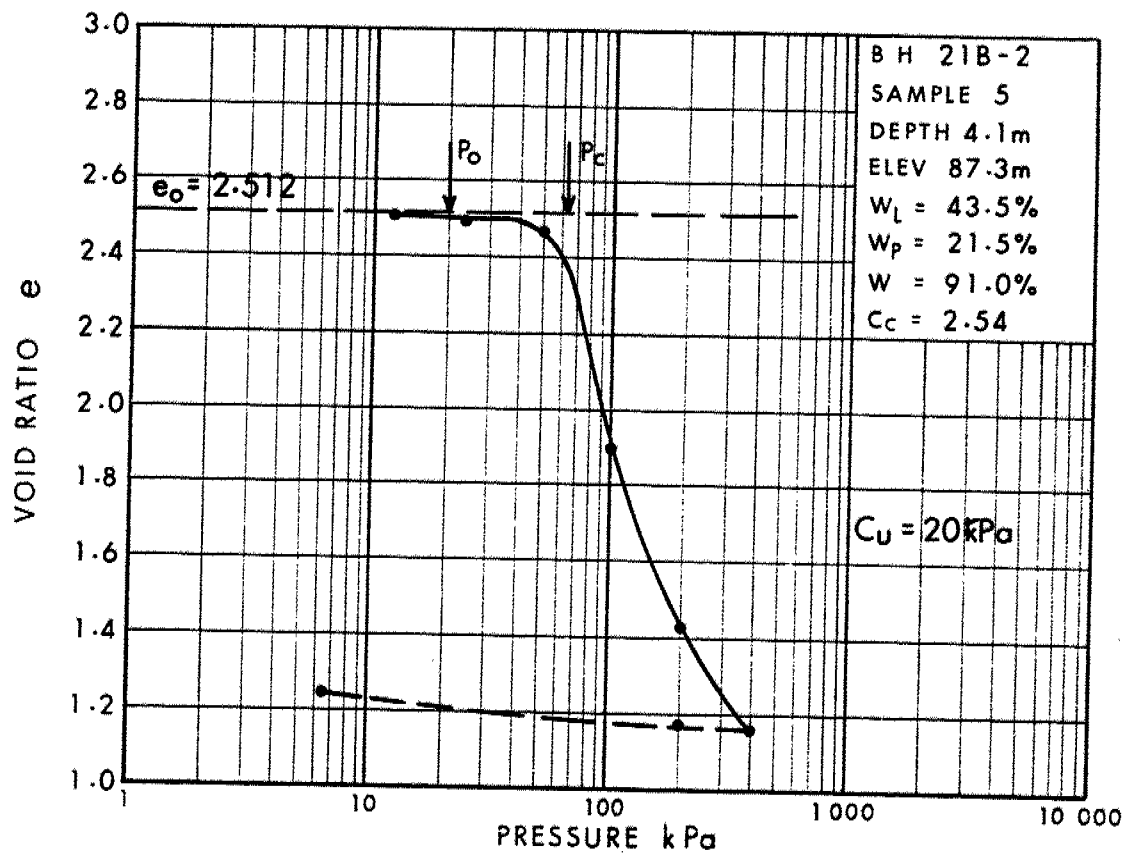
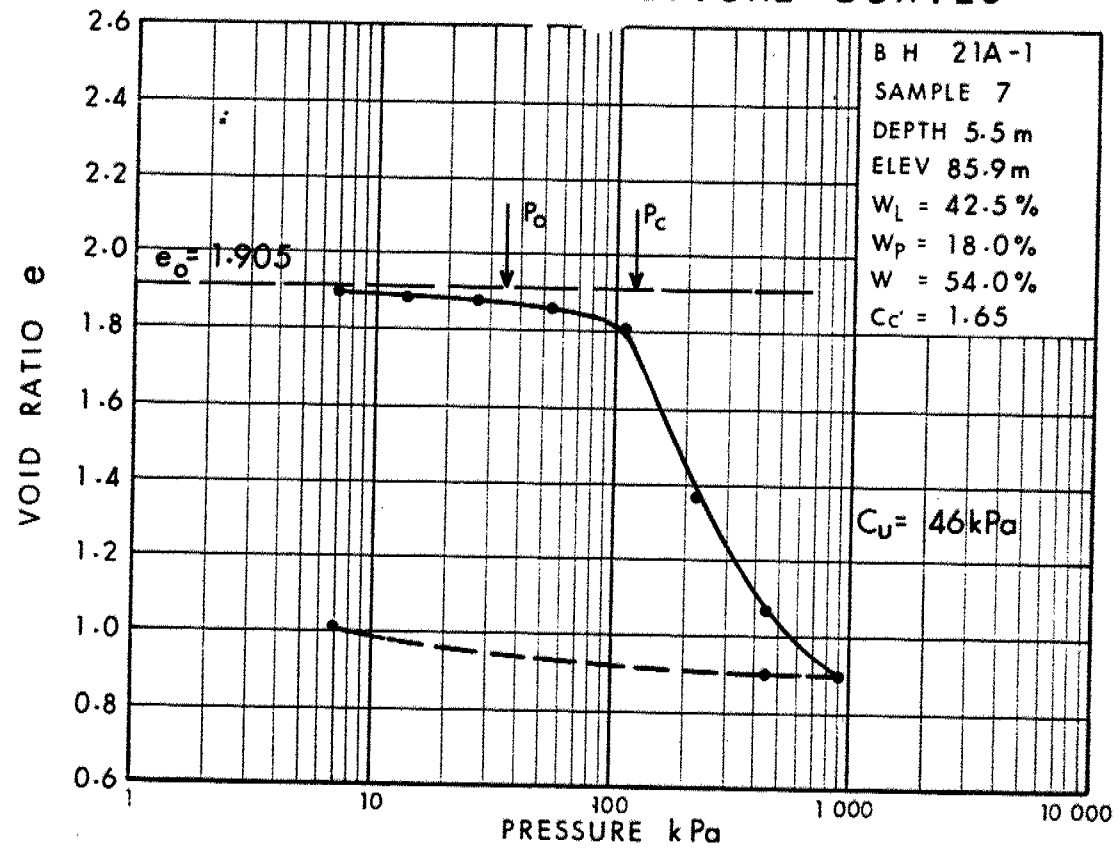
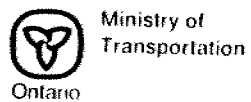


Fig 9

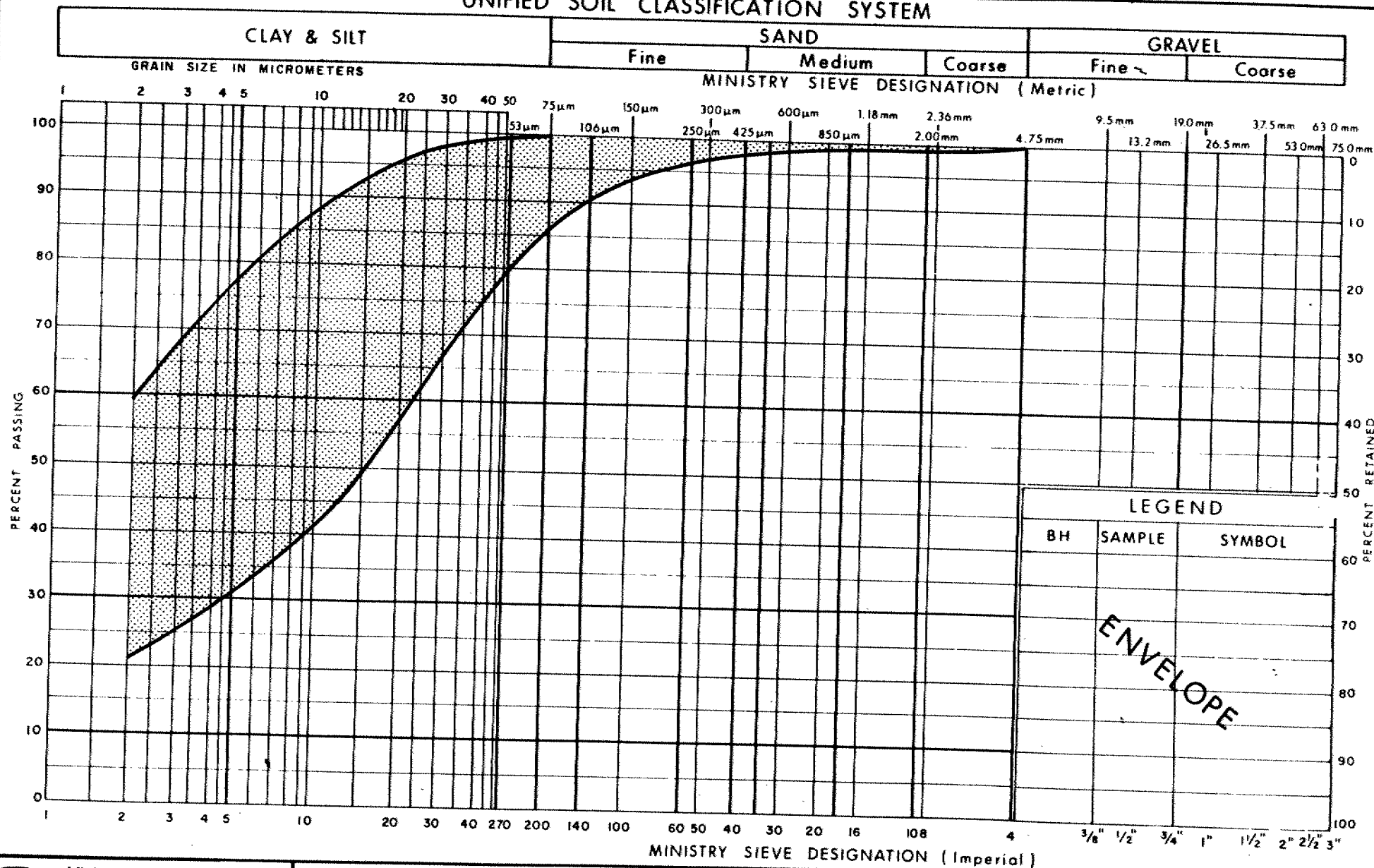
W P 128-87-05/06



WITH OCC SAND SEAMS, TRACE OF ORGANICS

W P 128-87-00

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
SILTY CLAY  
WITH OCC SAND SEAMS, TRACE OF ORGANICS

FIG No 7

W P 129-87-00

# VOID RATIO - PRESSURE CURVES

106

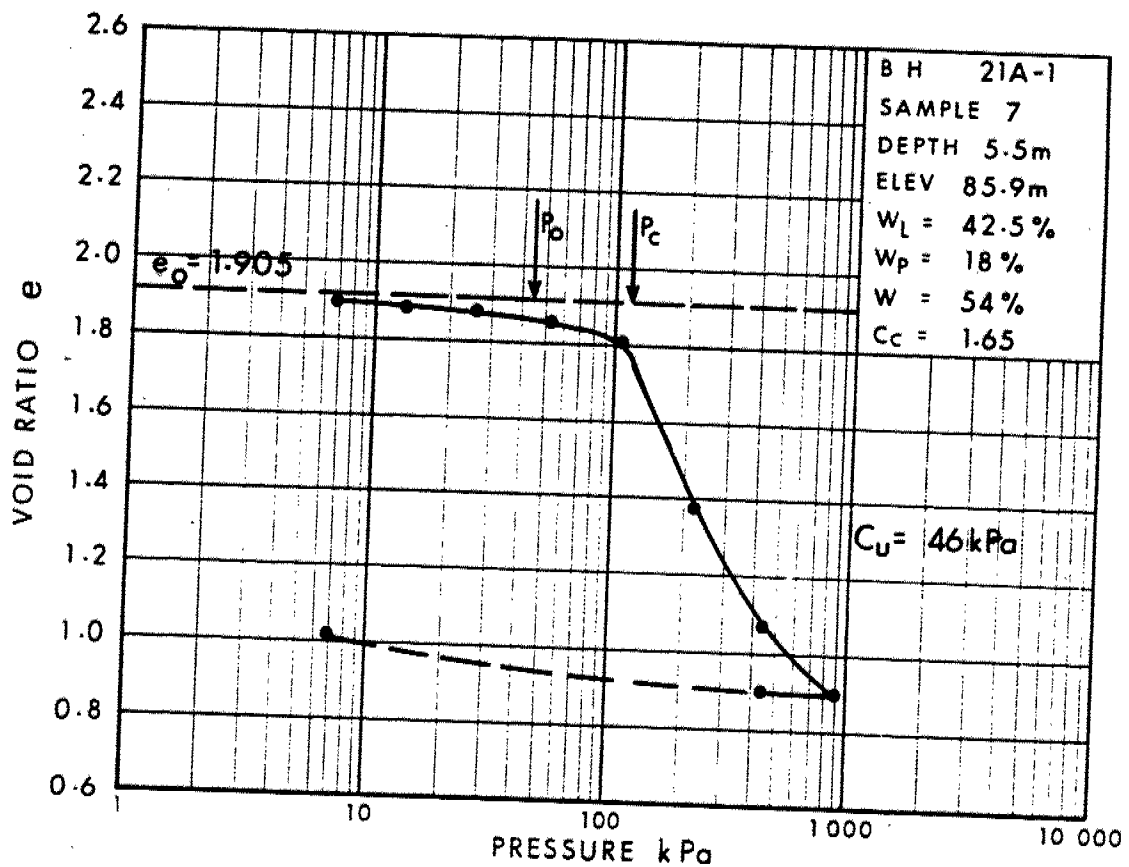
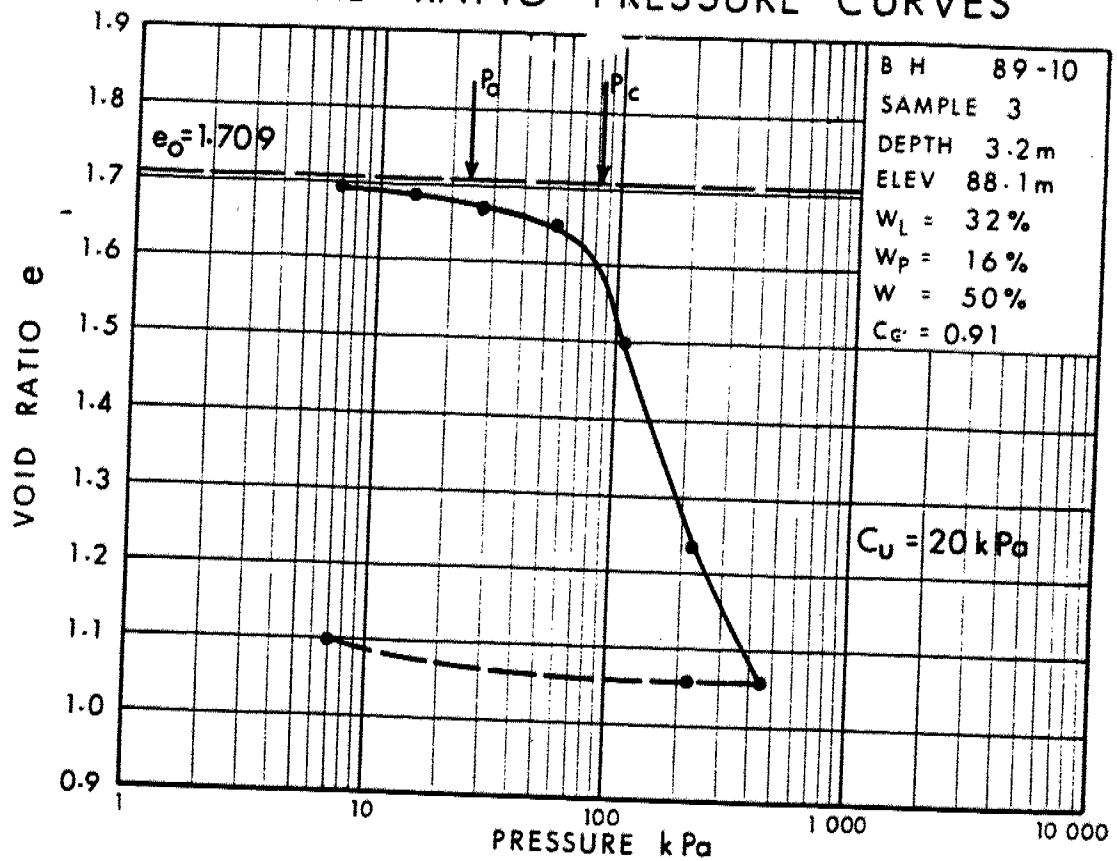


Fig 8 FILL AREA -1

W P 128-87-00

# VOID RATIO - PRESSURE CURVES

107

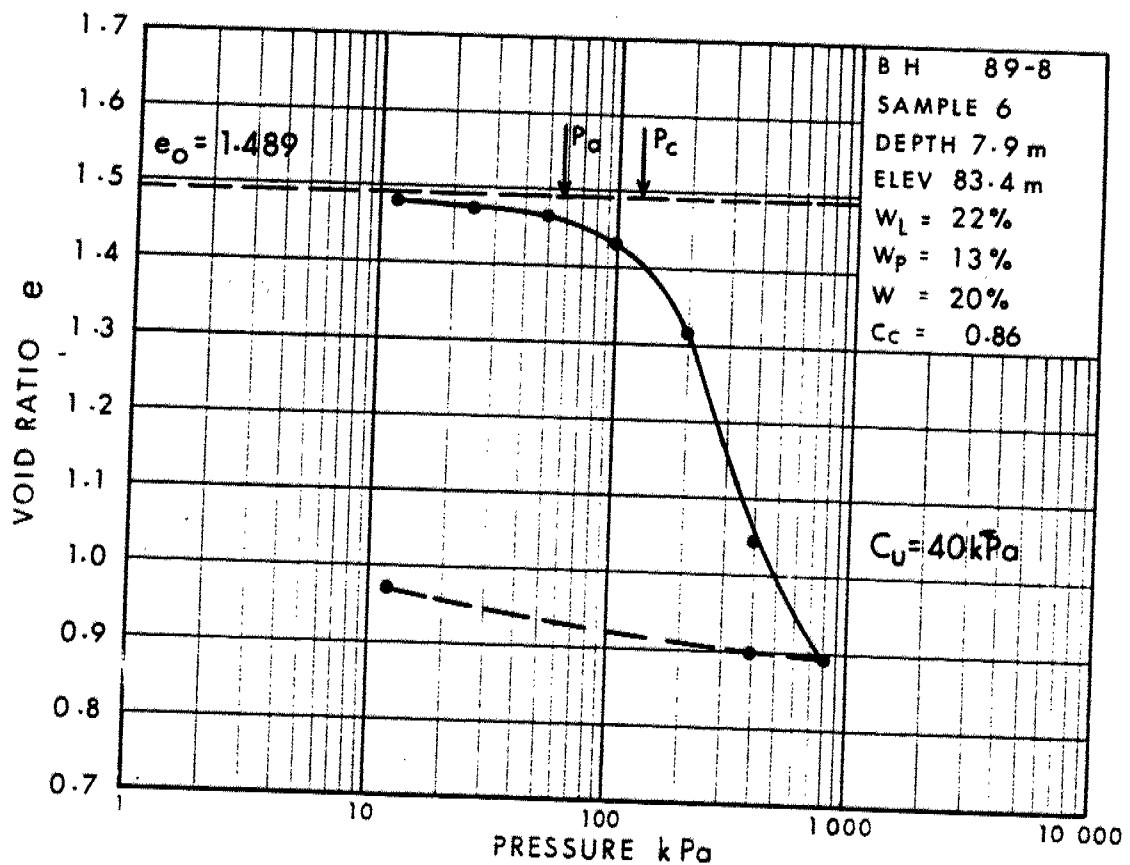
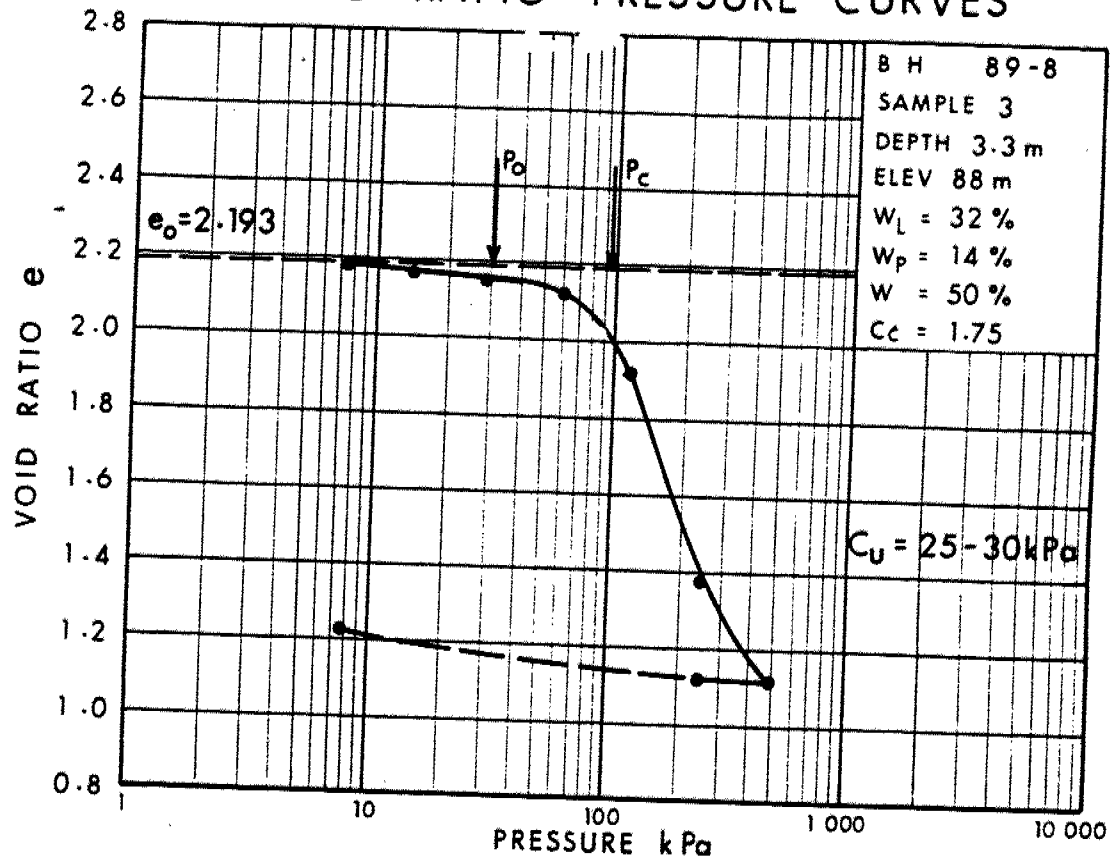


Fig 9- FILL AREA-2

W P 128-87-00

## VOID RATIO - PRESSURE CURVES

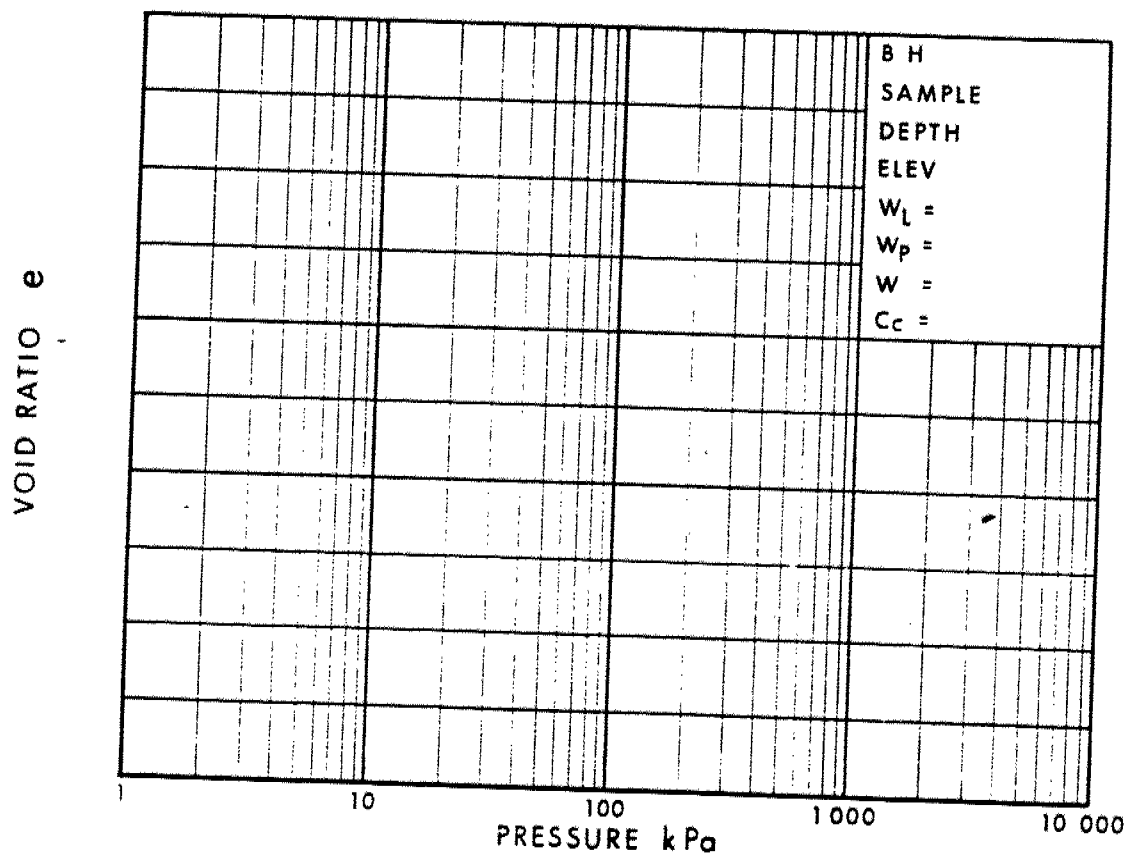
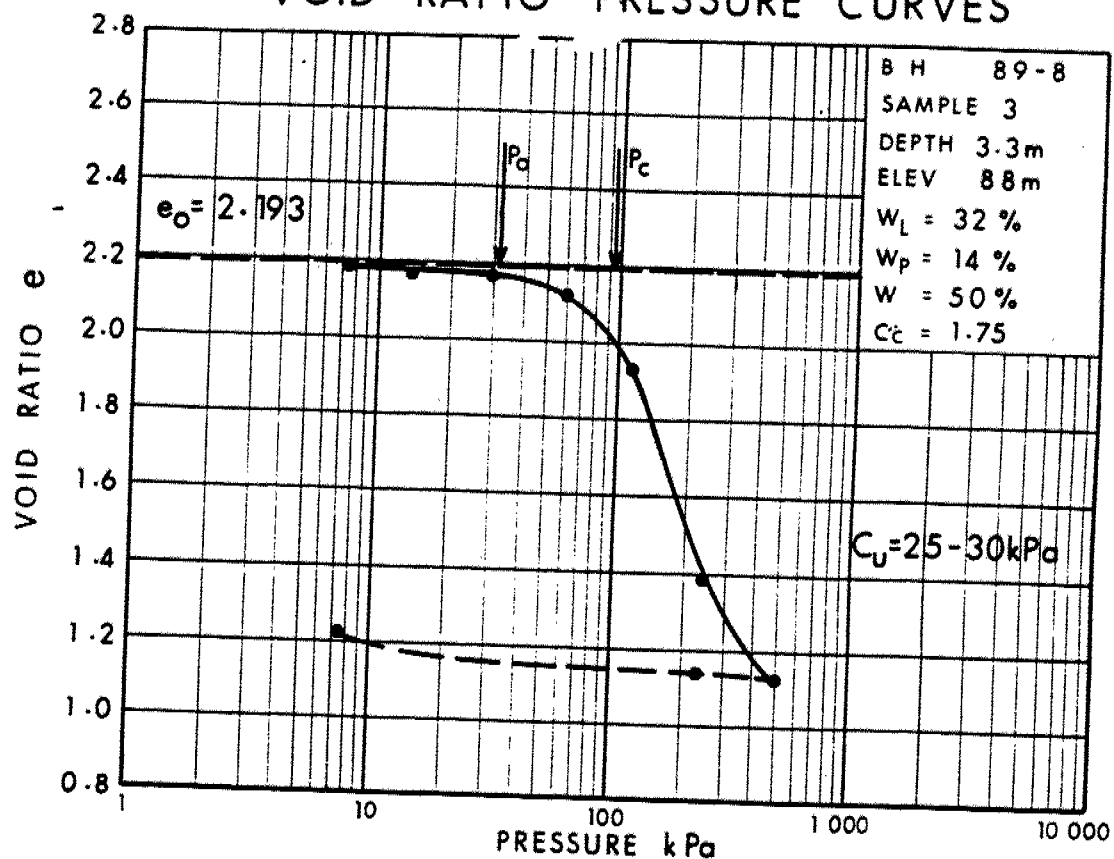


Fig 9a FILL AREA-2A W P 128-87-00

# VOID RATIO - PRESSURE CURVES

109

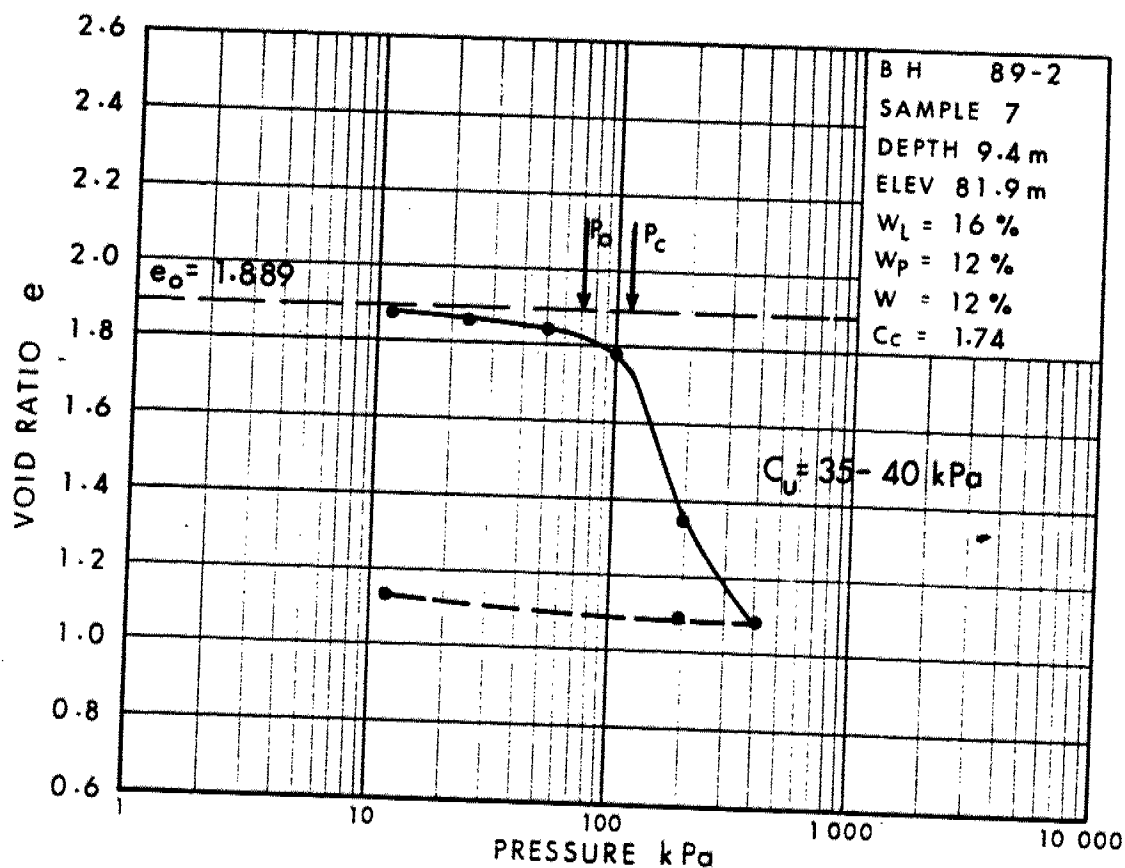
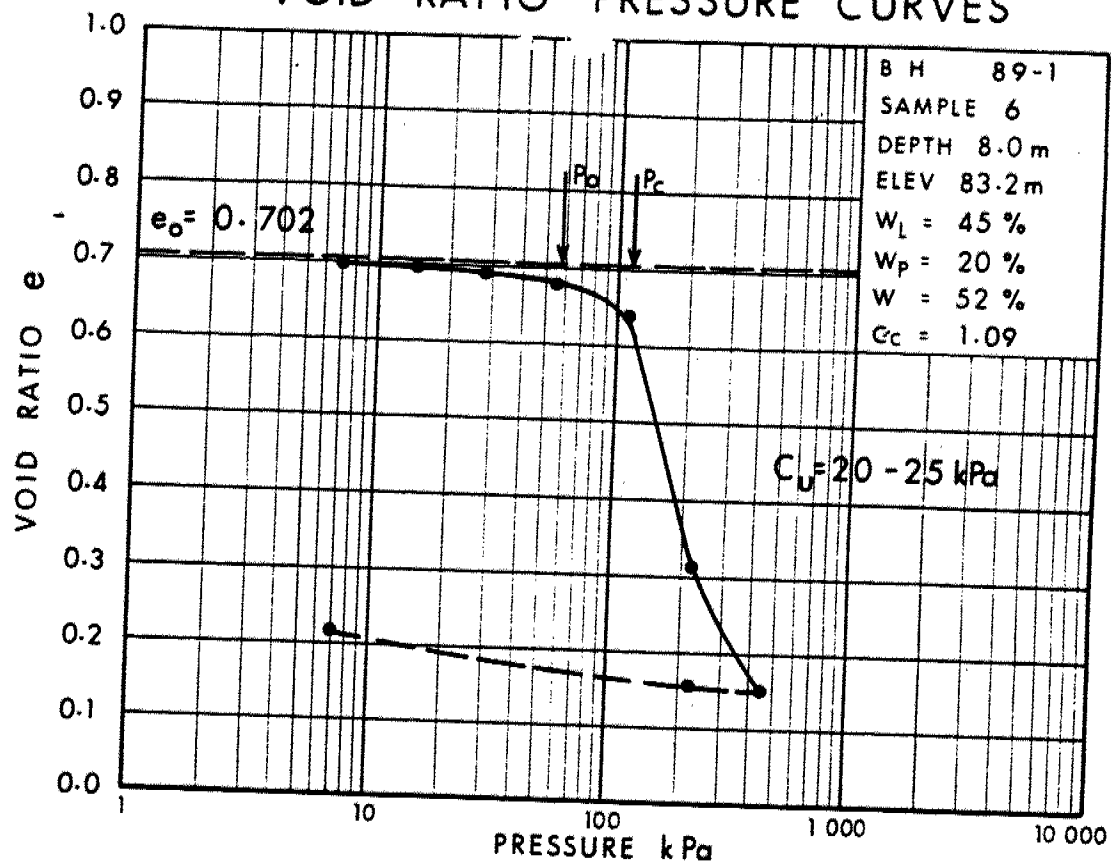


Fig 10 - FILL AREA-3

WP 128-87-00

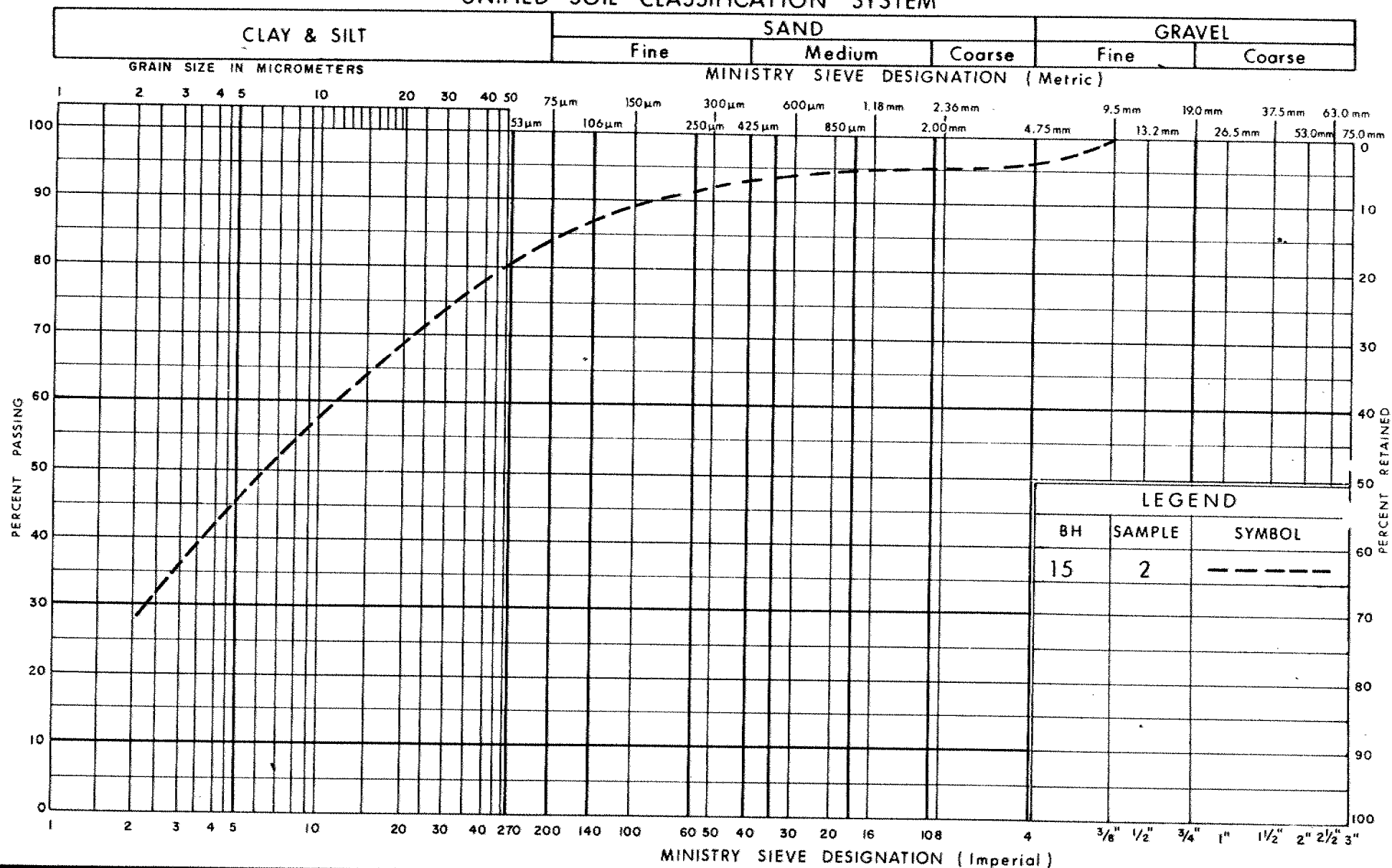


### GRAIN SIZE DISTRIBUTION HETEROGENEOUS MIXTURE OF SILT, SAND & GRAVEL

W P 128-87-00



## UNIFIED SOIL CLASSIFICATION SYSTEM

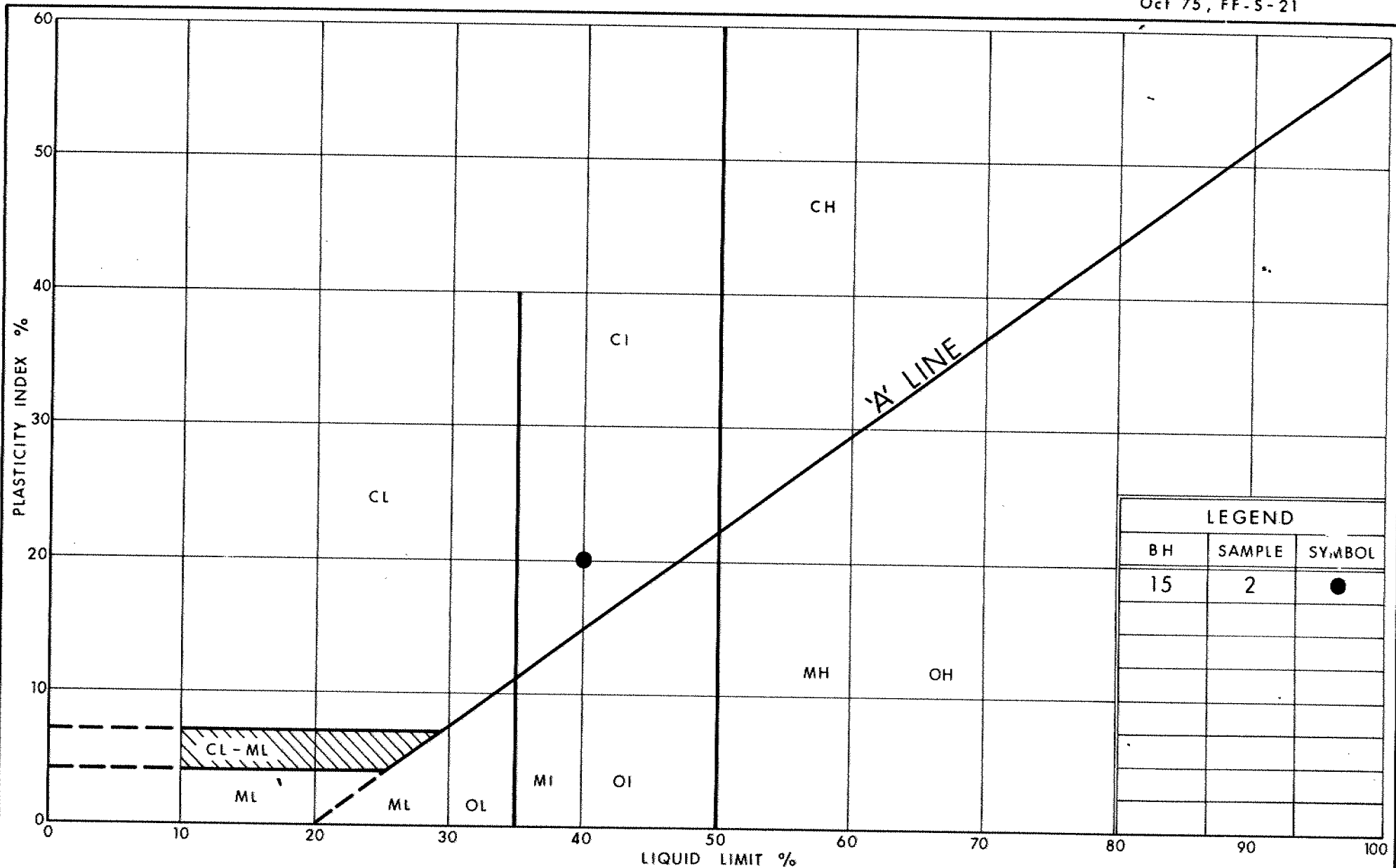


Ministry of  
Transportation

**GRAIN SIZE DISTRIBUTION**  
**CLAYEY SILT**  
TRACE SAND, GRAVEL, INCLUSION OF ORGANICS

FIG No 18

W P 128-87-05/06



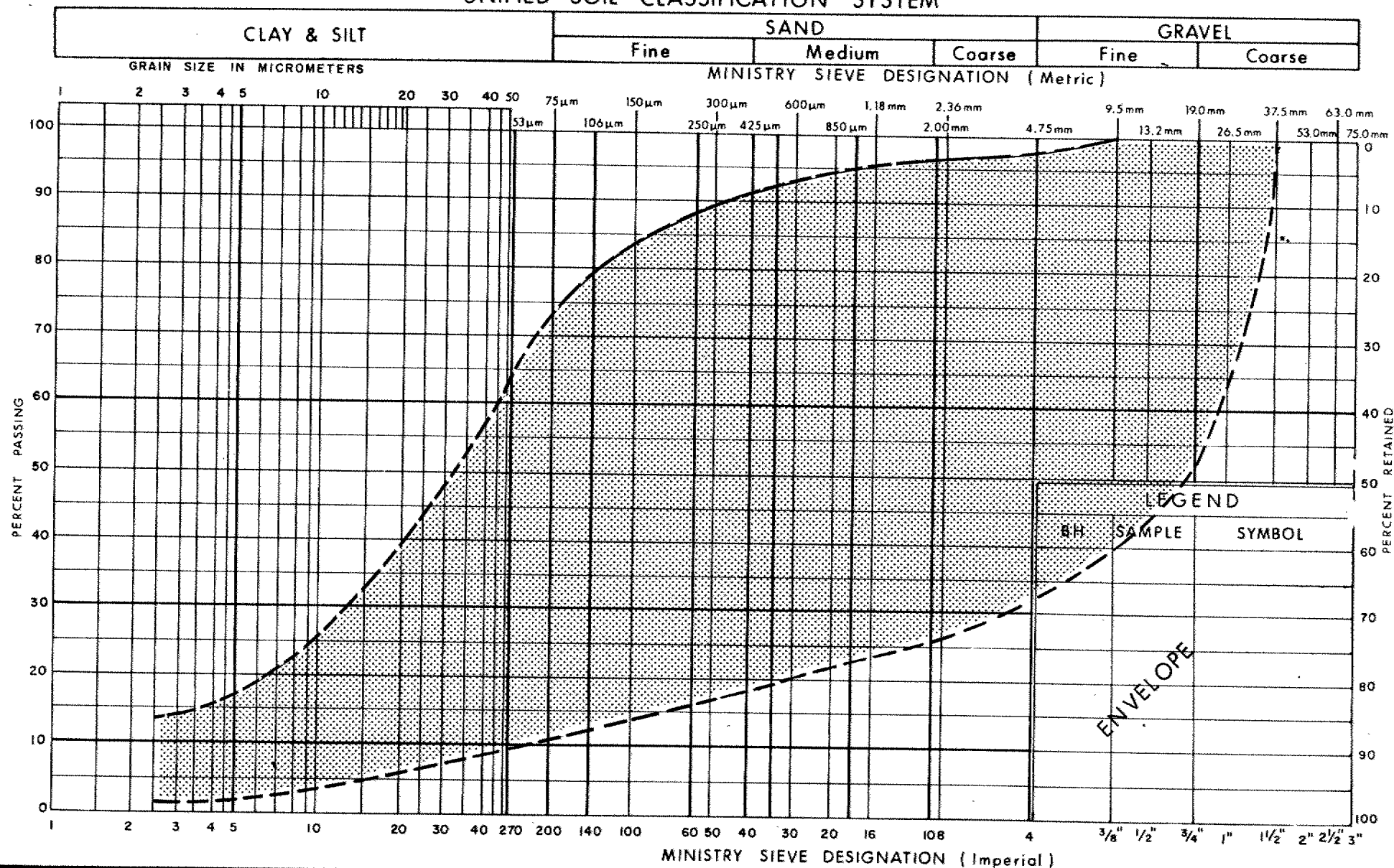
Ministry of  
Transportation

# PLASTICITY CHART CLAYEY SILT TRACE SAND, GRAVEL, INCLUSION OF ORGANICS

FIG No 19

W P 128-87-05/06

## UNIFIED SOIL CLASSIFICATION SYSTEM

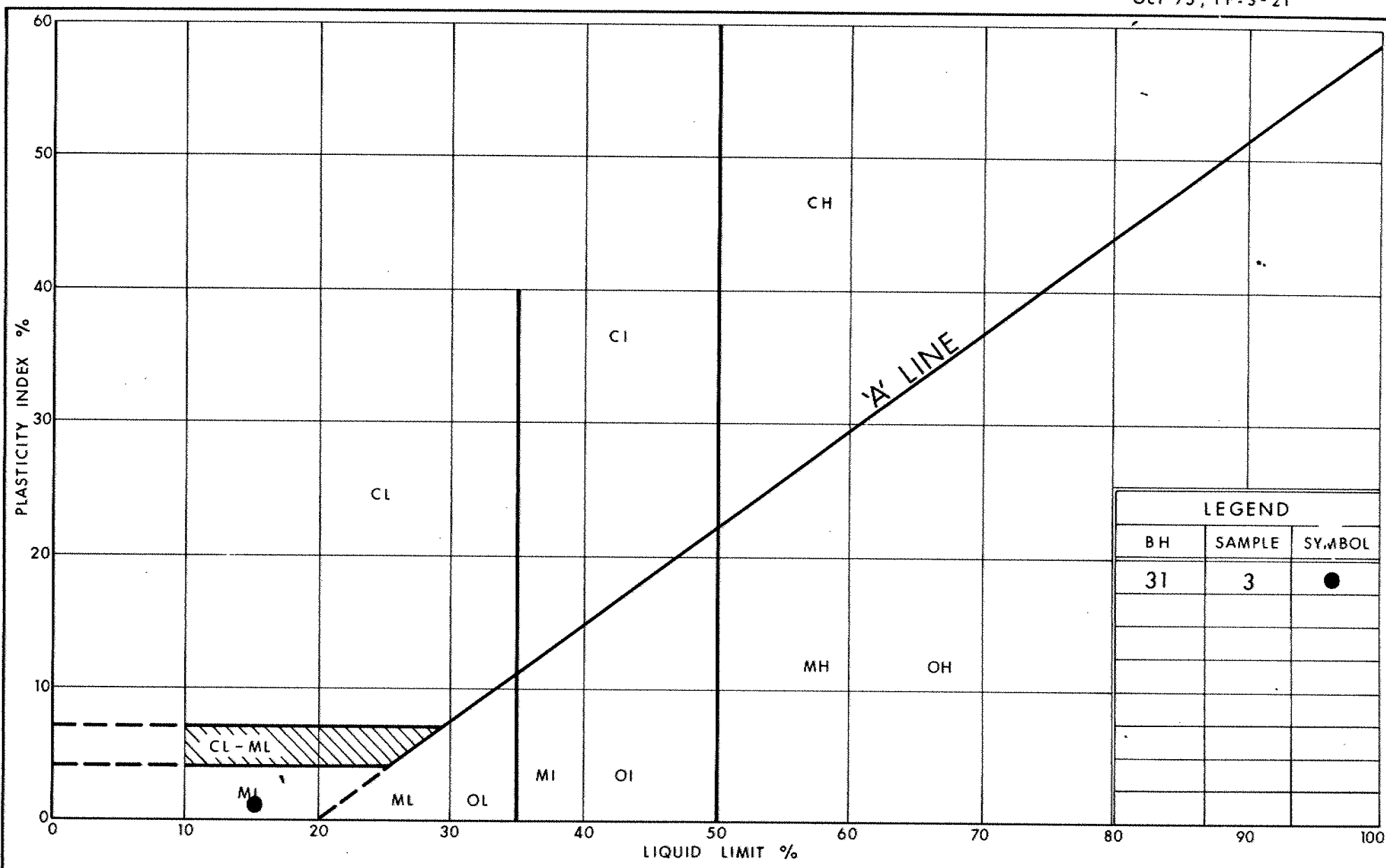


Ministry of  
Transportation

**GRAIN SIZE DISTRIBUTION**  
HETEROGENEOUS MIXTURE OF  
**SILT, SAND AND GRAVEL,**  
TRACE CLAY

FIG No 20

W P 128-87-05/06



LEGEND		
BH	SAMPLE	SYMBOL
31	3	●

**APPENDIX G**

**Vertical Seismic Profile Testing Results  
(Golder, 2018)**

## TECHNICAL MEMORANDUM

**DATE** July 6, 2018

**Project No.** 1417217/1231

**TO** Alex Meacoe, Golder Associates Ltd

**FROM** Stephane Sol, Christopher Phillips

**EMAIL** ssol@golder.com, cphillips@golder.com

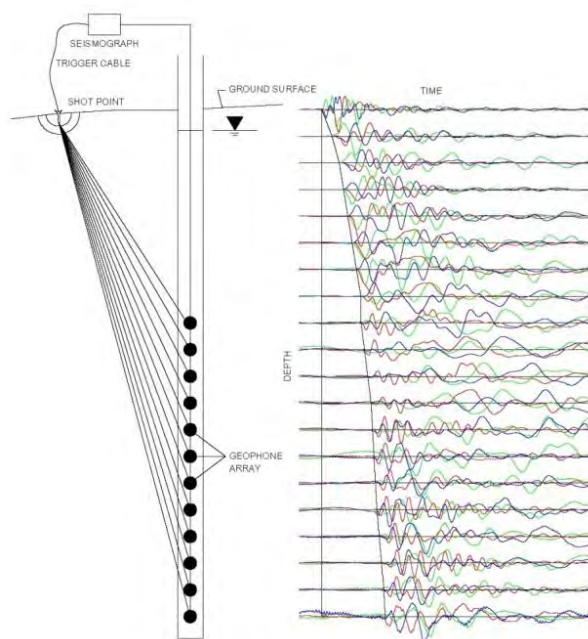
### **VERTICAL SEISMIC PROFILING TEST RESULTS OVERPASS ALONG HIGHWAY 16, OTTAWA, ONTARIO**

This memorandum presents the results of two Vertical Seismic Profiling (VSP) testing carried out at an overpass located along Highway 416 near McKenna Casey Drive in Ottawa, Ontario. VSP testing was carried out on June 25, 2018. Borehole 18-01, located north of the overpass, was drilled to an approximate depth of 26.6 m below the existing ground surface and then cased with a 2.5 inch PVC pipe grouted in place. The borehole consisted of approximately 5.3 m of gravelly sand fill, 15.5 m of gravelly silty sand, 3.2 m of sandstone, and shale/dolestone bedrock to bottom of the borehole. Borehole 18-02, located in south of the overpass, was drilled to an approximate depth of 20.2 m below the existing ground surface and then cased with a 2.5 inch PVC pipe grouted in place. The borehole consisted of approximately 4.4 m of sand fill, 5.7 m of silty clay, 4.7 m of silty sand, and dolestone bedrock.

### **Methodology**

For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth. The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear-wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole.

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the 2014 Canadian Highway Bridge Design Code (CHBDC 2014).



**Example 1: Layout and resulting time traces from a VSP survey.**

## Fieldwork

The fieldwork was carried out on June 25, 2018, by personnel from the Golder Mississauga and Ottawa offices.

At BH18-01, the compression and shear-wave seismic sources were used and they were located 2 m, and 2.03 m from the borehole. The seismic source for the compression wave test consisted of a 9.9 kilogram sledge hammer vertically impacted on a metal plate. The seismic source for the shear-wave test consisted of a 1 metre long aluminium plate, hammer into the ground and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced mostly at 1-metre intervals below the ground surface to a depth of 20 m and 0.5-metre intervals down to the maximum depth of the casing (25.7 m).

At BH18-02, the compression and shear-wave seismic sources were used and they were located 2 m, and 1.93 m from the borehole. The seismic source for the shear-wave test consisted of a 2.4 metre long, 150 millimetre by 150 millimetre wooden beam, weighted by a vehicle and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. The shear source was coupled to the ground surface by parking a vehicle on top of it. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced mostly at 1-metre intervals below the ground surface to a depth of 14 m and 0.5-metre intervals down to the maximum depth of the casing (19.7 m).

The seismic records collected for each source location were stacked a minimum of five times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.341 seconds was collected for each seismic shot.

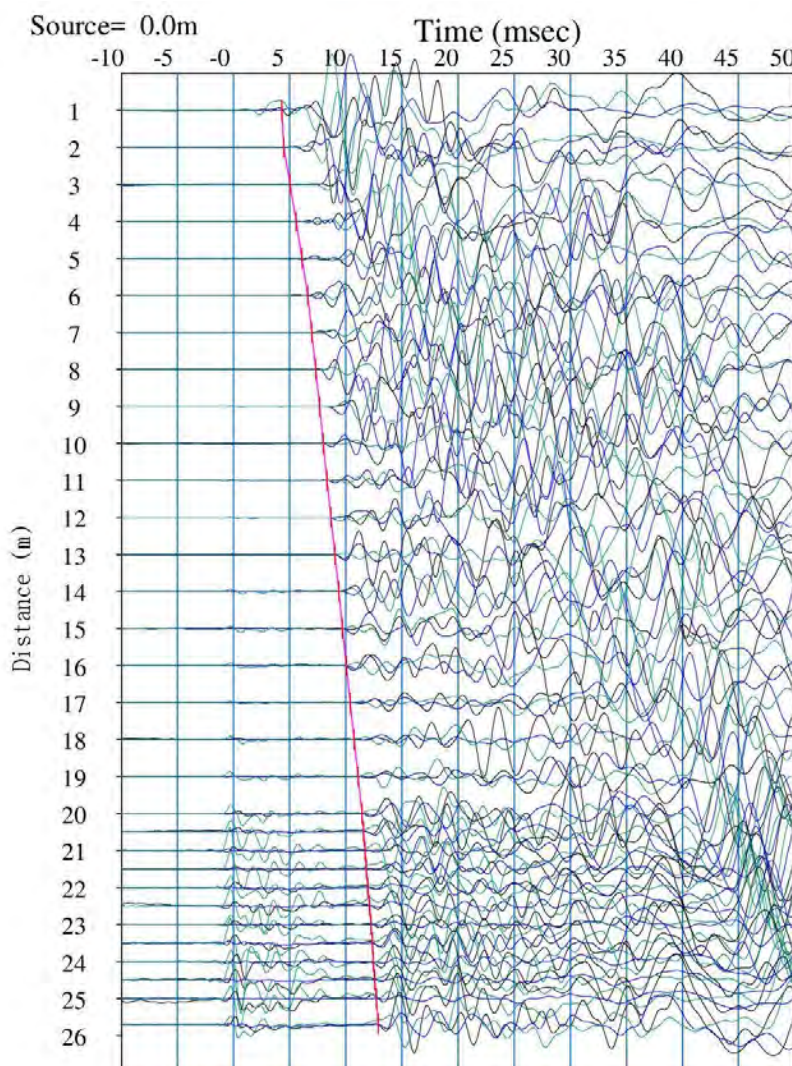


## Data Processing

Processing of the VSP test results consisted of the following main steps:

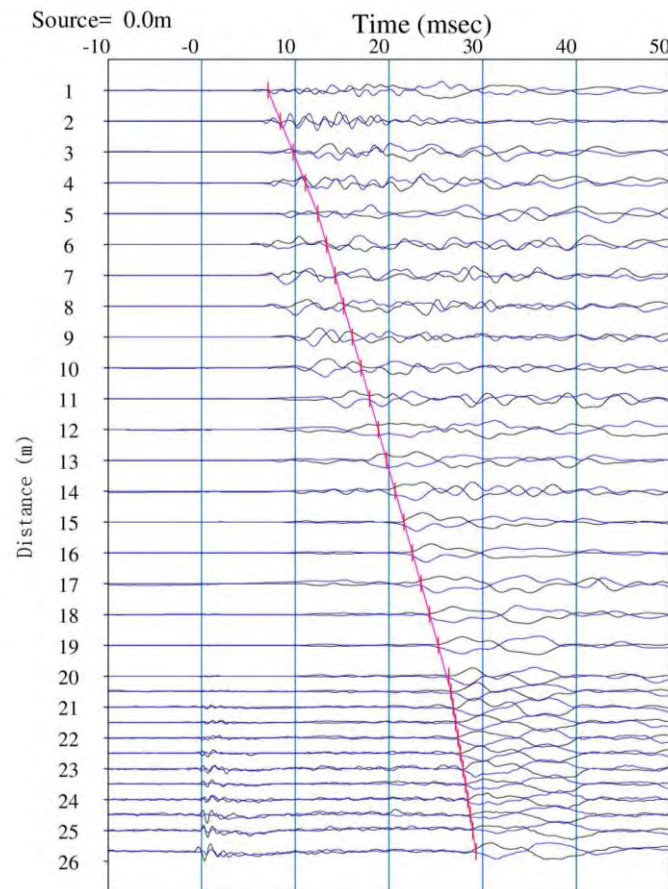
- 1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;
- 2) Low Pass Filtering of data to remove spurious high frequency noise;
- 3) First break picking of the compression and shear-wave arrivals; and,
- 4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records at BH18-01 are presented on the following two plots and show the first break picks of the compression wave (Figure 1) and shear wave arrivals (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.



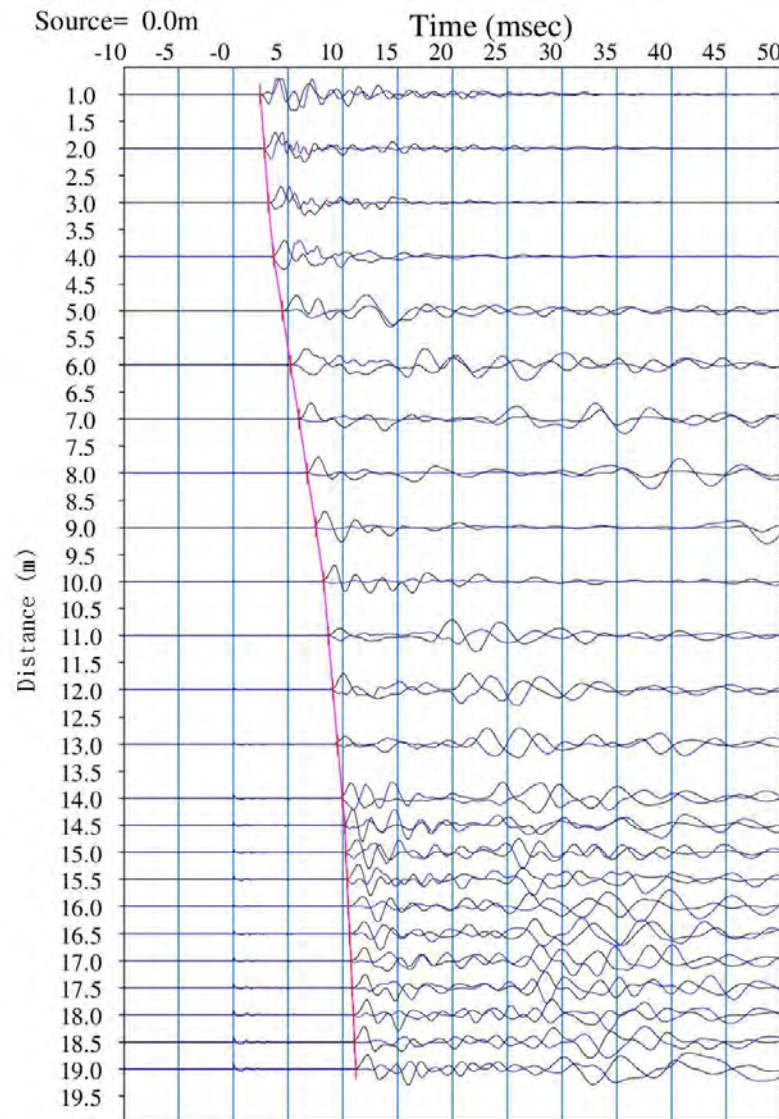
**Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole BH18-01.**



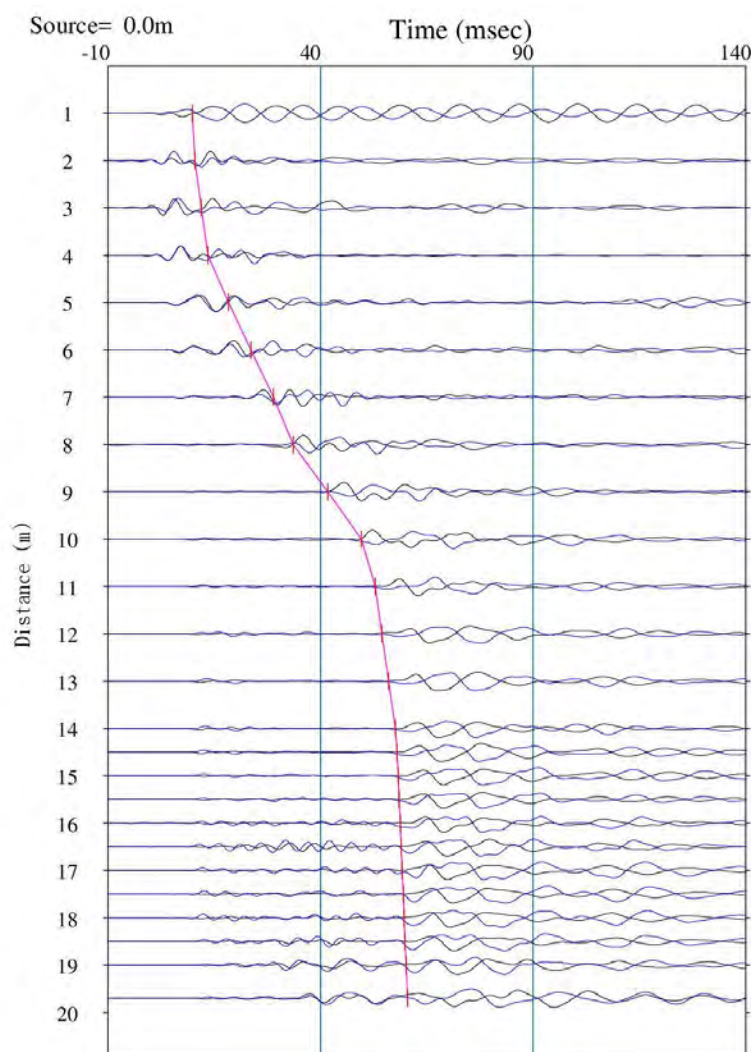


**Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 18-01.**

The seismic records at BH18-02 are presented on the following two plots and show the first break picks of the compression wave (Figure 3) and shear wave arrivals (Figure 4) overlaid on the seismic waveform traces recorded at the different geophone depths. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.



**Figure 3: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 18-02.**



**Figure 4: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 18-02.**

## Results

The VSP results at BH18-01 and BH18-02 are summarized in Tables 1, and Table 2, respectively. The shear wave and compression wave layer velocities were calculated by best fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Tables 1 and 2. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. At borehole 18-01, an estimated bulk density of 2000 kg/m<sup>3</sup> was used for fill and gravelly silty sand and an estimated bulk density of 2,300 kg/m<sup>3</sup> was used for the sandstone and shale bedrock. At borehole 18-02, an estimated bulk density of 2000 kg/m<sup>3</sup> was used for fill, 1,550 kg/m<sup>3</sup> for silty clay, 2,000 kg/m<sup>3</sup> for the silty sand and 2,600 kg/m<sup>3</sup> for the dolostone bedrock.

At borehole 18-01, the average shear wave velocity from ground surface to a depth of 30 metres was measured to be 953 metres per second. The average velocity at 18-01 was calculated assuming that the

velocity from 25.7 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 1,900 m/s which is equal to the velocity at the bottom of the borehole.

At borehole 18-02, the average shear wave velocity from ground surface to a depth of 30 metres was measured to be 460 metres per second. The average velocity at 18-02 was calculated assuming that the velocity from 19.7 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 2,000 m/s which is equal to the velocity at the bottom of the borehole.

## Limitations

This technical memorandum, which specifically includes all tables, figures and attachments, is based on data and information collected by Golder Associates Ltd. and is based solely on the conditions of the properties at the time of the work, supplemented by historical information and data obtained by Golder Associates Ltd. as described in this memo.

Golder Associates Ltd. has relied in good faith on all information provided and does not accept responsibility for any deficiency, misstatements, or inaccuracies contained in the reports as a result of omissions, misinterpretation, or fraudulent acts of the persons contacted or errors or omissions in the reviewed documentation.

The services performed, as described in this memo, were conducted in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practicing under similar conditions, subject to the time limits and financial and physical constraints applicable to the services.

Any use which a third party makes of this memo, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this memo.

The findings and conclusions of this memo are valid only as of the date of this memo. If new information is discovered in future work, including excavations, borings, or other studies, Golder Associates Ltd. should be requested to re-evaluate the conclusions of this memo, and to provide amendments as required.

## Closure

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

### **GOLDER ASSOCIATES LTD.**



Stephane Sol, Ph.D., P. geo.  
*Senior Geophysicist*

SS/CRP

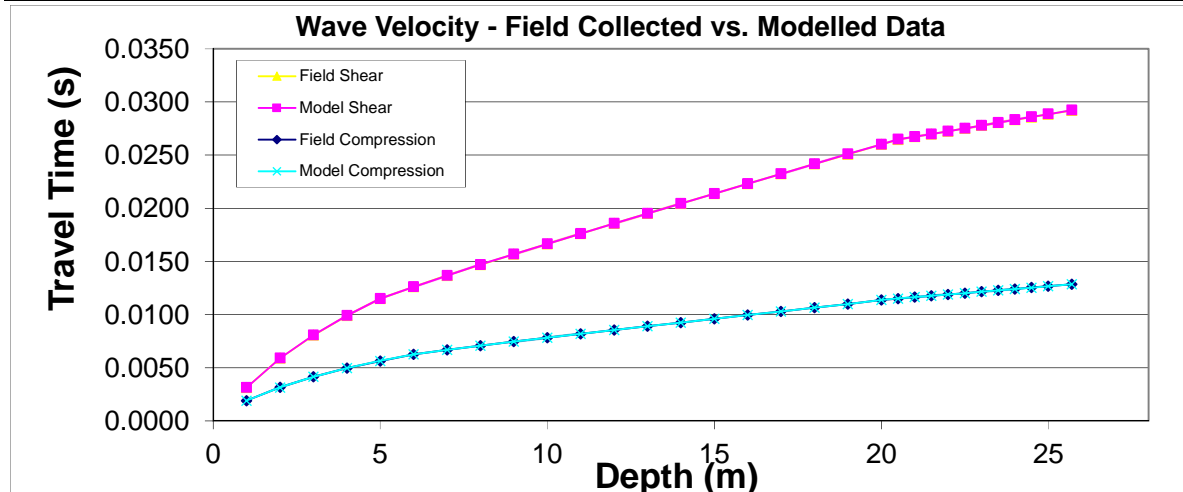


Christopher Phillips, M.Sc., P. Geo.  
*Senior Geophysicist, Principal*

Attachment: Tables 1 & 2

**TABLE 1**  
**SHEAR WAVE VELOCITY PROFILE AT BOREHOLE 18-01**

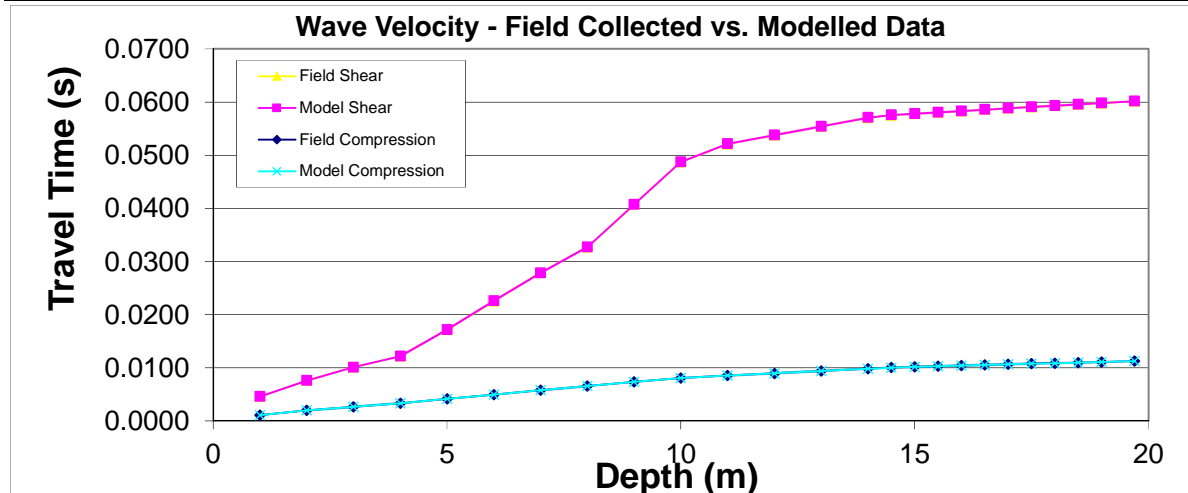
Layer Depth (m)		Velocities (m/s)		Estimated Bulk Density (kg/m <sup>3</sup> )	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1.0	525	320	2000	0.20	205	493	278
1.0	2.0	810	360	2000	0.38	259	714	967
2.0	3.0	990	460	2000	0.36	423	1153	1396
3.0	4.0	1250	550	2000	0.38	605	1670	2318
4.0	5.0	1450	620	2000	0.39	769	2134	3180
5.0	6.0	1600	900	2000	0.27	1620	4110	2960
6.0	7.0	2400	950	2000	0.41	1805	5080	9113
7.0	8.0	2530	970	2000	0.41	1882	5321	10293
8.0	9.0	2650	1020	2000	0.41	2081	5881	11271
9.0	10.0	2750	1030	2000	0.42	2122	6019	12296
10.0	11.0	2750	1040	2000	0.42	2163	6129	12241
11.0	12.0	2800	1050	2000	0.42	2205	6254	12740
12.0	13.0	2800	1060	2000	0.42	2247	6366	12684
13.0	14.0	2830	1070	2000	0.42	2290	6487	12965
14.0	15.0	2850	1070	2000	0.42	2290	6494	13192
15.0	16.0	2870	1070	2000	0.42	2290	6500	13421
16.0	17.0	2870	1080	2000	0.42	2333	6614	13363
17.0	18.0	2870	1080	2000	0.42	2333	6614	13363
18.0	19.0	2870	1080	2000	0.42	2333	6614	13363
19.0	20.0	2700	1080	2000	0.40	2333	6554	11470
20.0	20.5	3400	1080	2000	0.44	2333	6737	20010
20.5	21.0	4000	2000	2300	0.33	9200	24533	24533
21.0	21.5	4000	2000	2300	0.33	9200	24533	24533
21.5	22.0	3900	2000	2300	0.32	9200	24317	22716
22.0	22.5	3850	1800	2300	0.36	7452	20271	24156
22.5	23.0	3850	1800	2300	0.36	7452	20271	24156
23.0	23.5	3900	1800	2300	0.36	7452	20339	25047
23.5	24.0	3900	1900	2300	0.34	8303	22325	23912
24.0	24.5	3900	1900	2300	0.34	8303	22325	23912
24.5	25.0	3900	1900	2300	0.34	8303	22325	23912
25.0	25.7	3900	1900	2300	0.34	8303	22325	23912

**Notes**

1. Depth Presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.

**TABLE 2**  
**SHEAR WAVE VELOCITY PROFILE AT BOREHOLE 18-02**

Layer Depth (m)		Velocities (m/s)		Estimated Bulk Density (kg/m <sup>3</sup> )	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1.0	920	220	2000	0.47	97	285	1564
1.0	2.0	1110	330	2000	0.45	218	632	2174
2.0	3.0	1520	400	2000	0.46	320	936	4194
3.0	4.0	1580	485	2000	0.45	470	1362	4366
4.0	5.0	1190	200	1550	0.49	62	184	2112
5.0	6.0	1220	185	1550	0.49	53	158	2236
6.0	7.0	1240	190	1550	0.49	56	167	2309
7.0	8.0	1250	205	1550	0.49	65	194	2335
8.0	9.0	1260	125	1550	0.50	24	72	2428
9.0	10.0	1350	125	1550	0.50	24	72	2793
10.0	11.0	2300	290	2000	0.49	168	502	10356
11.0	12.0	2400	610	2000	0.47	744	2181	10528
12.0	13.0	2250	610	2000	0.46	744	2174	9133
13.0	14.0	2250	610	2000	0.46	744	2174	9133
14.0	14.5	2600	990	2000	0.42	1960	5548	10906
14.5	15.0	3600	2000	2000	0.28	8000	20429	15253
15.0	15.5	3800	2000	2600	0.31	10400	27215	23677
15.5	16.0	4300	2000	2600	0.36	10400	28329	34207
16.0	16.5	4300	2000	2600	0.36	10400	28329	34207
16.5	17.0	4400	2000	2600	0.37	10400	28492	36469
17.0	17.5	4400	2000	2600	0.37	10400	28492	36469
17.5	18.0	4400	2000	2600	0.37	10400	28492	36469
18.0	18.5	4400	2000	2600	0.37	10400	28492	36469
18.5	19.0	4400	2000	2600	0.37	10400	28492	36469
19.0	19.7	4400	2000	2600	0.37	10400	28492	36469

**Notes**

1. Depth Presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.

**APPENDIX H**

**Acceleration Time History Development  
(Golder, 2018)**



## TECHNICAL MEMORANDUM

**DATE** March 12, 2019

**Project No.** 1417217.1231

**TO** Matt Kennedy, P.Eng  
Golder Associates Ltd., Ottawa

**CC** Mike Snow, P.Eng

**FROM** Jie Cao, Feng Li, Alan Hull

**EMAIL** ahull@golder.com

### **EARTHQUAKE ACCELERATION TIME HISTORIES FOR STRANDHERD/CNR BRIDGES, OTTAWA, ONTARIO, CANADA**

#### **1.0 SUMMARY**

This technical memorandum presents earthquake acceleration time histories developed to support site-specific ground response analysis for the Strandherd/CNR Bridges in Ottawa, Ontario, Canada. The target acceleration response spectra are uniform hazard response spectra (UHRS) for two return periods for a soil Site Class A site as per the 2015 National Building Code of Canada (NBCC 2015). The two return periods are 975 years and 2,475 years. Golder has scaled a suite of 11 single-horizontal-component earthquake acceleration time histories to match each target response spectrum. Digital records of the scaled acceleration time histories are attached to this technical memorandum.

#### **2.0 INTRODUCTION**

Golder Associates Inc. (Golder) understands that earthquake acceleration time histories are needed for a site-specific ground response analysis for the Strandherd/CNR Bridges site in Ottawa, Ontario, Canada. The Strandherd/CNR Bridges are at present undergoing structural assessment. Golder will perform site-specific ground response analysis using site-specific earthquake acceleration time histories to assess the liquefaction potential at the site and provide design input to the structural assessment. This technical memorandum provides information on the development and results of earthquake acceleration time history scaling for the site response analysis.

#### **3.0 WORK SCOPE**

The work scope of this study is to:

- Select 11 sets of seed earthquake acceleration time histories for each of the 975-year and 2,475-year return period ground motions (22 total sets)
- Scale 11 single-horizontal-component seed acceleration time histories to the target acceleration response spectrum for each specified return period
- Provide digital scaled time history data to incorporate into the site-specific ground response analysis
- Prepare this technical memorandum that describes the data, procedures and results of the acceleration time history development.



## 4.0 INPUTS FOR ACCELERATION TIME HISTORY DEVELOPMENT

The target acceleration response spectra for acceleration time history development are the UHRS (5%-damped) for NBCC 2015 soil Site Class A for 975-year and 2,475-year return periods. Table 1 lists the spectra acceleration values for the target acceleration response spectra at the two earthquake levels.

**Table 1: Target 975-year and 2,475-year return period acceleration response spectra for NBCC 2015 soil Site Class A**

Period (s)	Spectral Acceleration (g)	
	975-Year RP	2,475-year RP
0	0.136	0.238
0.000001	0.164	0.285
0.05	0.164	0.285
0.1	0.164	0.285
0.2	0.164	0.285
0.3	--	--
0.5	0.075	0.128
1	0.038	0.064
2	0.018	0.031
5	0.005	0.009
10	0.002	0.003

The vibration periods of the Strandherd/CNR Bridges are understood to range from about 0.05 to 2 seconds; with the range of significant contributing vibration periods estimated to be from 0.5 to 2 seconds (Information of Modal Analysis provided by Ali El-Husseini, WSP, June 20, 2018). According to Commentary J to NBCC 2015, the period range for earthquake acceleration time history selection and scaling should have an upper bound greater than or equal to  $2T$  but not less than 1.5 seconds where  $T$  is the fundamental period of the structure to be analyzed. Similarly, the range of periods from the lower bound to the upper bound should include at least the periods of the vibration modes necessary to achieve 90% mass participation but not more than  $0.157T$ . The period range used for earthquake acceleration time history development in this study, therefore, ranges from 0.075 to 4 seconds.

From the hazard deaggregation analysis, primary contributors to the 975-year return period site seismic hazard are: earthquakes of magnitude **M**5.5 to **M**6.5 with the hypocentral distance of about 30 km from the site; earthquakes of magnitude **M** ≤5.5 close to the site (distances about 20 km) for 0.01- to 0.2-second spectral periods; and earthquakes of **M**6.5 to **M**7.5 located about 60 km from the site for 0.2- to 5-second spectral periods.

For the 2,475-year return period, deaggregation results indicate that the primary contributors are: earthquakes of magnitude **M6** to **M7** with the hypocentral distance of about 30 km from the site; earthquakes of magnitude **M**  $\leq 6$  located within about 10 km from the site for 0.01- to 0.2-second spectral periods; and earthquakes of **M7** to **M7.5** located about 50 km from the site for 0.2- to 5-second spectral periods.

There are no known seismogenic faults mapped close to the Strandherd/CNR Bridges site. Golder, therefore, has excluded earthquake acceleration time histories with clear pulse features when selecting seed acceleration time histories.

## 5.0 SELECTION OF SEED ACCELERATION TIME HISTORIES

### 5.1 General

Seed earthquake acceleration time histories are often selected based on the earthquake magnitude and source-to-site distance ranges that represent the earthquake scenarios contributing most to the total seismic hazard as evaluated for the return period of interest from a probabilistic seismic hazard analysis (PSHA) study. The earthquake scenarios are usually identified by interpretation of the PSHA deaggregation results at the return period of interest and at the structural period of interest, as discussed above.

Once the scenario earthquakes are selected, recorded acceleration time histories are selected that have similar source, path, and site properties to the scenario earthquakes. The selection criteria generally include earthquake magnitude, source-to-site distance, style of faulting, rupture directivity, and site ground condition.

In general, and particularly in eastern Canada, there are often insufficient recordings in the earthquake strong-motion databases to meet all of the desired selection criteria. An alternative to actual earthquake recordings is to use numerical simulations that have all of the desired source earthquake features. Still, to acquire enough candidate acceleration time histories, some of these conditions (e.g., style-of-faulting and site soil condition) are often relaxed.

### 5.2 Strandherd/CNR Bridges Site

Earthquake acceleration time history principal selection criteria used in this study are that the:

- Overall shape of the acceleration response spectrum of the seed time history is similar to the target spectrum
- Scaling factor ranges between 0.5 and 4.0
- Main intensity parameters of the scaled seed earthquake acceleration time histories are comparable to the anticipated measures of the intensity for the representative earthquake scenarios at the site.

For this study, in addition to the PEER NGA West2 database and NGA East database, simulated earthquake records were also used. These simulated records were developed by Atkinson (2009) using a stochastic finite-fault model technique. For eastern Canada, the simulated records contain four sets of records: an **M6** event at fault distances of 10 to 15 km (**M6** set 1), **M6** event at 20 to 30 km (**M6** set 2), **M7** at 15 to 25 km (**M7** set 1), and **M7** at 50 to 100 km (**M7** set 2) for Site Classes A, C, D and E, respectively.

Based on the deaggregation analysis and shape of the target response spectrum, for the 975-year return period, Golder has selected acceleration time histories recorded/simulated from the following earthquakes: one greater than **M7**, one less than **M5.5**, and nine between **M5.5** to **M7**. For the 2,475-year return period, Golder has

selected acceleration time histories recorded/simulated from earthquakes the following earthquakes: one greater than **M7**, one less than **M6**, and nine between **M6** to **M7**.

## 6.0 TIME HISTORY SCALING RESULTS

### 6.1 975-year Return Period Time Histories

Table 2 lists the key parameters of the selected seed earthquake acceleration time histories for 975-year return period. The record sequence number (RSN) for the real earthquake records are assigned by the PEER NGA West2 database and NGA East database. For the simulated records (Atkinson 2009), the RSN is created by combining the information of region, earthquake magnitude, site class, set, and record number in the selected set.

Figure 1 shows comparisons between the response spectra of scaled seed acceleration time histories and the target spectrum for the 975-year return period. The geometric mean response spectrum agrees reasonably well to the target spectrum for the period range of interest, i.e. from 0.075 to 4 seconds; and above 90% of the target spectrum within the same period range.

Earthquake magnitude and distance are causal parameters, and are only an indirect measure of the characteristics of the selected time histories. Significant durations ( $SD_{5-75}$  and  $SD_{5-95}$ ), Arias intensity (AI), cumulative absolute velocity (CAV), peak ground velocity (PGV), and peak ground displacement (PGD) are important measures of the intensity of the scaled earthquake acceleration time histories. Table 3 lists the main intensity parameters of the scaled seed acceleration time histories. As shown in Table 3, the scale factors for the seed acceleration time histories range from 0.5 to 3.5, which is within the typically accepted scale factor range of 0.5 to 4.0.

Golder further compared the intensity parameters of the scaled seed acceleration time histories to predictions of these intensity parameters based on the empirical relations listed in Table 4. Empirical relations considered are Abrahamson and Silva (1996) for significant durations, Abrahamson et al. (2016) for AI, Bullock et al. (2017) for CAV, and NBCC 2015 ground motion model for PGV. The considered earthquake scenarios for empirical relations are the following magnitude-distance pairs: **M6** at 30 km, **M5** at 10 km and **M7** at 50 km. Golder notes that there are no reliable empirical relations available for significant durations and AI for eastern North America or similar stable continental regions. Accordingly, the empirical relations for seismically active regions were used.

The comparison between Table 3 and 4 indicates that:

- The significant duration ( $SD_{5-95}$ ) of the five scaled seed time histories from the simulated records (Atkinson 2009) are slightly shorter than the predicted reference values.  $SD_{5-75}$  and  $SD_{5-95}$  of earthquake “Sparks\_2011-11-06” from the NGA-East database are relatively long and CAV is relatively large. But its AI and PGV are within 16<sup>th</sup> to 84<sup>th</sup> percentiles of the predicted reference values
- The AI of earthquake “40204628” from the NGA-West2 database is slightly smaller than the predicted preference value.

Overall the intensity parameters of the scaled seed acceleration time histories are judged to be acceptable for this analysis.

**Table 2: Key parameters of the selected seed earthquake acceleration time histories for 975-year return period**

ID	Database	Record Sequence Number	Component Direction	Earthquake Name	Year	Station Name	Earthquake Magnitude (M) <sup>1</sup>	Earthquake Mechanism	Rjb <sup>2</sup> (km)	Rrup <sup>3</sup> (km)
1	NGA-West2	146	320	Coyote Lake	1979	Gilroy Array #1	5.74	Strike Slip	10	11
2		455	230	Morgan Hill	1984	Gilroy Array #1	6.19	Strike Slip	15	15
3		2753	N	Chi-Chi_Taiwan-04	1999	CHY102	6.2	Strike Slip	39	39
4		8709	E	40204628	2007	Lick Observatory_Mt. Hamilton_CA_USA	5.45	Strike Slip	13	14
5		1836	360	Hector Mine	1999	Twentynine Palms	7.13	Strike Slip	42	42
6	NGA-East	10056	E	Sparks_2011-11-06	2011	Luther Middle School	5.68	Strike Slip	40	41
7	Simulated Record	east7a2-26	257.9	--	--	--	7	--	--	70.2
8		east7a2-40	90.6				7			94.3
9		east7a2-2	304.4				7			41.6
10		east7a2-30	328.4				7			47.8
11		east7a2-10	174.8				7			50.3

Notes:

1. Moment magnitude
2. Rrup = rupture distance
3. Rjb = Joyner-Boore distance

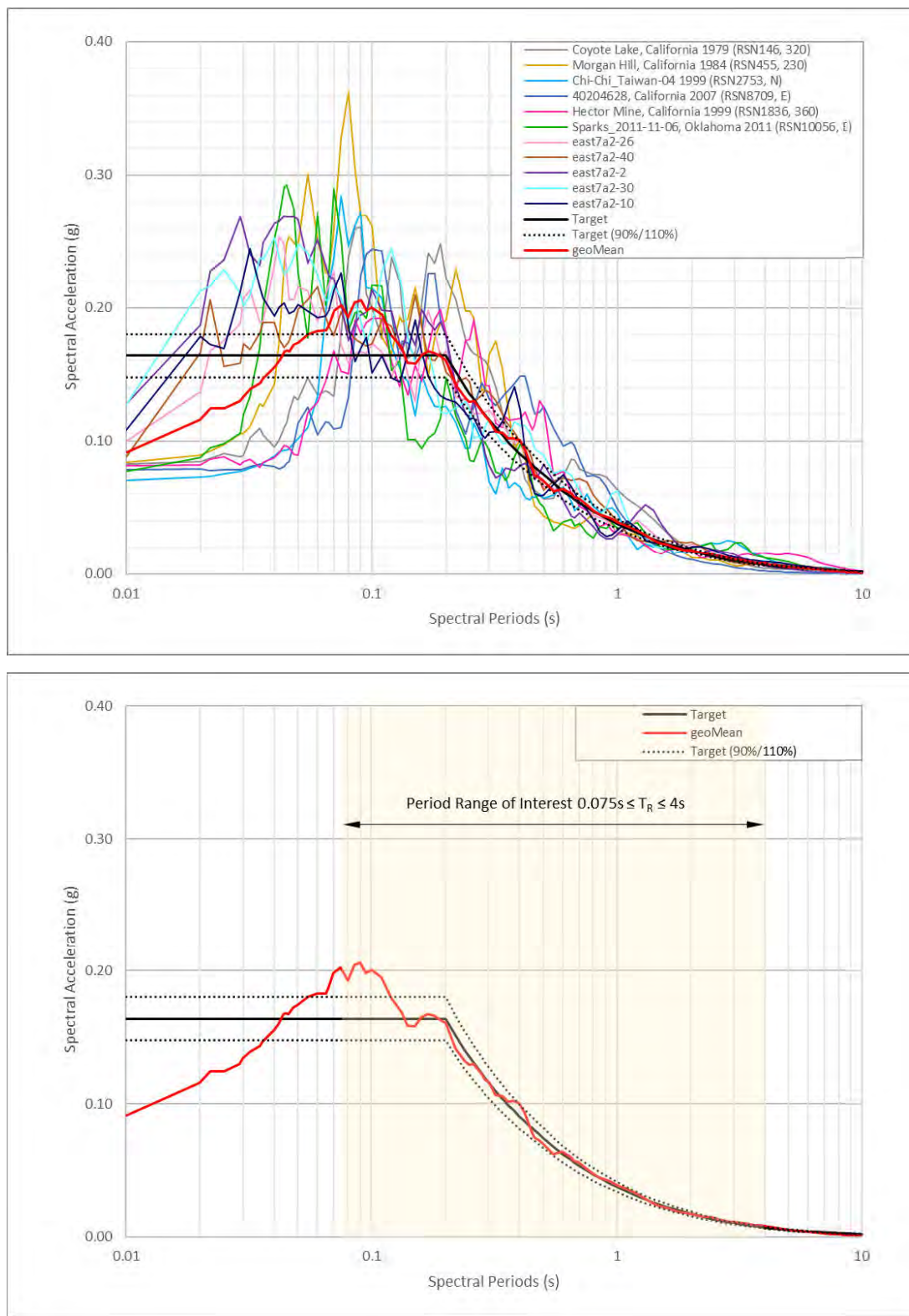


Figure 1: Target spectrum of 975-year return period comparing to (top) the individual response spectra of scaled seed earthquake acceleration time histories, and (bottom) the geometric mean response spectrum

**Table 3: Key intensity parameters of scaled seed earthquake acceleration time histories for 975-year return period**

ID (Table 2)	Scale Factor	SD <sub>5-75</sub> (s)	SD <sub>5-95</sub> (s)	AI (m/s)	CAV (m/s)	PGV (cm/s)	PGD (cm)
1	0.7	2.0	5.8	0.04	1.2	7.6	2.1
2	1.2	5.3	9.5	0.1	2.1	3.2	1.1
3	1.3	9.2	15.1	0.1	2.3	4.4	3.0
4	1.6	2.2	4.2	0.03	1.1	5.6	0.6
5	1.2	12.1	18.0	0.1	3.2	5.4	3.9
6	3.5	19	37.6	0.1	4.7	5.1	1.8
7	0.9	5.9	9.2	0.1	2.0	6.1	2.3
8	0.9	5.0	7.7	0.1	1.7	4.8	2.9
9	0.5	5.3	8.1	0.1	2.1	4.3	1.4
10	0.8	5.9	9.7	0.1	2.4	3.6	1.9
11	0.7	5.0	9.2	0.1	1.9	5.9	2.1

**Table 4: Predicted range of earthquake strong ground motion intensity measures from empirical relations for representative earthquake magnitude-distance pairs for 975-year return period**

Intensity Parameters	Empirical Equation Source	Earthquake Magnitude (M)	Earthquake Distance (km)	Median	16th Percentile	84th Percentile
SD <sub>5-75</sub> (s)	Abrahamson and Silva (1996)	6	30	4	2	7
		5	10	1	0.6	2
		7	50	9	5	15
SD <sub>5-95</sub> (s)		6	30	9	5	14
		5	10	2	1	4
		7	50	20	12	32
AI (m/s)	Abrahamson et al. (2016)	6	30	0.1	0.07	0.14
		5	10	0.06	0.04	0.1
		7	50	0.2	0.1	0.3

Intensity Parameters	Empirical Equation Source	Earthquake Magnitude (M)	Earthquake Distance (km)	Median	16th Percentile	84th Percentile
Cumulative Absolute Velocity (m/s)	Bullock et al. (2017)	6	30	1	0.7	2
		5	10	0.6	0.4	1
		7	50	2	1	4
PGV (cm/s)	NBCC 2015	6	30	3	2	6
		5	10	6	3	10
		7	50	6	3	11

## 6.2 2,475-year Return Period Time Histories

Table 5 lists the key parameters of the selected seed earthquake acceleration time histories for the 2,475-year return period. Figure 2 shows comparisons between the response spectra of scaled seed acceleration time histories and the target response spectrum. The geometric mean response spectrum agrees reasonably well to the target spectrum for the period range of interest. The geometric mean response spectrum does not fall more than 10% below the target spectrum within the same period range, and in some periods is a little more than 110% of the target.

Table 6 lists the direct measures of the intensity calculated for the scaled seed earthquake acceleration time histories. As indicated by the values listed in Table 5 and 6, the scale factors for the seed acceleration time histories range from 0.5 to 2.5, which is within the generally accepted scale factor range. The maximum number of seed acceleration time histories from the same earthquake event is two.

Table 7 lists predicted values of these intensity parameters based on the empirical relations for the following earthquake scenarios: **M6.5** at 30 km, **M5.25** at 10 km and **M7** at 50 km. The comparison between values listed in Table 6 and 7 shows that:

- The significant durations ( $SD_{5-75}$  and  $SD_{5-95}$ ) of the seven scaled seed time histories from the simulated records (Atkinson 2009) are slightly shorter than the predicted reference values
- AI, CAV and PGV measures are generally within 16<sup>th</sup> to 84<sup>th</sup> percentiles of the predicted reference values.

**Table 5: Key parameters of the selected seed earthquake acceleration time histories for 2,475-year return period**

ID	Database	Record Sequence Number	Component Direction	Earthquake Name	Year	Station Name	Earthquake Magnitude (M) <sup>1</sup>	Earthquake Mechanism	Rjb <sup>2</sup> (km)	Rrup <sup>3</sup> (km)
1	NGA-West2	1011	185	Northridge-01	1994	LA - Wonderland Ave	6.69	Reverse	15	20
2		1091	90	Northridge-01	1994	Vasquez Rocks Park	6.69	Reverse	23	24
3		1836	360	Hector Mine	1999	Twentynine Palms	7.13	Strike Slip	42	42
4		156	EW	Norcia_ Italy	1979	Cascia	5.9	Normal	1	5
5	Simulated Record	east7a1-40	276.5	--	--	--	7	--	--	25.6
6		east7a1-31	256.5				7			25.8
7		east7a1-37	203.2				7			25.7
8		east7a1-39	203.2				7			25.7
9		east7a2-1	304.4				7			41.6
10		east7a1-33	256.5				7			25.8
11		east7a2-26	257.9				7			70.2

Notes:

1. Moment magnitude
2. Rrup = rupture distance
3. Rjb = Joyner-Boore distance



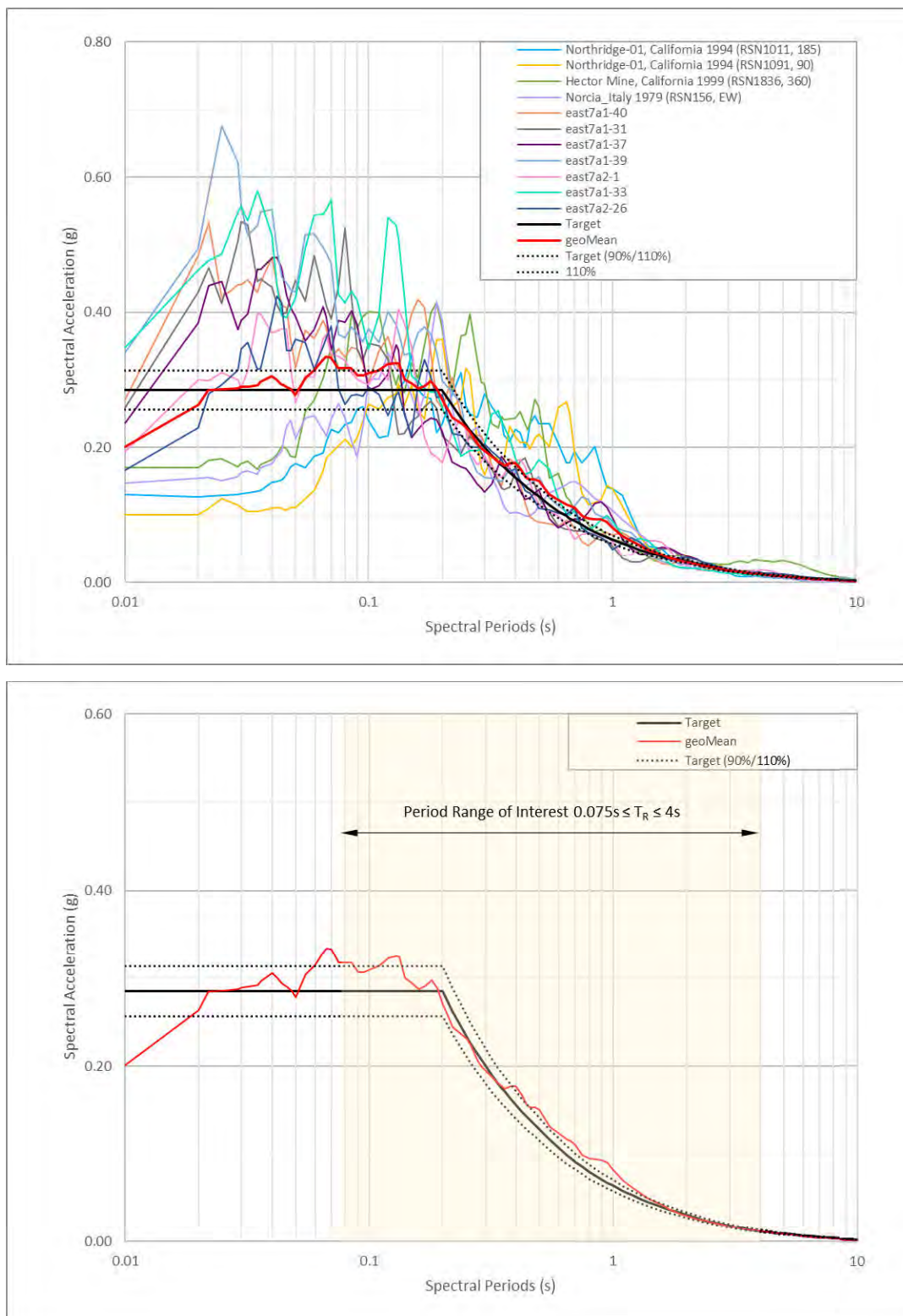


Figure 2: Target spectrum of 2,475-year return period comparing to (top) the individual response spectra of scaled seed earthquake acceleration time histories and (bottom) the geometric mean response spectrum

**Table 6: Key intensity parameters of scaled seed earthquake acceleration time histories for 2,475-year return period**

ID (Table 2)	Scale Factor	SD <sub>5-75</sub> (s)	SD <sub>5-95</sub> (s)	AI (m/s)	CAV (m/s)	PGV (cm/s)	PGD (cm)
1	0.8	5.1	6.7	0.1	2.5	11.7	1.8
2	0.7	4.3	7.3	0.2	3.0	7.7	2.0
3	2.5	12.1	18.0	0.4	6.7	11.3	8.0
4	0.7	2.3	5.2	0.1	1.8	10.3	2.4
5	0.5	4.9	7.7	0.3	3.5	7.4	4.1
6	0.5	4.0	6.9	0.3	3.2	6.3	4.0
7	0.5	3.3	7.1	0.2	2.7	8.4	5.3
8	0.7	3.6	7.6	0.3	3.7	7.0	6.0
9	0.7	5.2	8.0	0.2	3.0	6.9	3.8
10	0.6	4.3	7.2	0.3	3.8	10.0	4.7
11	1.5	5.9	9.2	0.2	3.3	10.1	3.8

**Table 7: Predicted range of earthquake strong ground motion intensity measures from empirical relations for representative earthquake magnitude-distance pairs for 2,475-year return period**

Intensity Parameters	Empirical Equation Source	Earthquake Magnitude (M)	Earthquake Distance (km)	Median	16th Percentile	84th Percentile
SD <sub>5-75</sub> (s)	Abrahamson and Silva (1996)	6.5	30	5	3	9
		5.25	10	1	0.8	2
		7	50	9	5	15
SD <sub>5-95</sub> (s)		6.5	30	12	7	20
		5.25	10	3	2	5
		7	50	20	12	32
AI (m/s)	Abrahamson et al. (2016)	6.5	30	0.3	0.2	0.5
		5.25	10	0.2	0.1	0.3
		7	50	0.4	0.3	0.6

Intensity Parameters	Empirical Equation Source	Earthquake Magnitude (M)	Earthquake Distance (km)	Median	16th Percentile	84th Percentile
Cumulative Absolute Velocity (m/s)	Bullock et al. (2017)	6.5	30	3	2	5
		5.25	10	1	0.8	2
		7	50	3	2	5
PGV (cm/s)	NBCC 2015	6.5	30	8	5	16
		5.25	10	10	6	19
		7	50	8	4	15

## 7.0 CONCLUSIONS

Two suites of 11 single-horizontal-component earthquake acceleration time histories have been selected from strong motion recordings and simulated records developed by Atkinson (2009). The records were scaled to match the 975-year return period UHRS and the 2475-year return period UHRS for soil Site Class A in NBCC 2015 at the Strandherd/CNR Bridges site. For each return period of interest, the geometric mean response spectrum of the scaled seed acceleration time histories agrees reasonably well to the target response spectrum for the period range of interest identified for this study. Comparisons between the measures of the intensity estimated from the scaled motions and from empirical relations indicate that the scaled motions are acceptable.

## 8.0 CLOSURE

Golder trusts that the explanation and results contained in this technical memorandum meet your requirements. Please contact us if you have questions or require clarification or additional information.

**GOLDER ASSOCIATES INC.**



Jie Cao, PhD  
Staff Engineer



Feng Li, PhD, PE  
Senior Project Engineer



Alan Hull, Ph.D., CEG  
Principal, Senior Practice Leader

Attachments: Digital files

JC/FL/AH/sb

[https://golderassociates-my.sharepoint.com/personal/sbrionez\\_golder\\_com/documents/1417217-strandheard bridges/1417217-tm-rev0-strandheard bridges ottawa-031219.docx](https://golderassociates-my.sharepoint.com/personal/sbrionez_golder_com/documents/1417217-strandheard%20bridges/1417217-tm-rev0-strandheard%20bridges%20ottawa-031219.docx)

## 9.0 REFERENCES

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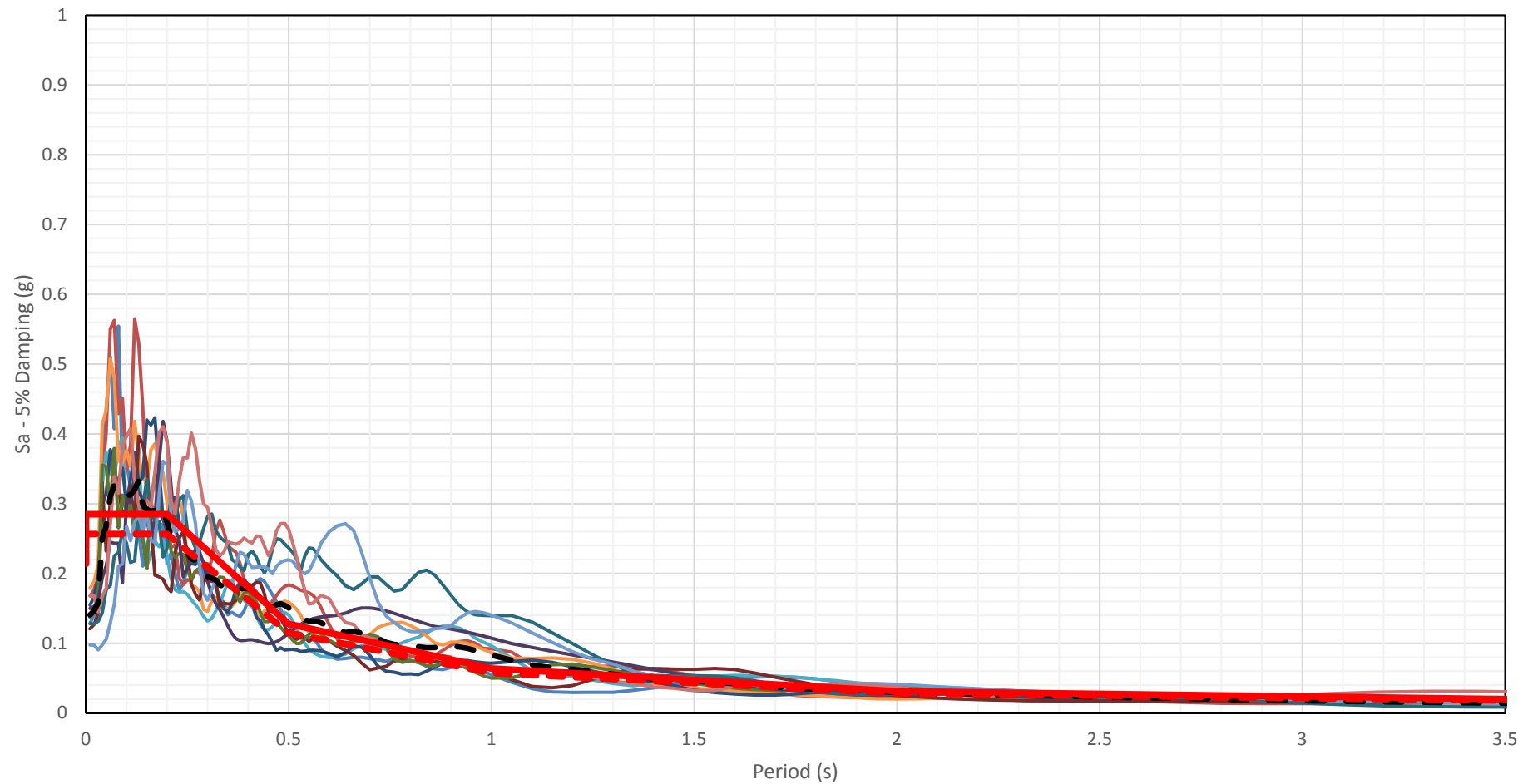
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**APPENDIX I**

# Seismic Analysis Results



East7 A1 31	East7 A1 33	East7 A1 37	East7 A1 39	East7 A1 40
East7 A2 01	East7 A2 26	Norcia Italy	Northridge Wonderland	Northridge Vasquez
Hector Mine	Average	Target (Site Class A)	90% Target (Site Class A)	

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YYYY-MM-DD	2018/10/03
PREPARED	SG
DESIGN	SG
REVIEW	MJK
APPROVED	MSS

PROJECT  
STRANDHERD DRIVE AND CNR OVERPASS BRIDGES  
SITE NO. 3-549, HIGHWAY 401  
OTTAWA, ONTARIO

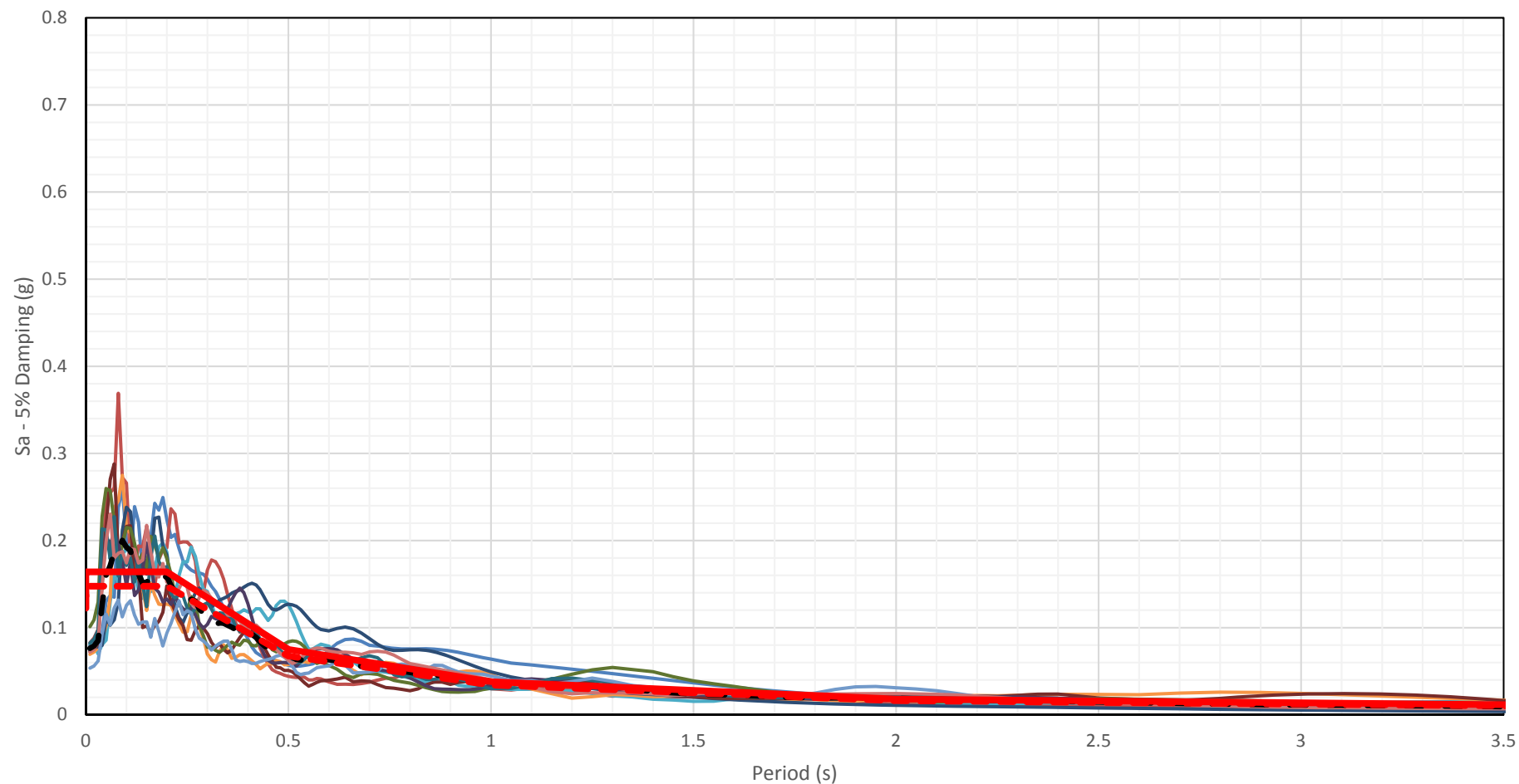
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**SITE CLASS A SCALED INPUT RESPONSE SPECTRA  
2,475-YEAR DESIGN EARTHQUAKE**

PROJECT No.  
**1417217**

Phase  
**1231**

Rev.  
**1**

Figure  
**11**



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DESIGN	SG
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APPROVED	MSS

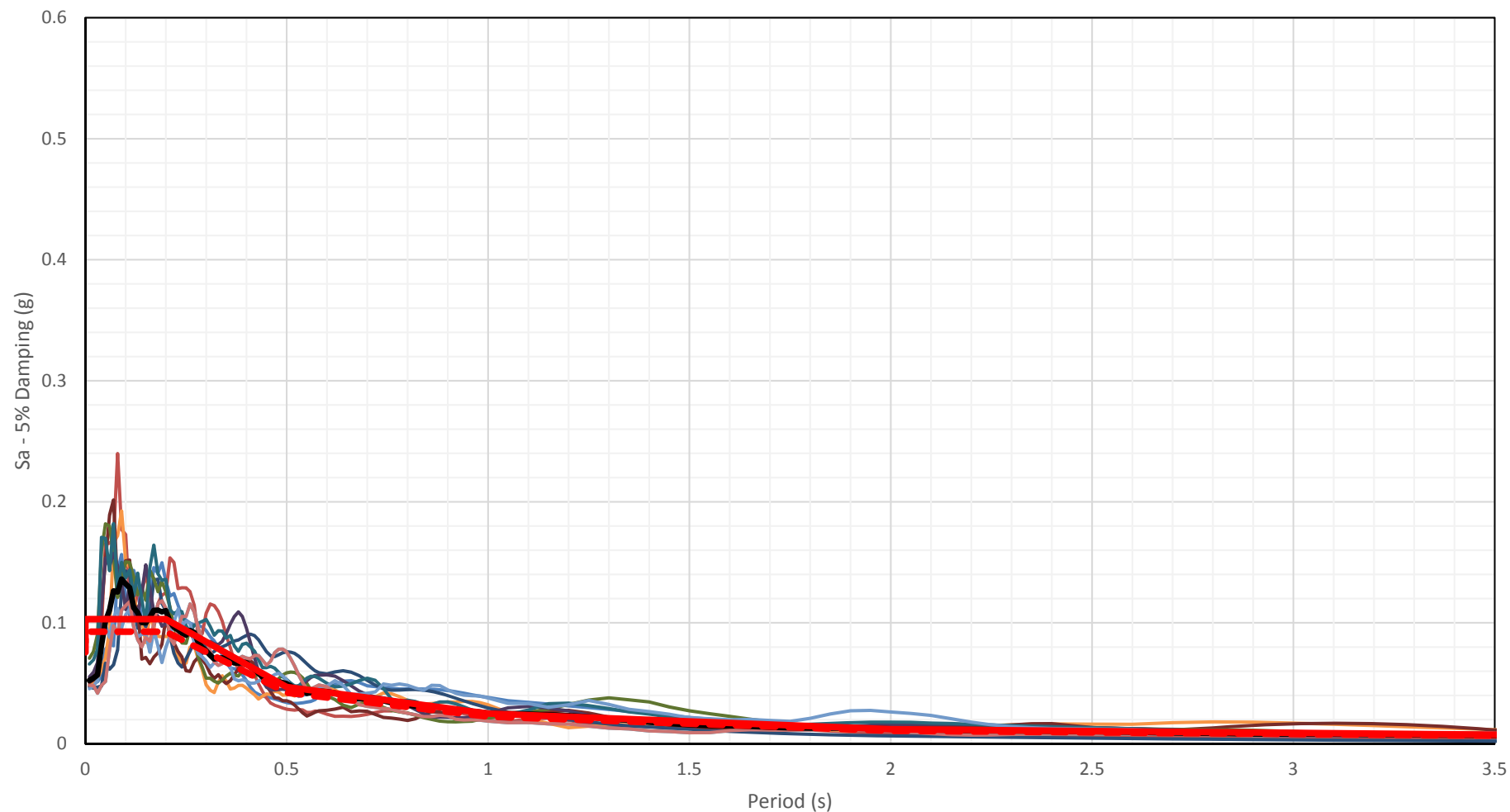
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STRANDHERD DRIVE AND CNR OVERPASS BRIDGES  
SITE NO. 3-549, HIGHWAY 401  
OTTAWA, ONTARIO

TITLE  
**SITE CLASS A SCALED INPUT RESPONSE SPECTRA  
975-YEAR DESIGN EARTHQUAKE**

PROJECT No.	Phase	Rev.	Figure
1417217	1231	1	12

1 in IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM ANSI A





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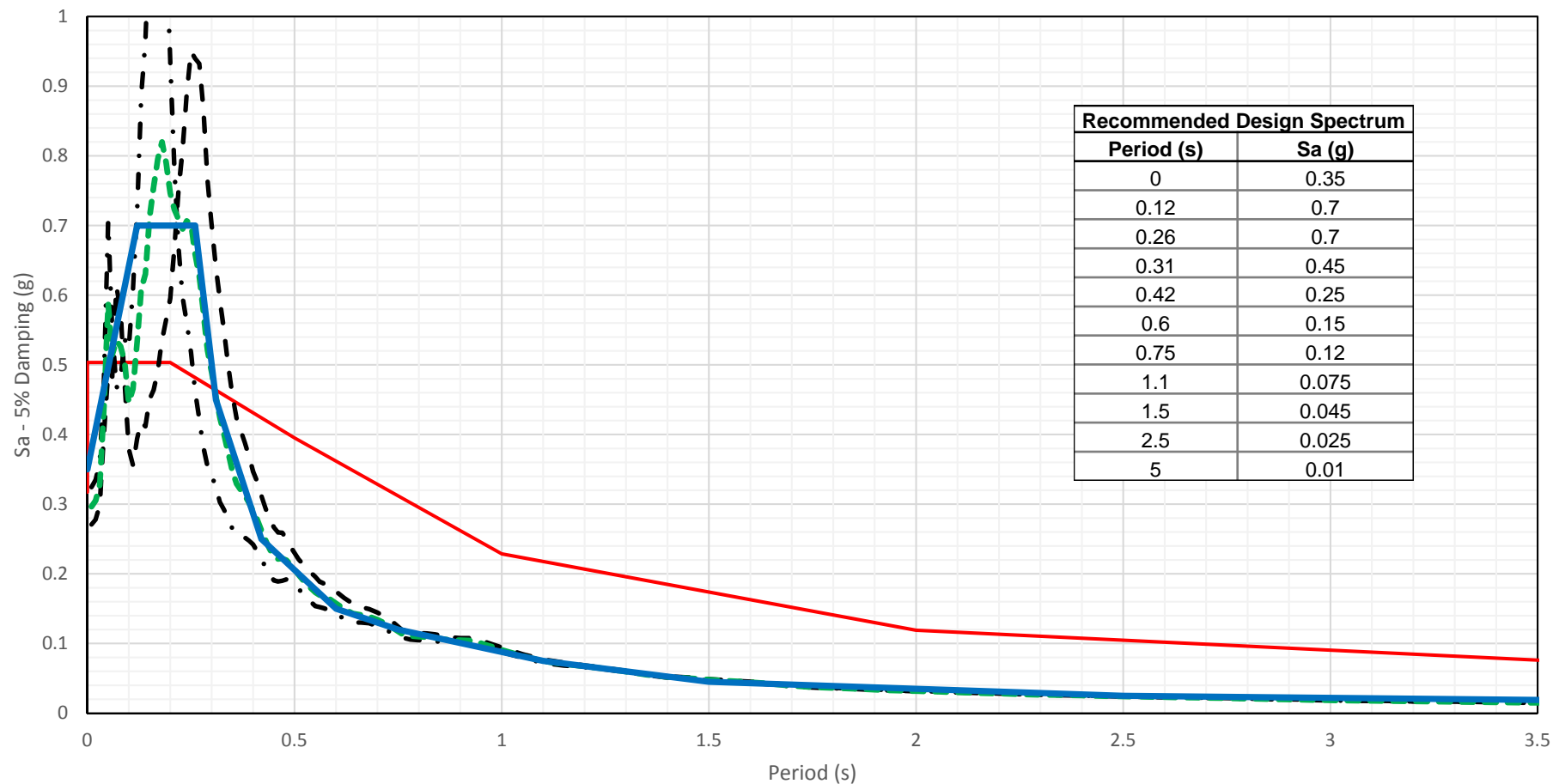
YYYY-MM-DD	2018/10/03
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DESIGN	SG
REVIEW	MJK
APPROVED	MSS

PROJECT  
STRANDHERD DRIVE AND CNR OVERPASS BRIDGES  
SITE NO. 3-549, HIGHWAY 401  
OTTAWA, ONTARIO

TITLE  
**SITE CLASS A SCALED INPUT RESPONSE SPECTRA  
475-YEAR DESIGN EARTHQUAKE**

PROJECT No.	Phase	Rev.	Figure
1417217	1231	1	13

1 in IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM ANSI A



— Site Class E (CHBDC)   
 - . - North Profile   
 - - - South Profile   
 - - - North and South GeoMean   
 — Recommended Design Spectrum

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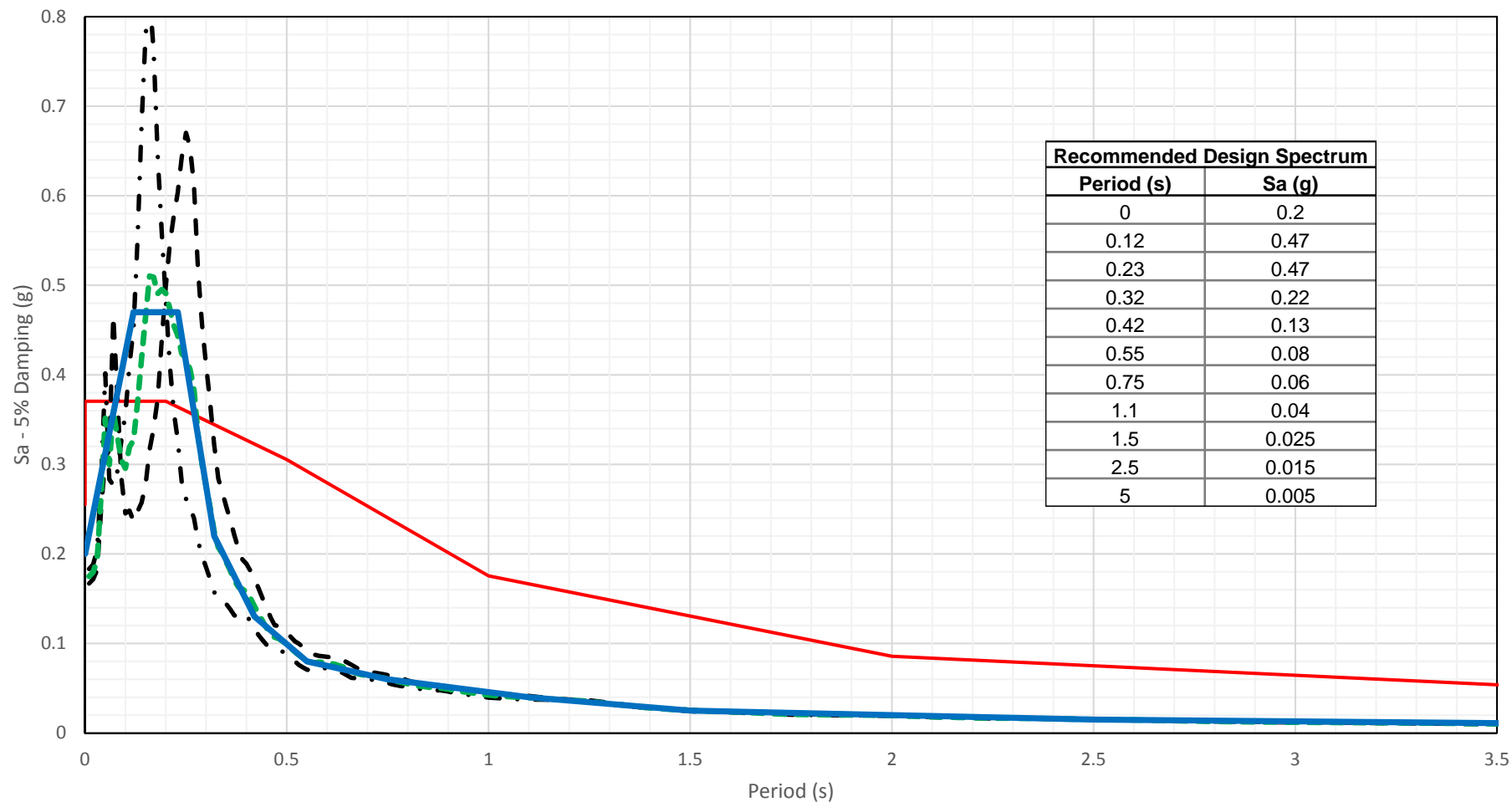


YYYY-MM-DD	2018/10/31
PREPARED	SG
DESIGN	SG
REVIEW	MJK
APPROVED	MSS

PROJECT  
 STRANDHERD DRIVE AND CNR OVERPASS BRIDGES  
 SITE NO. 3-549, HIGHWAY 401  
 OTTAWA, ONTARIO

TITLE  
**AVERAGE GROUND SURFACE RESPONSE SPECTRA  
 2,475-YEAR DESIGN EARTHQUAKE**

PROJECT No.	Phase	Rev.	Figure
1417217	1231	1	14



— Site Class E (CHBDC)   
 - . - North Profile   
 - - - South Profile   
 - - - North and South GeoMean   
 — Recommended Design Spectrum

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YYYY-MM-DD    2018/10/31

PREPARED    SG

DESIGN    SG

REVIEW    MJK

APPROVED    MSS

PROJECT

STRANDHERD DRIVE AND CNR OVERPASS BRIDGES  
SITE NO. 3-549, HIGHWAY 401  
OTTAWA, ONTARIO

TITLE

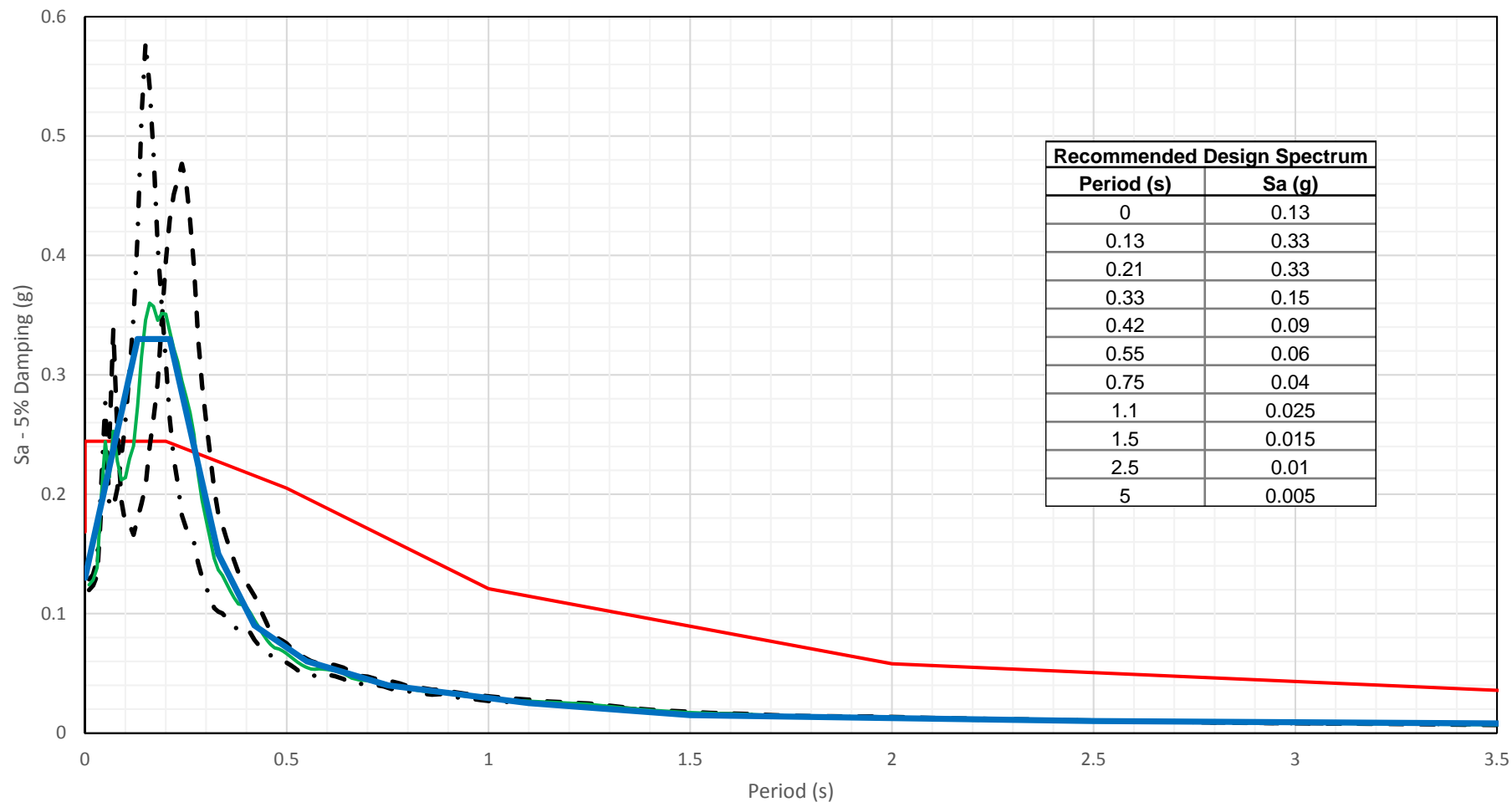
**AVERAGE GROUND SURFACE RESPONSE SPECTRA  
975-YEAR DESIGN EARTHQUAKE**

PROJECT No.  
**1417217**

Phase  
**1231**

Rev.  
**1**

Figure  
**15**



— Site Class E (CHBDC)   
 - . - North Profile   
 - - - South Profile   
 — North and South GeoMean   
 — Recommended Design Spectrum

CLIENT  
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YYYY-MM-DD	2018/10/31
PREPARED	SG
DESIGN	SG
REVIEW	MJK
APPROVED	MSS

PROJECT  
STRANDHERD DRIVE AND CNR OVERPASS BRIDGES  
SITE NO. 3-549, HIGHWAY 401  
OTTAWA, ONTARIO

TITLE  
**AVERAGE GROUND SURFACE RESPONSE SPECTRA  
475-YEAR DESIGN EARTHQUAKE**

PROJECT No.	Phase	Rev.	Figure
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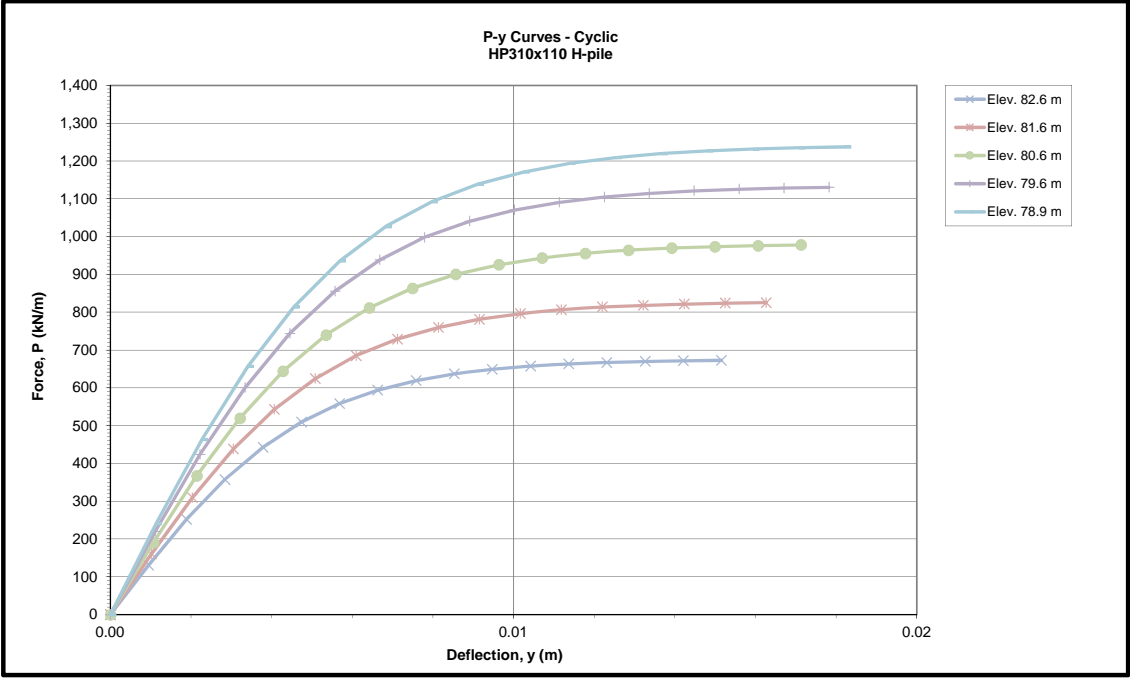
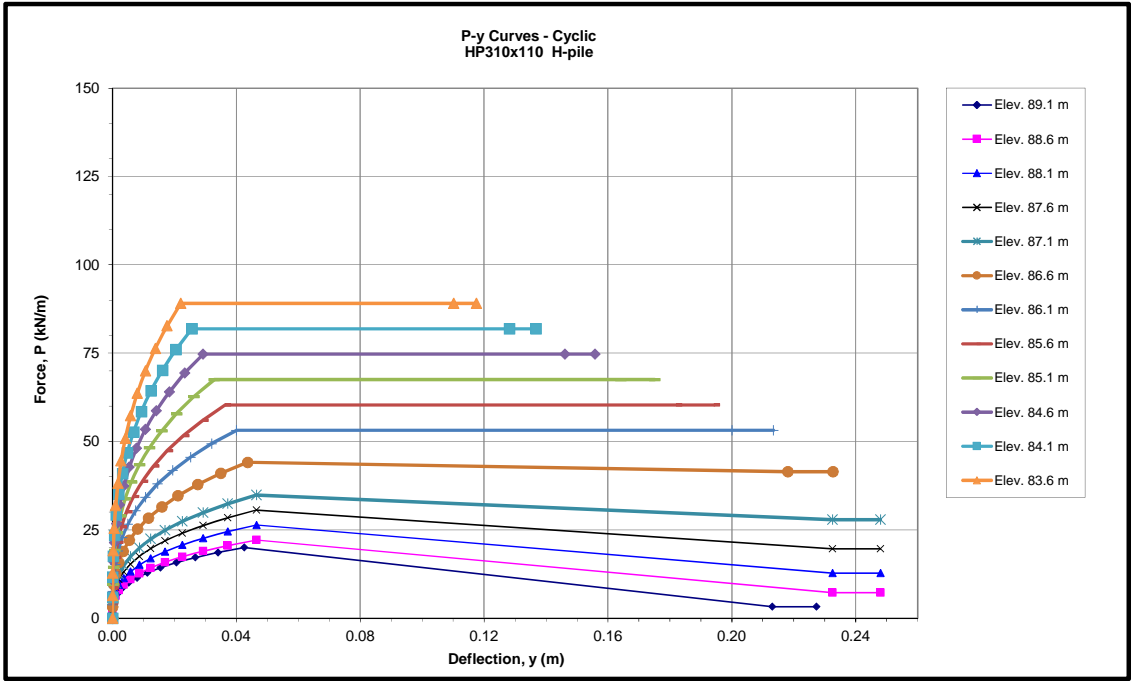
**APPENDIX J**

# P-y Curves for Deep Foundations

SUMMARY OF P-y CURVES FOR A HP310x110 H-pile

Description Depth (z) * Elevation P-y Curves	Soft to Firm Silty Clay																								Very Loose to Very Dense Silt Sand and Gravel											
	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= 10.7 m			
	Elev. 89.1 m		Elev. 88.6 m		Elev. 88.1 m		Elev. 87.6 m		Elev. 87.1 m		Elev. 86.6 m		Elev. 86.1 m		Elev. 85.6 m		Elev. 85.1 m		Elev. 84.6 m		Elev. 84.1 m		Elev. 83.6 m		Elev. 82.6 m		Elev. 81.6 m		Elev. 80.6 m		Elev. 79.6 m		Elev. 78.9 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	1.429	0.000	1.581	0.000	1.884	0.000	2.187	0.000	2.490	0.000	3.151	0.000	3.798	0.000	4.311	0.000	4.825	0.000	5.338	0.000	5.851	0.000	6.364	0.001	130.932	0.001	160.609	0.001	190.287	0.001	219.965	0.001	240.740			
0.000	2.859	0.000	3.162	0.000	3.768	0.000	4.374	0.000	4.980	0.000	6.302	0.000	7.596	0.000	8.623	0.000	9.649	0.000	10.676	0.000	11.702	0.000	12.729	0.002	252.378	0.002	309.584	0.002	366.789	0.002	423.995	0.002	464.039			
0.000	4.288	0.000	4.743	0.000	5.652	0.000	6.561	0.000	7.470	0.000	9.454	0.000	11.394	0.000	12.934	0.000	14.474	0.000	16.013	0.000	17.553	0.000	19.093	0.003	357.417	0.003	438.431	0.003	519.446	0.003	600.460	0.003	657.170			
0.001	5.717	0.001	6.324	0.001	7.536	0.001	8.748	0.001	9.960	0.001	12.605	0.001	15.192	0.001	17.245	0.001	19.298	0.001	21.351	0.001	23.404	0.001	25.457	0.004	442.908	0.004	543.301	0.004	643.693	0.004	744.086	0.005	814.361			
0.002	7.147	0.002	7.905	0.002	9.420	0.002	10.935	0.002	12.450	0.002	15.756	0.002	18.990	0.002	21.556	0.001	24.123	0.001	26.689	0.001	29.255	0.001	31.821	0.005	509.114	0.005	624.513	0.005	739.912	0.006	855.311	0.006	936.090			
0.003	8.576	0.004	9.486	0.004	11.304	0.004	13.122	0.004	14.941	0.003	18.907	0.003	22.788	0.003	25.868	0.003	28.947	0.002	32.027	0.002	35.106	0.002	38.186	0.006	558.436	0.006	685.015	0.006	811.593	0.007	938.172	0.007	1026.777			
0.005	10.006	0.006	11.067	0.006	13.188	0.006	15.309	0.006	17.431	0.005	22.059	0.005	26.586	0.005	30.179	0.004	33.772	0.004	37.365	0.003	40.957	0.003	44.550	0.007	594.130	0.007	728.800	0.007	863.469	0.008	998.139	0.008	1092.407			
0.008	11.435	0.009	12.647	0.009	15.072	0.009	17.496	0.009	19.921	0.008	25.210	0.007	30.384	0.007	34.490	0.006	38.596	0.005	42.702	0.005	46.808	0.004	50.914	0.008	619.423	0.008	759.826	0.009	900.229	0.009	1040.631	0.009	1138.913			
0.011	12.864	0.012	14.228	0.012	16.956	0.012	19.683	0.012	22.411	0.012	28.361	0.011	34.182	0.010	38.802	0.009	43.421	0.008	48.040	0.007	52.659	0.006	57.279	0.009	637.080	0.009	781.485	0.010	925.890	0.010	1070.295	0.010	1171.378			
0.016	14.294	0.017	15.809	0.017	18.840	0.017	21.870	0.017	24.901	0.016	31.512	0.015	37.980	0.013	43.113	0.012	48.245	0.011	53.378	0.009	58.510	0.008	63.643	0.009	649.277	0.010	796.447	0.011	943.616	0.011	1090.786	0.011	1193.805			
0.021	15.723	0.023	17.390	0.023	20.724	0.023	24.057	0.023	27.391	0.021	34.663	0.019	41.779	0.018	47.424	0.016	53.070	0.014	58.716	0.012	64.361	0.011	70.007	0.010	657.642	0.011	806.708	0.012	955.773	0.012	1104.839	0.013	1209.185			
0.027	17.152	0.029	18.971	0.029	22.608	0.029	26.245	0.029	29.881	0.027	37.815	0.025	45.577	0.023	51.736	0.021	57.895	0.018	64.054	0.016	70.213	0.014	76.372	0.011	663.351	0.012	813.710	0.013	964.070	0.013	1114.429	0.014	1219.681			
0.034	18.582	0.037	20.552	0.037	24.492	0.037	28.432	0.037	32.371	0.035	40.966	0.032	49.375	0.029	56.047	0.026	62.719	0.023	69.391	0.021	76.064	0.018	82.736	0.012	667.233	0.013	818.472	0.014	969.712	0.014	1120.951	0.015	1226.819			
0.043	20.011	0.047	22.133	0.047	26.376	0.047	30.619	0.047	34.861	0.044	44.117	0.040	53.173	0.036	60.358	0.033	67.544	0.029	74.729	0.026	81.915	0.022	89.100	0.013	669.867	0.014	821.703	0.015	973.540	0.016	1125.376	0.016	1231.662			
0.213	3.263	0.233	7.195	0.233	12.745	0.233	19.638	0.233	27.873	0.218	41.426	0.200	53.173	0.182	60.358	0.164	67.544	0.146	74.729	0.128	81.915	0.110	89.100	0.014	671.651	0.015	823.892	0.016	976.133	0.017	1128.374	0.017	1234.943			
0.227	3.263	0.248	7.195	0.248	12.745	0.248	19.638	0.248	27.873	0.233	41.426	0.213	53.173	0.194	60.358	0.175	67.544	0.156	74.729	0.137	81.915	0.118	89.100	0.015	672.859	0.016	825.374	0.017	977.888	0.018	1130.403	0.018	1237.164			

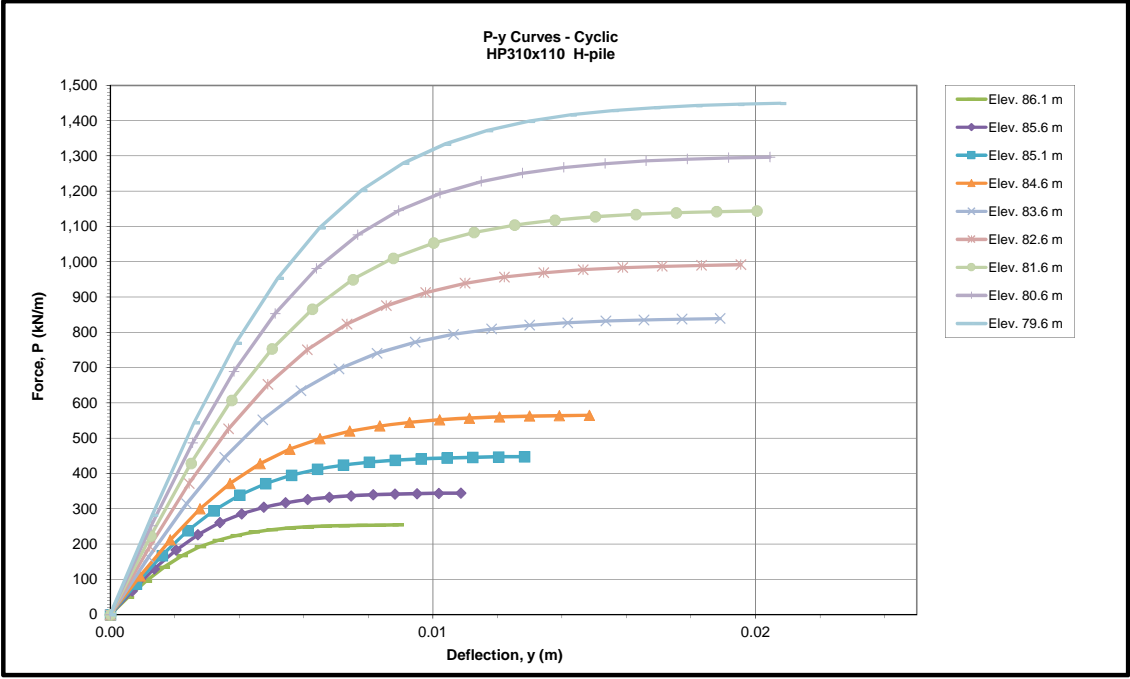
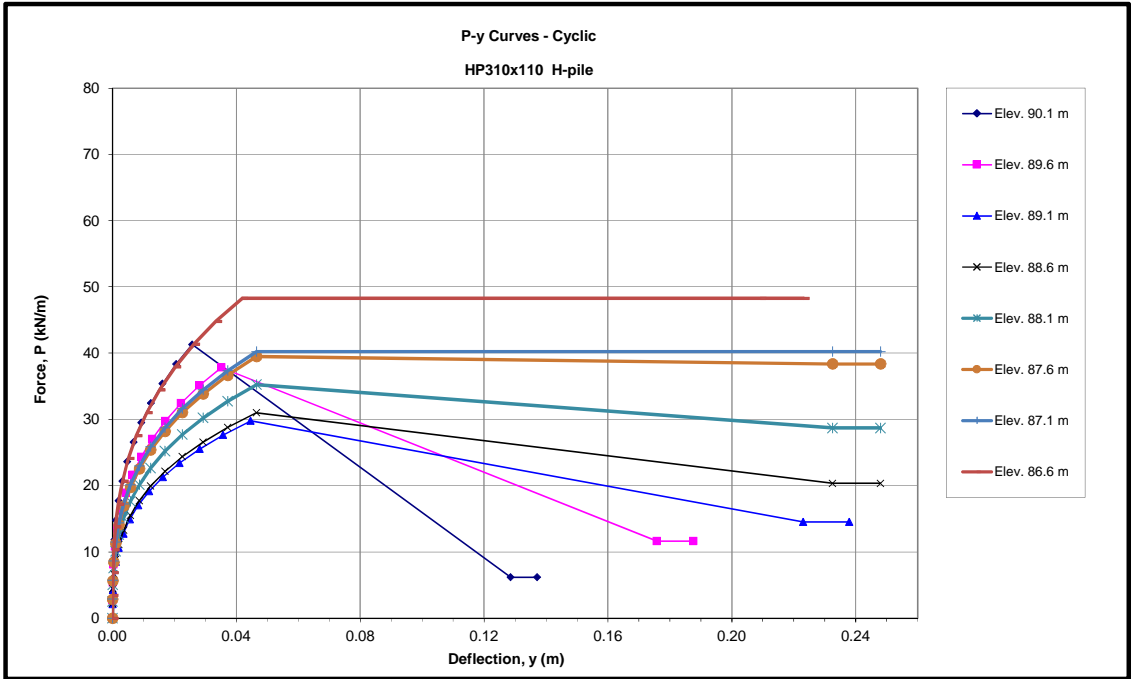
NOTES: \* Depth (z) is measured to be positive below the underside of the pier pile cap (Elevation 89.6 m).  
The P-y curves have been generated based on the following assumptions:  
1. P-y curves are generated for vertical piles (i.e. no inclination)  
2. Cyclic loading condition is considered.  
3. There are no pile group effects (i.e. analysis is based on a single pile).  
4. Resistance from pile cap embedment has not been considered.



SUMMARY OF P-y CURVES FOR A HP310x110 H-pile

Description Depth (z) * Elevation P-y Curves	Soft to Firm Silty Clay																Very Loose to Very Dense Silt Sand and Gravel																					
	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= 11.7 m			
	Elev. 90.1 m		Elev. 89.6 m		Elev. 89.1 m		Elev. 88.6 m		Elev. 88.1 m		Elev. 87.6 m		Elev. 87.1 m		Elev. 86.6 m		Elev. 86.1 m		Elev. 85.6 m		Elev. 85.1 m		Elev. 84.6 m		Elev. 83.6 m		Elev. 82.6 m		Elev. 81.6 m		Elev. 80.6 m		Elev. 79.6 m		Elev. 78.9 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)	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	0.000	
0.000	2.950	0.000	2.703	0.000	2.127	0.000	2.215	0.000	2.518	0.000	2.821	0.000	2.874	0.000	3.449	0.001	49.520	0.001	67.024	0.001	87.172	0.001	109.963	0.001	163.301	0.001	192.979	0.001	222.657	0.001	252.334	0.001	282.012	0.001	302.787	0.001	302.787	0.001
0.000	5.900	0.000	5.407	0.000	4.255	0.000	4.430	0.000	5.036	0.000	5.642	0.000	5.748	0.000	6.898	0.001	95.452	0.001	129.193	0.002	168.028	0.002	211.959	0.002	314.771	0.002	371.977	0.003	429.183	0.003	486.389	0.003	543.594	0.003	583.638	0.003	583.638	0.003
0.000	8.850	0.000	8.110	0.000	6.382	0.000	6.645	0.000	7.554	0.000	8.463	0.000	8.623	0.000	10.347	0.002	135.179	0.002	182.963	0.002	237.961	0.003	300.176	0.004	445.778	0.004	526.793	0.004	607.807	0.004	688.822	0.004	769.836	0.004	826.546	0.004	826.546	0.004
0.001	11.800	0.001	10.813	0.001	8.509	0.001	8.860	0.001	10.072	0.001	11.284	0.001	11.497	0.001	13.796	0.002	167.513	0.003	226.726	0.003	294.880	0.004	371.975	0.005	552.405	0.005	652.797	0.005	753.190	0.005	853.583	0.005	953.975	0.005	1024.250	0.005	1024.250	0.005
0.001	14.749	0.002	13.517	0.002	10.636	0.002	11.075	0.002	12.590	0.002	14.105	0.002	14.371	0.002	17.245	0.003	192.553	0.003	260.617	0.004	338.958	0.005	427.578	0.006	634.978	0.006	750.377	0.006	865.776	0.006	981.175	0.006	1096.574	0.007	1177.353	0.007	1177.353	0.007
0.002	17.699	0.003	16.220	0.004	12.764	0.004	13.290	0.004	15.108	0.004	16.927	0.004	17.245	0.003	20.694	0.003	211.207	0.004	285.865	0.005	371.796	0.006	469.001	0.007	696.494	0.007	823.072	0.008	949.651	0.008	1076.230	0.008	1202.809	0.008	1291.414	0.008	1291.414	0.008
0.003	20.649	0.004	18.923	0.006	14.891	0.006	15.505	0.006	17.626	0.006	19.748	0.006	20.119	0.005	24.143	0.004	224.707	0.005	304.137	0.006	395.561	0.006	498.978	0.008	741.012	0.009	875.682	0.009	1010.351	0.009	1145.021	0.009	1279.690	0.009	1373.959	0.009	1373.959	0.009
0.005	23.599	0.007	21.627	0.008	17.018	0.009	17.720	0.009	20.144	0.009	22.569	0.009	22.994	0.008	27.592	0.004	234.273	0.005	317.084	0.006	412.400	0.007	520.221	0.009	772.559	0.010	912.961	0.010	1053.364	0.010	1193.767	0.010	1334.169	0.010	1432.451	0.010	1432.451	0.010
0.007	26.549	0.009	24.330	0.012	19.146	0.012	19.935	0.012	22.662	0.012	25.390	0.012	25.868	0.011	31.041	0.005	240.951	0.006	326.123	0.007	424.156	0.008	535.050	0.011	794.580	0.011	938.985	0.011	1083.390	0.012	1227.795	0.012	1372.200	0.012	1473.283	0.012	1473.283	0.012
0.009	29.499	0.013	27.033	0.016	21.273	0.017	22.150	0.017	25.180	0.017	28.211	0.017	28.742	0.015	34.490	0.006	245.565	0.007	332.367	0.008	432.276	0.009	545.294	0.012	809.793	0.012	956.963	0.013	1104.132	0.013	1251.302	0.013	1398.471	0.013	1501.490	0.013	1501.490	0.013
0.012	32.449	0.017	29.736	0.022	23.400	0.023	24.365	0.023	27.698	0.023	31.032	0.023	31.616	0.020	37.939	0.006	248.728	0.007	336.649	0.009	437.846	0.010	552.319	0.013	820.226	0.013	969.292	0.014	1118.357	0.014	1267.423	0.014	1416.489	0.014	1520.834	0.014	1520.834	0.014
0.016	35.399	0.022	32.440	0.028	25.527	0.029	26.580	0.029	30.216	0.029	33.853	0.029	34.490	0.026	41.388	0.007	250.887	0.008	339.571	0.010	441.646	0.011	557.113	0.014	827.346	0.015	977.705	0.015	1128.065	0.015	1278.424	0.016	1428.784	0.016	1534.035	0.016	1534.035	0.016
0.021	38.349	0.028	35.143	0.036	27.655	0.037	28.795	0.037	32.735	0.037	36.674	0.037	37.365	0.034	44.837	0.007	252.355	0.009	341.558	0.010	444.231	0.012	560.374	0.015	832.188	0.016	983.427	0.016	1134.666	0.017	1285.906	0.017	1437.145	0.017	1543.013	0.017	1543.013	0.017
0.026	41.298	0.035	37.846	0.045	29.782	0.047	31.010	0.047	35.253	0.047	39.495	0.047	40.239	0.042	48.287	0.008	253.352	0.010	342.907	0.011	445.985	0.013	562.586	0.017	835.473	0.017	987.309	0.018	1139.146	0.018	1290.982	0.018	1442.819	0.018	1549.105	0.018	1549.105	0.018
0.129	6.179	0.176	11.637	0.223	14.511	0.233	20.341	0.233	28.700	0.233	38.401	0.233	40.239	0.209	48.287	0.008	254.027	0.010	343.820	0.012	447.173	0.014	564.084	0.018	837.699	0.018	989.940	0.019	1142.181	0.019	1294.422	0.019	1446.663	0.020	1553.231	0.020	1553.231	0.020
0.137	6.179	0.188	11.637	0.238	14.511	0.248	20.341	0.248	28.700	0.248	38.401	0.248	40.239	0.223	48.287	0.009	254.483	0.011	344.438	0.013	447.977	0.015	565.099	0.019	839.205	0.020	991.719	0.020	1144.234	0.020	1296.749	0.021	1449.264	0.021	1556.024	0.021	1556.024	0.021

NOTES: \* Depth (z) is measured to be positive below the underside of the pier pile cap (Elevation 90.6 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. Cyclic loading condition is considered.  
3. There are no pile group effects (i.e. analysis is based on a single pile).  
4. Resistance from pile cap embedment has not been considered.





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