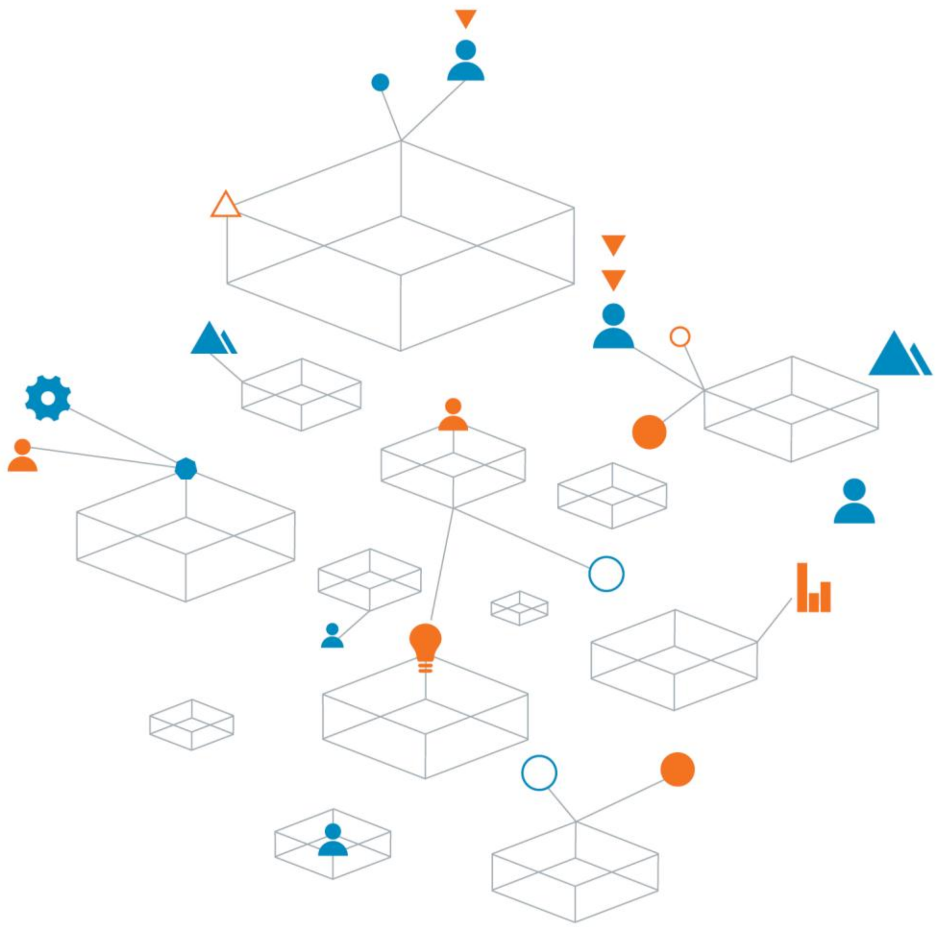
McCormick Rankin

Foundation Investigation and Design Report

Detour Bridge for Replacement of Glass's Bridge over The Innisfil Creek,
Site No. 30-254, Highway 89, Town of Innisfil, MTO Central Region,
W.P. 2108-11-00, GEOCRES NO. 31D-573
TRANETOB20462AA
16 April 2014



Trust is the cornerstone of all our projects coffey

16 April 2014

McCormick Rankin
2655 North Sheridan Way, Suite 300
Mississauga, Ontario
L5K 2P8

Attention: Ben Hui, P. Eng., M. Eng., Senior Project Manager

Dear Mr. Hui:

RE: Foundation Investigation and Design Reports
Detour Bridge for Replacement of Glass's Bridge over Innisfil Creek, Bridge Site No. 30-254, Town of Innisfil, MTO Central Region, W.P. 2108-11-00, GEOCRES No. 31D-573

Please find attached our foundation investigation and design reports for the above noted bridge site. The bridge site number refers to the existing bridge.

If you have any comments or enquiries please contact the undersigned.

For and on behalf of Coffey

Cam Mirza, P. Eng.
Principal

McCormick Rankin

Foundation Investigation Report

Detour Bridge for Replacement of Glass's Bridge over The Innisfil Creek,
Site No. 30-254, Highway 89, Town of Innisfil, MTO Central Region,
W.P. 2108-11-00, GEOCRES NO. 31D-573
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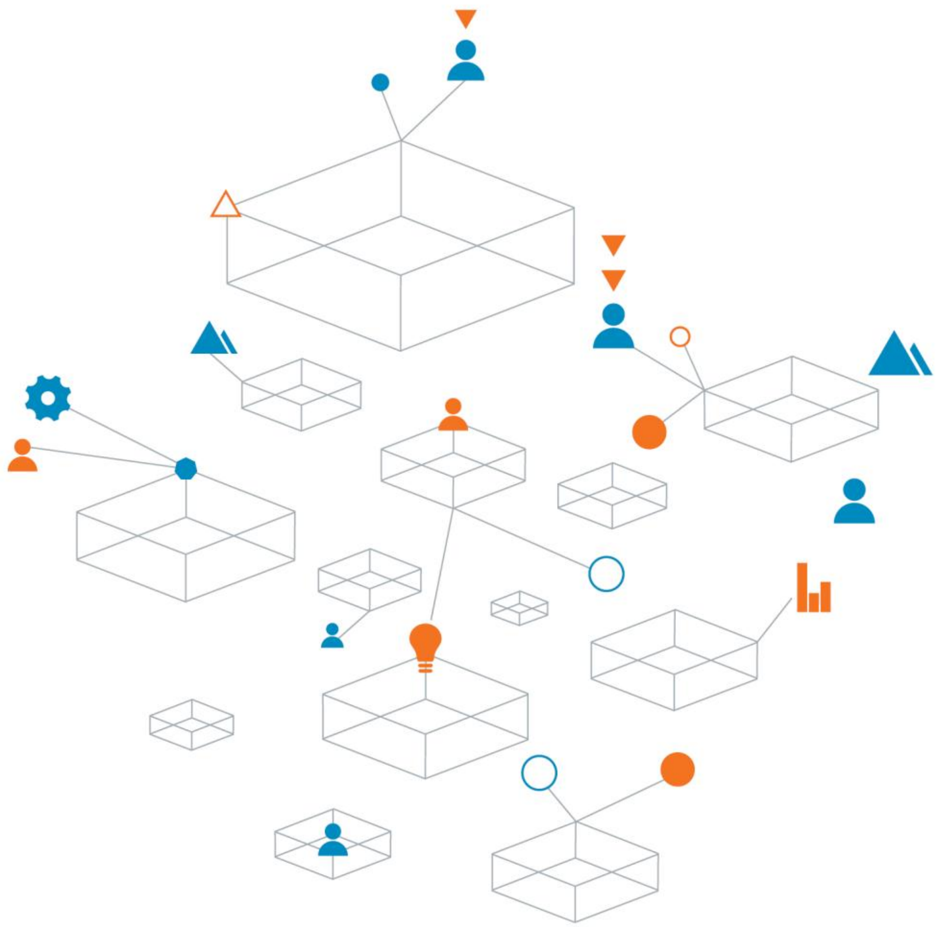

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CONTENTS

FOUNDATION INVESTIGATION REPORT
DETOUR BRIDGE FOR REPLACEMENT OF GLASS'S BRIDGE
OVER INNISFIL CREEK, SITE NO. 30-254, TOWN OF INNISFIL
MTO CENTRAL REGION, W.P. 2053-11-00

1 INTRODUCTION

Coffey was retained by McCormick Rankin (MRC) to carry out a foundation investigation for the proposed replacement of the existing Glass's Bridge over Innisfil Creek in the Town of Innisfil, Ontario. The structure (MTO Bridge Site No. 30-254) is located on Highway 89, about 3 km east of Cookstown or about 1 km west of Highway 400. The existing Highway 89 is a two lane arterial road, aligned more or less east-west.

To facilitate construction of the replacement bridge on the revised alignment, it is proposed to divert Highway 89 traffic to a temporary detour bridge, located about 30 m north of the existing bridge. This report provides geotechnical engineering subsurface data for the design and construction of the proposed detour bridge foundations and approaches. The detour structure will be a Baily bridge with its deck raised about one metre above existing ground level.

2 SITE DESCRIPTION AND GEOLOGY

The proposed detour bridge site is located about 30 m north of the existing structure. An MTO Patrol Yard is located about 150 m north-west of the present bridge location, on the north side of Highway 89. The proposed detour alignment runs more or less in a bow shape between the existing Highway 89 alignment and the south property limit of the Patrol Yard. The surrounding area is rural. The topography is flat in the immediate vicinity of the proposed bridge location but rises towards the north, east and west. Site photographs are shown in Appendix E.

Innisfil Creek flows from north to south beneath Highway 89 at the existing bridge location. The severe meandering of the stream suggests a mature hydrological regime and the possibility of buried ox bow lake deposits where meanders may have been cut off in the past. The stream banks, which are relatively steep, are stable. Deep erosion gullies are evident in a few locations in the stream reaches located within the project limits.

The project site is situated geologically in the southern portion of the Nottawasaga Basin which was at one time part of the floor of glacial Lake Algonquin. Surface deposits are of deltaic origin, underlain by lacustrine deposits. The southern portion of the Nottawasaga Basin represents a bay, separated from the main basin by moraine uplands. According to the "Physiography of Southern Ontario" (L.J. Chapman and D.F. Putnam, 1984) the site lies in the 'Simcoe Lowland' physiographic region.

A subsurface investigation in 2009 at the MTO Patrol Yard showed surficial fill, underlain by sandy silt over a thick deposit of silty clay extending well past the investigative maximum depth of 11 m.

Geological mapping suggests the depth to bedrock may be in excess of 40 m. The bedrock consists of shales and sandstones of the Ottawa and Simcoe Groups, Shadow Lake Formation.

3 FIELD INVESTIGATION

The field investigation consisted of laying out boreholes and sounding locations by reference to the detour centreline staking and NAD 83 northing and easting coordinates. All underground services were cleared prior to commencement of drilling and cone penetrometer test (CPT) soundings. Geodetic elevations at the borehole and sounding ground levels were provided by MRC. Borehole drilling, sampling and in situ testing was supervised by Coffey personnel. CPT soundings were supervised by DownUnder Geotechnical Limited. Boreholes were advanced with a track-mounted drill rig and hollow stem augers using mud-rotary techniques after a depth of 3-4 m. The fieldwork was performed between August 23 and September 04, 2013. A plan showing the location of the boreholes and CPT soundings is shown in Drawing No. 1. Table 3-1 shows the borehole and sounding locations. BH and CPT positions with respect to chainage and coordinates are given on the log sheets (Appendix A) and are shown on Drawings 1 and 2.

Table 3-1. Borehole (BH) / Sounding (CPT) Locations and Depths

BH / CPT No.	Location	Final Depth (m)	Remarks
BH11	W. Abutment, N side	12.8	-
CPT11	W. Abutment, N side	15.0	-
BH12	W. Abutment, S side	31.1	-
BH13	E. Abutment, N side	30.9	Piezometer, artesian condition
CPT14	E. Abutment, S side	15.0	-
BH15	W. Approach	11.3	-
BH16	E. Approach	11.3	-

Samples were taken at 0.76 to 1.5 m depth intervals down to 15 m and at 3 m or lesser depth intervals below 15 m in the Standard Penetration Test (SPT - ASTM D1586). The SPT N values were recorded in blows/0.3 m. In cohesive strata 75 mm dia. thin wall tube samples were taken by hydraulic pushing, followed by in situ vane shear testing with an MTO vane. All boreholes were decommissioned upon completion using regulatory MOE/MTO protocols.

The CPT soundings were made with a 35 mm diameter instrumented cone and friction sleeve assembly pushed hydraulically into the soil at an average rate of 2 cm/s, to a depth of 16 m below grade. At this depth the downward force pulled out one of the rig anchors and the test was stopped. The soundings were conducted with a 10 tonne capacity audio Geotech AB cone with a tip area of 10 cm^2, a friction sleeve area of 150 cm^2 and a porewater pressure u_2 pre-saturated filter. A cordless audio-cone device transmitted tip resistance, friction and pore pressure values to surface receivers. All measurements were corrected for verticality with the built-in inclinometer. A report on the CPT soundings is given in Appendix B.

Soil samples were placed in moisture proof containers for visual examination, classification and further laboratory testing. The testing included determination of moisture content, unit weight, plasticity, gradation analysis (both sieve and hydrometer) and one-dimensional consolidation. Laboratory test results are presented on the Office Record of Borehole Sheets (Appendix A) and in Appendix C.

4 SUBSURFACE CONDITIONS

4.1 General

The detour bridge site stratigraphy consists of 2-3 m thickness of surficial loose compressible fine sand and silt deposits of detrital fill and somewhat organic rich random zones, followed by 6-8 m thickness of loose to compact silty sand overlying a thick deposit of silty clay with silt-clayey silt and fine sand stringers in the upper and lower thirds of the deposit. This massive cohesive deposit is underlain at depths of 20-25 m by very dense silt, a source of artesian water.

4.2 Surficial Deposit

Surficial deposits are 100-200 mm of topsoil underlain by detritus fill material consisting of organic rich silty sand and silt resulting from sporadic flooding of the low lying land and importation of erosion by-products through surface flow into the site from higher surrounding ground. The thickness of this surficial deposit ranges between 1 m and 4 m. SPT N values of 0-13 blows /0.3 m indicate the deposit is generally very loose to loose, being occasionally compact in zones not rich in organic content. The organic content includes very thin slivers of peat and decayed wood in random spatial array. The soil is not considered organic silt or organic sand in the sense that those terms imply with respect to long term settlements. In one organic rich sample the moisture content was over 50 per cent. In other samples which contained organics, the natural moisture content was generally about 30-35 per cent.

4.3 Silty Sand

The surficial deposits are underlain by a grey silty sand (or sandy silt in some cases) that is loose to compact on the basis of recorded N values between 0 and over 35 blows/0.3 m. Some very low N values shown on the log sheets may be the result of unbalanced hydrostatic heads causing unintended boiling during hollow stem augering. The thickness of this cohesionless stratum varies between 6 m and 8 m. CPT11 and CPT14 soundings indicate a thickness range of 5.5 – 6.8 m.

The natural moisture content of the soil is consistently about 20 per cent. The wet unit weight of the soil in this stratum is 20 kN/m^3. The average gradation characteristics are shown below (see also Figure C-1 in Appendix C):

Gravel:	0 %
Sand:	36-64 %
Silt:	25-51 %
Clay size:	9-15 %

On the basis of gradation analysis, the soil is classified as sandy a silt to silty sand (SP-SM)

4.4 Silty Clay

The major stratigraphic unit is a 17-20 m thick deposit of grey silty clay that contains frequent stringers of clayey silt, silt and fine sand. The presence of these stringers is illustrated in the CPT soundings where an examination of pore pressure dissipation characteristic indicates frequently occurring very permeable thin seams in an otherwise homogeneous cohesive soil deposit. This deposit extended to depths of nearly 29 m below ground surface or to about elev. 196 m. CPT11 and CPT14 soundings were terminated within this deposit at depths of 15-16 m below the ground surface due to equipment anchor failure (pull out).

Hydrometer gradation results are shown below (see also Figure C-2 in Appendix C).

Gravel:	0-4 %
Sand:	0-13 %
Silt:	32-60 %
Clay size	40-59 %

The ranges in Atterberg Limits are shown below (see also Figure C-3 in Appendix C):

Liquid Limit:	26-51 %
Plastic Limit:	16-23 %
Plasticity Index:	12-28

On the basis of these tests, the soil is classified as silty clay (CI) with some segments being classified as clayey silt to silt (CL-ML). Occasionally, some clay rich seams classify as clay of high plasticity (CH).

The natural moisture content of this deposit lies between the liquid and plastic limits and ranges from 15% to 40%. The higher moisture content is associated with silty clay-clay and the lower with clayey silt-silt and fine sand. The liquidity index averages about 0.5. However, in siltier portions, the liquidity index is much greater than unity. The average unit weight of the soil in this deposit is 18-19 kN/m^3.

SPT N values of 6 blows/0.3 m to over 100 blows/0.3 m suggest a firm to hard consistency, confirmed by in situ vane undrained shear strengths of 90-200 kPa. Some very high values (± 200 kPa) may be the result of the field vane penetrating silt or fine sand stringers. The variation of undrained shear strength with depth is shown on Figure D-1 in Appendix D. Also plotted on Figure D-1 is the effective overburden stress (P'_o), and $0.23P'_o$. It is commonly understood for most cohesive soil deposits that if the measured undrained shear strength is in excess of $0.23P'_o$, the deposit is likely to be overconsolidated. Figure D-1 suggests this deposit is overconsolidated.

The result of a single 1-D consolidation test is given in Figure C-4, Appendix C. The wet unit weight of the tested sample was 19.6 kN/m^3, being somewhat higher than the average, and indicative of the presence of very thin silt and fine sand inclusions within the trimmed sample. From the e-log p curve, the estimated pre-consolidation pressure, P_c, is about 350 kPa, yielding an overconsolidation ratio (OCR) of about 3. The compression index (C_c) is 0.15 the recompression index (C_r) is 0.03.

4.5 Silt

The silty clay deposit is underlain below about elev. 196-199 m by a very dense uniformly grey coarse silt deposit. Prior to testing by dispersion with sodium hexametaphosphate for hydrometer analysis, the visual appearance of this soil is that of a fine sand; a gritty feel is evident upon tactile examination. BH12 and BH13 were terminated within this deposit at a depth of about 31 m.

The SPT N values in this deposit ranged from 67 blows/0.3 m to in excess of 100 blows/0.3 m, indicating it is very dense.

This silt stratum was the source of an artesian head of 4 m above ground level after being penetrated more than a metre by mud drilling methods in BH 13.

5 GROUNDWATER CONDITIONS

The phreatic surface (surface groundwater level) is located about 2 m below ground surface.

In BH13, an initial 4 m artesian head above the ground surface was measured by extending the hollow stem auger casings. This hole was decommissioned three days after piezometer installation due to continued flow of groundwater. Decommissioning was accomplished by re-drilling the borehole and grouting it in accordance with regulatory requirements.

The stream level in Innisfil Creek on September 20, 2012 was at elev. 222.8 and at elev. 223.4 on August 1, 2013. The 50 year flood level is said to reach elev. 224.9 m (information supplied by others).

Surface groundwater levels are subject to seasonal fluctuations, stream level changes, prior weather events and rates of infiltration and evapotranspiration.

6 CLOSURE

Borehole drilling services were provided by Davis Drilling of Milton, Ontario, working under the supervision of Mr. Lorne Granville, EIT, reporting to the undersigned. CPT field and testing services were provided by DownUnder Geotechnical Limited who used special equipment and a drill rig supplied by Strata Soil Sampling Inc. of Richmond Hill, Ontario.

We appreciate the opportunity provided to Coffey Geotechnics to present factual subsurface data for the proposed detour bridge across Innisfil Creek. Please call if you require assistance or need clarification.

For and on behalf of Coffey

Gwangha Roh, Ph. D., P. Eng.
Senior Geotechnical Engineer

C. V. Wijeyakulasuriya

Vasantha Wijeyakulasuriya, P. Eng.
Senior Principal

LICENSED PROFESSIONAL ENGINEER
V. WIJEYAKULASURIYA
100186910
17 April 2014
PROVINCE OF ONTARIO

Cam Mirza, P. Eng.
Principal

LICENSED PROFESSIONAL ENGINEER
C. MIRZA
31976012
PROVINCE OF ONTARIO

Drawings

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

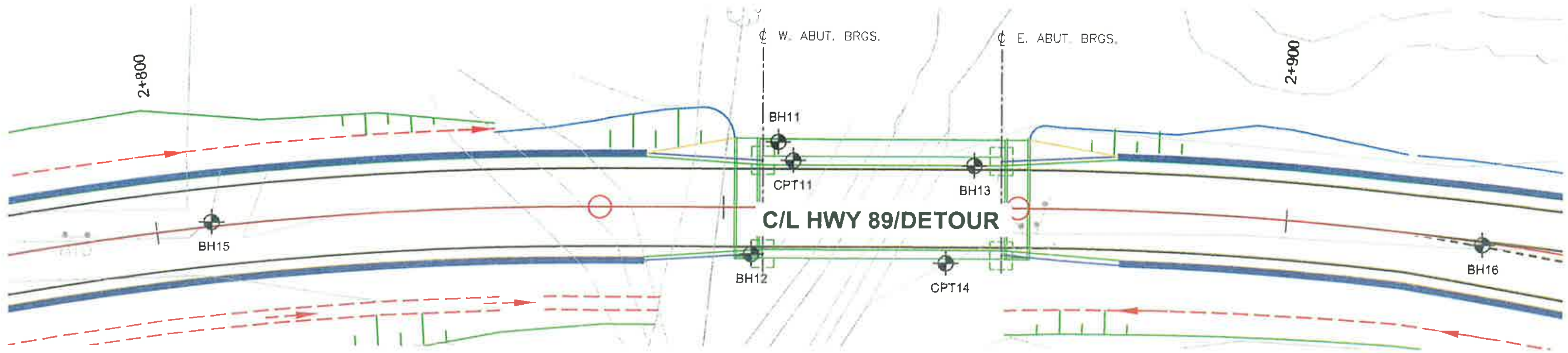
CONT No. -

WP: 2108-11-00

HIGHWAY 89 DETOUR
INNISFIL CREEK
BOREHOLE LOCATION PLAN
AND SOIL STRATA 1

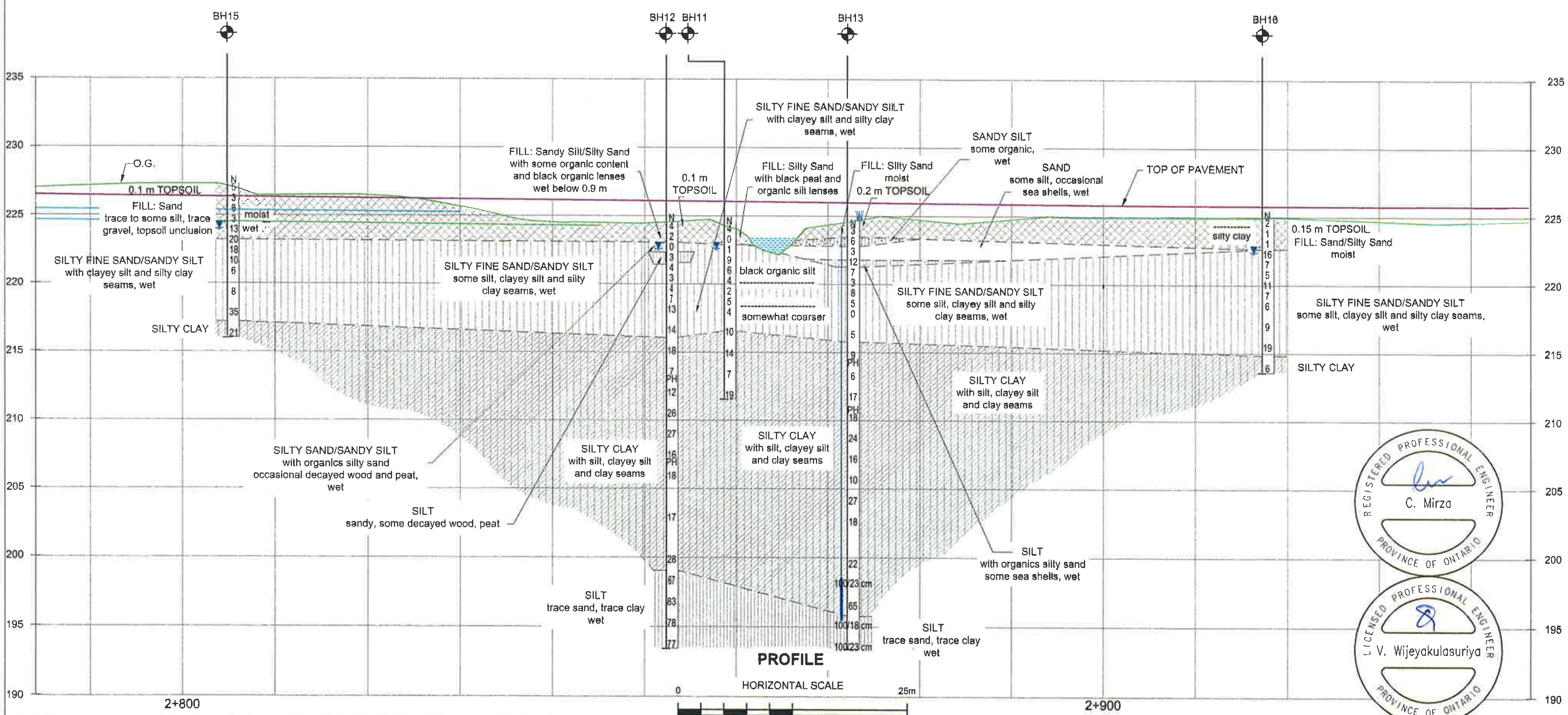
SHEET

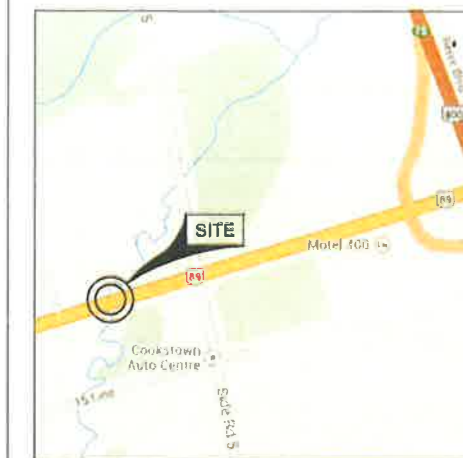
coffey



PLAN

0 SCALE 25m





KEY PLAN
N.T.S.

LEGEND

- Borehole
- N — Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation
- Water Level in Piezometer
- Piezometer
- Artesian

No.	ELEVATION	STATION	OFFSET
BH11	224.367	2+856	4.3 m Lt C/L
CPT11	224.598	2+855	5.9 m Lt C/L
BH12	224.551	2+862	4.1 m Rt C/L
BH13	224.261	2+872	3.9 m Lt C/L
CPT14	224.874	2+870	4.9 m Rt C/L
BH15	227.293	2+805	0.5 m Lt C/L
BH16	224.942	2+917	1.3 m Lt C/L

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

NOTE: This drawing is for subsurface information only.. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31D-573

TRANETOB20462AA			DIST
SUBM'D	CHECKED	DATE April , 2014	SITE 30-254
DRAWN SSH	CHECKED GR	APPROVED ZO	DWG 1

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

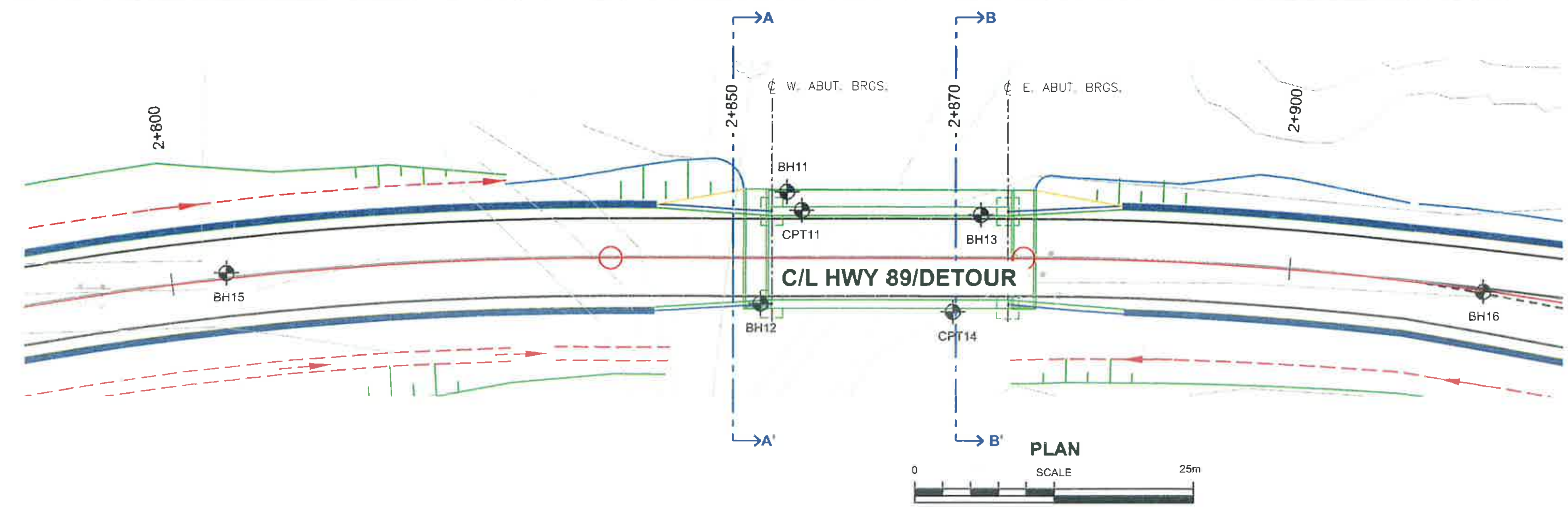
CONT No. -

WP: 2108-11-00

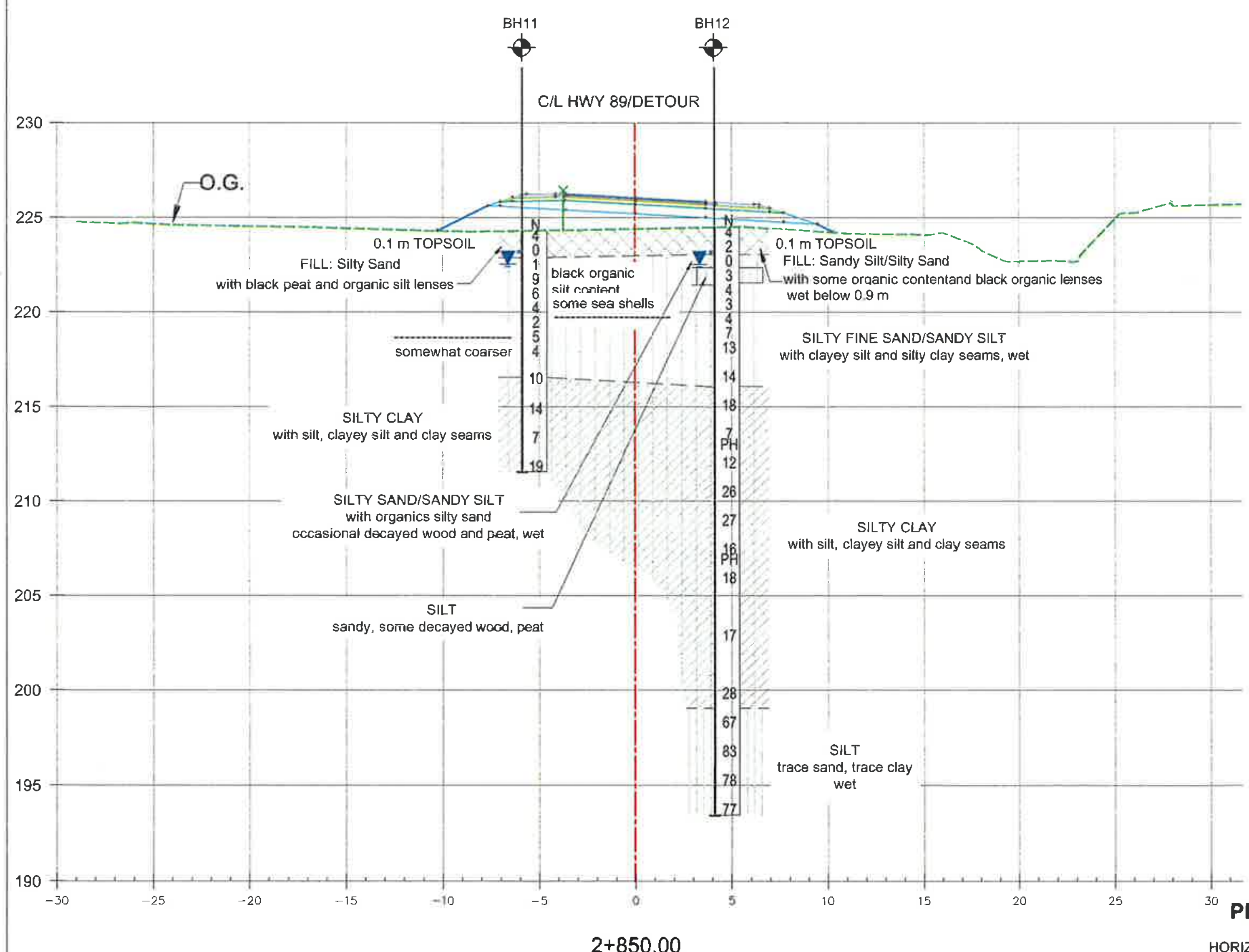
HIGHWAY 89 DETOUR
INNISFIL CREEK BRIDGE
BOREHOLE LOCATION PLAN
AND SOIL STRATA 2

SHEET

coffey

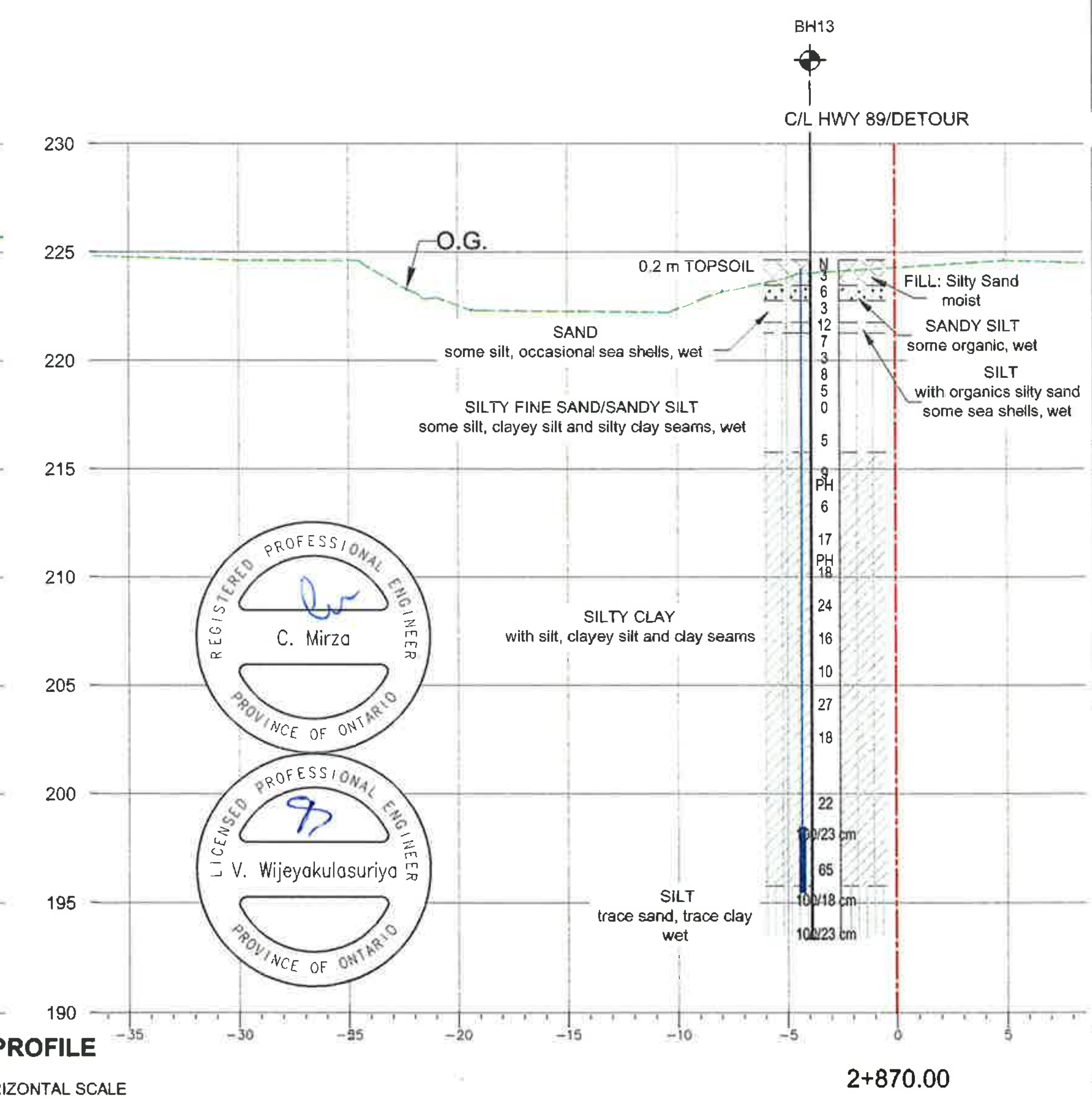


KEY PLAN
N.T.S.



2+850.00

SECTION A-A'



2+870.00

SECTION B-B'

LEGEND

⊕	Borehole
N	Blows/0.3m (Std. Pen. Test, 475 J/blow)
▽	Water Level at Time of Investigation
▽	Water Level in Piezometer
	Piezometer
A A'	Section

No.	ELEVATION	STATION	OFFSET
BH11	224.367	2+856	4.3 m Lt C/L
BH12	224.551	2+852	4.1 m Rt C/L
BH13	224.261	2+872	3.9 m Lt C/L

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

NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31D-573

	TRANETOB20462AA			DIST	
SUBM'D	CHECKED	DATE	April , 2014	SITE	30-254
DRAWN SSH	CHECKED GR	APPROVED ZO		DWG	2

Appendix A

Record of Borehole Sheets

RECORD OF BOREHOLE No BH11

METRIC

GWP: WP 2108-11-00	LOCATION: 12+855, 4 m Lt C/L (N 4895322.851, E 291399.173)		ORIGINATED BY: LG
DIST: 5	HWY: 89	BOREHOLE TYPE: Hollow Stem Auger and Mud Rotary Drilling	COMPILED BY: SSH
DATUM: Geodetic	DATE: 28/08/2013 28/08/2013		CHECKED BY: ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
224.4	GROUND SURFACE					
0.0	0.1 m **TOPSOIL** **FILL:** Silty Sand with black peat and organic silt lenses brown, very loose	1	SS	4		
		2	SS	0		
222.9 1.5	black organic silt content some sea shells	3	SS	1	▼*	wet spoon
	SILTY FINE SAND/SANDY SILT with clayey silt and silty clay seams grey, very loose to loose, wet	4	SS	9		
		5	SS	6		
		6	SS	4		
		7	SS	2		
		8	SS	5		
	somewhat coarser	9	SS	4		
216.6 7.8	**SILTY CLAY** with silt, clayey silt and clay seams grey, firm to very stiff	10	SS	10		
		11	SS	14		1 13 32 54
		12	SS	7		
211.6		13	SS	19		
12.8	End of Borehole * Groundwater @ 1.8 m (El. 222.6 m) based on wetness of sample (not stabilized).					

ELEVATION SCALE: 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH12

METRIC

GWP	WP 2108-11-00	LOCATION	12+853, 4 m Rt C/L (N 4895312.509, E291399.825)
DIST	5 HWY 89	BOREHOLE TYPE	Hollow Stem Auger and Mud Rotary Drilling
DATUM	Geodetic	DATE	23/08/2013 27/08/2013

ORIGINATED BY LG
COMPILED BY SSH
CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
224.6	GROUND SURFACE						
0.0	0.1 m **TOPSOIL** **FILL:** Sandy Silt/Silty Sand with some organic content and black organic lenses brown/dark brown/black, very loose, wet below 0.9 m	1	SS	4		224	
		2	SS	2			
223.1 / 1.5	**SILTY SAND/SANDY SILT** with organics silty sand occasional decayed wood and peat black, very loose, wet	3	SS	0	▼	223	wet spoon
222.4 / 2.2	**SILT** sandy, some decayed wood, peat black, very loose	4	SS	3		222	
221.6 / 3.0	**SILTY FINE SAND/SANDY SILT** some silt, clayey silt and silty clay seams grey, wet	5	SS	4		221	0 54 37 9
		6	SS	3		220	
		7	SS	4			
	very loose	8	SS	7		219	
	compact	9	SS	13		218	
		10	SS	14		217	
216.1 / 8.5	**SILTY CLAY** with silt, clayey silt and clay seams grey, firm to very stiff					216	
		11	SS	18		215	
		12	SS	7		214	
		13	TW	PH		213	
		14	SS	12		212	
		15	SS	26		211	
						210	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/m^3

Water content plotted values: 48 (sample 2), 53 (sample 3)

Continued Next Page

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH12

METRIC

GWP	WP 2108-11-00	LOCATION	12+853, 4 m Rt C/L (N 4895312.509, E291399.825)
DIST	5 HWY 89	BOREHOLE TYPE	Hollow Stem Auger and Mud Rotary Drilling
DATUM	Geodetic	DATE	23/08/2013 27/08/2013
ORIGINATED BY	LG	COMPILED BY	SSH
CHECKED BY	ZO		

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10 20 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GRAIN SIZE (GR SA SI CL)
209.6	**SILTY CLAY** with silt, clayey silt and clay seams grey, very stiff		16	SS	27	209	
						208	
			17	SS	16		
			18	TW	PH	207	0 0 60 40
			19	SS	18	206	
						205	
						204	
			20	SS	17	203	
						202	
						201	
			21	SS	28	200	
199.1 25.5	**SILT** trace sand, trace clay grey, very dense, wet					199	
			22	SS	67	198	
			23	SS	83	197	
						196	
			24	SS	78	195	

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH12 METRIC

GWP WP 2108-11-00 LOCATION 12+853, 4 m Rt C/L (N 4895312.509, E291399.825) ORIGINATED BY LG

DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH

DATUM Geodetic DATE 23/08/2013 27/08/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
194.6											
	SILT trace sand, trace clay grey, very dense, wet						194				
			25	SS	77				○ (~20)		
193.5 31.1	End of Borehole * Groundwater @ 2 m (El. 222.6 m) based on wetness of sample (not stabilized).										

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

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH13

METRIC

GWP	WP 2108-11-00	LOCATION	12+870, 4 m Lt C/L (N 4895325.983, E 291416.424)
DIST	5 HWY 89	BOREHOLE TYPE	Hollow Stem Auger and Mud Rotary Drilling
DATUM	Geodetic	DATE	28/08/2013 29/08/2013

ORIGINATED BY LG
COMPILED BY SSH
CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT (%)	UNIT WEIGHT γ (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
224.3	GROUND SURFACE						
0.0	0.2 m **TOPSOIL** **FILL:** Silty Sand, dark brown, very loose, moist	1	SS	3			
223.5 / 0.8	**SANDY SILT** some organics, brown to dark brown, loose, wet	2	SS	6			
222.8 / 1.5	**SAND** some silt, occasional sea shells, grey, very loose, wet	3	SS	3			wet spoon
221.8 / 2.5	**SILT** with organics silty sand, some sea shells, dark grey, compact, wet	4	SS	12			
221.3 / 3.0	**SILTY FINE SAND/SANDY SILT** some silt, clayey silt and silty clay seams, grey, very loose to loose, wet	5	SS	7			
		6	SS	3			0 36 51 13
		7	SS	8			0 38 47 15
		8	SS	5			
		9	SS	0			
		10	SS	5			
215.8 / 8.5	**SILTY CLAY** with silt, clayey silt and clay seams, grey, firm to very stiff	11	SS	9			4 5 33 59
		12	TW	PH	51	19.6	consolidation test
		12	SS	6			
		13	SS	17			
		14	TW	PH			
		15	SS	18			

Field vane: 120.0 (at approx. elev. 214)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH13

METRIC

GWP WP 2108-11-00 LOCATION 12+870, 4 m Lt C/L (N 4895325.983, E 291416.424) ORIGINATED BY LG

DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH

DATUM Geodetic DATE 28/08/2013 29/08/2013 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
209.3					
	SILTY CLAY with silt, clayey silt and clay seams grey, firm to very stiff				209
		16	SS	24	
					208
		17	SS	16	
					207
					206
		18	SS	10	
					205
		19	SS	27	
					204
					203
		20	SS	18	
					202
					201
					200
		21	SS	22	
					199
		22	SS	100/23 cm	
					198
					197
		23	SS	65	
					196
195.8 28.5	**SILT** trace sand, trace clay grey, very dense, wet				
		24	SS	100/18 cm	
					195

Field vane shear strength values plotted: 360.0 (at approx. elev. 205), 310.0 (at approx. elev. 202).

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

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

Continued Next Page

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH13

METRIC

GWP WP 2108-11-00 LOCATION 12+870, 4 m Lt C/L (N 4895325.983, E 291416.424) ORIGINATED BY LG

DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH

DATUM Geodetic DATE 28/08/2013 29/08/2013 CHECKED BY ZO

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) (20 40 60 80 100)	WATER CONTENT (%) w_P – w – w_L (10 20 30)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
194.3											
	SILT trace sand, trace clay grey, very dense, wet						194				
193.4			25	SS	100/23 cm						
30.9	End of Borehole * Groundwater @ 1.8 m (El. 222.5 m) based on wetnes of sample (not stabilized). Artesian condition was noted in piezometer (3.9 m above grade, measured by casings, on Sept 4, 2013)										

Legend: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH15

METRIC

GWP WP 2108-11-00 LOCATION 12+805, @ C/L (N 4895301.125, E 291353.611) ORIGINATED BY LG

DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH

DATUM Geodetic DATE 28/08/2013 28/08/2013 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
227.3	GROUND SURFACE							
0.0	0.1 m **TOPSOIL**	1	SS	5		227		
	FILL: Sand trace to some silt, trace gravel topsol inclusion brown, very loose to compact	2	SS	3		226		
		3	SS	6		225		
	moist	4	SS	3				
	wet	5	SS	13	▼*	224		wet spoon
223.2 4.1	**SILTY FINE SAND/SANDY SILT** with clayey silt and silty clay seams grey, loose to compact, wet	6	SS	20		223		
		7	SS	18		222		0 64 25 11
		8	SS	10				
		9	SS	6		221		
						220		
		10	SS	8		219		
		11	SS	35		218		
217.2 10.1	**SILTY CLAY** grey, very stiff					217		
		12	SS	21				
216.0 11.3	End of Borehole * Groundwater @ 3.3 m (El. 224.0 m) based on wetness of sample (not stabilized).							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BH16

METRIC

GWP WP 2108-11-00 | LOCATION 12+918, @ C/L (N 4895314.493, E 291453.186) | ORIGINATED BY

DIST 5 HWY 89 | BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling | COMPILED BY

DATUM Geodetic | DATE 30/08/2013 30/08/2013 | CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS
224.9	GROUND SURFACE					
0.0	0.15 m **TOPSOIL**	1	SS	2		
	FILL: Sand/SILTY SAND brown, very loose, moist	2	SS	1	224	
		3	SS	1	223	
222.6	silty clay					
2.3	**SILTY FINE SAND/SANDY SILT** with clayey silt and silty clay seams trace gravel grey, very loose to compact, wet	4	SS	16	222	wet spoon
		5	SS	7		
		6	SS	5	221	
		7	SS	11	220	
		8	SS	7	219	
		9	SS	6	218	
		10	SS	9	217	
					216	
		11	SS	19	215	
214.8 / 10.1	**SILTY CLAY** grey, very stiff	12	SS	6	214	
213.6 / 11.3	End of Borehole * Groundwater @ 2.5 m (El. 222.4 m) based on wetness of sample (not stabilized).					FVT @ 11.6 m

Field vane: 150.0 >>

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Appendix B

Cone Penetration Test Report

PIEZOCONE PENETRATION TESTING
PROPOSED BRIDGE CROSSING AT INNISFIL CREEK
HIGHWAY 89, ONTARIO

For:
Strata Drilling Group
147 West Beaver Creek Road, Unit 2
Richmond Hill, Ontario
L4B 1C6

April 2014
Ref. No. D13109

DownUnder Geotechnical Limited
P.O. Box 96737, Jane/Major Mackenzie P.O., 2943 Major Mackenzie Drive, Maple, Ontario L6A 0A2
Tel 905-553-2483 Toll Free Fax 1-866-478-4593 Email office@downundergeotechnical.com

TABLE OF CONTENTS

1.0 INTRODUCTION

Downunder Geotechnical Limited (DownUnder Geotechnical) was retained by Strata Drilling Group to carry out PiezoCone Penetration Tests (CPT) at a proposed bridge crossing over Innisfil Creek at Highway 89 in Ontario. This report contains the findings of piezocone soundings advanced by DownUnder Geotechnical Limited.

2.0 FIELD TESTING PROCEDURES

The CPT soundings were carried out on September 4 and 5, 2013. All CPT soundings were carried out in general accordance with ASTM standards (D 5778). The CPT soundings were carried out using a direct push Geoprobe drill rig owned and operated by Strata Soil Sampling Inc. of Richmond Hill, Ontario, under the full-time supervision of DownUnder Geotechnical Limited. The light weight drill rig was anchored into the soil using solid stem augers.

At CPT-11a and CPT-14 locations a 35mm diameter instrumented cone and friction sleeve assembly was hydraulically thrust into the soil at a rate of about 2 cm/s to depths of 14.9 to 15.7m below grade where refusal was encountered due to pull-out of one of the anchors. The soundings were conducted using a 10 tonne capacity audio GEOTECH AB cone with a tip area of 10 cm^2, a friction sleeve area of 150 cm^2 and a u_2 filter location. The pore pressure brass filters were saturated overnight with glycerine under pressure. The cordless audio-cone uses sound waves to transmit the measured tip resistance, friction and pore pressure results up through the rods to a microphone at the surface. Measurements were taken at about 2 cm depth intervals during penetration and corrected for verticality based on the inclinometer readings in the cone. The sound waves are then decoded by a CPT-interface and sent to a laptop computer on-site. The cone calibration record is included in Appendix A.

At CPT-4 location a 35mm diameter instrumented cone and friction sleeve assembly was hydraulically thrust into the soil at a rate of about 2 cm/s to a depth of 23.0 m below grade where refusal was encountered due to pull-out of one of the anchors. The sounding was conducted using a 100 MPa capacity VERTEK cone with a tip area of 10 cm^2, a friction sleeve area of 150 cm^2 and a u_2 filter location. The pore pressure plastic filters were purchased pre-saturated with silicone oil. Measurements were taken at about 2 cm depth intervals during penetration and corrected for verticality based on the inclinometer readings in the cone. The cone calibration record is included in Appendix A.

Figure No.1 presents the approximate CPT locations. The CPT soundings are included graphically in Appendix B.

The GEOTECH AB cone had difficulty maintaining contact with the microphone during dissipation tests likely due to the stiff clays and dilatant nature of the silts. The VERTEK cone was used to obtain consistent dissipation results.

3.0 CPT RESULTS

The results of the soundings are presented in Appendix B. Each sounding log comprises the measured results and soil behaviour classification. Interpreted geotechnical parameters are discussed in Section 4.0. The following provides a brief discussion on each of the measured results.

Tip Resistance

The CPT provides a continuous measurement of the cone resistance, q_c. The measured cone resistance is corrected to total cone resistance, q_t, using the following equation,

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

where u_2 = pore pressure acting behind the cone

a = cone area ratio = A_n/A_c

= 0.57 for GEOTECH AB audio cone

= 0.82 for VERTEK SCPTu cone

A_n = cross-sectional area of the load cell or shaft

A_c = projected area of the cone

Sleeve Friction and Friction Ratio

The friction along the cone sleeve, f_s, is continuously measured during cone penetration. Friction Ratio is a commonly used parameter for determination of soil profiling and classification. Friction ratio is determined by the following equation.

$$FR\ (\%) = \frac{f_s}{q_t}$$

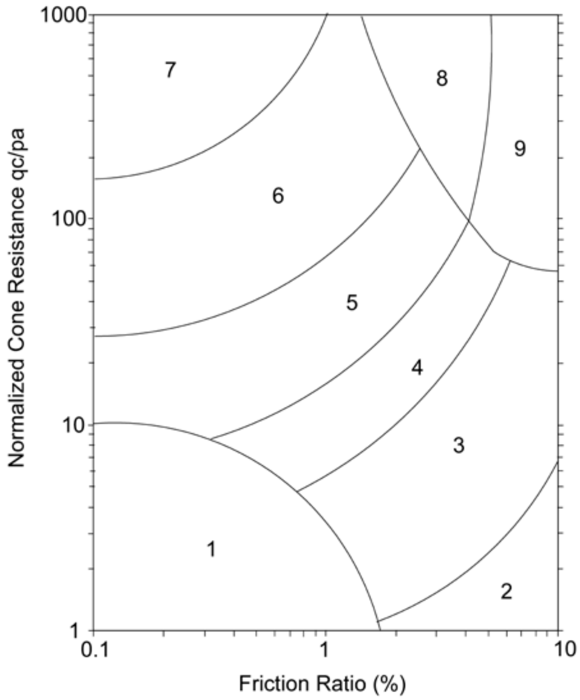
Pore Pressure

Continuous measurements of porewater pressure are taken during penetration. Due to the dynamic nature of the cone penetration, the porewater pressure measurements within fine grained soils are not representative due to undrained conditions and may even be negative in overconsolidated soils or dilatant silts.

Dissipation tests within fine grained soils are carried out by stopping penetration and measuring the change in excess porewater pressure over time. These results can provide an indication of hydraulic conductivity and consolidation characteristics, as well as soil behaviour – drained or undrained. In normally consolidated soils the excess porewater pressures dissipate during the test. In heavily overconsolidated or dilatant soils there is a delay in porewater pressure dissipation due to redistribution of the excess pore pressure behind the shoulder of the cone tip and the excess porewater pressures increase to a maximum before dissipating. The time for 50% dissipation is also an indicator of drained or undrained behaviour. Seventeen (17) dissipation tests were carried out during stoppage in penetration.

Soil Behaviour Type

One of the main applications of CPT soundings is for rapid soil profiling and classification. Normalized soil behaviour type (SBTn) on the sounding logs is based on the classification chart by Roberston (1990). A reproduction of one of the charts and the soil behaviour types are presented in the chart below. The chart is typically a 2-chart system, one assessing normalized cone resistance vs. friction ratio and the second chart assessing normalized cone resistance vs. pore pressure ratio (which is not presented).



NORMALIZED SOIL BEHAVIOUR TYPE (after Robertson 1990)

ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty Clay to Clay
5		Silty Sand to Sandy Silt
6		Sand to Silty Sand
7		Sand
8		Very dense/stiff soil*
9		Very dense/stiff soil*

* heavily overconsolidated and/or cemented

To simplify the SBTn charts, Jefferies and Davies (1993) proposed a CPT Soil Index I_C, which is also used as an indicator for soil stratigraphy, and was further normalized by Robertson (2009).

$$I_C = [(3.47-\log(Q_t))^2 + (1.22 + (\log F))^2]^{0.5}$$

where Q_t = normalized tip resistance = $(q_t - \sigma_{V0}) / \sigma_{V0}'$
F = normalized sleeve friction = $f_s / (q_t - \sigma_{V0})$

It should be noted that the above chart is an indication of soil behaviour and not an indication of grain size distribution.

4.0 INTERPRETATION

Undrained Shear Strength

The relationship between cone resistance and undrained shear strength can be empirically represented by the following equation.

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

where S_u = undrained shear strength (kPa)
σ_V= vertical stress (kPa)
N_{kt}= dimensionless constant

Typically N_{kt} varies from 10 to 20, with higher results in fissured clay, silts or varved clay deposits. Published empirical correlations also exist relating undrained shear strength, in situ effective vertical stress and OCR. In order to maintain the following empirical relationship $S_u/\sigma_{v0}' \sim 0.22$ OCR, a N_{kt} of 23 provides reasonable results. Undrained shear strengths were determined for SBTn 3 and 4 (silty clay to clayey silt) and SBTn of 5 (silt). The N_{kt} value can be confirmed by comparison with in situ shear vane test results.

Equivalent N_{60} SPT Value

Based on Jefferies and Davies (1993) the following empirical equation is used to correlate to equivalent Standard Penetration Test results.

$$N_{60} = \frac{q_C}{0.85 \times (1 - I_C/4.75)}$$

where q_C = tip resistance (MPa)
I_C = Soil Classification Index

Overconsolidation Ratio (OCR)

The estimate of the overconsolidation ratio, OCR, in clays is based on the following equation,

$$OCR = k\,(q_t - \sigma_v)/\sigma'_v$$

Where k is constant typically ranging from 0.3 to 0.5 for clays. A 'k' value of 0.2 was used for the clayey deposits at the site, which is typical of Greater Toronto Area soils at other sites tested.

Constrained Modulus

The constrained modulus, M, represents the deformation characteristics of soils for preconsolidation stresses, and is a function of the stress history, drainage condition and the stress path direction of the soil. The estimate of M for sands is based on the Robertson (2009) method. The estimate of M for clayey soils can be estimated using the Robertson (2009) method or that proposed by Senneset et al (1982) method. The Senneset et al method is presented in Appendix B as it provides a more conservative result.

$$M = 1/m_v = \alpha_m\,(q_t - \sigma_V)$$

where m_v = coefficient of volume change
α_m = constant

Robertson Method:

For I_c <2.2 (Sands):

$$\alpha_m = 0.0188\,[\,10^{(0.55\,Ic + 1.68)}]$$

For I_c >2.2 (Clays):

$\alpha_m = Q_t$ when $Q_t < 14$
$\alpha_m = 14$ when $Q_t > 14$

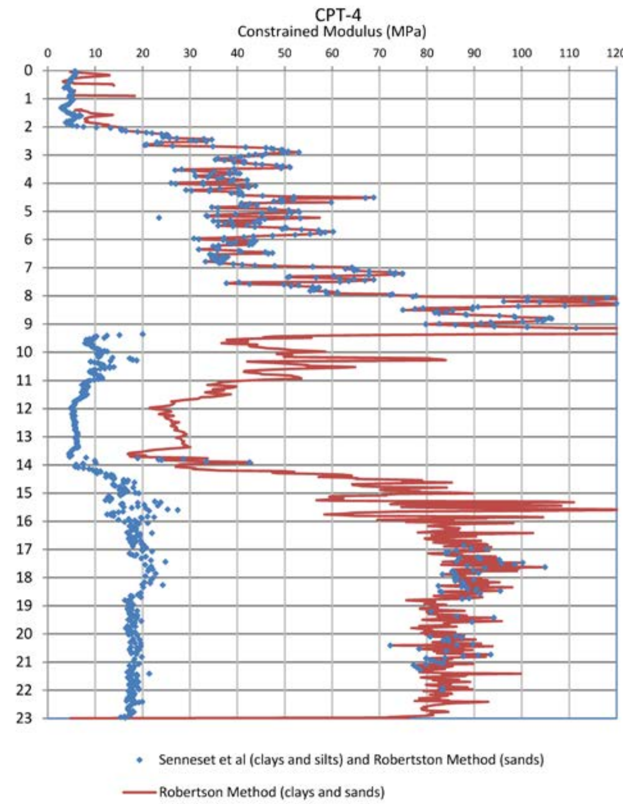
Senneset et al Method:

For SBTn <6 (Silts, Clays and Clayey Silts):

$\alpha_m = 3$

The α_m value above was selected based on odeometer testing in similar soils.

Below are comparison of the two above methods for CPT-4 location.



M is generally equivalent to 90% of Young's Modulus (E). It should be noted that E (and M) is a stress dependent value and non-linear in nature. In order to provide a more accurate value comparison with consolidation test results should be made.

Effective Friction

The following equation was used for SBTn 6 to 8 ("clean sands") and SBTn 5 (silt/sandy silt to silty sand).

$$\text{Friction Angle (degrees)} = \varphi_{p'} = 17.6^0 + 11 \text{ LOG } Q_t$$

For cohesive soils (plastic silts and clays) the following equation can be used to estimate friction angles, however it is only valid where $0.1 < B_q < 1.0$. This equation is not shown in the CPT soundings as the results provide high friction angles at this site and are not considered representative.

Friction Angle (degrees) = $29.5\ B_q^{\ 0.121}\ (0.256+0.336B_q+\log Q_t)$

Where Q_r = normalized tip resistance
B_q= normalized excess porewater pressure reading

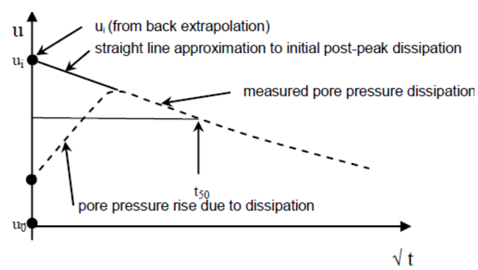
Coefficient of Consolidation

The horizontal coefficient of consolidation (C_h) of the soil can be estimated from the pore pressure dissipation test results. Montonic and dilatory excess pore pressure dissipation was observed in the seventeen (17) tests carried out at the site. The method by Houlsby and Teh (1988) was used to determine C_h, as follows.

$$C_h = \frac{T^*_{50}}{t_{50}}\ r^2\ I_R^{0.5}$$

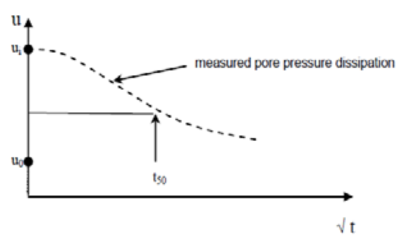
where T^*_{50}= time factor from theoretical solutions = 0.245
t_{50}= measured time for 50% dissipation
r = penetrometer radius = 17.8 cm
I_R = undrained rigidity index = G/S_u
G = shear modulus

Due to the dilative nature of most of the soils, the excess pore pressure increased during the test before dissipating (dilatory behaviour). In order to determine the 50% dissipation the test measurements were plotted for excess pore pressure vs root time scale. The initial excess pore pressure was then estimated by extrapolating back to time zero as presented in the following sketch.



Overconsolidated or Dilative Soils

In other tests an increase in pore pressure was not observed and the following sketch represents the monotonic behaviour observed.



From the initial pore pressure estimation, the normalized excess pore pressure was determined and plotted vs time. Normalized excess pore pressure was determined based on the following equation and the graphs are included in Appendix C.

$$U = \frac{u_t - u_0}{u_i - u_0}$$

where u_t = excess pore water pressure measurement at time t
u_0= in situ pore pressure based on the CPT results
u_i = initial excess pore water pressure at beginning of dissipation test

To correlate C_h to the vertical coefficient of consolidation (C_v) the following equation was used:

$$C_v = C_h\, k_v/k_h$$

where k_v/k_h ratio is suggested in the table below from Jamiolkowski (1985).

Nature of Clay	k_h/k_v
No macrofabric or slightly developed macrofabric (homogeneous deposit)	1 to 1.5
Fairly well to well developed macrofabric (eg. sedimentary clays with discontinuous lenses and layers of more permeable material)	2 to 4
Varved clays and other deposits containing embedded and more or less continuous permeable layers	3 to 15

The results are considered to be approximate and reasonable to within an order of magnitude.

5.0 SUMMARY OF RESULTS

The subsurface and statistical analysis of the results is divided into four zones as follows:

Borehole Description	Depth below existing grade (m)			Inferred Consistency or Compactness
	CPT-4	CPT-11a	CPT-14	
Sandy Silt to Clayey Silt	2.1	3.1	2.7	firm to stiff
Sand to Silty Sand	9.4	8.6	9.5	compact
Clayey Silt to Silty Clay	16.1	>14.9	>15.7	stiff to hard
Silt	>23.0	-	-	hard

The following tables summarize the interpreted geotechnical parameters from the CPT testing as per the above groupings. For each of the geotechnical parameters the mean value and standard deviation is provided. A characteristic value can be assigned as opposed to the mean value based on the designer's judgement.

Sandy Silt to Clayey Silt behaviour

Location	Depth (m)	Mean q_t (MPa)	Mean N_{60} (blows/0.3m)	Mean *M* (MPa)	OCR	S_u (kPa)
CPT-4	0 to 2.1	1.7 σ=0.5	3 σ=3	4.9 σ=1.4	28.0 σ=22.0	71 σ=20
CPT-11a	0 to 3.1	1.4 σ=0.5	3 σ=1	4.3 σ=1.3	20.0 σ=14.8	47 σ=10
CPT-14	0 to 2.7	1.1 σ=0.5	2 σ=1	3.4 σ=1.6	13.9 σ=8.9	43 σ=15

Silty Sand to Sand behaviour

Location	Depth (m)	Mean q_t (MPa)	Mean N_{60} (blows/0.3m)	Mean φ'	Mean *M* (MPa)
CPT-4	2.1 to 9.4	11.0 σ=3.0	20 σ=5	42^0 σ=2	55 σ=23
CPT-11a	3.1 to 8.6	10.2 σ=2.7	18 σ=4	41^0 σ=3	48 σ=14
CPT-14	2.7 to 9.4	35.3 σ=23.5	18 σ=4	41^0 σ=3	49 σ=11

*Clayey Silt to Silty Clay behaviour**

Location	Depth (m)	Mean q_t (MPa)	Mean N_{60} (blows/0.3m)	Mean *M* (MPa)	OCR	S_u (kPa)
CPT-4	9.4 to 16.1	3.7 σ=1.6	10 σ=4	10.7 σ=5.6	5.3 σ=2.2	150 σ=71
CPT-11a	8.6 to 14.9	3.7 σ=1.6	10 σ=4	10.6 σ=7.5	5.0 σ=1.6	127 σ=40
CPT-14	9.5 to 15.7	2.8 σ=0.7	8 σ=2	7.6 σ=2.0	3.8 σ=1.0	107 σ=27

*for a more accurate characterization this deposit should be split into an upper, middle and lower zone based on the shear strength, OCR and M of the cohesive soil.

Silt to Sandy Silt behaviour (undrained)

Location	Depth (m)	Mean q_t (MPa)	Mean N_{60} (blows/0.3m)	Mean *M* (MPa)	OCR	S_u (kPa)
CPT-4	16.1 to 23.0	6.6 σ=0.6	15 σ=1	18.2** σ=1.9	7.1 σ=1.0	265 σ=21

**SBTn>6 removed from analysis

Seventeen dissipation tests were carried out within the above noted soils. The results are summarized below.

Location	Depth below grade (m)	C_h (cm^2/min)	Inferred Soil Behaviour from CPT
CPT-11a	11.33	8.18	Silty Clay
	12.09	3.06	Clayey Silt
CPT-4	9.55	10.54	Clayey Silt
	10.57	4.22	Clayey Silt
	11.56	0.26	Clayey Silt
	12.57	0.32	Clayey Silt to Silt
	13.55	4.31	Clayey Silt to Silt
	14.54	0.50	Silt
	15.52	0.0047	Clayey Silt
	16.52	3.83	Clayey Silt to Silt
	17.57	1.28	Silt to Sandy Silt
	18.53	7.03	Silt to Sandy Silt
	19.53	1.92	Silt
	20.54	2.81	Silt to Sandy Silt
	21.53	0.33	Silt
	22.53	0.20	Silt
	23.00	0.11	Silt

To correlate C_h to C_v the table proposed by Jamiolkowski (1985) can be used, or the C_v from consolidation tests can be used to correlate the C_h values, which indicates C_h~>84C_v. The results are considered to be approximate and reasonable to within an order of magnitude.

6.0 REFERENCES

Houlsby, G.T. and Teh, C.I. 1988. Analysis of the piezocone in clay. Proceedings of the International Symposium on Penetration Testing, ISOPT-1, Orlando, pp 777-83, Balkema Pub., Rotterdam.

Jamiolkowski, M. Ladd, C.C., Germaine, J.T. and Lancelotta, R. 1985. New developments in field and laboratory testing of soils. State-of-the-art report. Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, 1, pp.57-153, Balkema Pub., Rotterdam.

Jefferies, M., and Davies, M. Use of CPTu to Estimate Equivalent SPT N60. Geotechnical Testing Journal, December 1993.

Mayne, P.W. 2005. Invited Keynote: “Integrated Ground Behaviour: In-Situ and Lab Tests”. Deformation Characteristics of Geomaterials, Vol. 2 (Proc. IS-Lyon), Taylor & Francis Group, London: 155-177.

Senneset, K. Janbu, N. and Svano, G. Strength and deformation parameters from cone penetration tests. Proceedings of the 2nd European Symposium on Penetration Testing, ESOPT-II, Amersterdam, Balkema Pub., Rotterdam, 1982.

Robertson, P.K. 2009. Interpretation of cone penetration tests – a unified approach. Canadian Geotechnical Journal, Vol. 46, pp 1337-1355.

7.0 LIMITATION OF REPORT

Subsurface and groundwater conditions beyond the CPT locations may differ from those encountered at the CPT locations. The information herein in no way reflects on the environmental aspects of the project.

This report has been prepared for this specific project and the information herein is not applicable to any other project or site location. This report is for use by the client, the Ministry of Transportation (owner) and the owner's geotechnical consultant. Any use of this report by another third party, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. DownUnder Geotechnical does not take any responsibility for the use of the soil parameters summarized in this report unless consulted during geotechnical design.

Report prepared by:

Andrew Drevininkas, P. Eng.
President

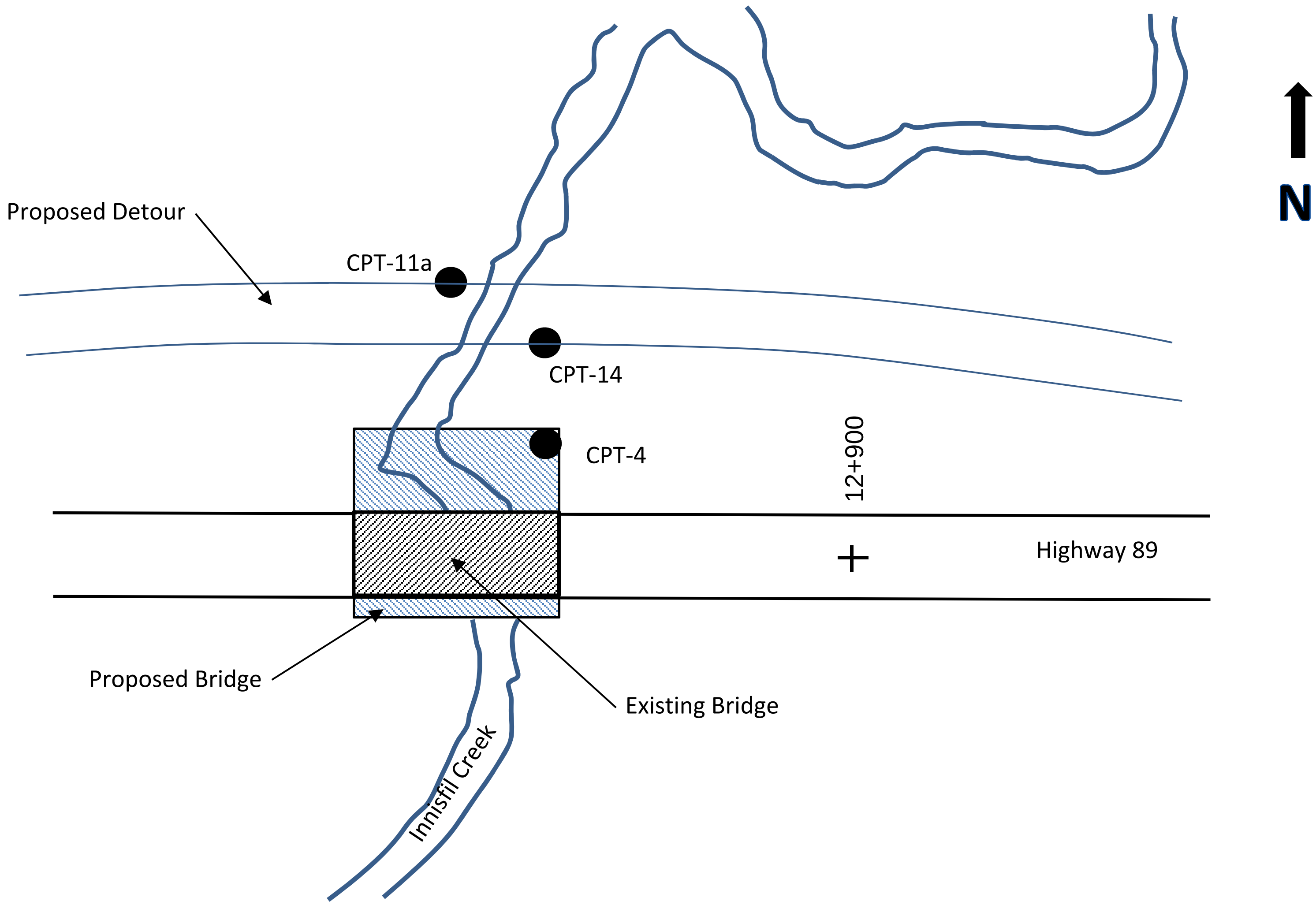


Figure No.1
CPT Location Plan

DownUnder Geotechnical Limited

APPENDIX A – Calibration Certificate

Geoprobe Systems
THE WORLD LEADER IN DIRECT PUSH TECHNOLOGY

CERTIFICATE FOR CPT PROBE 4143

PROBE NUMBER	4143 (Groundtech)
DATE OF CALIBRATION	May 20, 2013
CALIBRATED BY	Sean Bigler Geoprobe® Systems

POINT RESISTANCE

Sensor Range	100.00 MPa
Scaling Factor	**902**
Net Area Factor	0.57

LOCAL FRICTION

Sensor Range	0.50 MPa
Scaling Factor	**6135**
Net Area Factor	0.017

PORE PRESSURE

Sensor Range	2.50 MPa
Scaling Factor	**2522**

TILT ANGLE

Range	0-40 deg.

CALIBRATION EQUIPMENT

Sensotec® Precision Load Cell Model 73/2537-11-02 — Calibrated June 27, 2012
Serial No. 804409
Calibration at 0.0, 3000, 6000, 9000, 12000, 15000, 18000, 21000, 24000, 27000, 30000, 27000, 24000, 21000, 18000, 15000, 12000, 9000, 6000, 3000, 0.0 lbs

Sensotec® Pressure Transducer Model A-10/6076-08 — Calibrated June 27, 2012
Serial No. 544931
Calibration at 0.0, 30, 60, 90, 120, 150, 180, 210, 240, 270, 300, 270, 240, 210, 180, 150, 120, 90, 60, 30, 0.0 psi

Documentation of NIST Traceability available upon request.

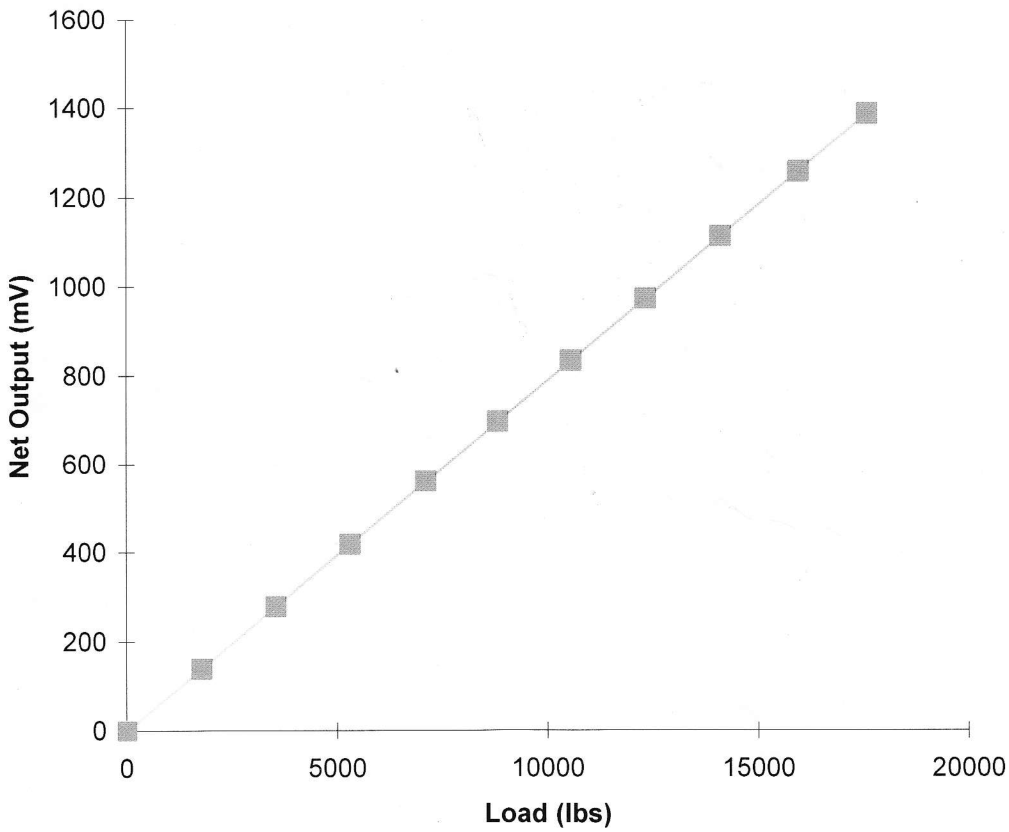
Cone penetration test probe calibration results are accurate at the time of calibration. Geoprobe® Systems does not guarantee probe accuracy at the time of field testing. ISSMFE international reference test procedure for cone penetration testing recommends probe calibration at least every 3 months.

250 Beanville Road
Randolph, Vermont 05060
phone: (800)639-6315 fax: (802)728-9871

Cone Penetrometer Calibration

Digital Cone Tip

Cone Serial No.:	4644.103	Date:	27-Oct-11
Rated Range:	22000 lbs		
Load Reference:	Ref LC-SN: 322089A	Calibrated By:	WJC
Ref. DVM:	MY47026116		
Ref. Excitation:	5.049 V_{dc}	Approved By:	



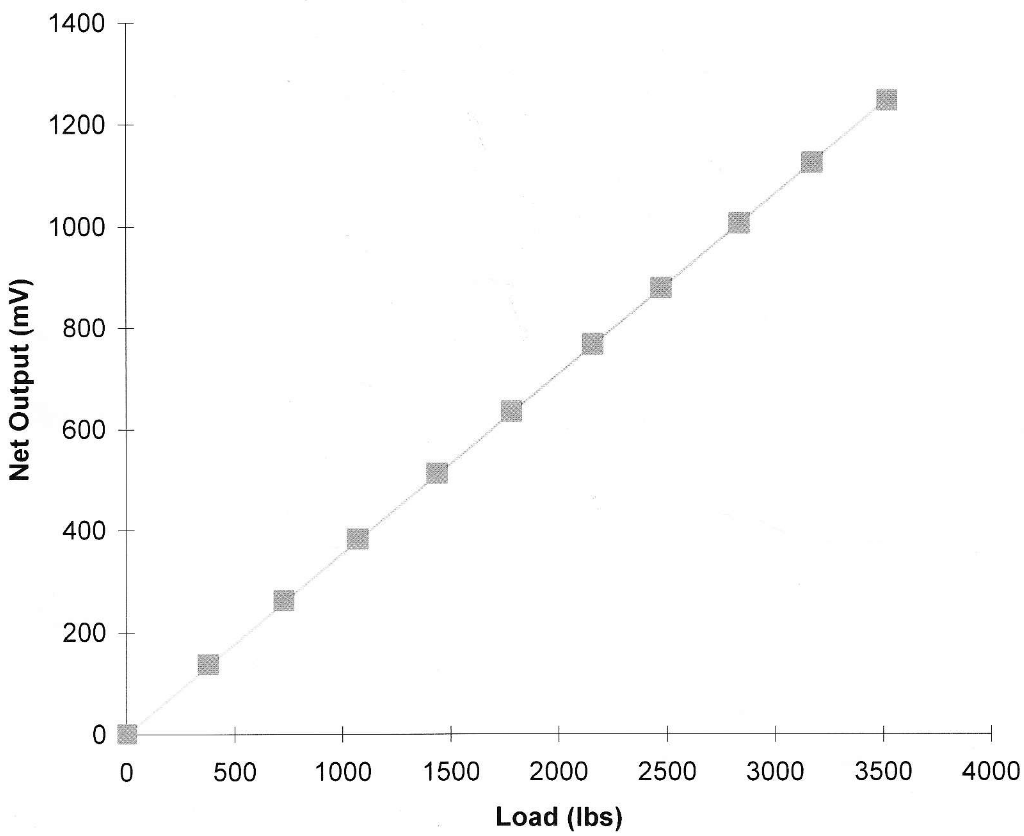
Cal Factor:	**78.781E-3 mV/lbs**	*75.000E-3 nominal*
R^2:	1.00000	
Nonlinearity:	0.05	
Zero Load Output:	250.641E-3 V	

produced with Cone_Cal v 2003.02.04

VERTEK
250 Beanville Road
Randolph, Vermont 05060
phone: (800)639-6315 fax: (802)728-9871

Cone Penetrometer Calibration
Digital Cone Sleeve

Cone Serial No.:	4644.103	Date:	27-Oct-11
Rated Range:	4400 lbs		
Load Reference:	Ref LC-SN: 322089A	Calibrated By:	WJC
Ref. DVM:	MY47026116		
Ref. Excitation:	5.049 V_{dc}	Approved By:	



Cal Factor:	**353.139E-3 mV/lbs**	*350.000E-3 nominal*
R^2:	0.99999	
Nonlinearity:	0.34	
Zero Load Output:	295.359E-3 V	

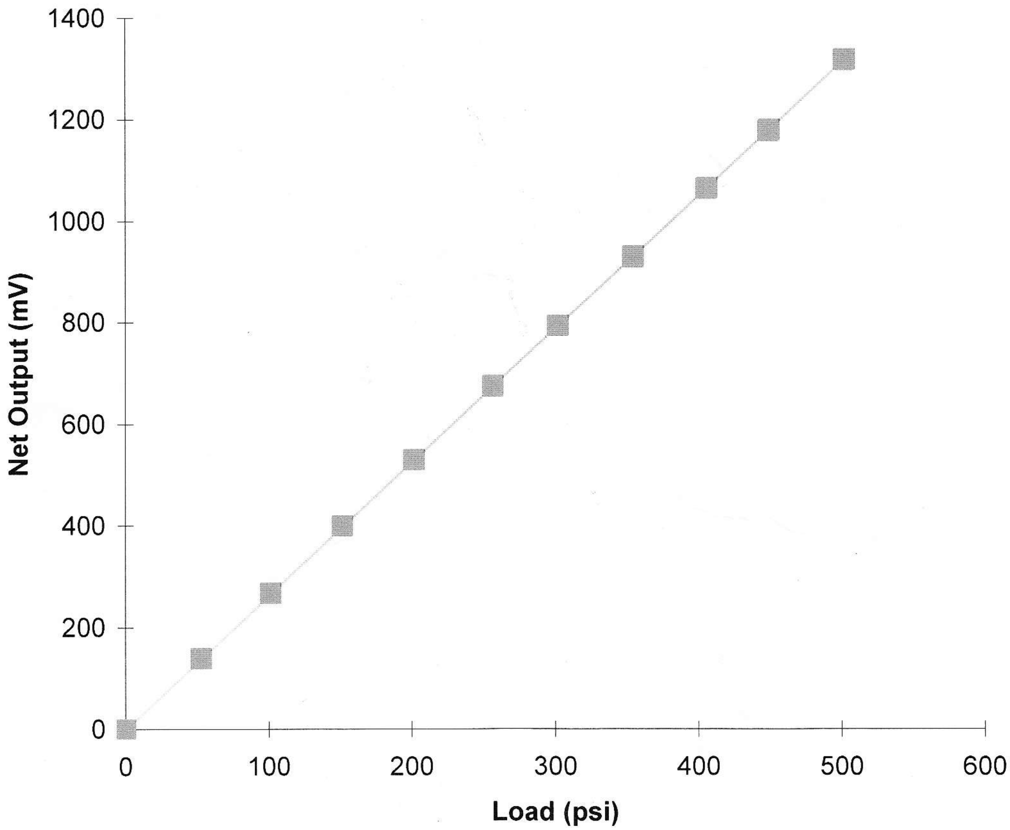
produced with Cone_Cal v 2003.02.04

250 Beanville Road
Randolph, Vermont 05060
phone: (800)639-6315 fax: (802)728-9871

Cone Penetrometer Calibration

Digital Cone Pore Pressure

Cone Serial No.:	4644.103	Date:	27-Oct-11
Rated Range:	2000 psi		
Load Reference:	Ref PT-SN:0937-016VMC	Calibrated By:	WJC
Ref. DVM:	MY47026116		
Ref. Excitation:	5.049 V_{dc}	Approved By:	

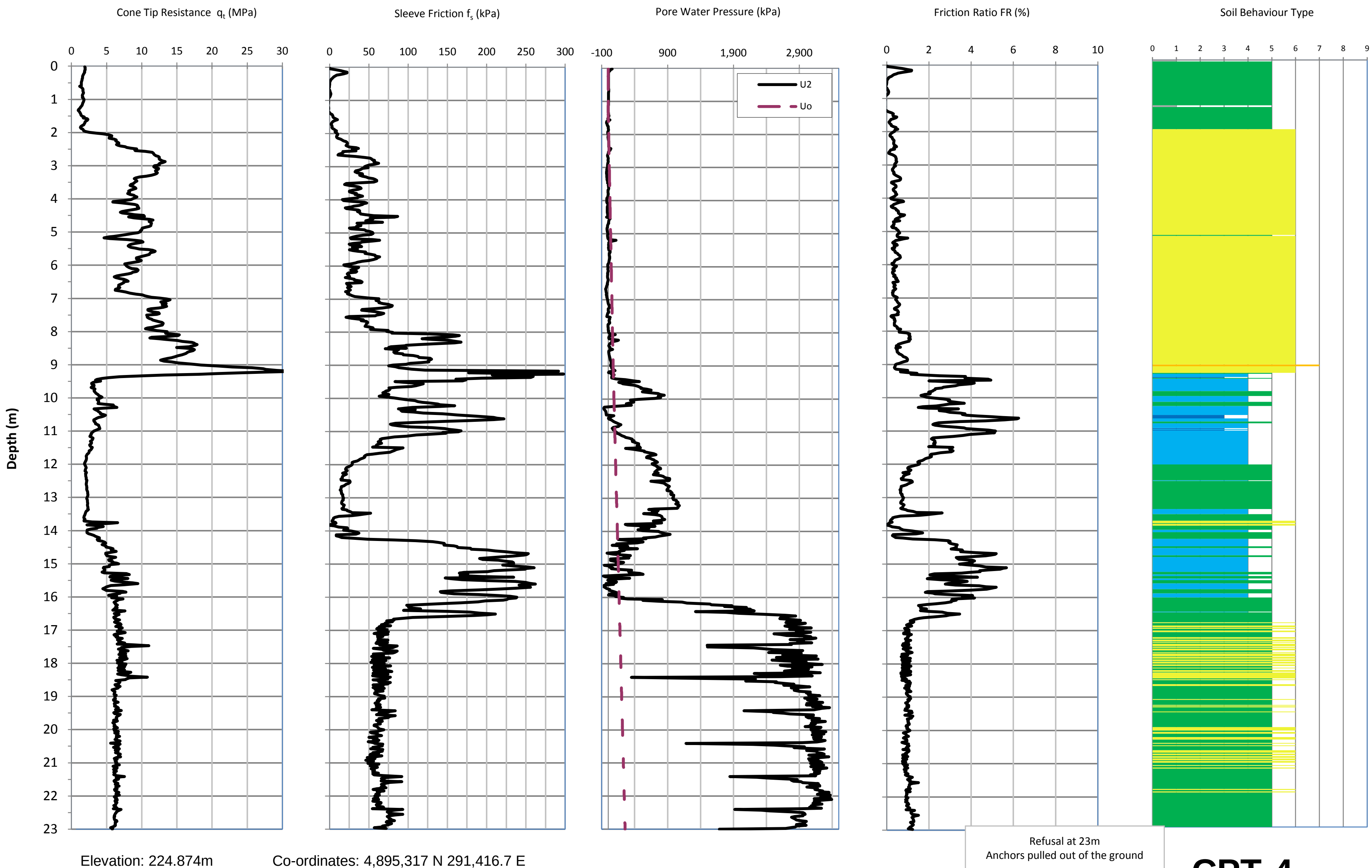


Cal Factor:	**2.621E+0 mV/psi**	*2.500E+0 nominal*
R^2:	1.00000	
Nonlinearity:	0.11	
Zero Load Output:	212.940E-3 V	

produced with Cone_Cal v 2003.02.04

APPENDIX B – Piezocone Soundings

PiezoCone Penetration Test



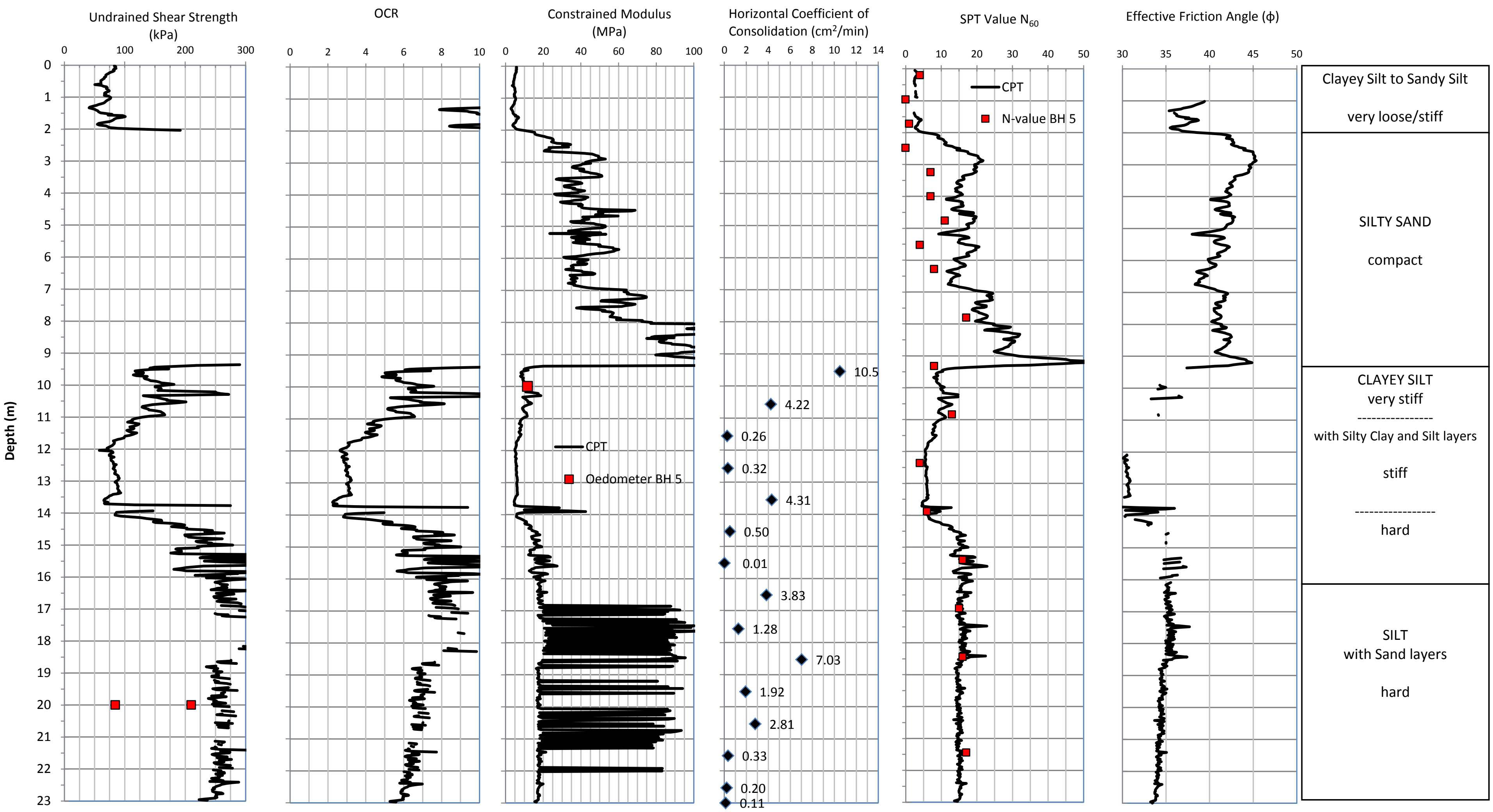
Elevation: 224.874m Co-ordinates: 4,895,317 N 291,416.7 E
Date:September 5, 2013
Location: Highway 89 and Innisfil Creek, Ontario
Engineer: A. Drevininkas
Cone: VERTEK 10 tonne
Tip Area: 10 cm^2
Friction Sleeve Area: 150 cm^2
Filter Location: U_2

CPT-4

CPT Probe 4644.103

DownUnder Geotechnical Limited

PiezoCone Penetration Test



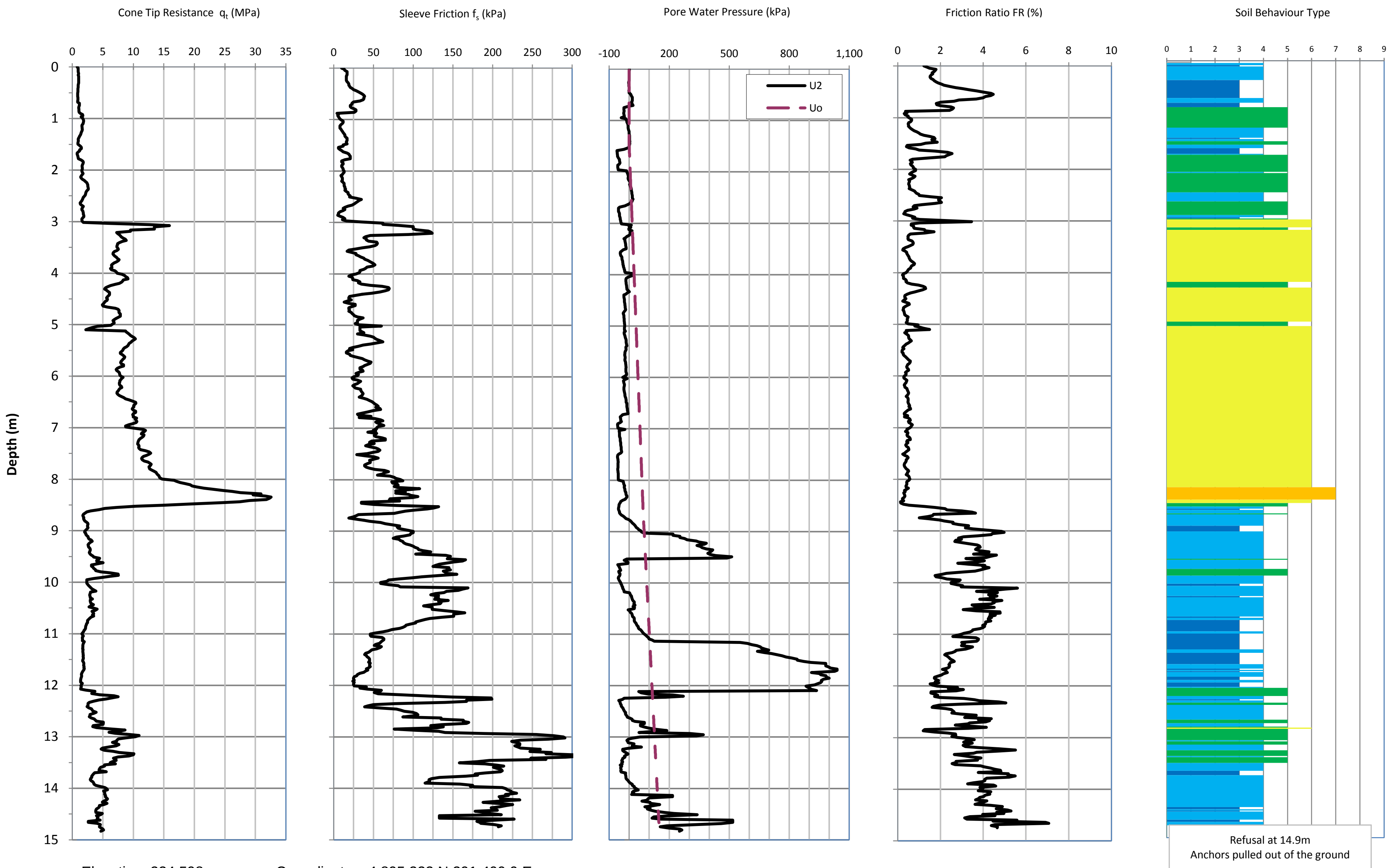
Elevation: 224.874m Co-ordinates: 4,895,317 N 291,416.7 E
Date:September 5, 2013
Location: Highway 89 and Innisfil Creek, Ontario
Engineer: A. Drevininkas
Cone: VERTEK 10 tonne
Tip Area: 10 cm^2
Friction Sleeve Area: 150 cm^2
Filter Location: U_2

CPT-4

CPT Probe 4644.103

DownUnder Geotechnical Limited

PiezoCone Penetration Test



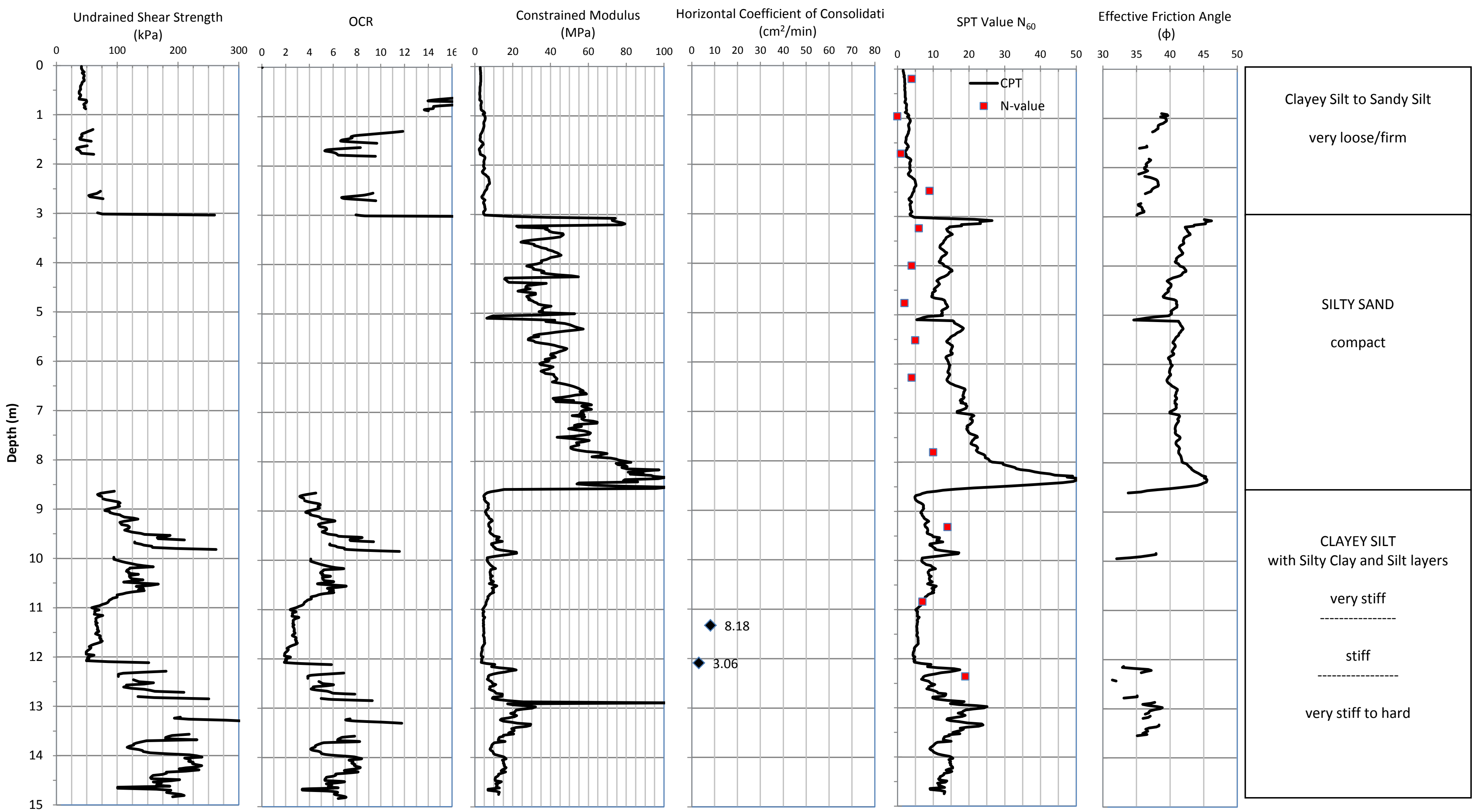
Elevation: 224.598m Co-ordinates: 4,895,322 N 291,400.9 E
Date:September 4, 2013
Location: Highway 89 and Innisfil Creek, Ontario
Engineer: A. Drevininkas
Cone: GEOTECH AB 10 tonne
Tip Area: 10 cm^2
Friction Sleeve Area: 150 cm^2
Filter Location: U_2

CPT-11A

CPT Probe 4143

DownUnder Geotechnical Limited

PiezoCone Penetration Test



Elevation: 224.598m Co-ordinates: 4,895,322 N 291,400.9 E
Date:September 4, 2013
Location: Highway 89 and Innisfil Creek, Ontario
Engineer: A. Drevininkas
Cone: GEOTECH AB 10 tonne
Tip Area: 10 cm^2
Friction Sleeve Area: 150 cm^2
Filter Location: U_2

CPT-11A

CPT Probe 4143

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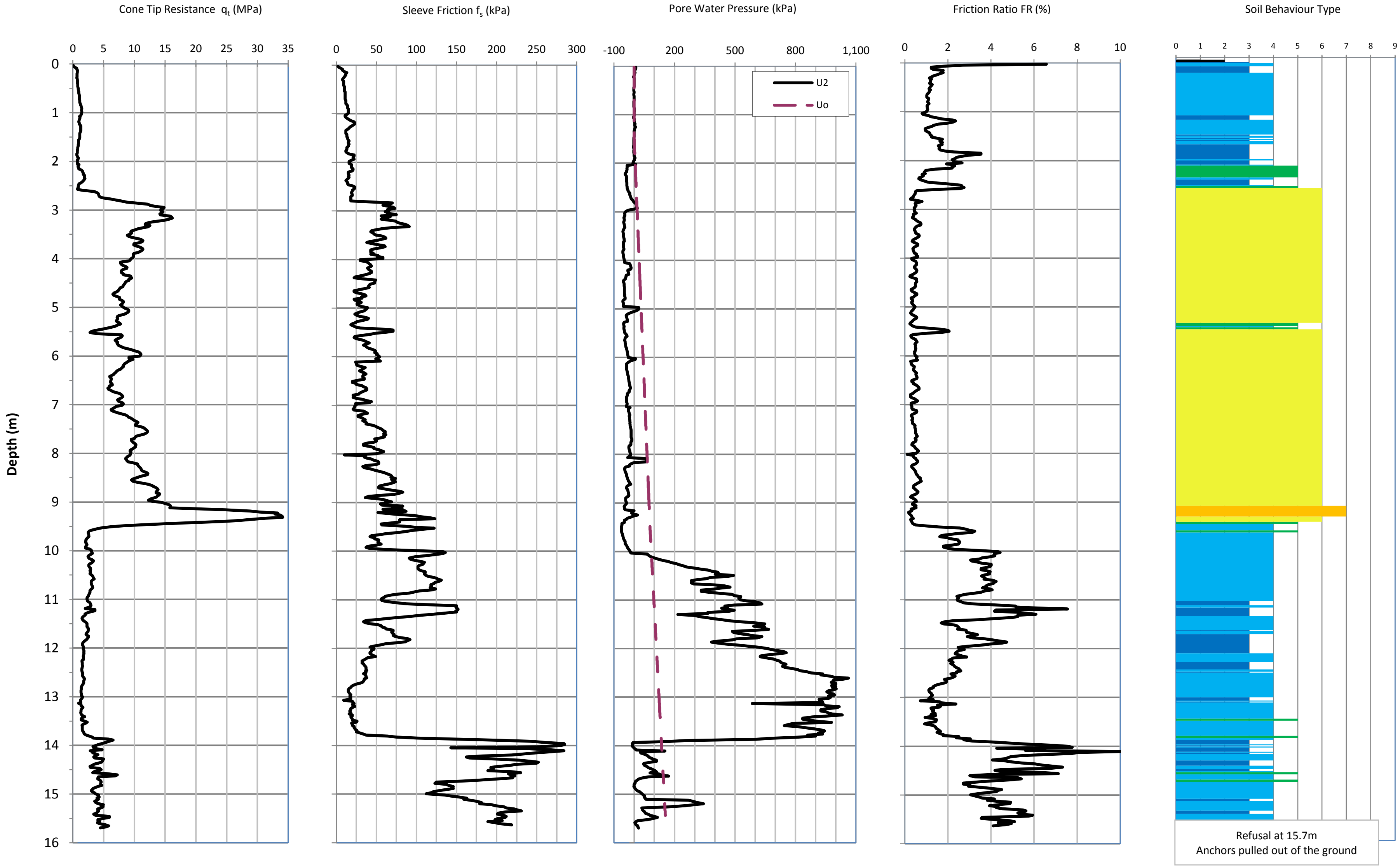
PiezoCone Penetration Test



CPT-14

CPT Probe 4143

Elevation: 224.549m Co-ordinates: 4,895,307N 291,420.4E
Date:September 4, 2013
Location: Highway 89 and Innisfil Creek, Ontario
Engineer: A. Drevininkas
Cone: GEOTECH AB 10 tonne
Tip Area: 10 cm^2
Friction Sleeve Area: 150 cm^2
Filter Location: U_2

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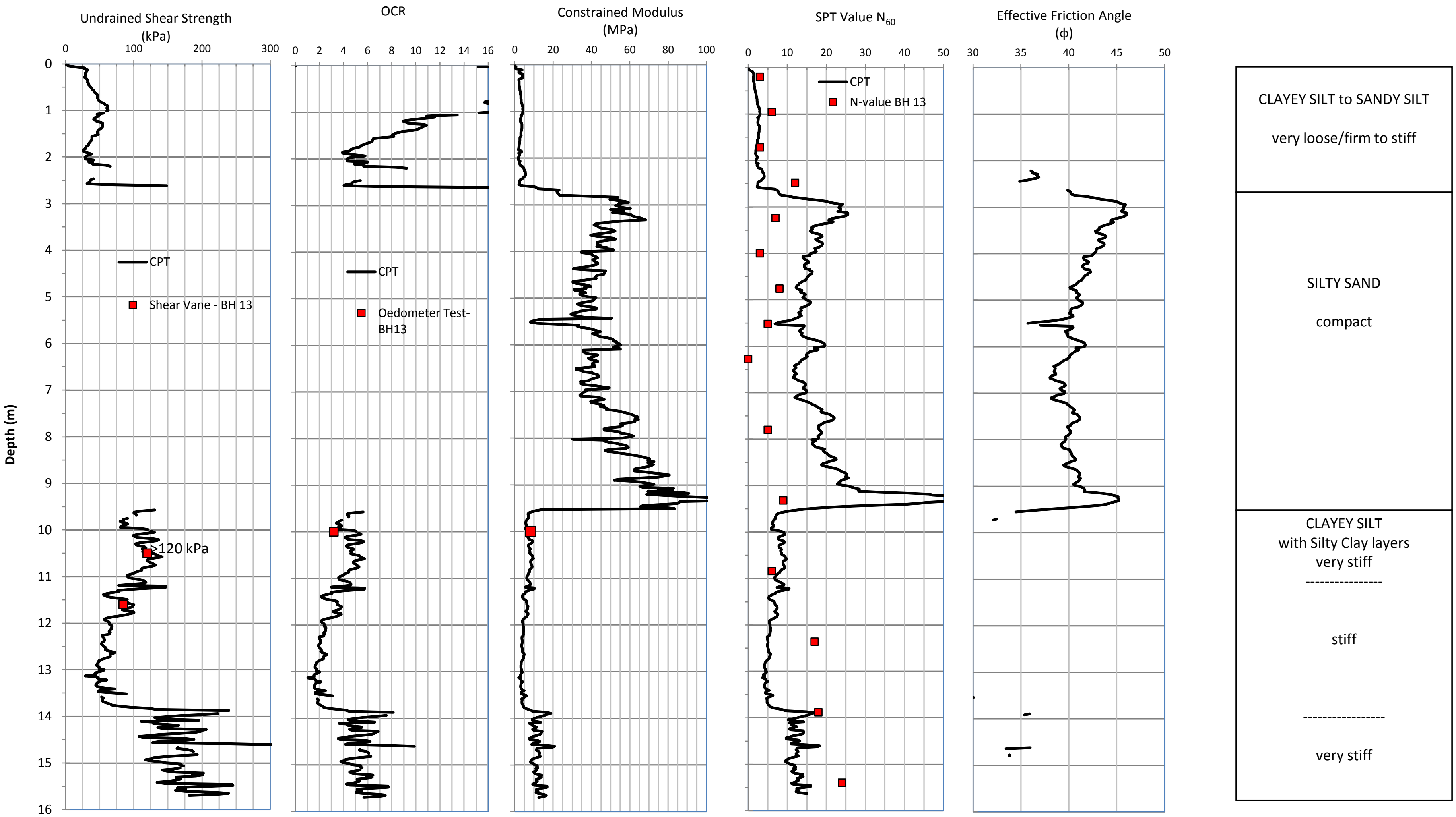
PiezoCone Penetration Test



Elevation: 224.549m Co-ordinates: 4,895,307N 291,420.4E
Date:September 4, 2013
Location: Highway 89 and Innisfil Creek, Ontario
Engineer: A. Drevininkas
Cone: GEOTECH AB 10 tonne
Tip Area: 10 cm^2
Friction Sleeve Area: 150 cm^2
Filter Location: U_2

CPT-14

CPT Probe 4143

DownUnder Geotechnical Limited

APPENDIX C – Dissipation Test Results

Summary of Dissipation Test Results

CPT	Depth (m)	u_i (kPa)	u_0 (kPa)	t_{50} (min)	I_r (kPa)	C_v (cm^2/min)	Response
11a	11.3	1280	105	0.43	20	8.18	Dilatory
	12.1	912	115	1.15	20	3.06	Monotonic
4	9.6	1450	80	0.33	20	10.54	Dilatory
	10.6	580	93	0.83	20	4.22	Dilatory
	11.6	487	106	13.42	20	0.26	Monotonic
	12.6	882	119	10.92	20	0.32	Monotonic
	13.6	1270	132	0.82	20	4.31	Dilatory
	14.5	1225	145	7.08	20	0.50	Dilatory
	15.5	650	157	750*	20	0.0047	Dilatory
	16.5	2216	170	0.92	20	3.83	Dilatory
	17.6	2956	183	2.75	20	1.28	Monotonic
	18.5	2241	196	0.50	20	7.03	Monotonic
	19.5	3030	209	1.83	20	1.92	Monotonic
	20.5	2897	222	1.25	20	2.81	Monotonic
	21.5	3026	235	10.5	20	0.33	Monotonic
	22.5	2862	248	17.5	20	0.20	Monotonic
	23.0	2609	254	31.67*	20	0.11	Monotonic

*extrapolated value

u_i = initial measured excess pore pressure for Monotonic response
= extrapolated maximum excess pore pressure for Dilatory response (as per Houlsby and Teh)

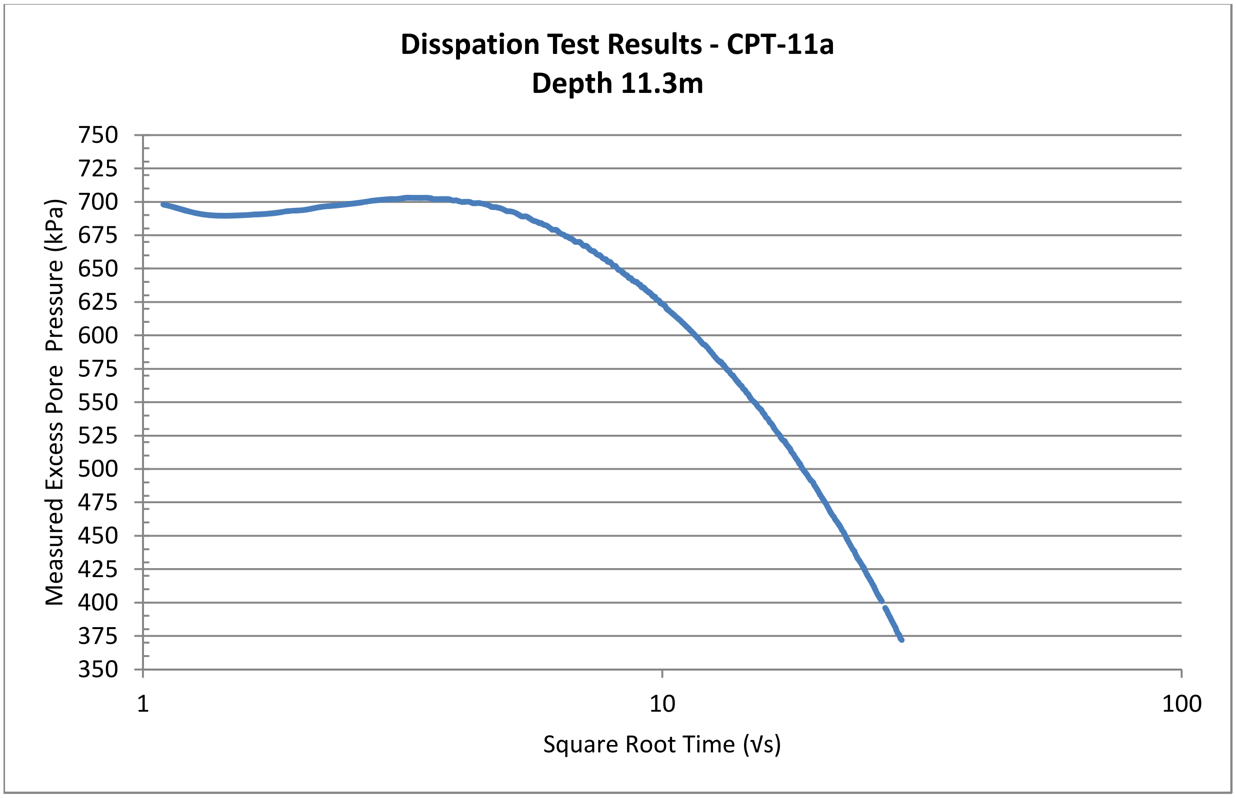
u_0 = pore water pressure at rest
Assumed hydrostatic within Fill and Silty Sand/Sand
Assumed to be 30% higher than hydrostatic in Silty Clay/Clayey Silt and Silt/Sandy Silt due to artesian pressures in lower sands encountered in the adjacent boreholes

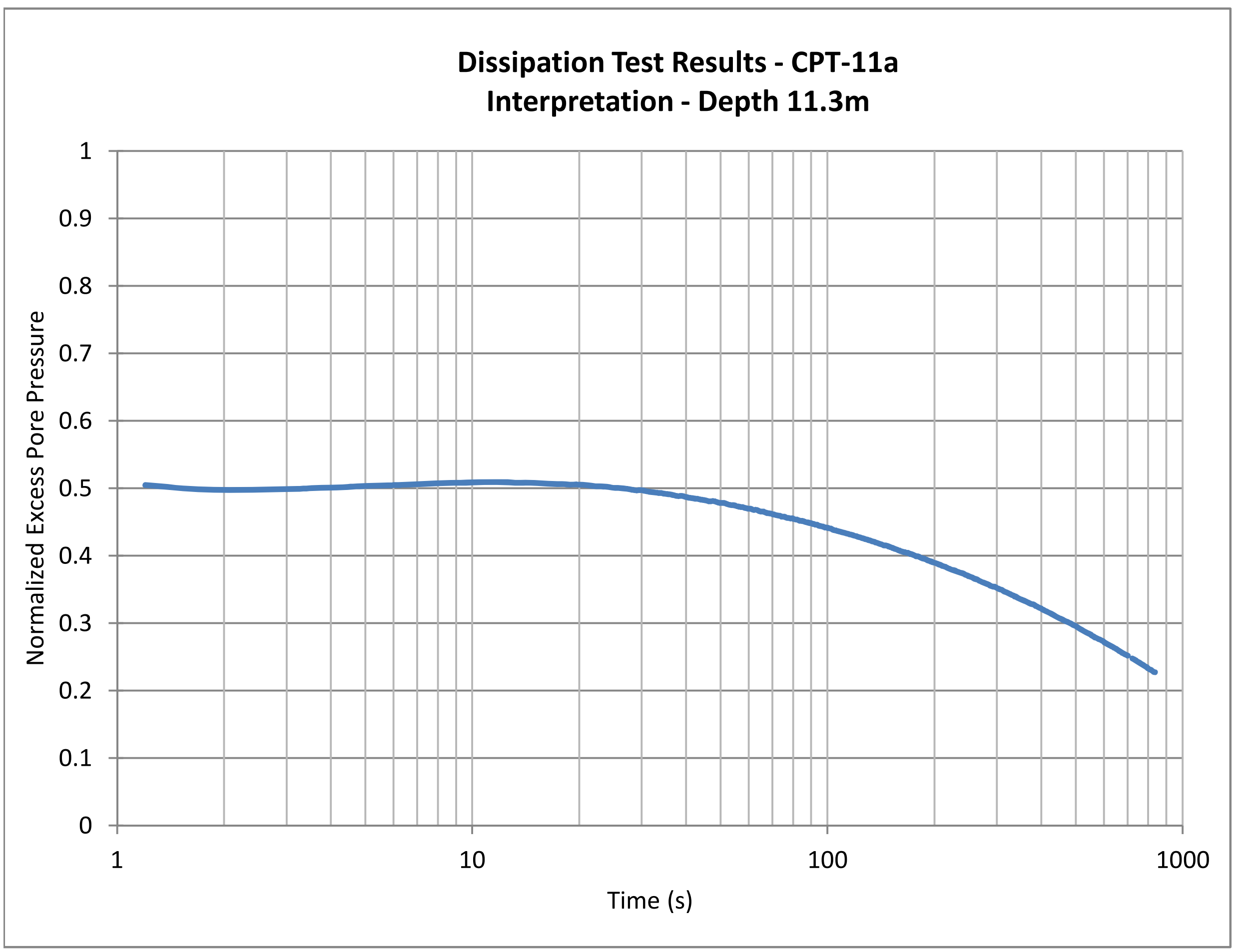
t_{50} = time for 50% excess pore water pressure dissipation

I_r = Undrained Rigidity Index = Shear Modulus/Undrained Shear Strength

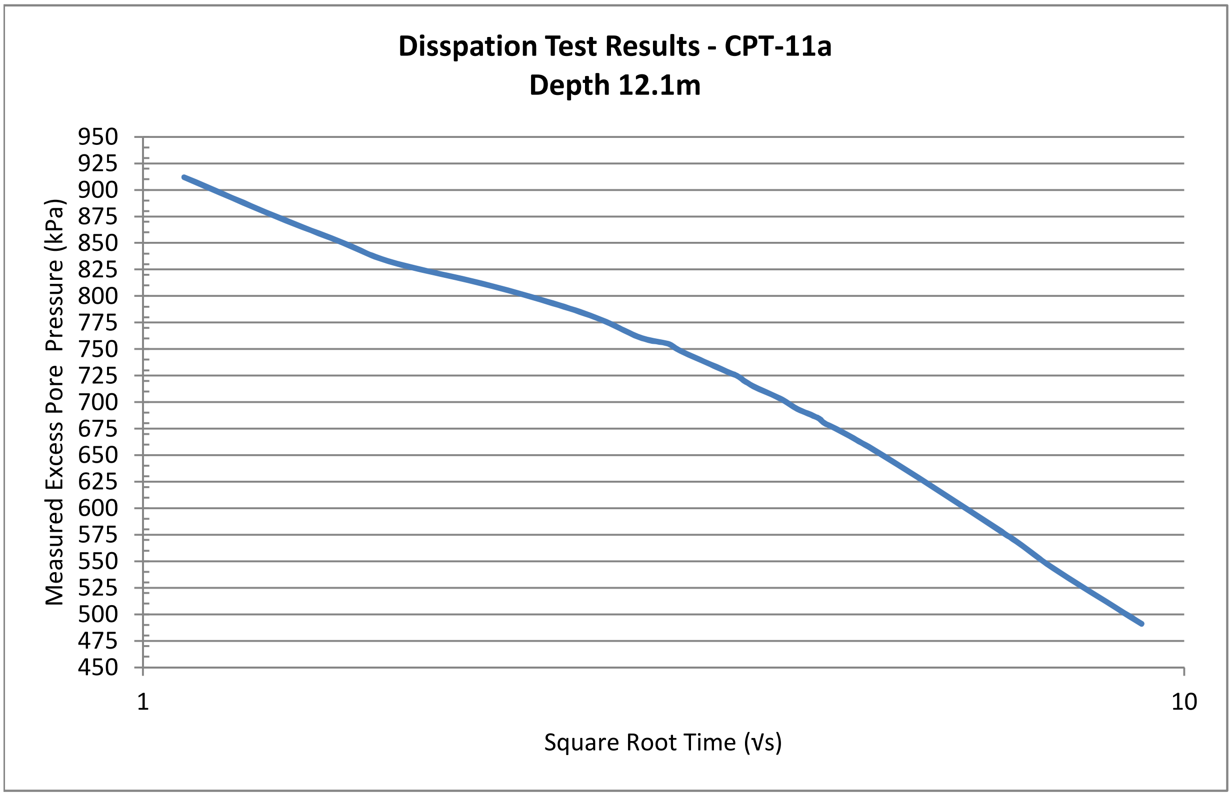
Inferred shear modulus (G) = E/3

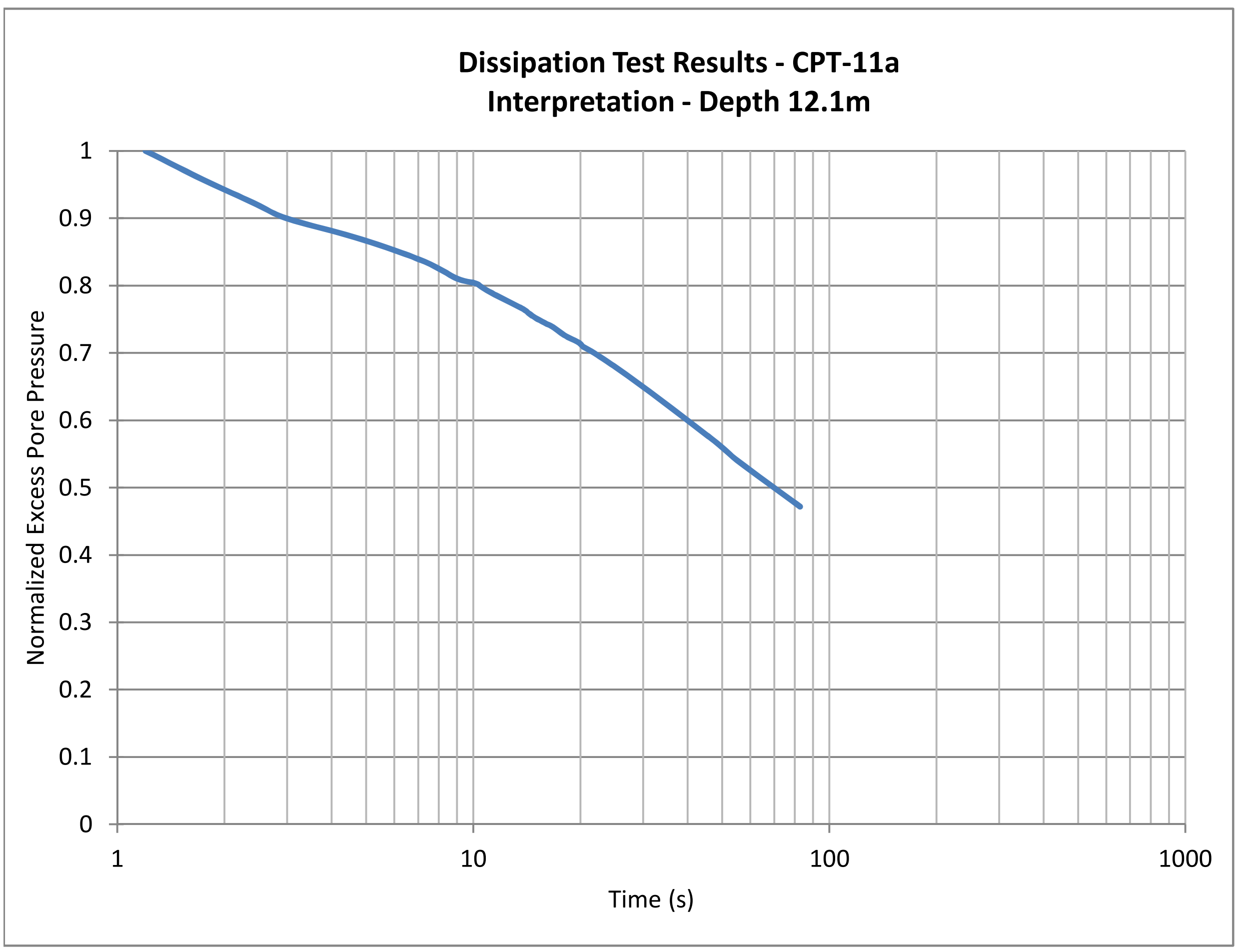
Assumed Bulk Unit Weight (ɣ)
Fill ~ 18 kN/m^3
Silty Sand/Sand ~ 20 kN/m^3
Silty Clay/Clayey Silt ~ 19.5 kN/m^3
Silt/Sandy Silt ~ 19.5 kN/m^3

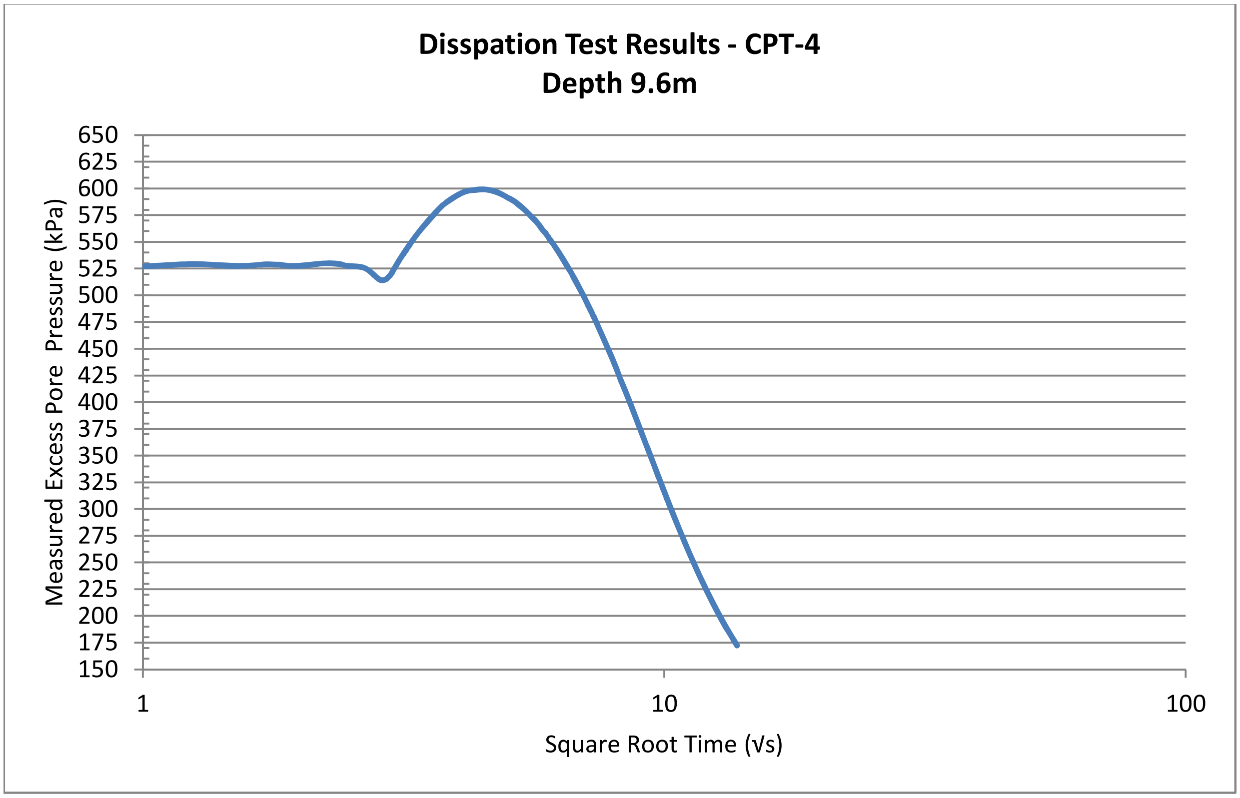


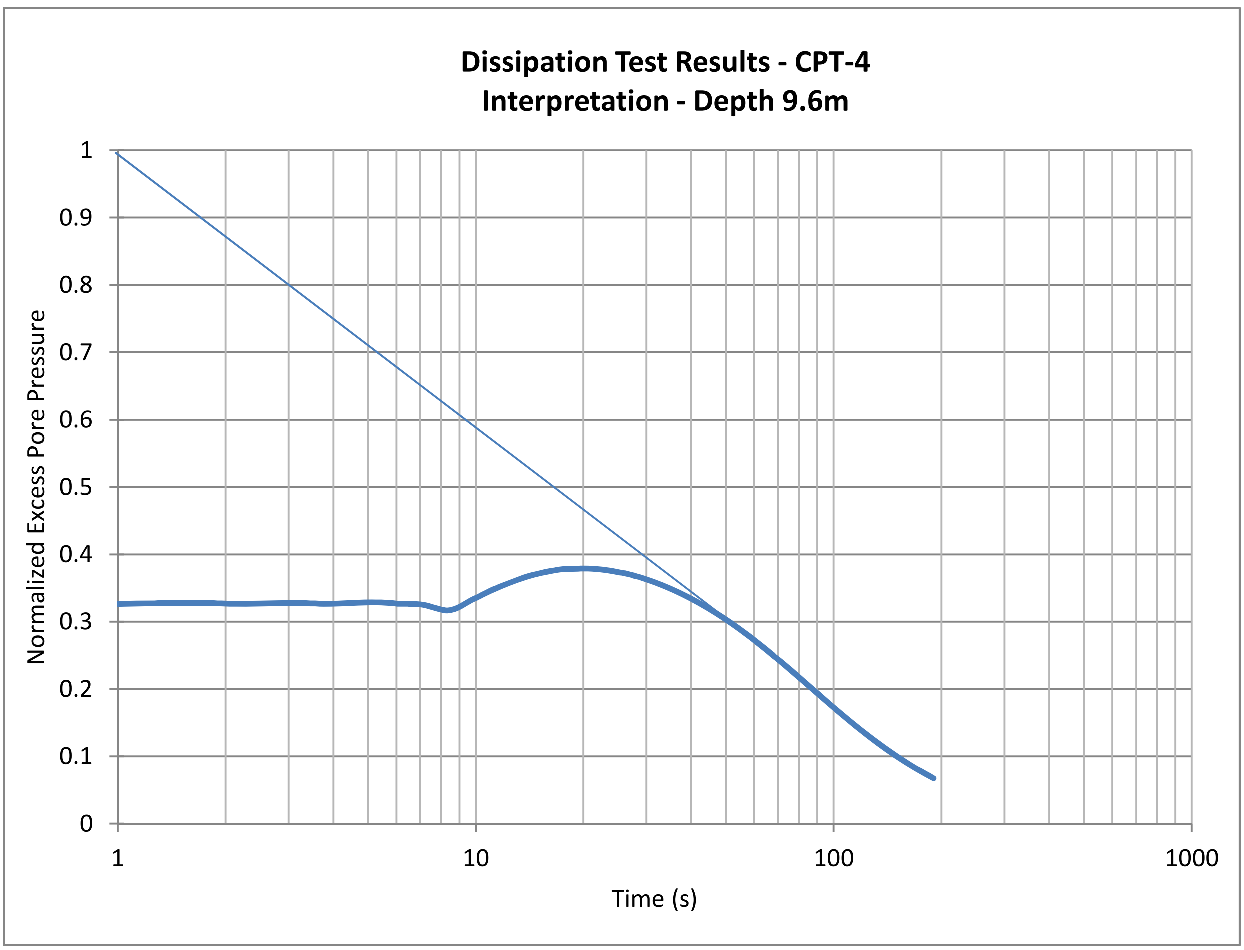


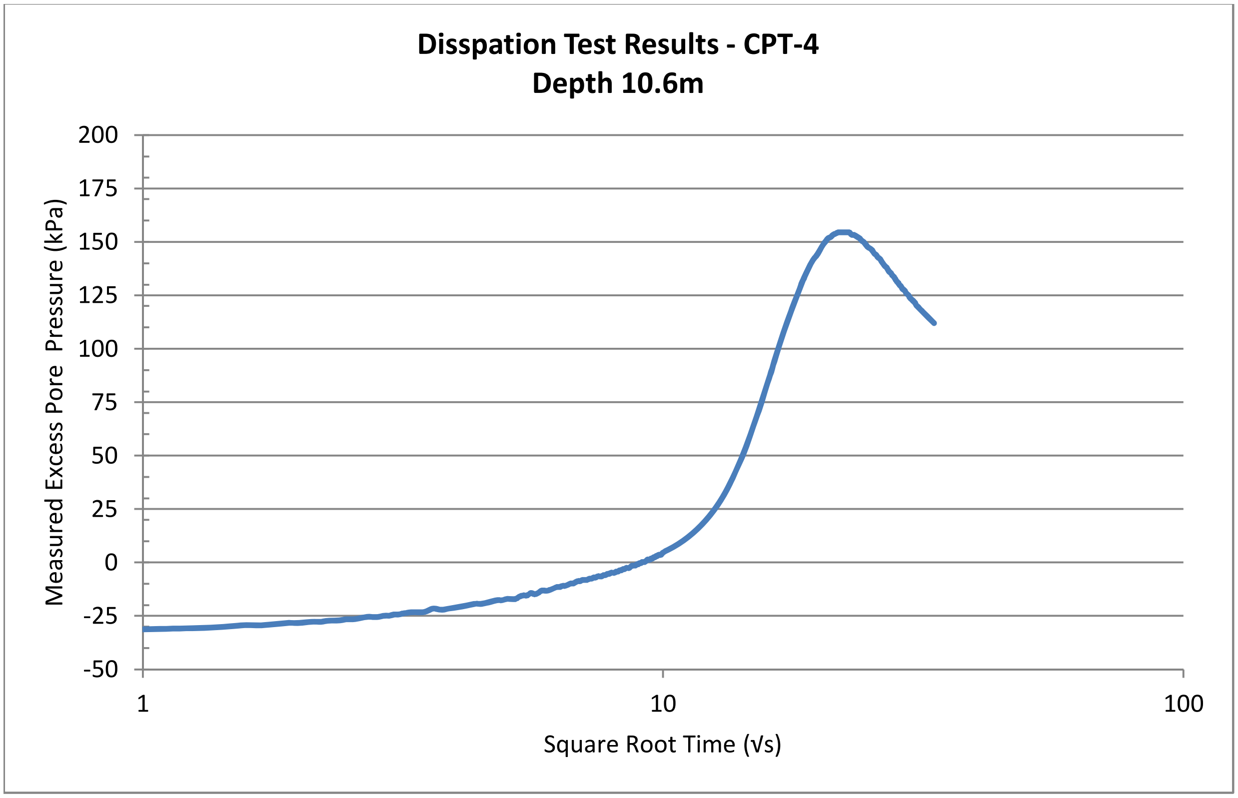
DownUnder Geotechnical Limited

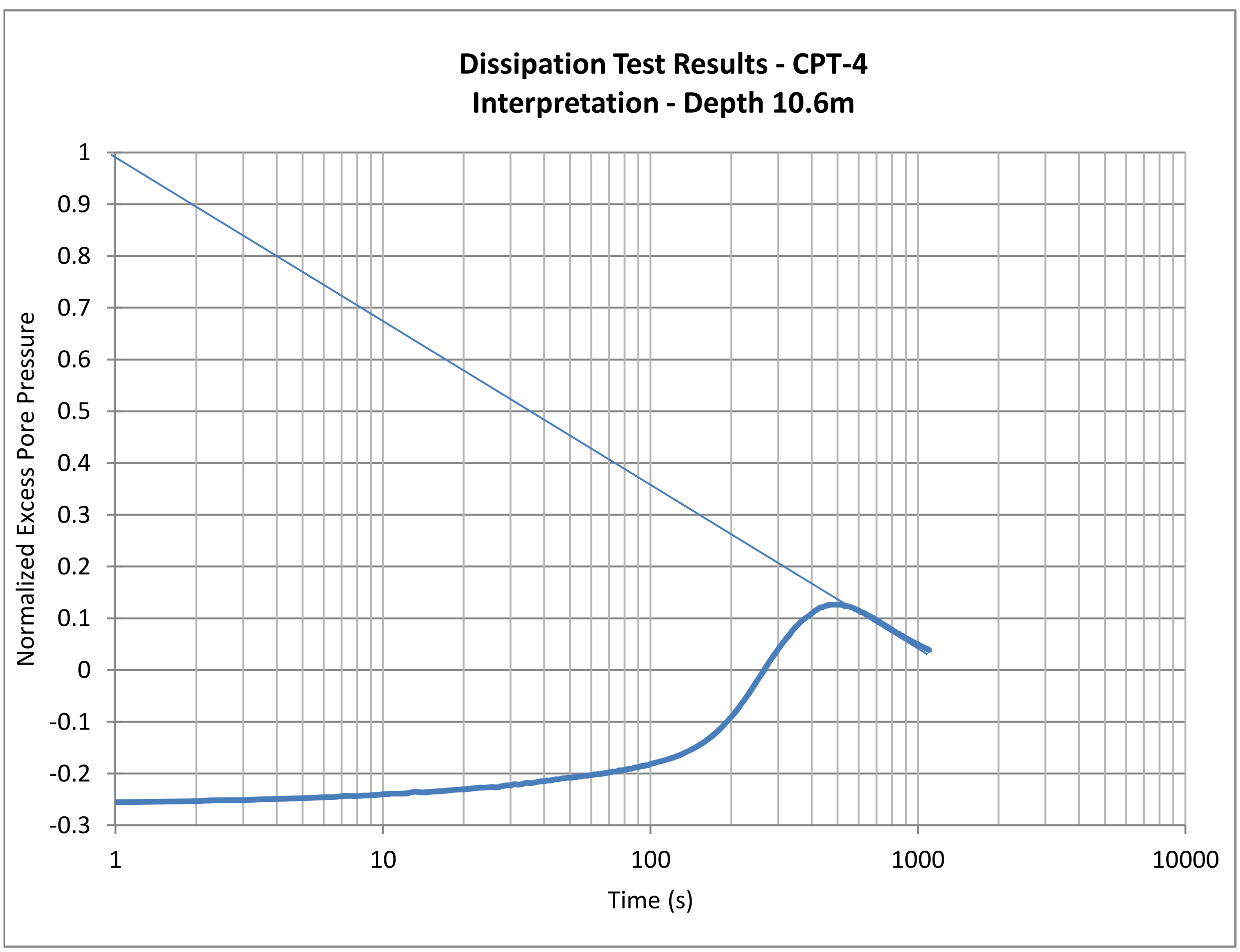

Disspation Test Results - CPT-11a
Depth 12.1m
Measured Excess Pore Pressure (kPa)
950
925
900
875
850
825
800
775
750
725
700
675
650
625
600
575
550
525
500
475
450
1
10
Square Root Time (√s)

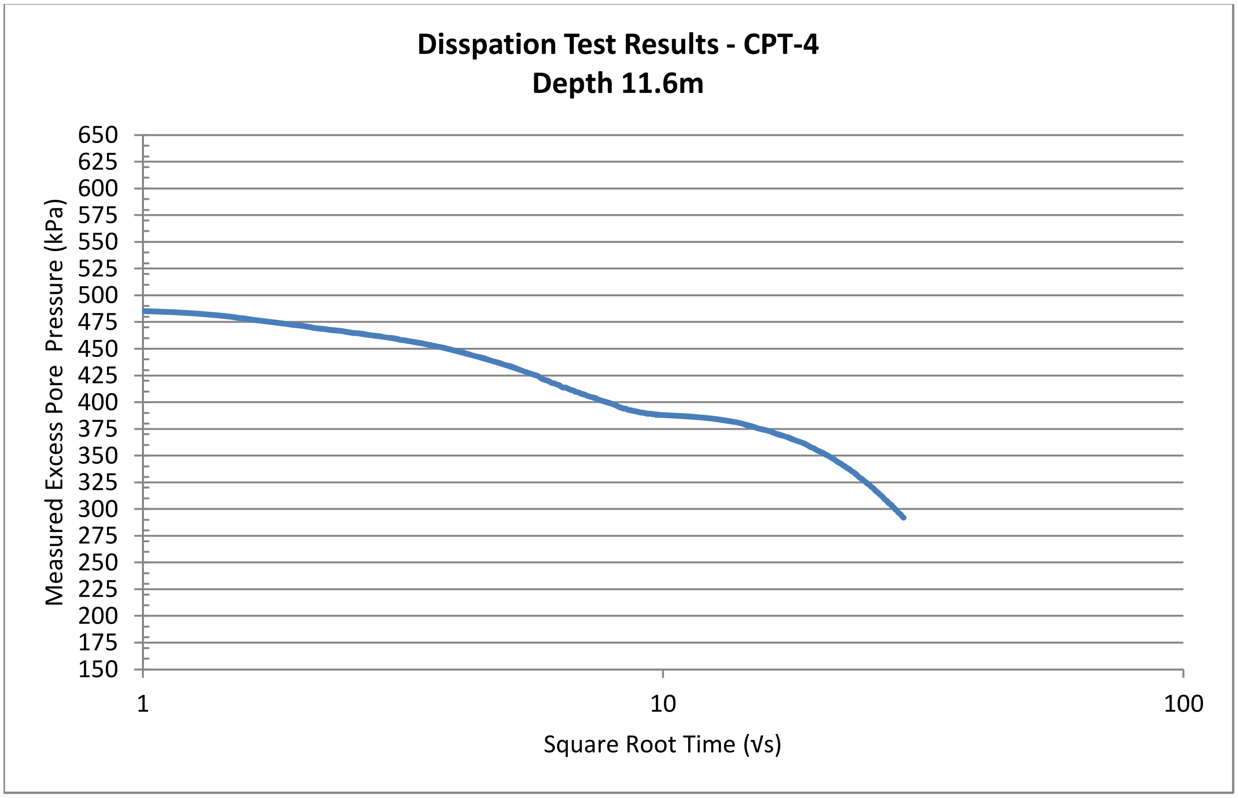

Dissipation Test Results - CPT-11a
Interpretation - Depth 12.1m
Normalized Excess Pore Pressure
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1
10
100
1000
Time (s)

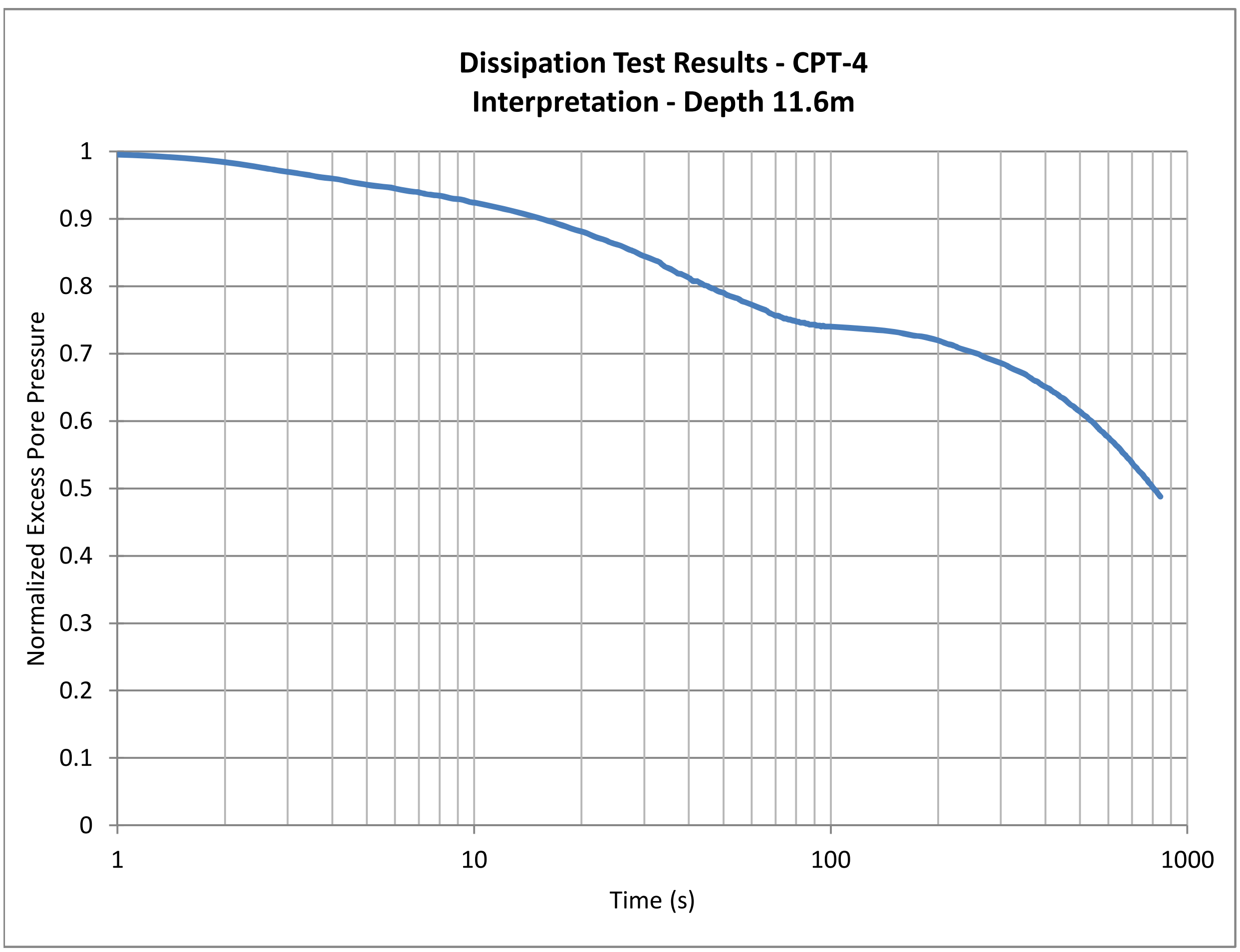

Disspation Test Results - CPT-4
Depth 9.6m
Measured Excess Pore Pressure (kPa)
650
625
600
575
550
525
500
475
450
425
400
375
350
325
300
275
250
225
200
175
150
1
10
100
Square Root Time (√s)

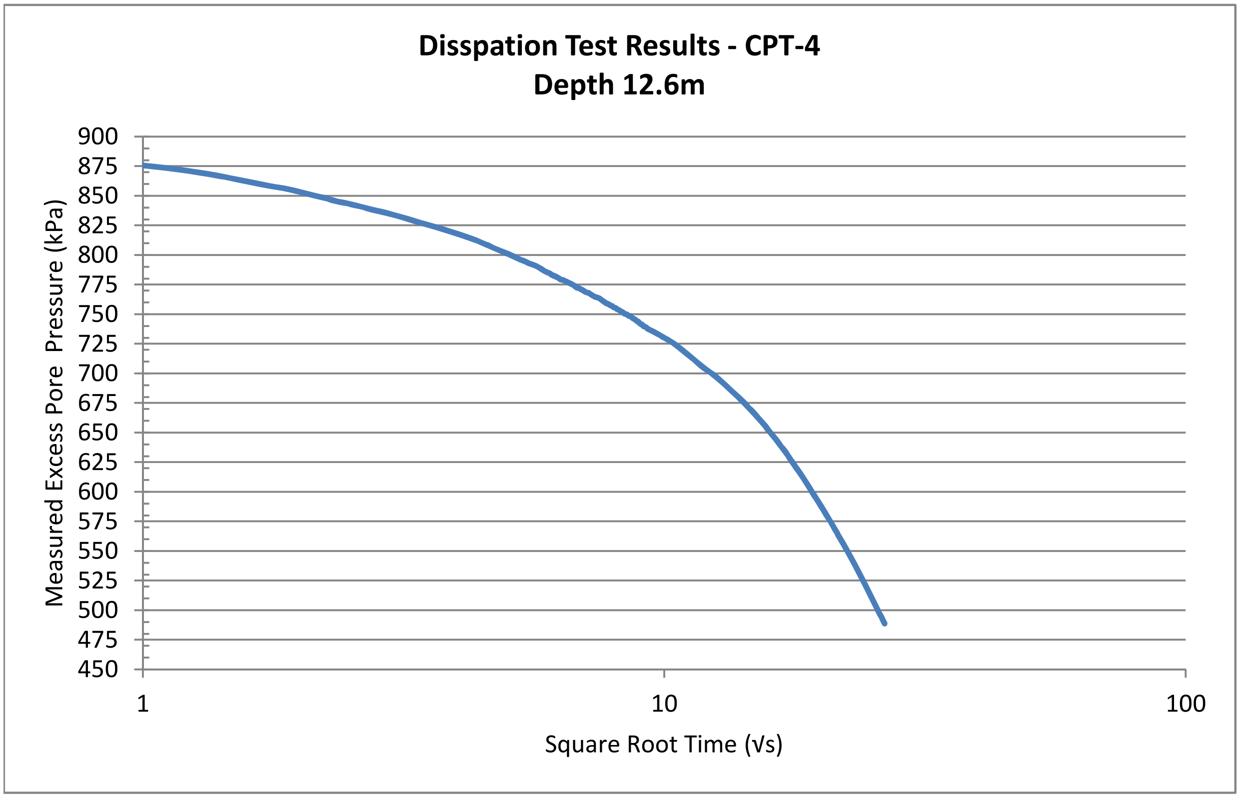

Dissipation Test Results - CPT-4
Interpretation - Depth 9.6m
Normalized Excess Pore Pressure
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1
10
100
1000
Time (s)

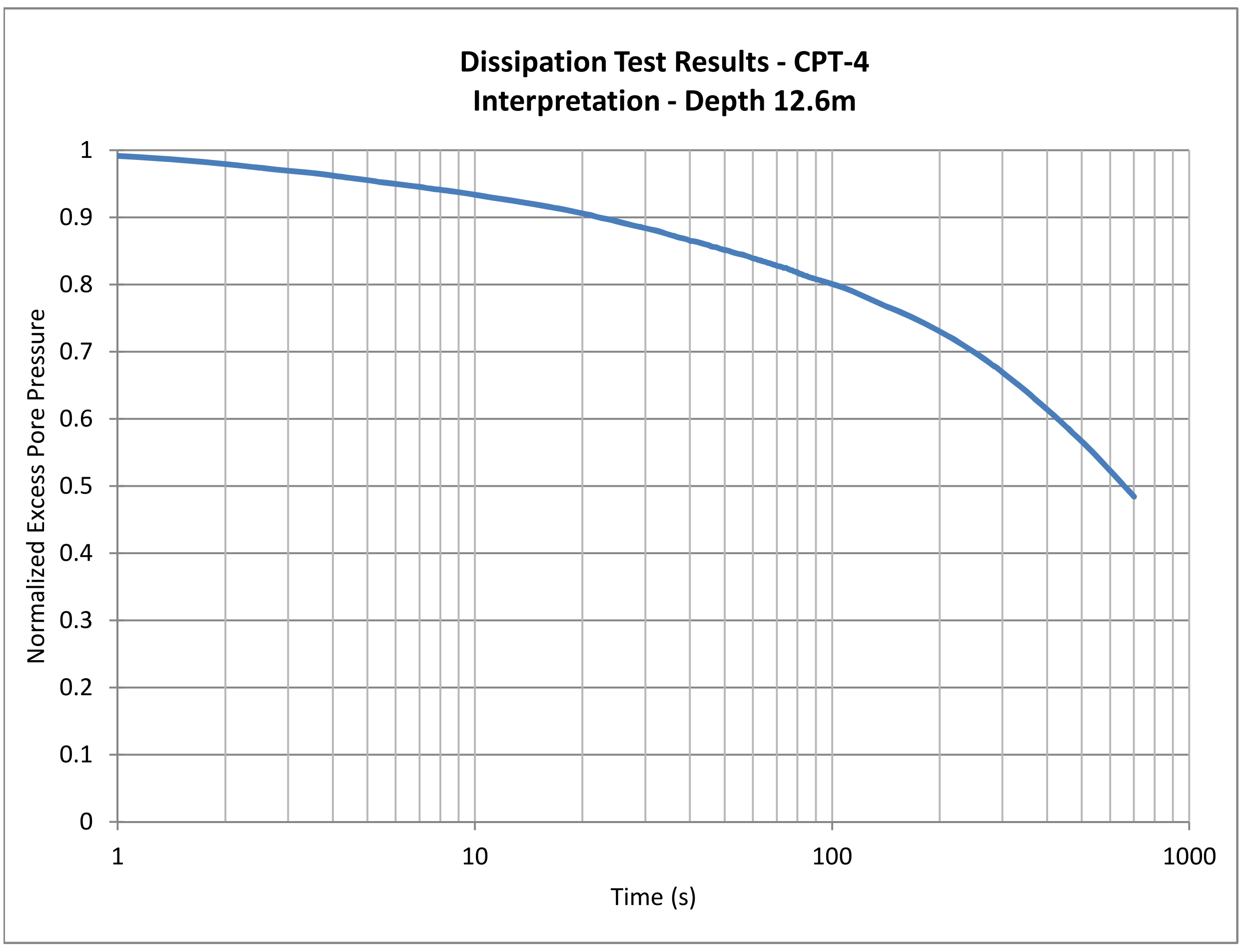

Disspation Test Results - CPT-4
Depth 10.6m
200
175
150
125
100
75
50
25
0
-25
-50
Measured Excess Pore Pressure (kPa)
1
10
100
Square Root Time (√s)


Dissipation Test Results - CPT-4
Interpretation - Depth 10.6m
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
Normalized Excess Pore Pressure
1
10
100
1000
10000
Time (s)


Disspation Test Results - CPT-4
Depth 11.6m
Measured Excess Pore Pressure (kPa)
650
625
600
575
550
525
500
475
450
425
400
375
350
325
300
275
250
225
200
175
150
1
10
100
Square Root Time (√s)


Dissipation Test Results - CPT-4
Interpretation - Depth 11.6m
Normalized Excess Pore Pressure
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1
10
100
1000
Time (s)





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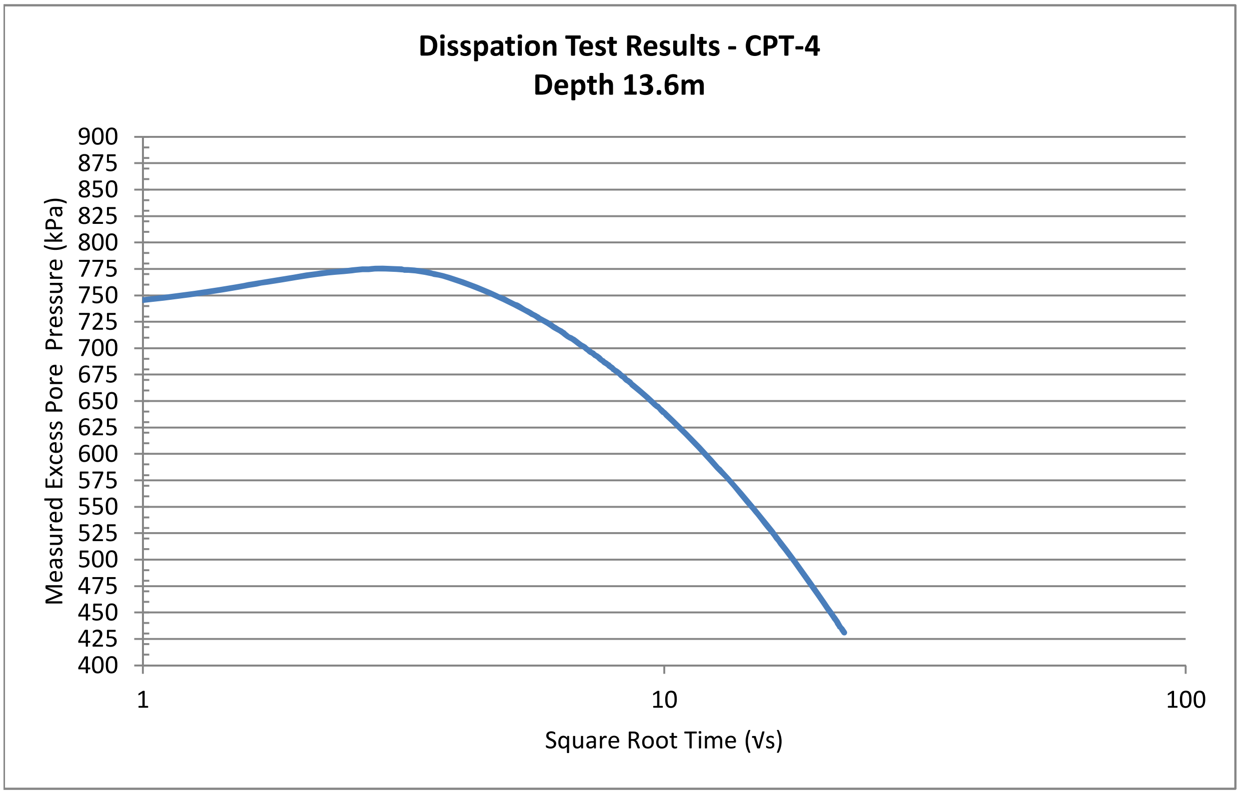
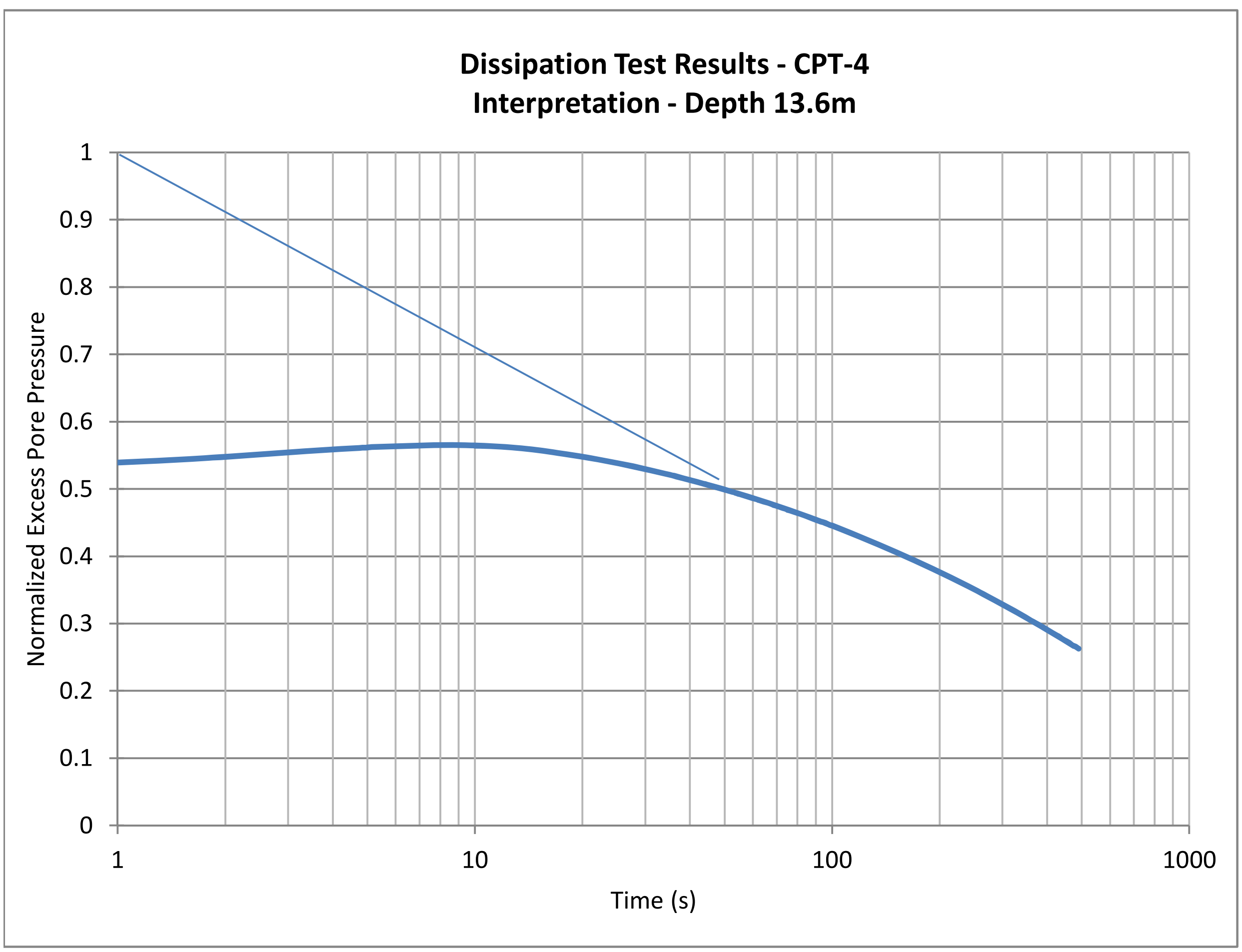
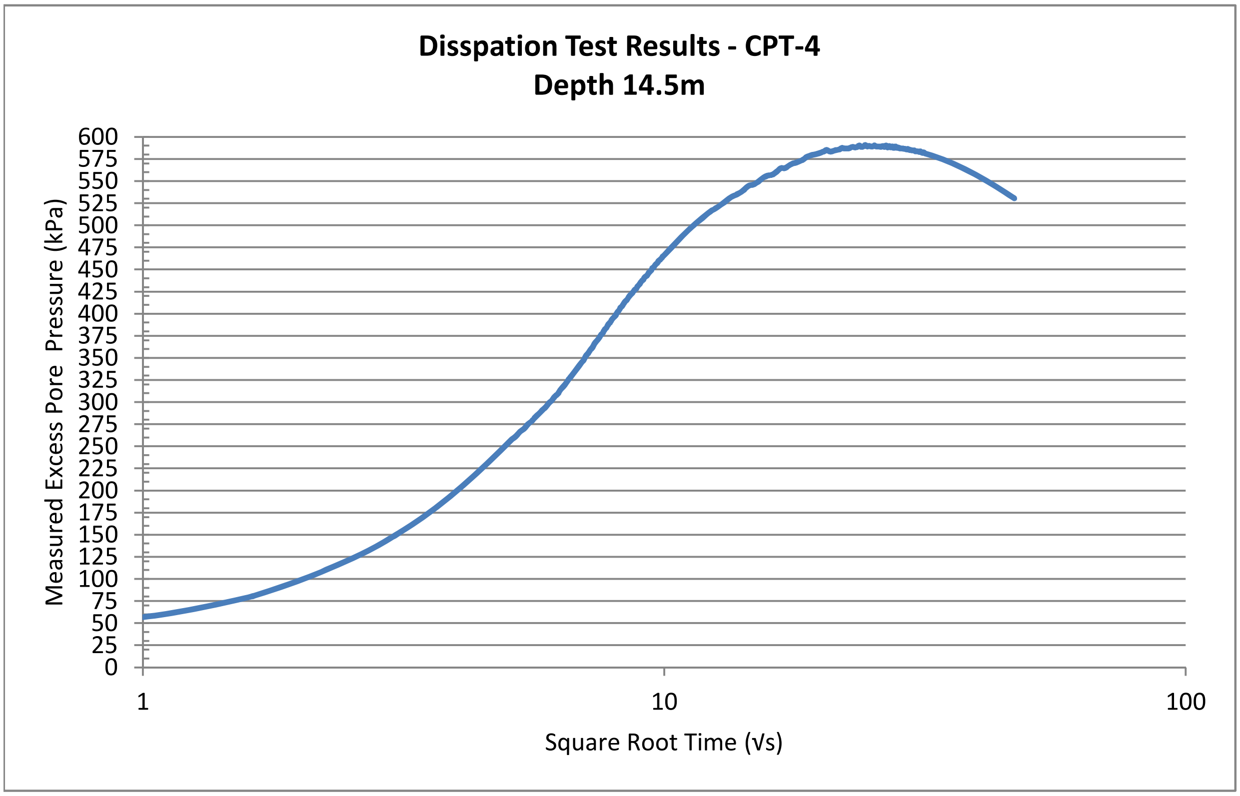
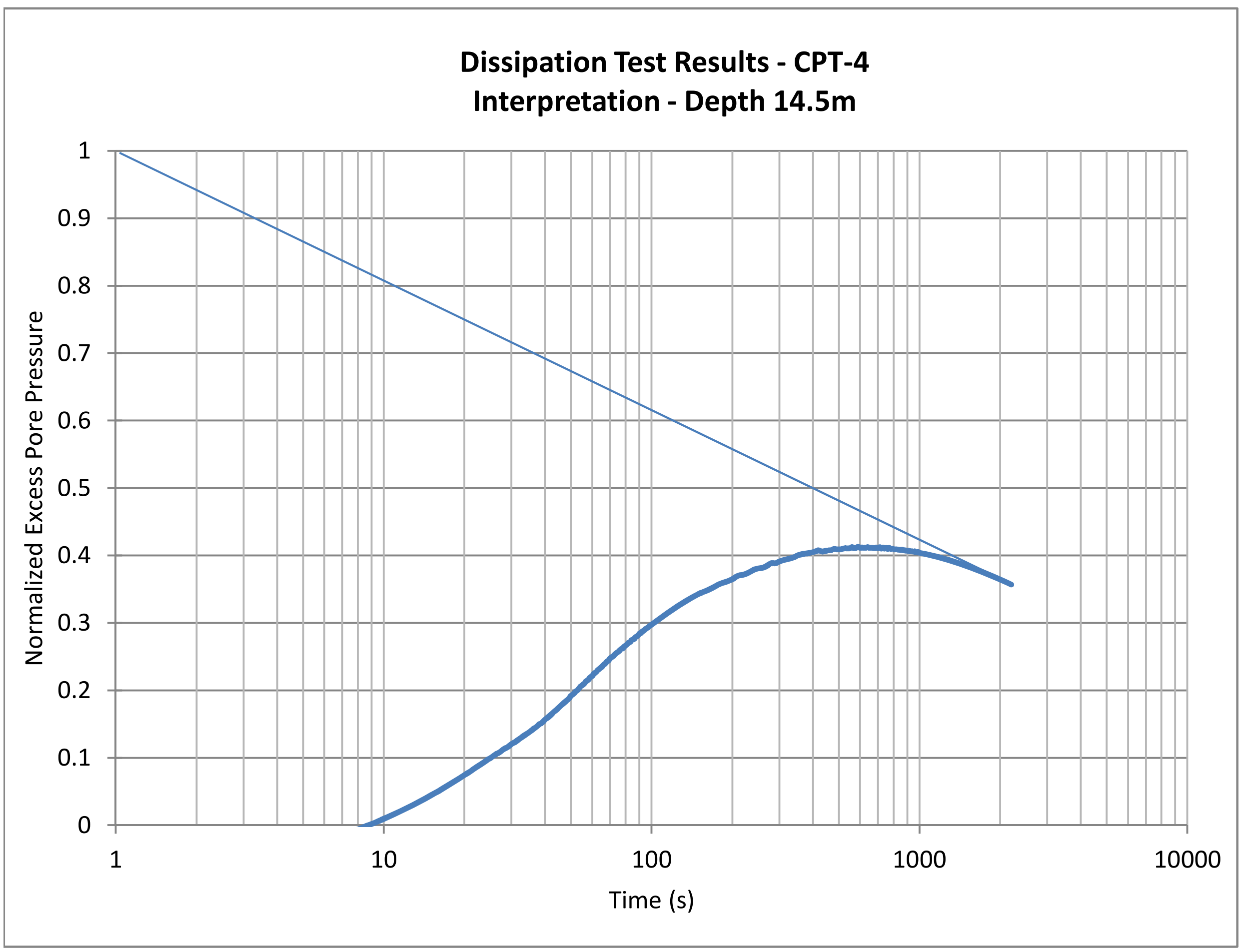

Disspation Test Results - CPT-4
Depth 13.6m
Measured Excess Pore Pressure (kPa)
900
875
850
825
800
775
750
725
700
675
650
625
600
575
550
525
500
475
450
425
400
1
10
100
Square Root Time (√s)


Dissipation Test Results - CPT-4
Interpretation - Depth 13.6m
Normalized Excess Pore Pressure
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1
10
100
1000
Time (s)





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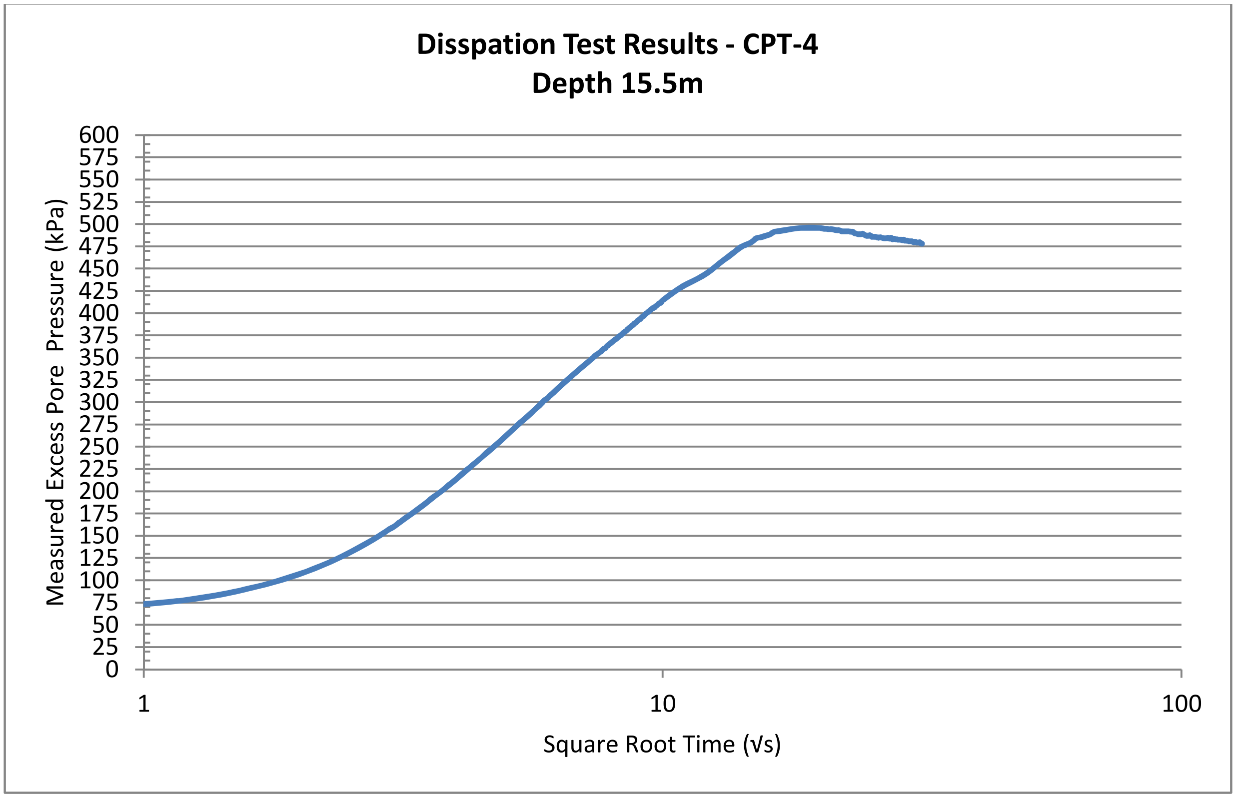
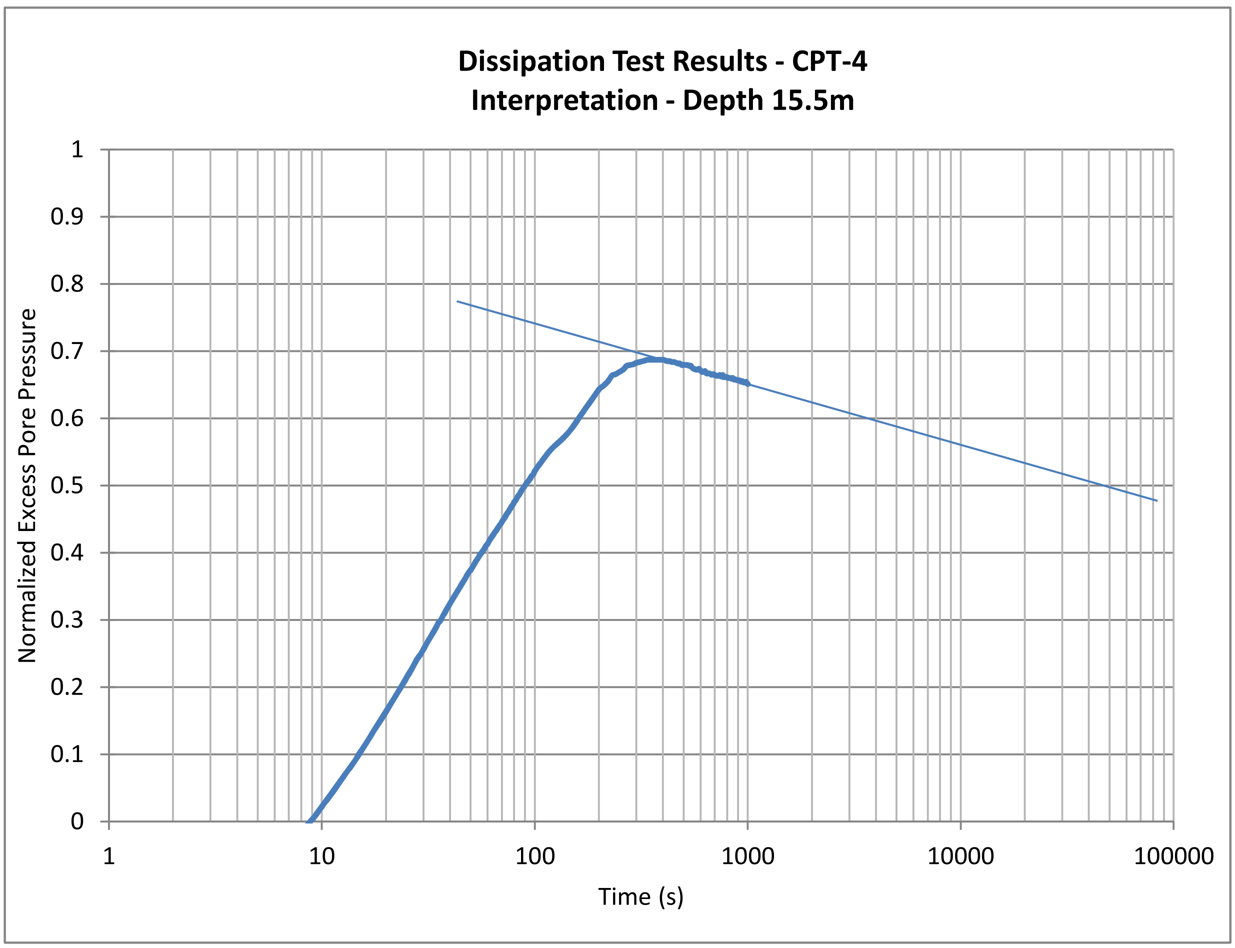
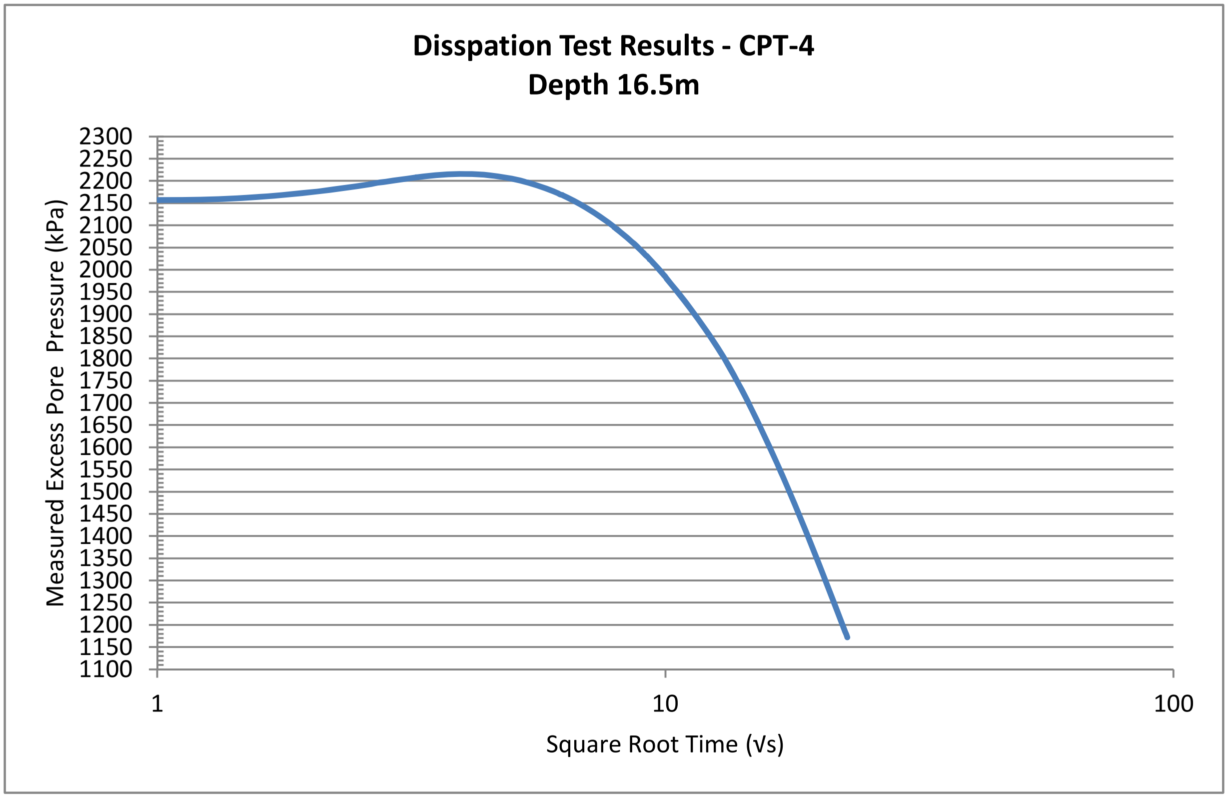
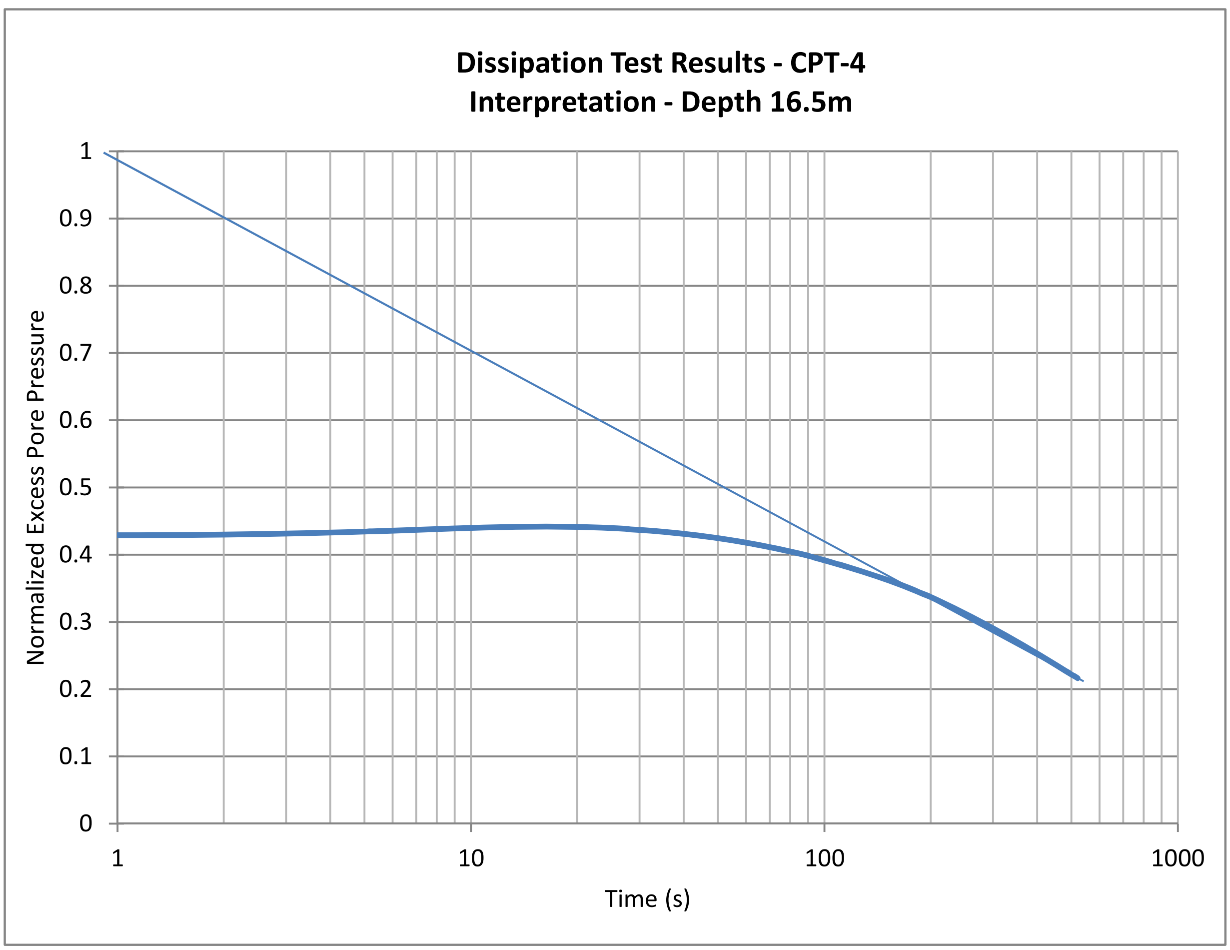

Disspation Test Results - CPT-4
Depth 15.5m
Measured Excess Pore Pressure (kPa)
600
575
550
525
500
475
450
425
400
375
350
325
300
275
250
225
200
175
150
125
100
75
50
25
0
1
10
100
Square Root Time (√s)

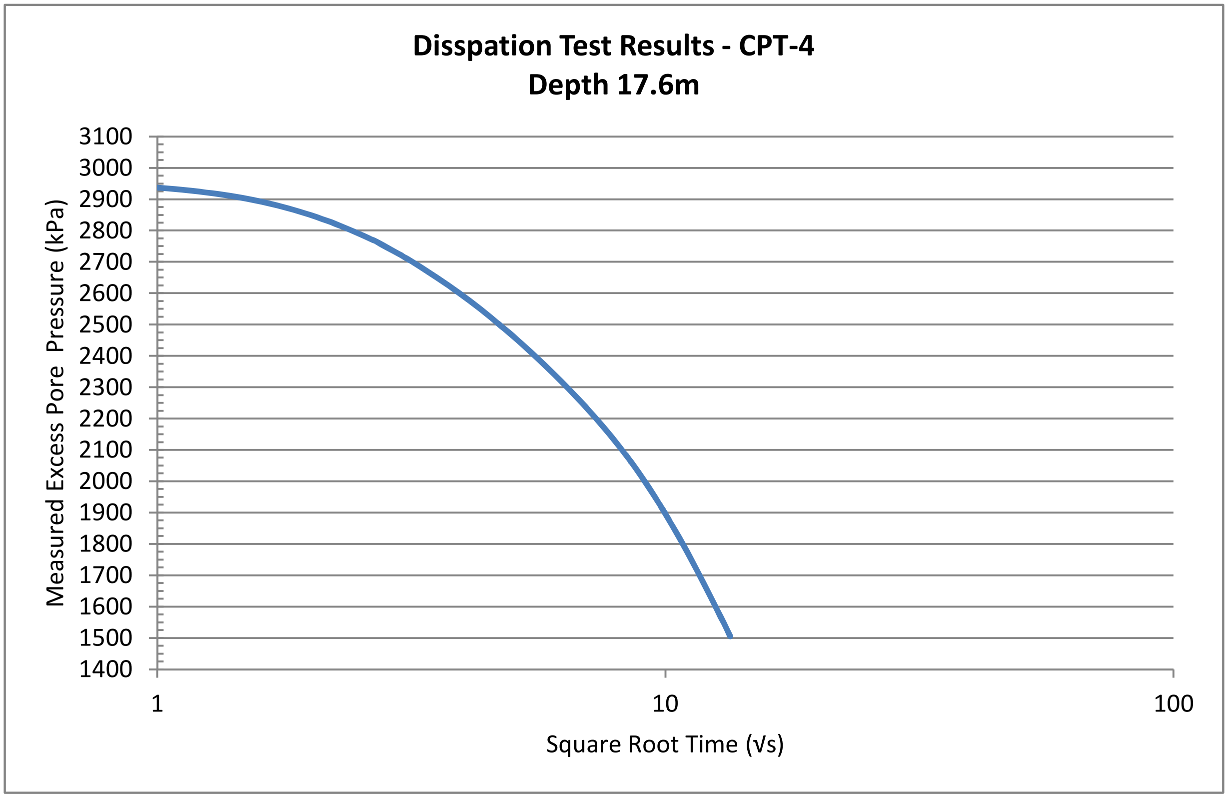

Dissipation Test Results - CPT-4
Interpretation - Depth 15.5m
Normalized Excess Pore Pressure
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1
10
100
1000
10000
100000
Time (s)

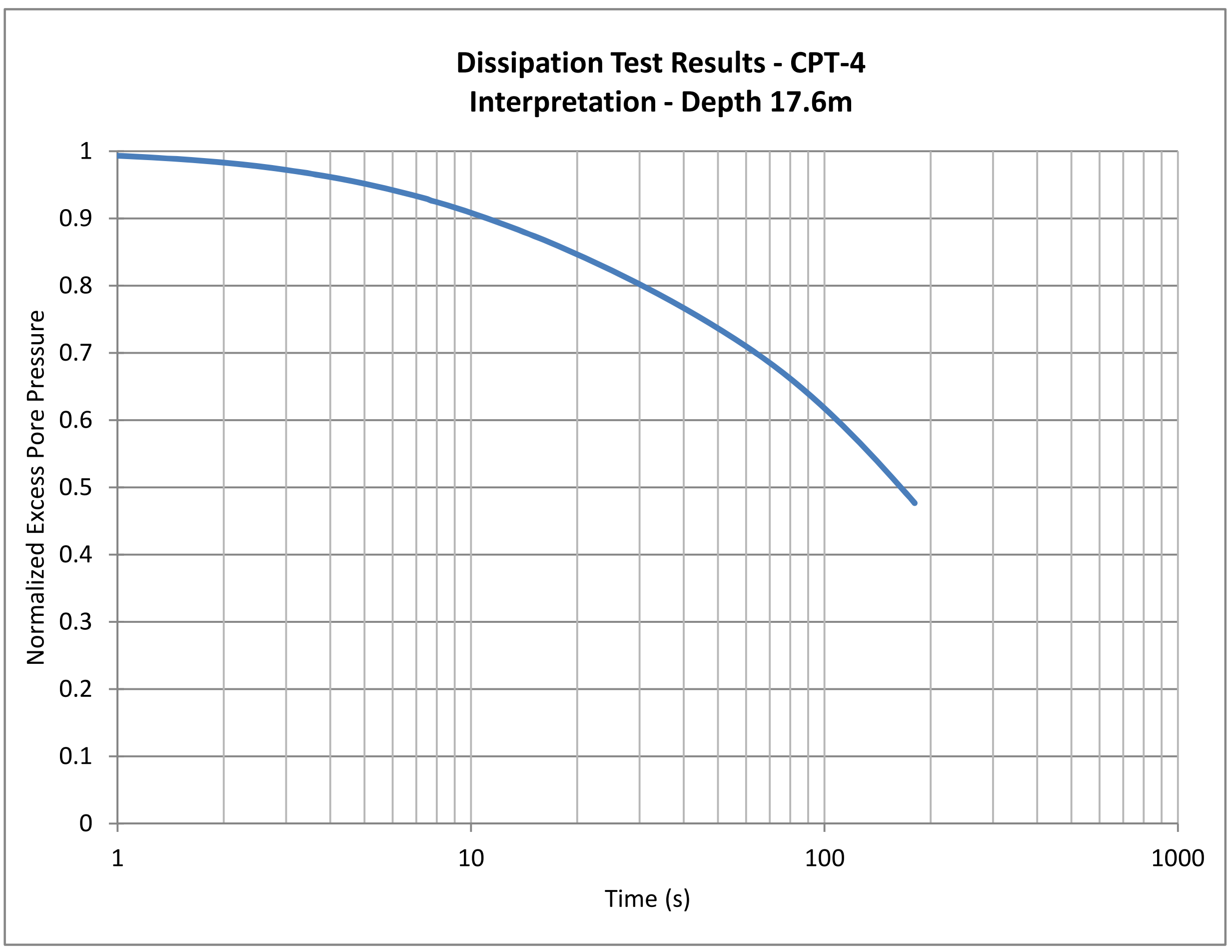


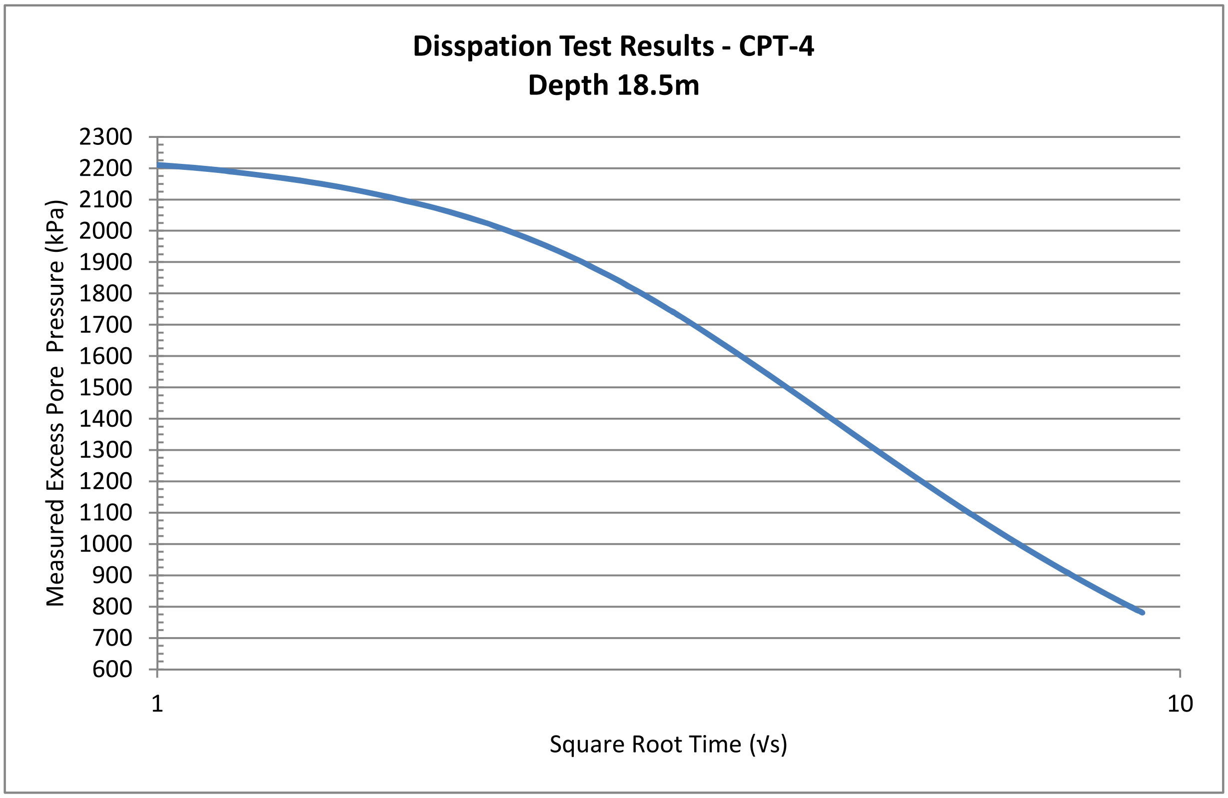
Disspation Test Results - CPT-4
Depth 16.5m
Measured Excess Pore Pressure (kPa)
2300
2250
2200
2150
2100
2050
2000
1950
1900
1850
1800
1750
1700
1650
1600
1550
1500
1450
1400
1350
1300
1250
1200
1150
1100
1
10
100
Square Root Time (√s)

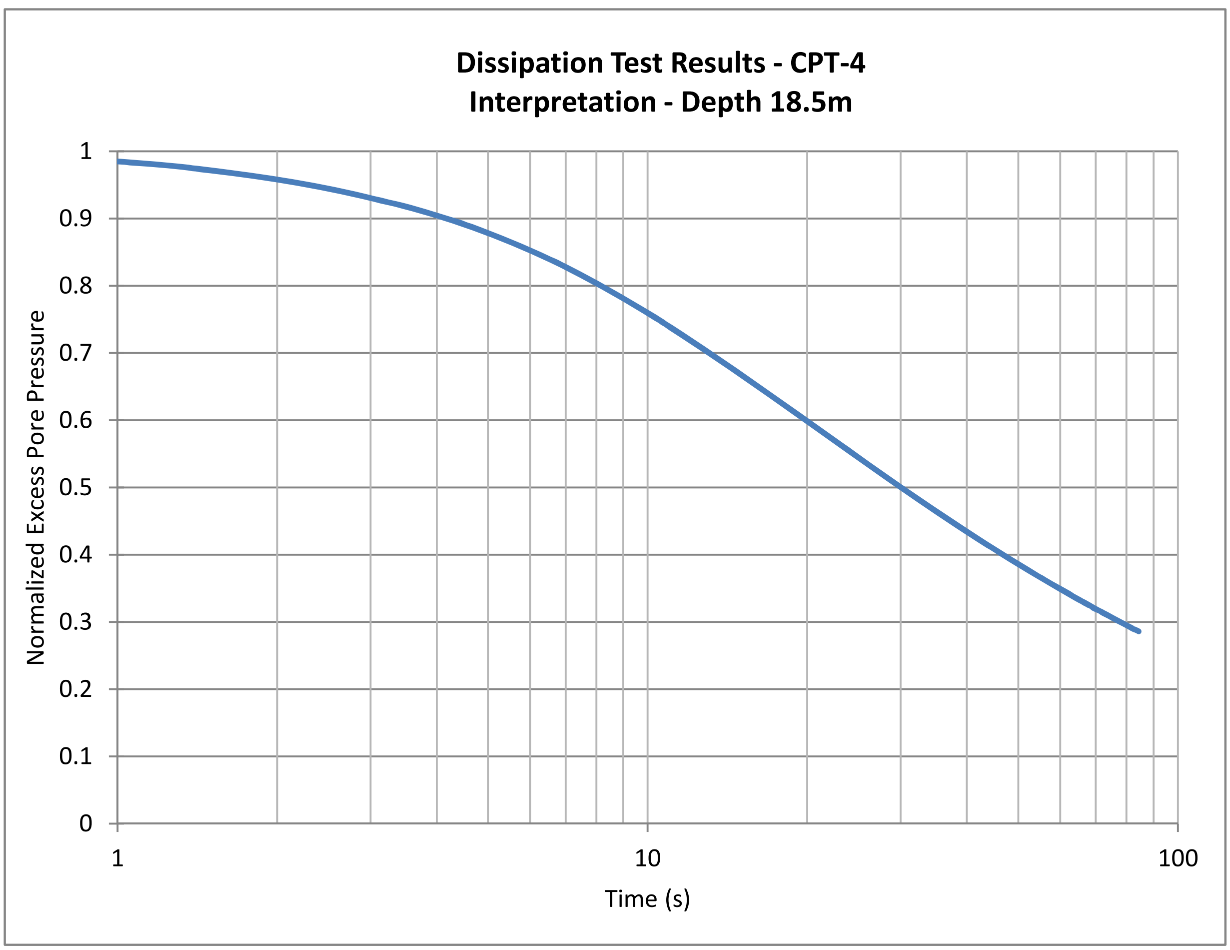


Dissipation Test Results - CPT-4
Interpretation - Depth 16.5m
Normalized Excess Pore Pressure
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1
10
100
1000
Time (s)

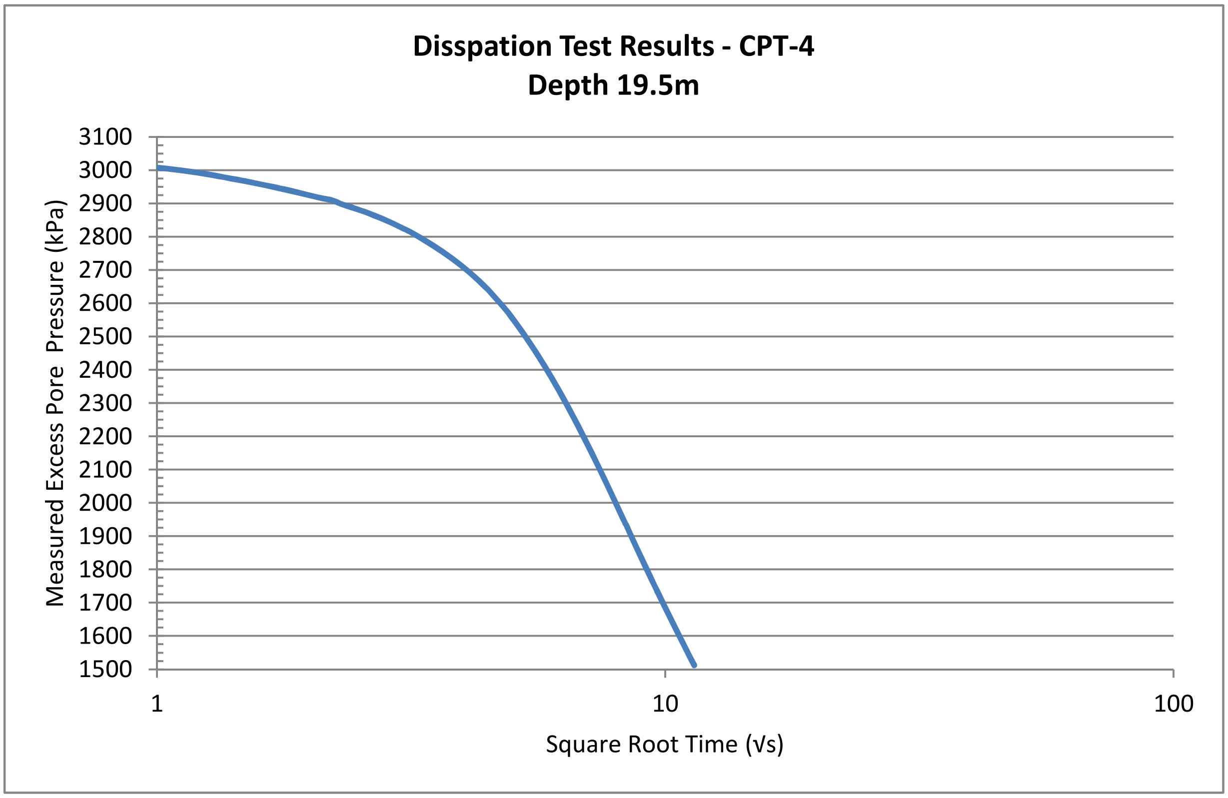

Disspation Test Results - CPT-4
Depth 17.6m
Measured Excess Pore Pressure (kPa)
3100
3000
2900
2800
2700
2600
2500
2400
2300
2200
2100
2000
1900
1800
1700
1600
1500
1400
1
10
100
Square Root Time (√s)

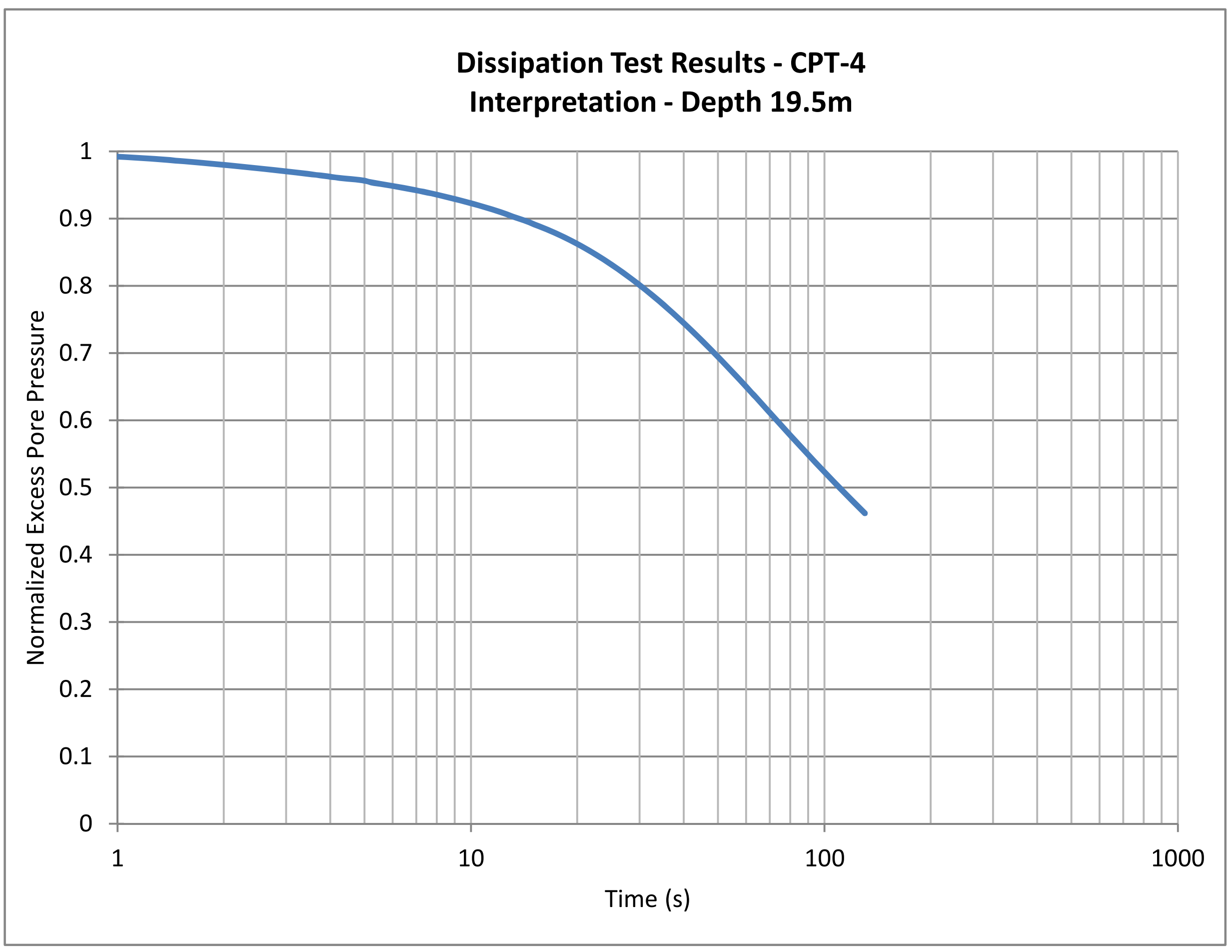

Dissipation Test Results - CPT-4
Interpretation - Depth 17.6m
Normalized Excess Pore Pressure
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1
10
100
1000
Time (s)

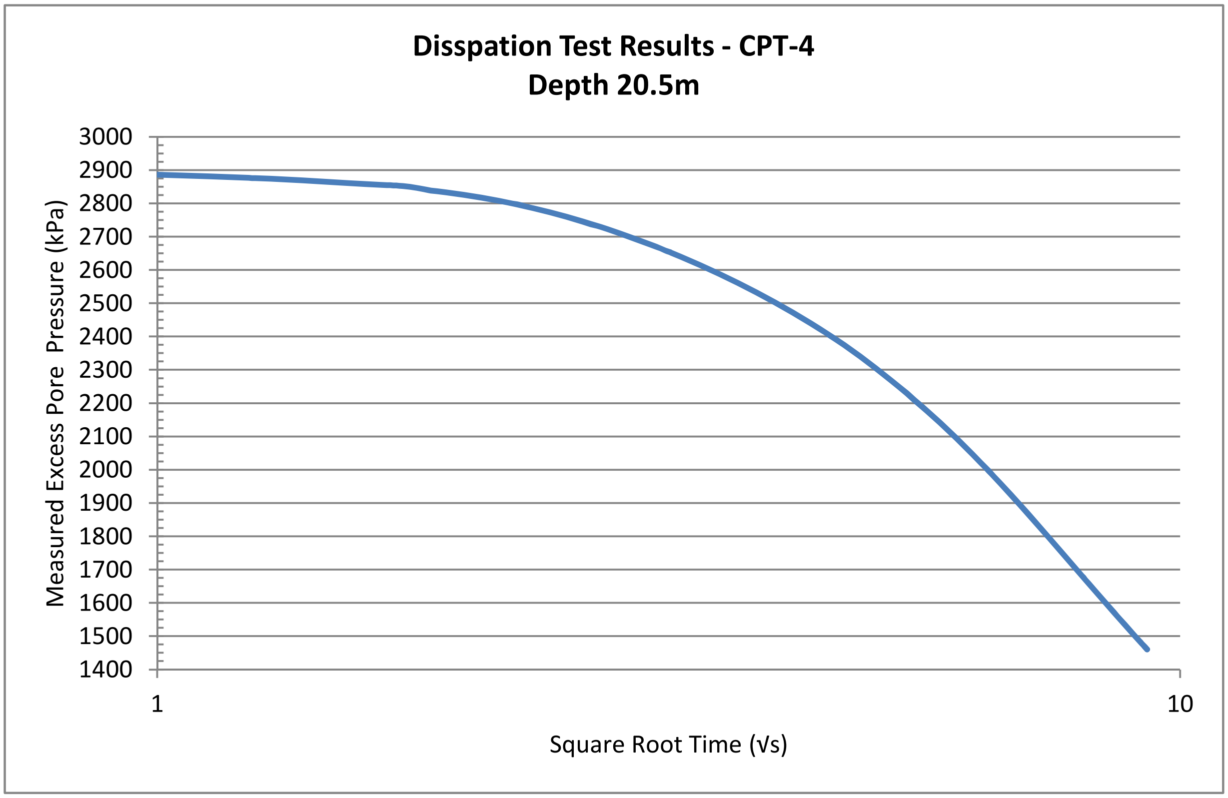


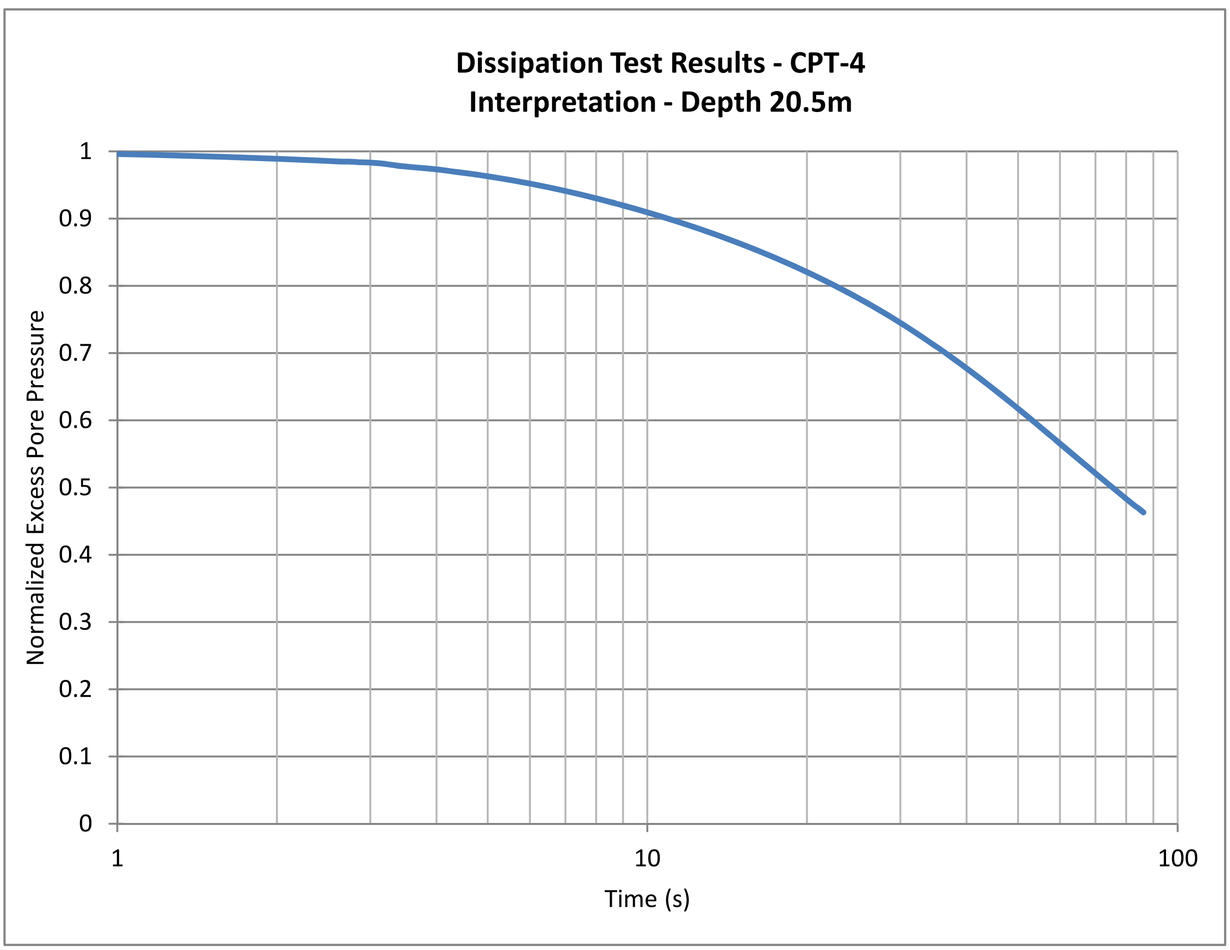


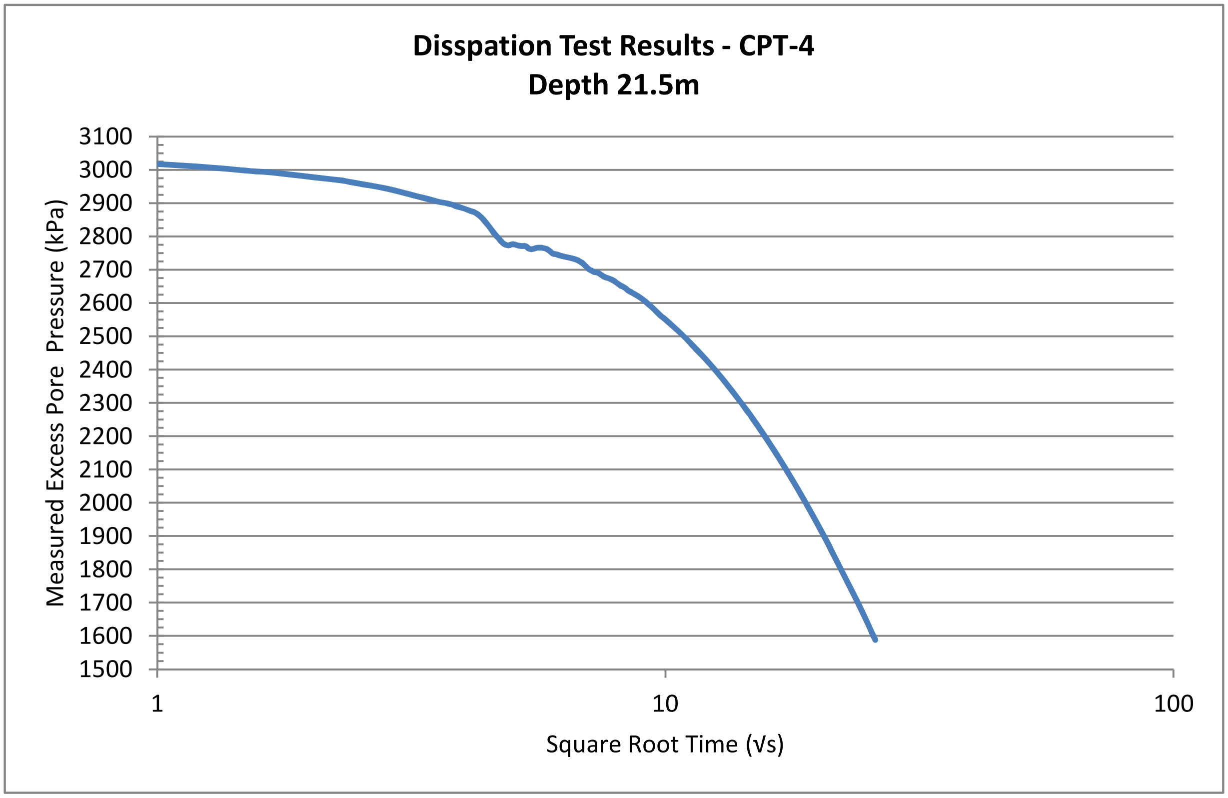
DownUnder Geotechnical Limited

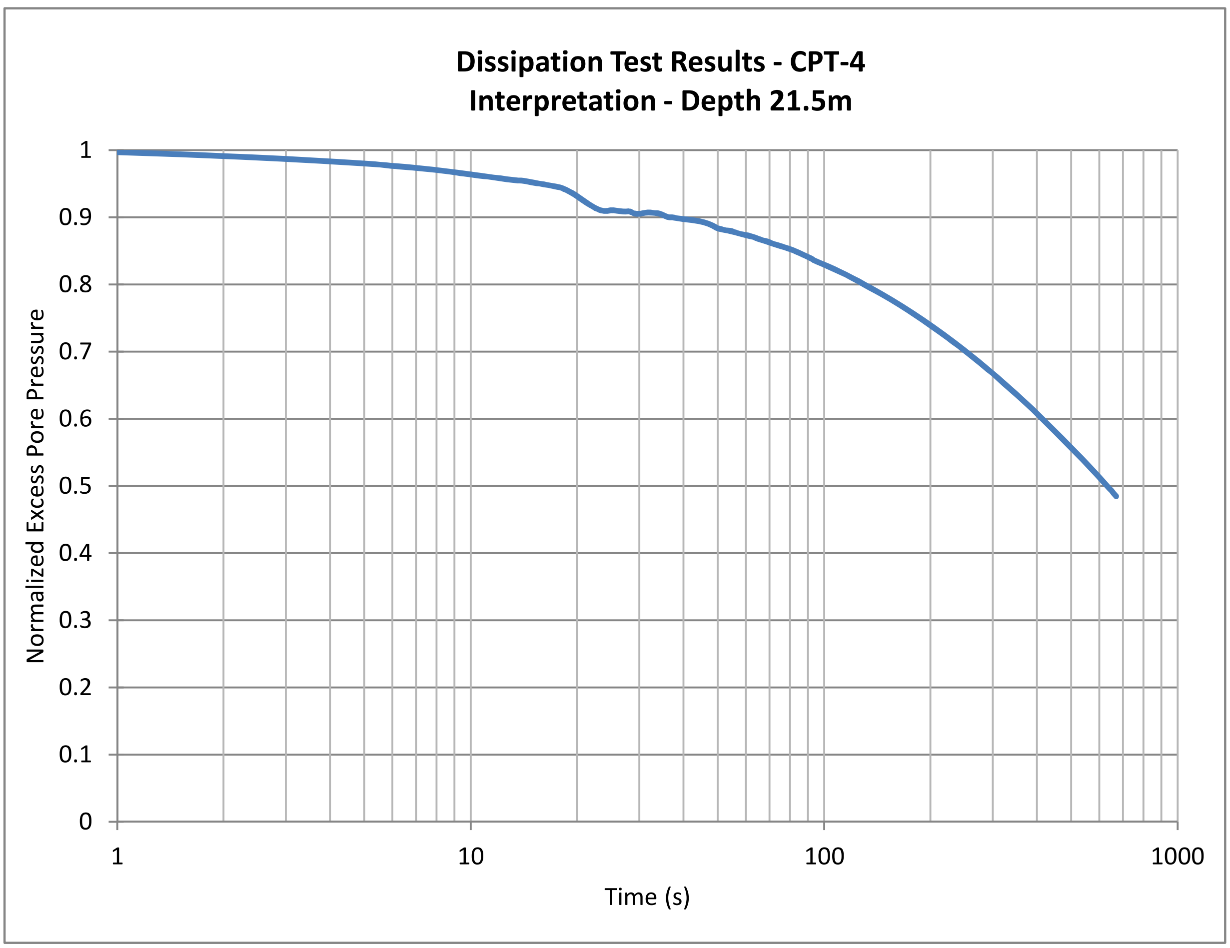

Disspation Test Results - CPT-4
Depth 19.5m
Measured Excess Pore Pressure (kPa)
3100
3000
2900
2800
2700
2600
2500
2400
2300
2200
2100
2000
1900
1800
1700
1600
1500
1
10
100
Square Root Time (√s)


Dissipation Test Results - CPT-4
Interpretation - Depth 19.5m
Normalized Excess Pore Pressure
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1
10
100
1000
Time (s)


Disspation Test Results - CPT-4
Depth 20.5m
3000
2900
2800
2700
2600
2500
2400
2300
2200
2100
2000
1900
1800
1700
1600
1500
1400
Measured Excess Pore Pressure (kPa)
1
10
Square Root Time (√s)


Dissipation Test Results - CPT-4
Interpretation - Depth 20.5m
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Normalized Excess Pore Pressure
1
10
100
Time (s)





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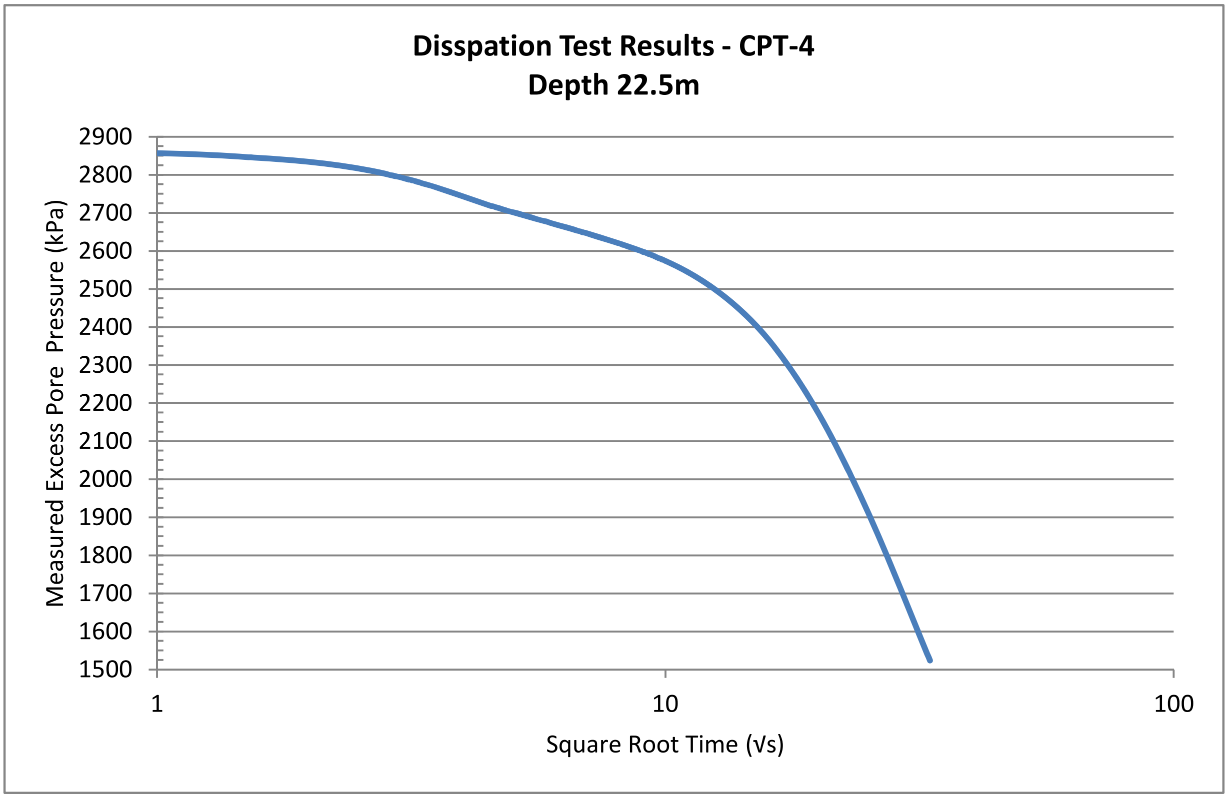

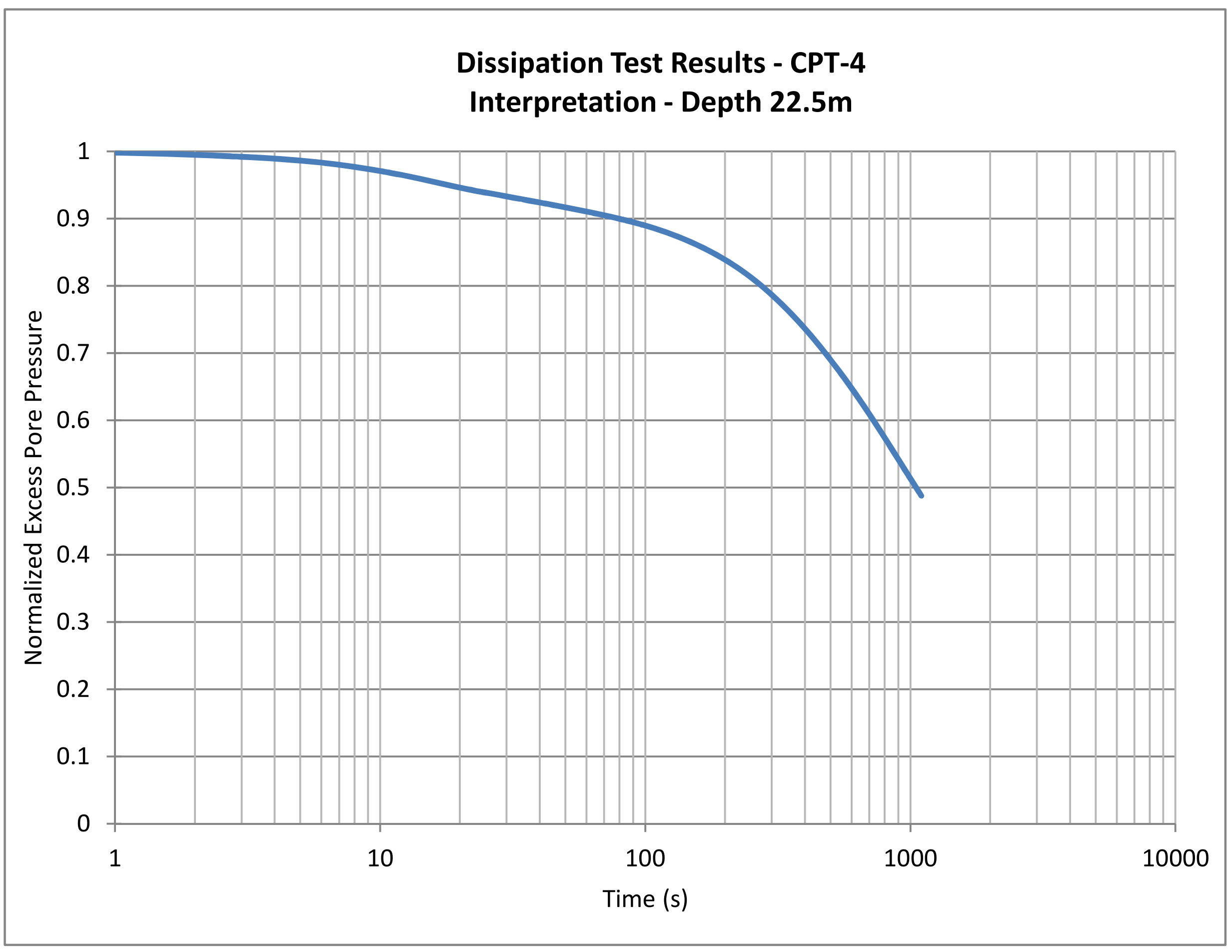

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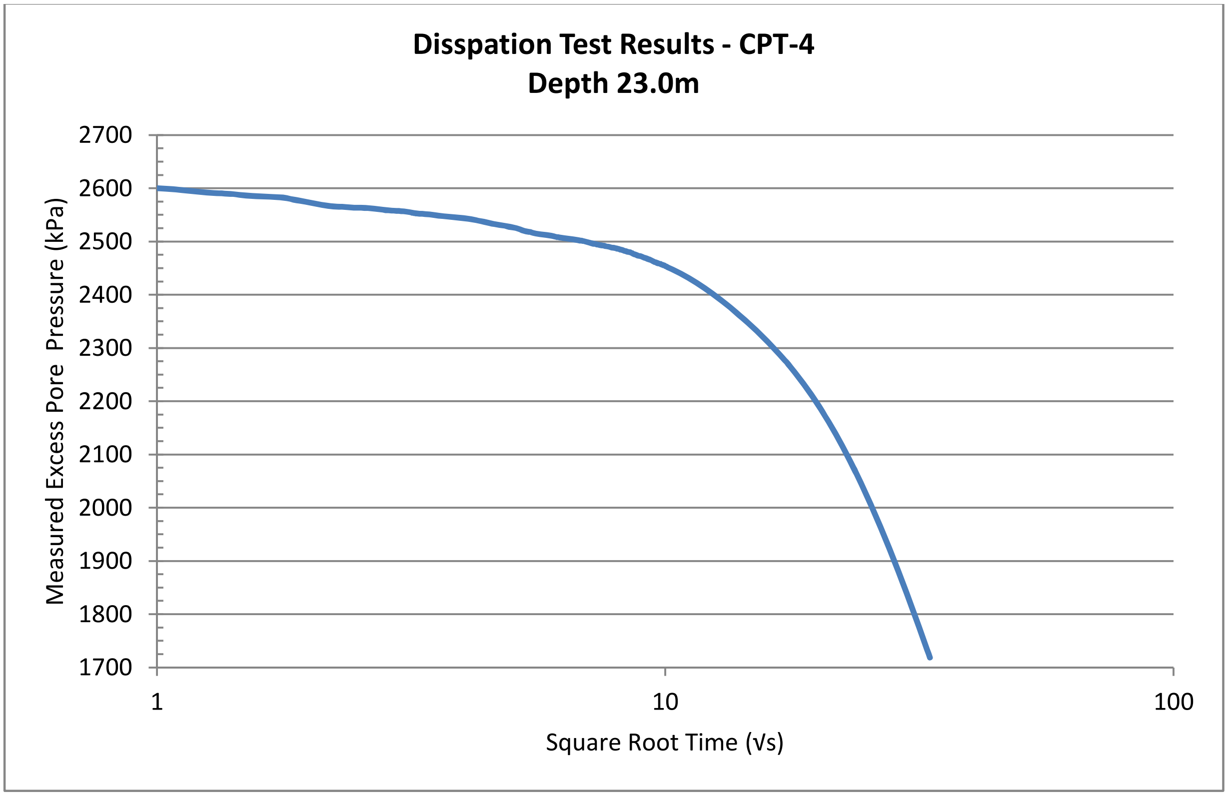

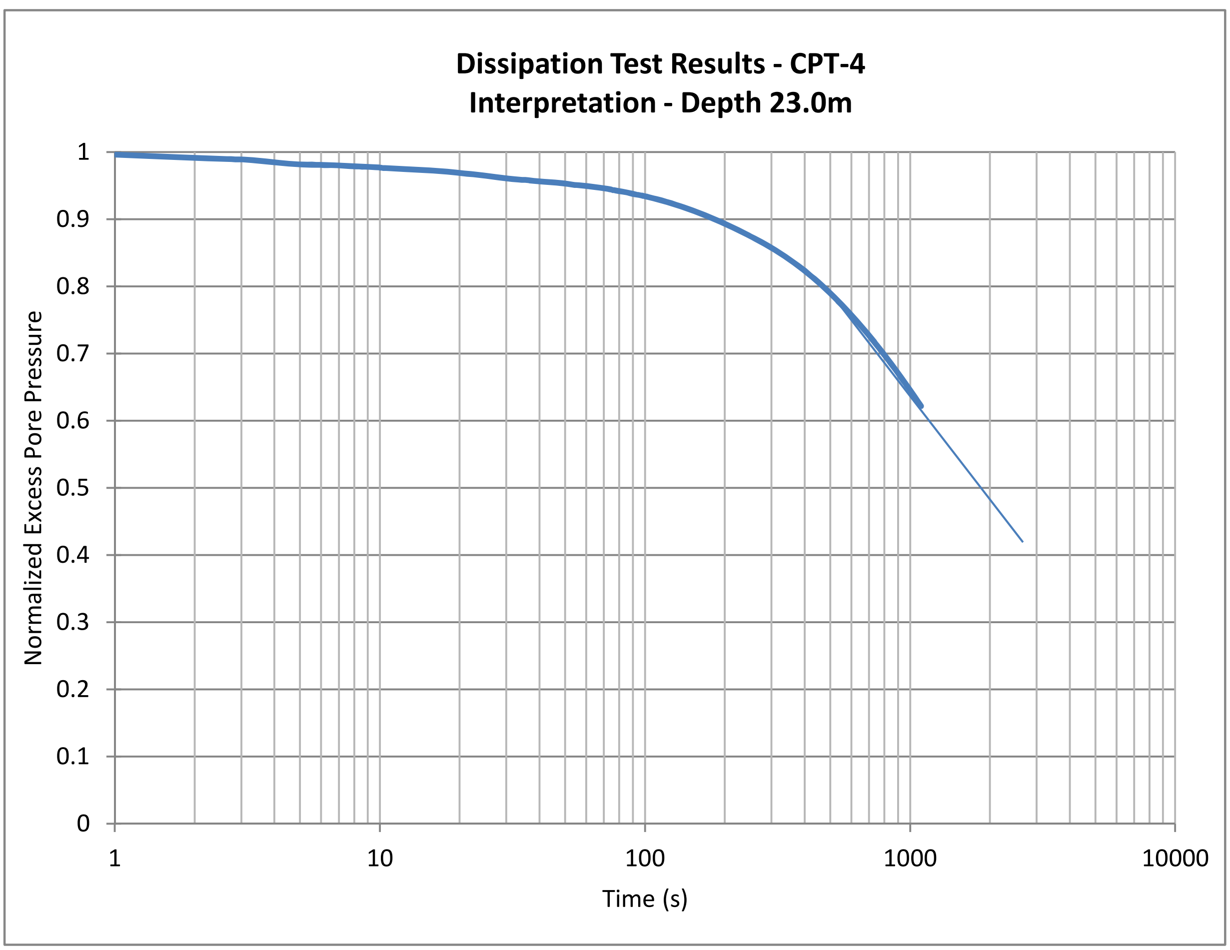

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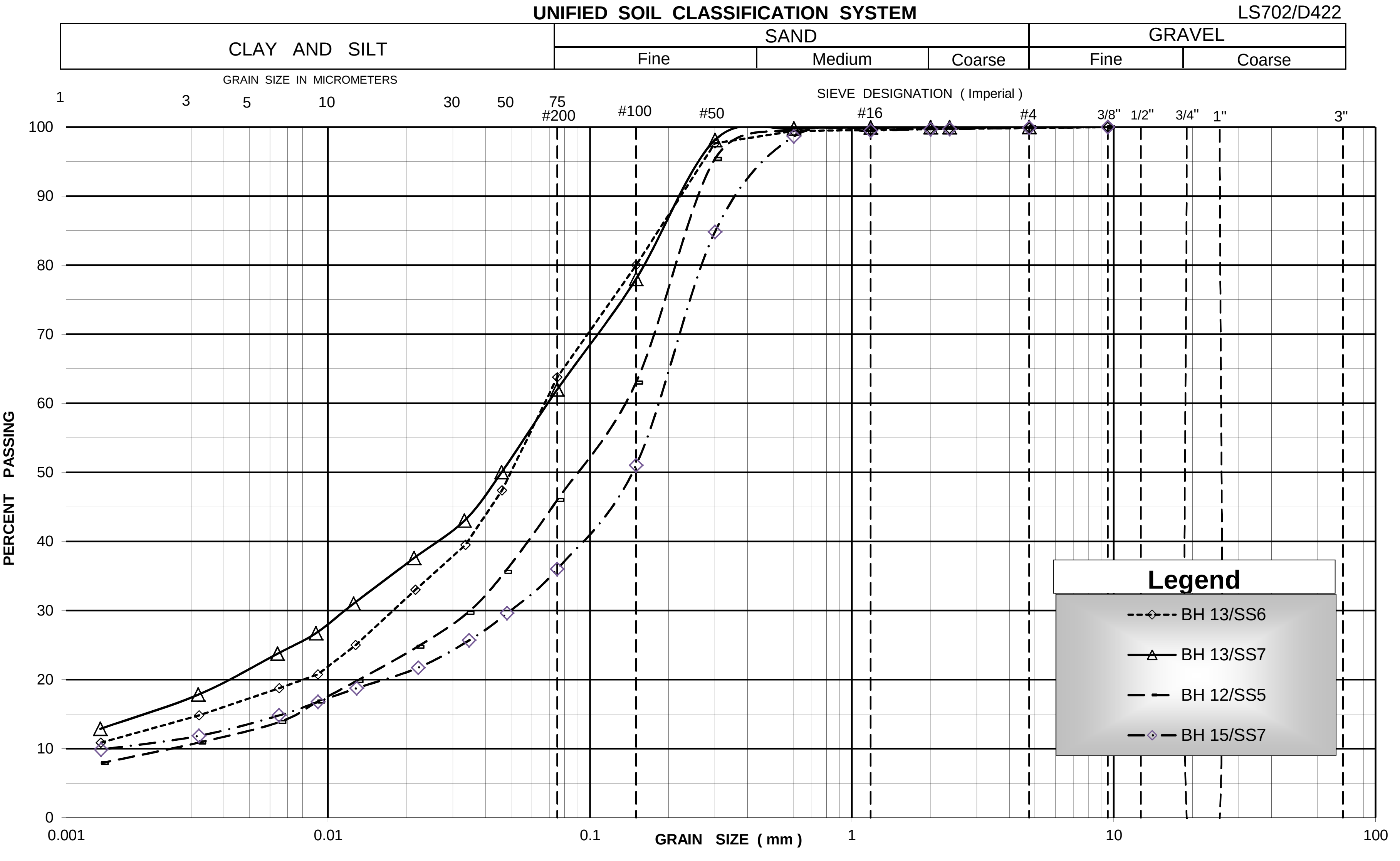
Appendix C

Laboratory Test Results



coffey

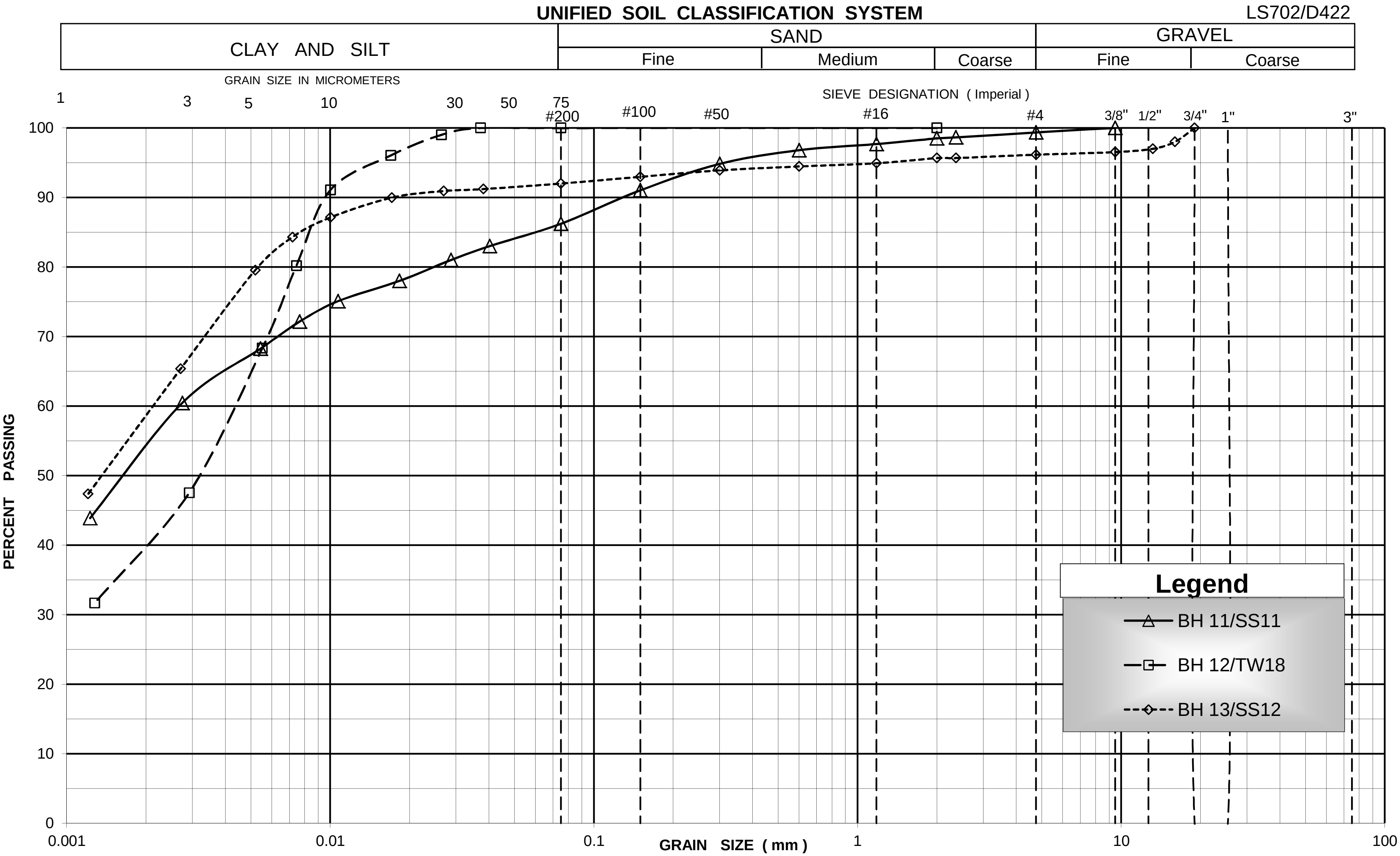
GRAIN SIZE DISTRIBUTION

Silty Fine Sand/Sandy Silt

FIGURE # : C-1

PROJECT # : TRANETOB20462AA

DATE : Nov 29, 2013



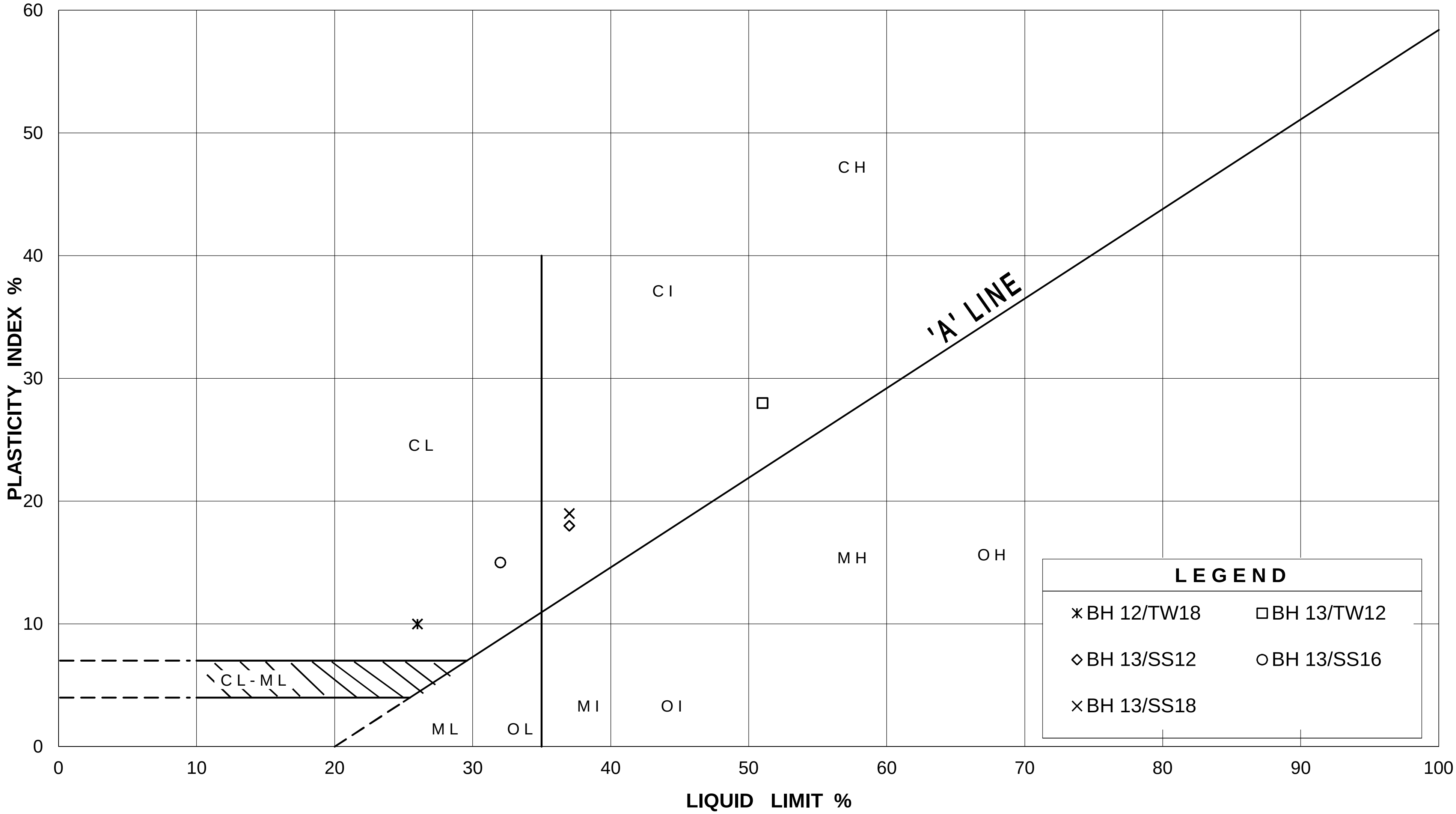
GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE # : C-2

PROJECT # : TRANETOB20462AA

DATE : Nov 29, 2013



coffey

PLASTICITY CHART

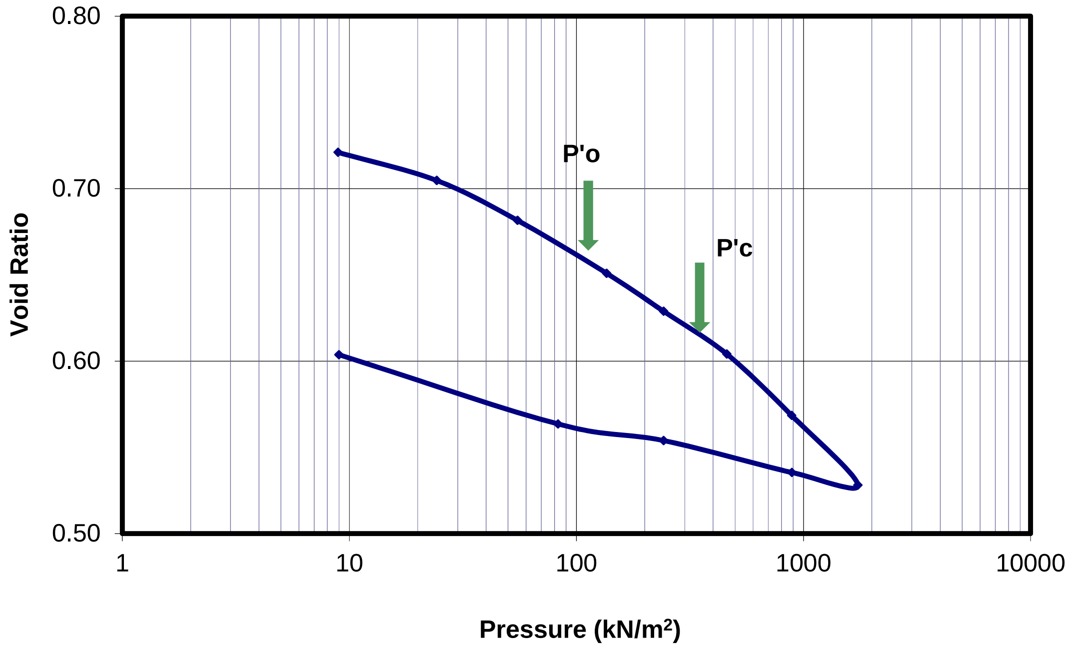
Silty Clay

Figure C-3

Project No. TRANETOB20462AA

DATE: Dec 09, 2013

Void Ratio versus Pressure



Coefficient of Consolidation vs. Pressure

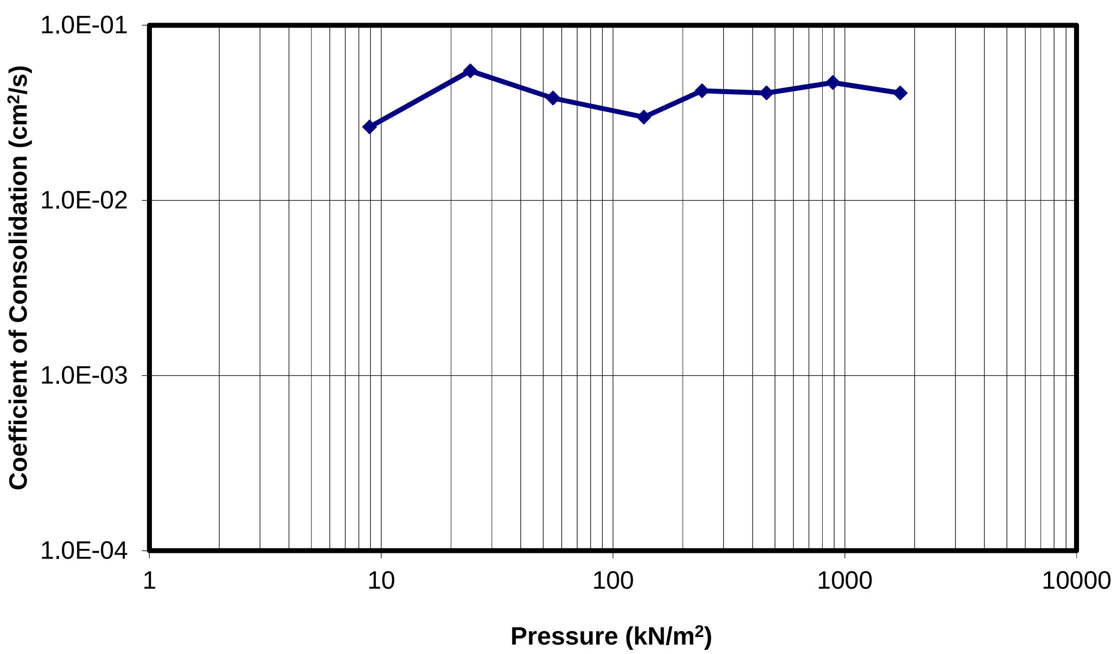


Figure C-4 Consolidation Test Results - BH 13 TW12

(P'c was estimated by strain energy method)

Appendix D

Field Vane Test Results

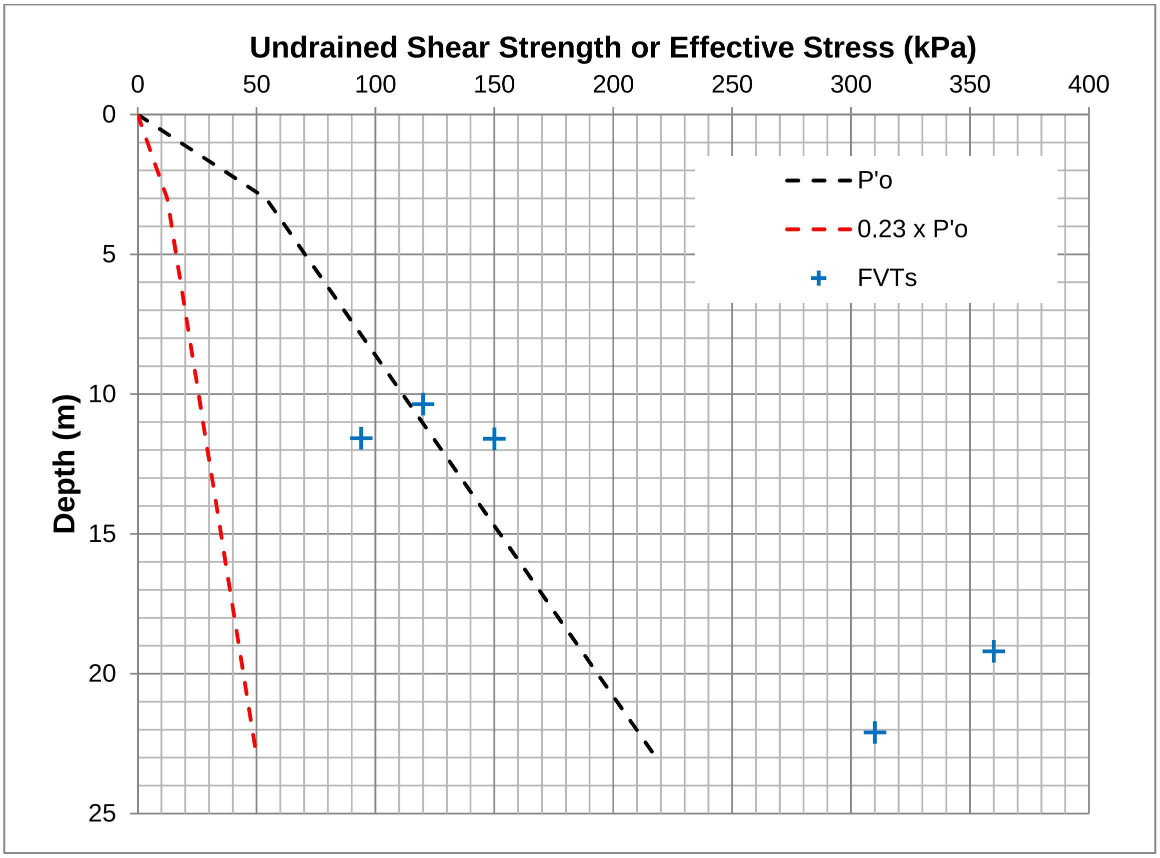
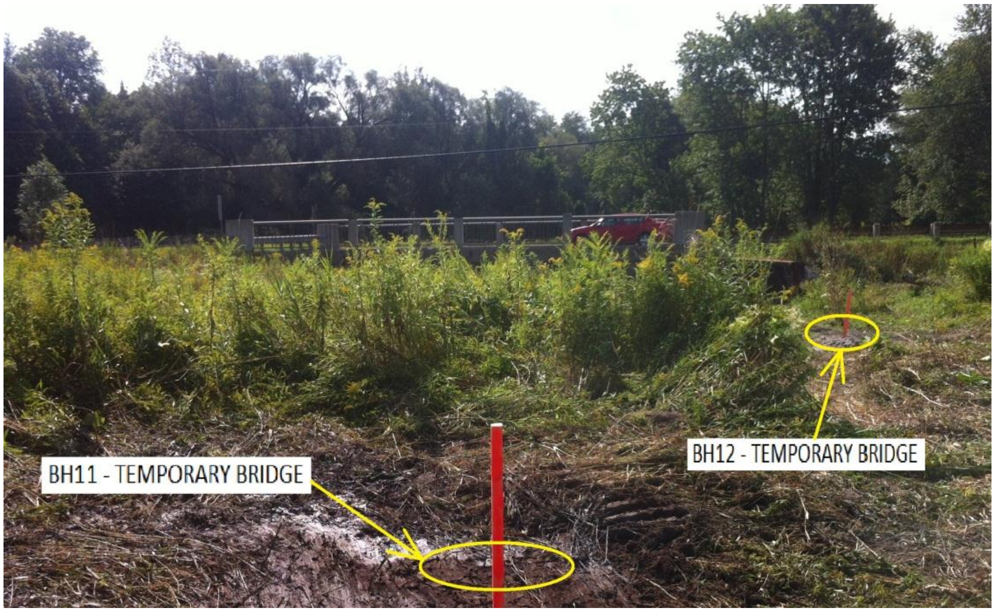


Figure D-1 Undrained Shear Strength or Effective Stress vs. Depth

Appendix E

Site Photographs



Photograph 1. Boreholes BH 11 and BH 12 (looking south)

Photograph 2. Boreholes BH 13 and BH 14
(looking north, BH14 was replaced with CPT 14)

Photograph 3. Borehole BH 15 (looking east)

Photograph 4. Borehole BH 16 (looking west)

Appendix F

Explanation of Terms Used in the Report

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
Φ'	$-^o$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	$-^o$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

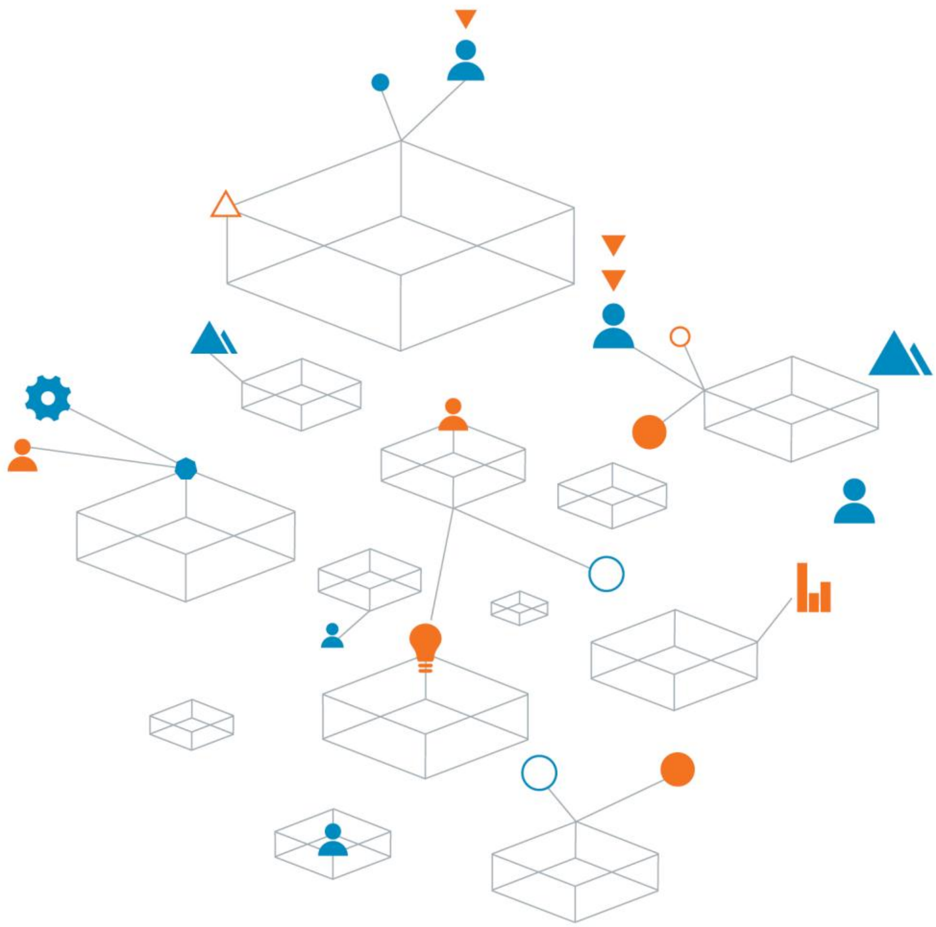
PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{,max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L – W_L)$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W – W_P)/ I_P$	i	1	HYDAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L – W) / 1_P$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERED SOIL	$e_{,max}$	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

McCormick Rankin

Foundation Design Report

Detour Bridge for Replacement of Glass's Bridge over The Innisfil Creek,
Site No. 30-254, Highway 89, Town of Innisfil, MTO Central Region,
W.P. 2108-11-00, GEOCRES NO. 31D-573
TRANETOB20462AA
16 April 2014



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of all our
projects

CONTENTS

FOUNDATION DESIGN REPORT
DETOUR BRIDGE FOR REPLACEMENT OF GLASS'S BRIDGE
OVER THE INNISFIL CREEK, SITE NO. 30-254, TOWN OF INNISFIL
MTO CENTRAL REGION, W.P. 2108-11-00

7 DISCUSSION AND RECOMMENDATIONS

7.1 General

The existing Glass's Bridge over the Innisfil Creek will be replaced. To maintain highway traffic during bridge demolition and new construction, a temporary bridge will be constructed on a detour alignment, about 30 m north of the existing bridge. The proposed detour structure will be a 21.3 m long, 7 panel triple single (TSR3) bailey bridge. The proposed width of the bridge is 10.6 m, including a footwalk assembly, as shown on the GA Drawings in Appendix G. The finished grade elevation will be 226 m at both abutment locations. As the existing grade elevations at the abutment locations are 224.5 m (west) and 225.0 m (east), the proposed construction will entail average grade raises of about 1.0 m (east) and 1.5 m (west) at the approaches.

Innisfil Creek flows in a southerly direction. It is 4-5 m wide at normal Creek water level. It meanders and suffers from extensive erosion on its banks. At the detour bridge location, the water level in the creek was at elev. 222.8 m on September 20, 2012 and at elev. 223.4 on August 1, 2013. The 50 year flood level is said to be at elev. 224.9 m (see GA drawings in Appendix G).

The geotechnical site investigation, described in Part 1, shows the presence of some surficial fill and fine grained non-cohesive surficial soils within 1-4 m of the original ground level, mixed with some organics in a random fashion, underlain by fine grained granular soils consisting of sandy silt with silty fine sand and occasional clayey silt and silty clay seams. These fine grained granular soils extend to depths of 8-10 m, or to elev. 215-217 m. On the basis of SPT N values, this deposit is very loose to occasionally dense, but typically loose to compact.

The major stratigraphic unit at this site, below the upper silty sand surficial deposits, is a silty clay deposit containing thin clayey silt, clay, silt and fine sand seams. Boreholes drilled at the approach embankment locations were terminated within this deposit at depths of 11-13 m below ground surface or elev. 212-216 m. In this cohesive deposit, the SPT N values were 6 blows/0.3 m to in excess of 100 blows/0.3 m. In situ field vane tests gave undrained shear strengths of 94 kPa to in excess of 200 kPa, indicating a firm to hard consistency.

The massive silty clay deposit was fully penetrated in Boreholes BH12 and BH13. It is underlain by a lower non-cohesive soil deposit (coarse silt with some fine sand) at depths of 26-29 m, or elev. 196-199 m. Boreholes BH12 and BH13 were terminated in the silt deposit at a depth of about 31 m or Elev. 193.5 m. SPT N values ranged from 67 blows/0.3 m to in excess of 100 blows/0.3 m, indicating the silt stratum is very dense. It is also a source of artesian water with heads above ground surface of 3-4 m.

The groundwater at the time of the investigation was found to be generally between elev. 224 m and 222.5 m. It can be expected to be largely controlled by the creek water level and to major weather events. Creek and groundwater levels at elev. 222.8 m have been assumed for foundation design and stability analyses.

7.2 Detour Bridge Foundations

Typically, bailey bridges are supported on rock-filled timber cribs, or spread footings on engineered fill. Deep foundations can also be considered for bailey bridge structures, depending on site conditions.

7.2.1 Shallow Foundations

As shown on the GA drawing (see Appendix G), the proposed bailey bridge can be supported on spread footings on engineered fill. Timber crib ballast walls will be placed at each abutment location for the approaches. Continuous grade beams (about 10 m long) are recommended. The grade beams will reduce the bearing pressure on the subgrade and minimize distortion and differential settlement of and between individual footings.

The width of the concrete footings shown on the GA drawing is 1.5 m and the thickness is 1.7 m. The underside of the concrete footings will be at elev. 223.3 m. The following procedure is recommended to prepare the base for the footing.

- Excavate to at least 1 m below the underside of the proposed footings to accommodate two layers of geogrid within a granular engineered fill. The excavation should be carried out when the creek water level is low and certainly not when higher than excavation base level to avoid dewatering.
- The base of the excavation should extend 1.5 m beyond the perimeter of the footing(s).
- Temporary excavation slopes should be no steeper than 1H:1V above the groundwater table and 2H:1V below the groundwater table.
- After excavation, the exposed subgrade at the base of the excavation should be inspected to ensure that it is free of organic and unsuitable soil. Should delays be anticipated in the placing of granular soils for the approach fills, a mud mat of lean concrete may be placed to protect the subgrade against time-related deterioration.
- If sub-excavation below the prevailing groundwater level is necessary to remove unsuitable soil,75 mm clear stone can be placed and pushed into the soil to provide a firm working base, to avoid sub-excavation.
- Place minimum 1 m thick Granular A fill, compacted to minimum 98% Standard Proctor Dry Density (SPMDD) in loose lift thicknesses of maximum 300 mm, with layers of biaxial geogrid placed within the granular fill at 0.4 m and 0.8 m above the bottom of the granular fill.

With the adoption of the aforementioned foundation preparation, the following resistance/reactions are recommended for the design of the 1.5 m wide footings:

Factored Geotechnical Resistance at ULS =	180 kPa
Geotechnical Reaction at SLS	120 kPa

For the evaluation of sliding resistance the friction factor (ultimate) between the underside of the concrete footing and the surface of compacted Granular A can be taken as 0.55.

At the SLS reaction, the anticipated immediate (during and immediately after construction) and longer term (5-10 years) footing settlements will be in the order of 40-50 mm and 20-25 mm respectively, provided the foundations are not underlain by extremely compressible organic rich soils. If these anticipated footing settlements are unacceptable, the use of deep foundations or preloading can be considered, depending on the construction schedule. If the schedule permits, preloading will help to reduce the magnitude of anticipated settlements by half. The effects of preloading are discussed in Section 7.4.

Excavation Dewatering

Excavations that need to be taken to base elevations below the prevailing groundwater levels can be made without the need for well point site dewatering by using interlocking steel sheet pile cofferdams. The depth of toe penetration, D, of cantilevered sheet pile walls should be, for preliminary design and cost estimating purposes, 2×H, where H = retained height of exterior soil above base level of excavation, assuming a horizontal exterior surface and zero surcharge. The contractor should provide stamped and sealed shop drawings. For cantilevered interlocking steel sheet pile design, the coefficients of active and passive earth pressures, K_A and K_P, can be assumed to be 0.36 and 1.0 respectively within the silty sand deposit above the silty clay stratum. The saturated unit weight of the silty sand may be assumed to be 18.0 kN/m^3.

7.2.2 Deep Foundations

Two deep foundation alternatives can be considered – timber piles and helical screw auger piles.

7.2.2.1 Helical Piles

A helical screw auger pile is a segmented deep foundation system with helical bearing plates (helixes) welded to a central square or round, solid or hollow, steel shaft. Applied loads are transferred from the shaft to the helical plates that bear down on the soil below them. Helical piles do not generate earth spoil and do not require excavation or dewatering. They are easy to install and can be readily load tested. Concrete grout can be used for added resistance when hollow shafts are used. These piles are generally installed by "design-build" subcontractors.

At the detour bridge abutment locations, the final depth of penetration will depend on the torque developed during installation. The lowest helix, however, should be kept above elev. 217 m. The centre to centre pile spacing should be at least three times the largest helix diameter. Typically, a 0.5 m thick granular pad is provided under footings to transfer loads to the helical piles.

For preliminary design, a factored geotechnical resistance of 100 kN per helical auger pile at ULS and a geotechnical reaction of 75 kN per pile at SLS may be used to control settlements to within 35 mm after installation and application of full loading. The lateral resistance of helical piles should be provided by the design-build subcontractor. Load tests should be conducted (with various helix diameters placed at various elevations to corroborate assumed axial and lateral load capacities and to serve as a demonstration project for future consideration by the MTO.

7.2.2.2 Driven Timber Piles

The following axial capacities may be used for Size 36 timber piles (OPSS 903) driven to toe elevations at or above elev. 210 m into the very stiff silty clay stratum:

Factored Geotechnical Axial Resistance at ULS =	180 kN
Geotechnical Axial Reaction at SLS=	120 kN

Minor consolidation settlement (30-40 mm) is anticipated for a one year construction period.

The horizontal resistance of a single Size 36 timber pile, based on Brom's method and literature searches of installations in similar soil deposits, may be taken as follows:

Factored Horizontal Resistance at ULS	25 kN / pile
Resistance at SLS	15 kN / pile

Pile driving should be controlled with a recognized pile driving formula, such as the Hiley Formula, and/or with a pile driving analyser (PDA with CAPWAP).

7.3 Seismic Design

Site Coefficient

The subsurface conditions encountered at the site are represented by Soil Profile Type III (see Clause 4.4.6.2 of CHBDC CAN/CSA-S6-06). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient, S, for the site is 1.5.

Seismic Zone and Zonal Acceleration Ratio (A)

Table A3.1.1 of the CHBDC provides a zonal Acceleration Ratio (A) of 0.05 and Velocity Related Seismic Zone (Z_v) of 1 for Barrie. As site coefficient (S) is 1.5, and the zonal acceleration is 0.05, the design zonal acceleration ratio for the site can be taken as A=0.075.

Temporary Structure

Seismic analysis may not be required for temporary structures.

Liquefaction Potential

Loose submerged silty fine sand and sandy silt soils may liquefy during earthquake events or from construction induced vibrations. The risk for liquefaction of the upper silty sand and surficial deposits at this site, under earthquake excitation, is very low based on earthquake magnitude data obtained from Natural Resources Canada.

7.4 Approach Embankments

7.4.1 Approach Embankment Stability

Slope stability analyses were carried out using the embankment cross sections provided by MRC (see Appendix G). The stability of the proposed embankments was analysed with Slope/W and the Morgenstern-Price method of analysis for both short term (undrained) and long term (drained) analysis. The soil parameters for analysis are summarized in Table 7.4.1.1. The results of the analyses are given in Appendix I.

In summary the analysis indicates that 1.5 m high embankments constructed with 2:1 side slopes and encroaching within 1.5 m of the creek bank will remain stable.

Table 7.4.1.1 Soil Parameters Used for Slope Stability

Soil Type	Unit Weight (kN/m^3)	Shear Strength Parameters			
		Undrained		Drained	
		Cohesion (kPa)	Angle of internal friction (degrees)	Cohesion (kPa)	Angle of internal friction (degrees)
New Embankment Fill	20.5	0	32	0	32
Existing Fill	18.5	0	27	0	27
Top 2-3 m of Upper Silty Fine Sand to Sandy Silt	17.5	0	27	0	27
Lower Portion of Upper Silty Fine Sand to Sandy Silt	18.0	0	28	0	28
Upper Silty Clay	18.0	80	0	0	28
Granular Pad	21.0	0	34	0	34

7.4.2 Forward Slope Stability

Analyses for forward slopes were carried out using the soil parameters contained in Table 7.4.1.1 and the profile provided by MRC, which we understand represents the profile along the south edge of the bridge (see GA drawing presented in Appendix G). The results of analysis are given in Appendix I. Based on these results the recommended forward slopes should be no steeper than 2H:1V.

7.4.3 Approach Embankment Settlements

The maximum grade raise at the west and east abutment approaches is 1.2 m and 1.8 m respectively. The estimated total settlement of approach fills are shown below:

1.0 m grade raise = 20-30 mm

1.5 m grade raise = 40-50 mm

These settlement magnitude estimates can be reduced by 50% if the embankments are left in place for two months, as anticipated from a revised construction schedule.

7.4.4. Embankment Materials and Construction

In general, about 200 mm stripping of topsoil will be required for new embankment construction, since the site is located within the flood plain of Innisfil Creek.

In areas designated for pile driving, the maximum nominal size of soil particles or rock fragments used in engineered fills or for site grading purposes should not be larger than 60 mm. The materials for new embankment construction should consist of approved soils such as Granular 'B' Type I or SSM. The fill material used for the approach embankment fills should satisfy OPSS 212. Fill placement should meet or exceed the requirements of OPSS 501 and OPSS 206. In general, fill should be placed in loose lift thicknesses not exceeding 300 mm. Each lift should be compacted to at least 95% SPMDD.

Where new fill abuts into an existing embankment, the side slope of the existing embankment should be benched as per OPSD 208.010.

7.5 Construction Considerations

All excavations must be carried out in conformance with the Occupational Health and Safety Act (OHSA) Regulation 213/91. In accordance with OHSA, the soils which can be expected to be encountered during site/subgrade preparation can be classified as Type 4 both below and above the groundwater table.

Excavation and backfilling should be carried out in accordance with OPSS 902.

The on-site excavated soil is considered unsuitable for re-use as backfill. It can be re-used for general site grading or for slope flattening beyond a 1:1 slope extending down from the shoulder rounding.

Since the temporary detour bridge and embankment will be constructed 30 m north of the existing bridge, construction-related disturbance to the existing Enbridge gas main pipe on the south side of the existing bridge is not an issue.

The impact of construction related vibrations needs to be assessed based on the choice of foundation types for both the detour bridge and the replacement structure. A non-standard Special Provision (NSSP) can be prepared once foundation design choices have been finalized.

7.6 Scour and Frost Protection

The surficial soils are highly erodible. Proper erosion controls and scour protection measures are required.

The design frost penetration depth for this area is 1.5 m. Therefore, a permanent soil cover of about 1.5 m or its thermal equivalent of insulation is required for frost protection. For rip-rap such as rockfill, only one-half of the rockfill thickness should be assumed to be effective in providing protection against frost penetration.

8 CLOSURE

The Limitations of Report, as quoted in Appendix K, are an integral part of this report.

For and on behalf of Coffey

pp

Gwangha Roh, Ph.D., P.Eng.
Senior Geotechnical Engineer

Vasantha Wijeyakulasuriya, P. Eng.
Senior Principal

LICENSED PROFESSIONAL ENGINEER
V. WIJEYAKULASURIYA
100186910
17 April 2014
PROVINCE OF ONTARIO

Cam Mirza, P.Eng.
Principal

LICENSED PROFESSIONAL ENGINEER
C. MIRZA
31976012
2014 04 17
PROVINCE OF ONTARIO

Appendix G

GA Drawing and Embankment Cross-Sections

DISTRICT
CONT. No.
WP No. 2108-11-00

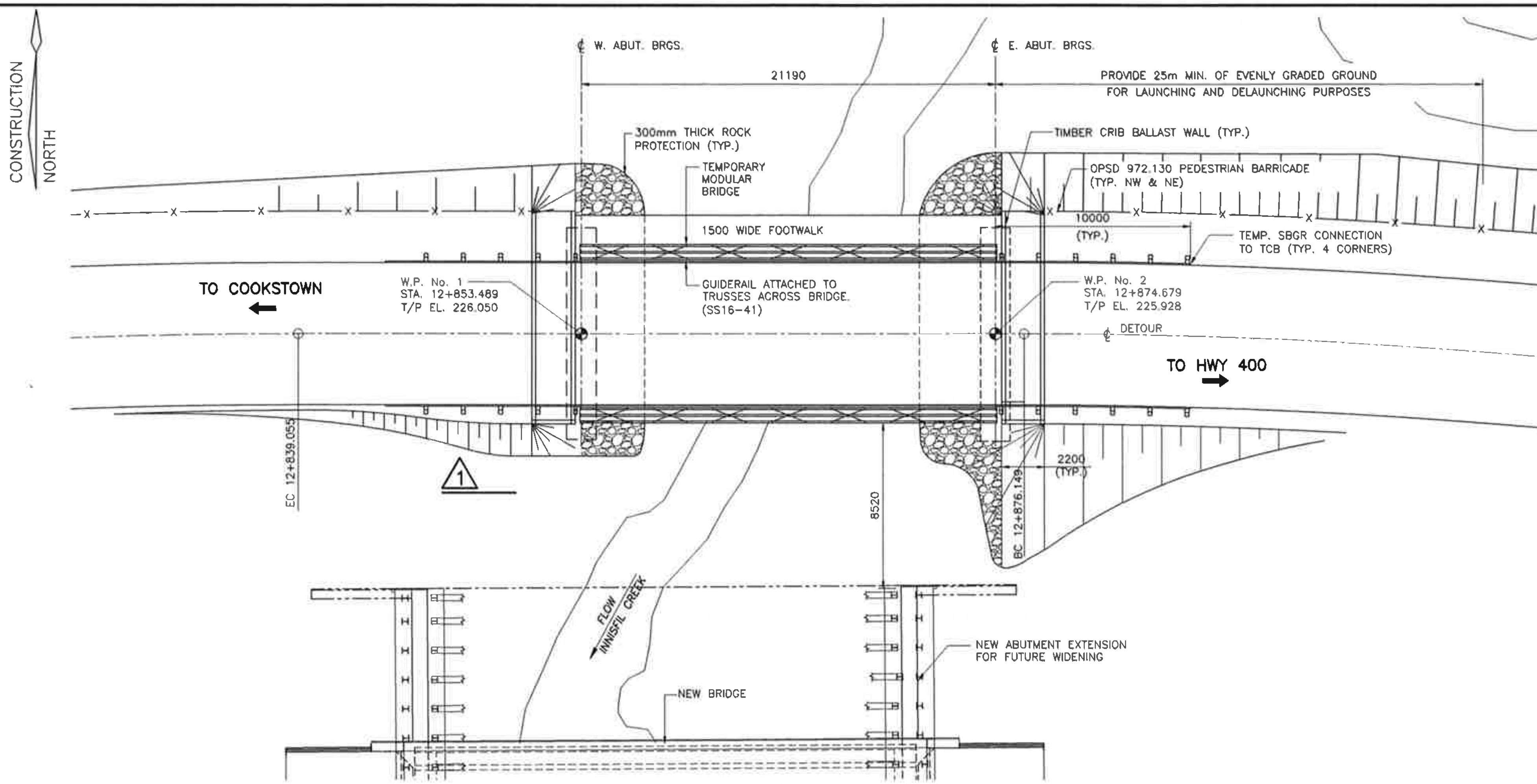
HWY 89 DETOUR
INNISFIL CREEK

TEMPORARY MODULAR BRIDGE
GENERAL ARRANGEMENT

SHEET

MRC McCORMICK RANKIN
A member of MMM GROUP

METRIC



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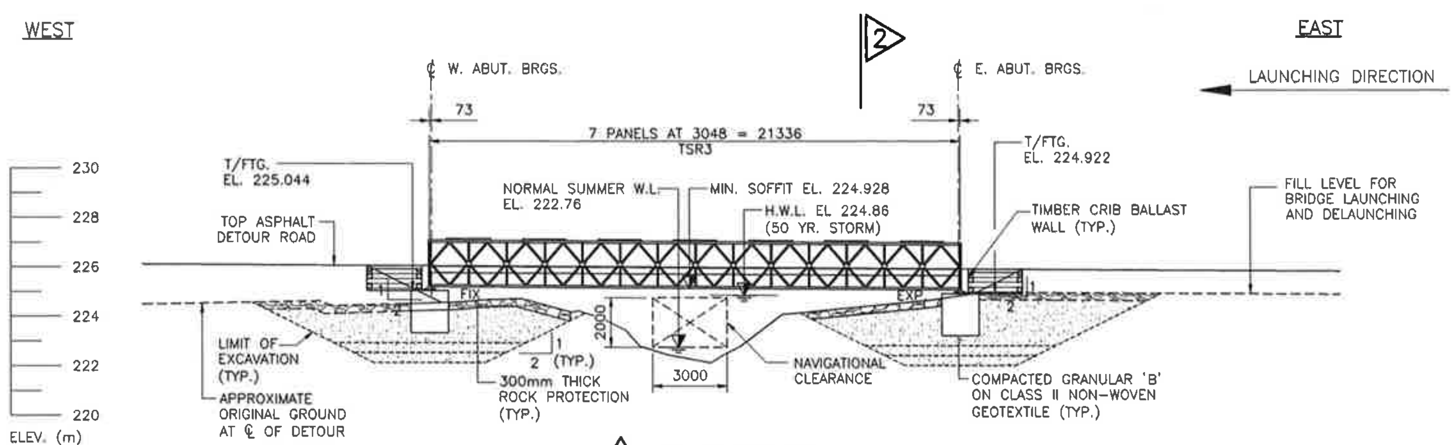

ELEVATION
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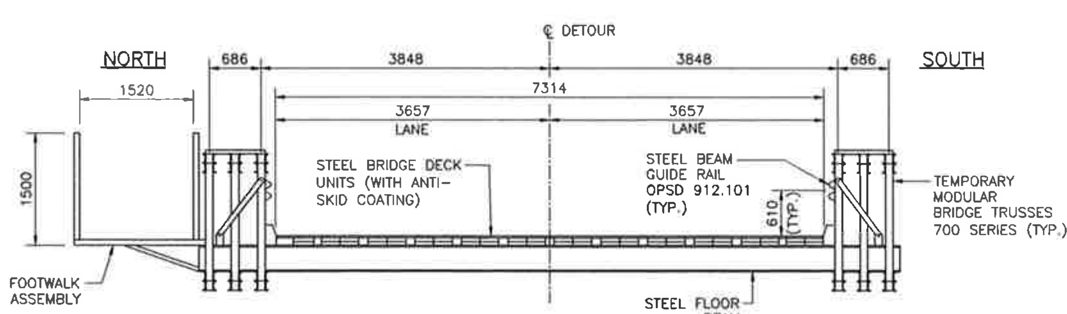
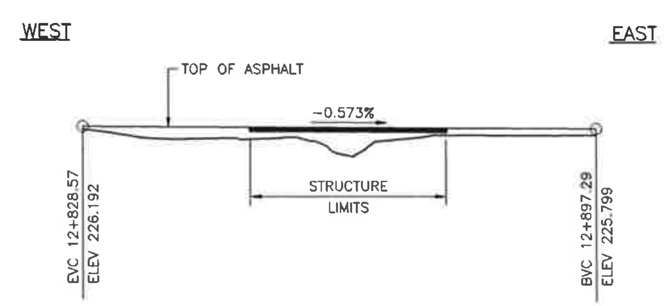


2
1:50



DETOUR PROFILE OF HWY 89
N.T.S.

LICENSED PROFESSIONAL ENGINEER J.S. YUN 100079999 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER C.L. BOND 100122728 PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

GENERAL NOTES

1. DESIGN OF SUBSTRUCTURE IS IN ACCORDANCE WITH THE CANADIAN HIGHWAY BRIDGE DESIGN CODE 2006 (CAN/CSA-S6-06)
2. THE TEMPORARY MODULAR BRIDGE SUPERSTRUCTURE COMPONENTS SHALL BE ACROW PANEL 700 SERIES AND SHALL BE SUPPLIED BY MTO AND SHALL REMAIN THE PROPERTY OF MTO.
3. THE CONTRACTOR SHALL BE RESPONSIBLE FOR CONSTRUCTION AND REMOVAL OF THE TEMPORARY MODULAR BRIDGE SUBSTRUCTURE, AND FOR TRANSPORTATION, ASSEMBLY, LAUNCHING, REMOVAL, DISASSEMBLY AND RETURN TO MTO OF THE TEMPORARY MODULAR BRIDGE SUPERSTRUCTURE COMPONENTS.
4. CONTRACTOR TO CONFIRM ALL BEARING LOCATIONS & ELEVATIONS AND ADJUST THE LAYOUT OR ELEVATION AS REQUIRED.
5. CLASS OF CONCRETE 30MPa
6. REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
7. UNLESS SHOWN OTHERWISE TENSION LAP SPLICES SHALL BE CLASS B
8. CLEAR COVER TO REINFORCING STEEL 70±20mm

ERECTION & LAUNCHING

1. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE ERECTION, LAUNCHING AND DELAUNCHING OF THE STRUCTURE.
2. LAUNCHING OF THE MODULAR BRIDGE SHALL NOT COMMENCE UNTIL CONCRETE IN THE TEMPORARY ABUTMENTS HAS REACHED 30 MPaSTRENGTH.
3. (i) THE CONTRACTOR SHALL SUBMIT WORKING DRAWINGS WHICH DESCRIBE THE METHOD OF ERECTION, LAUNCH AND DELAUNCH AND SHOW THE LAYOUT OF THE LAUNCHING AND CONSTRUCTION ROLLERS.
   (ii) THE TOPS OF THE ROLLERS SHALL BE AT THE SAME ELEVATION, UNLESS SPECIFIED OTHERWISE ON DRAWINGS.
   (iii) THE TOPS OF THE ROLLERS SHALL BE LEVELLED ACROSS IN PAIRS AT RIGHT ANGLES TO THE CENTRE LINE OF THE STRUCTURE.
   (iv) LAUNCHING ROLLERS AND RECEIVING ROLLERS TO BE LOCATED AT THE CENTRE LINE OF BEARING LOCATIONS.
4. (i) ALL PINS, BOLTS AND THREADED PARTS MUST BE FREE OF DIRT AND BE LUBRICATED AT THE TIME OF INSTALLATION.
   (ii) TRANSOM CLAMP TIGHTENING BARS MUST BE WIRED TO THE PANEL VERTICALS. SWAY BRACES SHALL BE FULLY TIGHTENED TO GAUGE BLOCKS AND ALL LOCK NUTS BE SECURED.
   (iii) ALL PANEL PINS ON STRUCTURE MUST BE KEYED.
5. THE DECK UNITS SHALL BE INSTALLED BEFORE LAUNCHING.

MAINTENANCE

1. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE MAINTENANCE OF THE TEMPORARY MODULAR BRIDGE AND APPROACHES, INCLUDING THE FOLLOWING:
   (i) CHECK THAT ALL BEARING BOLTS, CHORD BOLTS AND TRANSOM CLAMPS ARE AND REMAIN FULLY TIGHTENED.
   (ii) KEEP BASE PLATES AND BEARINGS FREE OF DEBRIS, INSPECT BASE PLATES AND ABUTMENTS PERIODICALLY AND CORRECT ANY UNEVEN SETTLEMENT TO THE SATISFACTION OF THE CONTRACT ADMINISTRATOR.
   (iii) NOTIFY THE CONTRACT ADMINISTRATOR IMMEDIATELY OF ANY DAMAGE TO THE BRIDGE OR SUPPORTS.

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. TEMPORARY ABUTMENTS
3. STANDARD DETAILS

APPLICABLE STANDARD DRAWINGS

OPSD 972.130 FENCE, CHAIN-LINK INSTALLATION – ROADWAY

REVISIONS

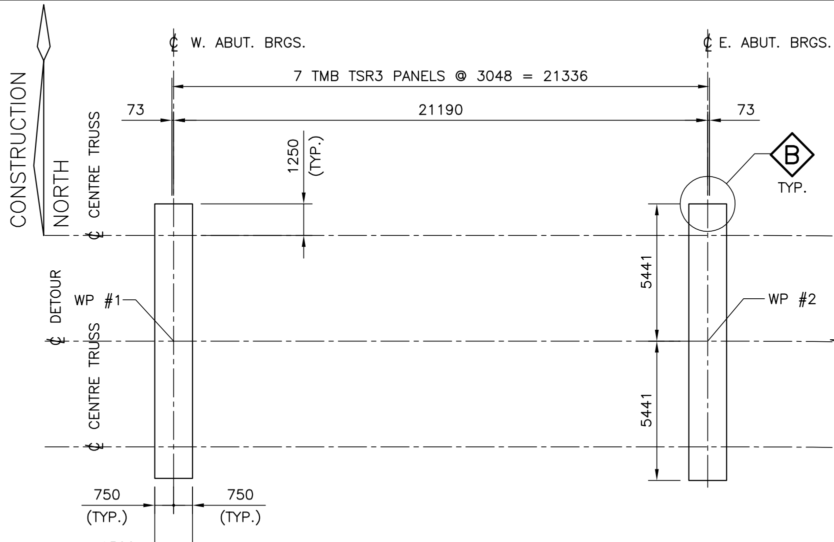
			DESCRIPTION		
DESIGN AY	CHK BB	CODE CHBDC-06	LOAD CL-625-ONT	DATE FEB/14	
DRAWN WA	CHK AY	SITE 30-254	STRUCT	SCHEME	DWG 17

CAD FILE LOCATION AND NAME: n:\2012\32\3212093\320 INNISFIL\S3212093-320-017MB.dwg
MODIFIED 2/7/2014 9:33:48 AM BY VILLASENORD
DATE PLOTTED 2/13/2014 11:33:44 AM BY: DANI VILLASENOR

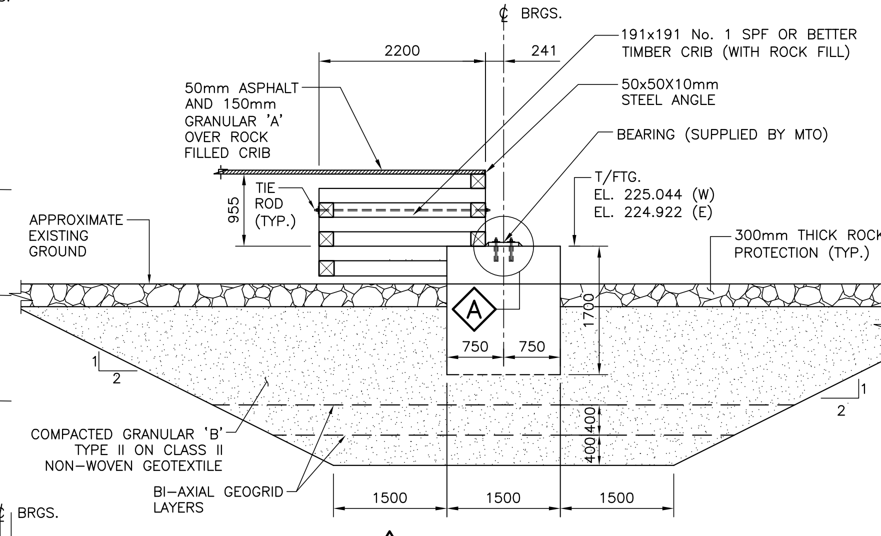
MINISTRY OF TRANSPORTATION, ONTARIO PH-D-707 88-05

WORKING POINT DATA			
LOCATION	STATION	NORTHING	EASTING
WEST ABUT.	12+853.489	4895316.780	291399.617
EAST ABUT.	12+746.679	4895322.991	291419.877

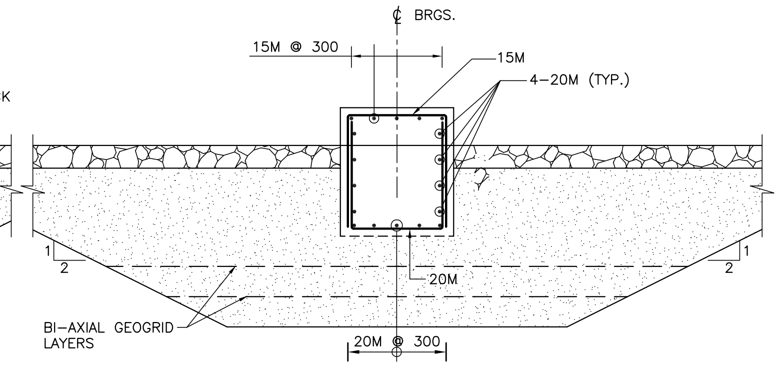
DISTRICT CONT. No. WP No. 2108-11-00	
HWY 89 DETOUR INNISFIL CREEK	SHEET
TEMPORARY MODULAR BRIDGE ABUTMENTS	S23
MRC McCORMICK RANKIN A member of MMM GROUP	METRIC



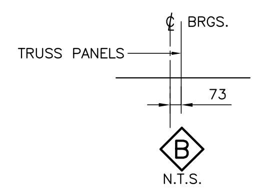
FOUNDATION LAYOUT
1:150

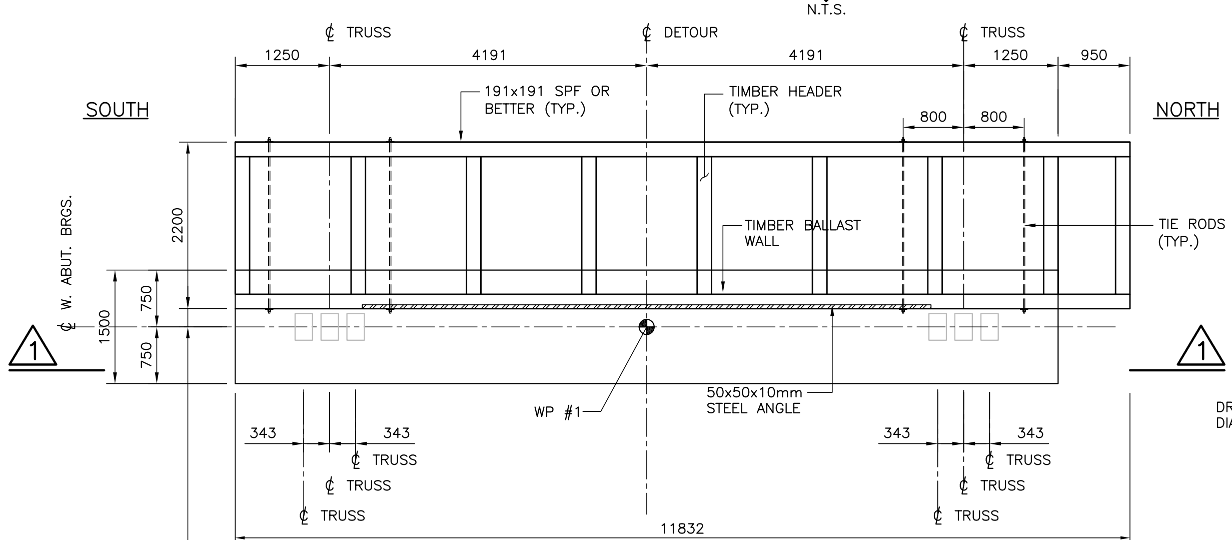


2 DIMENSIONS
1:50



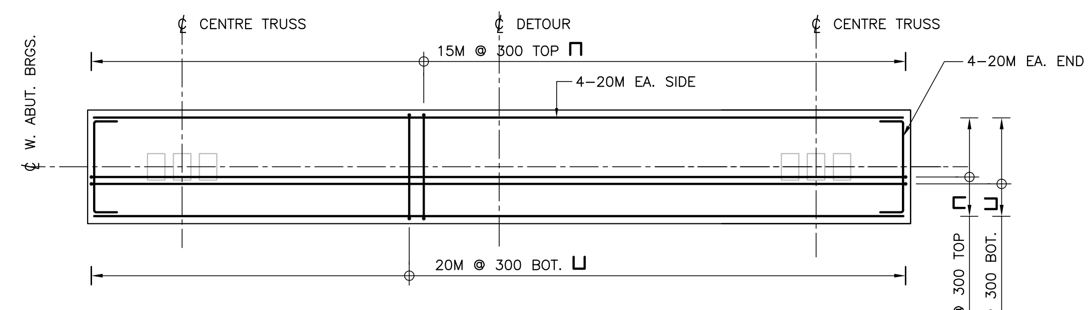
2 REINFORCING
1:50





PLAN
1:50

(W. ABUT. SHOWN, E. ABUT. SIMILAR)



PLAN REINFORCING
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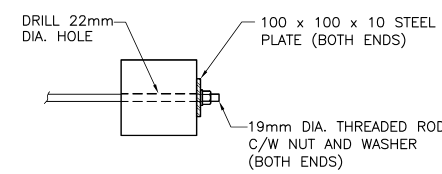
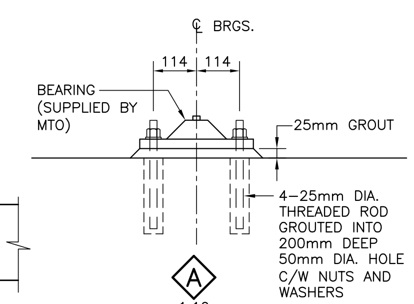


TIE ROD DETAIL
1 : 10

(24 LOCATIONS PER ABUTMENT)



TIMBER CRIB NOTES:

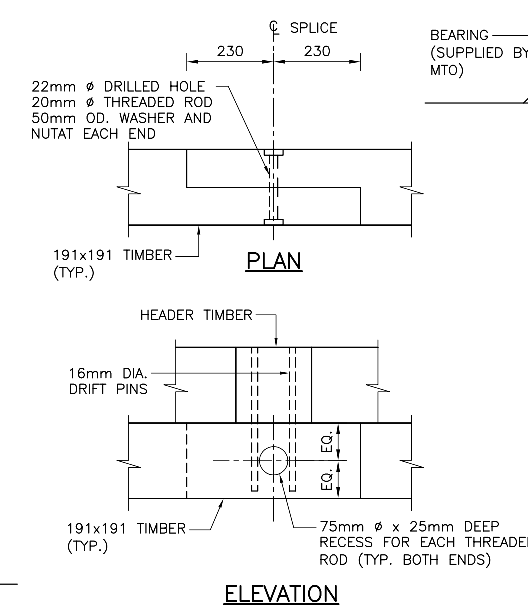
1. OPSS 907 SHALL GOVERN THE CONSTRUCTION OF THE ROCK FILLED TIMBER CRIBS FOR THE ABUTMENTS.
2. CARE SHALL BE EXERCISED DURING THE INSTALLATION OF THE CRIB WALLS TO ENSURE EACH FACE IS VERTICAL AND PLUMB AND THAT EACH TIER IS SET HORIZONTAL.
3. WARPED OR TWISTED TIMBERS SHALL BE DISCARDED OR USED AS HEADERS BUT SHALL NOT BE INCORPORATED AS STRETCHERS IN THE CRIB CONSTRUCTION.
4. HEADERS SHALL BE SPACED AS SHOWN ON THE DRAWINGS OR AS MAY OTHERWISE BE APPROVED BY CONTRACT ADMINISTRATOR.
5. EACH SUCCESSIVE TIER SHALL BE DRIFT BOLTED TO THE ONE UPON WHICH IT RESTS BY DRIFT PINS NOT LESS THAN 16mm DIAMETER AND OF SUFFICIENT LENGTH TO EXTEND THROUGH TWO TIERS. DRIFT BOLTS SHALL BE STAGGERED AND NOT MORE THAN 2.5m CENTRE TO CENTRE IN EACH TIER.
6. ALL END JOINTS SHALL BE LAPPED AND DRIFT PINNED IN TWO LOCATIONS. HEADERS SHALL BE DRIFT PINNED TO FACE TIMBERS (STRETCHERS) IN A SIMILAR MANNER.
7. DRIFT PINS AND OTHER HARDWARE DO NOT HAVE TO BE GALVANIZED.
8. TIMBER SPLICES SHALL BE BOLTED AS SHOWN ON THE DRAWINGS.
9. THE FILLING OF THE INTERIOR OF THE CRIB SHALL FOLLOW CLOSELY THE ERECTION OF THE SUCCESSIVE TIERS OF TIMBER AND SHALL NOT AT ANY TIME BE MORE THAN 1000mm BELOW THE HEIGHT OF THE TIMBER CONSTRUCTION.
10. COMPACTION SHALL BE ONLY WITH A PLATE TAMPER. CARE IS TO BE TAKEN NOT TO CAUSE SUCH INTERNAL FORCES DURING COMPACTION TO RESULT IN SPLITTING OF ANY OF THE HEADERS. THE CONTRACTOR SHALL INSPECT THE ENDS OF THE HEADERS DURING COMPACTION.
11. ASSUMED DIMENSION FROM TOP OF STEEL DECK TO UNDERSIDE OF STEEL BEARING PLATE IS 981mm. CONTRACTOR SHALL ADJUST FOOTING ELEVATIONS TO SUIT ANY VARIANCE OF THIS DIMENSION.

LEGEND

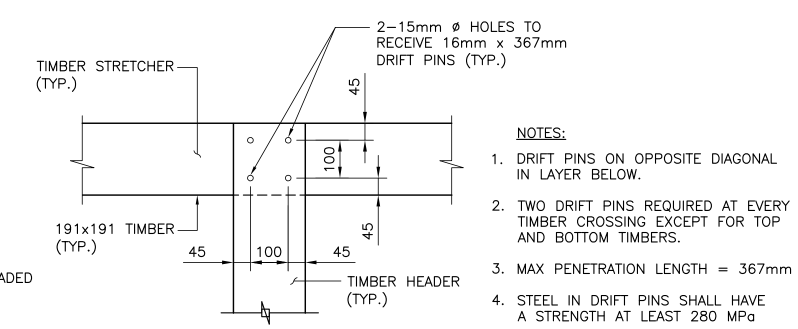
TMB DENOTES TEMPORARY MODULAR BRIDGE

FOUNDATION NOTES:

1. SUBGRADE SHALL BE INSPECTED AND APPROVED BY QUALITY VERIFICATION ENGINEER PRIOR TO PLACING GRANULAR BACKFILL.
2. CONTRACTOR SHALL SUB-EXCAVATE TO THE EXTENT DIRECTED BY THE QUALITY VERIFICATION ENGINEER.
3. GRANULAR BACKFILL SHALL BE COMPACTED TO 100% STANDARD PROCTOR MAXIMUM DRY DENSITY AT A MOISTURE CONTENT WITHIN 2% OF OPTIMUM.



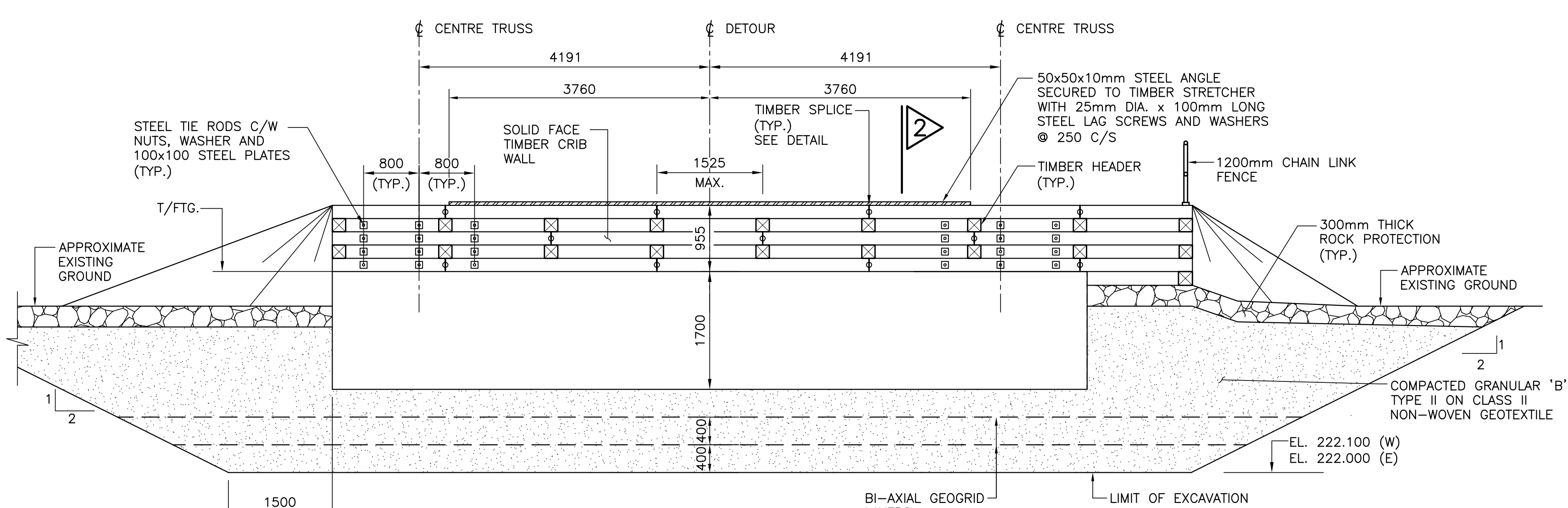
TIMBER SPLICE DETAIL
1 : 10



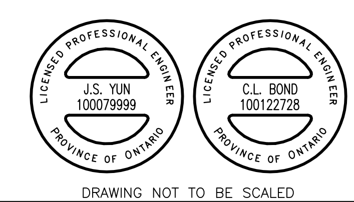
NOTES:

1. DRIFT PINS ON OPPOSITE DIAGONAL IN LAYER BELOW.
2. TWO DRIFT PINS REQUIRED AT EVERY TIMBER CROSSING EXCEPT FOR TOP AND BOTTOM TIMBERS.
3. MAX PENETRATION LENGTH = 367mm
4. STEEL IN DRIFT PINS SHALL HAVE A STRENGTH AT LEAST 280 MPa

TIMBER CONNECTION ARRANGEMENT
1 : 10

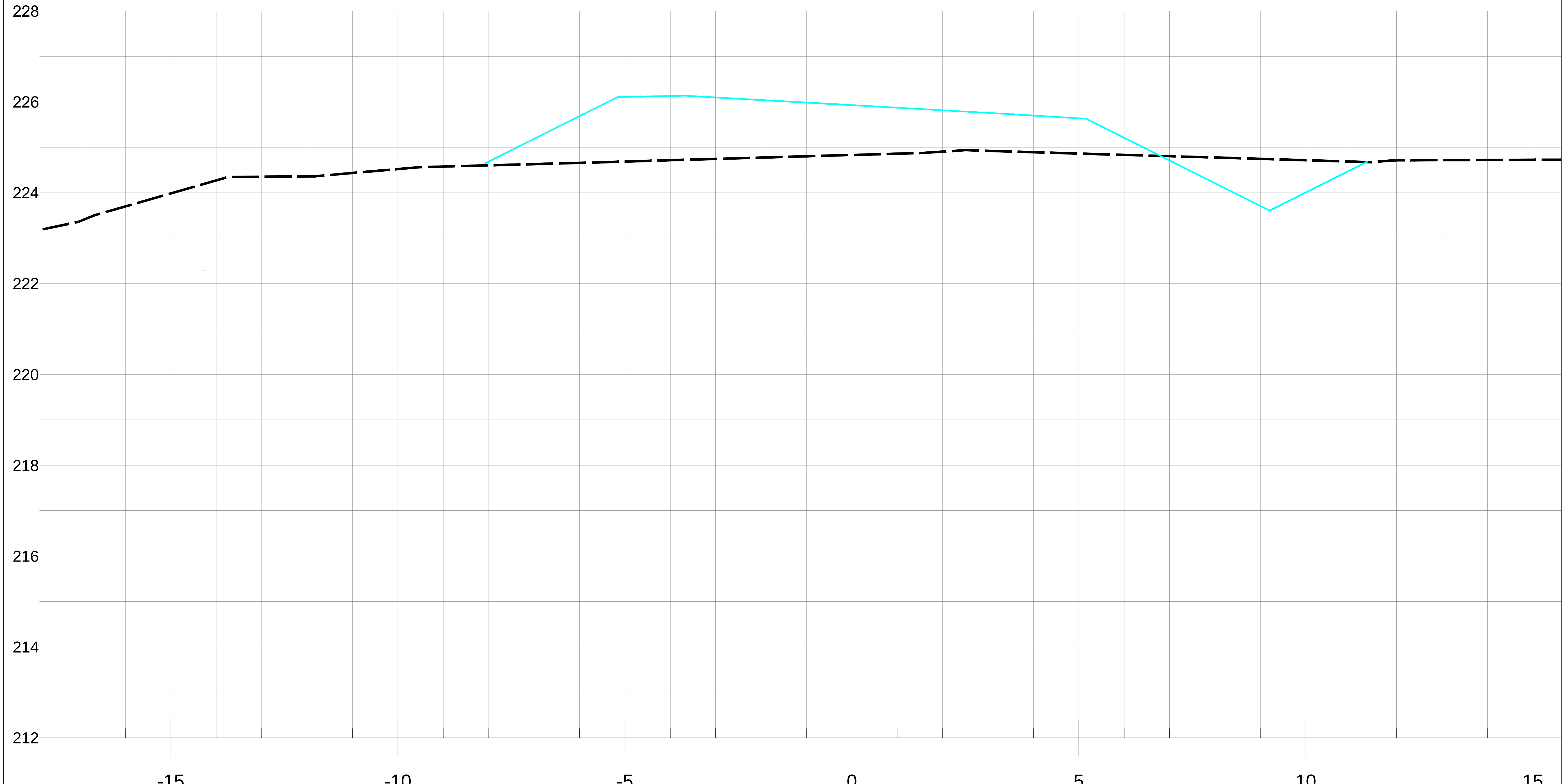


1 ELEVATION
1:50

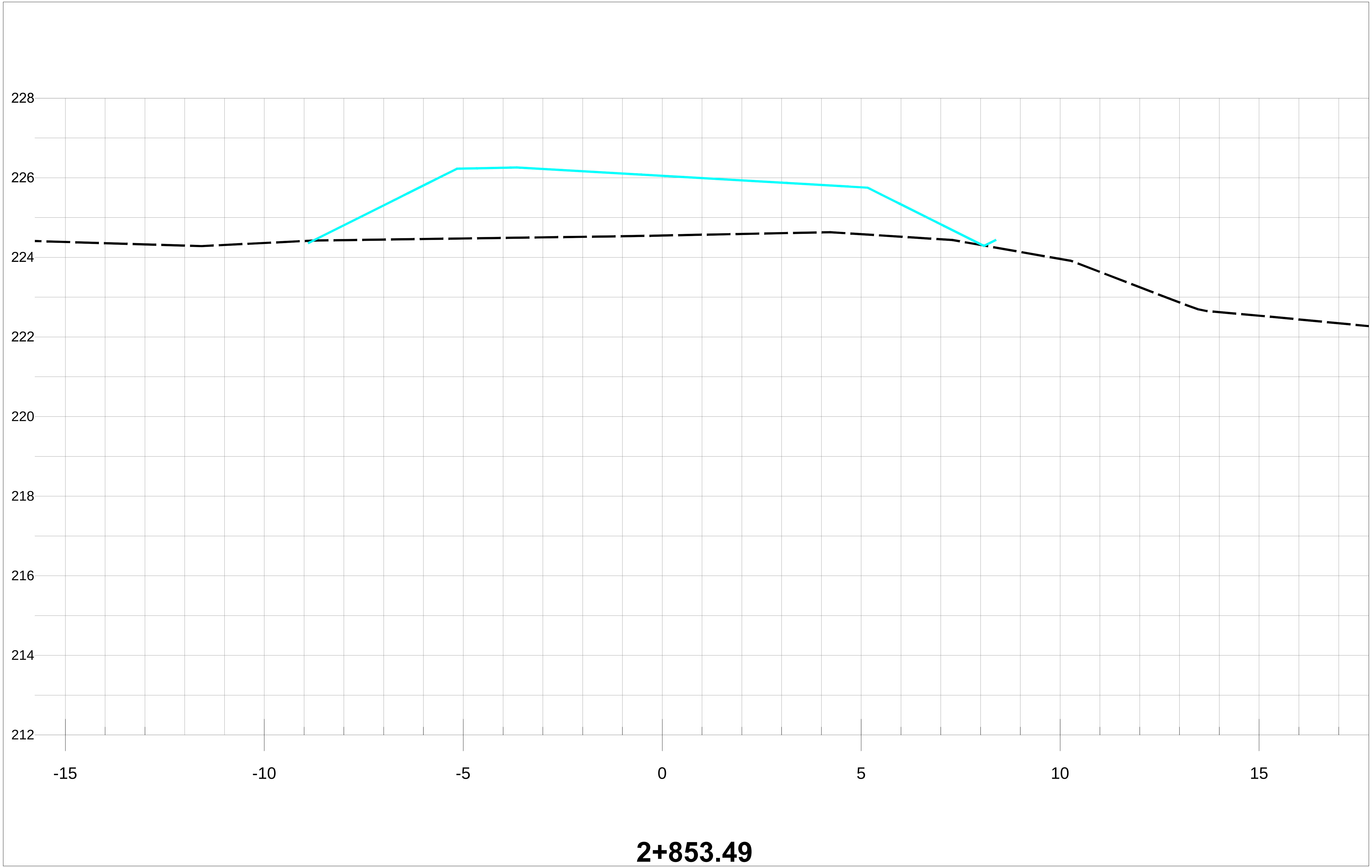


DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS				
DESCRIPTION				
DESIGN AY	CHK BB	CODE CHBDC-06	LOAD CL-625-ONT	DATE MAR/14
DRAWN DPV	CHK AY	SITE 30-254	STRUCT	SCHEME / DWG 4



2+874.68


228
226
224
222
220
218
216
214
212
-15
-10
-5
0
5
10
15
2+853.49

Appendix H

Advantages, Disadvantages, Costs and Risks/Consequences of Foundation Alternatives

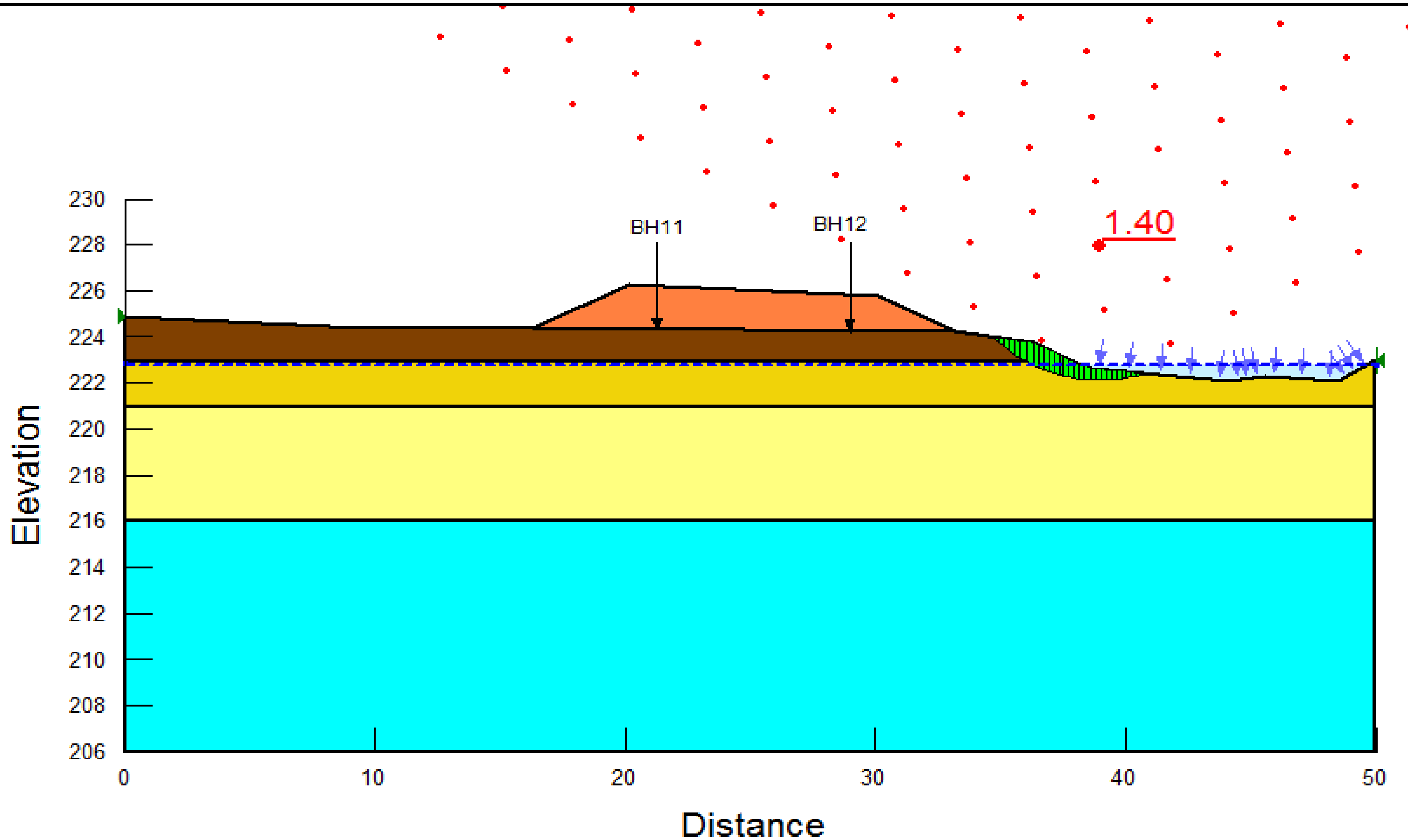
Table H-1

Foundation Options for New Bridge Over Innisfil Creek

Foundation Type	**Advantage/ Disadvantage**	**Risks/Consequences**	**Relative Costs**	**Recommendations**
Shallow foundations	-Low cost -Sheet pile cofferdam may be required depending on the extent of excavation	- Large settlement - High groundwater - Organic rich soils	Low to moderate cost	-Feasible for the proposed temporary structure
Helical Piles	-No dewatering is required -Minimal excavation -No excessive cuttings -Small equipment	-New to MTO work	Low to moderate cost	-Feasible for the proposed temporary structure
Timber Piles	-Pile driving equipment is required		higher cost in comparison with shallow foundations and helical piers	-Not recommended based on cost

Appendix I

Slope Stability Analyses Results



Section / Location	:	2+853 (south)
Static/Seismic	:	static
Drained Condition	:	undrained
GWT	:	El. 222.8 m
(assumed as a normal operational water level)		
Analysis Method	:	Morgenstern - Price

Stratum	γ (kN/m^3)	c (kPa)	ϕ (^{o})
New Embankment Fill	20.5	0	32
Existing Fill	18.5	0	27
Upper Si-Sa/Sa-Si (upper)	17.5	0	27
Upper Si-Sa/Sa-Si (lower)	18.0	0	28
Upper Si-Cl	18.5	80	0

coffey

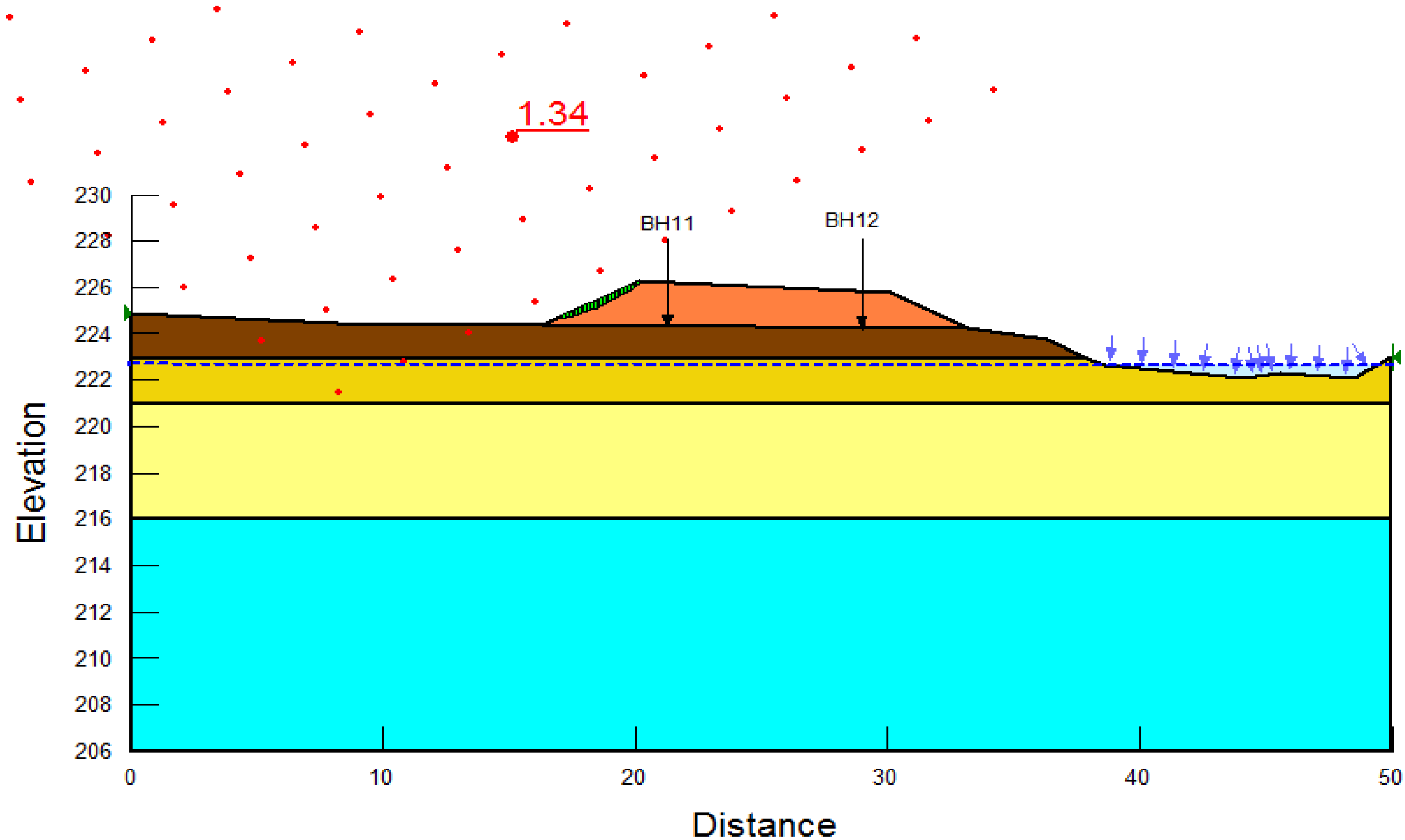
SLOPE STABILITY ANALYSIS

Innisfil Creek Temporary Bridge Approach Embankment

PROJECT NO.:	TRANETOB20462AA	DATE:	Jan, 2014
Analyzed by	GR	Reviewed by	ZO

HIGHWAY 89

FIGURE I-1



Section / Location	:	2+853 (north)
Static/Seismic	:	static
Drained Condition	:	undrained
GWT	:	El. 222.8 m
(assumed as a normal operational water level)		
Analysis Method	:	Morgenstern - Price

Stratum	γ (kN/m^3)	c (kPa)	ϕ (^{o})
New Embankment Fill	20.5	0	32
Existing Fill	18.5	0	27
Upper Si-Sa/Sa-Si (upper)	17.5	0	27
Upper Si-Sa/Sa-Si (lower)	18.0	0	28
Upper Si-Cl	18.5	80	0

coffey

SLOPE STABILITY ANALYSIS

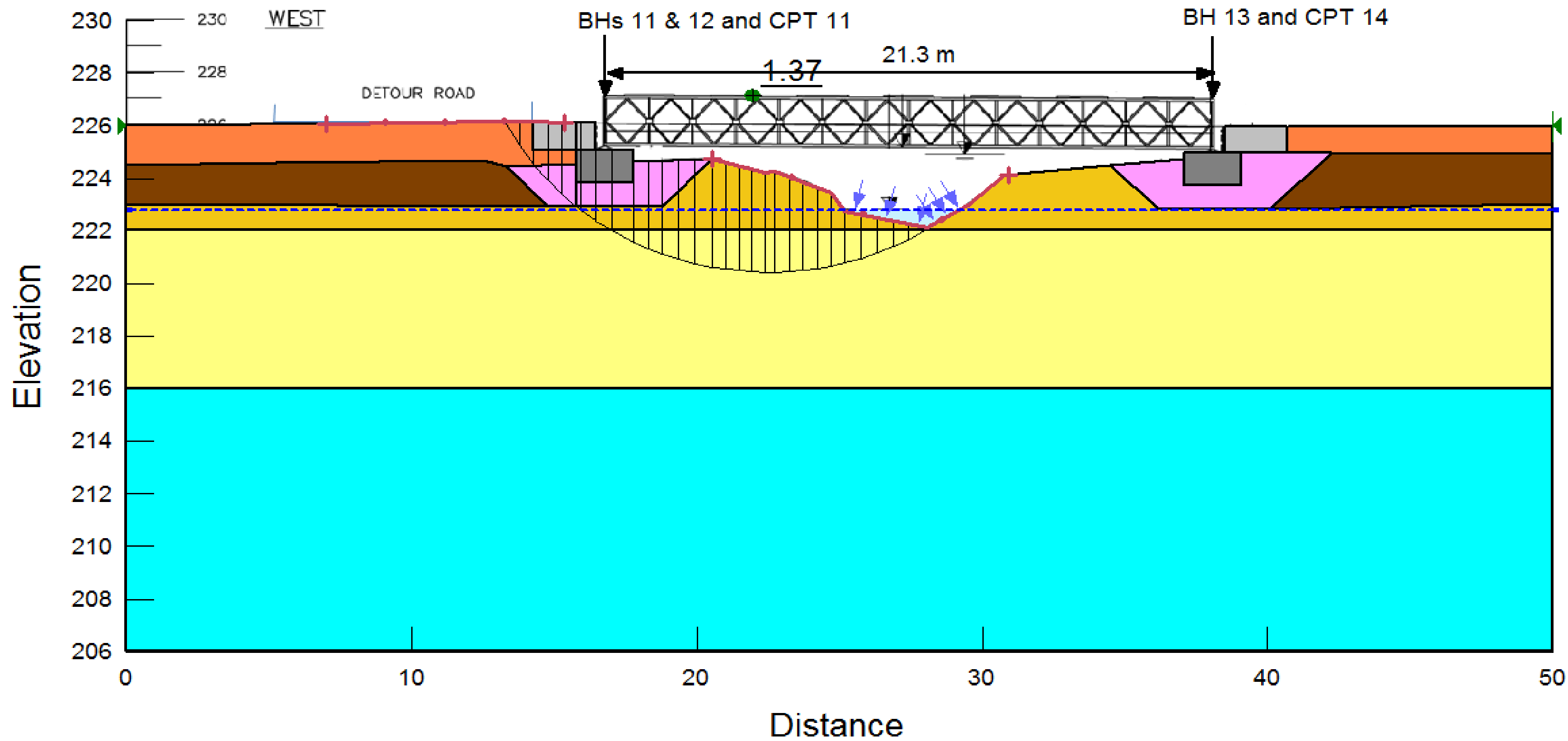
Innisfil Creek Temporary Bridge Approach Embankment

PROJECT NO.:	TRANETOB20462AA	DATE:	Jan, 2014
Analyzed by	GR	Reviewed by	ZO

HIGHWAY 89

FIGURE I-2

based on the GA drawing width of the concrete block is 2 m and length will be at least 9 m
(thickness is about 1.2 m-typical foundation thickness)



Section / Location	:	Profile (West)
Static/Seismic	:	static
Drained Condition	:	undrained
GWT	:	El. 222.8 m
(assumed as a normal operational water level)		
Analysis Method	:	Morgenstern - Price

*Concrete grade beam with structural loading was considered
**Failure surface can not intercept crib wall and foundation

Stratum	γ (kN/m^3)	c (kPa)	ϕ (^{o})
New Embankment Fill	20.5	0	32
Existing Fill	18.5	0	27
Upper Si-Sa/Sa-Si (upper)	17.5	0	27
Upper Si-Sa/Sa-Si (lower)	18.0	0	28
Upper Si-Cl	18.5	80	0
Granular Pad	21.0	0	34

coffey

SLOPE STABILITY ANALYSIS

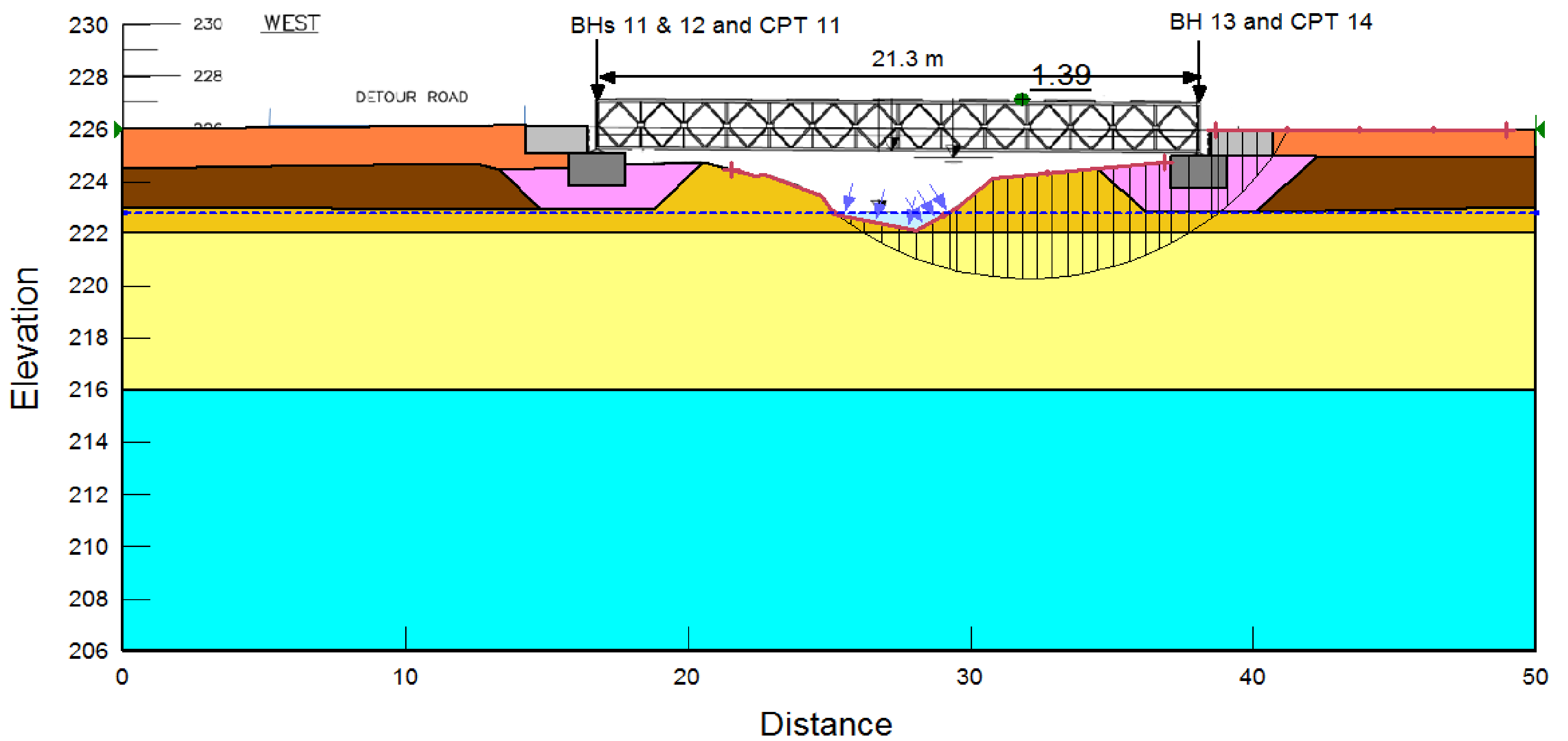
Innisfil Creek Temporary Bridge Forward Slope

PROJECT NO.:	TRANETOB20462AA	DATE:	Jan, 2014
Analyzed by	GR	Reviewed by	ZO

HIGHWAY 89

FIGURE I-3

based on the GA drawing width of the concrete block is 2 m and length will be at least 9 m
(thickness is about 1.2 m-typical foundation thickness)



Section / Location	: Profile (East)
Static/Seismic	: static
Drained Condition	: undrained
GWT	: El. 222.8 m
(assumed as a normal operational water level)	
Analysis Method	: Morgenstern - Price

*Concrete grade beam with structural loading was considered
**Failure surface can not intercept crib wall and foundation

Stratum	γ (kN/m^3)	c (kPa)	φ (^{o})
New Embankment Fill	20.5	0	32
Existing Fill	18.5	0	27
Upper Si-Sa/Sa-Si (upper)	17.5	0	27
Upper Si-Sa/Sa-Si (lower)	18.0	0	28
Upper Si-Cl	18.5	80	0
Granular Pad	21.0	0	34

coffey

SLOPE STABILITY ANALYSIS

Innisfil Creek Temporary Bridge Forward Slope

PROJECT NO.:	TRANETOB20462AA	DATE:	Jan, 2014
Analyzed by	GR	Reviewed by	ZO

HIGHWAY 89

FIGURE I-4

Appendix J

List of OPSS, OPSD and Non-standard Specifications

List of OPSDs, OPSSs and Non-standard Specifications

OPSDs

OPSD 208.01 Benching of Earth Slopes

OPSSs

OPSS206 - Construction Specification for Grading

OPSS212 - Construction Specification for Borrowing

OPSS 501 - Construction Specification for Compacting

OPSS 803 - Construction Specification for Sodding

OPSS804 - Construction Specification for Seed and Cover

OPSS 902 – Construction Specification for Excavating and Backfilling-Structures

OPSS 903 – Construction Specification for Deep Foundations

NSSP Wording

Vibration Monitoring

The vibration monitoring equipment shall be placed on the site during construction.

Impact of construction vibration on the temporary bridge structure should also be assessed during construction of new highway 89 bridge.

The Contractor shall take readings during the construction. The results shall be submitted to the Contract Administrator frequently.

If the readings are beyond the criteria, the Contractor must alter his/her construction procedures until the vibrations are within the acceptable ranges.

Appendix K

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Coffey at the time of preparation. Unless otherwise agreed in writing by Coffey, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

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