

McCormick Rankin

Foundation Investigation and Design Report

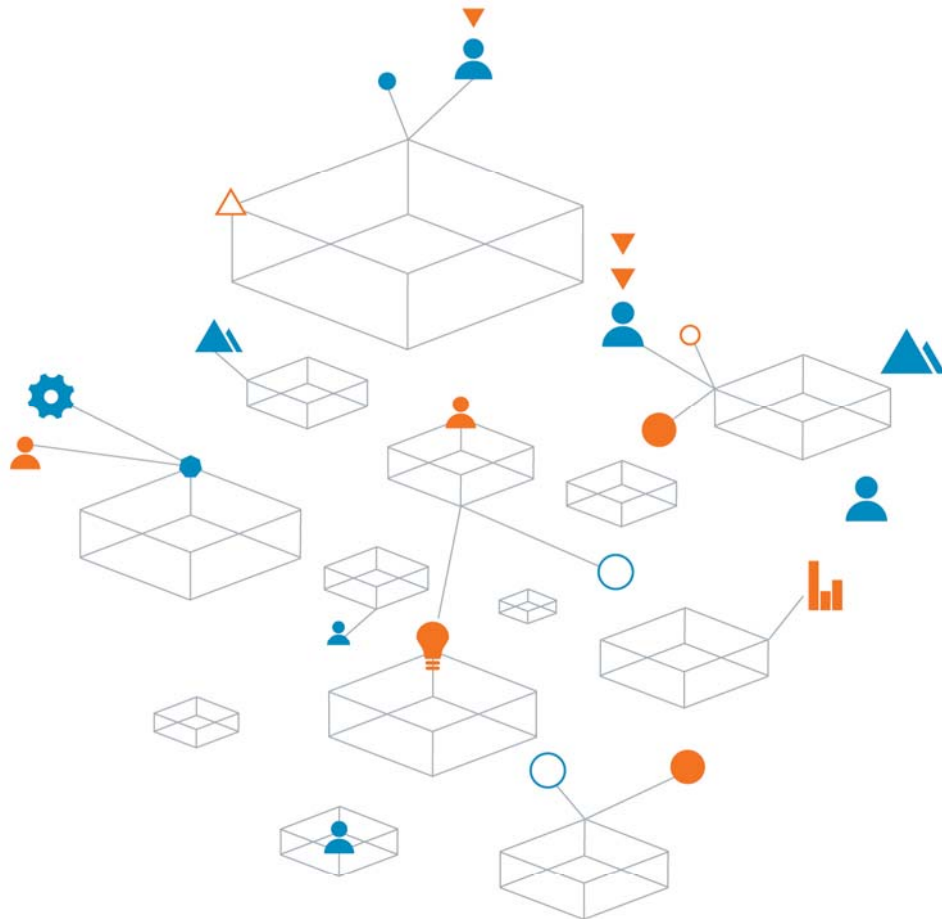
Replacement of Glass's Bridge over Innisfil Creek

Highway 89, Site No. 30-254, Town of Innisfil,

MTO Central Region, W.P. 2108-11-00, GEOCRES NO. 31D-572

TRANETOB20462AA

25 August 2014



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projects



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25 August, 2014

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

Dear Mr. Hui:

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

Please find attached our foundation investigation and design reports for the above noted site.

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

For and on behalf of Coffey.

A handwritten signature in blue ink, appearing to read "G. Roh", written over a horizontal line.

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

McCormick Rankin

Foundation Investigation Report

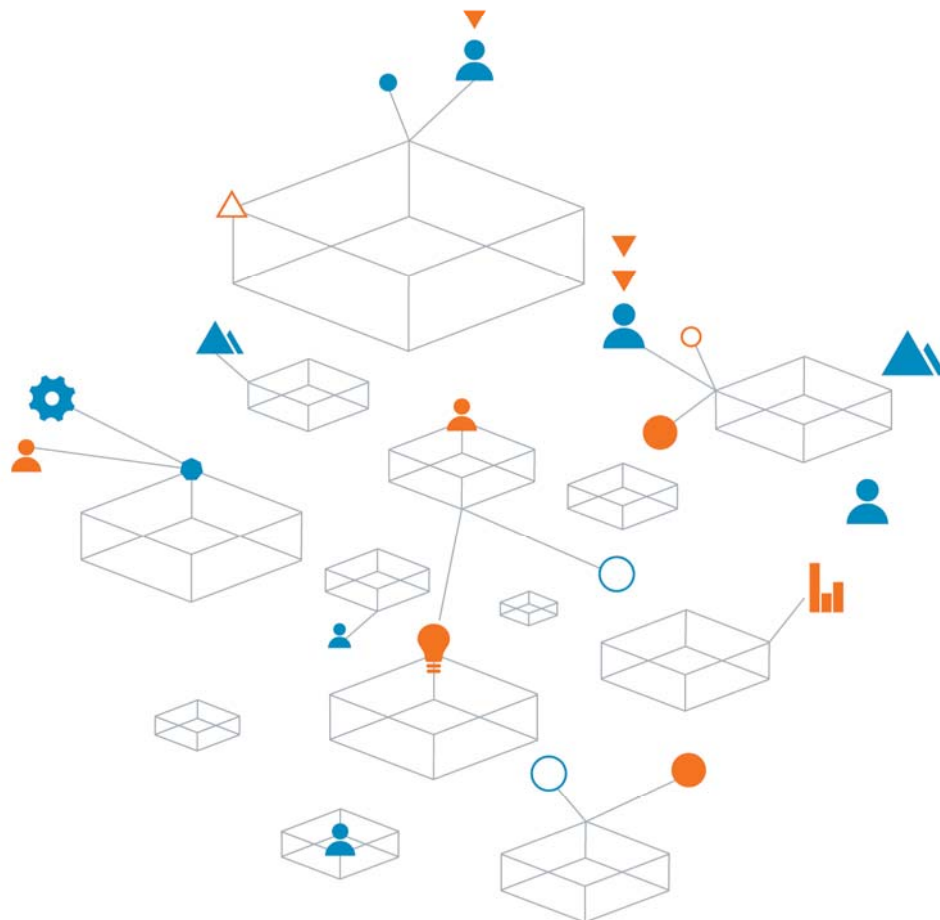
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**FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF GLASS'S BRIDGE OVER THE INNISFIL CREEK
SITE NO. 30-254, HIGHWAY 89, TOWN OF INNISFIL
MTO CENTRAL REGION, W.P. 2108-11-00, GEOCRES 31D-572**

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.

The existing bridge is a single span T-beam concrete structure, about 11 m long and 10.7 m wide. It is believed to have been built in the early 1900's and is supported on spread footings. Erosion control works included the installation of interlocking steel sheet piling in the northwest corner. Based on hydrological considerations, it is proposed to raise the new bridge deck level by 1.6 m. The replacement bridge substructure will be built wider towards the north to accommodate future deck widening, when needed, to accommodate a 4-lane Highway 89. The centreline of Highway 89 will be shifted by about a lane width towards the north, resulting in the construction of the replacement bridge about 4 m north of its current position.

During new bridge construction, Highway 89 traffic will be diverted to a proposed temporary bridge structure, which will be located about 30 m north of the existing highway centreline. A foundation investigation report for the proposed temporary bridge is being submitted under separate cover.

The purpose of this investigation was to determine the subsurface and groundwater conditions at the proposed new bridge location by means of boreholes and soundings, and to determine the engineering characteristics of the overburden soils by means of field and laboratory tests.

The findings of the investigation are presented in this report.

2 SITE DESCRIPTION AND GEOLOGY

An MTO Patrol Yard is located about 150 m north-west of the present bridge location, on the north side of Highway 89. The area surrounding the existing bridge is rural. The topography is flat in the immediate vicinity of the existing bridge, 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 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 INVESTIGATION PROCEDURES

The field investigation consisted of laying out boreholes and sounding locations by reference to the alignment 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 below 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/Sounding Locations and Depths

BH / CPT No.	Location	Final Depth (m)	Comments
BH1	W. Abutment, N. side	37.2	Dynamic cone penetration test (DCPT) from borehole bottom to 41.5 m. Nearby DCPT to 22 m.
BH2	W. Abutment, near C/L	31.1	drilled from highway pavement
BH3	W. Abutment, S. side	47.9	Piezometer
CPT4	E. Abutment, N. side	23.0	-
BH5	E. Abutment, N. side shoulder	29.3	Piezometer
BH6	E. Abutment, S. side	30.7	drilled from highway pavement
BH7	W. Approach, N. side	11.3	-
BH8	W. Approach, S. side	11.3	drilled from highway pavement
BH9	E. Approach, N. side	11.3	-
BH10	E. Approach, S. side	11.3	drilled from highway pavement

Samples were taken at 0.76 m to 1.5 m depth intervals down to the 15 m depth and at 3 m or lesser intervals of depth 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 obtained 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 cm², a friction sleeve area of 150 cm² and a porewater pressure u₂ 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 on the laboratory reports in Appendix C.

4 SUBSURFACE CONDITIONS

4.1 General

Leaving aside the pavement structure (hot mix asphaltic concrete or HMA and underlying base and subbase granular layers), the subsurface stratigraphy consists of a surficial deposit of topsoil and organic rich silty sand and detritus fill followed by loose to compact silty sand overlying a very thick deposit of silty clay over very dense silt. The silt deposit is underlain by very stiff to hard clayey silt over a glacial till soil that was not fully penetrated. Bedrock was not encountered to the maximum depth of exploration of 48 m.

4.2 Topsoil

The topsoil thickness ranged between 200 mm and 600 mm, being thicker in the low lying areas and within drainage swales and ditches.

4.3 Pavement

The pavement structure consists of 200-250 mm thickness of HMA underlain by 200-450 mm granular base sand and gravel over 250-300 mm of a gravelly sand subbase over subgrade.

4.3 Embankment Fill

Boreholes BH2, BH6, BH8, and BH10, drilled from the existing Highway 89 pavement and Borehole BH7 from the existing highway shoulder contacted 1.3 to 3.1 m thick silty sand to sandy silt, and sand fill materials mostly below the pavement fill. Trace to some gravel and clay and trace organic were also present within this fill layer. This fill was found to be extended to 2.2 m to 3.8 m below the ground surface. The basal portion of this fill may be mixed with natural pre-filling surficial topsoil.

The SPT N-values ranged randomly from lows of 2 blows/0.3 m to highs of 26 blows/0.3m. On this basis, the non-cohesive fill under the highway (at Boreholes BH2, BH6, BH7, BH8 and BH10) is considered to be very loose to compact.

4.4 Surficial Deposits

Surficial deposits beyond the existing highway pavement and shoulders consist of detritus fill material with organic inclusions. The surficial deposit is essentially 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 3 m. SPT N values of 0-4 blows /0.3 m indicate the deposit is generally very loose to loose. 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 80 percent. In other samples that contained organics, the natural moisture content was generally about 30-35 percent. Gradation analysis was not performed on samples

recovered from this surficial deposit, as the samples showed wide variations in gradation characteristics and organic content, even within one sample itself.

4.5 Silty Sand

The surficial deposits beyond the highway cross-section 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 of between 0 and over 30 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. Gradation test results are shown in Figures C-1 to C-3, Appendix C. The range in grain size distribution was as follows]:

Gravel:	0-2 %
Sand:	73-90 %
Silt	5-17 %
Clay sized	5-10 %

Figure C-2 presents a sample from a coarser layer, with 16% gravel, 67% sand and 17% silt & clay sized particles.

The grain size distribution for six silty fine sand to sandy silt samples is given in Figure C-3. The gradation range is shown below:

Gravel:	0-2 %
Sand:	28-58 %
Silt:	30-56 %
Clay sized:	9-14 %

The results of Atterberg limits test performed on a clayey silt to silt (ML) stringer (BH3 SS9) is given in Figure C-4.

Liquid Limit:	14 %
Plastic Limit:	11 %
Plasticity Index:	3 %

The moisture content of specimens selected from the various split-spoon samples varied greatly, ranging from below 15 percent to over 50 percent. The higher moisture content values were always associated with the presence of decomposed vegetative organic matter, such as peat and decayed wood. As shown on the log sheets, the moisture content averages 20 percent throughout the deposit, excluding extreme values. For a saturated sand with a solids specific gravity of 2.65, the in situ void ratio is about 0.5. In the densest state, sand has a void ratio of 0.35, whereas very loose sand has a void ratio of about 0.8. On this basis, the deposit is considered to be generally compact.

4.6 Silty Clay

The silty sand stratum is underlain by a thick clayey silt to silty clay deposit that is present below depths of 8-11 m, or below elev. 214-217 m. The total thickness of this deposit ranged from 16 m to 19 m, suggesting

a base elevation of about 197-199 m. It is inferred from the surface elevations of this deposit that it has an easterly trending dip or slope gradient of about 2.6 percent, the surface of the deposit being higher in the west (BH 7) and lower in the east (BH 9 and 10).

The silty clay is grey. It contains frequent clayey silt to silt, and occasionally, highly plastic clay seam inclusions within the upper and lower 5 m of the overall thickness. The mid-portion of the deposit is generally massive and homogeneous in appearance.

The grain-size distribution of six samples from the silty clay deposit is given in Figure C-5, Appendix C. Test ranges are:

Gravel:	0 %
Sand:	0-4 %
Silt:	22-57 %
Clay sized	43-76 %

The results of Atterberg limits tests on 17 samples are given in Figure C-6 and are summarized below:

Liquid Limit:	9-56 %
Plastic Limit:	4-25 %
Plasticity Index:	5-31

The soil deposit is characterized as silty clay of low to medium plasticity (CL-CI). The natural moisture content of the tested samples lies generally between the plastic and liquid limits, yielding an average liquidity index of about 0.5, suggesting the deposit may be somewhat overconsolidated.

SPT N values ranged from 3 blows/0.3 m to 32 blows/0.3 m. The in-situ undrained shear strength (from field vane tests) ranged from 36 kPa to in excess of 200 kPa, indicating a firm to hard consistency, being mostly very stiff to hard. It is possible that the higher strength values (200 kPa +) resulted from testing in silt or sand seams that are prevalent randomly as stringers throughout this deposit, and as evidenced by the porewater pressure response plots in the CPT soundings (Appendix B).

Figure D-1 in Appendix D shows the variation of the in-situ undrained shear strength, s_u , against depth. Also plotted on Figure D-1 is the effective overburden stress (P'_o), and $0.23P'_o$ with depth. 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 over-consolidated.

Three odometer (one-dimensional consolidation) tests were performed on 75 mm thin wall tube samples. The test results are presented in Figures C-7 to C-9, Appendix C. They show a possible range of 200-300 kPa between the pre-consolidation pressure, P'_c and the existing effective overburden stress, P'_o , suggesting an OCR (overconsolidation ratio) of about 2-3.

The consolidation tests give average compression indices (C_c) of in the order of 0.1 to 0.2 and recompression indices (C_r) of about 0.03 to 0.05. These low values also confirm overconsolidation.

4.7 Silt

The massive silty clay deposit is underlain by a very dense silt deposit, the source of artesian water at this site. Table 4.7.1 summarizes deposit elevation data. 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.

Table 4.7.1 Silt Deposit Elevations

Borehole	Top Elevation (m)	Base Elevation (m)	Thickness (m)
BH1	197.4	192.4	5.0
BH2	198.6	unknown	unknown
BH3	199.3	191.3	8.0
BH5	198.8	unknown	unknown
BH6	198.4	unknown	unknown

The grain-size distribution of six samples from this stratum, Figure C-10, Appendix C, is summarized below:

Gravel:	0 %
Sand:	4-13 %
Silt:	81-89 %
Clay sized	4-7%

On the basis of SPT N values of 20-100+ blows/0.3 m, but generally over 50 blows/0.3 m, this stratum is considered to be dense to very dense. The lower N values are attributed to soil loosening under artesian head.

This silt stratum was the source of an artesian head of 3 m above ground level after being penetrated more than a metre by mud drilling methods in boreholes BH3 and BH5.

4.8 Clayey Silt to Silty Clay

A clayey silt deposit lies beneath the dense silt stratum. Borehole BH1 was terminated within this deposit at a depth of 37.2 m below the ground surface, or at elev. 187.2 m. In one location (BH3) the silty clay was underlain by a thin silt layer at a depth of 47.4 m, or at elev. 177.4 m.

The hydrometer analysis on two samples is given in Figure C-11 in Appendix C. The results show the following average gradation:

Gravel:	0 %
Sand:	0 %
Silt:	60-62 %
Clay sized	38-40 %

The results of Atterberg limits tests are given in Figure C-12 in Appendix C. The limit ranges are shown below:

Liquid Limit:	27-41 %
Plastic Limit:	16-19 %
Plasticity Index:	11-22

These results indicate clayey soils (basically cohesive soils) of low to intermediate plasticity (CI-CL). The natural moisture content was typically closer to the plastic limit, indicating over-consolidation.

SPT N values of 21-40 blows/0.3 m indicate a very stiff to hard consistency.

4.9 Glacial Till

A 0.2 m thick basal silt deposit overlying a clayey glacial till matrix was contacted at a depth of 47 m, or at about elev. 177 m. SPT N values were in excess of 100 blows/0.3 m suggesting the glacial till is very dense where non-cohesive and hard where cohesive.

5 GROUNDWATER CONDITIONS

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

Piezometers were installed in Boreholes BH3 and BH5 at depths of 32 m and 29 m respectively within the very dense silt deposit. A 3 m artesian head above the ground surface was measured in BH3 and BH5 by a pressure gauge placed on the piezometer standpipes. These two boreholes were decommissioned two to three days after piezometer installation. Decommissioning was accomplished by re-drilling the boreholes and grouting them in accordance with regulatory requirements.

The stream level in Innisfil Creek on September 20, 2013, was at elev. 222.8 m, representing the adjacent surface groundwater level. The 50 year flood level is said to reach elev. 224.9 m.

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 Highway 89 replacement bridge across Innisfil Creek. Please call us if you require assistance or need clarification.

For and on behalf of Coffey



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



Vasantha Wijeyakulasuriya, P. Eng.
Senior Principal



Cam Mirza, P. Eng.
MTO Designated Contact, Principal



Drawings

12+800

12+900

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
INNISFIL CREEK BRIDGE
BOREHOLE LOCATION PLAN
AND SOIL STRATA 1



SHEET

coffey



LEGEND

- Borehole
- 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
BH1	224.425	12+844	12.6 m LI CL
BH2	226.617	12+843	2.0 m RI CL
BH3	224.862	12+846	13.3 m RI CL
CPT4	224.848	12+808	13 m LI CL
BH5	224.286	12+870	6.2 m LI CL
BH6	225.464	12+869	6.8 m RI CL
BH7	225.918	12+814	1.3 m LI CL
BH8	225.866	12+824	8.8 m RI CL
BH9	224.693	12+901	10.6 m LI CL
BH10	225.355	12+903	6.0 m RI CL

-NOTE-

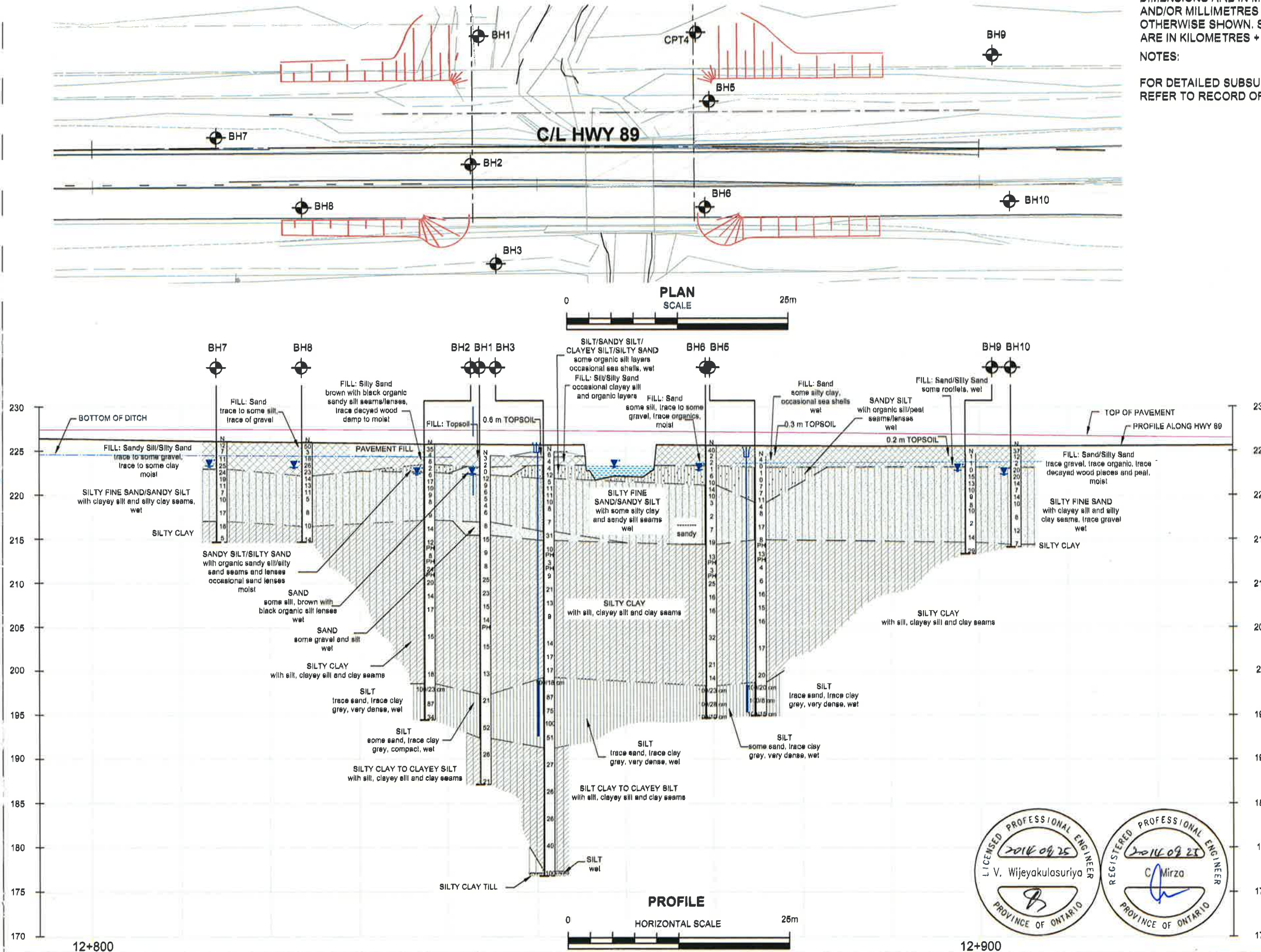
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31D-572

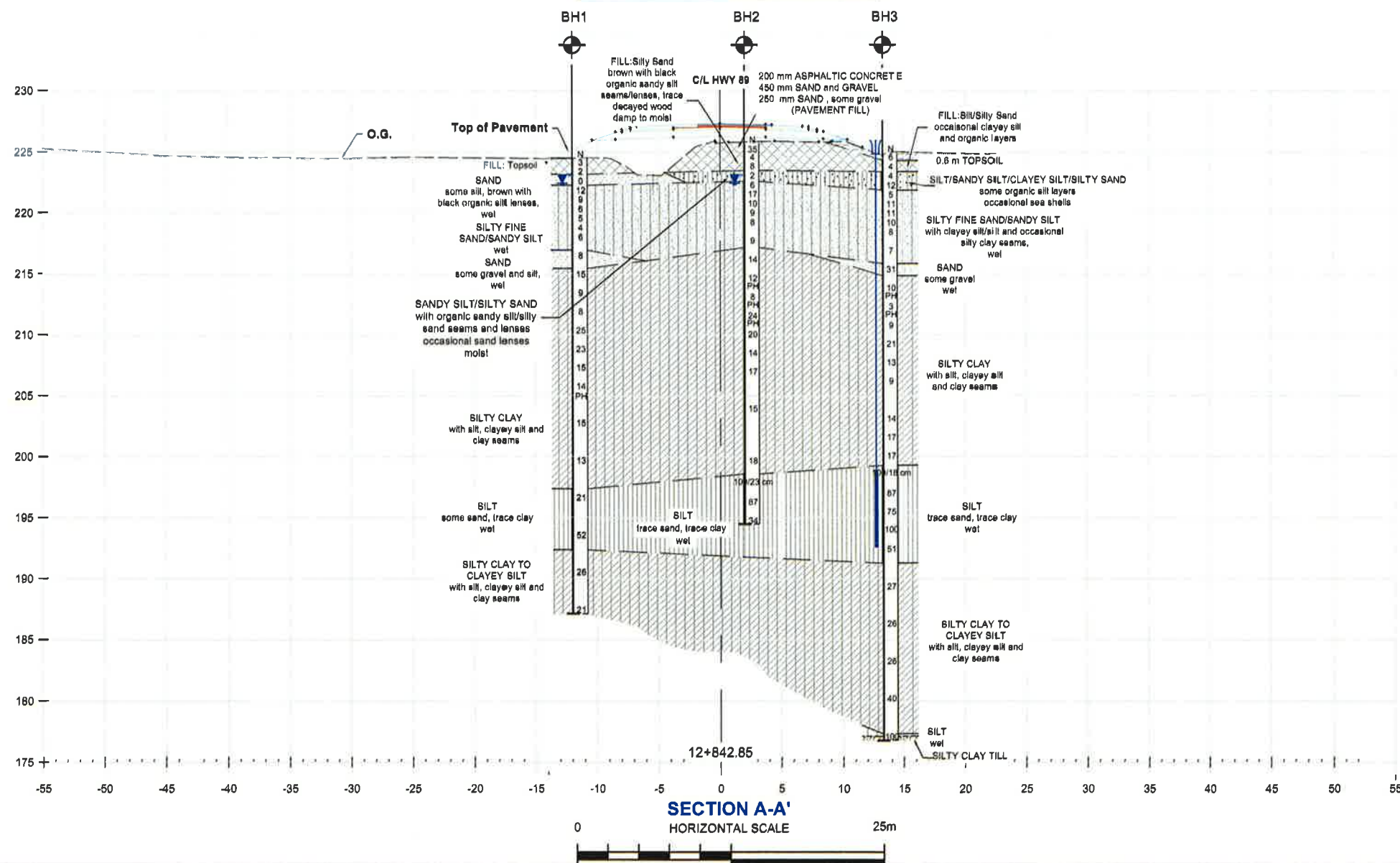
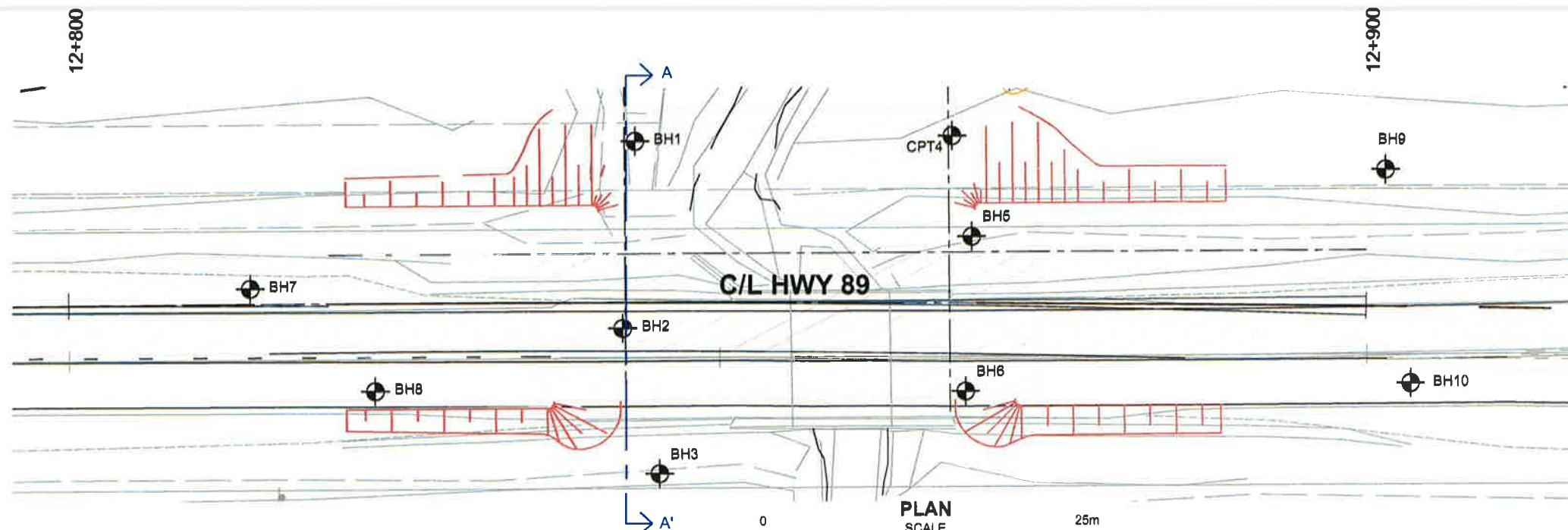
SUBMD	CHECKED	DATE	SITE	DIST
DRAWN	88H	CHECKED GR	APPROVED CM	DWG 1



PROFILE

HORIZONTAL SCALE





METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

CONT No. -

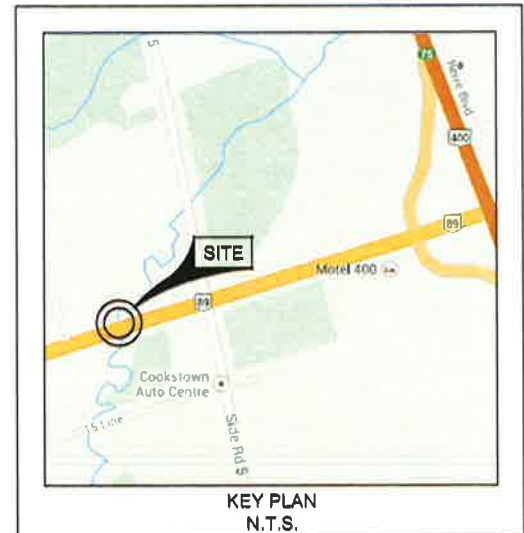
WP: 2108-11-00

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



SHEET

coffey



LEGEND

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

No.	ELEVATION	STATION	OFFSET
BH1	224.425	12+844	12.5 m LI CL
BH2	225.617	12+843	2.0 m RI CL
BH3	224.852	12+845	13.3 m RI CL

-NOTE-

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

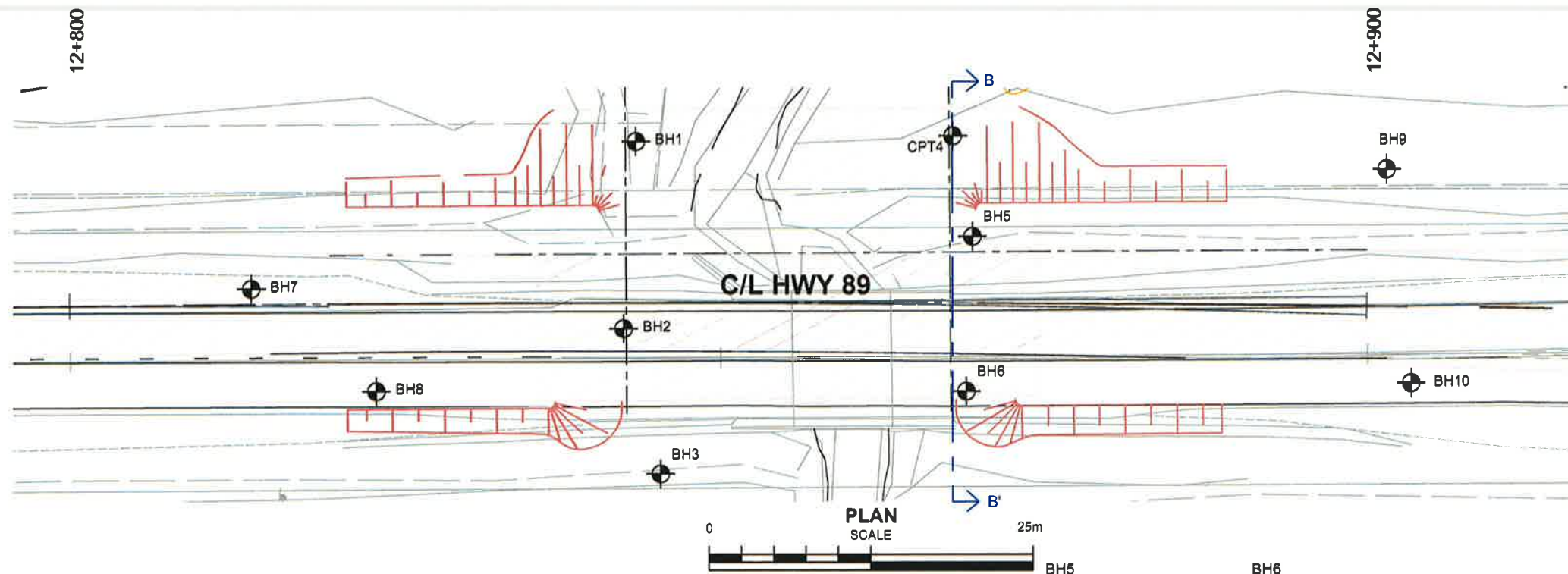
NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31D-672

TRANETO20482AA				DIST	
SUBMD	CHECKED	DATE	August, 2014	SITE	30-254
DRAWN	SSH	CHECKED	GR	APPROVED	CM
				DWG	2





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. -

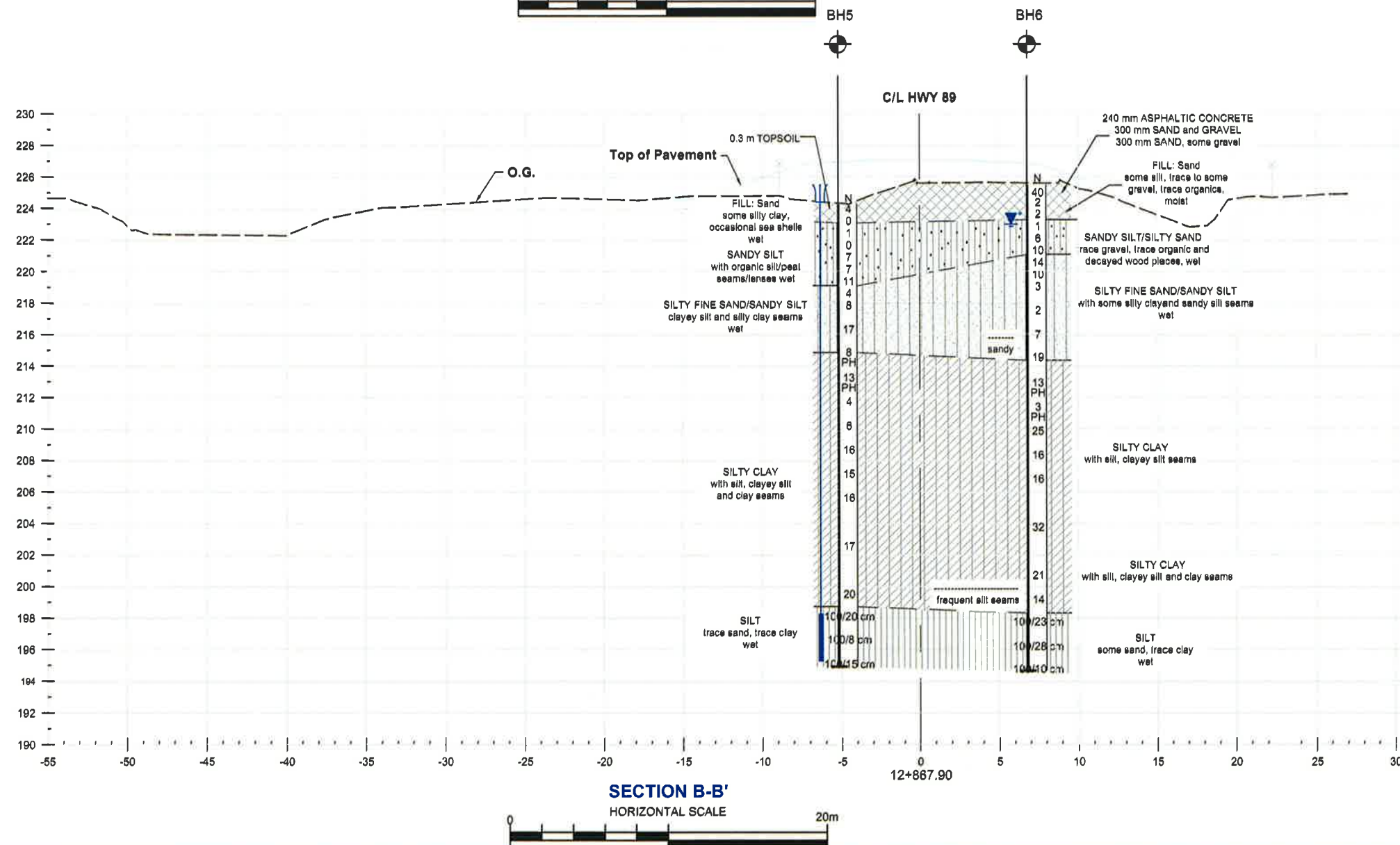
WP: 2108-11-00

HIGHWAY 89
INNISFIL CREEK BRIDGE
BOREHOLE LOCATION PLAN
AND SOIL STRATA 3



SHEET

coffey



LEGEND

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

No.	ELEVATION	STATION	OFFSET
BH5	224.266	12+879	5.2 m Li CL
BH6	225.404	12+869	6.8 m Ri CL

-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-572

TRANETOB20482AA				DIST	
SUBMD	CHECKED	DATE	August, 2014	SITE	30-254
DRAWN	SSH	CHECKED	GR	APPROVED	CM

Appendix A

Record of Borehole Sheets

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH1

1 OF 3

METRIC

GWP WP 2108-11-00 LOCATION 12+845, 16.7 m Lt C/L (N 4895299.517, E 291397.132) ORIGINATED BY LG
 DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and DCPTs COMPILED BY SSH
 DATUM Geodetic DATE 21/08/2013 23/08/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W		
224.4 0.0	GROUND SURFACE												
	FILL: Topsoil		1	SS	3		224						Additional DCPT Performed adjacent to the borehole from the ground surface
223.1 1.3			2	SS	2		223						
222.2 2.2	SAND some silt, brown with black organic silt lenses very loose, wet		3	SS	0		222						wet spoon
	occasional organic lenses		4	SS	12		221						
	SILTY FINE SAND/SANDY SILT grey, compact to very loose, wet		5	SS	9		220						0 45 45 10
	occasional silt and clayey silt lenses		6	SS	6		219						
			7	SS	5		218						0 45 46 9
			8	SS	4		217						
			9	SS	6		216						
216.9 7.5	SAND some gravel and silt grey, loose, wet		10	SS	8		215						16 67 (17)
215.4 9.0			11	SS	15		214						
	SILTY CLAY with silt, clayey silt and clay seams grey, stiff to very stiff		12	SS	9		213						0 3 45 52
			13	SS	8		212						
			14	SS	25		211						0 1 43 56
							210						

Continued Next Page

+³ ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 89

2 OF 3

METRIC

(%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH1

3 OF 3

METRIC

GWP WP 2108-11-00 LOCATION 12+845, 16.7 m Lt C/L (N 4895299.517, E 291397.132) ORIGINATED BY LG
 DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and DCPTs COMPILED BY SSH
 DATUM Geodetic DATE 21/08/2013 23/08/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _P	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)								WATER CONTENT (%)
								○ UNCONFINED ● POCKET PENETR.	+ FIELD VANE × LAB VANE							
194.4							20 40 60 80 100			10 20 30				GR SA SI CL		
	SILT some sand, trace clay grey, very dense, wet		22	SS	52											
192.4																
32.0	SILTY CLAY TO CLAYEY SILT with silt, clayey silt and clay seams grey, very stiff		23	SS	26									0 0 62 38		
187.2			24	SS	21											
37.2	End of Borehole * Groundwater in open borehole @ 2 m (El. 222.4 m) upon completion (not stabilized). Caved in @ 10.7 m upon completion. Dynamic Cone Penetration Test was performed from 37.2 to 41.5 m.															
183.0																
41.5	End of Dynamic Cone Penetration Test.															

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH2

1 OF 3

METRIC

GWP WP 2108-11-00 LOCATION 12+843, 4 m Lt C/L (N 4895285.415, E 291400.479) ORIGINATED BY LG
DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Auger Drilling COMPILED BY SSH
DATUM Geodetic DATE 12/09/2013 17/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
225.6	GROUND SURFACE													
0.0	200 mm Asphaltic Concrete 450 mm Sand and Gravel 250 mm Sand, some gravel (PAVEMENT FILL)		1	SS	35		225							
224.7			2	SS	4		224							
0.9	FILL: Silty Sand brown with black organic sandy silt seams/lenses, trace decayed wood very loose to compact, damp to moist		3	SS	8		223							
223.4			4	SS	2		222							
2.2	SANDY SILT/SILTY SAND with organic sandy silt/ silty sand seams and lenses occasional sand lenses black, very loose, moist		5	SS	6		221							
222.5			6	SS	17		220							
3.1			7	SS	10		219							
	SILTY FINE SAND/SANDY SILT clayey silt and silty clay seams grey, loose to compact, wet		8	SS	9		218							
			9	SS	8		217							
			10	SS	9		216							
217.1			11	SS	14		215							
8.5			12	SS	12		214							
	SILTY CLAY with silt, clayey silt and clay seams grey, firm to very stiff		13	TW	PH		213							
			14	SS	8		212							
			15	TW	PH		211							
			16	SS	24									
			17	TW	PH									

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+³, ×³: Numbers refer to
Sensitivity

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15 10 5
(%) STRAIN AT FAILURE



TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH2

2 OF 3

METRIC

GWP WP 2108-11-00 LOCATION 12+843, 4 m Lt C/L (N 4895285.415, E 291400.479) ORIGINATED BY LG
 DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Auger Drilling COMPILED BY SSH
 DATUM Geodetic DATE 12/09/2013 17/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)			WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE								
210.6							20	40	60	80	100	10	20	30		
	SILTY CLAY with silt, clayey silt and clay seams grey, stiff to very stiff		18	SS	20	210										
						209										
			19	SS	14	208										
						207										
			20	SS	17	206										
						205										
						204										
			21	SS	15	203										
						202										
						201										
						200										
						199										
198.6 27.0	SILT trace sand, trace clay grey, very dense, wet		23	SS	100/23 cm	198										
					197											
24			SS	87	196											

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH2

3 OF 3

METRIC

GWP WP 2108-11-00 LOCATION 12+843, 4 m Lt C/L (N 4895285.415, E 291400.479) ORIGINATED BY LG
 DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Auger Drilling COMPILED BY SSH
 DATUM Geodetic DATE 12/09/2013 17/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
195.6																	
194.5	SILT trace sand, trace clay grey, dense, wet		25	SS	34		195										
31.1	End of Borehole * Groundwater @ 3.2 m (El. 222.4 m) based on wetness of sample (not stabilized).																

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH3

1 OF 4

METRIC

GWP WP 2108-11-00 LOCATION 12+845, 8.5 m Rt C/L (N 4895285.415, E 291400.479) ORIGINATED BY LG
DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
DATUM Geodetic DATE 04/09/2013 05/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _P W W _L				
224.8 0.0	GROUND SURFACE													
	0.6 m TOPSOIL		1	SS	6									
224.2 0.6	FILL: Silt/Silty Sand occasional clayey silt and organic layers brown, very loose		2	SS	4		224							
223.3 1.5	SILT/SANDY SILT/CLAYEY SILT/SILTY SAND some organic silt layers, occasional sea shells brown/dark grey, very loose to compact, wet		3	SS	4		223							
			4	SS	12		222							wet spoon
221.8 3.0	SILTY FINE SAND/SANDY SILT with clayey silt/silt and occasional silty clay seams grey, loose to compact, wet		5	SS	5		221							
			6	SS	11		220							0 32 54 14
			7	SS	11		219							
			8	SS	10		218							
			9	SS	8		217							2 28 56 14
			10	SS	7		216							
			11	SS	31		215							
215.8 9.0	SAND some gravel grey, dense, wet						214							
214.8 10.0	SILTY CLAY with silt, clayey silt and clay seams grey, firm to very stiff		12	SS	10		213							
			13	TW	PH		212							0 4 32 64
			14	SS	3		211							
			15	TW	PH									
			16	SS	9									
							210							

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+³, ×³: Numbers refer to
Sensitivity

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15
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(%) STRAIN AT FAILURE




TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH3

2 OF 4

METRIC

GWP WP 2108-11-00 LOCATION 12+845, 8.5 m Rt C/L (N 4895285.415, E 291400.479) ORIGINATED BY LG
DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
DATUM Geodetic DATE 04/09/2013 05/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE		WATER CONTENT (%) w _P w w _L				
209.8								20 40 60 80 100	20 40 60 80 100	10 20 30				
	SILTY CLAY with silt, clayey silt and clay seams grey, stiff to hard		17	SS	21									0 1 45 54
							209							
							208							
							207							
							206							
							205							
							204							
							203							
							202							
							201							
199.3 25.5	SILT trace sand, trace clay grey, very dense, wet													
			23	SS	100/18 cm	199								0 5 88 7
						198								
			24	SS	87	197								
						196								
			25	SS	75									0 7 86 7

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+³ ×³: Numbers refer to
Sensitivity

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(%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH3

3 OF 4

METRIC

GWP WP 2108-11-00 LOCATION 12+845, 8.5 m Rt C/L (N 4895285.415, E 291400.479) ORIGINATED BY LG
DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
DATUM Geodetic DATE 04/09/2013 05/09/2013 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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
194.8	SILT trace sand, trace clay grey, very dense, wet		26	SS	100		194				
							193				
								192			
	SILTY CLAY with silt, clayey silt and clay seams grey, very stiff		27	SS	51		191				
191.3							190				
33.5							189				
							188				
							187				
							186				
							185				
							184				
							183				
							182				
			28	SS	27		181				
						180					
			29	SS	26						
			30	SS	26						
			31	SS	40						

Continued Next Page

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

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH3

4 OF 4

METRIC

GWP WP 2108-11-00 LOCATION 12+845, 8.5 m Rt C/L (N 4895285.415, E 291400.479) ORIGINATED BY LG
 DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
 DATUM Geodetic DATE 04/09/2013 05/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
179.8														
	SILTY CLAY TO CLAYEY SILT with silt, clayey silt and clay seams grey, hard													
177.4														
177.2														
177.4	SILT grey wet		32	SS	100									
176.9	SILT CLAY TILL grey, hard													
176.9	End of Borehole * Groundwater @ 2.4 m (El. 222.4 m) based on wetness of sample (not stabilized). Artesian condition was recorded in piezometer (about 3 m above grade, measured by pressure guage).													





TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH5

1 OF 2

METRIC

GWP WP 2108-11-00 LOCATION 12+869, 9 m Lt C/L (N 4895300.153, E 291424.125) ORIGINATED BY LG
DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
DATUM Geodetic DATE 30/08/2013 04/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _P	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							WATER CONTENT (%)
								○ UNCONFINED ● POCKET PENETR.	+ FIELD VANE × LAB VANE						
								20 40 60 80 100	20 40 60 80 100						
224.3 0.0	GROUND SURFACE														
223.1 1.2	0.3 m TOPSOIL		1	SS	4		224						20.4	consolidation test	
	2		SS	0	223										
	SANDY SILT with organic silt/peat seams/lenses brown/black, very loose, wet	3	SS	1	222										
	frequent sea shells and organics														
		4	SS	0	221										
		5	SS	7	220										
		6	SS	7	219										
		7	SS	11	218										
219.1 5.2	SILTY FINE SAND/SANDY SILT clayey silt and silty clay seams grey, wet		8	SS	4	217									
			9	SS	8	216									
	very loose to loose compact														
			10	SS	17	215									
214.9 9.4	SILTY CLAY with silt, clayey silt and clay seams grey, firm to very stiff		11	SS	8	214									
			12	TW	PH	213									
			13	SS	13	212									
			14	TW	PH	211									
			15	SS	4	210									
			16	SS	6										

Continued Next Page

+³ ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH5

2 OF 2

METRIC

GWP WP 2108-11-00 LOCATION 12+869, 9 m Lt C/L (N 4895300.153, E 291424.125) ORIGINATED BY LG
DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
DATUM Geodetic DATE 30/08/2013 04/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
209.3														
	SILTY CLAY with silt, clayey silt and clay seams grey, very stiff		17	SS	16		209							
							208							
			18	SS	15		207							0 0 57 43
							206							
			19	SS	16		205							
							204							
							203							
			20	SS	17		202							
							201							
							200							
			21	SS	20		199							
198.8 25.5							198							0 4 89 7
	SILT trace sand, trace clay grey, very dense, wet		22	SS 100/20 cm			197							
							196							
			23	SS 100/8 cm										
195.0 29.3			24	SS 100/15 cm										
	End of Borehole * Groundwater @ 2.5m (El. 221.8 m) based on wetness of sample (not stabilized). Artesian condition was recorded in piezometer													

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH6

1 OF 3

METRIC

GWP WP 2108-11-00 LOCATION 12+868, 2.6 m Rt C/L (N 4895288.545, E 291427.189) ORIGINATED BY LG
DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
DATUM Geodetic DATE 09/09/2013 11/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
225.5	GROUND SURFACE													
0.0	240 mm ASPHALTIC CONCRETE 300 mm SAND and GRAVEL 300mm SAND, some gravel		1	SS	40		225							
	FILL: Sand some silt, trace to some gravel trace organics brown, very loose to dense, moist		2	SS	2		224							
223.3			3	SS	2		223							
2.2	SANDY SILT/SILTY SAND trace gravel, trace organic and decayed wood pieces grey, very loose to loose, wet (dialatant)		4	SS	1		222							
			5	SS	6		221							
221.1			6	SS	10		220							
4.4	SILT FINE SAND/SANDY SILT with some silty clay and sandy silt seams grey, very loose to compact, wet (dialatant)		7	SS	14		219							
			8	SS	10		218							
			9	SS	3		217							
			10	SS	2		216							
			11	SS	7		215							
			12A	SS	19		214							
214.4		sandy	12B	SS			213							
11.1	SILTY CLAY with silt, clayey silt seams grey, soft to stiff		13	SS	13		212							
			14	TW	PH		211							
			15	SS	3									
			16	TW	PH									

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+³ ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

TW16:
No recovery

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH6

2 OF 3

METRIC

GWP WP 2108-11-00 LOCATION 12+868, 2.6 m Rt C/L (N 4895288.545, E 291427.189) ORIGINATED BY LG
 DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
 DATUM Geodetic DATE 09/09/2013 11/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
210.5														
	SILTY CLAY with silt, clayey silt and clay seams grey, stiff to hard		17	SS	25		210							
							209							
			18	SS	16		208							
							207							
			19	SS	16		206							
							205							
							204							
			20	SS	32		203							
							202							
							201							
			21	SS	21		200							
							199							
			22	SS	14		198							
							197							
			23	SS	100/23 cm		196							
			24	SS	100/28 cm									
198.4 27.1	SILT some sand, trace clay grey, very dense, wet													

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH6

3 OF 3

METRIC

GWP WP 2108-11-00 LOCATION 12+868, 2.6 m Rt C/L (N 4895288.545, E 291427.189) ORIGINATED BY LG
 DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
 DATUM Geodetic DATE 09/09/2013 11/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
195.5																	
194.7	SILT some sand, trace clay grey, very dense, wet		25	SS	100/10 cm		195										
30.7	End of Borehole * Groundwater @ 2.6 m (El. 222.9 m) based on wetness of sample (not stabilized).																

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH7

1 OF 1

METRIC

GWP WP 2108-11-00 LOCATION 12+813, 5.3 m Lt C/L (N 4895279.919, E 291372.222) ORIGINATED BY LG
DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
DATUM Geodetic DATE 12/09/2013 12/09/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
225.9 0.0	GROUND SURFACE		1	SS	12		225							
	FILL: Sandy Silt/Silty Sand trace to some gravel trace to some clay brown, loose to compact, moist		2A	SS	7									
			2B	SS										
			3	SS	11		224							
			4	SS	25		223							
222.9 3.0	SILTY FINE SAND/SANDY SILT with clayey silt and silty clay seams grey, compact, wet		5	SS	24		222							
			6	SS	19		221							
			7	SS	11		220							
			8	SS	7		219							
			9	SS	10		218							
			10	SS	17		217							
			11	SS	18		216							
217.0 8.9	SILTY CLAY grey, firm to stiff		12	SS	5		215							
214.6 11.3	End of Borehole * Groundwater @ 2.6 m (El. 223.3 m) based on wetness of sample (not stabilized).													

+³, ×³: Numbers refer to
Sensitivity

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(%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 89

1 OF 1

METRIC

SOIL PROFILE					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES	GROUND WATER CONDITIONS	ELEVATION SCALE
			NUMBER	TYPE	"N" VALUES
226.0	GROUND SURFACE				
0.0	200 mm ASPHALTIC CONCRETE 200 mm SAND and GRAVEL 300 mm SAND,some gravel		1	SS	50
	FILL: Sand trace to some silt trace of gravel brown, very loose to dense		2	SS	3
			3	SS	18
			4	SS	26
			5	SS	23
222.2			6	SS	14
3.8	SILT FINE SAND/SANDY SILT with clayey silt and silty clay seams grey, very loose to compact, wet		7	SS	13
			8	SS	11
			9	SS	5
			10	SS	8
216.5			11	SS	10
9.5	SILT CLAY grey, firm to stiff		12	SS	14
214.7	End of Borehole * Groundwater @ 2.8 m (El. 223.2 m) based on wetness of sample (not stabilized).				
11.3					

+³, ×³: Numbers refer to Sensitivity

TRANETOB20462AA: Hwy 89

RECORD OF BOREHOLE No BH9

1 OF 1

METRIC

GWP WP 2108-11-00 LOCATION 12+902, 14 m Lt C/L (N 4895314.493, E 291453.186) ORIGINATED BY LG
DIST 5 HWY 89 BOREHOLE TYPE Hollow Stem Auger and Mud Rotary Drilling COMPILED BY SSH
DATUM Geodetic DATE 30/08/2013 30/08/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE								WATER CONTENT (%)	
224.6 0.0	GROUND SURFACE						20	40	60	80	100	10	20	30			
	0.2 m TOPSOIL		1	SS	1	☼*	224										
	FILL: Sandy Silt/Silty Sand some rootlets brown, very loose, wet		2	SS	1												
223.1 1.5	occasional sea shells and organics, very loose		3	SS	0		223										
	SILTY FINE SAND/SANDY SILT with clayey silt and silty clay seams, very occasional organics grey, loose to compact, wet		4	SS	15		222										
			5	SS	13		221										
			6	SS	10		220										
			7	SS	9		219										
			8	SS	8		218										
			9	SS	10		217										
			10	SS	2		216										
			11	SS	14		215										
214.6 10.0		SILTY CLAY grey, very stiff						214									
213.3 11.3		End of Borehole * Groundwater @ 1.8 m (El. 222.2 m) based on wetness of sample (not stabilized).															

TRANETOB20462AA: Hwy 89

1 OF 1

METRIC

+³, ×³: Numbers refer to Sensitivity

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

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2.0 FIELD TESTING PROCEDURES	1
3.0 CPT RESULTS	1
4.0 INTERPRETATION	3
5.0 SUMMARY OF RESULTS	7
6.0 REFERENCES	9
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FIGURE NO. 1 – CPT Location Plan

APPENDIX A – Calibration Certificate
APPENDIX B – Piezocone Soundings
APPENDIX C – Dissipation Test Results

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 cm², a friction sleeve area of 150 cm² and a u₂ 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 cm², a friction sleeve area of 150 cm² and a u₂ 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.

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

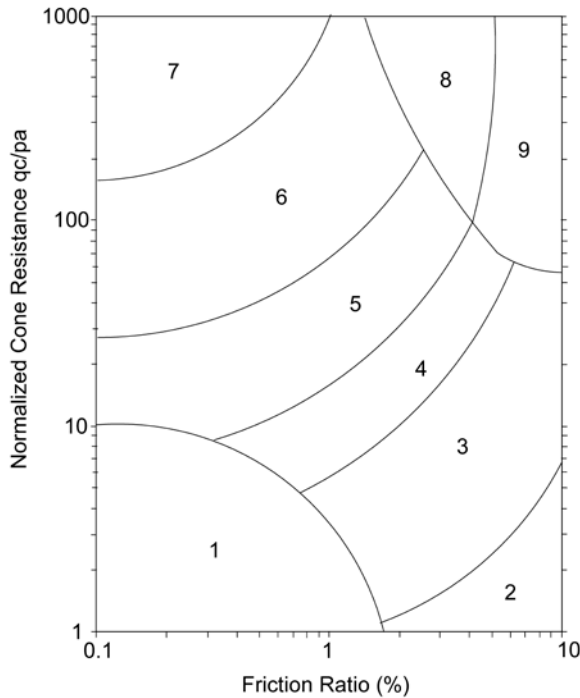
Pore Pressure

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 (SBT_n) 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 $S_u/\sigma_{v0}' \sim 0.22 \text{ OCR}$, 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,

$$\text{OCR} = k (q_t - \sigma_v)/\sigma_v'$$

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):

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

For $I_c > 2.2$ (Clays):

$$\alpha_m = Q_t \text{ when } Q_t < 14$$

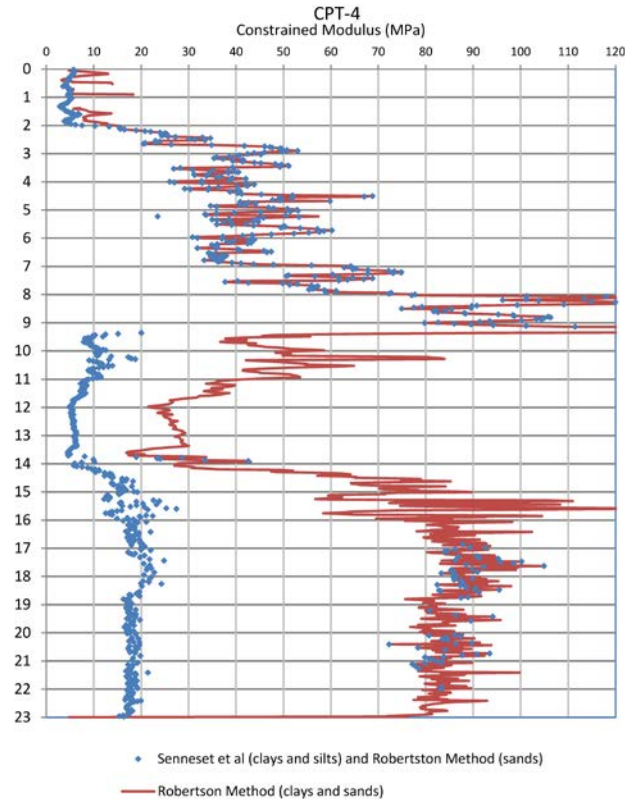
$$\alpha_m = 14 \text{ when } Q_t > 14$$

Senneset et al Method:

For SBTn <6 (Silts, Clays and Clayey Silts):
 $\alpha_m = 3$

The α_m value above was selected based on oedometer 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).

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

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.

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

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

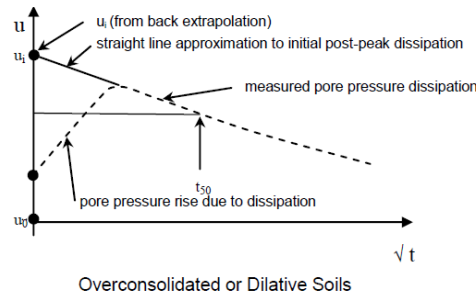
Coefficient of Consolidation

The horizontal coefficient of consolidation (C_h) of the soil can be estimated from the pore pressure dissipation test results. Monotonic and dilatatory 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.

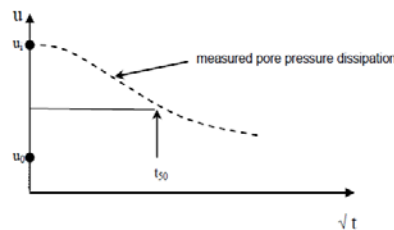
$$C_h = \frac{T_{50}^*}{t_{50}} r^2 I_R^{0.5}$$

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 (dilatatory 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.



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.

DownUnder Geotechnical Limited

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 $\sigma=0.5$	3 $\sigma=3$	4.9 $\sigma=1.4$	28.0 $\sigma=22.0$	71 $\sigma=20$
CPT-11a	0 to 3.1	1.4 $\sigma=0.5$	3 $\sigma=1$	4.3 $\sigma=1.3$	20.0 $\sigma=14.8$	47 $\sigma=10$
CPT-14	0 to 2.7	1.1 $\sigma=0.5$	2 $\sigma=1$	3.4 $\sigma=1.6$	13.9 $\sigma=8.9$	43 $\sigma=15$

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 $\sigma=3.0$	20 $\sigma=5$	42 ⁰ $\sigma=2$	55 $\sigma=23$
CPT-11a	3.1 to 8.6	10.2 $\sigma=2.7$	18 $\sigma=4$	41 ⁰ $\sigma=3$	48 $\sigma=14$
CPT-14	2.7 to 9.4	35.3 $\sigma=23.5$	18 $\sigma=4$	41 ⁰ $\sigma=3$	49 $\sigma=11$

*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 $\sigma=1.6$	10 $\sigma=4$	10.7 $\sigma=5.6$	5.3 $\sigma=2.2$	150 $\sigma=71$
CPT-11a	8.6 to 14.9	3.7 $\sigma=1.6$	10 $\sigma=4$	10.6 $\sigma=7.5$	5.0 $\sigma=1.6$	127 $\sigma=40$
CPT-14	9.5 to 15.7	2.8 $\sigma=0.7$	8 $\sigma=2$	7.6 $\sigma=2.0$	3.8 $\sigma=1.0$	107 $\sigma=27$

*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 $\sigma=0.6$	15 $\sigma=1$	18.2** $\sigma=1.9$	7.1 $\sigma=1.0$	265 $\sigma=21$

**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 $C_h \sim 84C_v$. 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.

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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, Amsterdam, 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

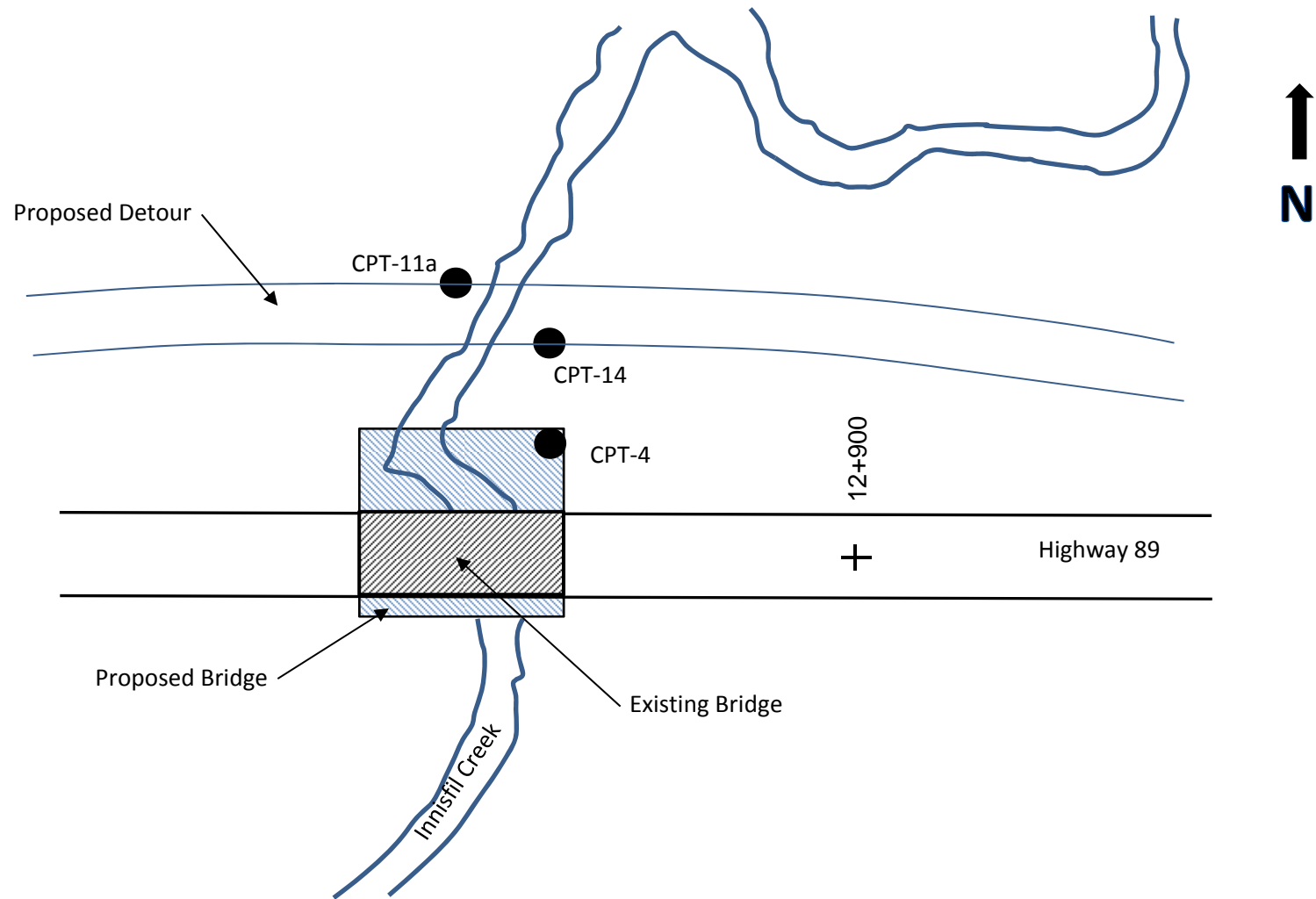


Figure No.1
CPT Location Plan

DownUnder Geotechnical Limited

APPENDIX A – Calibration Certificate

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 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	Calibrated June 27, 2012
--	--------------------------

Sensotec® Pressure Transducer Model A-10/6076-08 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	Calibrated June 27, 2012
---	--------------------------

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.

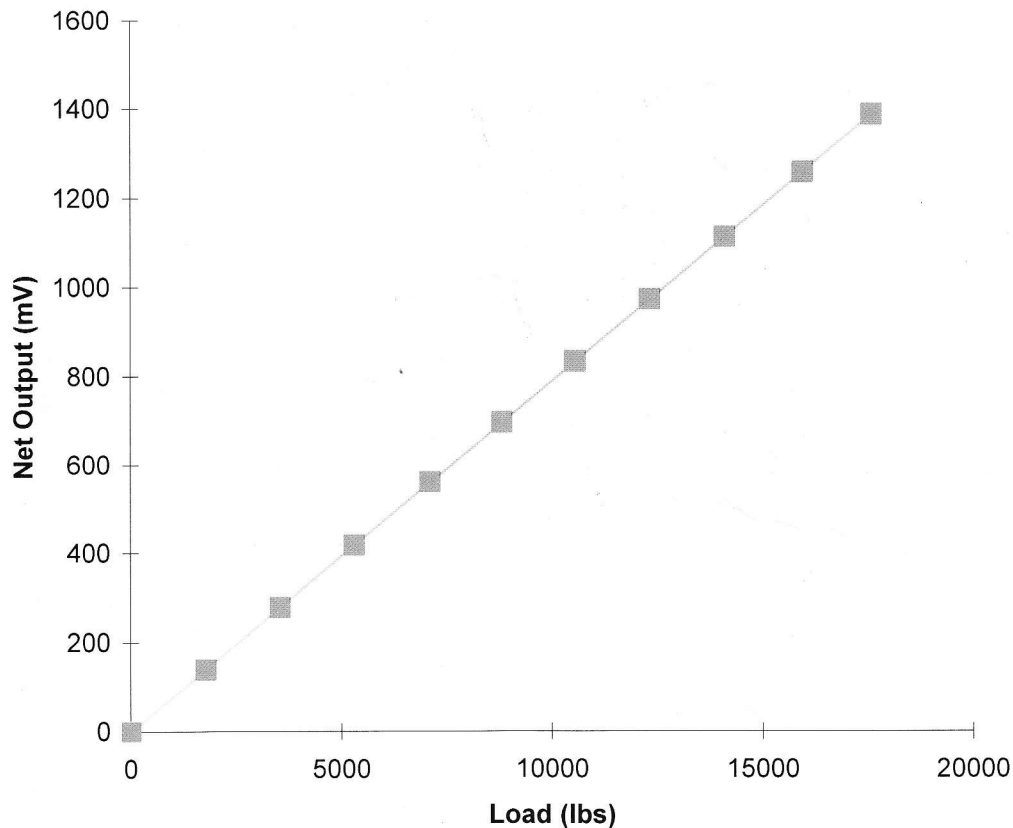


250 Beanville Road
Randolph, Vermont 05060
phone: (800)639-6315 fax: (802)728-9871

Cone Penetrometer Calibration
Digital Cone Tip

Cone Serial No.: 4644.103
Rated Range: 22000 lbs
Load Reference: Ref LC-SN: 322089A
Ref. DVM: MY47026116
Ref. Excitation: 5.049 V_{dc}

Date: 27-Oct-11
Calibrated By: WJC
Approved By: R. G. Hull



Cal Factor: 78.781E-3 mV/lbs 75.000E-3 nominal
R²: 1.00000
Nonlinearity: 0.05
Zero Load Output: 250.641E-3 V

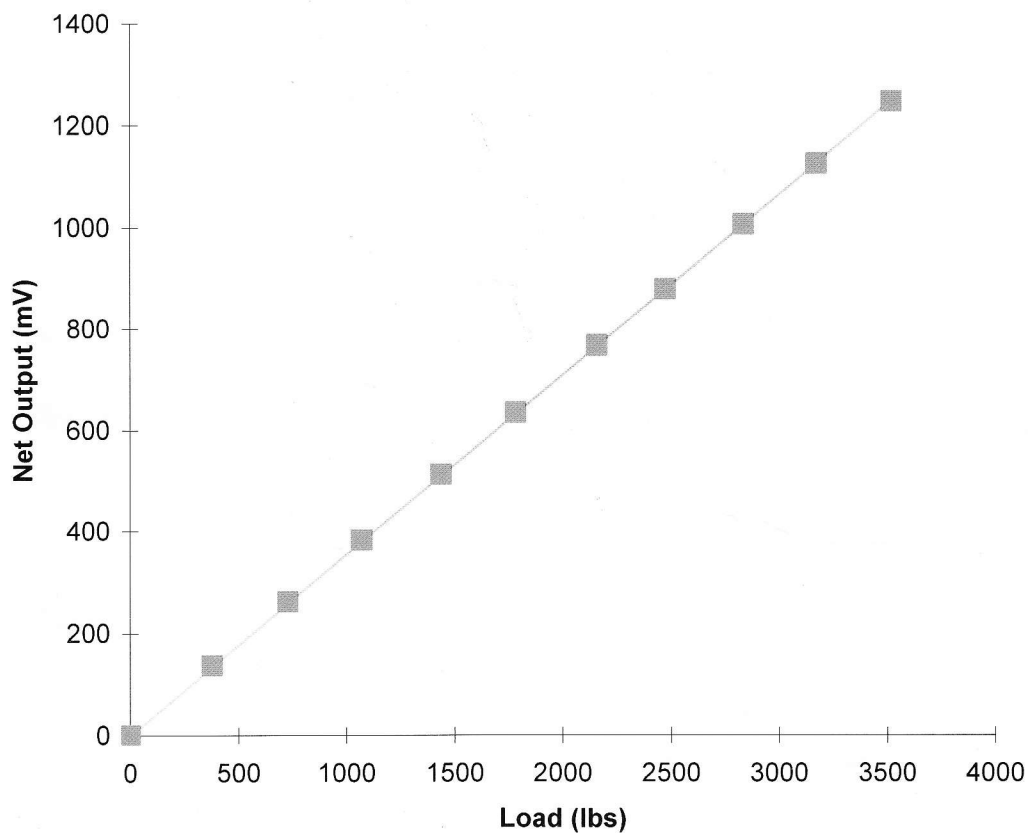


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Randolph, Vermont 05060
phone: (800)639-6315 fax: (802)728-9871

Cone Penetrometer Calibration
Digital Cone Sleeve

Cone Serial No.: 4644.103
Rated Range: 4400 lbs
Load Reference: Ref LC-SN: 322089A
Ref. DVM: MY47026116
Ref. Excitation: 5.049 V_{dc}

Date: 27-Oct-11
Calibrated By: WJC
Approved By: [Signature]



Cal Factor: 353.139E-3 mV/lbs 350.000E-3 nominal
R²: 0.99999
Nonlinearity: 0.34
Zero Load Output: 295.359E-3 V

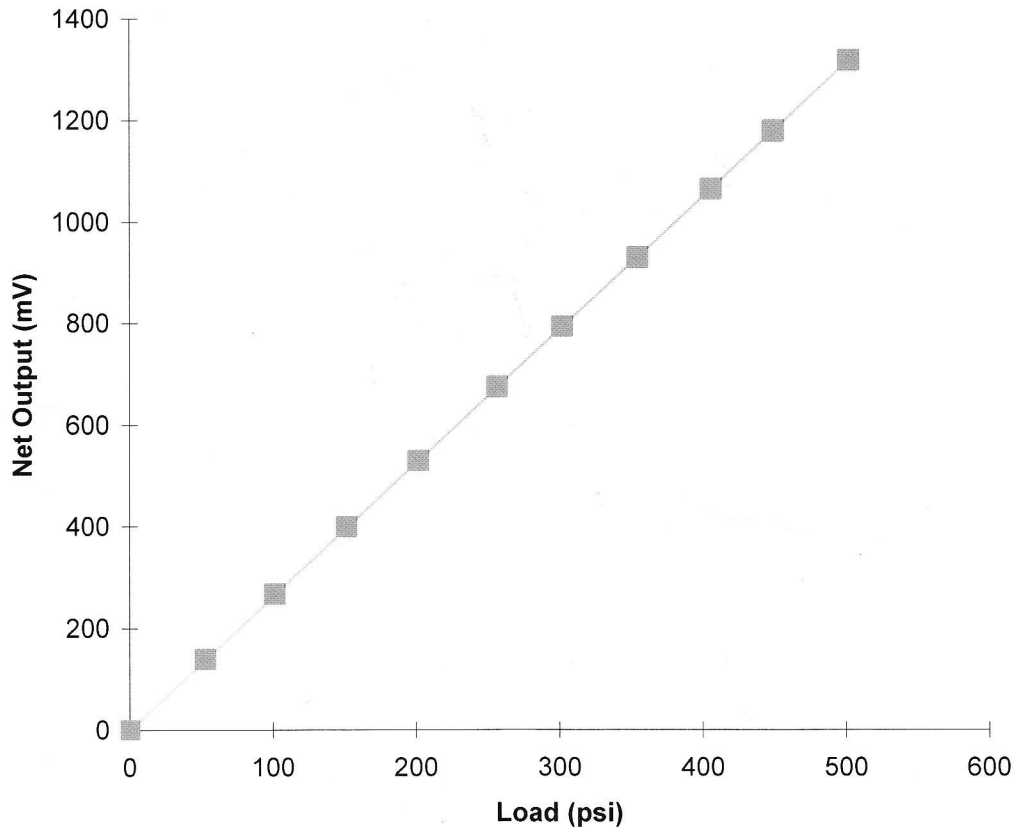


250 Beanville Road
Randolph, Vermont 05060
phone: (800)639-6315 fax: (802)728-9871

Cone Penetrometer Calibration
Digital Cone Pore Pressure

Cone Serial No.: 4644.103
Rated Range: 2000 psi
Load Reference: Ref PT-SN:0937-016VMC
Ref. DVM: MY47026116
Ref. Excitation: 5.049 V_{dc}

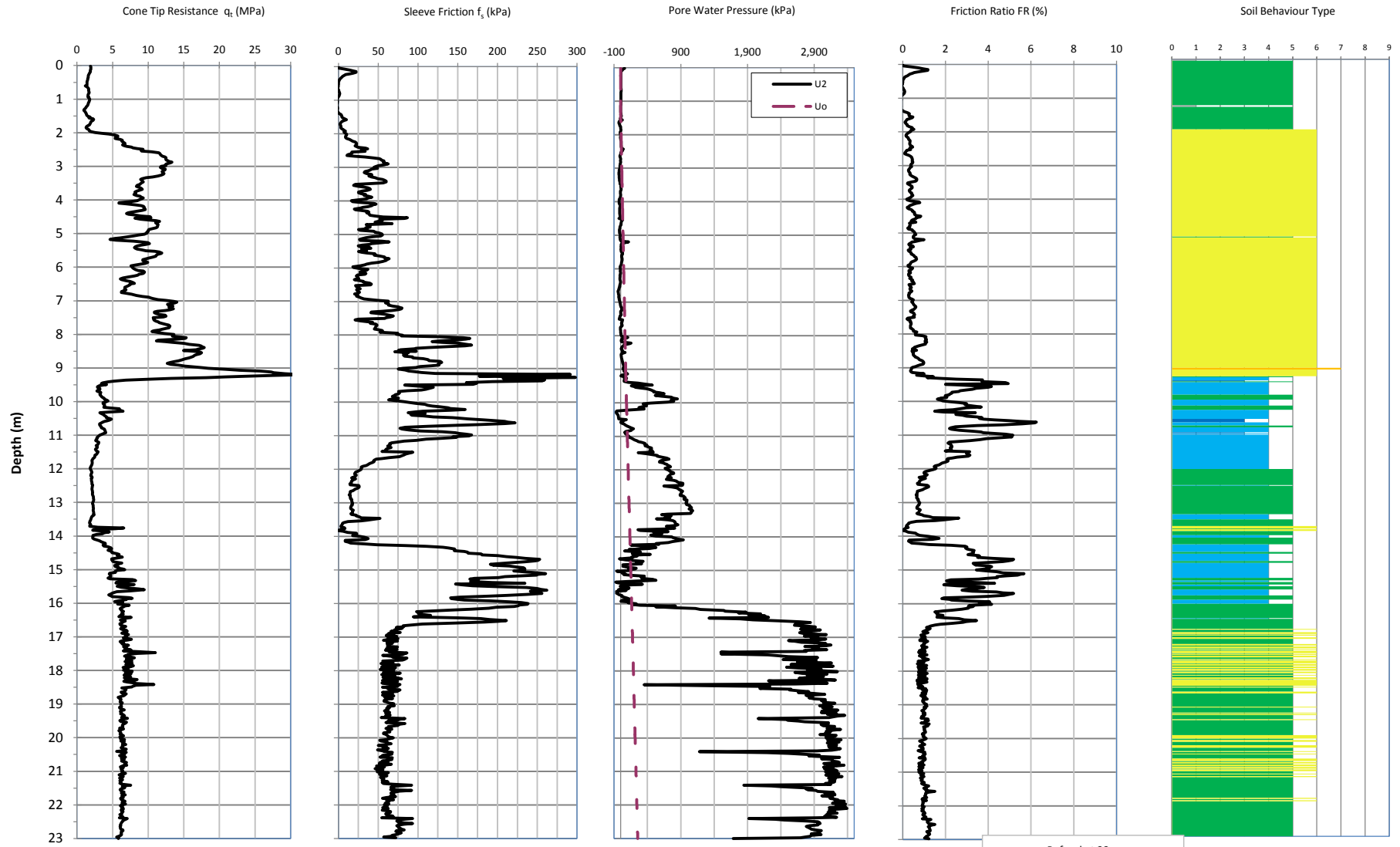
Date: 27-Oct-11
Calibrated By: WJC
Approved By: L. G. Hull



Cal Factor: 2.621E+0 mV/psi 2.500E+0 nominal
R²: 1.00000
Nonlinearity: 0.11
Zero Load Output: 212.940E-3 V

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 cm²
 Friction Sleeve Area: 150 cm²
 Filter Location: U_2

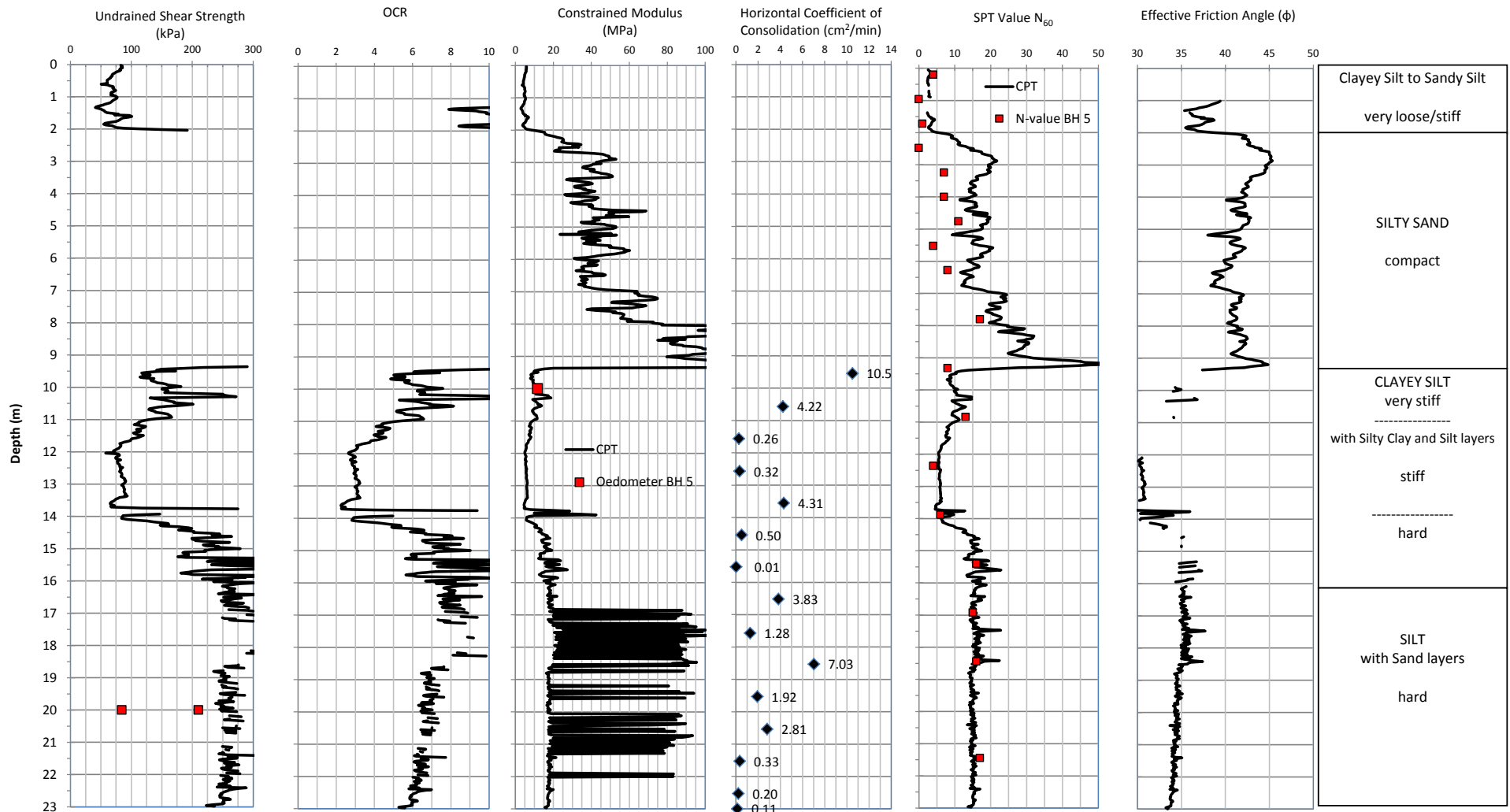
Refusal at 23m
 Anchors pulled out of the ground

CPT-4

CPT Probe 4644.103

DownUnder Geotechnical Limited

PiezoCone Penetration Test



Elevation: 224.874m
 Date: September 5, 2013
 Location: Highway 89 and Innisfil Creek, Ontario
 Engineer: A. Drevininkas
 Cone: VERTEK 10 tonne
 Tip Area: 10 cm²
 Friction Sleeve Area: 150 cm²
 Filter Location: U₂

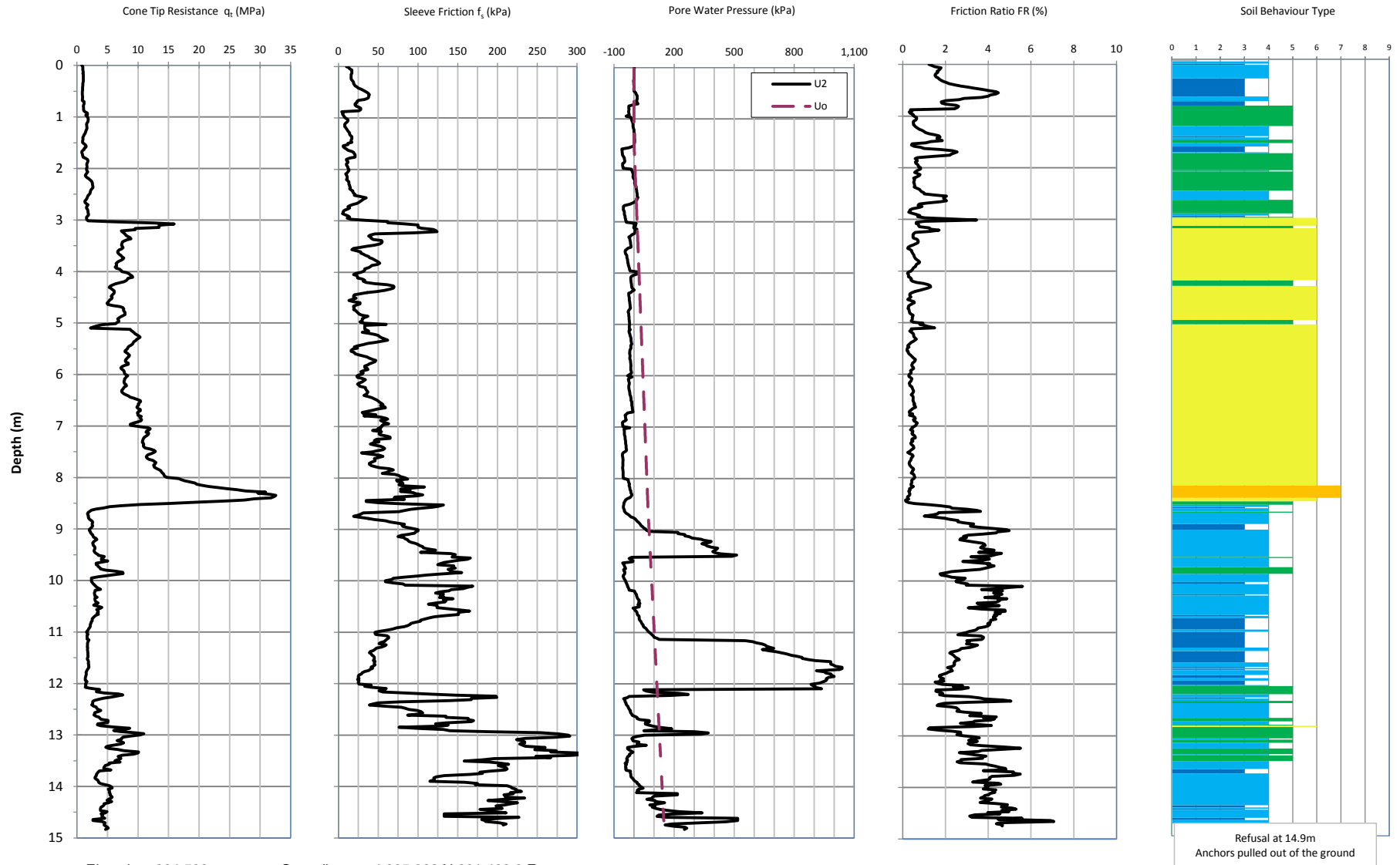
Co-ordinates: 4,895,317 N 291,416.7 E

CPT-4

CPT Probe 4644.103

DownUnder Geotechnical Limited

PiezoCone Penetration Test



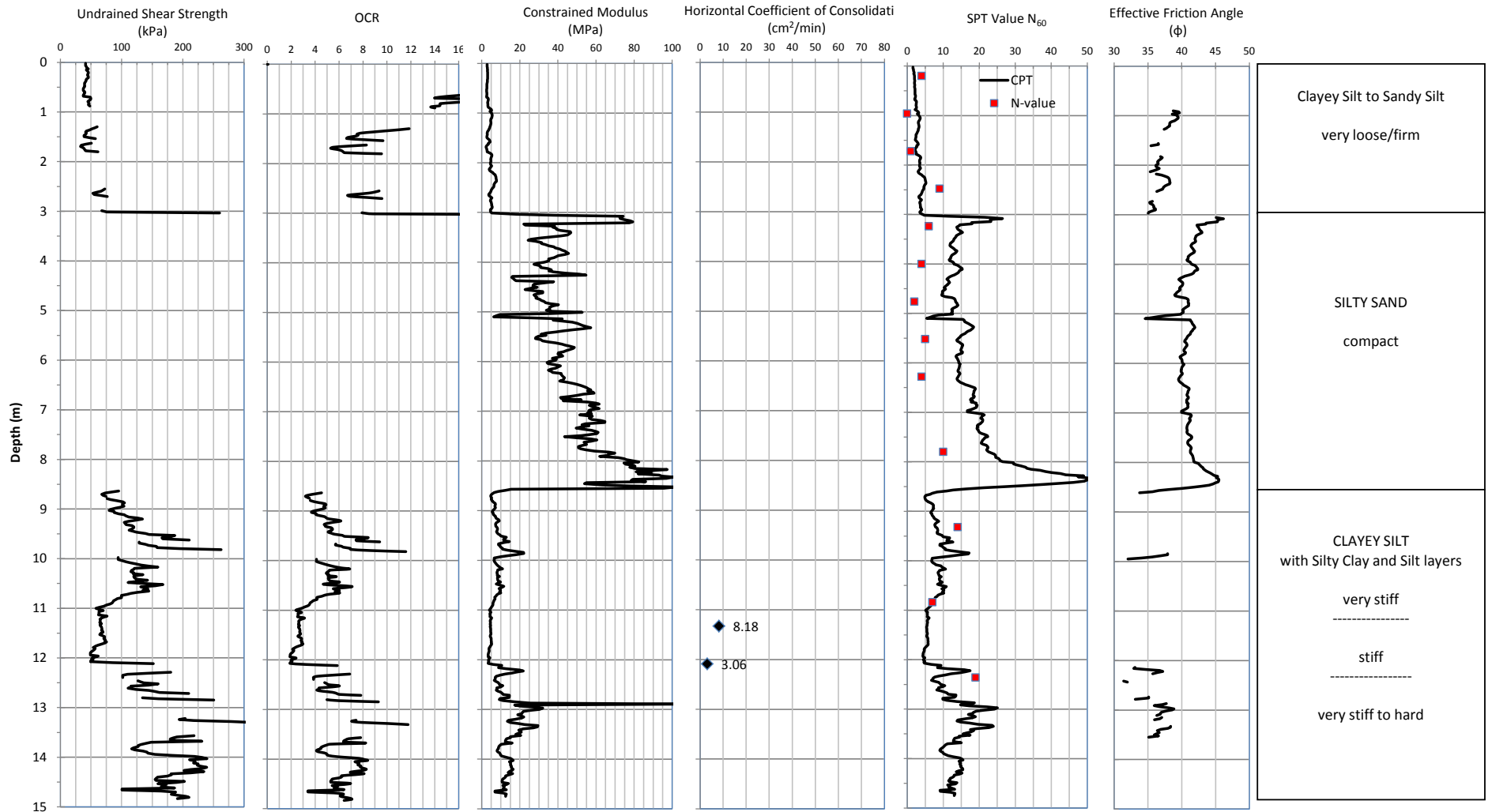
Elevation: 224.598m Co-ordinates: 4,895,322 N 291,400.9 E
 Date: September 4, 2013
 Location: Highway 89 and Innisfil Creek, Ontario
 Engineer: A. Drevininkas
 Cone: GEOTECH AB 10 tonne
 Tip Area: 10 cm²
 Friction Sleeve Area: 150 cm²
 Filter Location: U₂

DownUnder Geotechnical Limited

CPT-11A

CPT Probe 4143

PiezoCone Penetration Test



Elevation: 224.598m
 Date: September 4, 2013
 Location: Highway 89 and Innisfil Creek, Ontario
 Engineer: A. Drevininkas
 Cone: GEOTECH AB 10 tonne
 Tip Area: 10 cm²
 Friction Sleeve Area: 150 cm²
 Filter Location: U₂

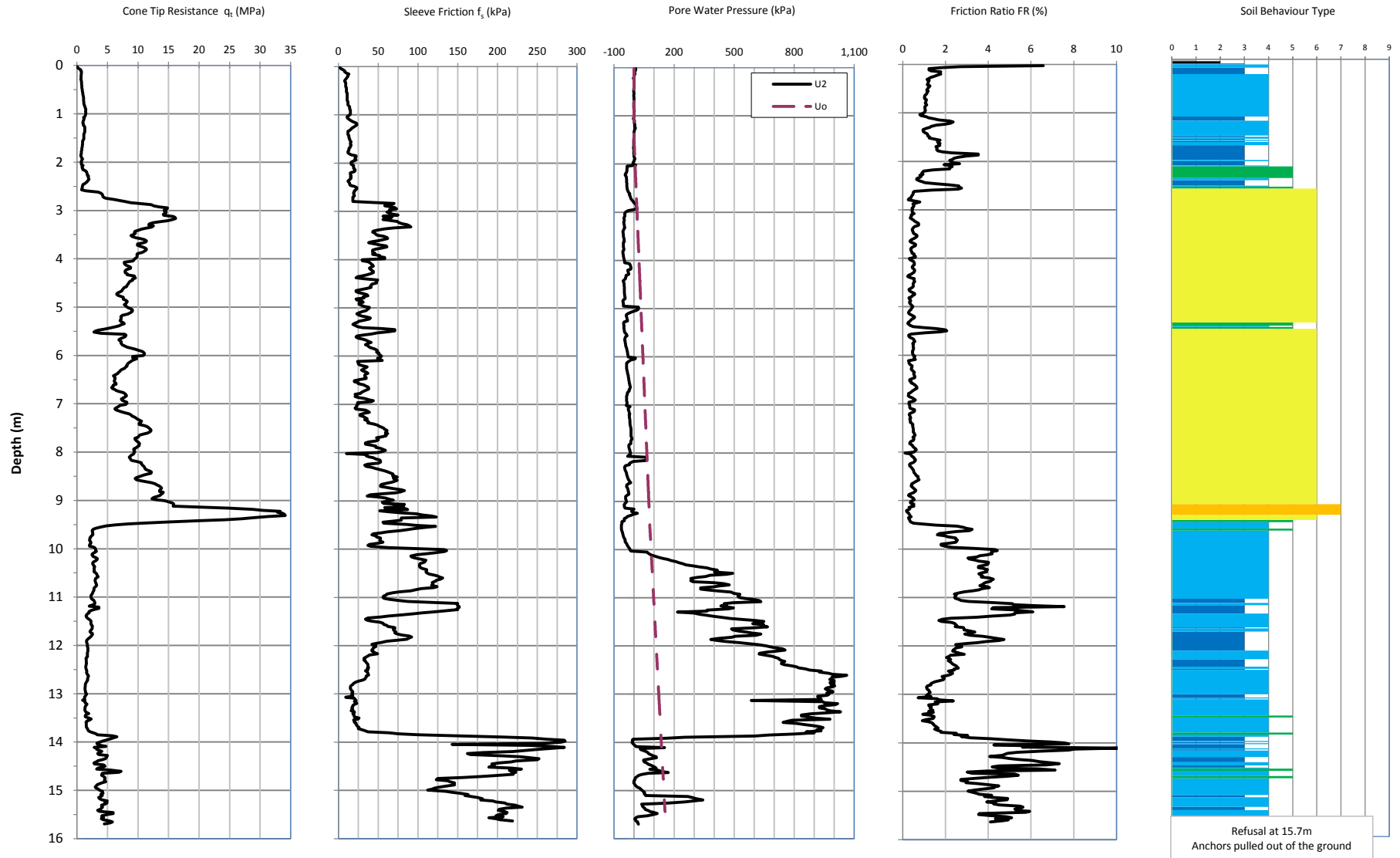
Co-ordinates: 4,895,322 N 291,400.9 E

CPT-11A

CPT Probe 4143

DownUnder Geotechnical Limited

PiezoCone Penetration Test



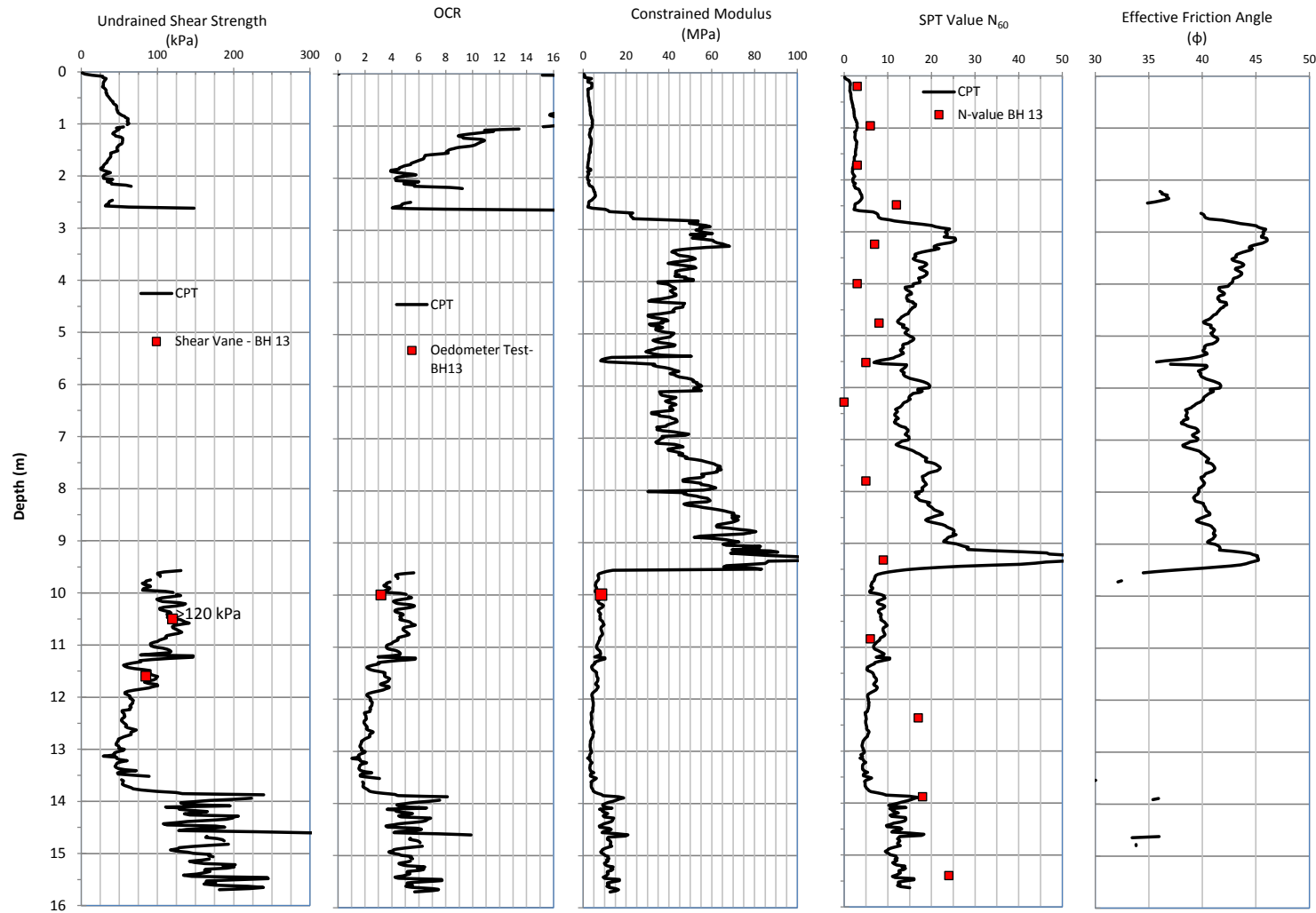
Elevation: 224.549m Co-ordinates: 4,895,307N 291,420.4E
 Date: September 4, 2013
 Location: Highway 89 and Innisfil Creek, Ontario
 Engineer: A. Drevininkas
 Cone: GEOTECH AB 10 tonne
 Tip Area: 10 cm²
 Friction Sleeve Area: 150 cm²
 Filter Location: U_2

DownUnder Geotechnical Limited

CPT-14

CPT Probe 4143

PiezoCone Penetration Test



CLAYEY SILT to SANDY SILT very loose/firm to stiff
SILTY SAND compact
CLAYEY SILT with Silty Clay layers very stiff ----- stiff ----- very stiff

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 cm²
 Friction Sleeve Area: 150 cm²
 Filter Location: U₂

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)

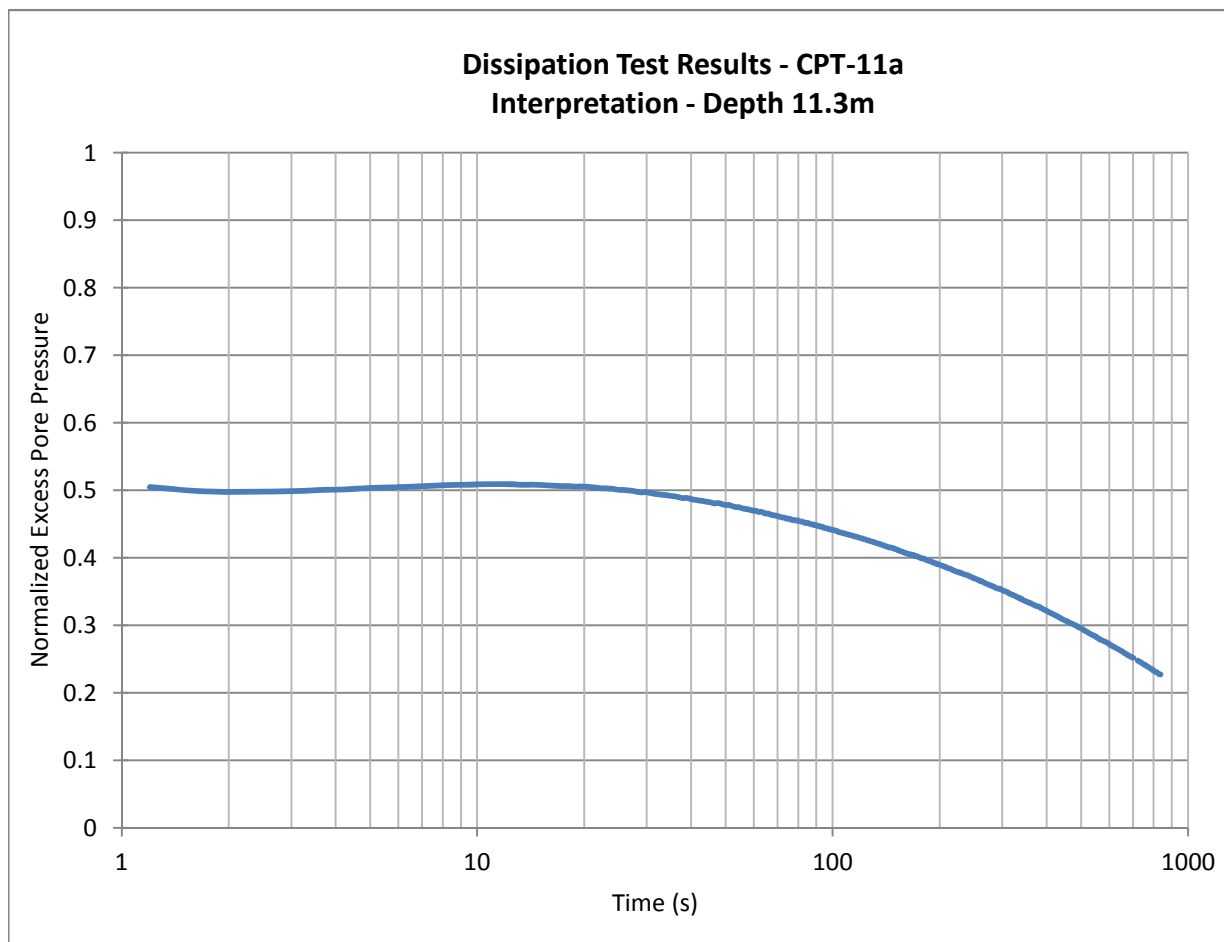
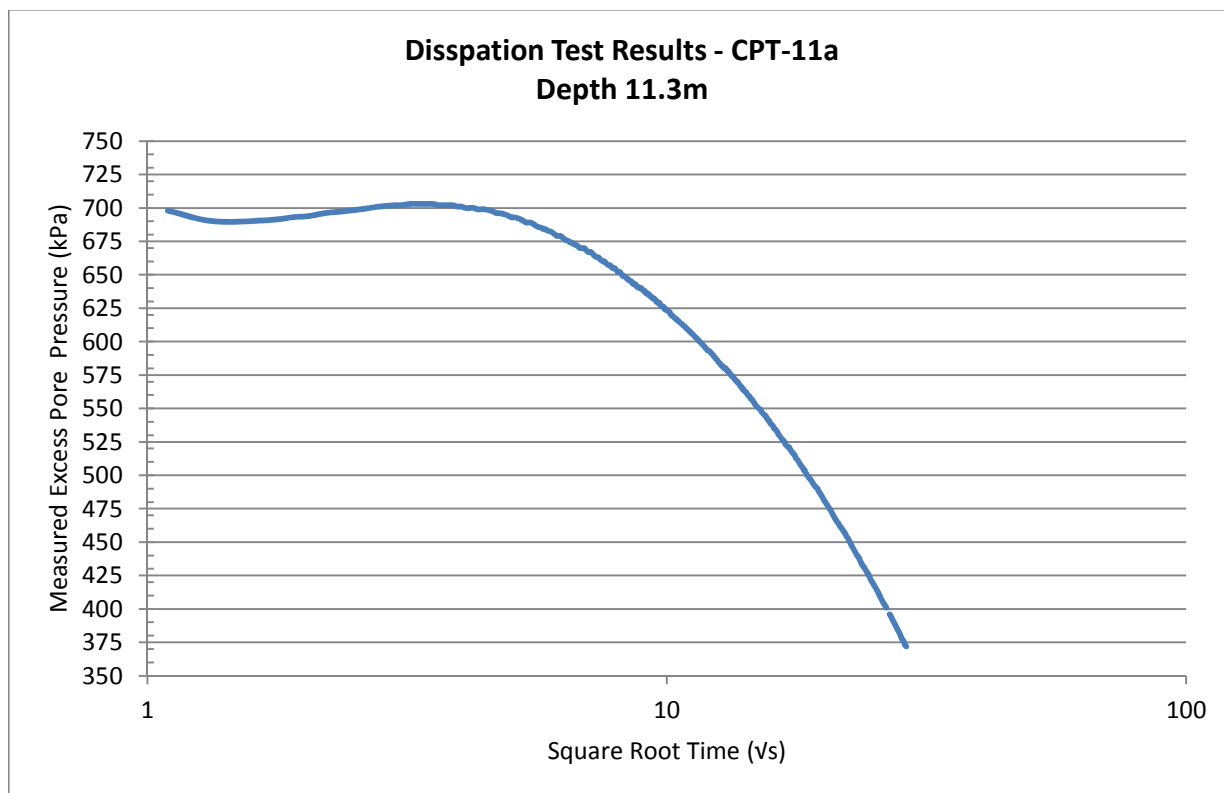
u_0 = pore water pressure at rest
Assumed hydrostatic within Fill and Silty Sand/Sand
Assumed to be 30% higher than hydrostatic in Silty Clay/Clayey Silt and Silt/Sandy Silt
due to artesian pressures in lower sands encountered in the adjacent boreholes

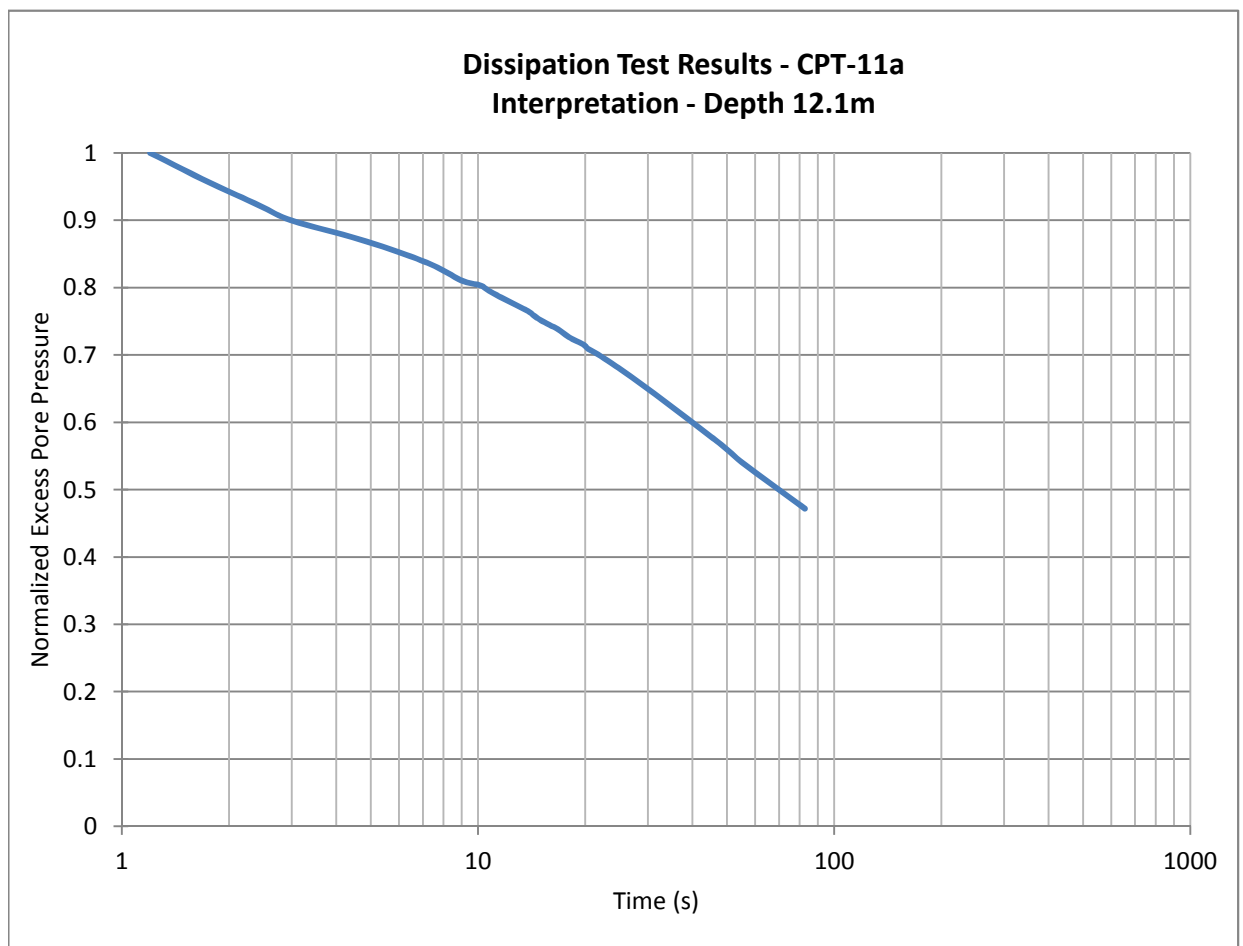
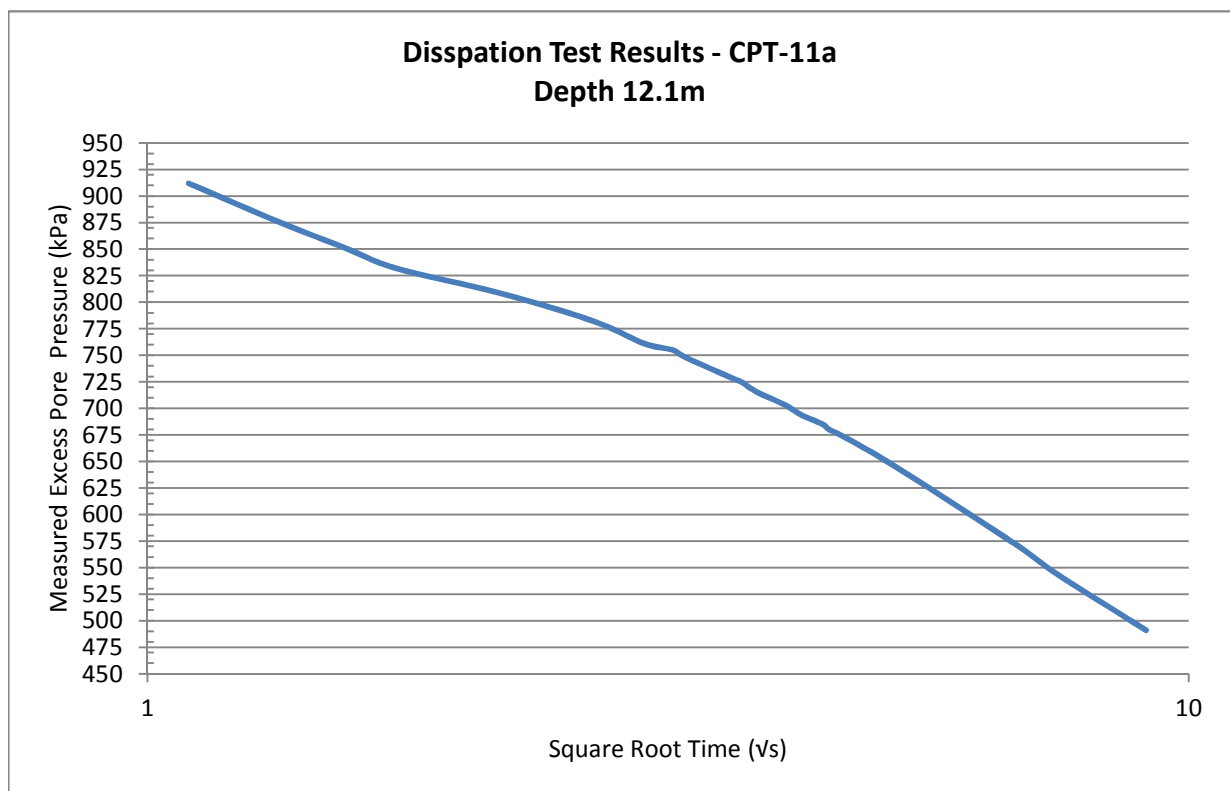
t_{50} = time for 50% excess pore water pressure dissipation

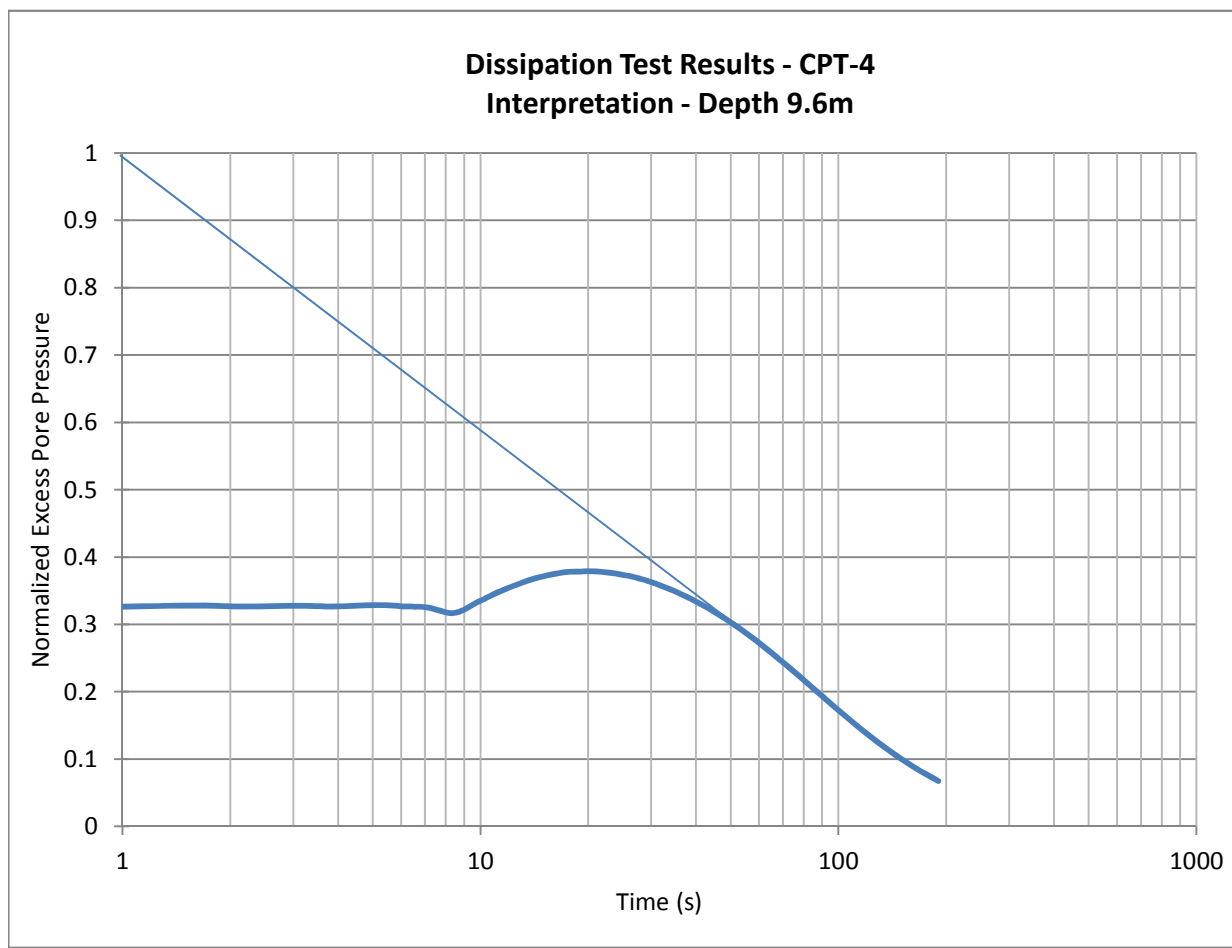
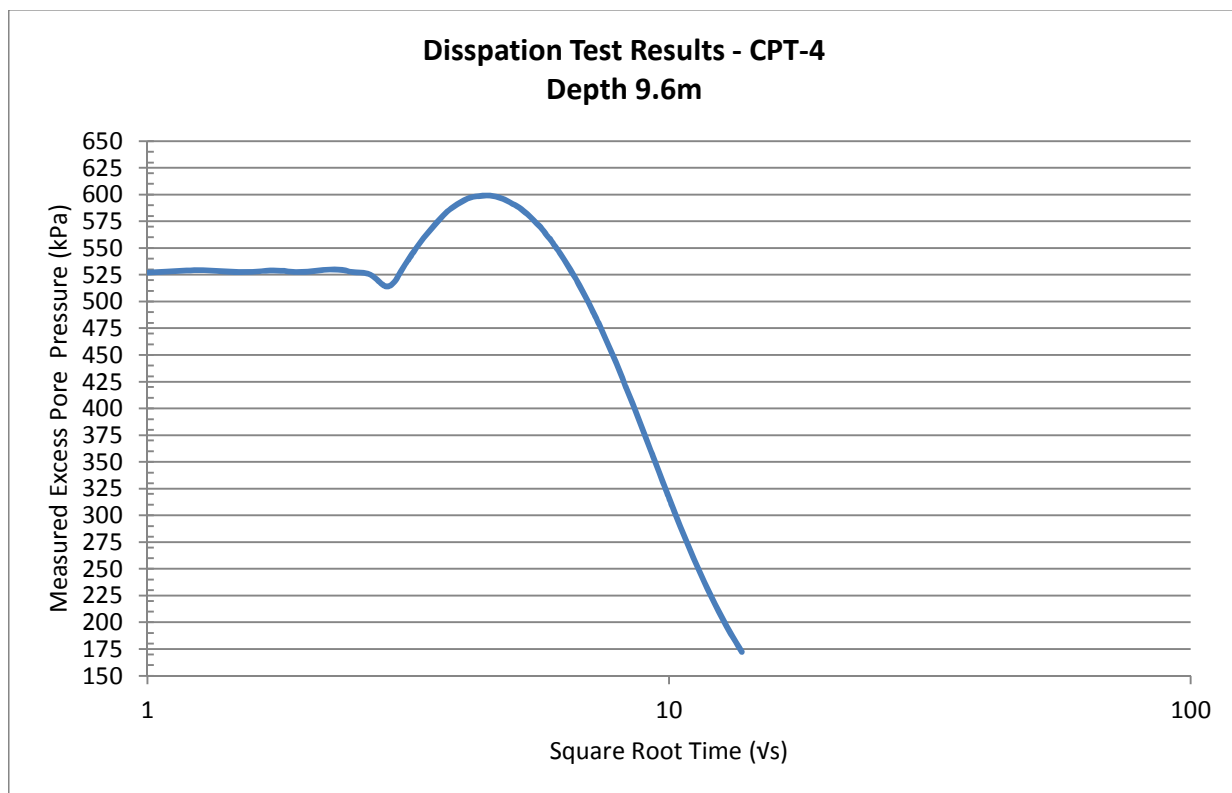
I_r = Undrained Rigidity Index = Shear Modulus/Undrained Shear Strength

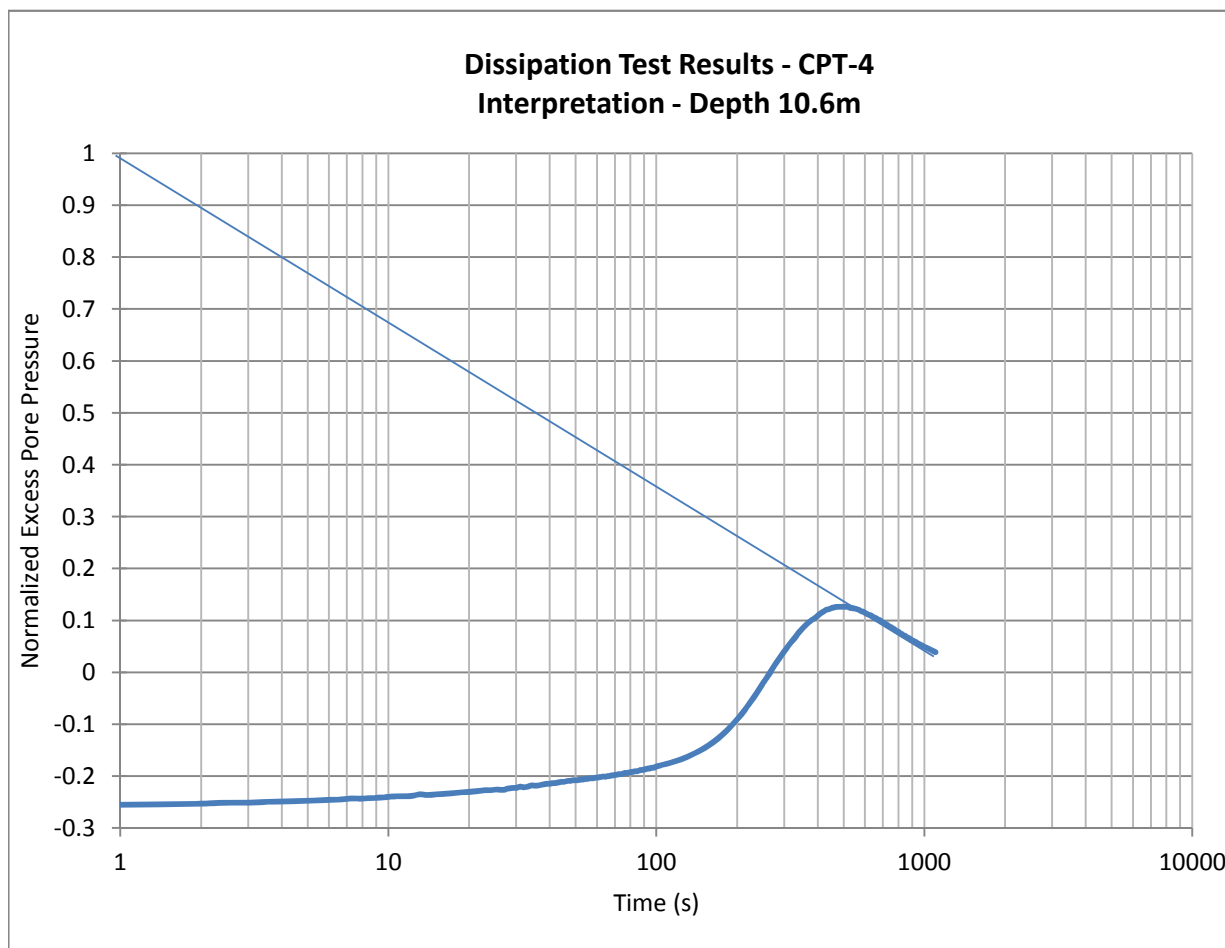
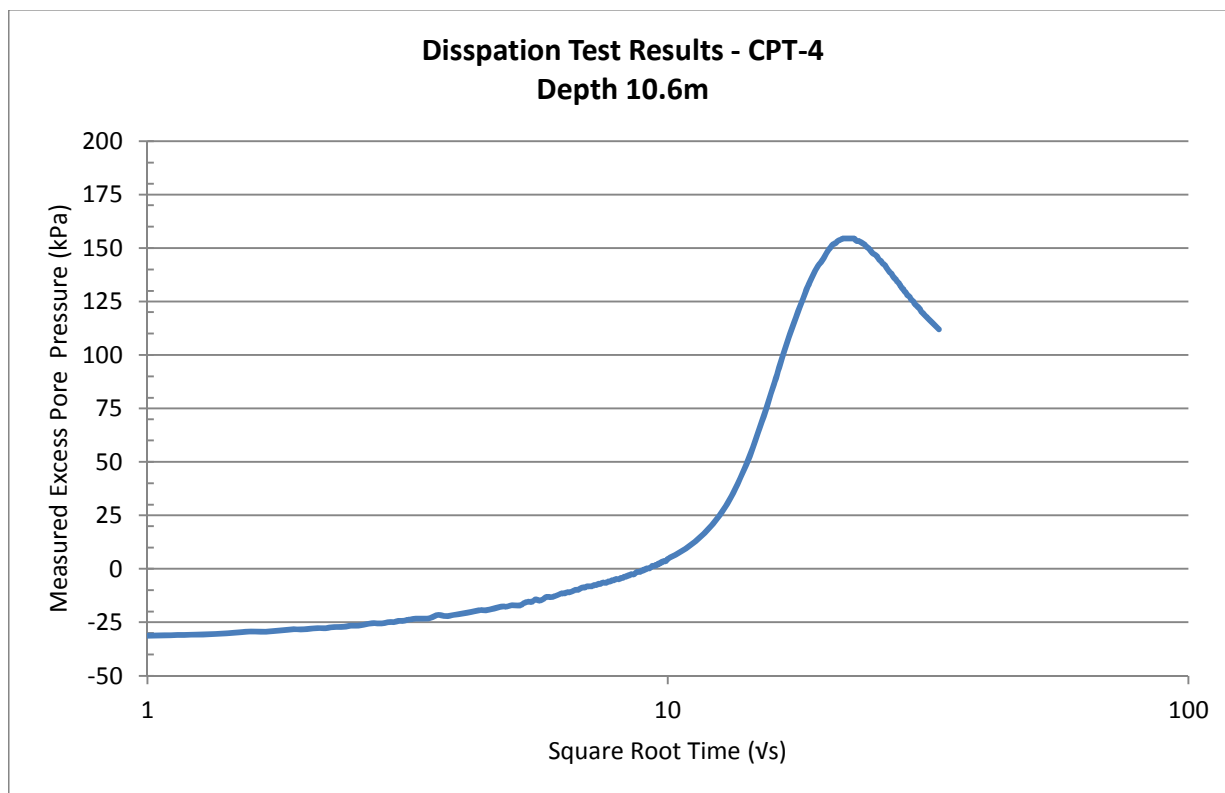
Inferred shear modulus (G) = E/3

Assumed Bulk Unit Weight (γ)
Fill ~ 18 kN/m³
Silty Sand/Sand ~ 20 kN/m³
Silty Clay/Clayey Silt ~ 19.5 kN/m³
Silt/Sandy Silt ~ 19.5 kN/m³

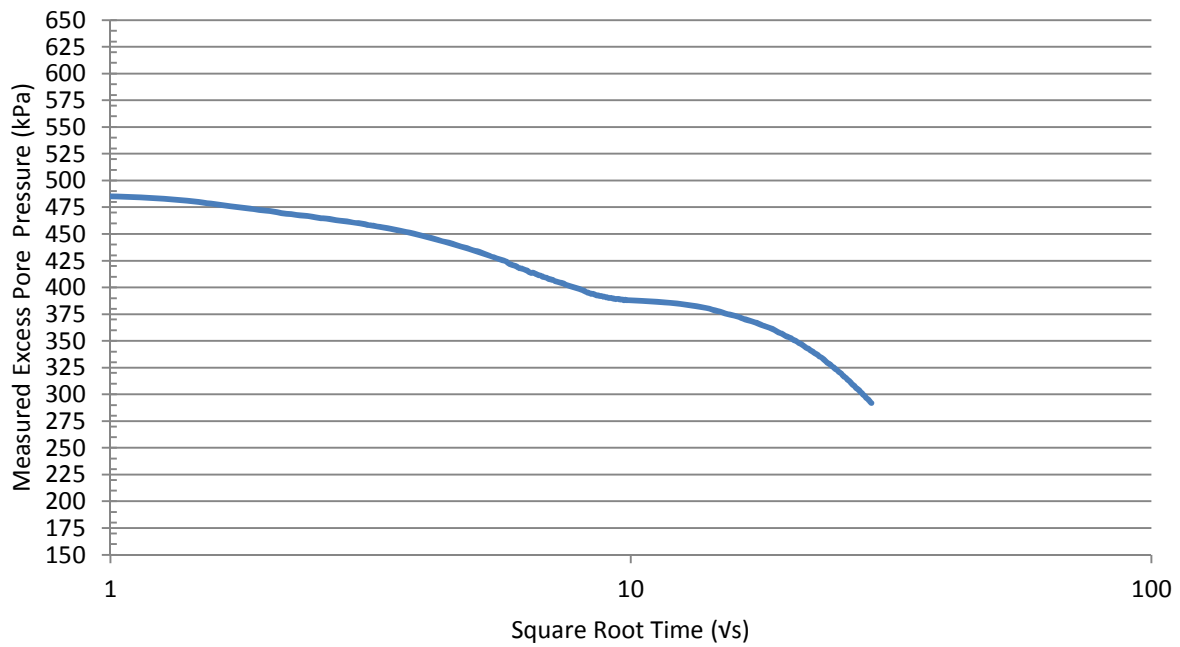




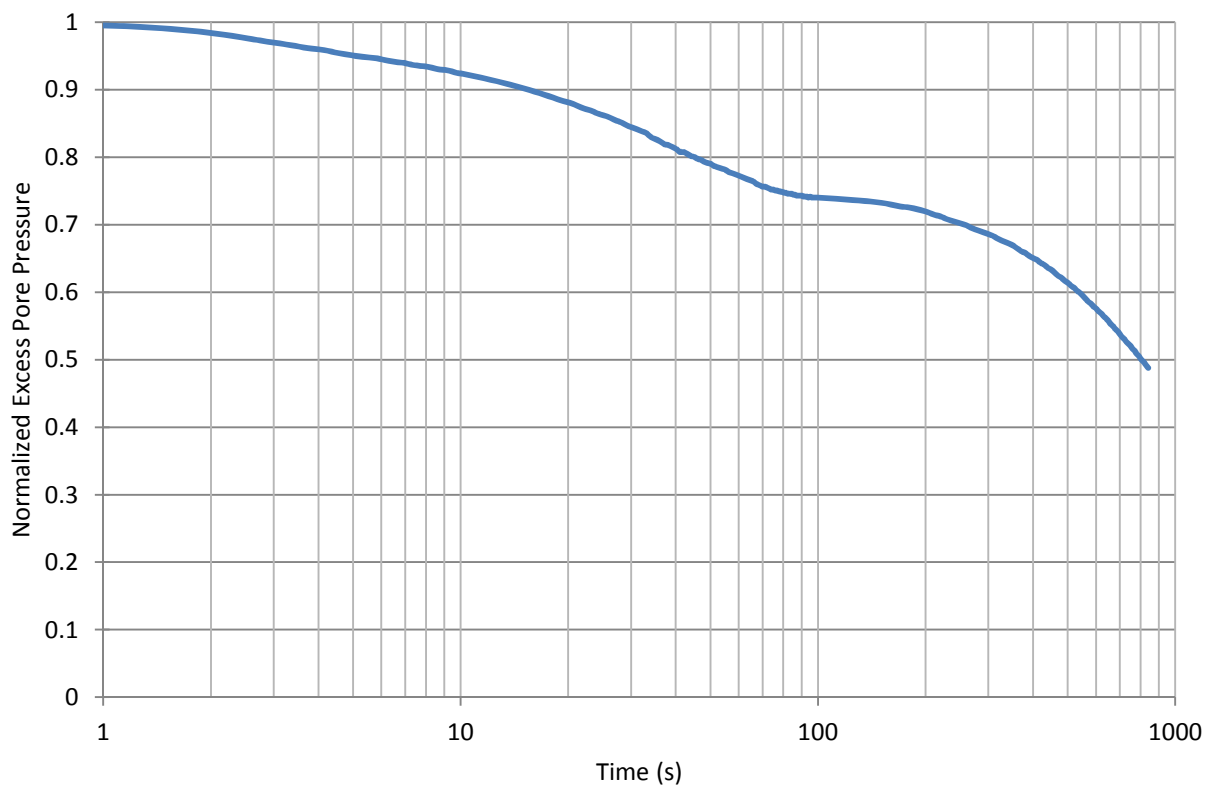


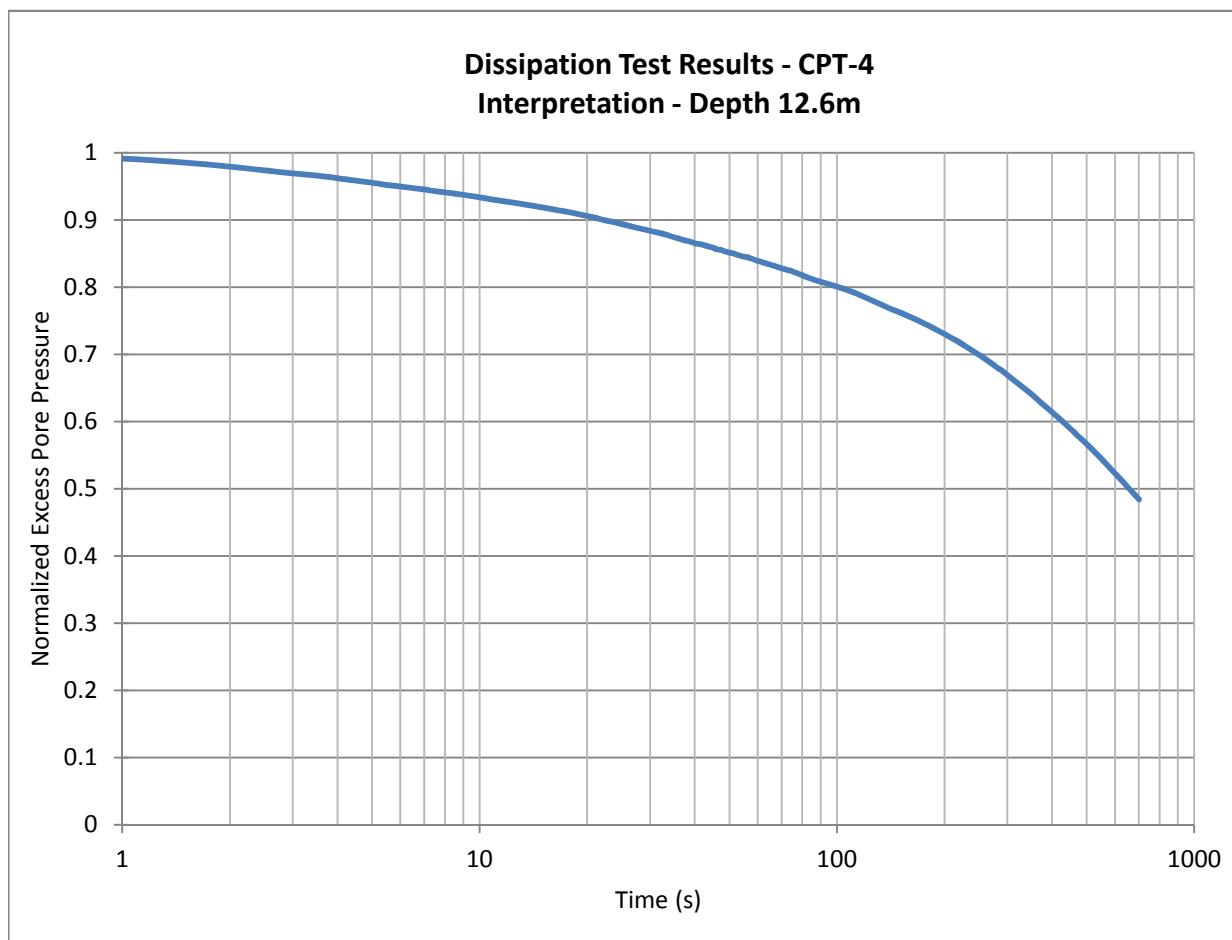
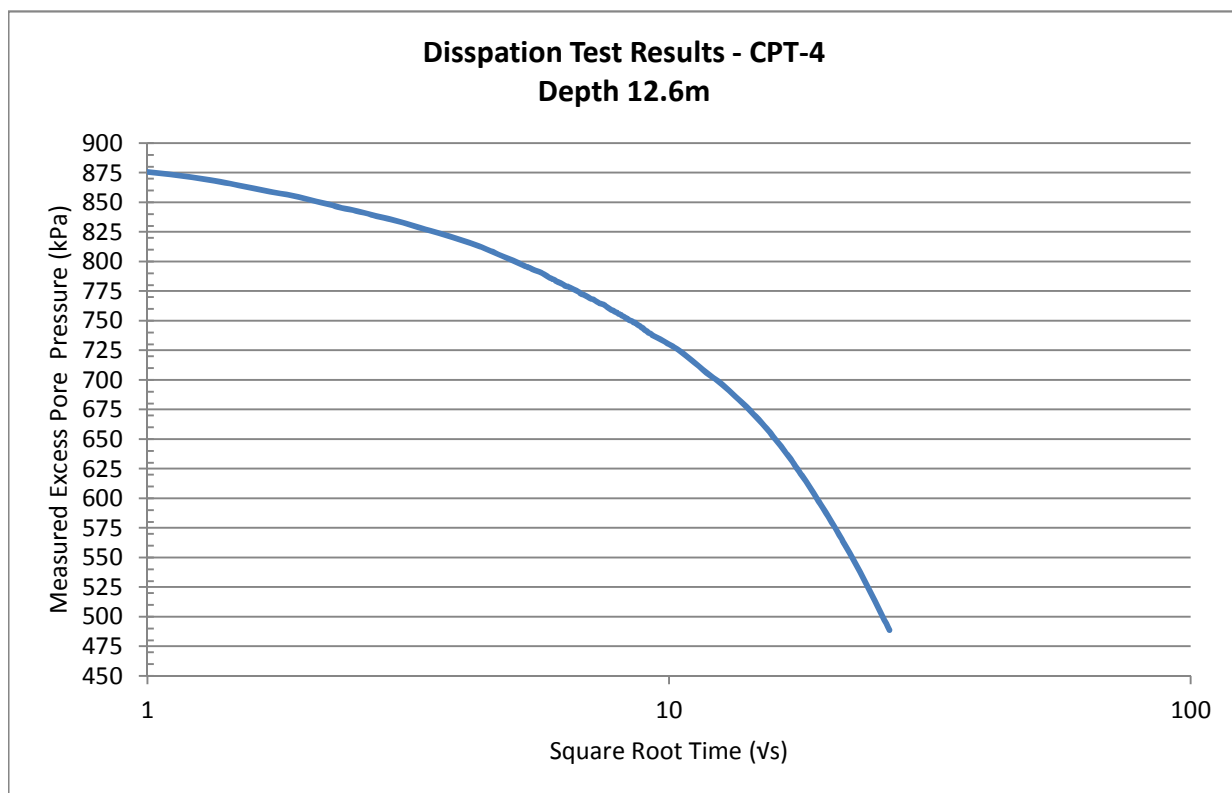


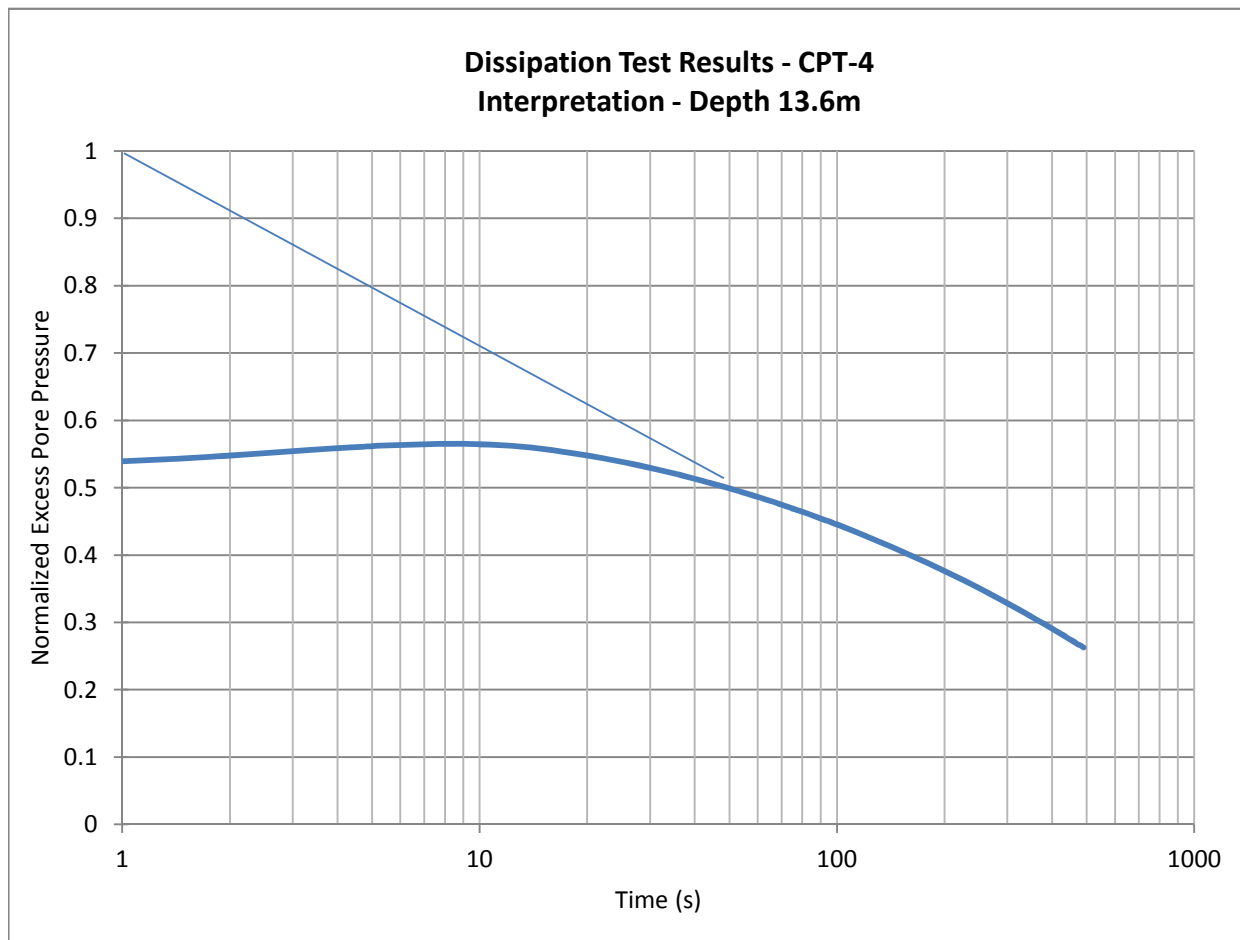
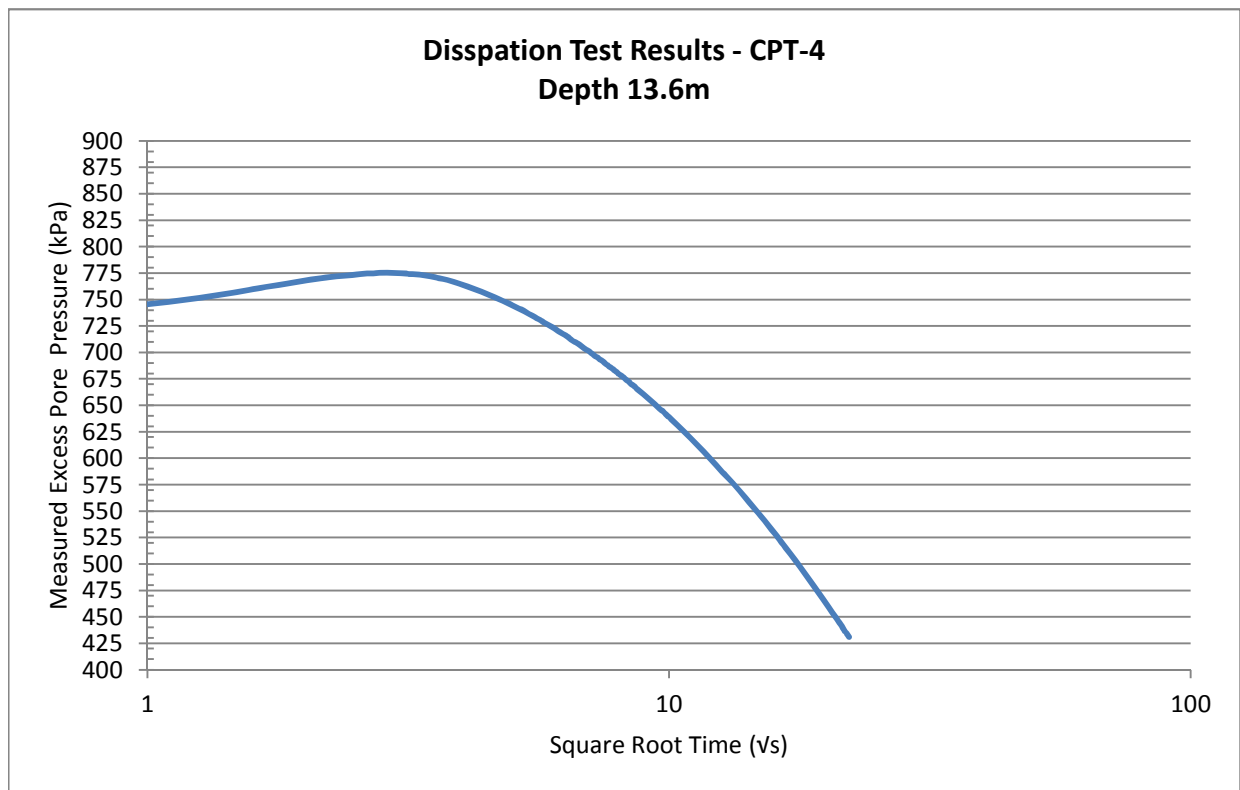
Dissipation Test Results - CPT-4
Depth 11.6m

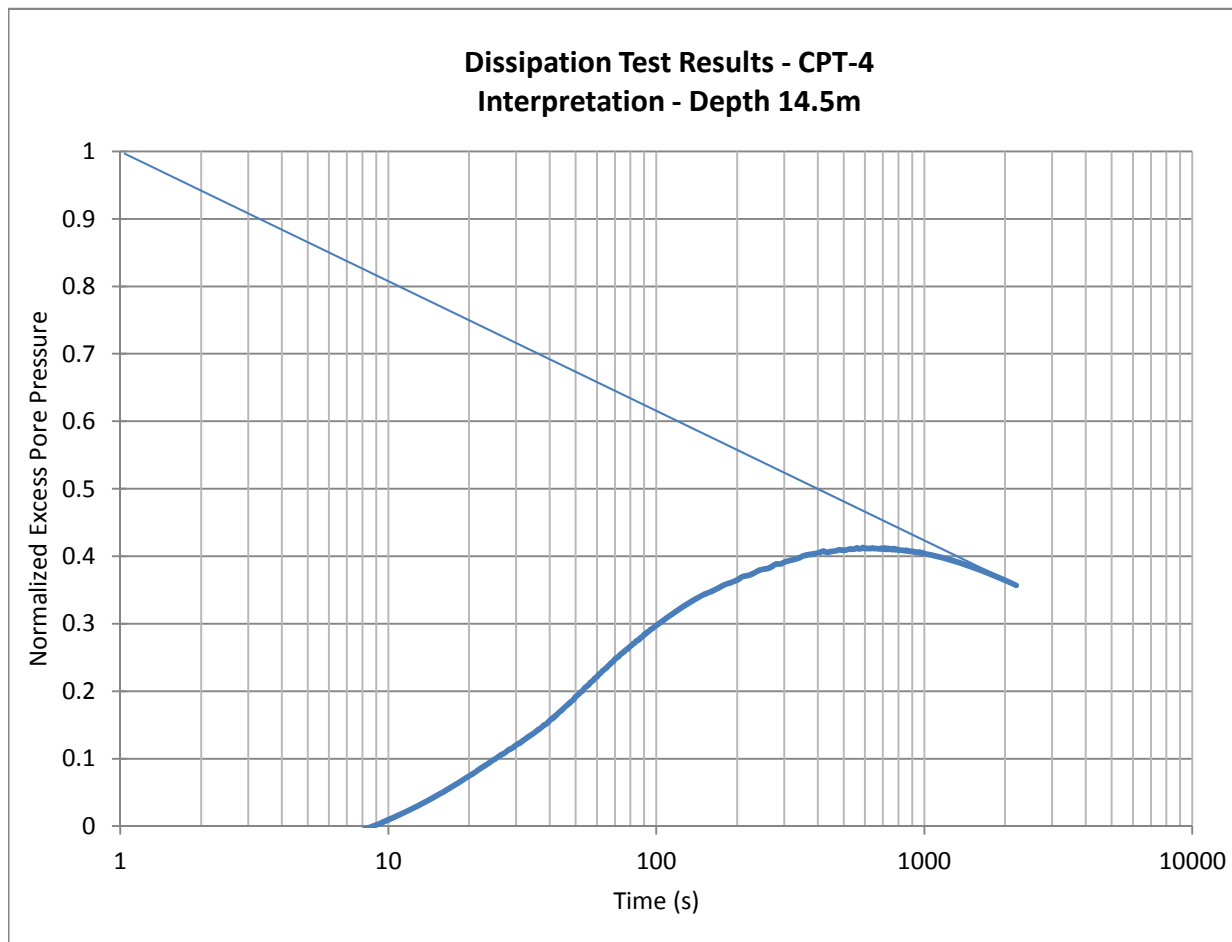
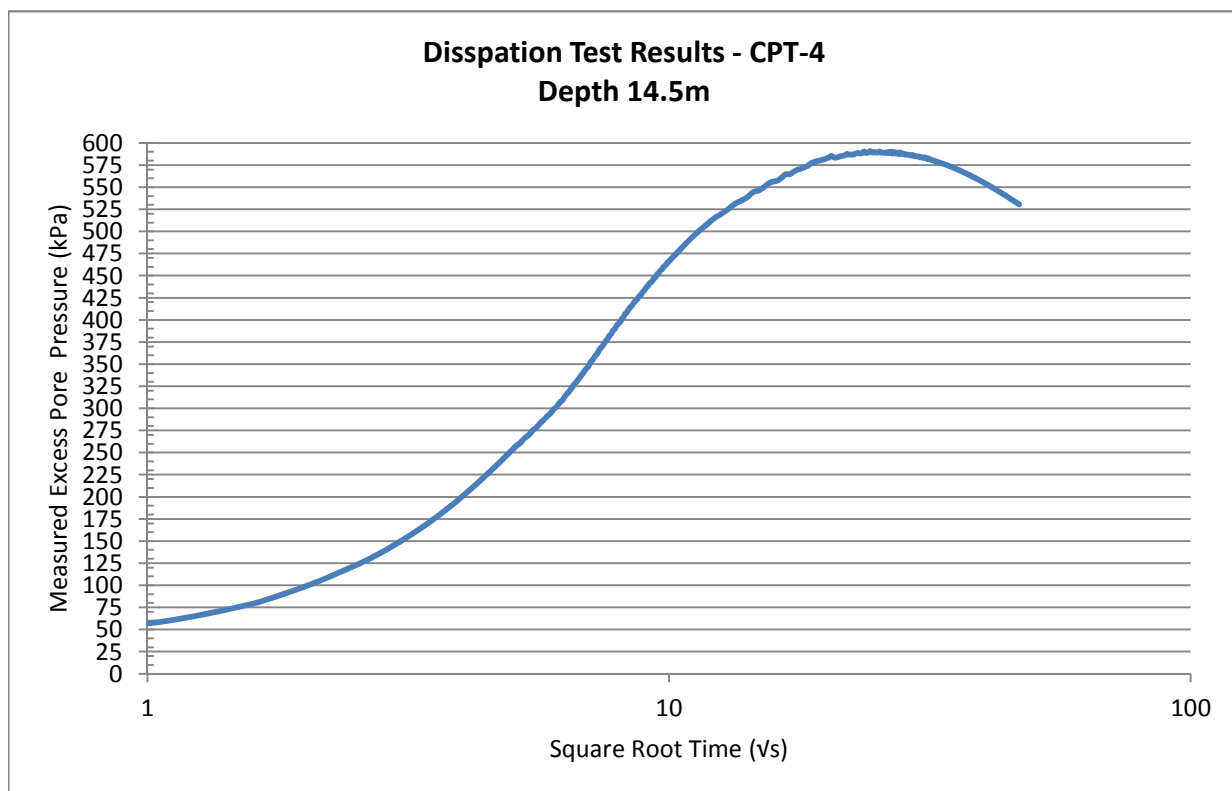


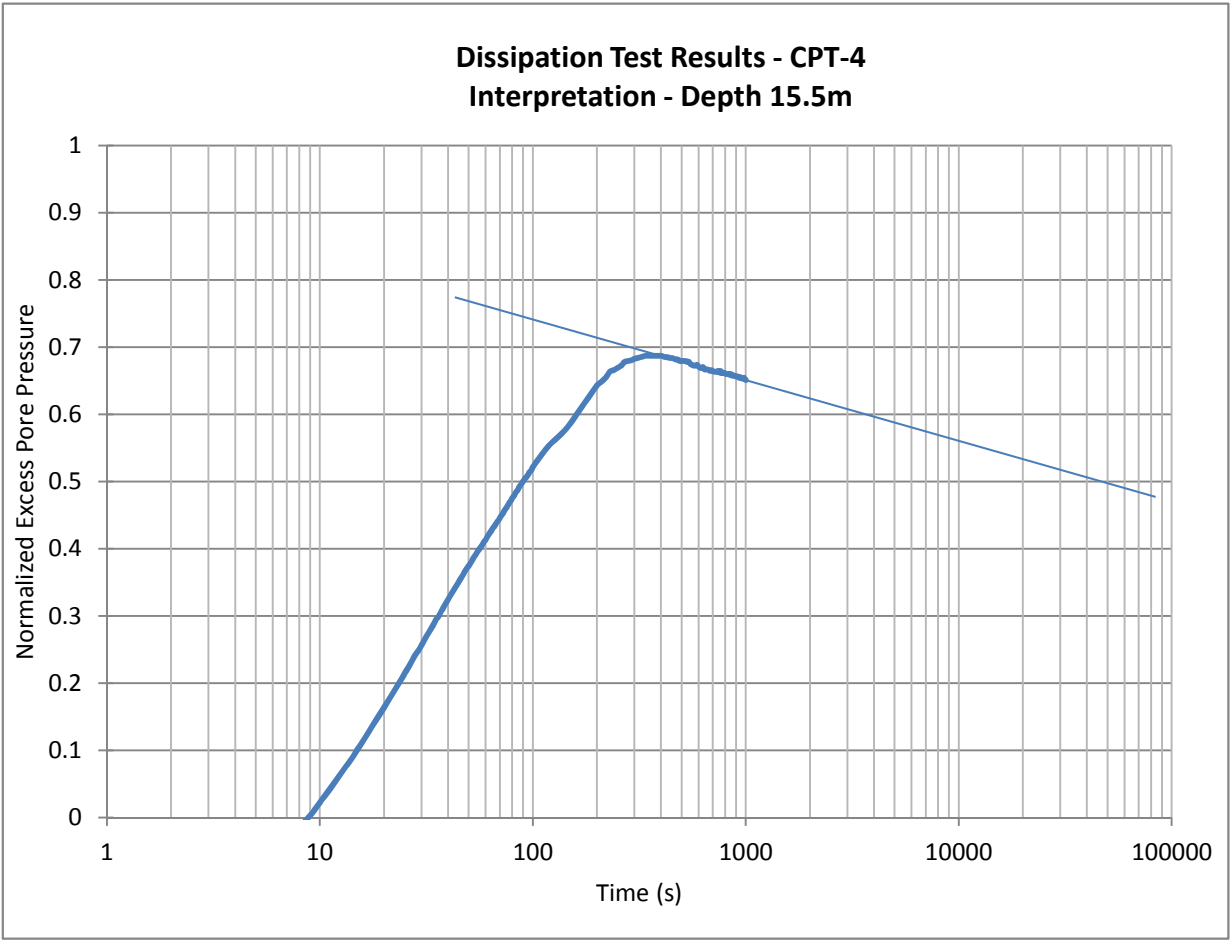
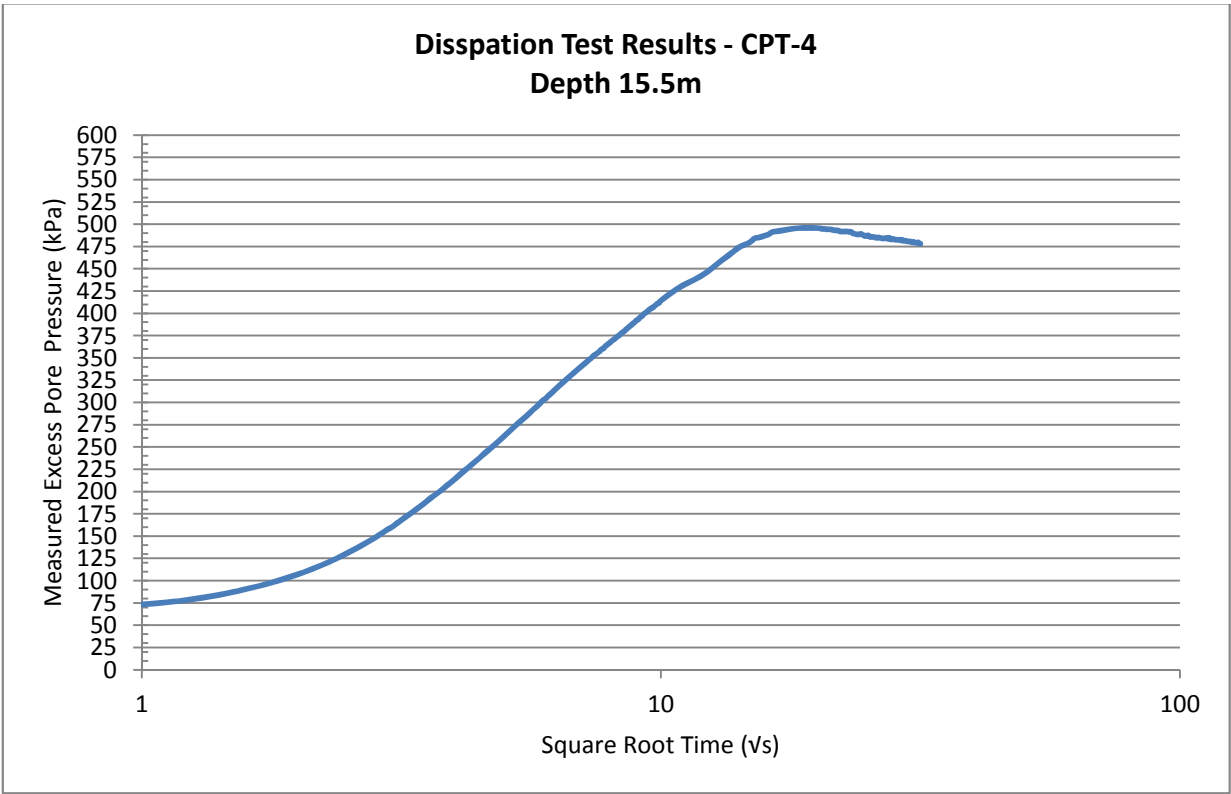
Dissipation Test Results - CPT-4
Interpretation - Depth 11.6m

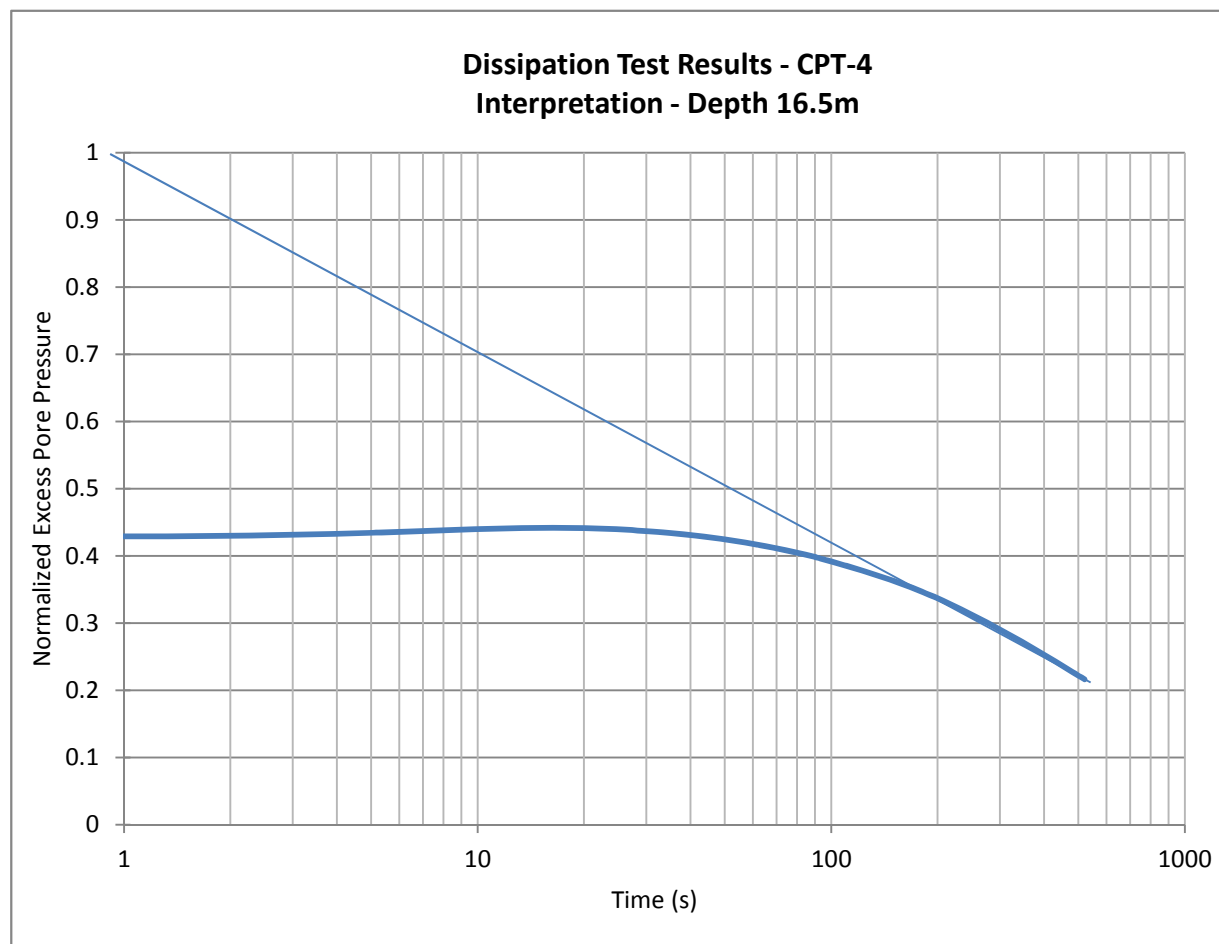
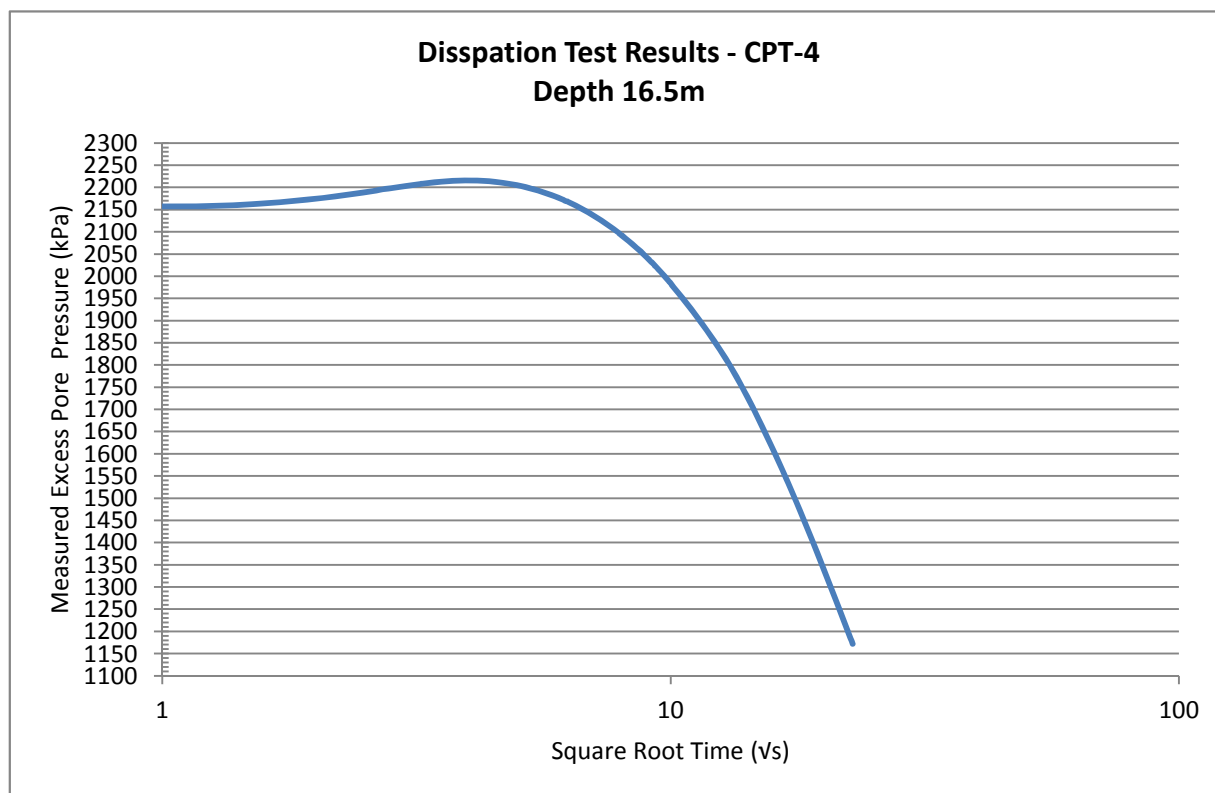


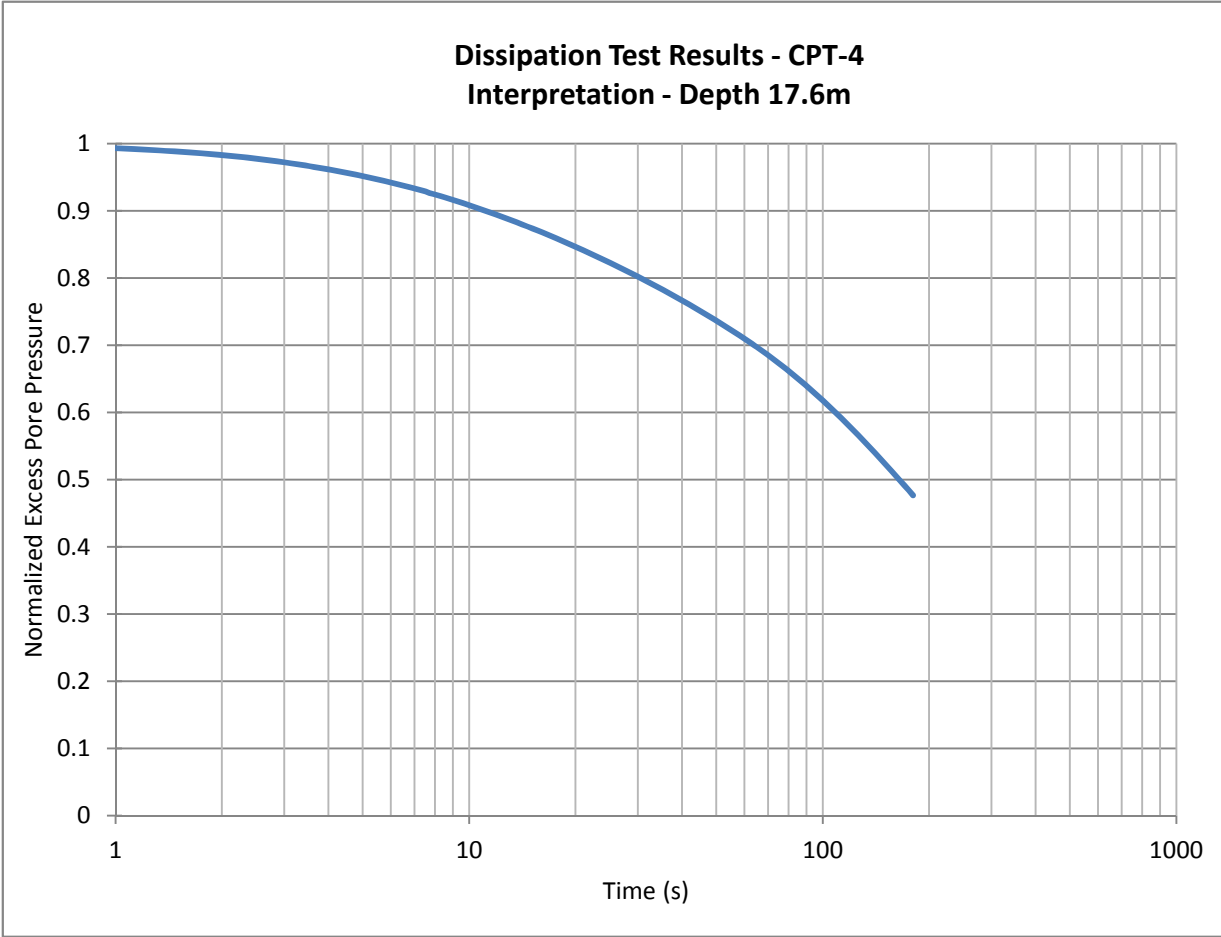
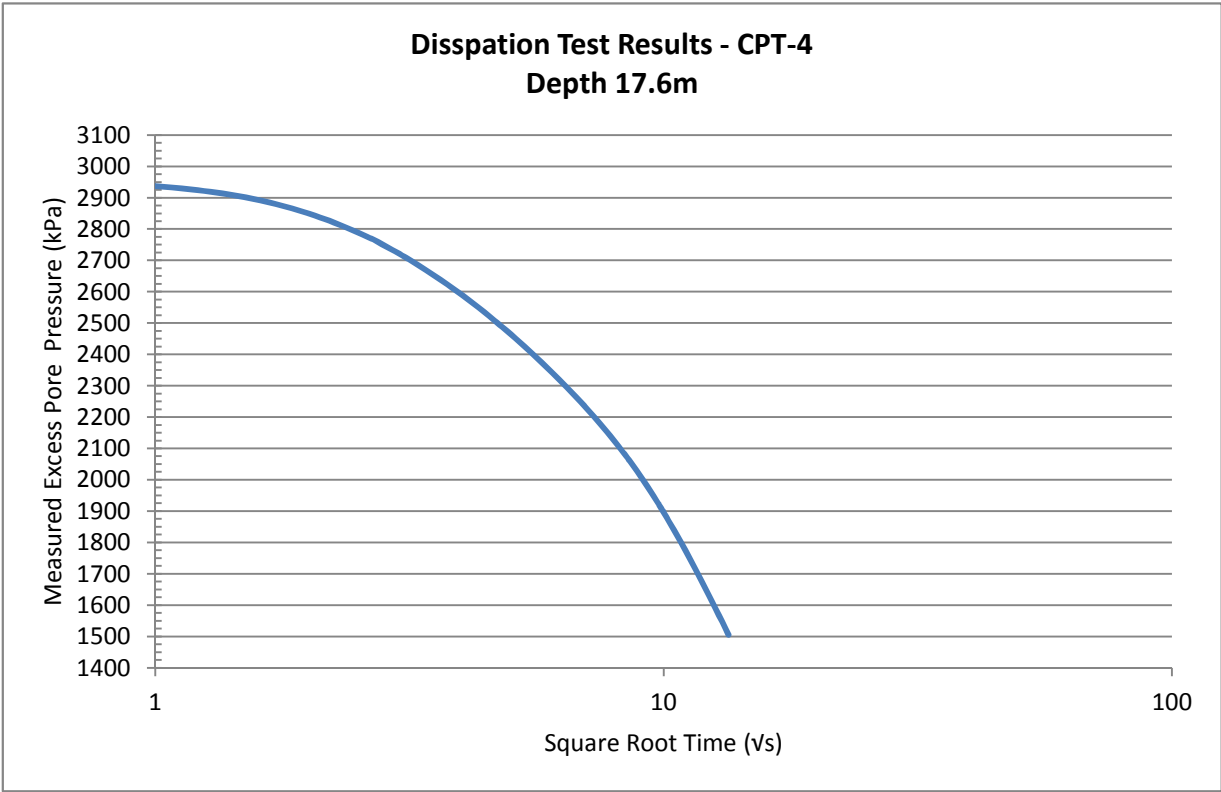


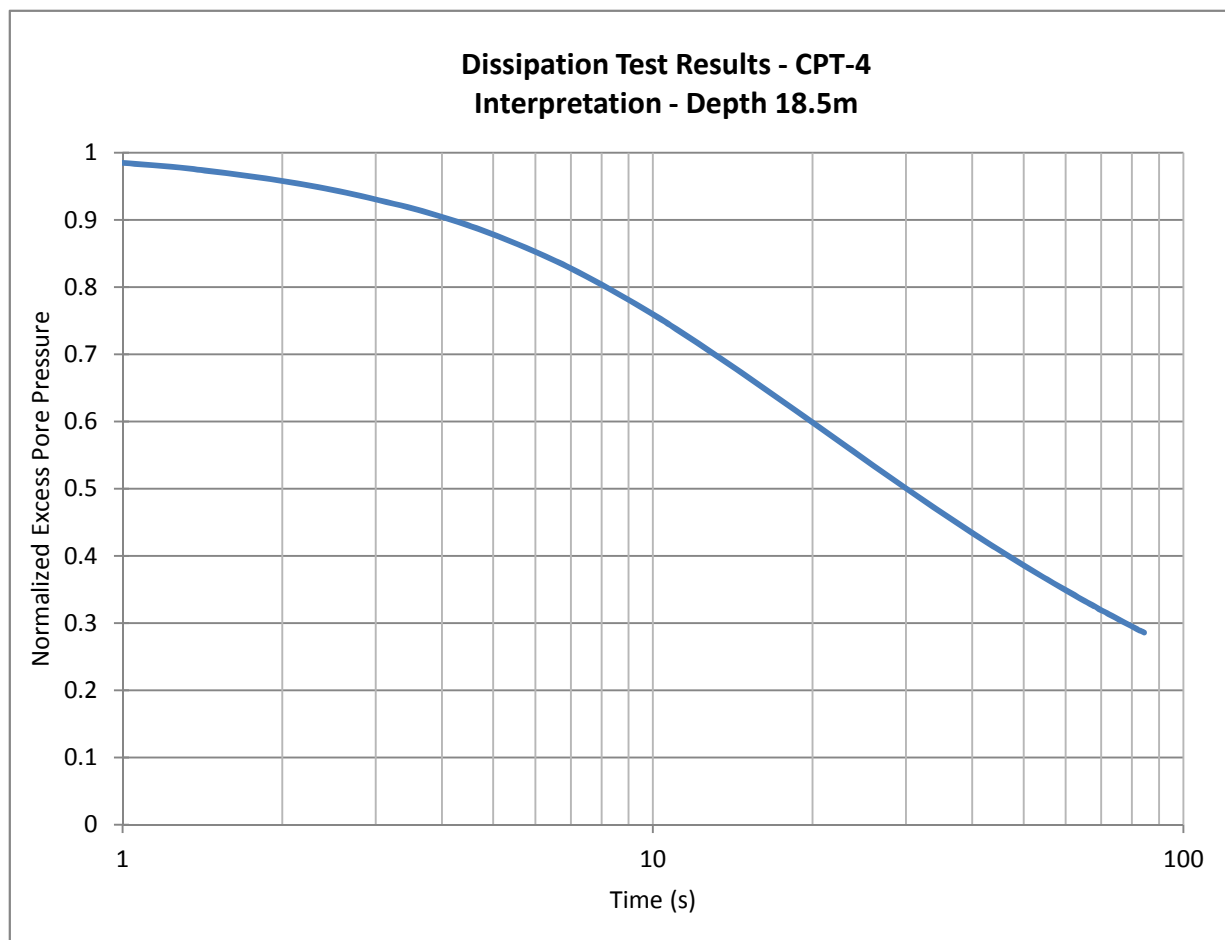
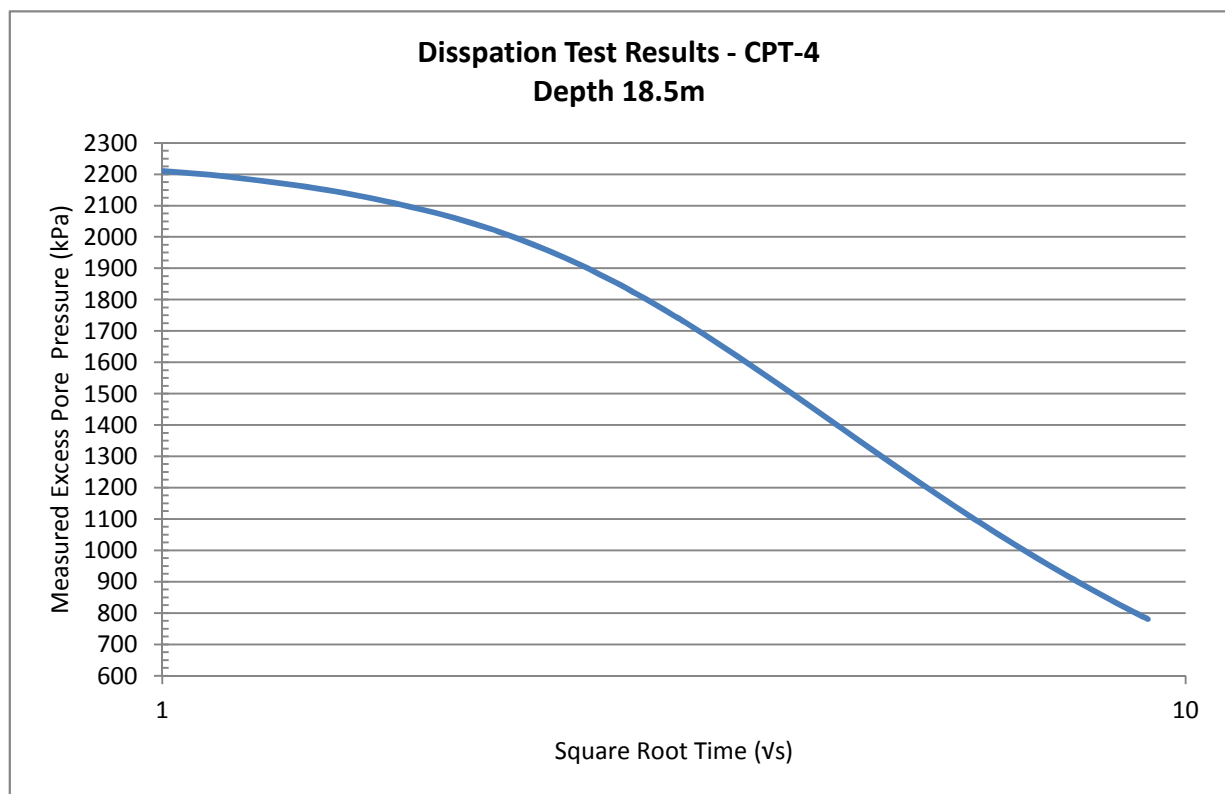


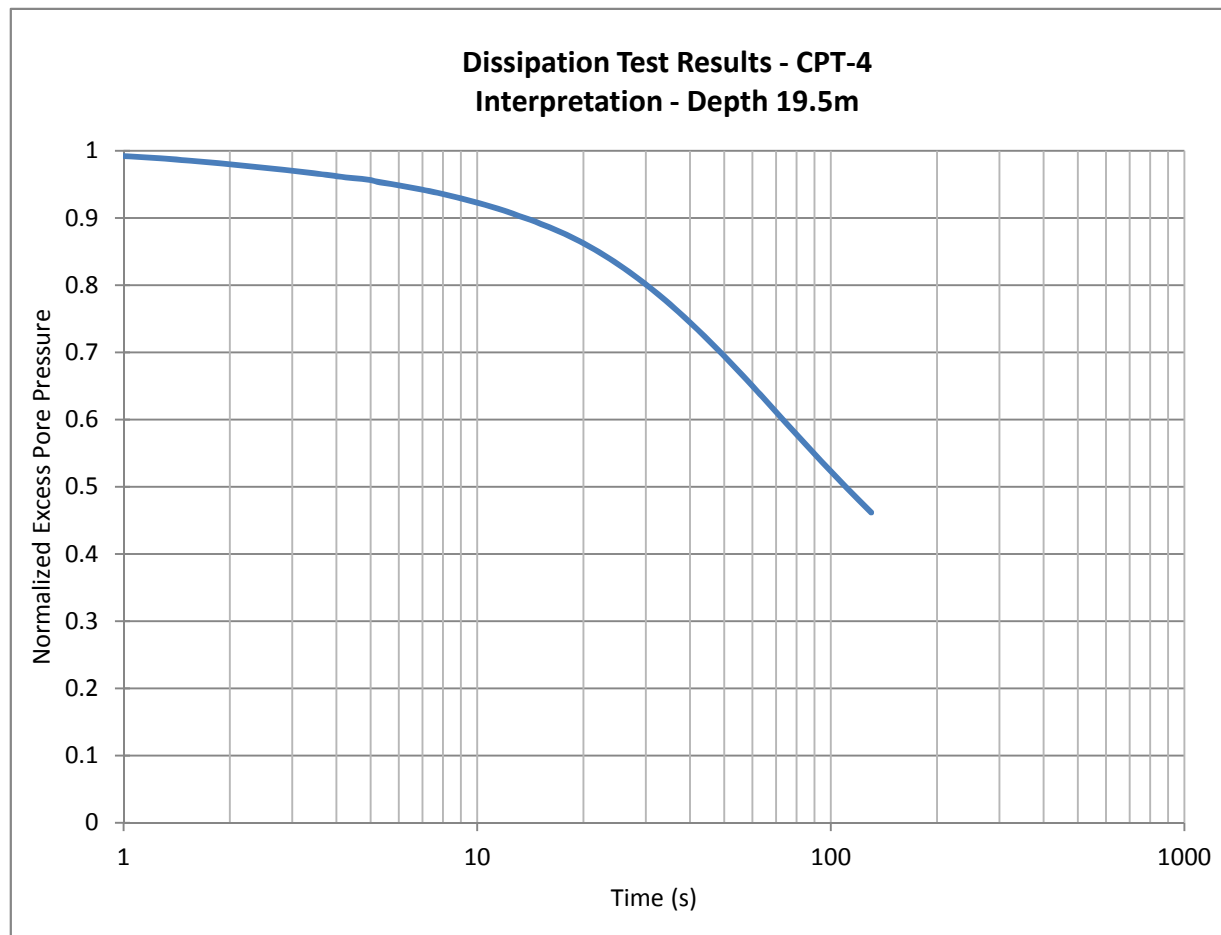
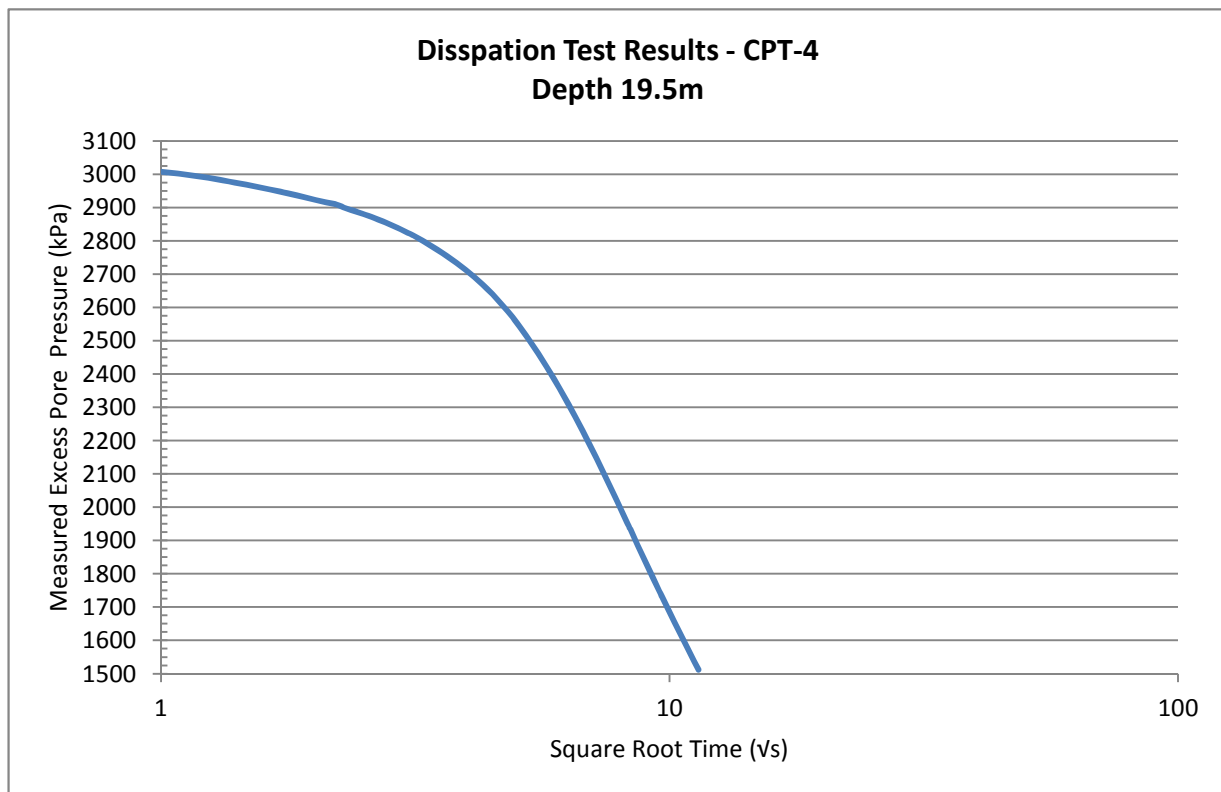


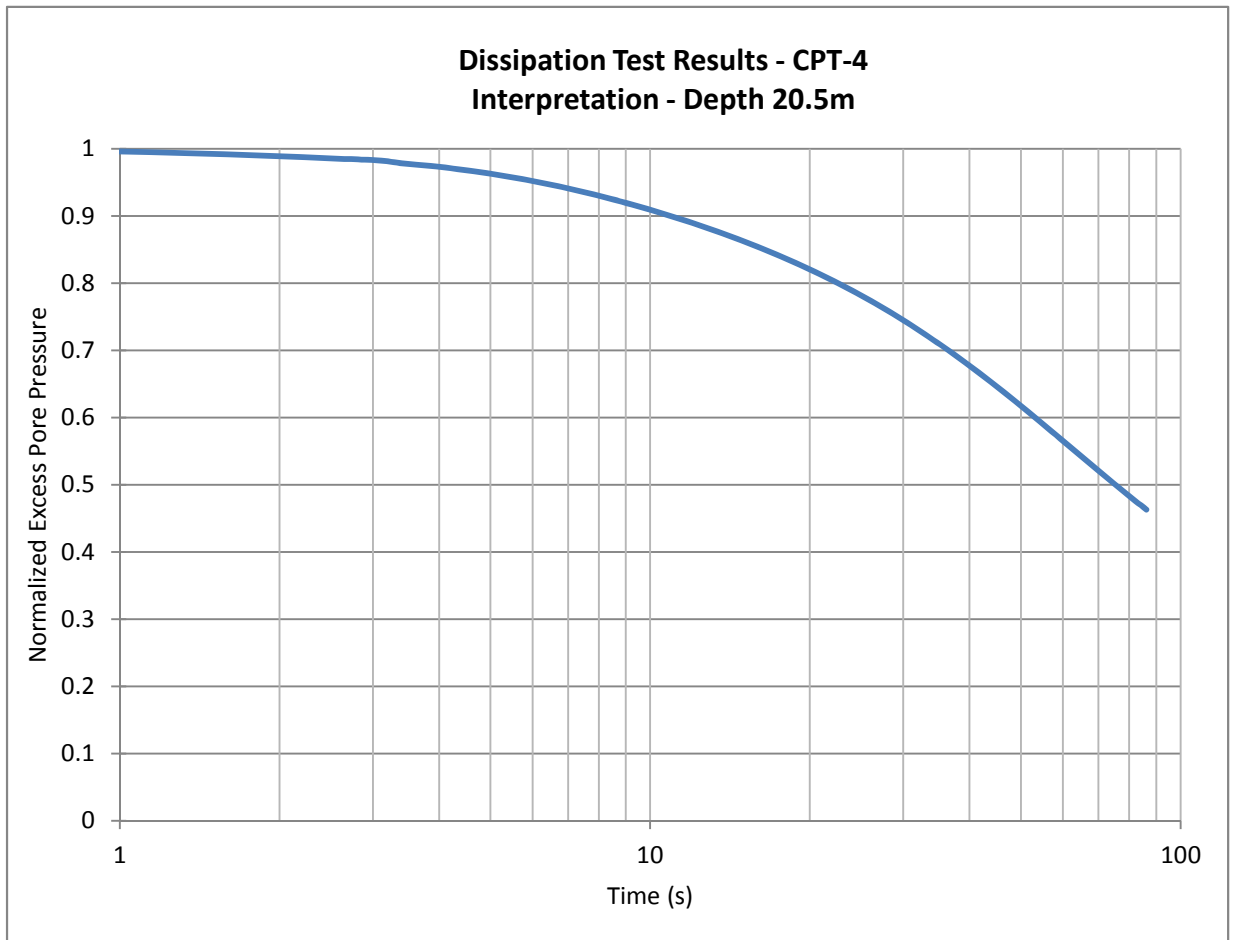
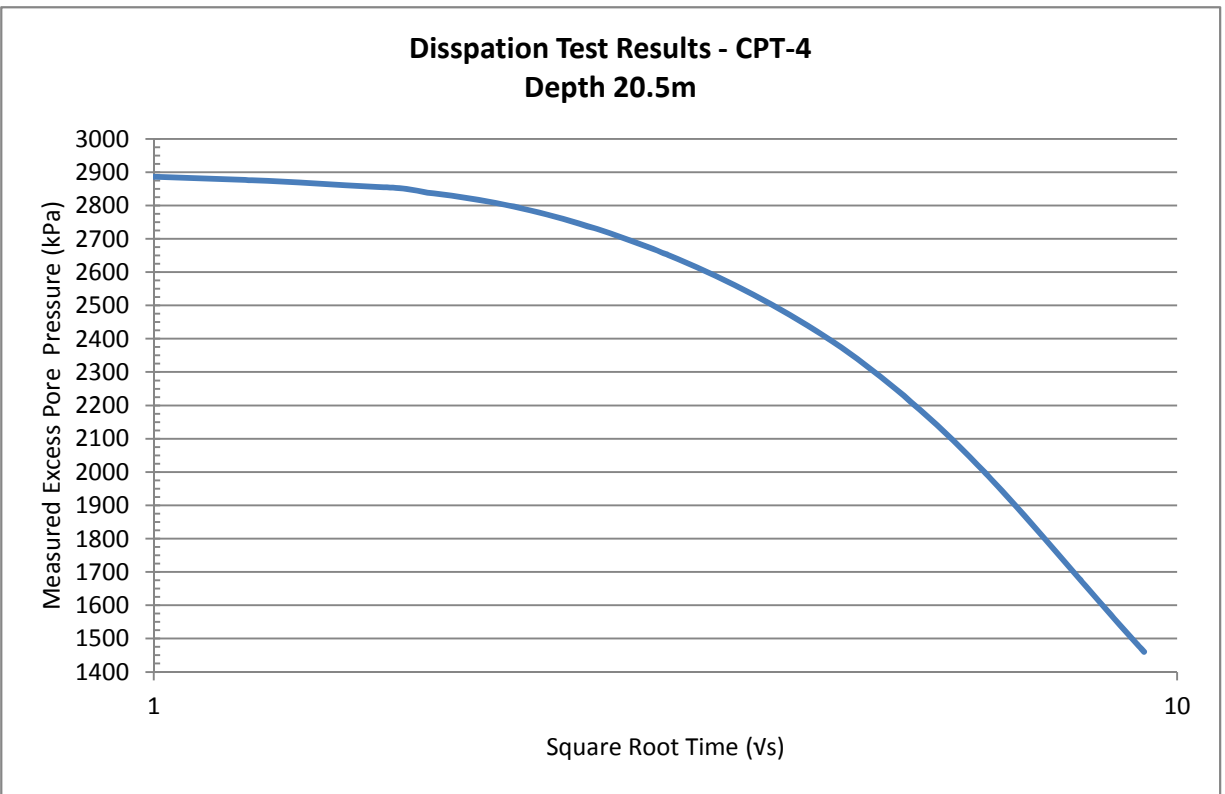


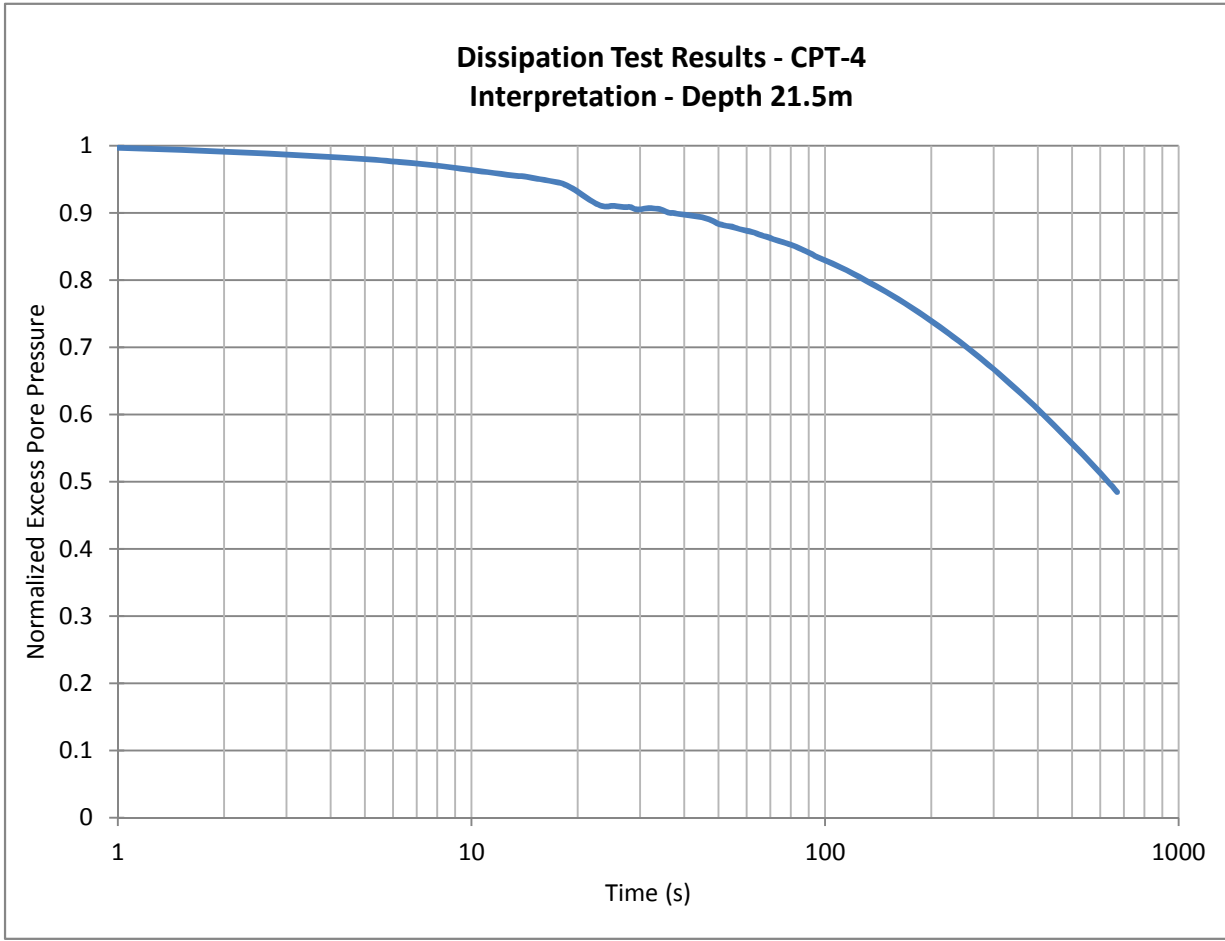
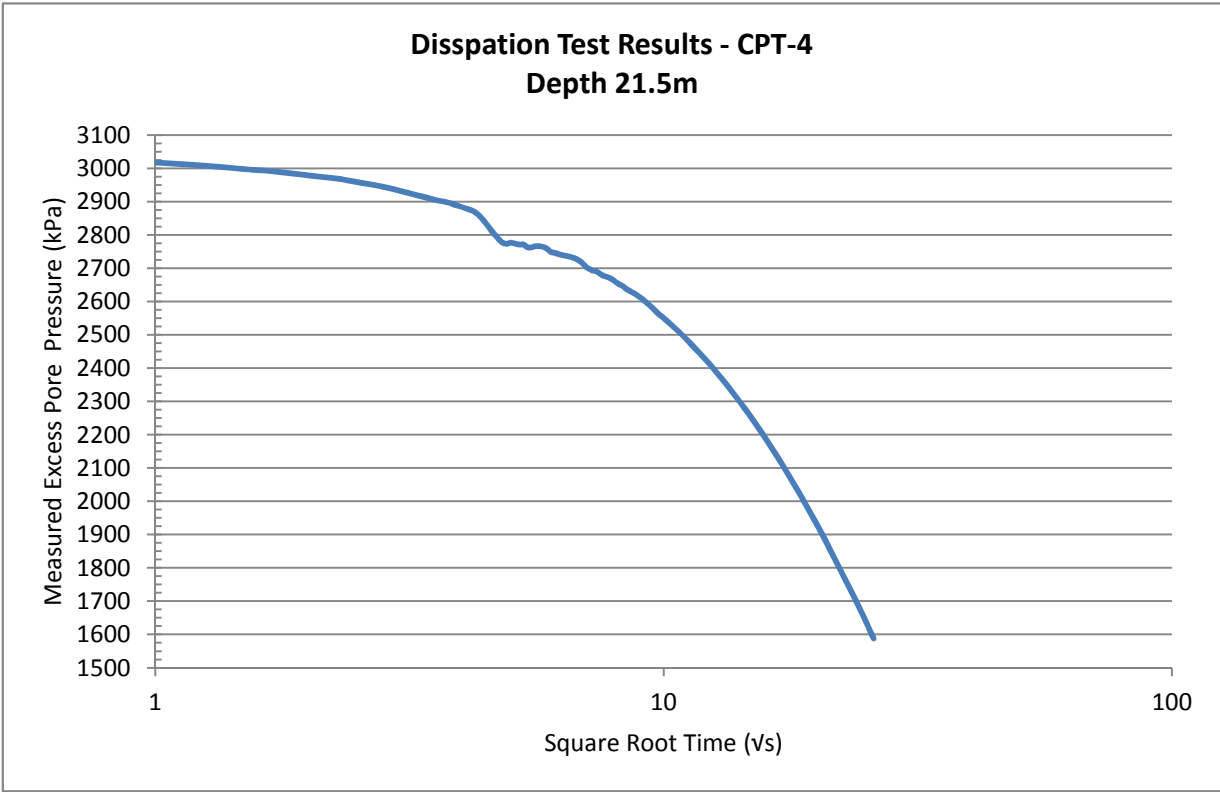


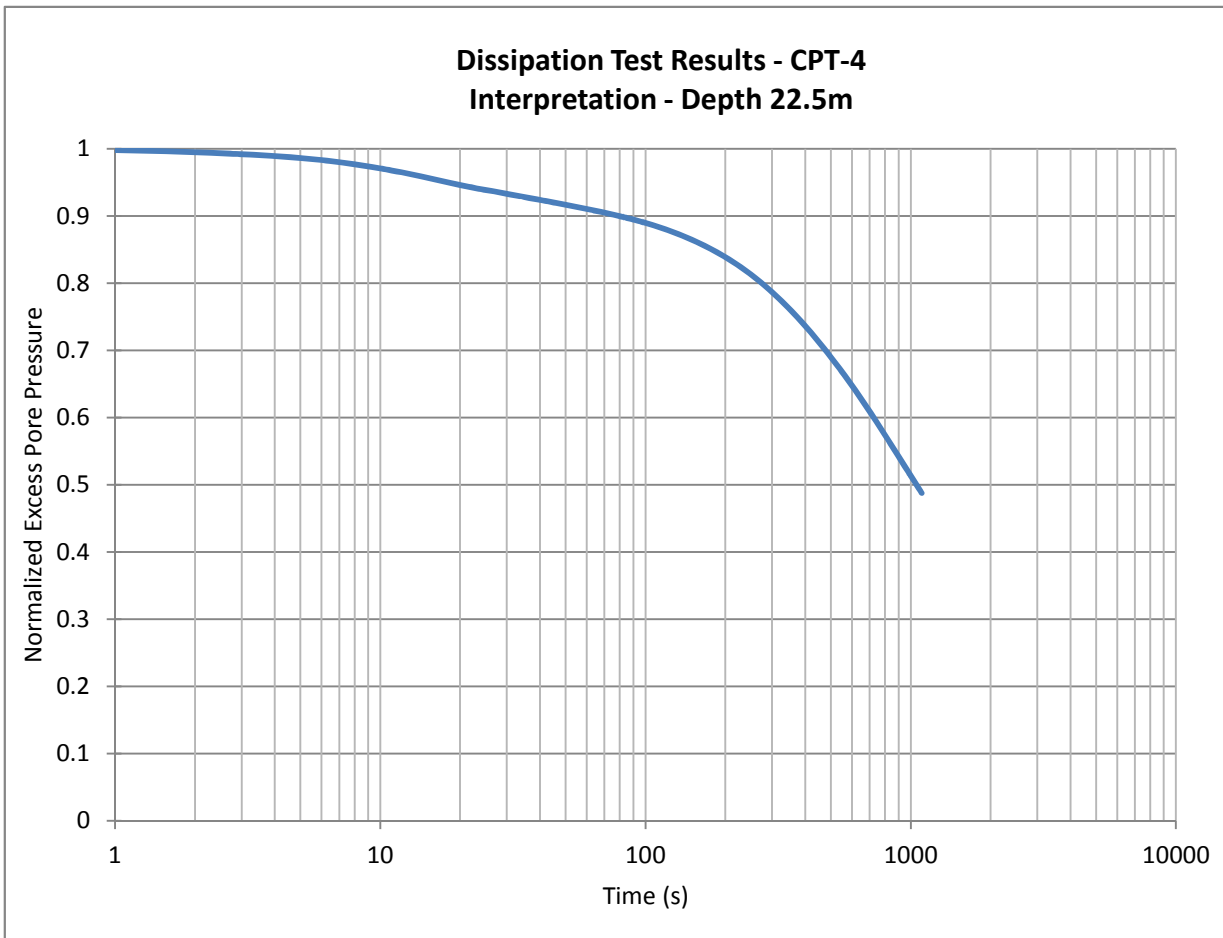
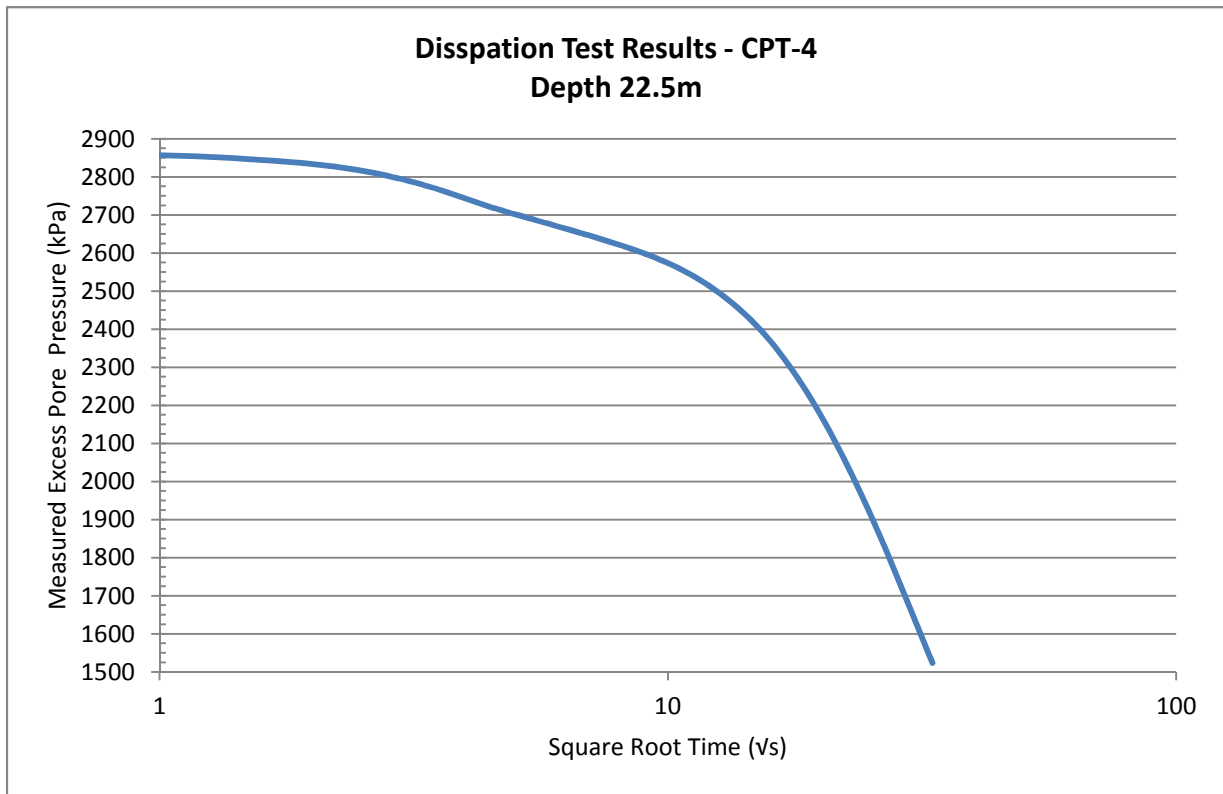


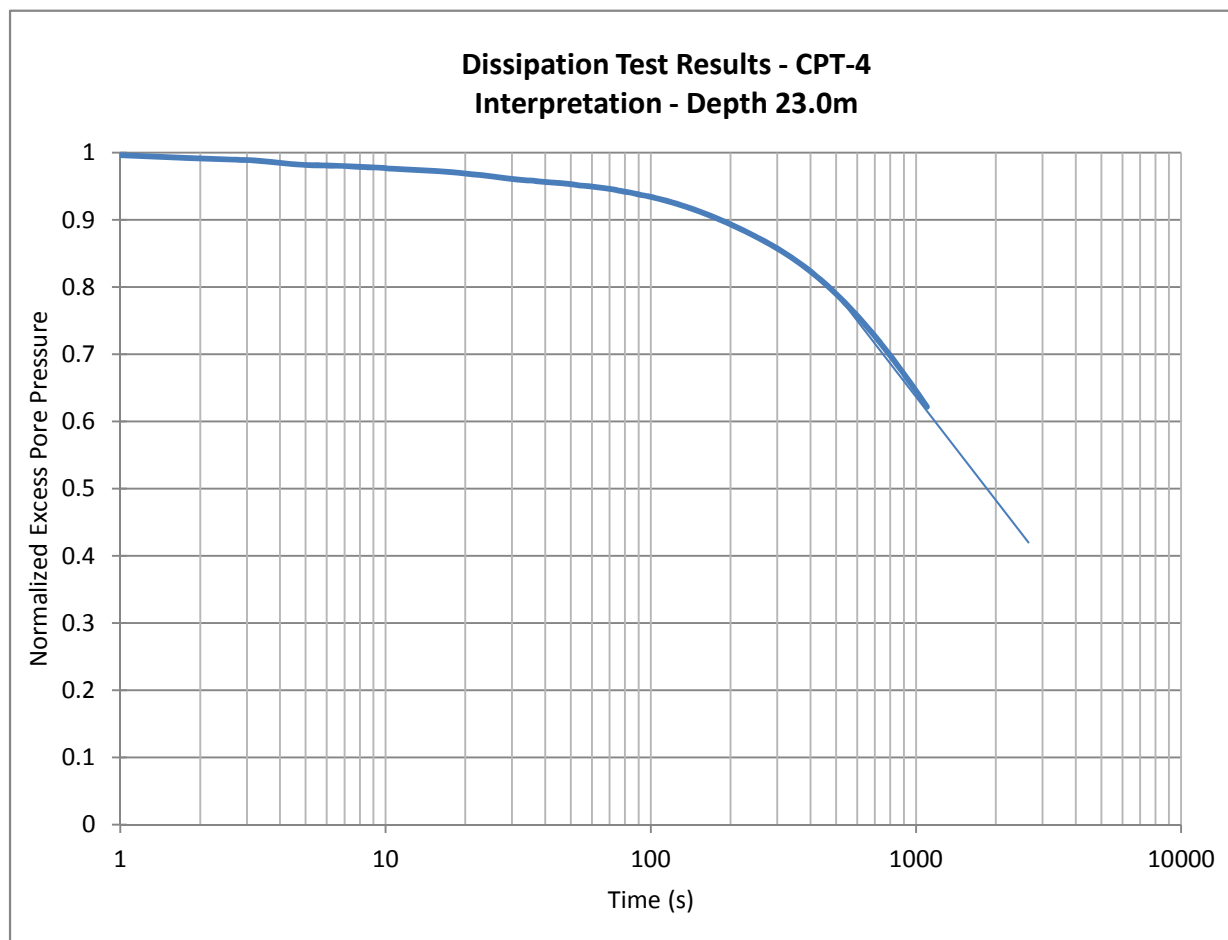
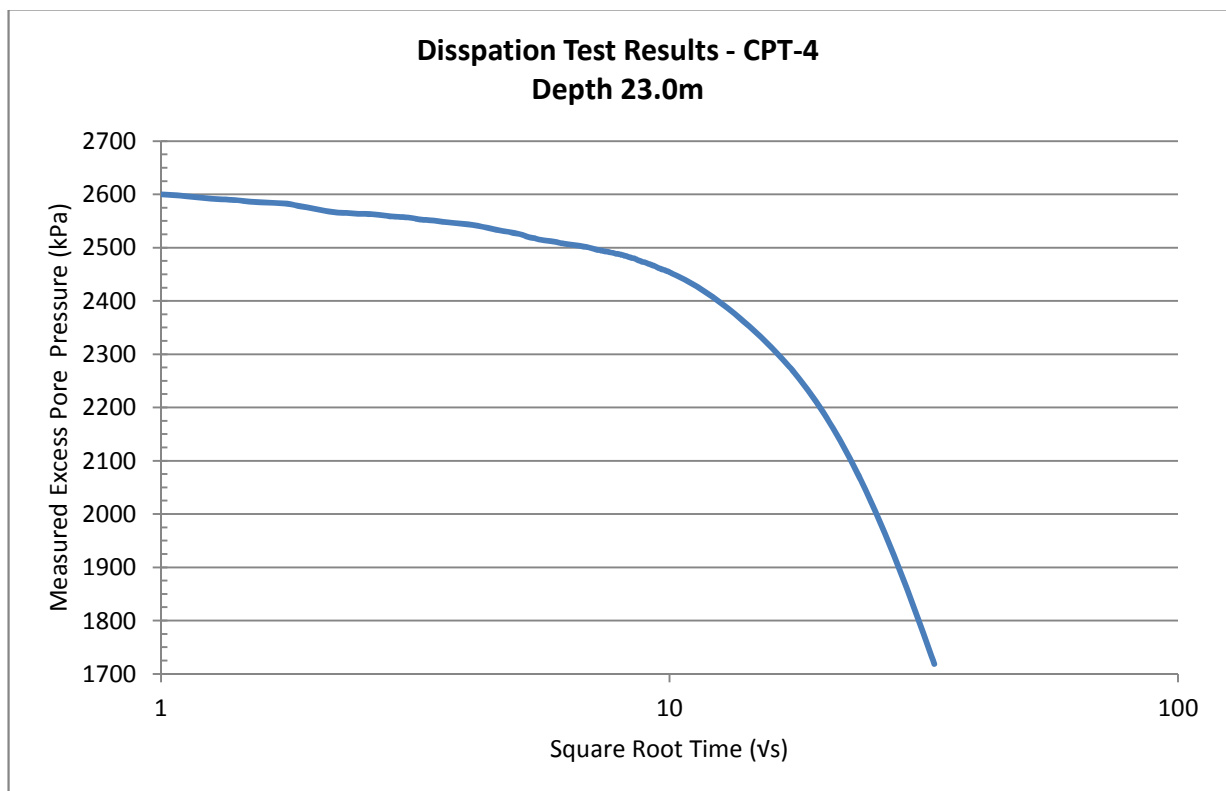










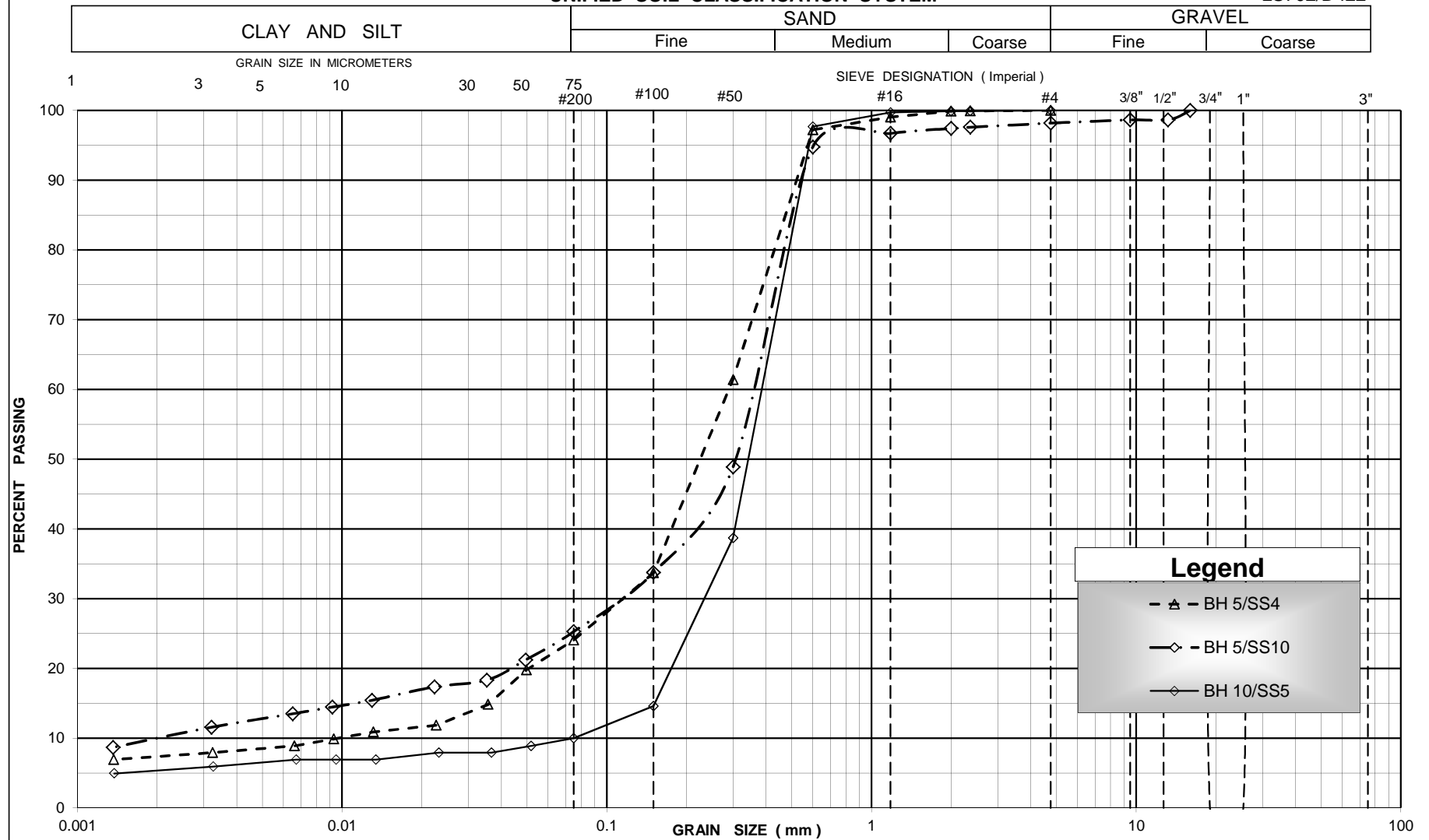


Appendix C

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

LS702/D422



GRAIN SIZE DISTRIBUTION
Sand, some gravel/silt, trace clay

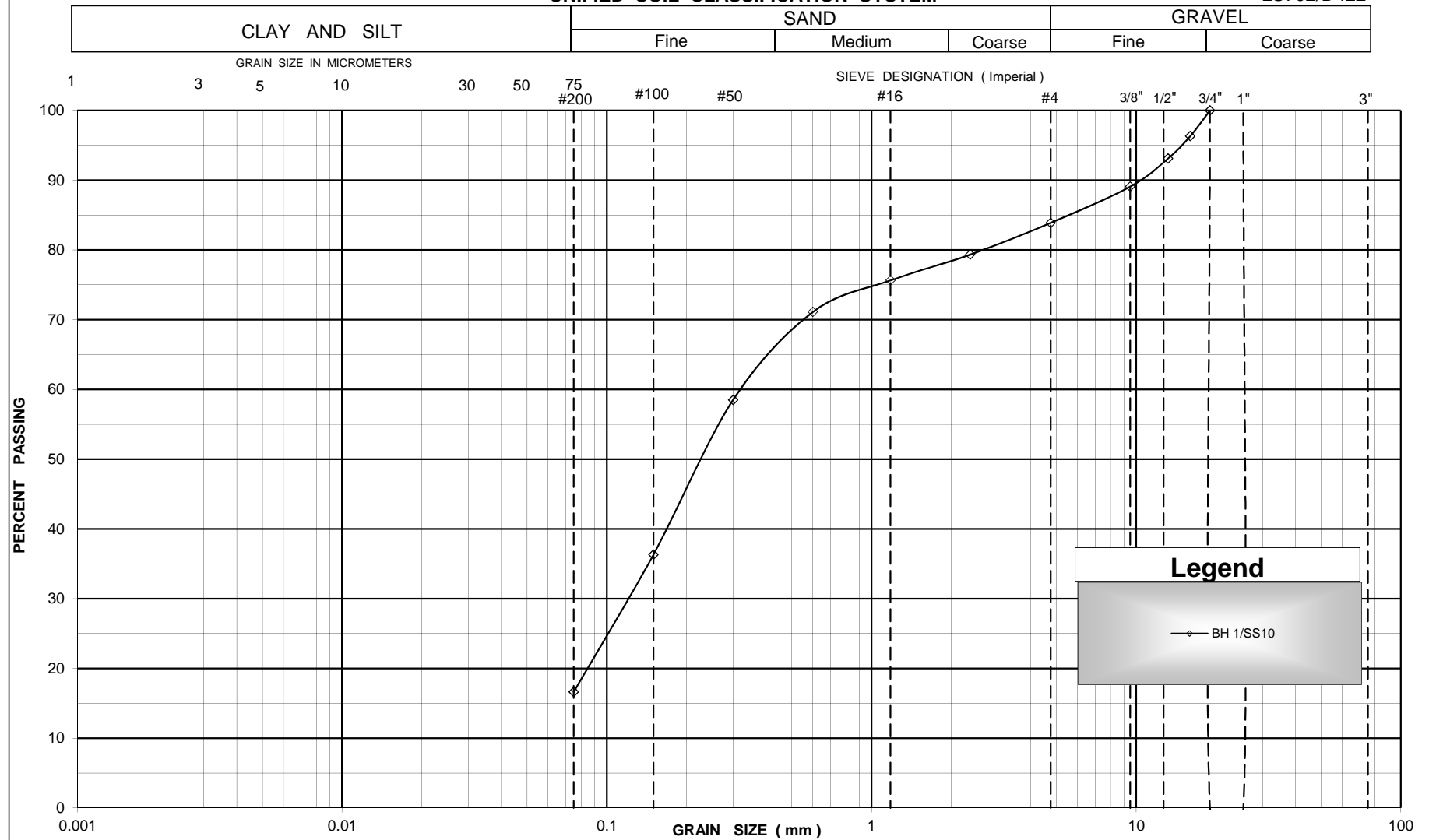
FIGURE #: C-1

PROJECT #: TRANETOB20462AA

DATE: Nov 26, 2013

UNIFIED SOIL CLASSIFICATION SYSTEM

LS702/D422



GRAIN SIZE DISTRIBUTION
coarser layer within Silty Sand

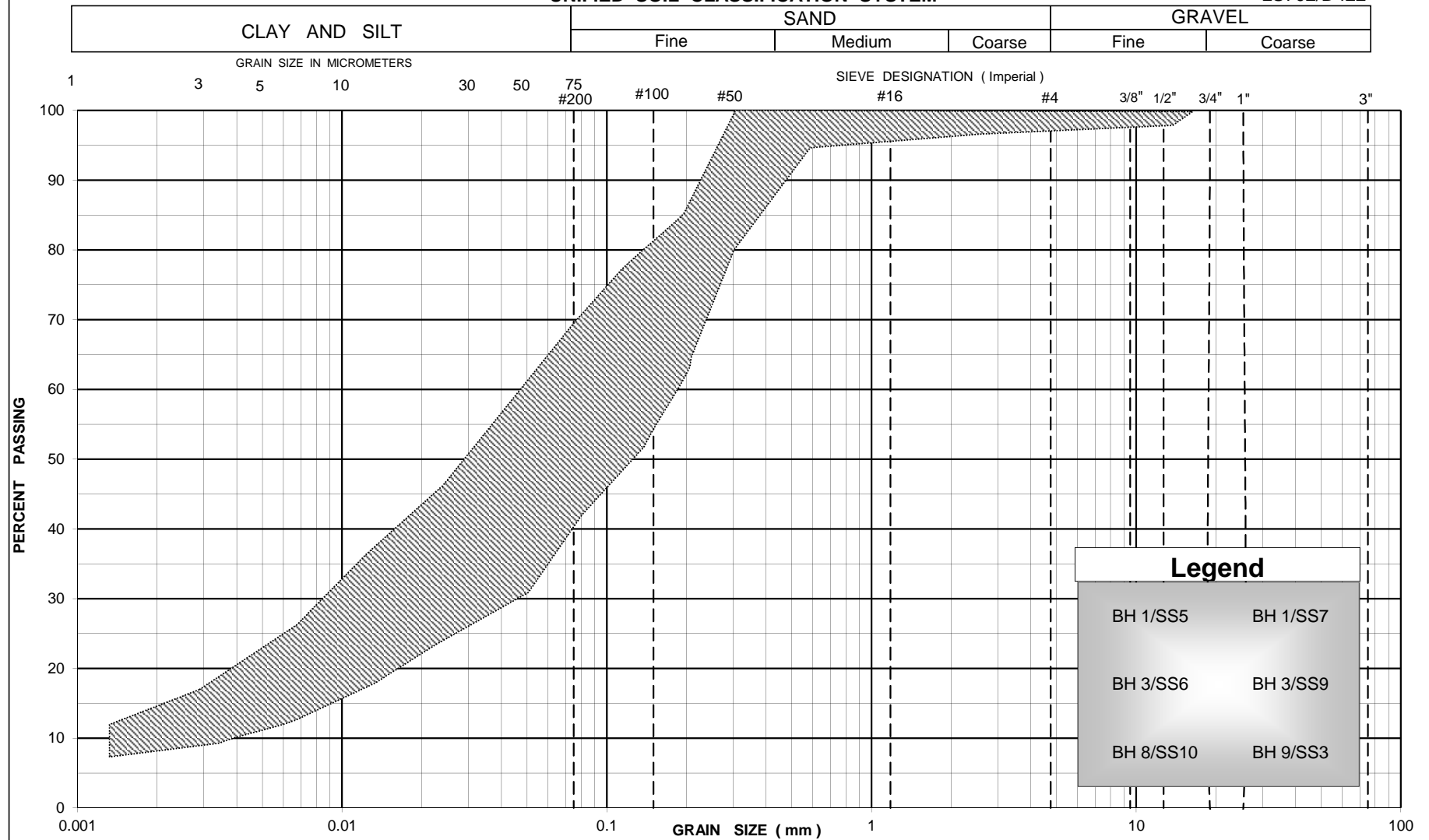
FIGURE #: C-2

PROJECT #: TRANETOB20462AA

DATE: Nov 26, 2013

UNIFIED SOIL CLASSIFICATION SYSTEM

LS702/D422



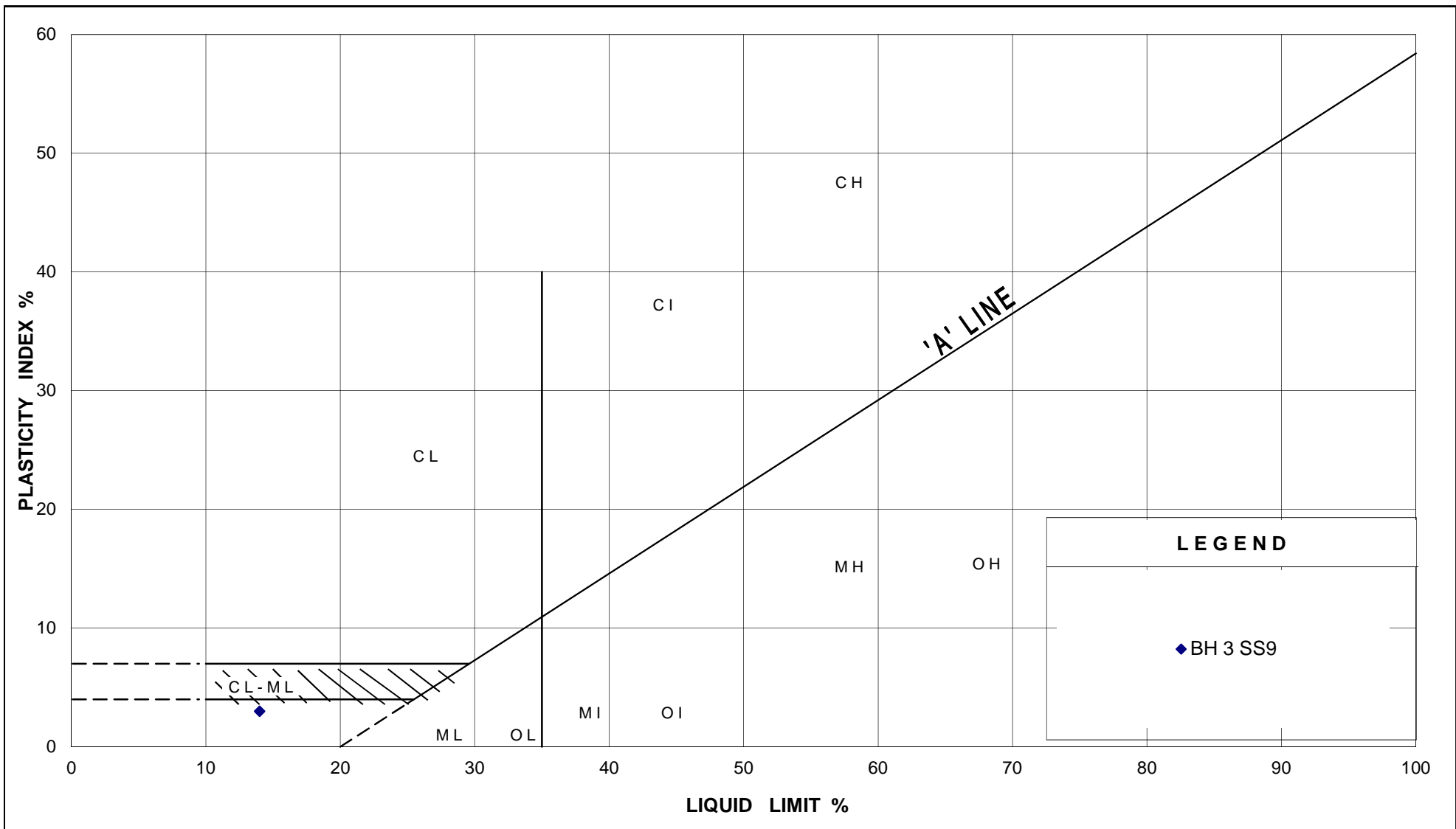
GRAIN SIZE DISTRIBUTION

Silty Sand

FIGURE #: C-3

PROJECT #: TRANETOB20462AA

DATE: Nov 26, 2013



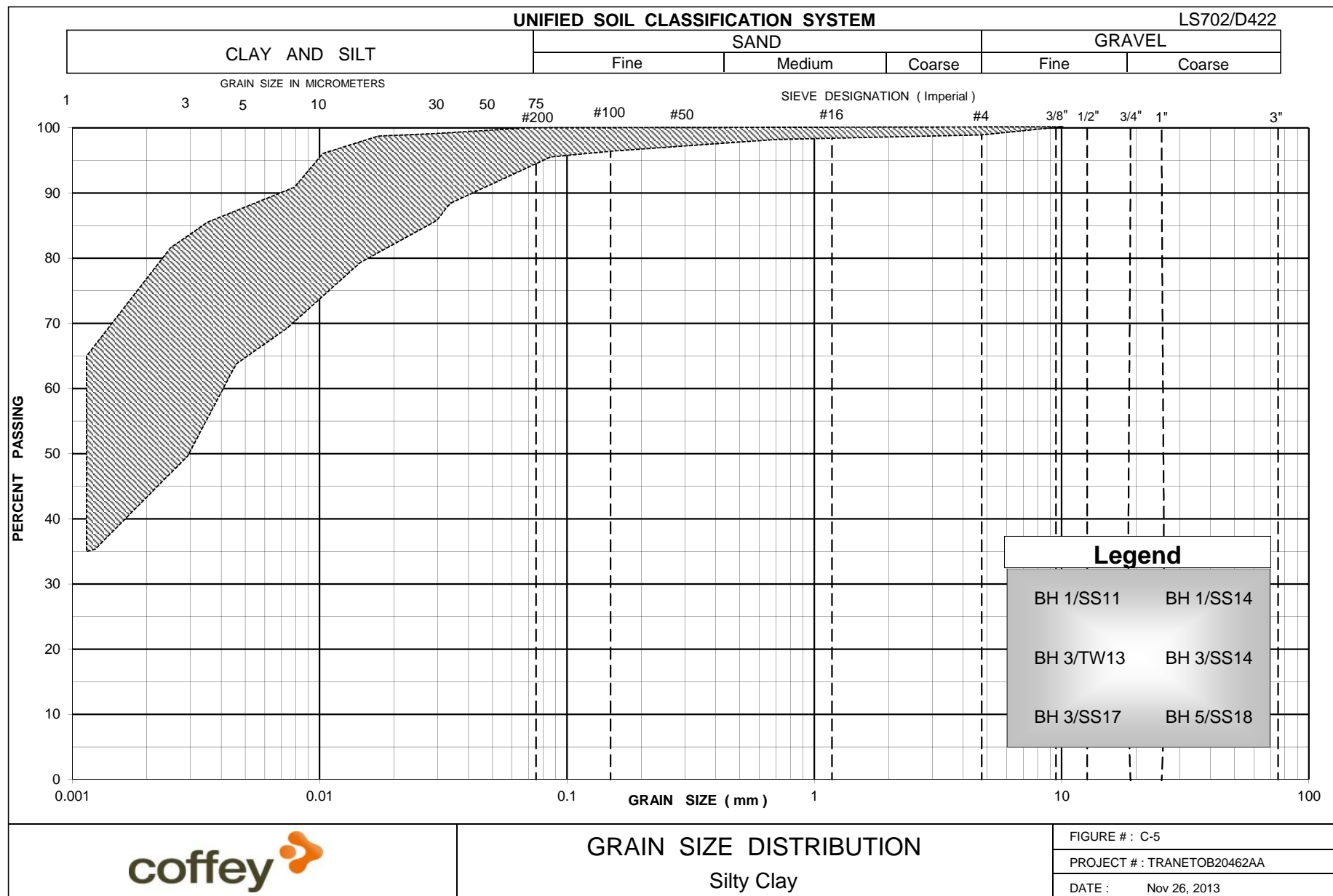
PLASTICITY CHART

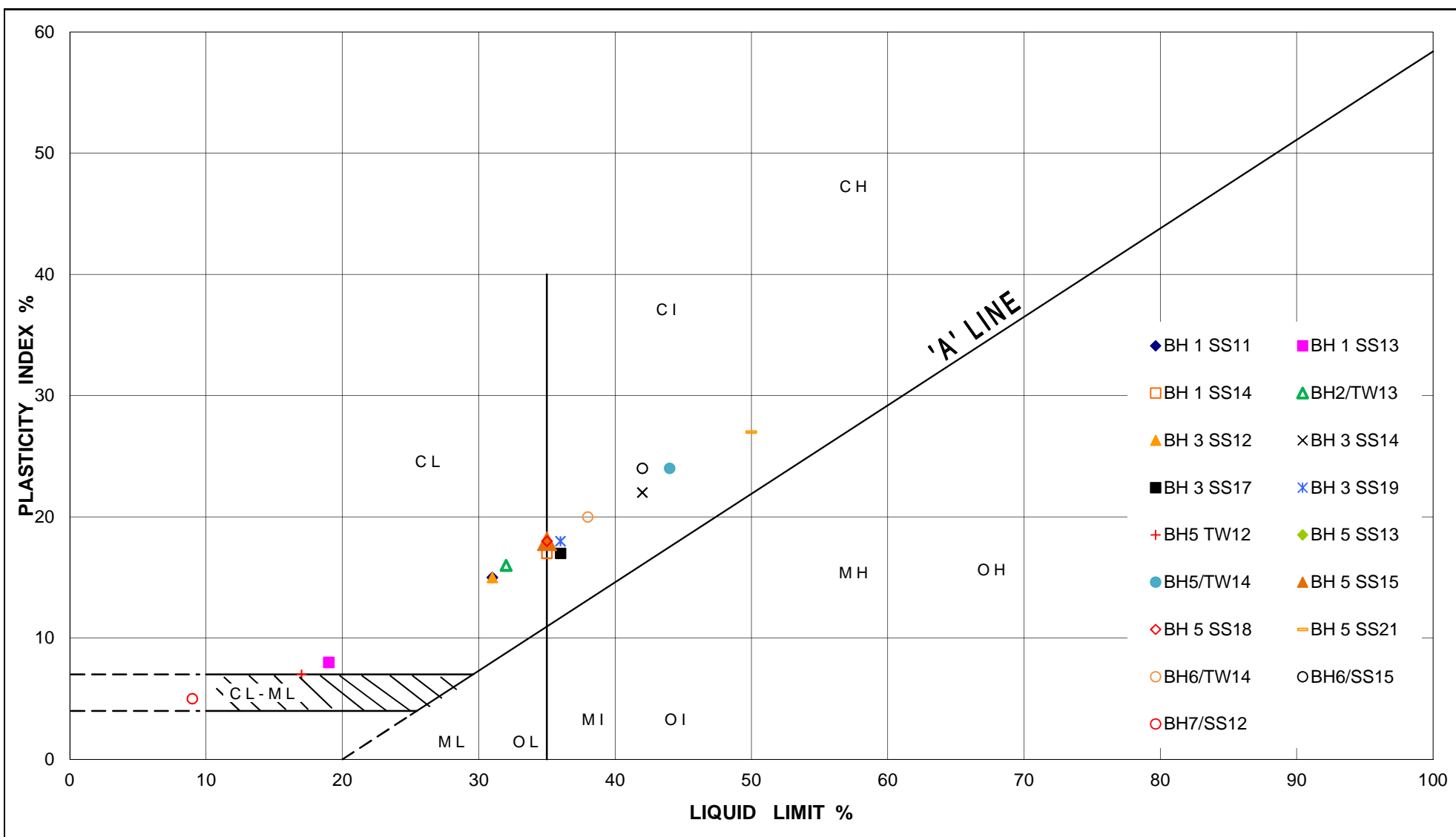
Silty Sand

Figure C-4

Project No. TRANETOB21844AA

DATE: Nov 26, 2013





PLASTICITY CHART

Silty Clay

Figure C-6

Project No. TRANETOB20462AA

DATE: Nov 26, 2013

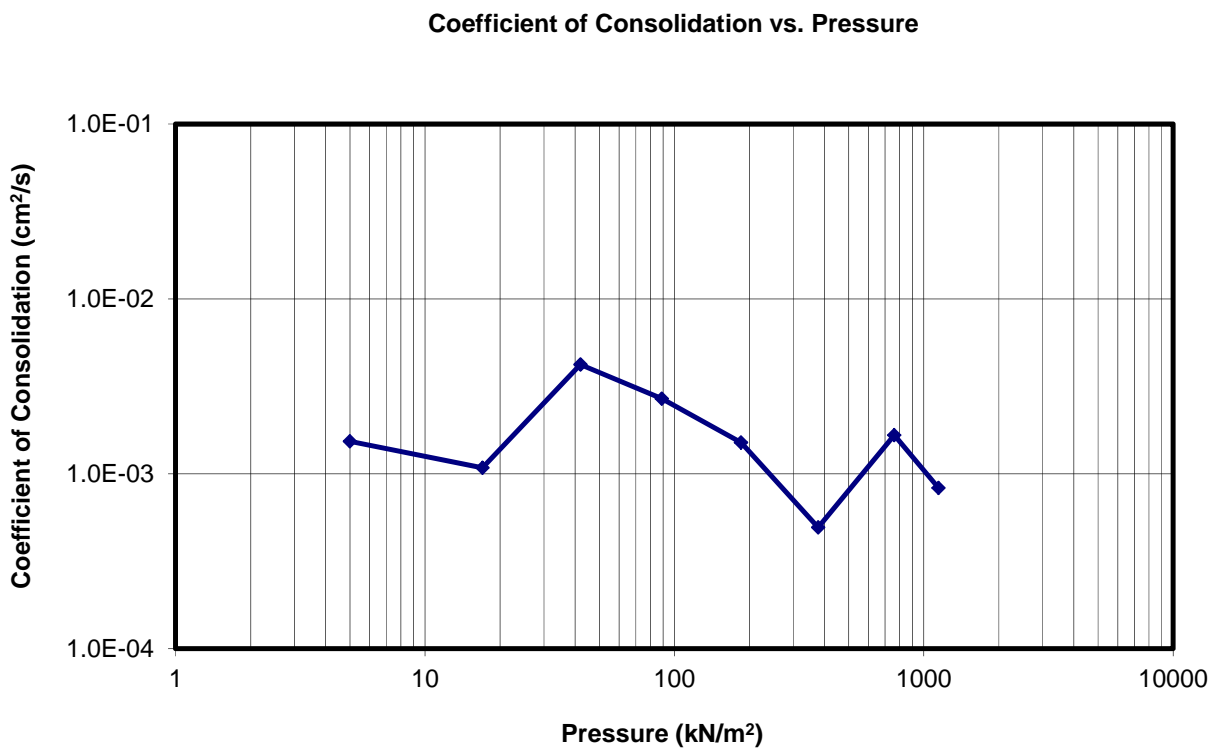
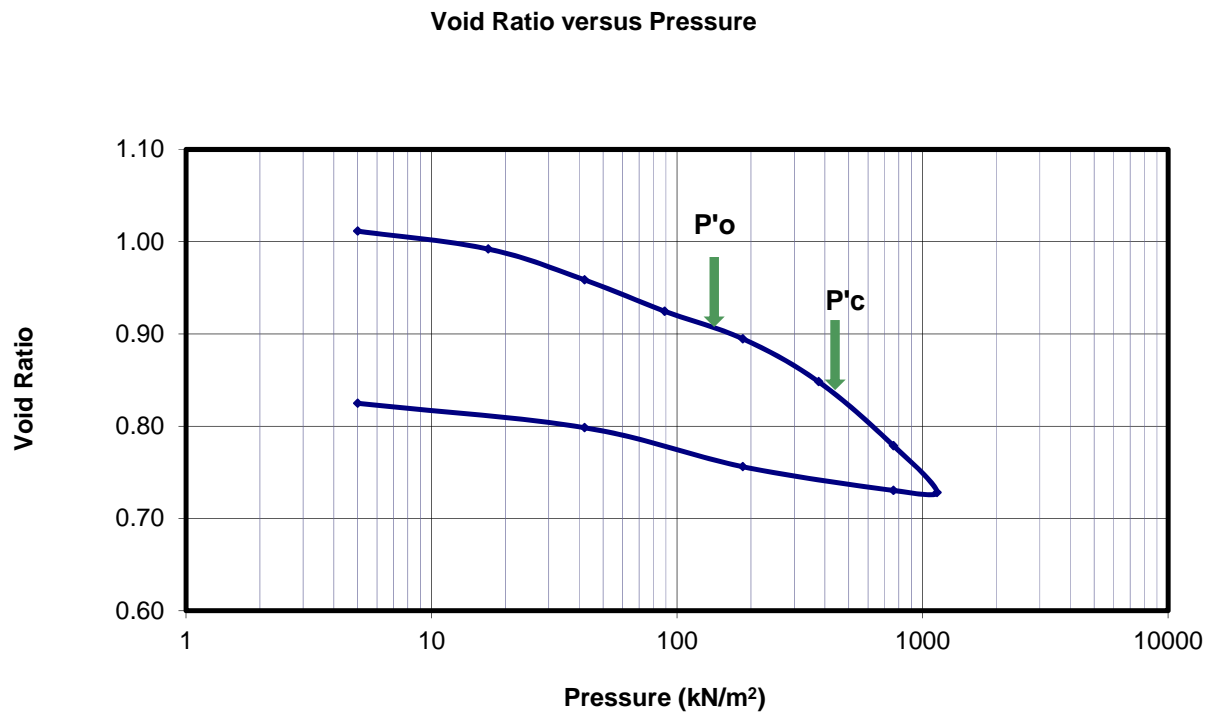
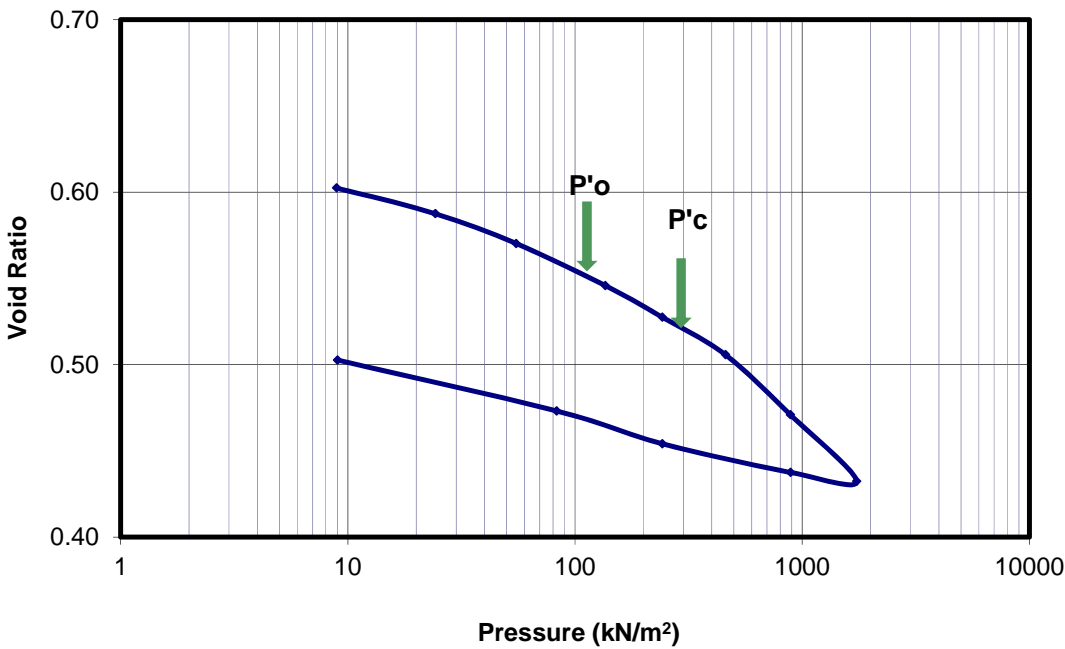


Figure C-7 Consolidation Test Results - BH 3 TW13
(P'c was estimated by strain energy method)

Void Ratio versus Pressure



Coefficient of Consolidation vs. Pressure

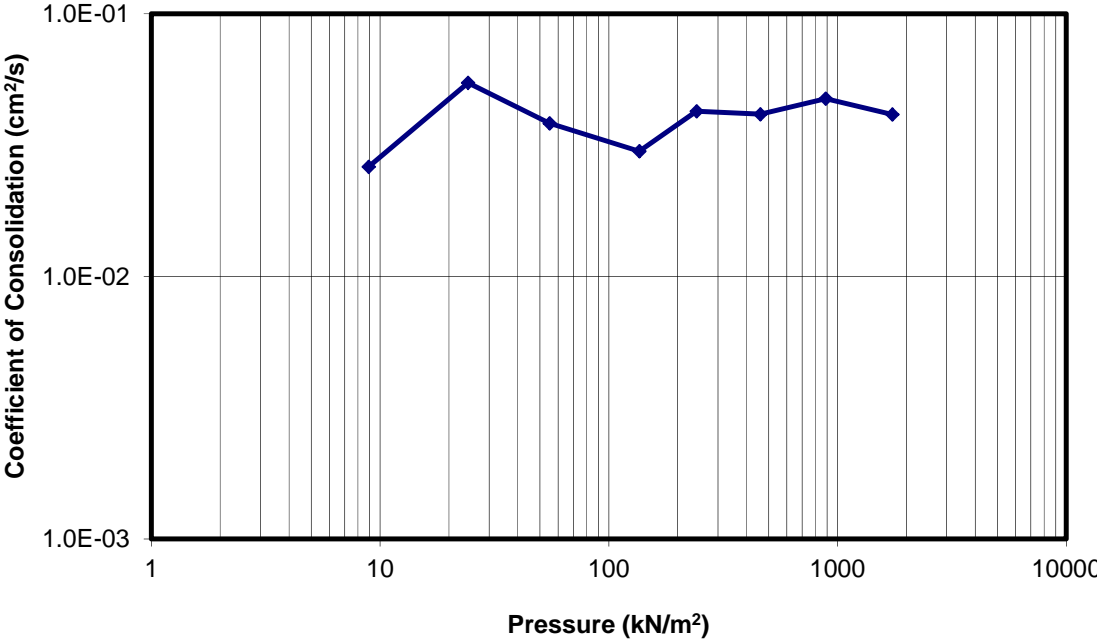
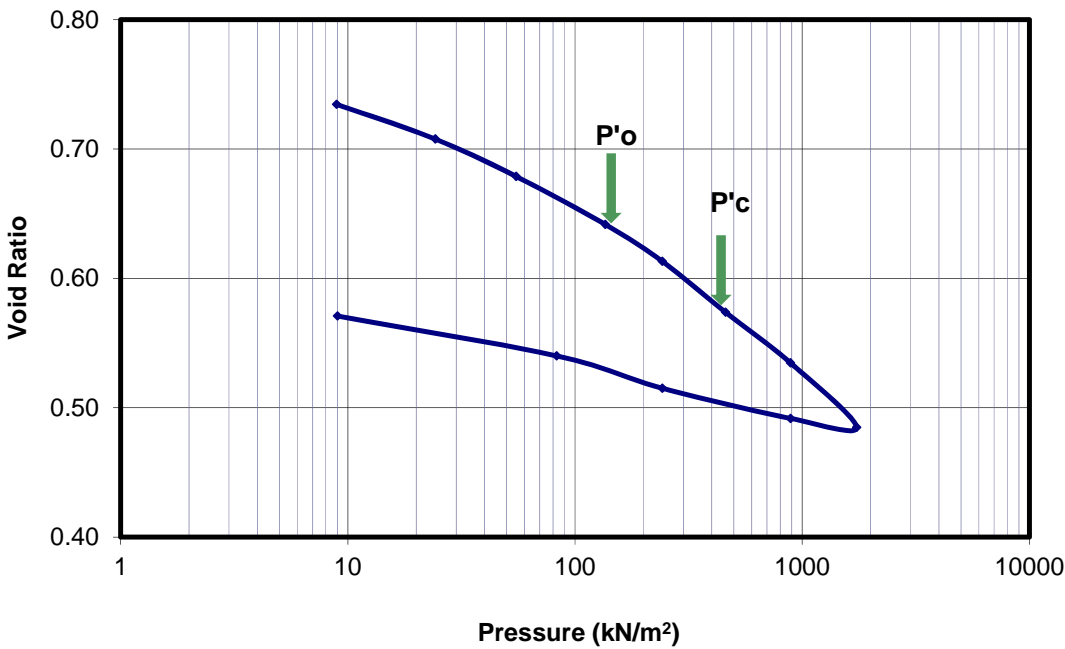


Figure C-8 Consolidation Test Results - BH 5 TW12
($P'c$ was estimated by strain energy method)

Void Ratio versus Pressure



Coefficient of Consolidation vs. Pressure

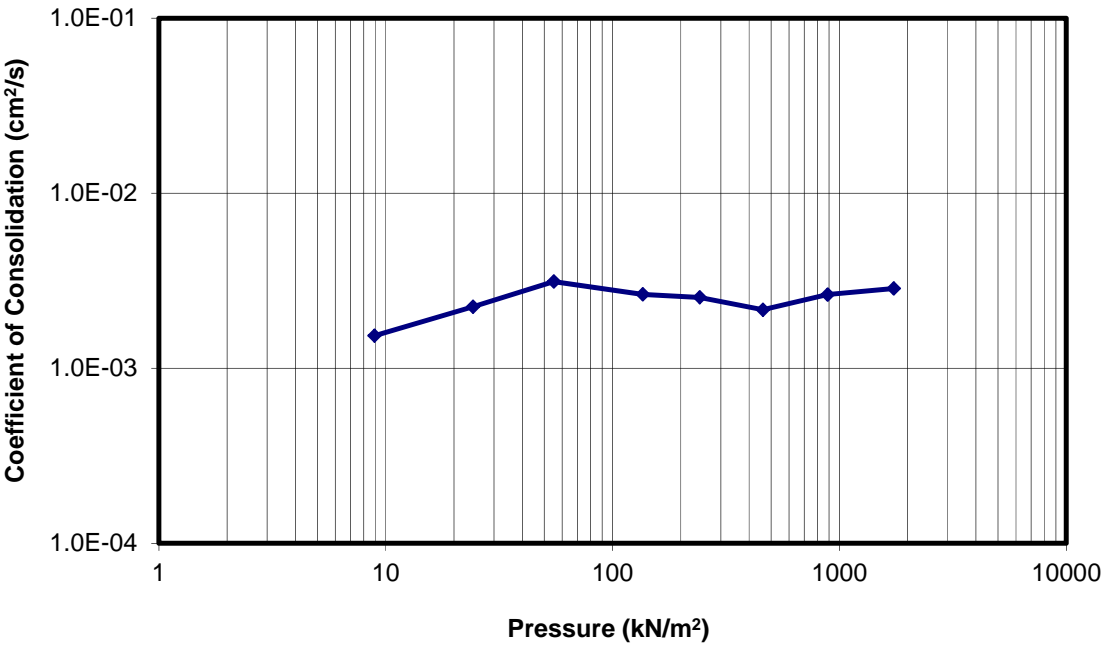
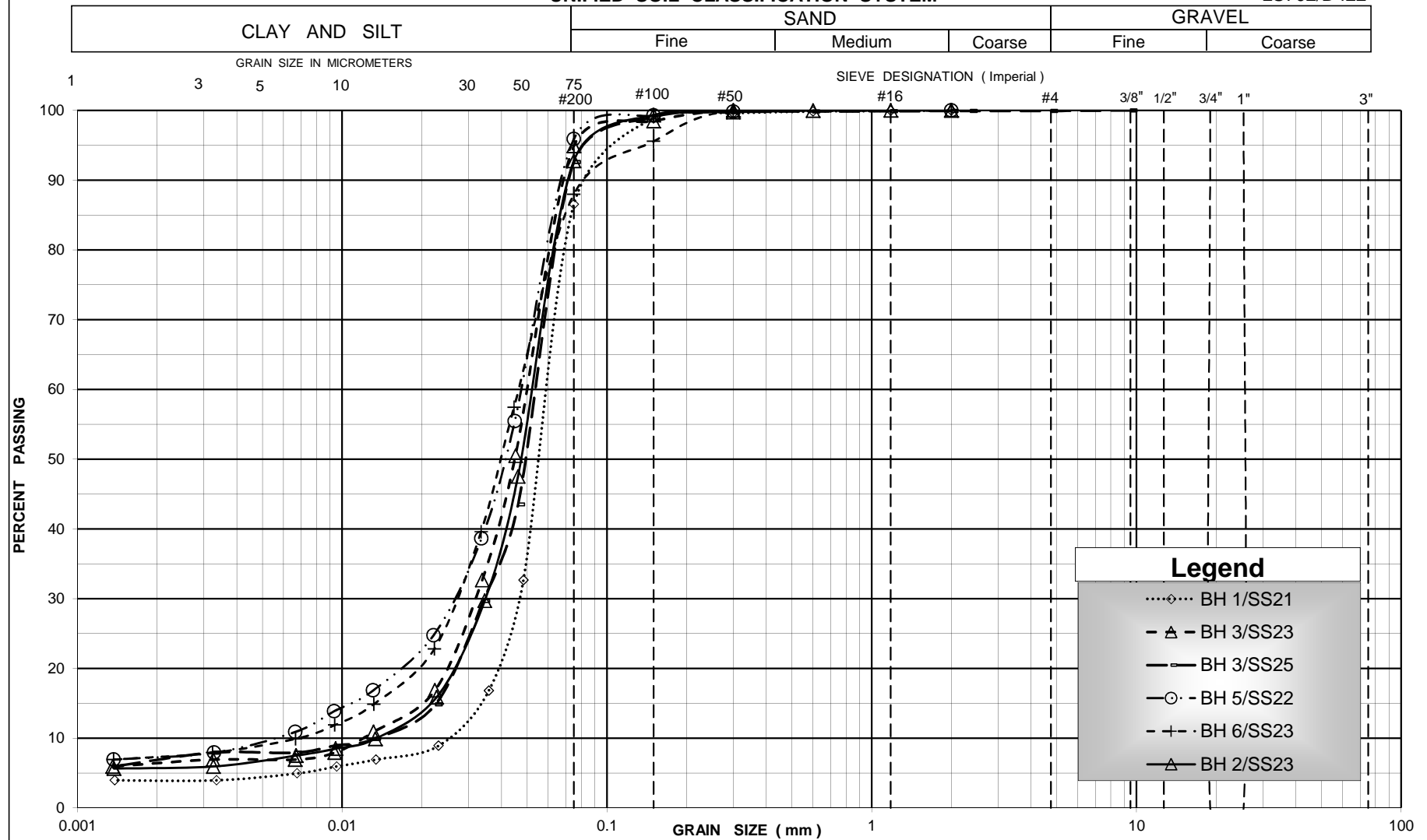


Figure C-9 Consolidation Test Results - BH 6 TW14
($P'c$ was estimated by strain energy method)

UNIFIED SOIL CLASSIFICATION SYSTEM

LS702/D422



GRAIN SIZE DISTRIBUTION
Silt, trace to some sand, trace clay

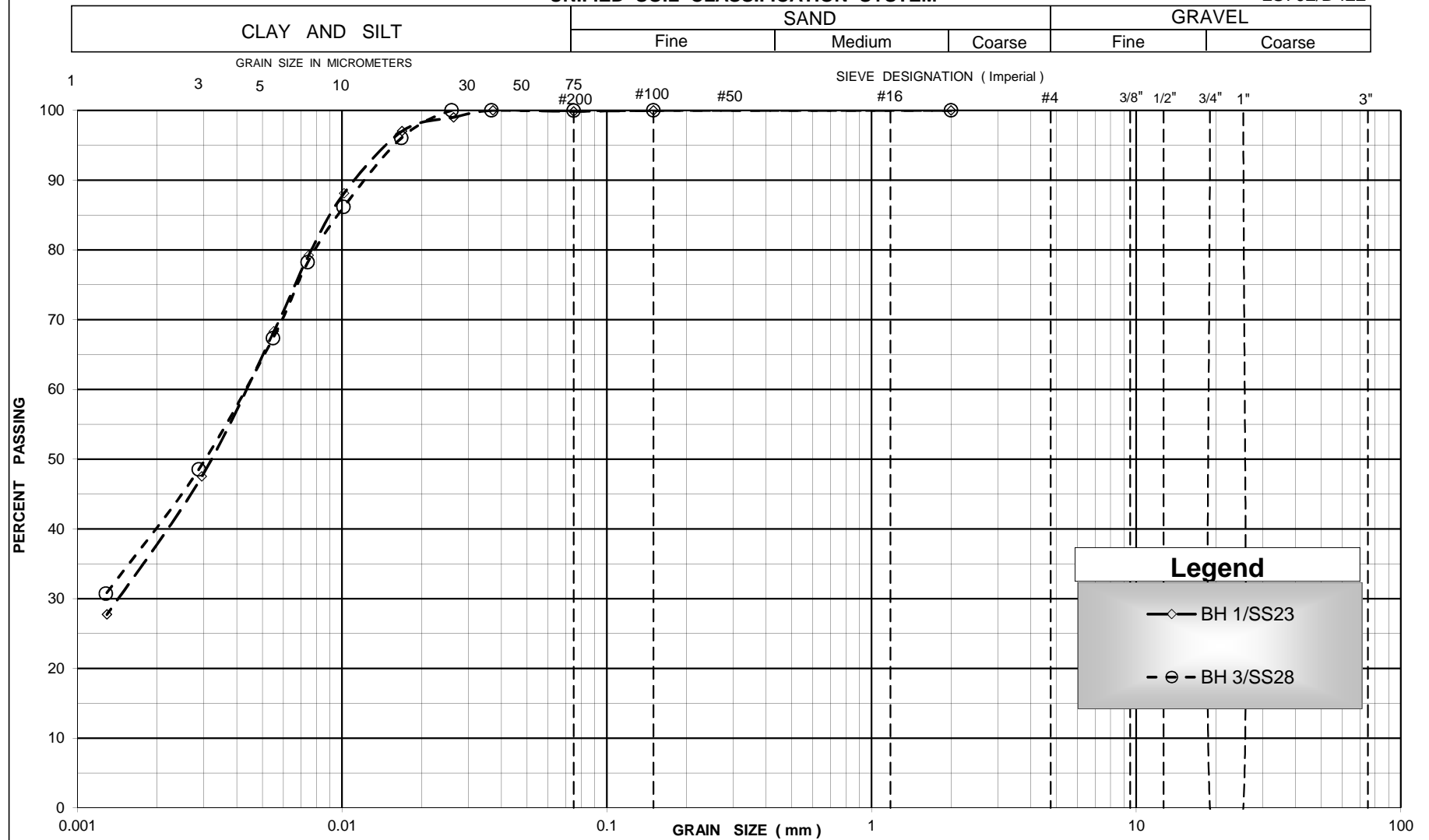
FIGURE #: C-10

PROJECT #: TRANETOB20462AA

DATE: Nov 26, 2013

UNIFIED SOIL CLASSIFICATION SYSTEM

LS702/D422

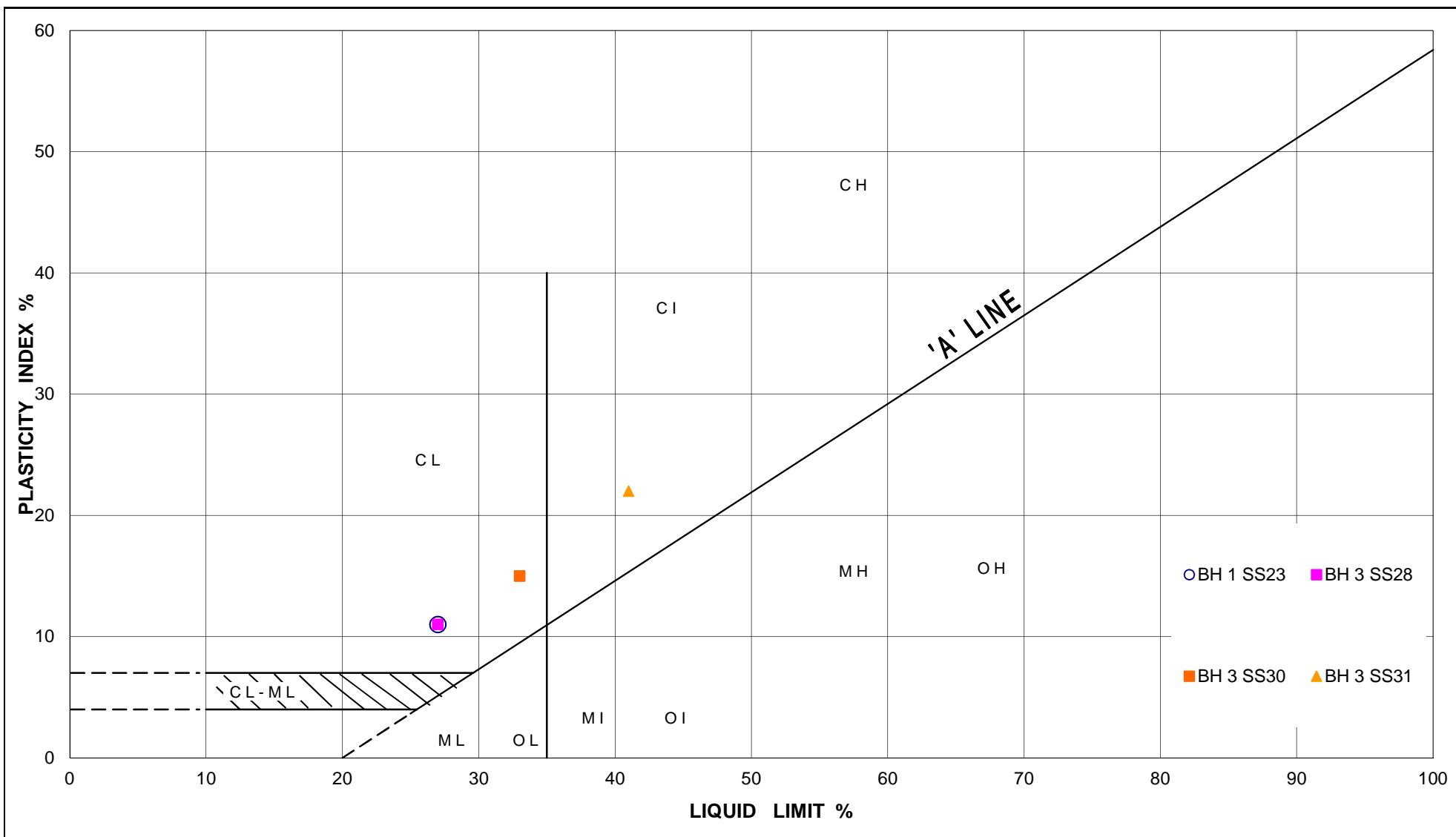


GRAIN SIZE DISTRIBUTION
Clayey Silt to Silty Clay

FIGURE #: C-11

PROJECT #: TRANETOB20462AA

DATE: Nov 26, 2013



PLASTICITY CHART

Clayey Silt to Silty Clay

Figure C-12

Project No. TRANETOB20462AA

DATE: Nov 26, 2013

Appendix D

Field Vane Test Results

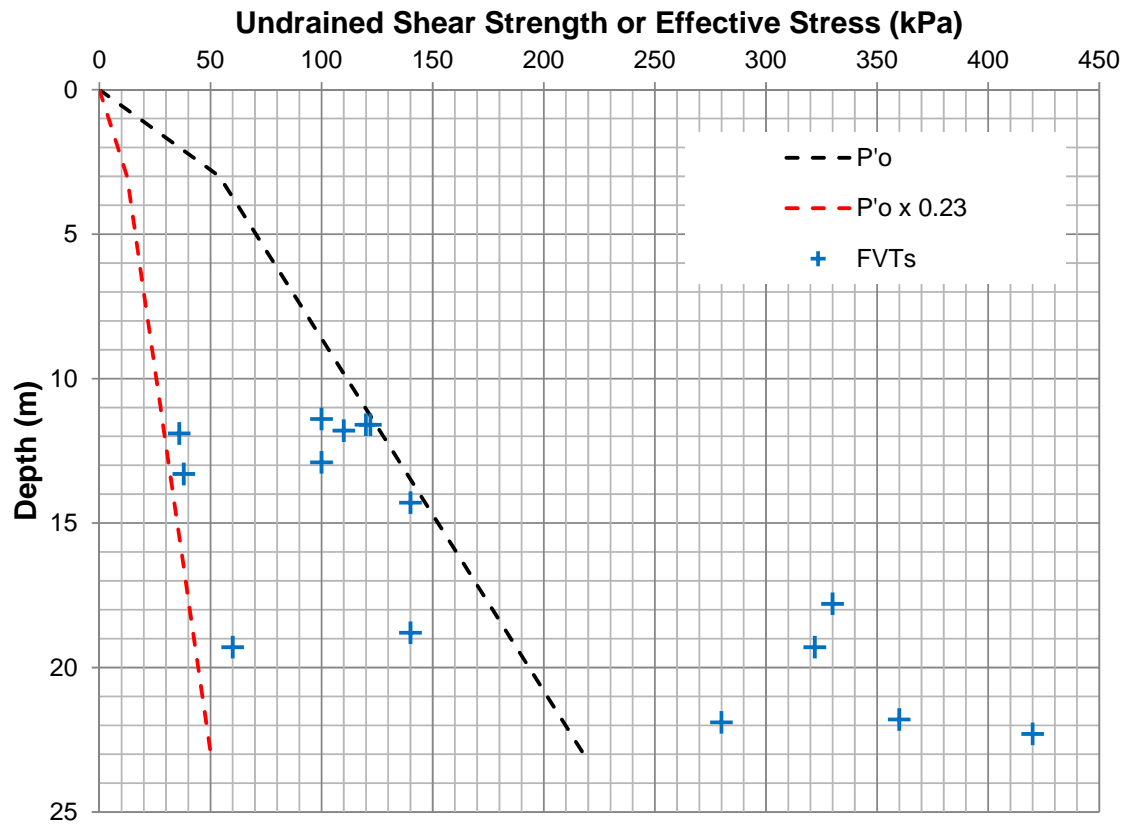


Figure D-1 Undrained Shear Strength or Effective Stress vs. Depth

Appendix E

Site Photographs



Photograph 1. Borehole BH1 (looking east)



Photograph 2. Borehole BH2 (looking west)



Photograph 3. Boreholes BH4 and BH5 (looking east)



Photograph 3. Borehole BH10 (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 \bar{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 COMPOSITION 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
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

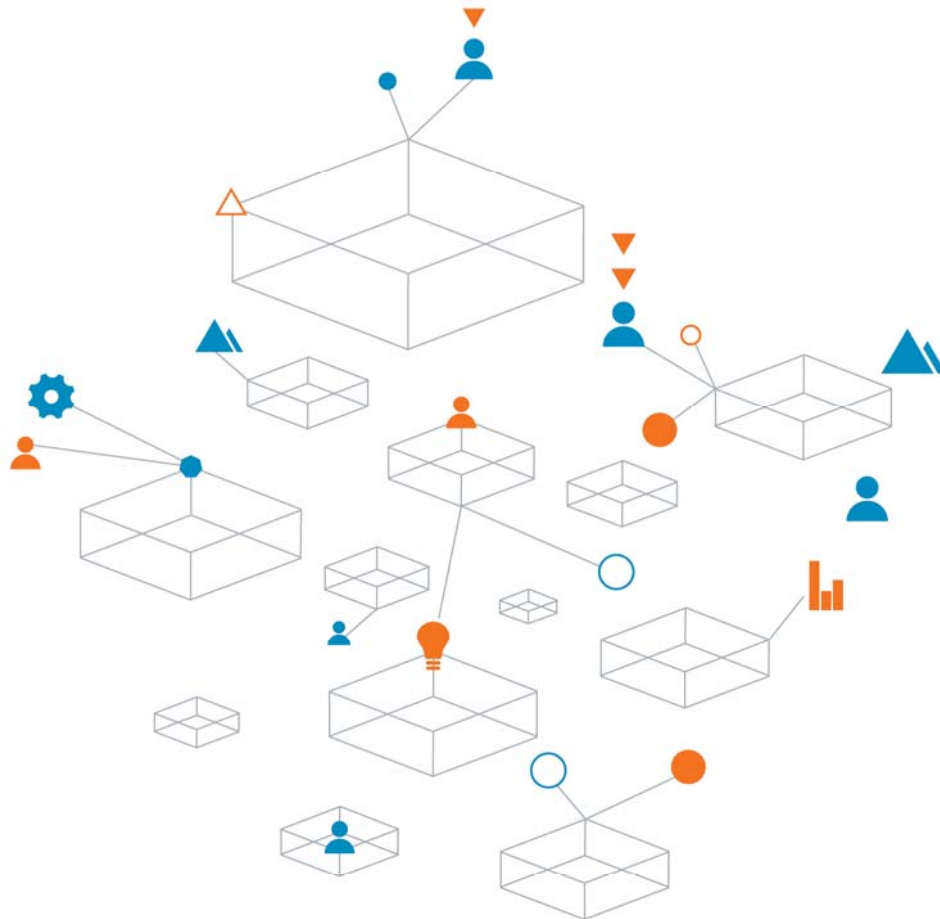
m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_e	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
Φ	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	-°	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

P_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
P	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 = $(W_L - W_p)$	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 = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	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**

Replacement of Glass's Bridge over Innisfil Creek,
Highway 89, Bridge Site No. 30-254, Town of Innisfil,
MTO Central Region, W.P. 2108-11-00, GEOCRES NO. 31D-572
TRANETOB20462AA
25 August 2014



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**FOUNDATION DESIGN REPORT
REPLACEMENT OF GLASS'S BRIDGE OVER THE INNISFIL CREEK
SITE NO. 30-254, TOWN OF INNISFIL
MTO CENTRAL REGION, W.P. 2108-11-00, GEOCRETS 31D-572**

7 DISCUSSION AND RECOMMENDATIONS

7.1 General

The existing Glass's Bridge over Innisfil Creek, located about 1 km west of Highway 400, will be replaced. The bridge is 11 m long by 10.7 m wide. It is a single span, reinforced cast-in-place concrete structure built in or around 1913. It may have been rehabilitated or widened in the 1930's. Based on drawings obtained from MTO GEOCRETS, the existing bridge appears to have been founded on spread footings, placed about 1.2 m below the then existing streambed elevation. The height of the bridge from the top of the foundations to the top of abutment level is about 4 m (see GA Drawing in Appendix G).

The new bridge, as presently proposed by MRC, will be a 25 m long single span structure. The proposed width of the bridge is 15 m, but provision will be made to widen the bridge on the north side by about 7.8 m in the future, as shown on the GA Drawing in Appendix G. This will entail constructing wider foundations during the presently proposed replacement, but the bridge widening (i.e. construction of the superstructure for the widened section) will be implemented at a later date. The final deck grade will be at elev. 227.2 m on the east side and 227.4 m on the west side. As the existing bridge grade is about 225.8 m, the proposed new deck grades will result in approach fill grade raises of about 1.5 m. At the widening towards the north, due to low ground, the approach fill grade raises will be 2.6 m to 3.1 m. The presence of an existing ditch on the north side of the road increases the grade raise locally to 4.0 m.

Innisfil Creek at the existing bridge location flows in a southerly direction. It is only about 4-5 m wide but is meandering and suffers from extensive streambank erosion. At the bridge location, the water level in the creek was recorded at elev. 222.8 m on September 20, 2012. The 50-year storm water level elevation is expected to be at elev. 224.9 m.

The following construction sequence is anticipated. A temporary modular bridge will be constructed about 30 m north of the existing bridge to carry the Highway 89 traffic on a detour alignment. Traffic will be diverted to the detour. The existing bridge will be demolished and replaced with the new longer and slightly higher bridge.

Two construction schedules are under consideration for the proposed bridge:

- Completion in one construction season
- Completion in two construction seasons

This investigation, described in Part 1 of this report, has shown the presence of surficial granular soils mixed with organics within 2-3 m of original ground near the Creek, underlain by fine grained granular soils consisting of generally loose to compact sandy silt and silty sand that extend to a depth of about 9 m, or to about elev. 217-214 m. The silty sand to sandy silt stratum has inclusions of thin stringers of clayey silt and silt.

The sandy silt-silty sand deposit is underlain by a massive silty clay deposit containing frequent thin clayey silt and silt seams and stringers. Most of the boreholes were terminated within this silty clay deposit at depths of about 9-37 m. In BH 3 the silty clay deposit was underlain at a depth of 47.4 m by a very dense silt layer. The silt stratum is 5-8 m thick and is the source of artesian water, with heads of 3 m above the

ground surface. The silty clay deposit extends to depths of about 25-27 m, or to elev. 199-197 m. A basal silty clay stratum was intersected in BH1 and BH3 at depths of about 32-34 m, or elev. 192-191 m.

The groundwater phreatic surface at the time of the investigation was generally between elev. 223.5 m and 222.3 m. It can be expected to be largely controlled by the water level in the creek and in response to major weather events. A groundwater and creek water level at elev. 222.8 m has been assumed for foundation design and stability analyses.

7.2 Foundations

Semi-integral abutments are proposed (see GA Drawing in Appendix G). Both shallow and deep foundation options were considered for the abutments.

7.2.1 Spread Footings

The very loose to compact upper fine grained non-cohesive soil (including organic rich soil layers near the creek in the upper 2-3 m) is not suitable for shallow spread footings, or spread footings on engineered fill, due to lack of adequate bearing capacity and anticipated unacceptable immediate and longer term settlements. Removal of the surficial organic rich upper zone of silty sand would require excavations below the prevailing groundwater table which would entail extensive site dewatering. For these reasons, a deep foundation solution is considered more appropriate for this site.

7.2.2 Deep Foundation Options

A number of deep foundation options were reviewed: driven timber, concrete and steel piles; caissons; and expanded base piles. A summary of these deep foundation alternatives is given in Table H-1 in Appendix H. The more feasible options are briefly discussed in the following paragraphs.

7.2.2.1 Driven Timber Piles

Timber piles will result in unacceptable long term settlements due to consolidation of the silty clay into which the pile toes must be situated to achieve an acceptable axial capacity for bridge abutment support. Timber piles have a short service life when exposed above water. For these reasons, they are not considered suitable for this site.

7.2.2.2 Driven Concrete Piles

It is possible to support the bridge abutments on precast concrete piles driven into the intermediate fine grained granular soils below elev. 199-198 m, beneath the thick silty clay deposit. However, they are more expensive than steel piles. Their axial load capacities are generally about 15-20 per cent less than those for steel piles owing to fibre stress considerations. For this reason, they are considered impractical for this site.

7.2.2.3 Expanded Base Piles

Drive expanded base piles are not feasible, as the upper silty sand layer is insufficiently dense to generate the formation of an adequately sized expanded base, or "bulb", before the dry concrete plug reaches into the very stiff to hard silty clay deposit. Once the bulb is formed in the silty clay deposit, intolerable long term settlements can be anticipated under full bridge loading conditions.

7.2.2.4 Drilled Caisson Foundations

Caissons (drilled cast-in-place concrete large diameter piles) with their toes located in the silty clay deposit are feasible, but impractical owing to low geotechnical resistance and unavoidable long term settlement. Extending them deeper into the very dense silt increases the risk of unwanted artesian flow due to the potential penetration of the advancing auger into the artesian silt stratum under high torque that will be required to advance the caisson hole through the silty clay deposit.

7.2.3 Driven Steel Piles

7.2.3.1 General

Driven steel H-piles or close ended steel tube piles are the most suitable deep foundation alternatives for the prevailing subsurface and groundwater conditions. They can be driven to rest on top of, or just within, the very dense silt deposit at about elev. 198-199 m, or to about elev. 202 m to avoid the risk of encountering artesian conditions in the very dense silt deposit. Both options are discussed in the following sections.

Closed-end driven steel pipe piles are more advantageous than steel H-piles to minimize the risk of toe penetration into the artesian very dense silt deposit. Steel H-piles have a greater risk of penetration, unless closely controlled with respect to toe elevation and determination of axial capacity achievement at the highest toe elevation possible. Low-displacement steel H-piles do not generate vibrations during driving to the extent that closed-end steel pipe piles do. Vibration control and/or mitigation is a requirement from Enbridge for a gas main located just south of the existing bridge.

7.2.3.2 Axial Capacity of Driven Steel Piles

Steel piles can be driven to about elev. 202 m to avoid the risk of encountering artesian conditions in the very dense silt deposit. The calculated resistances for HP 310 x 110 Steel H-piles and O.D. 324 mm x 6.3 mm steel tube piles (driven closed end, concrete filled) are as follows:

Factored Geotechnical Axial Resistance at ULS = 620 kN/pile

Axial Reaction at SLS = 440 kN/pile

Alternatively, steel piles may be driven to the surface of the dense silt deposit to toe elev. 199-198 m. The calculated resistances for HP 310 x 110 Steel H-piles and O.D. 324 mm x 6.3 mm steel tube piles (driven closed end, concrete filled) are as follows:

Factored Geotechnical Axial Resistance at ULS = 900 kN/pile

Axial Reaction at SLS = 600 kN/pile

Both steel H-piles and closed end driven concrete-filled steel pipe piles may not attain their calculated axial capacities immediately upon completion of driving owing to the development of excess porewater pressure in the silty clay deposit during pile driving.

To avoid vibrations that could impact the existing Enbridge gas main, it is recommended the abutments for the proposed new bridge be supported on steel H piles (HP 310 x 110) driven to no deeper than elevation 198-199 m and designed for factored geotechnical axial resistance at ULS = 620 kN and SLS = 440 kN.(see GA drawing in Appendix G).

7.2.3.3 Lateral Resistance

To estimate lateral resistance the coefficient of horizontal subgrade reaction, K_s , can be estimated from:

- in cohesionless soils $K_s = n_h z/d$
- In cohesive soils $K_s = 67 c_u/d$

where c_u = undrained shear strength; z = depth; d = pile width or diameter; n_h = coefficient related to soil density, Table 7.2.3.3.1.

Table 7.2.3.3.1 Soil Parameters

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m ³)	Angle of Internal Friction (φ) Degrees	Recommended n_h Value (MN/m ³)	Recommended Undrained Shear Strength, c_u (kPa)
West Abutment						
BH1	223.1-222.2	Sand with org. lenses	16.0	24	1.0	-
	222.2-215.4	Silty Sand/Sandy Silt/Sand	18.0	28	1.1	-
	215.4-197.4	Silty Clay	18.0	-	-	130
	197.4-192.4	Sandy Silt/Silty Sand	18.5	32	4.4	-
BH2	223.4-222.5	Sandy Silt/Silty Sand	16.0	25	0.8	-
	225.5-217.1	Silty Fine Sand/Sandy Silt	17.5	29	1.5	-
	217.1-198.6	Silty Clay	17.5	-	-	120
	198.6-194.5	Sandy silt	19.0	32	11.0	-
BH3	223.3-221.8	Silt/Sand/Clayey Silt	17.0	27	1.2	-
	221.8-215.8	Silty Fine Sand/Sandy Silt	17.5	29	1.5	-
	215.8-214.8	Sand	19.0	30	1.6	-
	214.8-199.3	Silty Clay	17.5	-	-	100
	199.3-193.0	Sandy Silt	19.0	33	11.0	-
East Abutment						
BH5	223.1-222.0	Sandy Silt	16.5	25	0.8	-
	222.0-219.1	Sandy Silt	17.5	29	1.5	-
	219.1-214.9	Silty Fine Sand/Sandy Silt	17.5	28	1.3	-
	214.9-198.8	Silty Clay	17.5	-	-	140
	198.8-195.0	Sandy Silt	19.0	33	11.0	-
BH6	223.3-221.1	Sandy Silt/Silty Sand	17.0	28	1.3	-
	221.1-215.0	Silty Fine Sand/Sandy Silt	18.0	29	1.5	-
	215.0-214.4	Silty Sand	18.5	30	4.0	-
	214.4-198.4	Silty Clay	18.0	-	-	110
	198.4-194.7	Sandy Silt	19.0	33	11.0	-

The estimated horizontal resistance for HP 310 x 110 steel H-piles (and O.D. 324 mm x 6.3 mm steel tube piles) are as follows (based on CHBDC S6-06 and Brom's method):

Factored Horizontal Resistance at ULS = 120 kN / H-pile (and 140 kN / pipe pile)

Resistance at SLS (10 mm lateral movement) = 40 kN / H-pile (and 80 kN / pipe pile)

7.2.3.4 Installation of Steel Piles

All piles should be installed and monitored in accordance with OPSS 903. Pile spacing, centre to centre, should not be less than 3D, where D = largest pile width/dimension in plan. Pre-augering of embankment may be required to minimize pile driving induced vibrations, which can adversely affect the integrity of the nearby Enbridge gas pipe and the stability of loose wet granular soil under the embankment.

Pile driving should be monitored and controlled using a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles by the Hiley formula (MTO Standard Drawing SS 103-11) can be calculated by dividing the recommended axial resistance at ULS by a resistance factor of 0.5.

As the actual driving of piles in the field will be governed by the Hiley Formula and/or a pile driving analyser (PDA), design pile tip elevations are provided for guidance only. The actual pile toe elevations and pile lengths may vary from those estimated. After installation, pile re-tapping may be required due to potential pile upheaval when adjacent piles are driven.

The following NSSP should be provided in the bid documents: *“The very dense silt deposit below elev. 199-198 m, if penetrated sufficiently during pile driving and installation, may result in artesian flow that will be required to be controlled or stopped. The cost of controlling and stopping artesian flow, to the satisfaction of MTO and MOE, shall be the sole responsibility of the contractor”.*

Pile driving should be controlled as follows:

- Pile toes should not be lower than elev. 199 m
- With the pile toe at elev. 199 m, the pile capacity should be checked with the Hiley formula.
- The Hiley formula pile capacity should be confirmed with a PDA.
- A set and rebound criterion should be developed on the basis of the PDA analysis. The set criterion should be used to monitor the driving of other piles.
- It is recommended that at least one pile at each abutment location be so tested and set criteria developed for each abutment location.

It is estimated that a set-up time of minimum one week will be required to dissipate excess pore water pressure generated in the silty clay deposit after pile driving. If the design pile capacity has not been achieved upon completion of pile driving, re-striking is recommended at least a week later to confirm the axial load capacity.

7.3 Lateral Earth Pressures

Backfill behind abutments should consist of non-frost susceptible, free-draining granular materials (OPSD 3101.150). Computation of earth pressures should be in accordance with CSA/CAN S6-06. For design purposes, the following unfactored soil parameters are recommended.

Compacted Granular ‘A’ and Granular ‘B’ Type II

Angle of Internal Friction, $\phi = 35^\circ$ (unfactored)

Unit Weight, $\gamma = 22 \text{ kN/m}^3$

Coefficients of Lateral Earth Pressure: $K_A = 0.27$ $K_o = 0.43$

Compacted Granular ‘B’ Type I

Angle of Internal Friction, $\phi = 32^\circ$ (unfactored)

Unit Weight, $\gamma = 21 \text{ kN/m}^3$

Coefficients of Lateral Earth Pressure: $K_A = 0.31$ $K_o = 0.47$

These values assume that the backfill behind the retaining structure is free-draining; that adequate drainage has been provided to prevent the build-up of hydrostatic pressure; that there is no surcharge on a horizontal backfill surface behind the retaining wall.

Vibratory equipment for compaction will generate incremental and residual locked-in earth pressures that should be allowed for in structural design.

7.4 Seismic Design

Seismic analysis is not required for single span bridges, regardless of a seismic performance zone, except for single span truss bridges, as per Clause 4.4.5.2 of CHBDC CAN/CSA-S6-06. However, the following information is provided, if required for 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, the site coefficient, S , 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 the 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 $A = 0.075$.

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.

7.5 Approach Embankments

It is expected that the existing roadway will be widened by about 10 m on the north side while a minimal grade adjustment will be made on the south side. The existing top of pavement elevation is about 225.7 m. The new elevation at the west and east abutments will be 227.4 m and 227.2 m, respectively.

The proposed re-alignment and new profile grade will result in grade raises above the existing pavement, and above the original ground to the north, of about 1.0-1.5 m. The existing deep north side ditch and a variable ground surface elevation elsewhere will result in maximum localized fill heights of up to 4 m. This magnitude of grade raise has implications with respect to slope stability and total and differential settlements. These concerns are addressed in the following sections of this report.

7.5.1 Approach Embankment Stability

Slope stability analyses were carried out on supplied cross-sections (see Appendix G). The stability of the proposed embankments was analysed by the limit equilibrium approach using Slope/W and the Morgenstern-Price method of analysis for both short term (undrained) and long term (drained) scenarios. The soil parameters adopted in the analysis are summarized in Table 7.5.1.1.

Table 7.5.1.1 Soil Parameters Used for Slope Stability

Soil Type	Unit Weight (kN/m ³)	Shear Strength Parameters			
		Undrained		Drained	
		Cohesion (kPa)	Angle of internal friction (deg)	Cohesion (kPa)	Angle of internal friction (deg)
New Embankment Fill	20.5	0	32	0	32
Existing Embankment 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
Silty Clay	18.0	80	0	0	28
Surcharge	19.0	0	27	0	27

The results of the analyses are given in Appendix I. The calculated minimum factors of safety range from 1.4 to 1.5 for embankments up to 4 m in height constructed with 2H:1V side slopes.

7.5.2 Forward Slope Stability

Analysis for approach fill forward slopes was carried out using the soil parameters of Table 7.5.1.1 and the assumed 2H:1V slope profile with a mid-slope bench. The analysis indicates the proposed 2H:1V forward slopes are stable provided the forward slope toe and creek channel are provided appropriate erosion control measures, such as suitably sized rip-rap placed on geotextile. Periodic inspection and maintenance of the forward slopes and creek channel is necessary to ensure long term forward slope stability.

The stability analyses results are given in Appendix I.

7.5.3 Approach Embankment Settlements

The calculated maximum foundation settlement under the 1.0-1.5 m grade raise over the existing pavement is about 40 mm. The proposed widening on the north side has a grade raise of 4.0 m for which the calculated maximum settlement is 110 mm. On the east side the maximum height of fill is about 2.7 m and the calculated maximum foundation settlement is about 90 mm. It is estimated that about 70 to 80 per cent of these estimated settlements will occur during and upon completion of construction. The remaining residual settlements will be of a consolidation type and will occur over a much longer period of time.

Uneven embankment settlement will impact roadway ride and smoothness. The settlement potential can be reduced by preloading. Alternatives to minimize total and differential embankment settlement include the use of lightweight materials such as EPS, slag, and tire derived aggregates (TDA). or, consideration could be given to various ground improvement techniques such as deep soil mixing, vibro-compaction and aggregate piers. All of these alternatives entail high costs, special materials and special construction techniques. For these reasons, preloading is considered the most viable option to minimize post-construction differential settlements.

A minimum of three (3) months of preloading with 2.5 to 3.0 m grade raise on the widening section up to the existing Highway 89 level is recommended for the first year of construction. A minimum of one (1) month of preloading for an additional up to 1.5 m grade raise over the entire width of the new highway embankment is recommended in the second year of construction. The preloading is intended to reduce differential settlement between the existing and widened embankment to in the order of 20 mm or less.

Settlement monitoring is recommended to ensure the anticipated settlement is achieved by the proposed preloading. Time rate of settlement estimates are presented in Appendix J. A Non-standard Special Provision (NSSP) for ground settlement monitoring is included in Appendix K.

It is recommended that piles be driven upon completion of preloading with the approval of the contract administrator's geotechnical engineer or QVE. The approval will be based on the settlement monitoring results and the specified time rate of settlement criteria given in this report. An acceptable rate of settlement prior to driving piles is 3 mm / week over a period of at least two (2) weeks.

7.5.4 Embankment Construction

In areas where piles will be driven the maximum soil or rock particle size should not be greater than 60 mm.

The topsoil thickness varies from 200 mm to 600 mm. For estimating purposes assume an average stripping thickness of 350 mm. After topsoil stripping the exposed subgrade should be inspected, evaluated and approved by a Geotechnical Engineer. Proof rolling and compaction should avoid vibrations that could result in the generation of mud waves.

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

New embankment fill should consist of approved granular soils such as Granular 'B' Type I or SSM. The fill used for construction of the embankments should conform to OPSS 212. Fill placement should meet or exceed the requirements of OPSS 501 and OPSS 206. All fill should be placed in loose lifts not exceeding 300 mm in thickness before compaction. Each lift should be compacted to 95% Standard Proctor Maximum Dry Density (SPMDD) and the final lift to 98% SPMDD.

Proper surface erosion control measures should be implemented on new embankment slopes by seeding and cover (OPSS 804) or sodding (OPSS 803).

7.5.5 Preloading for Ultimate Highway Configuration

For the ultimate widening configuration shown on the GA drawing (Appendix G), preloading is recommended over the entire width of the highway including the future widening. A minimum 8 m (in the longitudinal direction) preloading behind the abutment will be required to minimize different settlement of the abutment between the future widening portion and the current proposed portion. In order to maintain highway drainage, roadway protection along the north edge of the abutments will be required.

It is understood that the preloading behind the future widening portion to the ultimate configuration will be removed after the aforementioned required preloading period has expired.

The embankment widening and preloading location is sufficiently far from existing MTO yard that could be impacted by the settlement deflection basin.

7.6 Construction Considerations

Excavations that need to extend below the prevailing groundwater table (phreatic surface) can be made watertight with interlocking steel sheet piles driven to a toe depth D below base of excavation equal to twice the height of the exterior soil retained above the base of the excavation. Entrapped water within enclosed sheet pile cofferdams can be pumped out from sumps placed within the excavation base. The above-mentioned sheet pile toe penetration requirement is given for a preliminary design purposes only. The soil parameters of Table 7.2.3.3.1 and Section 7.3 may be used for the detailed design of such cofferdams.

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

Open excavations should conform to the requirements of the latest Occupational Health and Safety Act (OHSA) Regulation 213/91, for Type 4 soil.

On-site excavated materials are not acceptable for re-use as engineered fill. They can be re-used for slope flattening or landscaping.

The Enbridge gas main south of the existing bridge imposes construction related limitations that include:

- Compaction equipment energy limits and minimum distance from the gas main
- Pile driving distance from the gas main
- Distance limitations for general widespread dynamic soil compaction
- Soil types fitting the description of Type 4 soil as defined in Article 226 of the Occupational Health and Safety Act.

Impact of construction vibration on the temporary bridge structure and the existing Enbridge gas line should be monitored/assessed during construction of the new highway bridge. An NSSP for vibration monitoring should be issued to cover this aspect.

7.7 Scour and Frost Protection

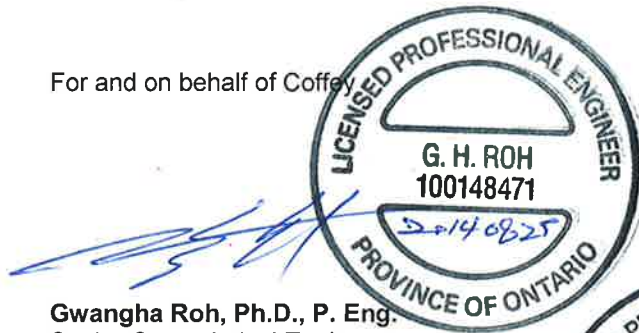
The surficial soils are highly erodible. Proper erosion control and scour protection measures should be provided to minimize creek bank undermining and scour.

The design frost protection depth for the bridge site is 1.5 m. Therefore, a permanent soil cover of about 1.5 m, or its thermal equivalent, should be provided for frost protection of foundations, including pile caps. For rip-rap covers, only half the rip-rap layer thickness should be assumed to be effective in providing frost protection.

8 CLOSURE

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

For and on behalf of Coffey



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Senior Geotechnical Engineer


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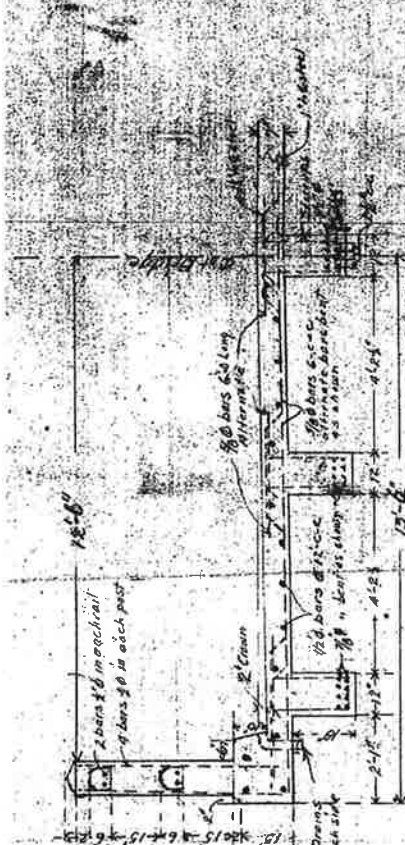
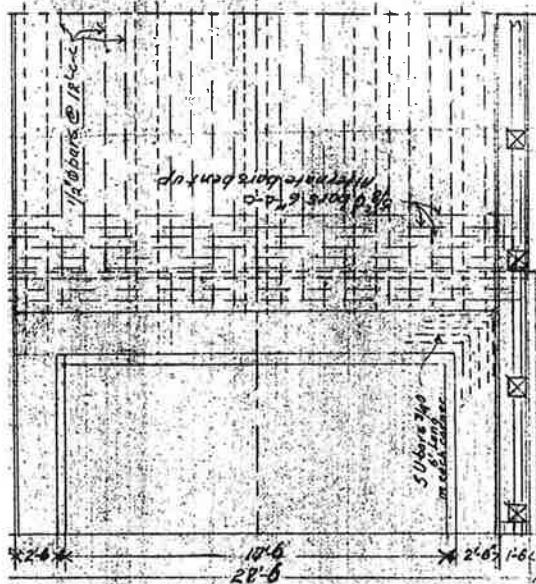



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MTO Designated Contact, Principal

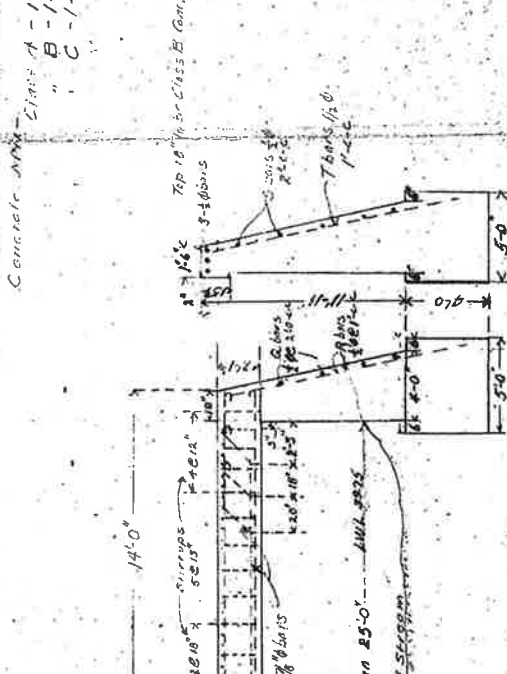
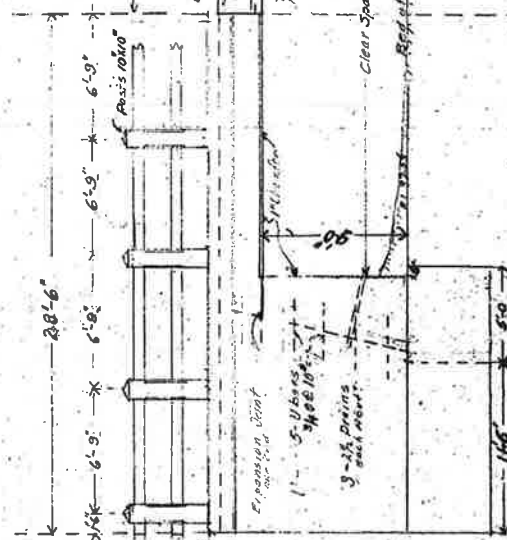


Appendix G

GA Drawing and Embankment Cross-Sections

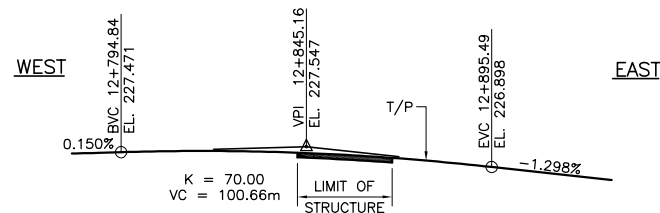


NOTE: Structure built in accordance with the D.H.O. Specifications for Highway Bridges 1933.
Depth of Footings subject to revision by the Engineer.
Concrete strength to be used at all breaks between pours in mass work.
All concrete to be finished.
Only washed, screened gravel shall be used.
Concrete shall be poured continuously.



COUNTY OF SIMCOE
GLASS'S BRIDGE
BRIDGE ON ROAD NO 3
LOT 5-1-1 CON. 1
T.B. OF INNISFIL
LONGING CLASS 'H-15'

H.A. Spellman, B.A.S.
Consulting Eng.
June 1913. Barrie



N.T.S.



DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS								
DESCRIPTION								
DESIGN	AY	CHK	CB	CODE	CHBDC-06	LOAD CL-625-ONT	DATE	JUL/14
DRAWN	WA	CHK	AY	SITE	30-254	STRUCT	SCHEME	DWG 1

SHEET

01



CLASS OF CONCRETE:

PRECAST GIRDERS	50MPa
REMAINDER UNLESS OTHERWISE NOTED	30MPa

CLEAR COVER TO REINFORCING STEEL:

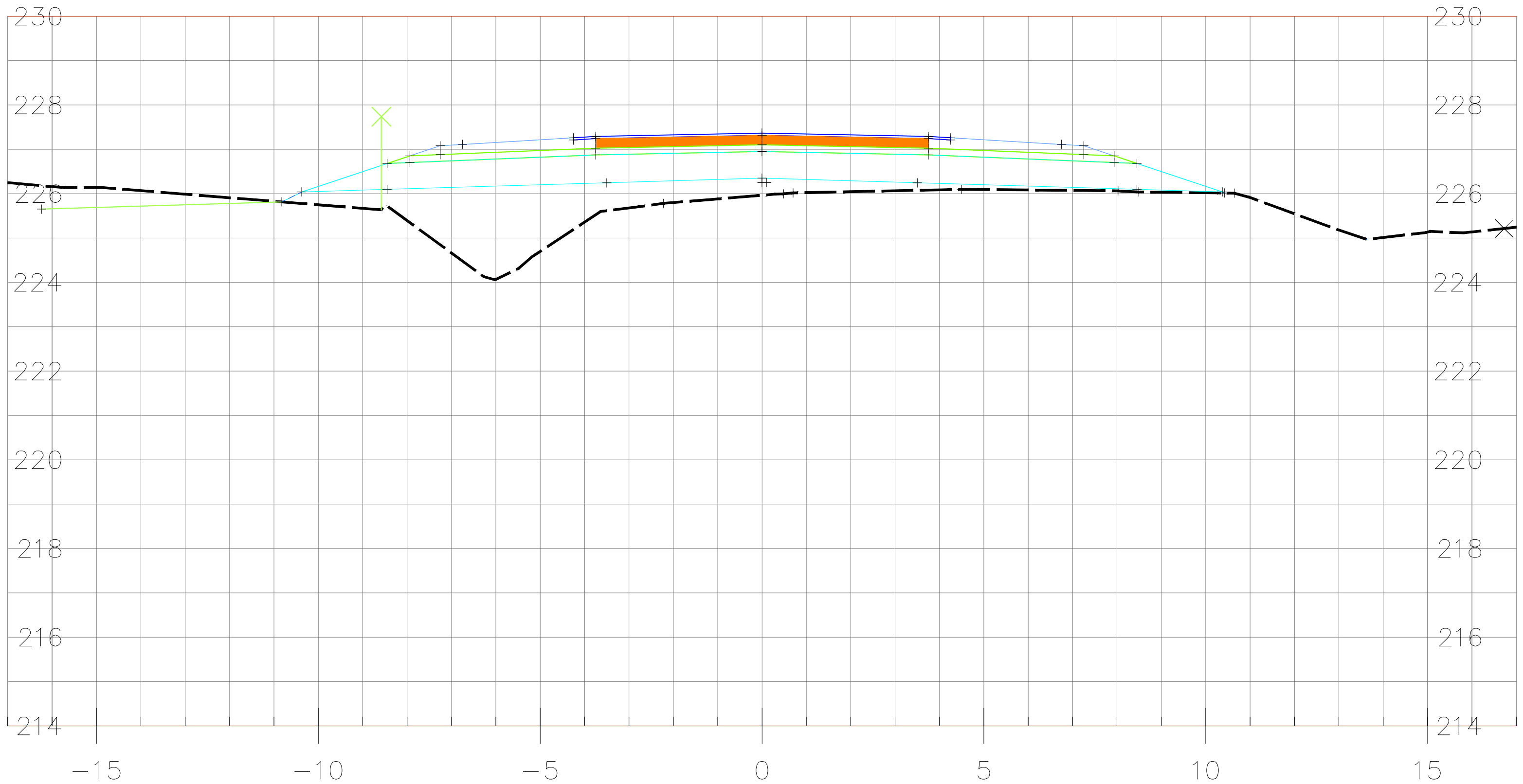
FOOTINGS	100±25mm
TOP OF DECK	70±20mm
REMAINDER UNLESS OTHERWISE NOTED	70±20mm

REINFORCING:

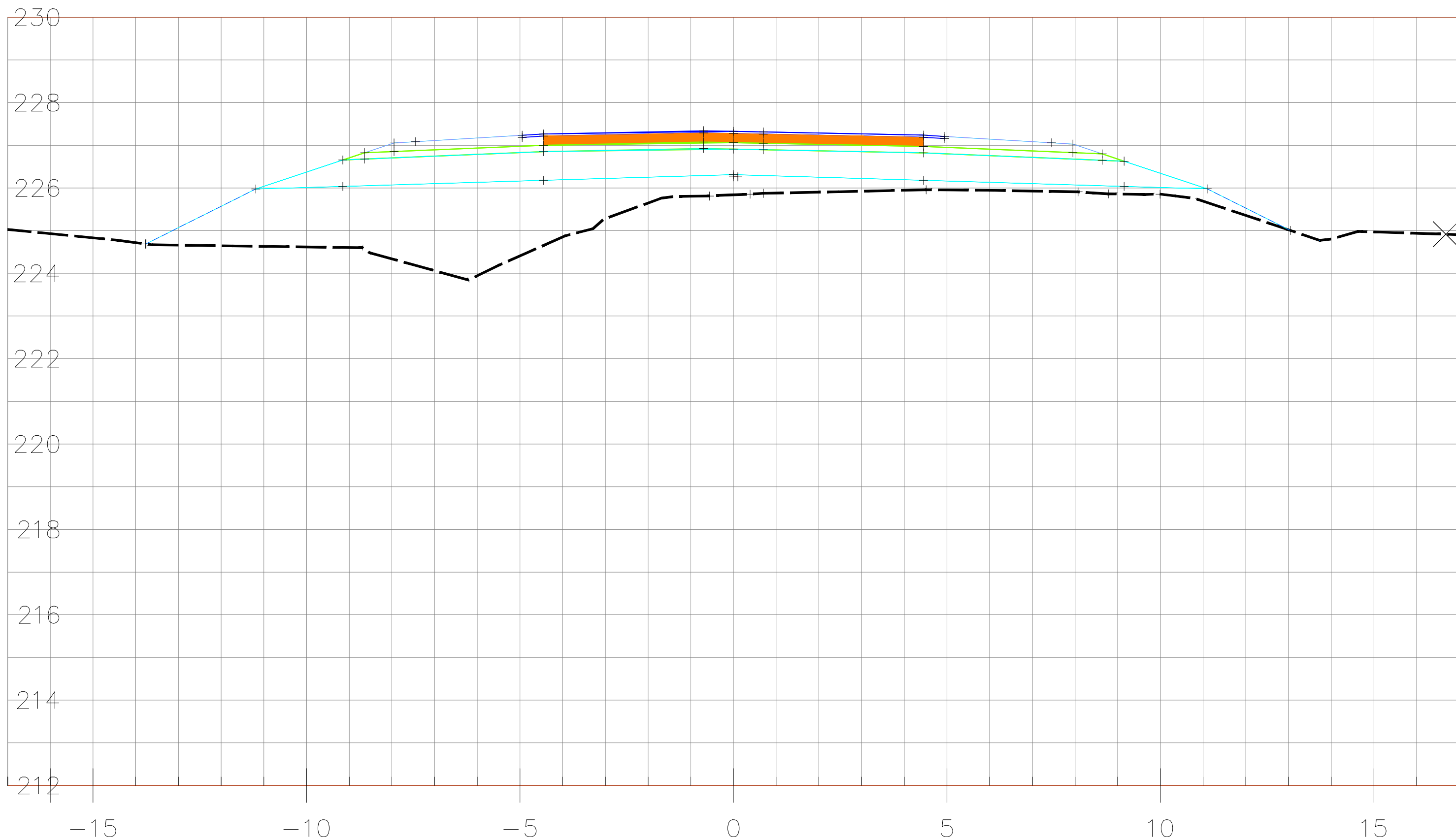
1. REINFORCING STEEL SHALL BE GRADE 400W.
2. UNLESS SHOWN OTHERWISE, TENSION LAP SPICES SHALL BE CLASS B.
3. STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE MINIMUM YIELD STRENGTH OF 500MPA, UNLESS OTHERWISE SPECIFIED.
4. BAR MARK WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
5. GLASS FIBRE REINFORCED POLYMER REINFORCING BARS SHALL BE GRADE I OR GRADE III AS SPECIFIED IN THE CONTRACT DRAWINGS. THE NOMINAL DIAMETER, TENSILE MODULUS OF ELASTICITY AND GUARANTEED MINIMUM TENSILE STRENGTH SHALL BE AS SPECIFIED IN THE CONTRACT DOCUMENTS. BAR MARKS WITH THE PREFIX GI DENOTE GRADE I GLASS FIBRE REINFORCED POLYMER BARS AND BAR MARKS WITH THE PREFIX GIII DENOTE GRADE III FIBRE REINFORCED POLYMER BARS.
6. BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12-1, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES:

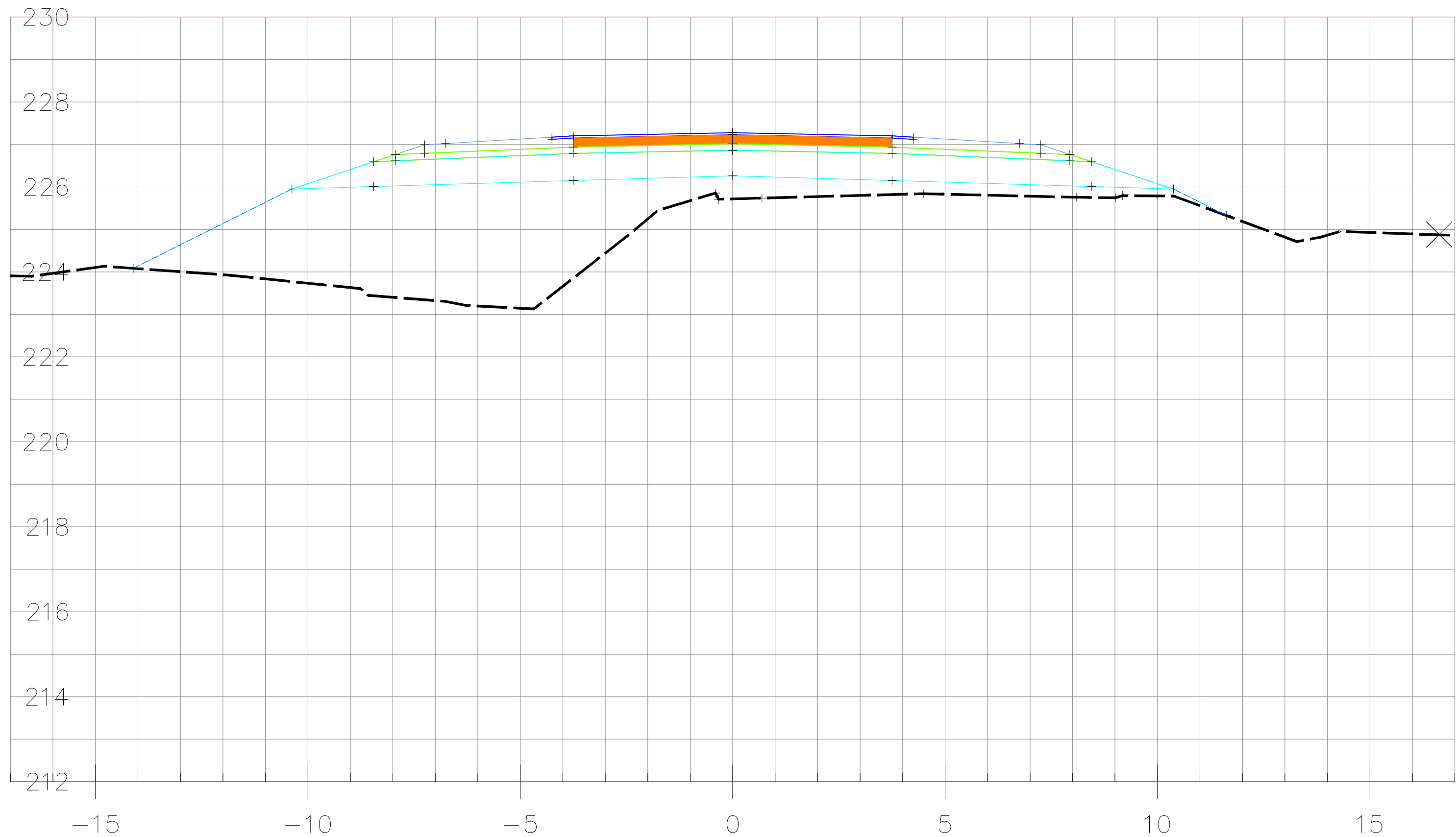
1. PILES TO BE DRIVEN IN ACCORDANCE WITH STANDARD SS103-11 USING AN ULTIMATE GEOTECHNICAL RESISTANCE OF 1240 kN PER PILE BUT NOT BELOW EL. 202.0 WITHOUT APPROVAL OF THE CONTRACT ADMINISTRATOR.
2. THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN IN THE DRAWINGS, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
3. NO BACKFILL SHALL BE PLACED ABOVE TOP OF ABUTMENT BEARING SEAT UNTIL DECK CONCRETE HAS REACHED 75% OF ITS STRENGTH.
4. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ENDS OF DECK KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 0.5m.
5. PROTECTION SYSTEMS REQUIRED TO COMPLETE THE WORK SHALL BE DESIGNED BY THE CONTRACTOR. LIMIT OF PROTECTION SYSTEMS TO BE DETERMINED BY THE CONTRACTOR.
6. ALL ELEVATIONS ARE TO GEODETIC DATUM.



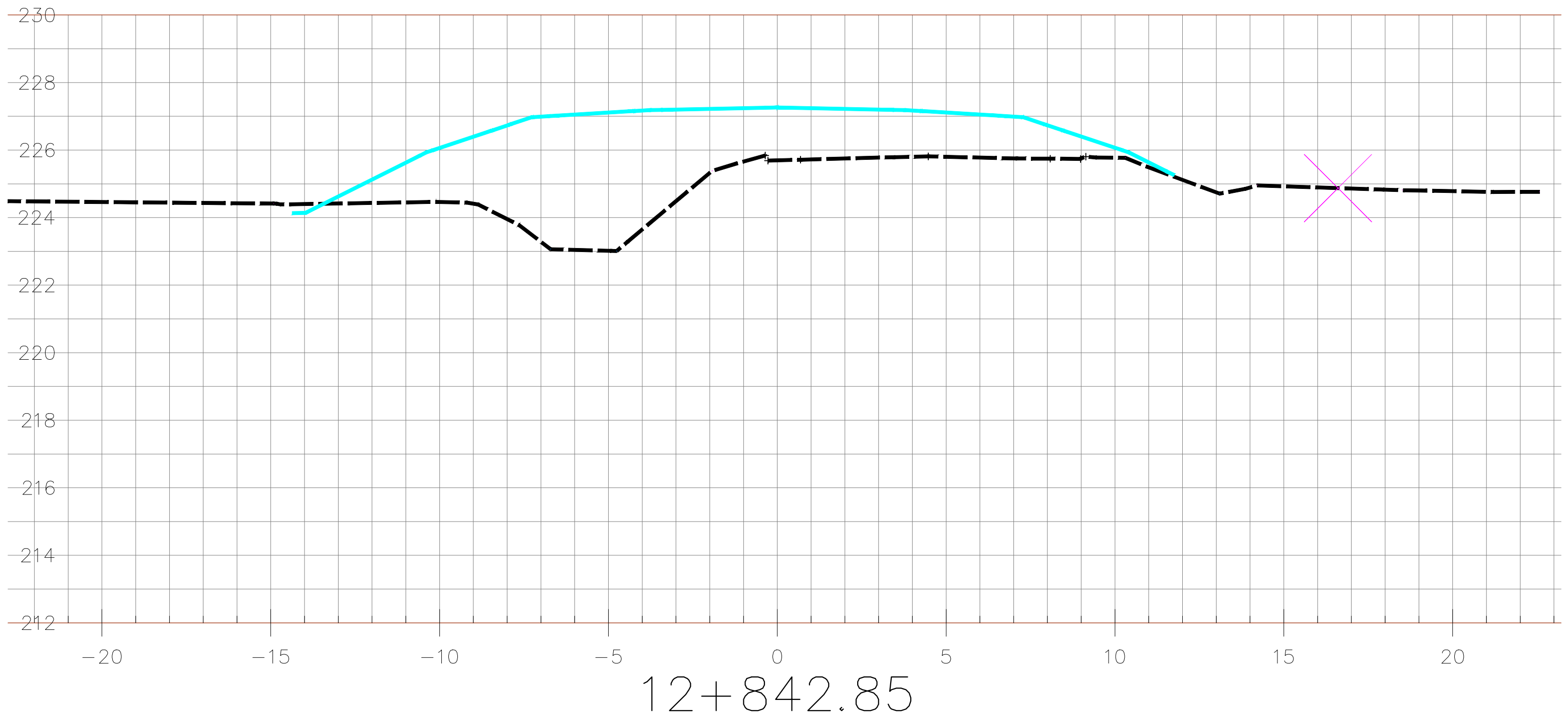
12+820.00

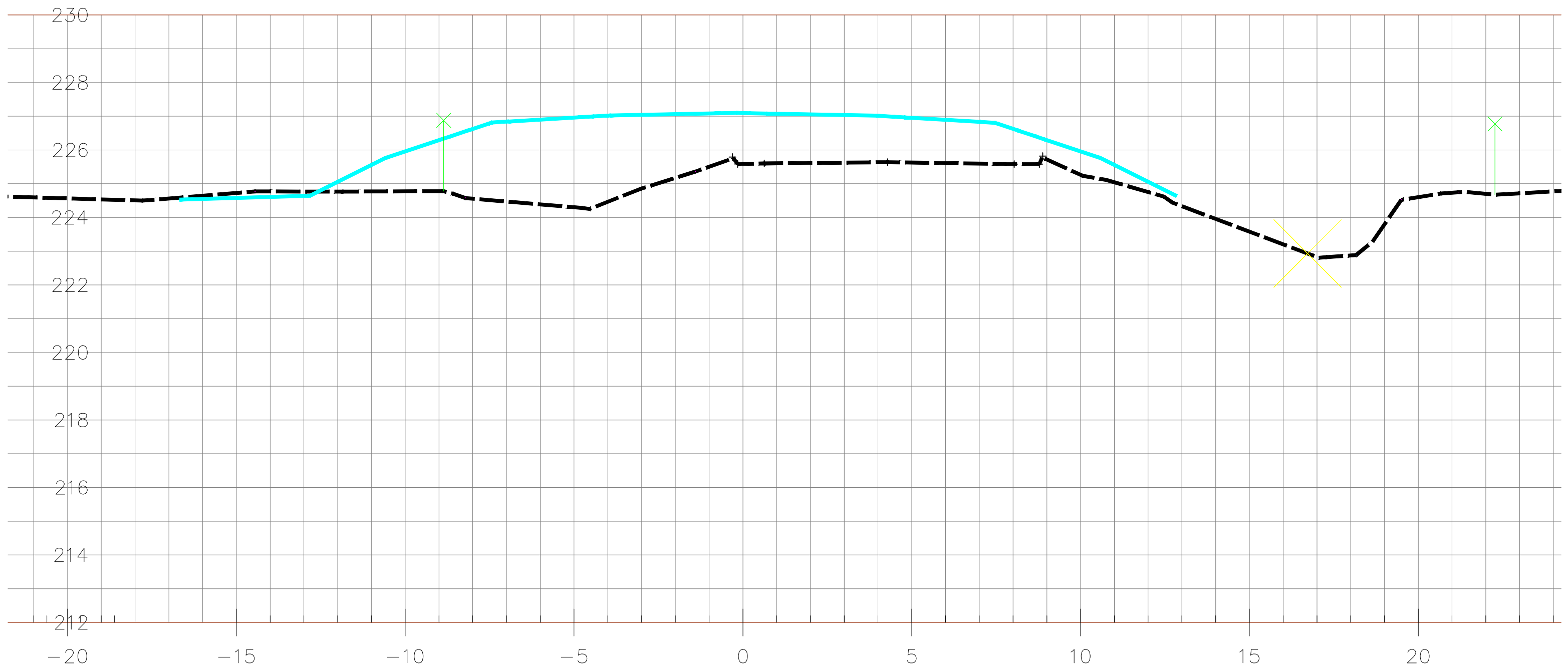


12+830.00

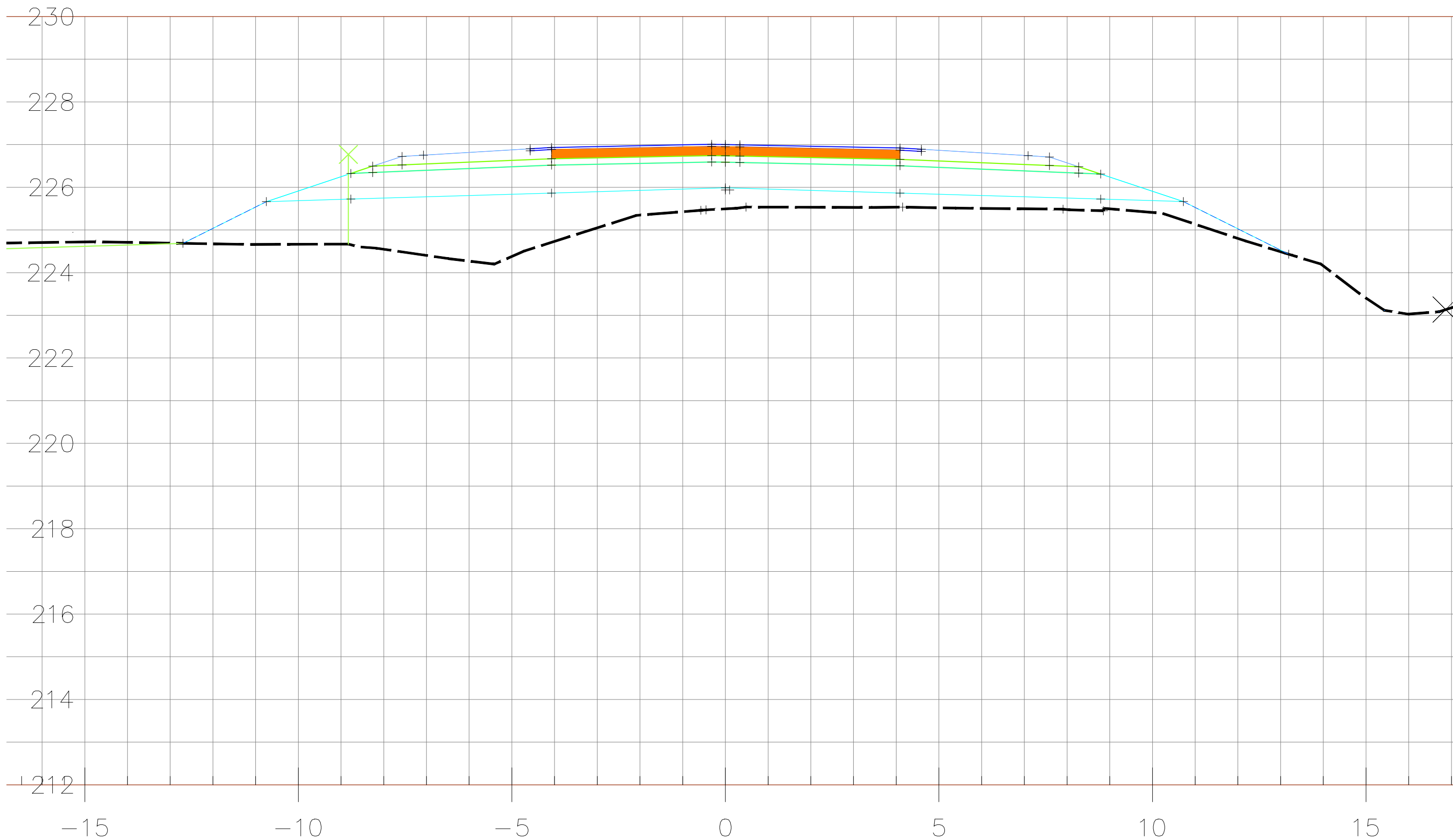


12+840.00

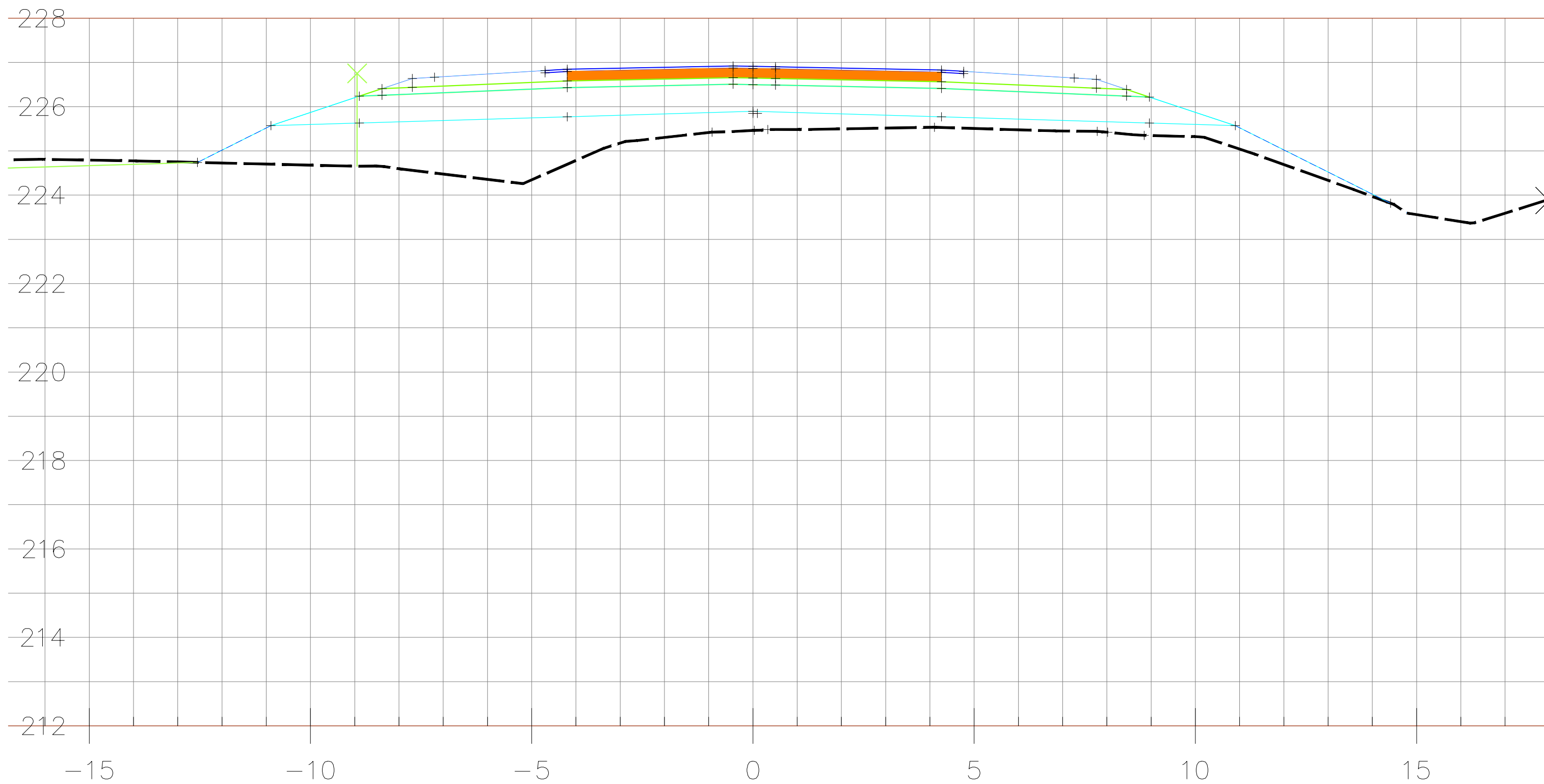




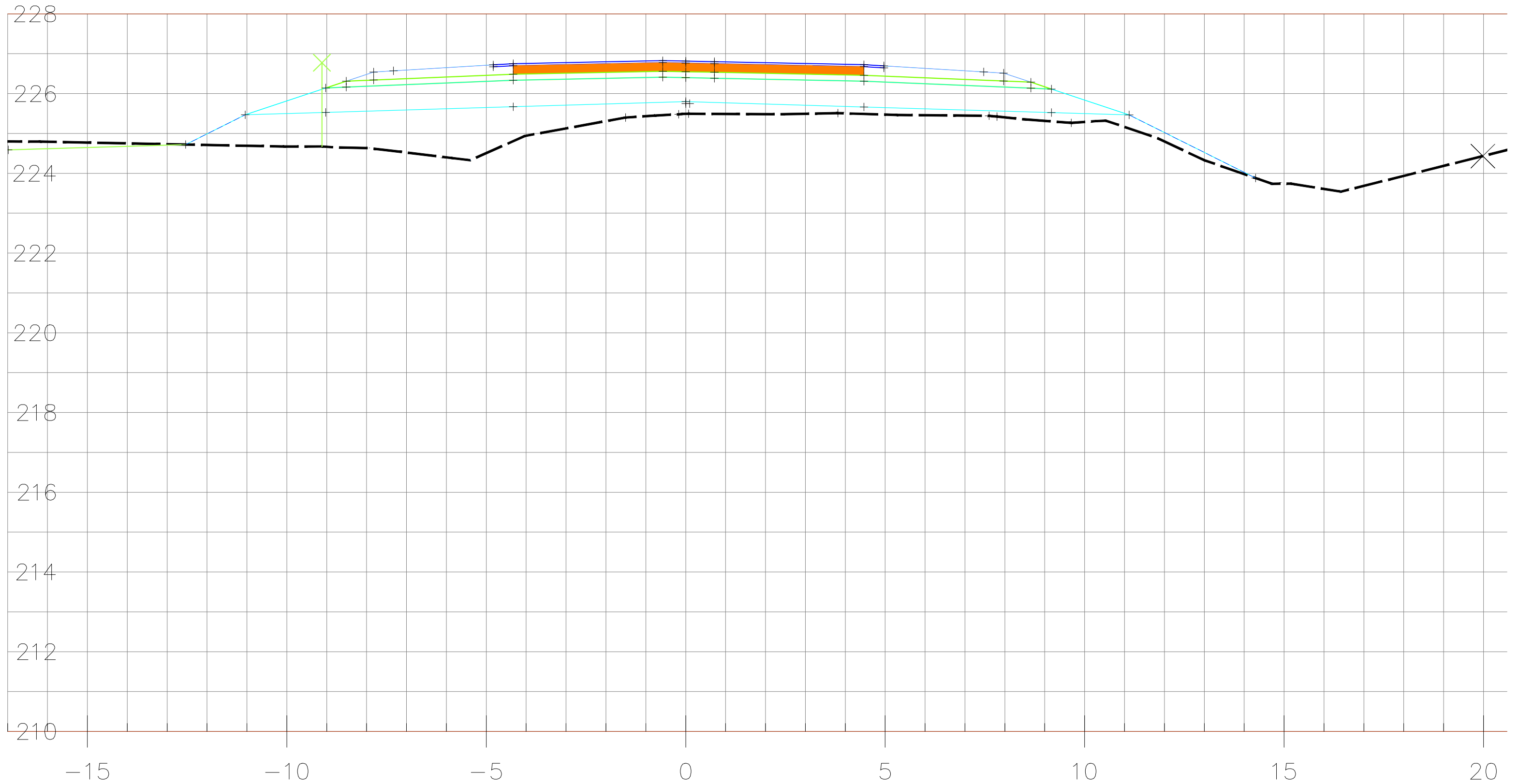
12+867.90



12+880.00



12+890.00



12+900.00

Appendix H

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

Table H-1

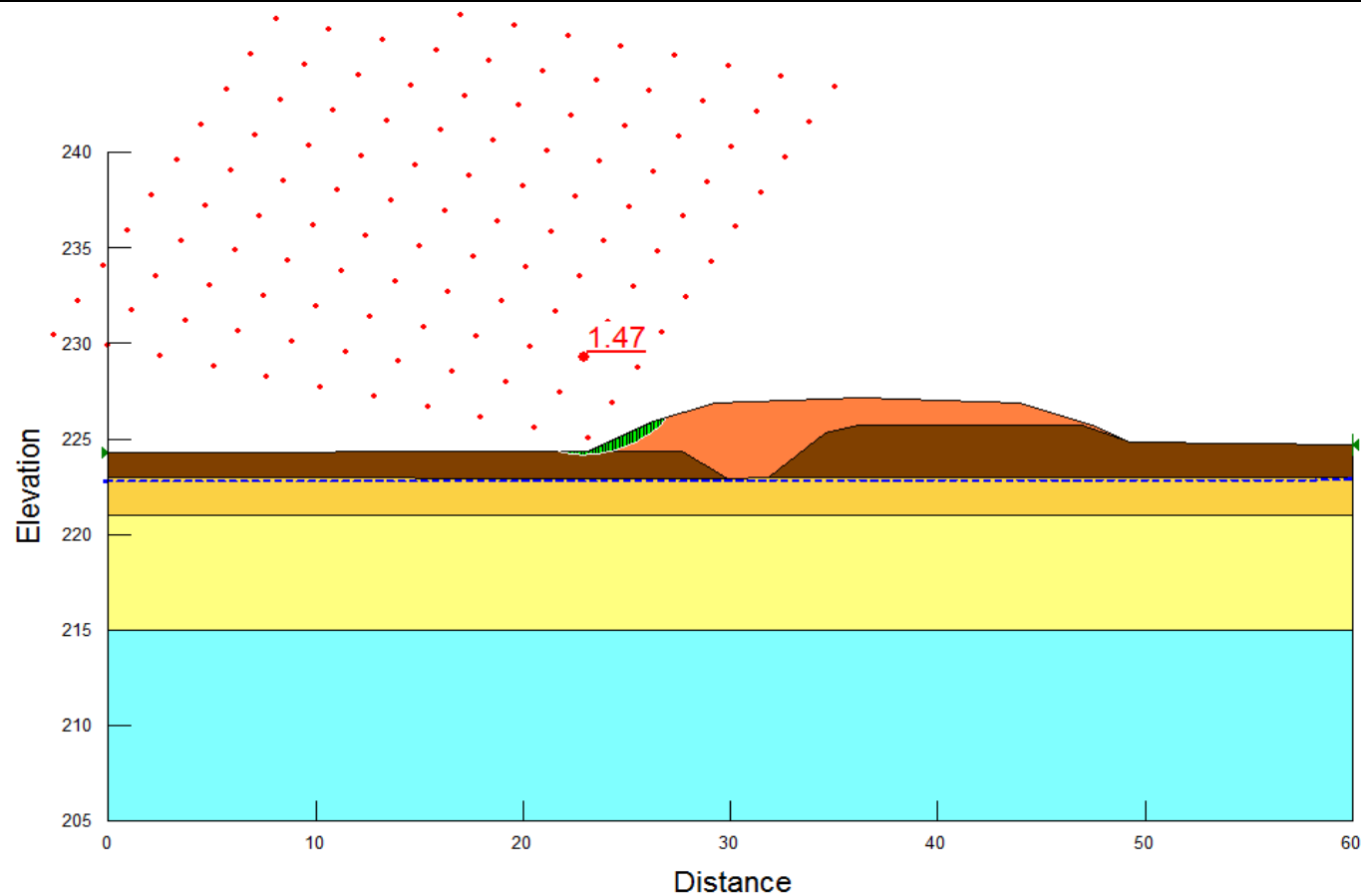
Foundation Options for New Bridge Over Innisfil Creek

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations	-Low cost but not suitable for the prevailing subsurface conditions	-Prone to excessive settlements	Low cost	-Not feasible due to the prevailing subsurface conditions and proposed grade raise
Driven steel H-piles foundations	-Low displacement piles and as such more suitable than other types of driven piles such as precast concrete or steel tube piles	-Vibration control is required due to proximity of existing gas line. -Artesian conditions may reduce resistance of pile if piles penetrate sufficient into the dense silt layer.	Moderate cost	-Feasible option -Vibration control during pile driving is required
Driven Steel Tube Pile Foundations	-Similar to driven steel H-piles, except they are higher displacement piles in comparison with H-piles	- Vibration control is required due to proximity of existing gas line. -Artesian conditions may reduce resistance of pile if piles penetrate sufficient into the dense silt layer.	Moderate Cost	-Feasible option -Vibration control during pile driving is required
Timber Piles	-Low cost but less reliable than steel piles due to environmental reasons.	-May be subject to deterioration in time -There may be problems during driving due to variable soil strength in the upper silty clay	Low	-Unlikely to be acceptable to MTO, not recommended

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	<ul style="list-style-type: none"> -Less vibrations and noise created than driven piles. -Deeper caisson may be required to maintain design loads 	<ul style="list-style-type: none"> -Possible hole instability issues during installation -Problems will arise if installed in the intermediate fine grained granular soils due to the observed artesian conditions. The caisson will therefore have to be terminated within the upper silty clay, despite low resistances 	More expensive than driven piles	-Feasible option but less suitable for the prevailing subsurface conditions from Geotechnical point of view in comparison with driven steel piles as well as offering low resistances
Expanded Base Piles	<ul style="list-style-type: none"> -Higher end bearing -Long term settlement 	-Vibration induced by forming bulb	More expensive than driven piles	-Not recommended due to the nearby existing gas pipe line

Appendix I

Slope Stability Analyses Results



Section / Location : Sta 12+842.85 (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 (°)
New Embankment Fill	20.5	0	32
Existing Embankment 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

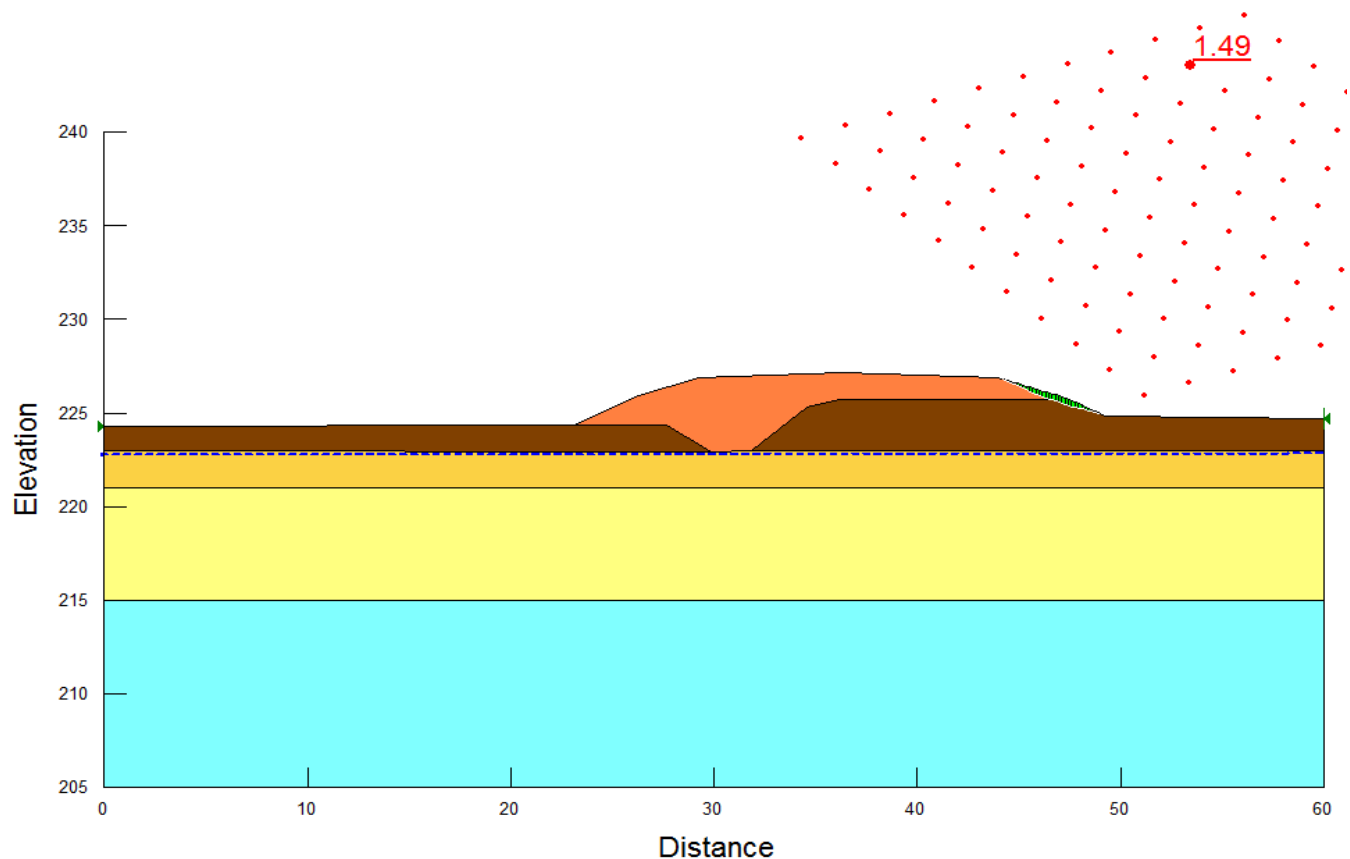


SLOPE STABILITY ANALYSIS
 Innisfil Creek Bridge Approach Embankment

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

HIGHWAY 89

FIGURE I-1



Section / Location : Sta 12+842.85 (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 (°)	
New Embankment Fill	20.5	0	32	
Existing Embankment 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	



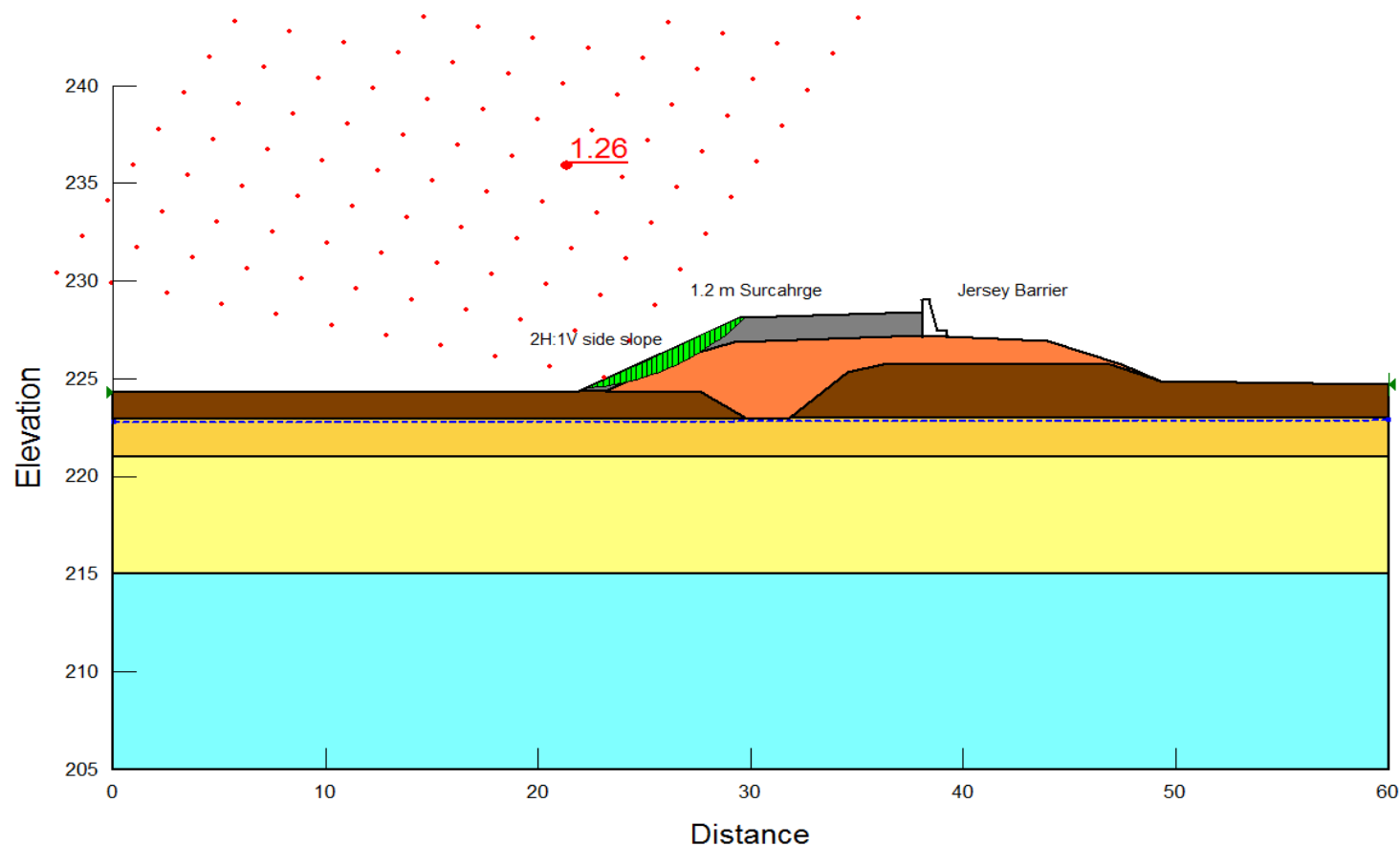
SLOPE STABILITY ANALYSIS

Innisfil Creek Bridge Approach Embankment

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

HIGHWAY 89

FIGURE I-2



Section / Location : Sta 12+842.85 (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 (°)
New Embankment Fill	20.5	0	32
Existing Embankment 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
Surcharge	19.0	0	27



SLOPE STABILITY ANALYSIS

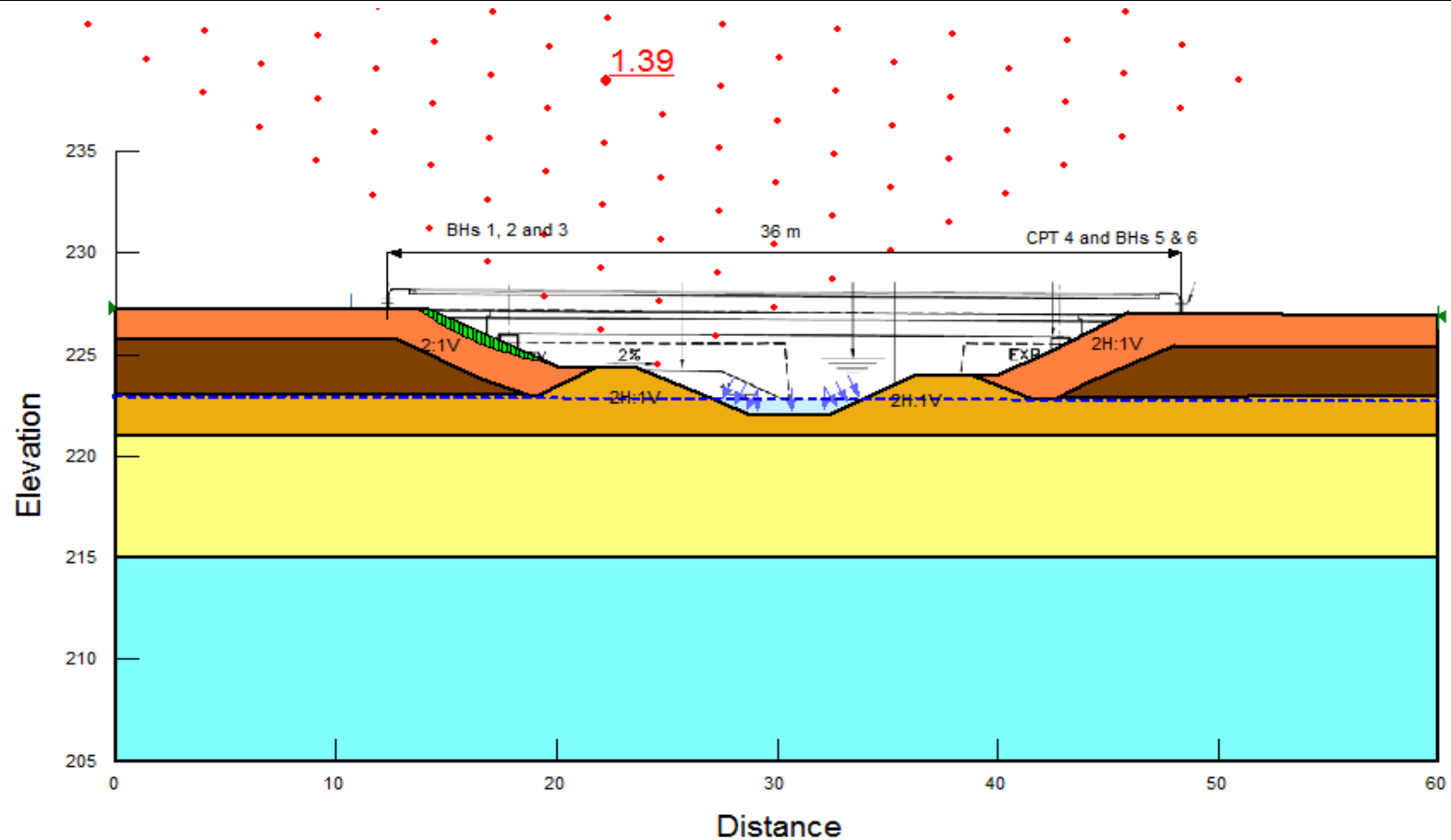
Innisfil Creek Bridge Embankment Surcharge Stability

PROJECT NO.: TRANETOB20462AA
 Analyzed by: GR

DATE: Jan, 2014
 Reviewed by: ZO

HIGHWAY 89

FIGURE I-3



Section / Location : centreline profile (west)
 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 ($^\circ$)
New Embankment Fill	20.5	0	32
Existing Embankment 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



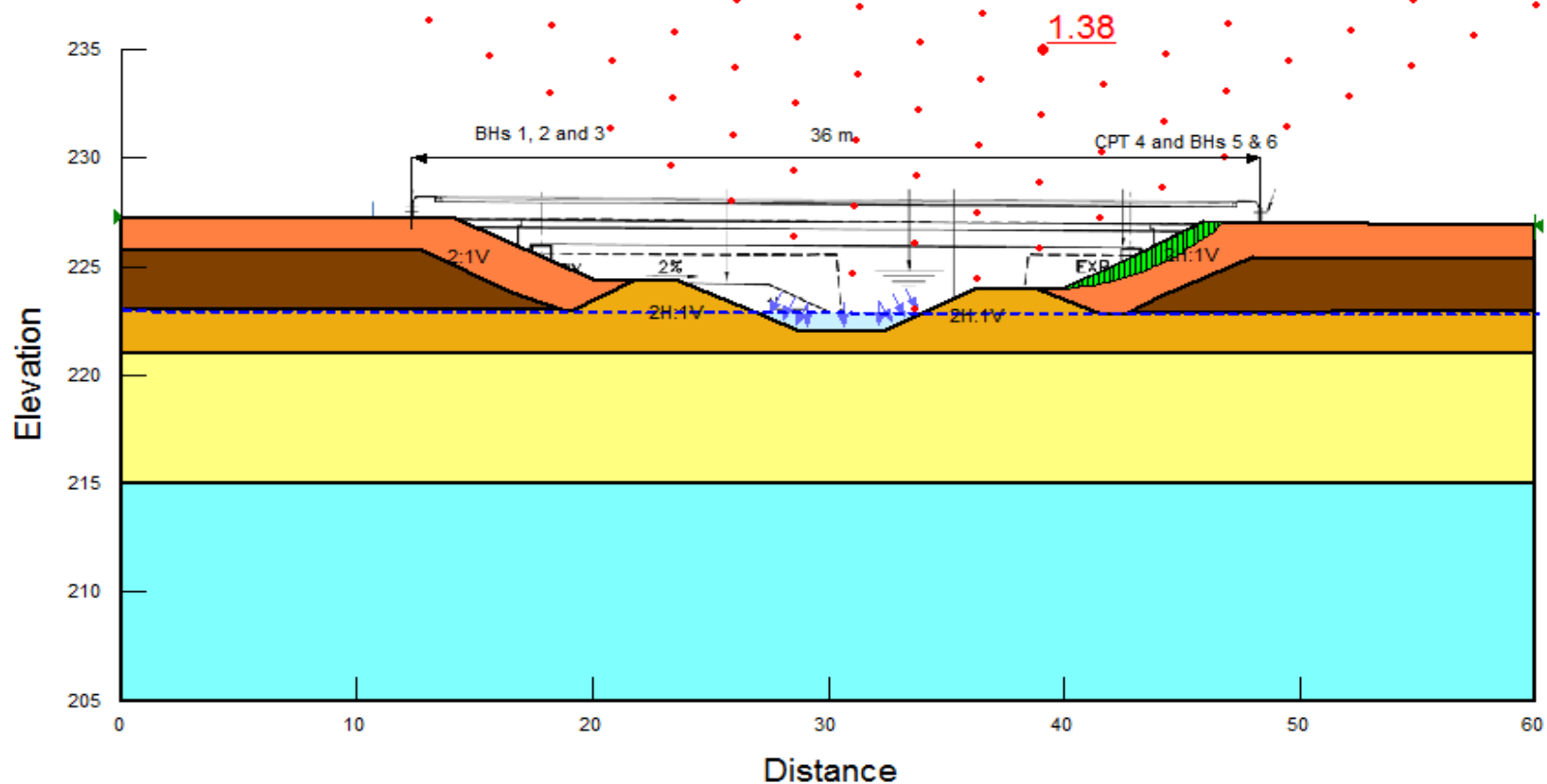
SLOPE STABILITY ANALYSIS

Innisfil Creek Bridge Forward Slope

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

HIGHWAY 89

FIGURE I-4



Section / Location : assumed profile (east)
 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 (°)
New Embankment Fill	20.5	0	32
Existing Embankment 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



SLOPE STABILITY ANALYSIS

Innisfil Creek Bridge Forward Slope

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

HIGHWAY 89

FIGURE I-5

Appendix J

Settlement-Time Estimation

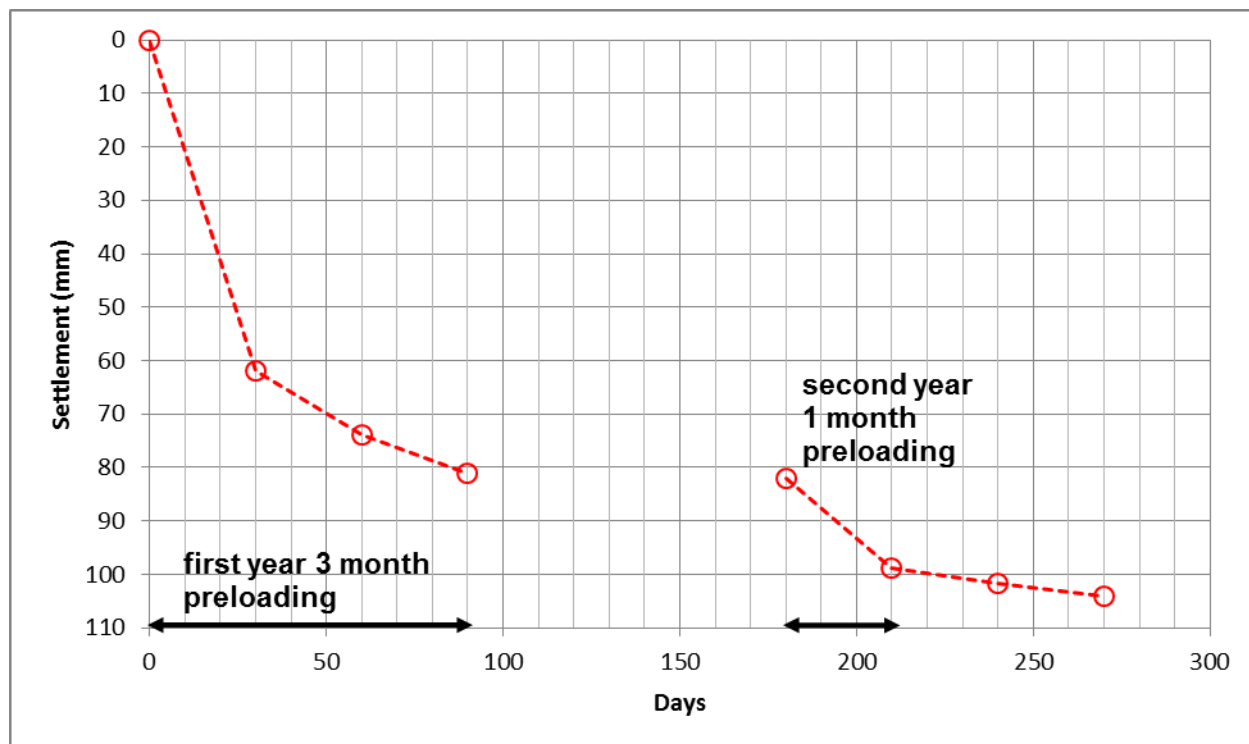


Figure J-1 Settlement-Time Estimation

Appendix K

List of OPSS, OPSD and Non-standard Specifications

List of OPSDs, OPSSs and Non-standard Specifications

OPSDs

OPSD 208.01 Benching of Earth Slopes

OPSD 3101.150 Walls, Abutment, Backfill, Minimum Granular Requirement

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

Driven Steel Piles

The very dense silt deposit below elev. 199-198 m, if penetrated sufficiently during pile driving and installation, may result in artesian flow that will be required to be stopped. The cost of controlling and stopping artesian flow, to the satisfaction of MTO and MOE, shall be the sole responsibility of the contractor”.

Artesian Condition

The presence of artesian condition should be noted before bidding this project. In addition, mitigating measures should be prepared in the contract to alleviate the effects of the artesian conditions.

Vibration Monitoring

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

Impact of construction vibration on the existing Enbridge gas line should be assessed by an independent engineer and monitoring criteria (review and alert levels) should be set up before 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.

SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT - Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the observation of ground settlement at locations as specified when roadway embankment preloading commences on the revised alignment of Highway 89 on either side of Innisfil Creek Bridge.

This special provision describes the requirements for the supply, installation and monitoring of square steel settlement plates with survey rods (the instruments) to determine the effectiveness of preloading. The purpose of these instruments is to monitor the settlement of the foundation soils under the preloading and embankment widening. The ground movements shall be measured by level surveying with total station equipment at the top of the settlement rods or as provided for in the alternative option described in Section 7.3.

2.0 REFERENCES - Not Used

3.0 DEFINITIONS - Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Frequency of Monitoring

Settlement monitoring data shall be provided to the Contract Administrator or QVE along with copies of all field notes, in accordance with the schedule shown below:

- Immediately after installation, indicating the location of the permanent reliable benchmarks used for ongoing settlement.
- When filling commences
- As the ground elevation above the monitoring location rises by 500 mm, for every 500 mm of elevation rise, or fraction thereof, until the final grade elevation has been achieved.
- Daily for the first two weeks after commencement of initial filling and once a week thereafter for the first three months or the duration of the preloading period. When/if new fill is added subsequently, settlement monitoring shall be performed once every two days for the first month and weekly thereafter.
- Further monitoring shall not be required once the time rate of settlement falls below 2 mm/week.
- The Ministry reserves the right to alter the frequency and duration of settlement monitoring

4.2 Measurement Tolerances

The tolerances for settlement monitoring shall be as follows:

- Instrument location: ± 100 mm.
- Error of survey closure from permanent bench mark ± 2 mm.

5.0 MATERIALS

5.1 Steel Base Plate

The Contractor shall supply minimum Grade 300 steel base plates with a thickness of at least 6.35 mm. The plates shall be square and at least 0.5 m by 0.5 m in plan dimensions.

5.2 Survey Rod

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described elsewhere. The rods shall be welded to the settlement monitoring plates. The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to for repeated observations.

5.3 Friction Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 - 50.8 mm O.D. PVC pipe cut perpendicular to the axis of the pipe.

5.4 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod within the embankment. The surround shall consist of 300 mm diameter corrugated steel pipe (CSP - OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the Friction Reduction Sleeve (PVC pipe) shall be filled with loose sand.

5.5 Survey Benchmarks

The Contractor shall provide local, stable and non-settling survey benchmarks located a minimum distance of 20 m from any instrument location. The number and locations of benchmarks shall be such that direct sighting is possible from all settlement plate locations to at least one bench mark. Elevations shall be surveyed to an accuracy of ± 2 mm or better. Prior to the installation of the instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground elevation at each instrument location.

6.0 EQUIPMENT

Total station survey equipment, calibrated to achieve the tolerance specified in 4.3.

7.0 CONSTRUCTION

7.1 Instrumentation Locations

The locations of the settlement plates are shown in Table 1.

Table 1 – Ground Settlement Monitoring Locations (see NOTES)

Station	Offset from CL (m)	H (m) ±	Remarks
12 + 770	6.0 Lt.	1.8 – 2.2	
12 + 820	6.0 Lt.	3.0 – 3.4	Low ground adjacent to existing road
12 + 820	0.0 on CL	1.0. – 1.5	On existing pavement
12 + 841	5.0 Lt	3.5 - 4.0	West Abutment
12+841	9.0 Lt	3.5 – 4.0	Future Widening of West Abutment
12 + 869	5.0 Lt	3.5 – 4.0	East Abutment
12 + 869	9.0 Lt	3.5 – 4.0	Future Widening of East Abutment
12 + 880	5.0 Lt.	2.5 – 3.0	
12 + 920	6.0 Lt.	2.0 – 2.5	

NOTES to Table 1:

1. CL = Centreline of Re-aligned Highway 89
2. Offset from CL – Left (Lt.) or Right (Rt.) facing increasing chainage
3. H = Anticipated height of preload fill.

7.2 Settlement Plates

The settlement plates shall be installed immediately after subgrade preparation is completed but prior to preload fill placement. As embankment widening and preload proceeds the rods shall be extended above the new top of embankment widening. Sleeves shall be installed around the rods to reduce friction and allow uninhibited movement of the rod with the plate. A protective surround shall be extended with the rods as embankment and preload construction proceeds.

The settlement plate shall be installed horizontally on undisturbed native soil. The elevation of the upward facing base of the plate shall be surveyed before backfilling. The rod shall be fixed to the centre of, and perpendicular to, the plate. The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings. The friction reducing sleeve shall be placed over the entire length of the rod that is below ground and within the embankment, except that the cap on top of the settlement rod shall extend 25 mm above the top of the friction sleeve at all times. The settlement rods shall be extended upwards as fill is placed such that the top of the rod is always at least 0.3 m but not more than 2 m above the surrounding fill. The CSP, Friction Reducing Sleeve and sand protective surround shall be extended with the rods. The settlement rod shall be in the centre of the CSP and friction-reducing sleeve. The annulus between the CSP and the friction-reducing sleeve shall be filled with loose sand to a level not higher than the top of the sleeve.

The elevation, easting and northing of the centre of the base of the plate and the top of the rod shall be surveyed prior to preload placement. The total distance from the base of the plate to the top of the rod shall be measured to an accuracy of ± 2 mm.

7.3 Alternate Methods of ground Settlement Monitoring

The Contractor may propose alternative methods for measuring the settlement of the ground beneath the re-aligned embankment widening and preload fill. The proposal must be signed off by a Professional Engineer and shall be approved by the Ministry prior to implementation. The alternative method shall provide progressive cumulative total and time rate of settlement data of the existing ground at locations indicated in Table 1, Section 7.1, and at the frequency and to the tolerances given respectively in Sections 4.1 and 4.2.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment, and materials required to do the work.

Appendix L

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.