

**Foundation Investigation and  
Design Report  
Highway 401 & Veterans  
Memorial Parkway  
Interchange Reconfiguration**

Highway 401, City of London

DB Contract 2015-3002

G.W.P. 3033-11-00

Geocres No.



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FOUNDATION INVESTIGATION AND DESIGN REPORT  
HIGHWAY 401 & VETERANS MEMORIAL PARKWAY INTERCHANGE RECONFIGURATION

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For  
DB 2015-3002  
G.W.P 3031-11-00

Highway 401 / Veterans Memorial Parkway Interchange Modifications  
City of London

## 1.0 INTRODUCTION

Dufferin Construction (Dufferin) is constructing the modifications to the Highway 401 / Veterans Memorial Parkway (VMP) interchange in the City of London, Ontario, for the Ministry of Transportation of Ontario (MTO) under a Design-Build agreement. Stantec Consulting Ltd. (Stantec) was retained by Dufferin to undertake the detailed design for modifications to the Highway 401 interchange at Veterans Memorial Parkway.

The proposed modifications include the reconfiguration of the existing interchange and the extension of the VMP to Wilton Grove Road located approximately 800 m south of Highway 401. As part of the proposed reconfiguration, the existing bridge carrying the VMP over Highway 401 will be replaced. The improvements will also include the following: revised highway access ramps and approach embankments including sections of geogrid-reinforced slopes, a new structural culvert where the realigned Crinklaw Drain channel passes beneath the VMP on the south side of Highway 401 and four new overhead signs.

This Foundation Investigation Report has been prepared specifically and solely for the proposed interchange modifications, culvert replacement and overhead signs.

Project Number: G.W.P.: 3033-11-00

Agreement Number: DB Contract 2015-3002

Project Location: Highway 401 at Veterans Memorial Parkway, London

## 2.0 SITE DESCRIPTION AND GEOLOGY

### Site Location

The site location is shown on the Key Plan inset to Drawing No. 1A, provided in Appendix A.

### General Site Description

At the project site, Highway 401 is a six-lane (three lanes in each direction) divided freeway that runs approximately in an east-west orientation. The existing Veterans Memorial Parkway is a four-lane (two lanes in each direction) divided roadway which runs north-south, is located on the north side of the highway and terminates at the Highway 401 interchange.

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The existing drainage at this site consists of catch basins along the paved center median of Highway 401, leading to storm sewers, along with ditches and culverts along the outside lanes of Highway 401 and Veterans Memorial Parkway. The Crinklaw Drain is present near the toe of the W-N ramp at the south side of the existing interchange.

In the vicinity of the project site the terrain is generally flat with gentle slopes. The areas located inside of the existing interchange ramps are typically higher than surrounding site grades and the site grade in this area is inferred to have been raised by placement of fill materials.

Chainage increases from west to east on Highway 401 and north to south on Veterans Memorial Parkway.

### Physiographic Description

The site is located within a physiographic region known as the Mount Elgin Ridges (Chapman and Putnam, 1984). The ridges are generally moraines of pale brown calcareous clay or silty clay, whereas the vales generally consist of alluvium deposits of gravel, sand or silt.

## 3.0 INVESTIGATION PROCEDURES

### 3.1 REVIEW OF EXISTING INVESTIGATION

Foundation borehole records for boreholes previously advanced in the immediate vicinity of the Highway 401 – VMP Interchange were reviewed as part of the preparation of the current report. The investigation results were made available to Stantec by MTO. The existing information included boreholes designated as Boreholes BH 1 to BH7 (1974 Investigation by MTO), Boreholes BH1 and BH2 (2012 investigation by the Infrastructure Engineering Group Inc.), and Boreholes 13-01 to 13-11 (2013 investigation by Thurber Engineering Ltd.). The borehole locations and strata plots incorporating boreholes information from the previous investigations are shown on the Borehole Location and Soil Strata Plans, Drawing Nos. 1A to 1D in Appendix A. Copies of borehole records from the previous investigations are included in Appendix B.

Review of the existing information from previous investigations at the site indicates that the subsurface soils within the general area of the interchange consist predominantly of surficial fill materials underlain by silty clay/clayey silt tills that typically extend to depths of approximately 10 m to 13 m below the original ground surface. The upper silty clay/clayey silt till is underlain by an interlayered sequence of sand and clayey silt/silty clay till deposits that extend to depths of at least 40 m below ground surface.

### 3.2 FIELD INVESTIGATION

The foundation investigation for the detail design of the proposed interchange modifications, culvert replacement and overhead signs consisted of testing at a total of 14 locations. The testing program included advancement of five boreholes (BH16-1, BH16-2, BH16-5, BH16-6 and BH16-8) and three electronic cone penetration tests (CPTs) (CPT16-6 through CPT16-8) in the

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area of bridge structure and interchange, 2 boreholes (BH16-3 and BH16-4) at the location of the new Crinklaw Drain culvert, and four boreholes (BH16-9 through BH16-12) at the overhead sign locations.

The locations of the boreholes and CPT soundings are shown on the Borehole Location and Soil Strata Plans, Drawing Nos. 1A to 1D, in Appendix A.

Prior to carrying out the investigation, Stantec contacted the public utility authorities to clear the borehole locations of both private and public utilities.

The field drilling program was carried out from April 4, 2016, to April 11, 2016. The boreholes were advanced using continuous flight hollow stem augers. Drilling was carried out with truck-mounted and track-mounted drill rigs, both equipped for soil sampling. The CPTs were advanced using a 25 ton truck-mounted CPT rig.

The subsurface stratigraphy encountered in each borehole was recorded in the field by an experienced Stantec field technician. Standard Penetration Tests (SPT) were carried out in the drilled holes and split spoon samples were collected at regular intervals. Fourteen relatively undisturbed Shelby Tube samples were also retrieved. All recovered SPT samples were returned to our Ottawa and Kitchener laboratories for detailed classification and testing. The Shelby Tube samples were sent to our Ottawa laboratory for consolidation and unconfined strength testing.

The undrained shear strengths of cohesive soils were determined using a combination of in-situ shear vane and laboratory unconfined compressive tests. As the clayey soils at the site were generally very stiff or hard, the in situ vane testing was completed using a vane with dimensions smaller than the standard MTO 'N' vane to allow for higher undrained shear strength measurements.

Groundwater monitoring wells were installed in three boreholes: BH16-1, BH16-2 and BH16-4; the monitoring wells were 50 mm in diameter. Slotted pipes were installed in these boreholes to depths of 1.5 m, 21.3 m and 5.7 m, respectively. The annulus around the slotted pipe section was backfilled with sand. The borehole annulus below and above the screened sections of the pipe was backfilled with bentonite.

Groundwater level measurements were carried out on April 22 and May 3, 2016. Groundwater was also observed in a number of open boreholes during drilling and dissipation testing was carried out in the CPTs to determine the water levels in the sand layer.

After completion of drilling, boreholes were backfilled with a mix of bentonite and drill cuttings. Boreholes advanced on the roadways were sealed with cold patch asphalt.

### 3.3 LOCATION AND ELEVATION SURVEY

The location and ground surface elevation at each borehole and CPT sounding location were surveyed by Stantec's Geomatics group in London, Ontario, after completion of the drilling investigation. Summary information pertaining to the Stantec boreholes included in this report is given in Table 3.1.

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**Table 3.1: Borehole and CPT Information Summary**

Test Hole	MTM Zone 11 Coordinates		Ground surface elevation (m)	Total depth drilled (m)	End of borehole elevation (m)	Number of soil samples
	Northing	Easting				
BH16-1	4757077.7	416890.5	274.4	9.8	264.6	9
BH16-2	4756895.6	416944.4	281.4	35.7	245.7	20
BH16-3	4756605.5	416987.5	273.1	9.8	263.3	9
BH16-4	4756620.4	417023.5	273.5	9.8	263.7	9
BH16-5	4756664.9	417017.1	273.8	9.8	264.0	9
CPT16-6	4756919.4	416946.2	282.0	30.6	251.4	-
BH16-6	4756914.9	416939.9	281.3	2.4*	278.9	-
CPT16-7	4756797.9	416976.9	281.8	25.0	256.8	-
CPT16-8	4756841.4	416985.5	275.7	19.0	256.7	-
BH16-8	4756841.0	416986.0	275.7	3.2*	272.5	1
BH16-9	4756672.4	415986.7	270.7	7.0	263.7	6
BH16-10	4756745.5	416490.7	272.4	7.0	265.4	7
BH16-11	4756938.7	417464.3	278.3	7.0	271.3	8
BH16-12	4757013.0	417899.1	280.8	7.0	273.8	7

Notes: \*Boreholes 16-6 and 16-8 were advanced in close proximity to CPT holes to allow for shear vane and/or collection of Shelby Tube samples for UCS testing to determine shear strengths for correlation with the CPT results.

### 3.4 LABORATORY TESTING

All samples were taken to Stantec's Ottawa and Kitchener laboratories where they were subjected to a detailed visual examination. The geotechnical laboratory testing program completed on the borehole samples is summarized in Table 3.2.

**Table 3.2: Geotechnical Laboratory Testing Program**

Test Description	Number of Tests	Remarks
Moisture Content	84	by Stantec
Atterberg Limits	19	by Stantec
Grain Size Distribution (sieve & hydrometer)	22	by Stantec
Consolidation (oedometer)	4	by Stantec
Unconfined Compression (soil)	14	by Stantec
Specific Gravity	3	by Stantec

Two soil samples were tested for pH, soluble sulphate content, chloride content, and resistivity.

Samples remaining after testing will be placed in storage for a period of one year after issuance of the final report. After the storage period, the samples will be discarded unless we are directed otherwise by MTO.

## 4.0 SUBSURFACE CONDITIONS

### 4.1 OVERVIEW

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are displayed on the Record of Borehole sheets and CPT records contained in Appendix B. An explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix B. The results of geotechnical laboratory testing are also presented on Figures C1 to C22 contained in Appendix C.

Borehole location plans are provided on Drawing Nos. 1A through 1D in Appendix A. A stratigraphic section of the soils encountered within the boreholes along the proposed bridge are shown on Drawing 1B. Stratigraphic sections along the proposed reinforced earth slope to the southeast of the bridge and at the Crinklaw Drain culvert site are displayed on Drawings 1C and 1D, respectively.

The stratigraphic boundaries on the borehole records and strata plots are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact boundaries between geological units. The subsoil conditions will vary between and beyond the borehole and CPT locations.

An overview of the subsurface conditions encountered in the boreholes is provided in the following sections. The subsurface conditions are presented separately for the new interchange site, the culvert replacement site and the overhead sign locations.

### 4.2 INTERCHANGE SITE

#### 4.2.1 General

The subsurface conditions observed in the five boreholes (BH16-1, BH16-2, BH16-5, BH16-6 and BH16-8) and three CPTS at the interchange site are presented in detail on the Borehole and CPT Records provided in Appendix B.

#### Previous Investigation

The geotechnical investigation results for the boreholes from previous foundation investigations at the site are also included in Appendix B. Based on these reports, the subsurface conditions at the site generally consist of the following distinct layers:

- Surficial fill or topsoil materials.
- An upper glacial till consisting of heterogeneous mixture of clayey silt to silty clay, some sand and trace of gravel (generally with a stiff to hard consistency).
- A deposit of fine sand with trace silt and gravel (generally compact to very dense).
- An intermediate glacial till unit consisting of heterogeneous mixture of clayey silt, some sand and traces of gravel (generally with a hard consistency)

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- Granular soils typically consisting of sand progressively transitioning into very stiff (clayey silt to silty clay) or dense sandy silt till deposits.

Groundwater measured in piezometers/monitoring wells was typically encountered at elevations between 259 and 263 m. Bedrock was not encountered in the previous boreholes advanced at the site which extended to depths of up to approximately 40.7 m below ground surface.

### Current Investigation

Based on the current investigation, the subsurface stratigraphy generally consisted of near-surface topsoil, asphalt and fill materials, overlying a clayey silt till, followed by a layer of silt, a layer of sand and then an intermediate clayey silt till deposit. Deposits of sandy silt and a lower clayey silt till were encountered in Borehole 16-2. Further details of the subsurface conditions encountered at the interchange site are given below.

### 4.2.2 Overburden

#### 4.2.2.1 Pavement and Roadway Fill

Asphalt pavement was encountered in BH16-8. The observed asphalt thickness was approximately 275 mm.

Roadway fill material was encountered below the asphalt in Borehole BH16-8. The thickness of the fill material was 0.9 m, extending to a base elevation of 274.5 m.

The fill material was composed predominantly of sand and gravel.

#### 4.2.2.2 Topsoil

Topsoil was encountered in Boreholes BH16-1, BH16-2 and BH16-5. The observed thickness of topsoil was approximately 150, 50 and 610 mm at these locations, respectively. The topsoil was typically comprised of clayey silt/silty clay varying amounts of sand, gravel, organic matter and rootlets. A sample of the topsoil from BH16-5 had a moisture content of approximately 25%.

#### 4.2.2.3 FILL

Fill material was encountered below the topsoil in Boreholes BH16-1 and BH16-2. The thickness of the fill material ranged from approximately 0.9 m (BH16-1) to 9.1 m (BH16-2) with the base of the fill encountered at elevations of 264.7 m (BH16-1) to 272.2 m (BH16-2).

Fill material was inferred in Borehole BH16-6 from ground surface to termination depth of the borehole at an approximate elevation of 278.8.

The fill material was predominantly composed of clayey silt to silty clay with some sand and trace gravel. Standard Penetration Test (SPT) 'N' values measured within the fill material ranged between 5 and 17 blows per 0.3 m of penetration. Undrained shear strength measurements were also made using a field vane and laboratory unconfined compression tests. The test results

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indicated that the undrained shear strength of the fill varies from 126 to 235 kPa indicative of a very stiff to hard consistency.

Index tests carried out on representative samples of the fill material yielded the following results:

Gravel:	2 to 6%
Sand:	14 to 19%
Silt:	49 to 57%
Clay:	22 to 31%
Moisture Content:	10 to 20%

Atterberg limit tests carried out on two representative samples of the fill materials measured Liquid Limits of 27 to 34 percent, Plastic Limits of 17 percent and corresponding plasticity indices of 10 to 17. The Unified Soil Classification System (USCS) group symbol for the fill material is CL (silty clay of low plasticity).

The results of grain size distribution testing and the corresponding plasticity charts for samples of the clayey fill materials are displayed on Figures C1 and C2 of Appendix C, respectively.

### 4.2.2.4 Upper Clayey Silt Till

A clayey silt till layer was encountered beneath the fill in Boreholes BH16-1, BH16-2 and BH16-8 and beneath the topsoil in BH16-5. This deposit was typically comprised of clayey silt with trace to some sand and trace gravel. Cobbles or boulders were noted in the upper till in BH16-2 while seams/interlayers of silty sand to silt were noted in BH16-1 as well as the CPTs.

This layer varied from about 8 to 9 m in thickness and extended to elevations of between about 264.6 to 264.7 m in BH16-2 and BH16-5, respectively.

The upper clayey silt till was also encountered in CPTs 16-06 to 16-08 where the upper clayey silt till was encountered to depths of about 9.8 m (below the Highway 401 grade in CPT16-08) to 16.5 (below the VMP road grade in CPT16-06), with corresponding bottom elevations of 266.0 m and 265.5 m.

Boreholes BH16-1 and BH16-8 were terminated within this layer upon reaching the planned depth of investigation. The drilled depths in these boreholes were 3.2 to 9.8 m from the existing ground surface with the boreholes terminating at elevations ranging from 272.5 m to 264.7 m.

SPT 'N' values measured within this deposit ranged from 14 to 28 blows per 0.3 m. Undrained shear strength measurements were also made using a field vane and laboratory unconfined compression test. Field vane tests attempted in the upper till in Boreholes 16-1 and 16-8 could not be turned suggesting undrained shear strengths in excess of 267 kPa. Unconfined compressive strength tests conducted on samples of the clayey silt till indicated undrained shear strengths ranging from 67 kPa to 308 kPa but generally greater than 100 kPa. Based on the above, the clayey silt till generally has a very stiff to hard consistency.



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Index tests carried out on representative samples from the upper clayey silt till layer yielded the following results:

Gravel: 0 to 5%  
Sand: 5 to 16%  
Silt: 56 to 80%  
Clay: 15 to 25%  
Moisture Content: 10 to 23%

Atterberg limit tests carried out on two representative samples from this layer indicated a plasticity index of 13 in BH16-2, the sample from BH16-1 was non-plastic. The USCS group symbol for this layer is clayey silt (CL).

The results of grain size distribution testing and the corresponding plasticity charts for samples of the upper clayey silt till layer are displayed on Figures C3 and C4 of Appendix C, respectively.

The results of consolidation tests carried out on three samples of the upper clayey silt till from Boreholes 16-1, 16-2 and 16-5 are provided on Figures C5 to C7 in Appendix C. The consolidation and index property test results for these samples are summarized in the following table.

**Table 4.1: Consolidation Test Results (Upper Clayey Silt Till)**

Sample ID	Sample Elevation	Moisture Content	Initial Void Ratio/Initial Unit Weight	Estimated Preconsolidation Stress, $P_c^1$	Recompression Index, $C_r$	Compression Index, $C_c$
BH16-1 Sa 6	269.5 m	20%	0.6/20.7 kN/m <sup>3</sup>	265 kPa	0.02	0.13
BH16-2 Sa 7	272 m	14%	0.4/22.6 kN/m <sup>3</sup>	680 kPa	0.015	0.11
BH16-5 Sa 3	272.0 m	14%	0.6/19.2 kN/m <sup>3</sup>	130 kPa	0.04	0.18

### 4.2.2.5 Upper Silt Deposit

A silt layer containing trace to some clay and trace sand was encountered beneath the upper clayey silt till in Boreholes BH16-2 and BH16-5 at depths of 16.8 m and 9.1 m, respectively. In addition, materials identified as silt/sandy silt/silty sand were encountered beneath the upper clayey silt till in CPTs 16-06 to 16-08 at depths ranging from about 9.5 m (from Highway 401 level) to 16.5 m below ground surface (from VMP road grade). The CPT records identified the presence of a silty clay layer near the base of this deposit.

The thickness of this layer was 1.5 m in BH16-2 and it extended to an elevation 263.1 m. The silt/sandy silt/silty sand strata identified in the CPTs were approximately 2 m to 2.5 m thick and extended to elevations ranging 262.3 m to 264.1 m.

Borehole BH16-5 was terminated within this layer at a depth of 9.8 m below existing ground surface corresponding to an elevation of 264.1 m.

A SPT 'N' value of 54 blows per 0.3 m was measured within the silt deposit in Borehole 16-5 indicating the silt deposit at that location was very dense. An unconfined compression test



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carried out on a sample of silt from Borehole 16-2 measured a compressive strength of approximately 170 kPa (corresponding to an undrained shear strength of 85 kPa).

Index tests carried out on a representative sample of the upper silt layer from Borehole 16-5 yielded the following results:

Gravel: 0 %  
Sand: 0%  
Silt: 96%  
Clay: 4%  
Moisture Contents: 16 to 20%

Atterberg limit tests carried out on one representative sample of this layer from Borehole 16-5 indicated the sample was non-plastic. The USCS group symbol for this layer is silt (ML).

A representative grain size distribution plot for this layer is presented on Figure C8 of Appendix C.

The results of a consolidation tests carried out on one sample of the upper silt from Borehole 16-2 are provided on Figure C9 in Appendix C. The consolidation and index property test results for this sample are summarized in the following table.

**Table 4.2: Consolidation Test Results (Upper Silt)**

Sample ID	Sample Elevation	Moisture Content	Initial Void Ratio/Initial Unit Weight	Estimated Preconsolidation Stress, $P_c^1$	Recompression Index, $C_r$	Compression Index, $C_c$
BH16-2 Sa 17	264.3 m	16%	0.5/21.4 kN/m <sup>3</sup>	700 kPa	0.01	0.07

#### 4.2.2.6 Upper Sand

A deposit of sand containing trace silt was encountered beneath the upper silt layer in Borehole BH16-2 as well as CPTs 16-06 to 16-08. The thickness of the sand layer varied from approximately 4.2 m (CPT16-06) to 5.8 m (CPT16-07). The top of the sand layer was encountered at elevations of approximately 263 m to 264 m. The base of the sand layer was encountered at elevations of approximately 258 m to 259 m.

SPT 'N' values measured within this deposit ranged from 20 to 39 blows per 0.3 m of penetration suggesting the sand deposit is compact to dense. Cone penetration test tip resistances (qt) measured within the sand typically ranged between 20 and 40 MPa.

Index tests carried out on a representative sample of the sand yielded the following results:

Gravel: 0 %  
Sand: 94%  
Fines: 6%  
Moisture Content: 7 to 20%

The USCS group symbol for this layer is sand (SP).

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A grain size distribution plot for a representative sample of this layer is displayed on Figure C10 in Appendix C.

#### **4.2.2.7 Middle Clayey Silt Till**

A till deposit comprised of clayey silt with trace sand and gravel was encountered beneath the sand in Borehole BH16-2. This deposit was also encountered at the locations of CPTs 16-06 to 16-08. Although not encountered within the middle till deposit at the borehole locations, the till deposits of Southern Ontario contain cobbles and boulders and these materials should be expected within the till deposits at this site.

The clayey silt till layer was 8.8 m thick at Borehole 16-2 with the base of the deposit encountered at an elevation of 249.4 m. CPTs 16-06 to 16-08 encountered effective refusal to further penetration within the middle till at depths of 19 to 30.6 m below ground surface.

SPT 'N' values measured within this layer ranged from 61 to 75 blows per 0.3 m suggesting the till has a hard consistency.

Index tests carried out on a representative sample from the middle clayey silt till layer yielded the following results:

Gravel:	2%
Sand:	18%
Silt:	57%
Clay:	23%
Moisture Content:	12 to 19%

An Atterberg limit test carried out on a representative sample from this layer measured a Liquid Limit of 20 percent, a Plastic Limit of 12 percent and a corresponding plasticity index of 8. The USCS group symbol for this layer is clayey silt (CL).

A representative grain size distribution plot and the corresponding plasticity chart for a sample of the middle clayey silt till layer are displayed on Figures C11 and C12 in Appendix C, respectively.

#### **4.2.2.8 Sandy Silt**

A deposit of sandy silt was encountered beneath the middle clayey silt till at a depth of 32 m in Borehole BH16-2. This layer was 3.1 m thick and extended to an elevation of 246.3 m.

A SPT 'N' value of 8 blows per 0.3 m of penetration was measured within this layer suggesting this layer is loose; however, the SPT 'N' value may have been affected by drilling disturbance.

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Index tests carried out on one representative sample from the sandy silt lower layer yielded the following results:

Gravel:	0%
Sand:	28%
Silt:	70%
Clay:	2%
Moisture Content:	22%

The USCS group symbol for this layer is sandy silt (ML).

The results of grain size distribution testing of a sample of this deposit are displayed on Figure C13 in Appendix C.

#### **4.2.2.9 Lower Clayey Silt Till**

A lower till deposit comprised of clayey silt with trace sand and gravel was encountered beneath the sandy silt at a depth of 35.1 m in Borehole BH16-2.

Index tests carried out on a representative sample from the lower clayey silt till layer yielded the following results:

Gravel:	1%
Sand:	8%
Silt:	74%
Clay:	17%
Moisture Content:	15%

The results of grain size distribution testing of a sample of this deposit are displayed on Figure C14 in Appendix C.

BH16-2 was terminated within the lower clayey silt till layer upon reaching the planned termination depth of investigation. The drilled depth in this borehole was 35.7 m below the existing ground surface corresponding to an elevation of 245.7 m.

#### **4.2.3 Bedrock**

All boreholes and CPTs were terminated above the bedrock level.

#### **4.2.4 Groundwater**

Monitoring wells were installed in Borehole BH16-1 and BH16-2. The groundwater levels in the monitoring wells were measured on April 22 and May 3, 2016. Dissipation tests were also carried out in the Upper Sand deposit at the locations of CPTS 16-6 to 16-8.

The groundwater level measurements are summarized in Table 4.3 below.

**Table 4.3: Measured Groundwater Levels – Interchange Site**

Borehole No	Ground Surface Elevation (m)	Date	Measured Groundwater Level		Comments
			Depth (m)	Elevation (m)	
BH16-1	274.4	April 22/16	0.2	274.2	MW sealed in Upper Clayey Silt Till
		May 3/16	0.2	274.2	
BH16-2	281.4	April 22/16	18.9	262.5	MW sealed in Upper Sand
		May 3/16	18.9	262.5	
CPT16-6	282.0	April 5/16	20.0	262.0	Dissipation test in Upper Sand
CPT16-7	281.8	April 5/16	19.9	261.9	Dissipation test in Upper Sand
CPT16-8	275.7	April 5/16	13.6	262.1	Dissipation test in Upper Sand

Groundwater levels at the site will be subject to seasonal fluctuations and precipitation events, and should be expected to be higher during the spring season or during and following periods of heavy precipitation or snow melt.

## **4.3 CRINKLAW DRAIN CULVERT SITE**

### **4.3.1 General**

The subsurface conditions observed in the two boreholes at the Crinklaw Drain culvert site (BH16-3 and BH16-4) are presented in detail on the Borehole Records provided in Appendix B.

In general, the subsurface conditions at the culvert location consist of topsoil over clayey silt to silty clay till containing interlayers/deposits of silt.

### **4.3.2 Overburden**

#### **4.3.2.1 Topsoil**

A deposit of clayey silt topsoil was encountered at ground surface in Boreholes BH16-3 and BH16-4. The thickness of the topsoil varied from 305 to 500 mm.

#### **4.3.2.2 Clayey Silt/Silty Clay Till**

A clayey silt to silty clay till layer containing trace to some sand and gravel was encountered beneath the topsoil in both boreholes. Although not encountered in the boreholes, the till deposits of Southern Ontario contain cobbles and boulders and these materials should be expected within the till deposits at this site.

In BH16-3, this deposit was 7.3 m in thickness and extended to an elevation of 265.5 m. In BH16-4, the clayey silt/silty clay till extended to the planned depth of the borehole; a layer of silt (described in more detail in Section 4.3.2.3) was encountered within the till between depths of 2.3 m and 4.6 m in this borehole. Drilling was terminated within the clayey silt till upon reaching the planned termination depth of the investigation. BH16-4 was advanced to a depth of 9.8 m corresponding to an elevation of 263.8 m.

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SPT 'N' values measured within the clayey silt/silty clay till ranged from 12 to 38 blows per 0.3 m of penetration. Unconfined compressive strength testing carried out on a sample of the clayey silt till measured an undrained shear strength of 171 kPa. Based on the SPT 'N' values and the laboratory testing results, the clayey silt/silty clay till is considered to have a very stiff to hard consistency.

Index tests carried out on representative samples from the clayey silt to silty clay till layer yielded the following results:

Gravel:	2 to 4%
Sand:	12 to 13%
Silt:	54 to 55%
Clay:	29 to 31%
Moisture Content:	12 to 20%

Atterberg limit tests carried out on two representative samples from this deposit measured Liquid Limits of 26 percent, Plastic Limits of 13 to 14 percent and corresponding plasticity indices of 12 to 13. The USCS group symbol for this layer is clayey silt to silty clay (CL).

Representative grain size distribution plots and the corresponding plasticity charts for this layer are given in Figures C15 and C16 of Appendix C, respectively.

### 4.3.2.3 Silt to Sandy Silt

Deposits of silt to sandy silt containing trace clay were encountered beneath the clayey silt till in Borehole BH16-3 and within the silty clay/clayey silt till in BH16-4. The silt layer was 2.3 m thick in BH16-4 and extended to an elevation of 269.0 m. In Borehole BH16-3, drilling was terminated within the silt to sandy silt deposit upon reaching the planned termination depth of investigation. The drilled depth of this borehole was 9.8 m below the existing ground surface corresponding to an elevation of 263.4 m.

SPT 'N' values measured within the silt strata varied between 19 and 27 blows per 0.3 m of penetration indicating these materials are compact.

Index tests carried out on representative samples from the silt to sandy silt layer yielded the following results:

Gravel:	0 %
Sand:	0 to 6%
Silt:	89 to 91%
Clay:	3 to 11%
Moisture Content:	19 to 20%

Atterberg limit tests carried out on representative samples from this layer indicated the samples were non-plastic. The USCS group symbol for this layer is silt to sandy silt (ML).

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The results of grain size distribution testing carried out samples of the silt to sandy silt are shown on Figure C17 of Appendix C.

### 4.3.3 Bedrock

Boreholes were terminated above the bedrock level.

### 4.3.4 Groundwater

A groundwater monitoring well was installed in BH16-4. The groundwater in the monitoring well was measured on April 22 and May 3, 2016.

The depth to groundwater in Borehole BH16-3 was also estimated at the time of drilling on April 11, 2016. This groundwater level is not considered to be a stabilized measurement, and hence will be referred to as "inferred".

The measured and inferred (i.e., at the time of drilling) groundwater levels are summarized in Table 4.4 below.

**Table 4.4: Measured and Inferred Groundwater Levels – Crinklaw Drain Culvert Site**

Borehole No	Ground Surface Elevation (m)	Date	Measured Groundwater Level		Comments
			Depth (m)	Elevation (m)	
Measured					
BH16-4	273.5	April 22/16	3.3	270.2	MW sealed in Silt Deposit
		May 3/16	3.5	270.0	
Inferred					
BH16-3	265.1	April 11/16	8.0	265.1	Water level observed in open borehole at time of drilling.

Groundwater levels at the site will be subject to fluctuations due to seasonal changes, precipitation events and the water level in the adjacent drain. The water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation or snow melt.

## 4.4 OVERHEAD SIGN SITES

### 4.4.1 General

The subsurface conditions observed in the four boreholes at the overhead sign locations (BH16-9 to BH16-12) are presented in detail on the Borehole Records provided in Appendix B.

In general, the subsurface conditions at the sign locations included asphalt over roadway fill followed by clayey silt till or clayey silt to sandy silt.

## **4.4.2 Overburden**

### **4.4.2.1 Asphalt**

Asphalt pavement was encountered in Boreholes BH16-9, BH16-10 and BH16-12. The observed asphalt thicknesses at these boreholes were approximately 125 mm, 115 mm and 115 mm, respectively.

### **4.4.2.2 Roadway Fill**

Roadway fill material was encountered below the asphalt in Boreholes BH16-9, BH16-10 and BH16-12, and at ground surface in BH16-11. The thickness of the fill material ranged approximately between 0.6 m and 2.2 m, extending to elevations of 280.1 to 268.5 m.

The fill material was composed predominantly of sand and gravel with some silt to silty sand and gravel. Borehole BH16-9 contained a layer of silty clay fill with some sand and trace gravel below the sand and gravel fill.

SPT 'N' values measured in the granular fill ranged between 22 and 55 blows per 0.3 m of penetration indicating these materials are compact to very dense. SPT 'N' values measured in the clayey fill ranged between 6 and 11 blows per 0.3 m of penetration suggesting these materials are firm to stiff.

Index tests carried out on representative samples of the fill material yielded the following results:

Gravel:	21%
Sand:	42%
Fines (silt & clay):	37%
Moisture Content:	4 to 16%

The results of grain size distribution testing carried out on one sample of the fill material is shown on Figure C18 in Appendix C. The USCS group symbol for the granular fill material is silty sand and gravel (SM).

### **4.4.2.3 Clayey Silt Till**

A clayey silt till layer containing trace to some sand and gravel was encountered beneath the fill in Boreholes BH16-10, BH16-11 and BH16-12.

Drilling in these boreholes was terminated within the clayey silt till upon reaching the planned termination depth of the investigation. Each of these boreholes were advanced to depths of 7.0 m below ground surface corresponding to elevations of 265.4 m to 273.8 m.

SPT 'N' values measured within the clayey silt till ranged from 3 to 51 blows per 0.3 m. Measurements of the undrained shear strength of the till were made by conducting in situ field vane testing and laboratory unconfined compression tests. The test results indicated the

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undrained shear strength of the till varies from about 128 to 240 kPa, suggesting a very stiff to hard consistency.

Index tests carried out on representative samples from the clayey silt till layer yielded the following results:

Gravel:	1 to 2%
Sand:	14 to 16%
Silt:	54 to 56%
Clay:	28 to 30%
Moisture Content:	11 to 17%

Atterberg limit tests carried out on four representative samples from clayey silt till measured Liquid Limits of 25 to 29 percent, Plastic Limits of 13 to 16 percent and corresponding plasticity indices of 12 to 13. The USCS group symbol for this layer is clayey silt (CL).

The results of grain size distribution and Atterberg Limit testing carried out on samples of the clayey silt till are given in Figures C19 and C20 in Appendix C, respectively.

#### **4.4.2.4 Clayey Silt to Sandy Silt**

A deposit ranging in composition from clayey silt containing trace sand to sandy silt was encountered beneath the fill in Borehole BH16-9. Drilling was terminated within this layer upon reaching the planned termination depth of the investigation. The drilled depth was 7.0 m corresponding to an elevation of approximately 263.8 m.

The SPT 'N' values measured within this layer ranged from 6 to 26 blows per 0.3 m with lowest SPT N-value measured within a cohesive portion of the deposit.

Index tests carried out on representative samples from the clayey silt to sandy silt layer yielded the following results:

Gravel:	0%
Sand:	5 to 20%
Silt:	69 to 79%
Clay:	11 to 16%
Moisture Content:	16 to 21%

Atterberg limit tests carried out on two representative samples from this layer indicated a Liquid Limit of 24 percent, a Plastic Limit of 17 percent and corresponding plasticity index of 7 in one sample while the other sample was non-plastic. The USCS group symbol for this layer is clayey silt (CL) to sandy silt (ML).

Representative grain size distribution plot and the corresponding plasticity charts for this layer are given in Figures C21 and C22 of Appendix C, respectively.



#### 4.4.3 Bedrock

Borehole advancement was terminated above the bedrock level.

#### 4.4.4 Groundwater

The groundwater conditions observed in the boreholes at the time of drilling on April 4<sup>th</sup> to 5<sup>th</sup>, 2016, are summarized in Table 4.5 below.

**Table 4.5: Measured and Inferred Groundwater Levels – Overhead Sign Locations**

Borehole No	Ground Surface Elevation (m)	Groundwater		Comments
		Depth (m)	Elevation (m)	
Inferred				
BH16-9	270.8	2.7	268.1	Water levels observed in open boreholes at time of drilling.
BH16-10	272.4	Dry	-	
BH16-11	278.4	Dry	-	
BH16-12	280.8	5.8	275.0	

The measurements in the table above are not considered to represent stabilized water levels. Based on visual examination and the measured water contents of the collected samples, the groundwater levels at these sites are expected to be in the range of 2 m below ground surface.

### 4.5 CHEMICAL TESTING

One representative sample of the native soils retrieved from each of the new underpass and culvert structure sites were tested for pH, sulphate and chloride concentrations, and resistivity. The analysis results are provided in Table 4.6.

**Table 4.6: Results of Chemical Analysis**

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
Underpass Site						
BH16-8	SS-1	2.1 to 2.7	8.2	699	29	7.3
Crinklaw Drain Culvert Site						
BH16-4	SS-4	2.3 to 2.7	8.5	9	254	30

## **5.0 MISCELLANEOUS**

The field work was carried out under the supervision of Rick Cluthe, Senior Engineering Technologist, under the direction of Kevin Nelson, P.Eng.

The private and public utility locates for the boreholes for the detailed design were carried out by Stantec personnel.

The truck and track-mounted drilling equipment was supplied and operated by London Soil Testing of London, Ontario. The CPT rig was supplied and operated by ConeTec (Geotechnical, Mining and Environmental Site Investigation Contractors).

Location and elevation survey of the Boreholes was carried out by Stantec's Geomatics Group located in Waterloo, Ontario.

Traffic control service was provided by On Track Safety Ltd. of Thornhill, Ontario.

Geotechnical laboratory testing was carried out at Stantec's Ottawa and Kitchener laboratories. Chemical testing for pH, soluble sulphate, and chloride content, and resistivity was carried out by Agat Laboratories of Mississauga and Paracel Laboratories of Ottawa.

This report was prepared by Katurah Firdawsi, P.Eng. and reviewed by Kevin Nelson, P.Eng. and Raymond Haché, P.Eng, Designated Principal MTO Foundation Contact.

## 6.0 CLOSURE

A subsurface investigation is a limited sampling of a site. The subsurface conditions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information.

Respectfully Submitted;

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For  
DB Contract 2015-3002  
G.W.P. 3033-11-00

Highway 401 & Veterans Memorial Parkway Interchange Reconfiguration  
City of London

## **7.0 DISCUSSIONS AND ENGINEERING RECOMMENDATIONS**

### **7.1 PROJECT DESCRIPTION AND BACKGROUND**

#### Project Purpose/Description

This project involves the reconfiguration of the existing Highway 401 and Veterans Memorial Parkway (VMP) Interchange and the extension of the VMP to Wilton Grove Road located approximately 800 m south of Highway 401. As part of the proposed reconfiguration, the existing four-span bridge carrying the VMP over Highway 401 will be removed and replaced with a new two span structure. The improvements will also include the following components: revised access ramps including high fill embankments with sections of geogrid-reinforced slopes, a new structural culvert for the realignment of the Crinklaw Drain beneath the VMP on the south side of Highway 401 and four new overhead signs.

#### Proposed Overpass Structure and Replacement Culvert

A new VMP underpass structure is proposed to be constructed along approximately the same alignment as the existing bridge as part of the revised interchange configuration. The available General Arrangement (GA) drawing for the new structure indicates that the proposed underpass will consist of a 27.9 m wide, two-span structure with an approximate length of 75 m. Consideration is being given to using integral abutments and supporting the central pier on caisson/drilled pier foundations in order to minimize the depth of the excavation, and the duration that the excavation is required to remain open, within the Highway 401 median area.

The proposed Crinklaw Drain culvert is understood to consist of an approximately 39 m long, precast, rigid-frame box culvert. Wingwalls comprised of RSS retaining wall systems are planned to be constructed at both the inlet and outlet of the culvert. The culvert wingwalls are understood to be classified as low performance and low appearance structures. The embankments above the culvert and wing walls will be constructed with side slopes of 3H:1V.

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Key approximate elevations associated with the proposed new underpass and replacement culvert are as follows:

Interchange Site

Proposed underside of pile cap elevation (north abutment):	278.1 m
Proposed underside of pile cap elevation (south abutment):	277.8 m
Proposed VMP final grade at centre pier:	283.7 m
Proposed Hwy 401 final grade (median):	±275.6 m

Culvert Site

Proposed invert of Crinklaw Drain culvert (box culvert) (inlet):	271.7 m
Proposed invert of Courtney Drain culvert (box culvert) (outlet):	271.6 m
Approximate streambed (inlet):	271.9 m
Approximate streambed (outlet):	271.8 m
Proposed grade at VMP centerline above culvert:	278.6 m
Water Level Elevation:	269.9 m (May 3, 2016)

## 7.2 GEOTECHNICAL DESIGN PARAMETERS

### 7.2.1 Underpass/Interchange Site

The soil conditions encountered in the area of the interchange, including at the site of the new underpass structure, generally consist of fill materials associated with the existing pavement structure and approach embankments underlain by a very stiff to hard clayey silt till that extends to depths in the range of 10 m to 13 m below the existing travelled surface of Highway 401. The upper clayey silt till is in turn underlain by a layered sequence of granular deposits (varying from silt to sand) and clayey silt till deposits that extend to depths in excess of 40 m below the VMP road grade.

For design purposes, the soil profile identified in Table 7.1 below and on Figures D1 to D2 in Appendix D can be used for the overpass/interchange site. This soil profile was developed using the subsurface information obtained from Boreholes 16-1, 16-2, 16-5, 16-6 and 16-8 and CPTs 16-6 to 16-8 from the current investigation, as well as borehole information available from Report Nos. 40I14-153, 40I14-147 and 40I14-107 in the MTO Geocres library. The geotechnical parameters identified in the soil profile were developed based on the synthesis of the CPT data, the shear strengths measured by in-situ shear vane testing and laboratory unconfined compression tests, measured SPT 'N' values and laboratory index test results (including moisture contents) of soil samples retrieved from the site.

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**Table 7.1: Representative Soil Profile at VMP and Highway 401 Interchange Site**

Elevation (m)		Soil Type	Design Parameters			
From	To		$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (°)	$S_u$ (kPa)	E (MPa)
Varies	274	Embankment Fill	See Figures D1 & D2 in Appendix D			
274.0	265	Upper Clayey Silt Till (Very Stiff to Hard)				
265	263.5	Silt / Sandy Silt (Compact to Dense)				
263.5	258.5	Upper Sand (Dense)				
258.5	249.5	Middle Clayey Silt Till (Hard)				
249.5	246.3	Sandy Silt (Loose to Dense)				
< 246.3		Lower Clayey Silt Till (Very Stiff to Hard)				

- Note: (1)  $\gamma$  = total unit weight,  $\phi$  = soil friction angle,  $S_u$  = undrained shear strength, and E = soil modulus.  
 (2) The groundwater level within the upper clayey silt till is perched and assumed to be near ground surface at elevation 274 m  
 (3) Groundwater in the Upper Sand and strata below this deposit may be assumed to be at an approximate elevation of 262.5 m for design purposes. Submerged unit weights ( $\gamma'$ ) should be used below this groundwater level  
 (4) Cobbles, boulders and coarse gravel particles were present in the upper clayey silt till layer at elevations of about 266 to 267 m in Borehole 16-2. These materials should be expected within all till deposits at the site.

## 7.2.2 Crinklaw Drain Culvert Site

The soil profile indicated in Table 7.2 below can be used for the design of the culvert and associated wingwalls. The measured subsurface information from Boreholes 16-3 and 16-4, and associated laboratory testing, was used to develop the design soil profile.

**Table 7.2: Representative Soil Profile at Crinklaw Drain Culvert**

Elevation (m)		Side of Culvert	Soil Type	Design Parameters			
From	To			$\gamma$ (kN/m³)	$\phi'$ (°)	S <sub>u</sub> (kPa)	E (MPa)
272.8	265.5	West Side	Silty clay to clayey silt (Till) (Very stiff to hard)	20.5	33	170	50
273	271.2	East Side					
And below 269							
Below 265.5		West Side	Silt to sandy silt (Compact to dense)	20.5	31	--	20
271.2	269	East Side					

## 7.3 FROST PENETRATION

In accordance with OPSD 3090.101, the design frost penetration depth for foundations,  $f$ , at the site is 1.2 m. Therefore, footings and pile caps should be provided with a minimum of 1.2 m of soil cover or equivalent insulation for protection against frost heaving.

This depth of frost penetration should also be considered in the design of frost tapers adjacent to the culvert and bridge abutment backfill zones.

## 7.4 SEISMIC DESIGN CONSIDERATIONS

Based on the subsurface conditions at the site, the Site Coefficient,  $S$ , for this site, in accordance with Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC, 2006), may be taken as 1.2 consistent with Soil Profile Type II.

The site is located in London, Ontario, and according to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio,  $A$ , applicable to this site is 0.0. Liquefaction of the foundation soils is not a concern for this site due to the negligible zonal acceleration ratio.

## 7.5 FOUNDATION OPTIONS

The use of both shallow and deep foundation options was evaluated for the proposed underpass structure. Shallow foundations would require removal of all existing fill materials in the foundation areas with the footings founded either within the very stiff to hard clayey silt till or 'perched' on granular fill pads at the abutment locations. Drilled piers/caisson foundations would extend into the upper sand deposit encountered at a depth of about 12 to 13 m below Highway 401 road grades. Driven pile foundations would develop the majority of their loads from shaft friction and would need to be driven into or beyond the middle clayey silt till depending on required loads.

Table 7.3 compares various foundation options for the central pier and abutments of the proposed underpass from foundation design and constructability perspectives:

**Table 7.3: Comparison of Foundation Options for Underpass Structure**

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Drilled Piers (Caissons)	<ul style="list-style-type: none"> <li>Large diameter caissons founded in Upper Sand deposit can resist very large axial and lateral loads</li> <li>Use of caissons at the central pier would reduce excavation and temporary support requirements</li> </ul>	<ul style="list-style-type: none"> <li>Not suitable for installation through an RSS wall</li> <li>Not compatible with integral abutments</li> <li>Caissons at central pier would encounter existing bridge footing.</li> </ul>	High	<ul style="list-style-type: none"> <li>Existing footing at central pier would pose obstruction to caissons. Coring through existing footing required for caisson installation.</li> <li>Installation of drilled piers through saturated granular soils could result in soil disturbance and collapse of sidewalls. Liners and/or drilling mud required to mitigate groundwater issues.</li> </ul>
Frictional H-Piles	<ul style="list-style-type: none"> <li>Reduced pile length compared to piles driven to rock</li> <li>Decreased settlement</li> </ul>	<ul style="list-style-type: none"> <li>Long piles required for relatively small capacities</li> <li>Structural capacity of</li> </ul>	Medium	<ul style="list-style-type: none"> <li>Potential for loss of capacity if splicing required due to pile-soil setup which occurs</li> </ul>

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Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
	<p>compared to shallow foundations</p> <ul style="list-style-type: none"> <li>Suitable for integral abutments</li> </ul>	<p>piles may not be fully utilized due to insufficient geotechnical resistance.</p> <ul style="list-style-type: none"> <li>Need for setup time after pile driving to develop full capacity</li> </ul>		<p>during stoppages in pile driving.</p> <ul style="list-style-type: none"> <li>Pile lengths may need to be modified during construction due to the wide range of possible adhesion values in hard clayey soils.</li> </ul>
Frictional Pipe Piles	<ul style="list-style-type: none"> <li>Reduced pile length compared to piles to rock</li> <li>Decreased settlement compared to shallow foundations</li> <li>Not subject to soil plugging problems allowing for piles to be spliced on site.</li> </ul>	<ul style="list-style-type: none"> <li>Structural capacity may not be fully utilized</li> <li>Not typically used for integral abutments in Ontario.</li> <li>Need for setup time after pile driving to develop full capacity</li> </ul>	Medium	<ul style="list-style-type: none"> <li>Long piles required for relatively small capacities</li> <li>Increased potential for encountering refusal on cobbles and boulders in till in comparison to H-piles</li> </ul>
Shallow foundations founded within upper clayey silt till	<ul style="list-style-type: none"> <li>Generally suitable for semi-integral abutment bridges</li> </ul>	<ul style="list-style-type: none"> <li>Larger excavations needed to found below existing fill materials.</li> <li>Larger foundation areas required compared to integral abutments or drilled piers</li> <li>Larger/deeper excavation required at central pier (to remove existing footing) requiring increased temporary protection</li> </ul>	Low to medium	<ul style="list-style-type: none"> <li>Excessive settlement under large loads</li> <li>Increased potential for differential settlement</li> </ul>
End bearing piles on bedrock	<ul style="list-style-type: none"> <li>High geotechnical resistances</li> <li>Negligible settlement</li> </ul>	<ul style="list-style-type: none"> <li>Excessive pile length</li> </ul>	High	<ul style="list-style-type: none"> <li>Bedrock depth is not known but is more than 42 m at this site</li> <li>Potential for damage to piles during installation</li> </ul>

Based on the above, the preferred option from a geotechnical/foundations perspective is to found the central pier on drilled caisson foundations founded within the upper sand and to support the abutments for the replacement bridge structures on driven steel H-piles that derive their load carrying capacity predominantly from shaft adhesion (i.e. friction piles). The steel H-piles would permit the use in an integral abutment configuration. Further details on the preferred foundation options are provided in the following sections.



## **7.6 DEEP FOUNDATIONS**

### **7.6.1 General**

The design recommendations presented in the following sections have been developed in accordance with the requirements and methods described in the Canadian Highway Bridge Design Code (CHBDC, 2006).

### **7.6.2 Abutment Foundations**

Pile foundations consisting of steel H-piles, deriving the majority of their capacity in shaft friction, are planned to be used to support the integral abutments of the proposed underpass. Based on the preliminary GA drawing, the underside of the pile caps at the abutments will be at approximate elevations of 277.8 m and 278.1 m for the south and north abutments, respectively. The minimum piles spacings should be greater than one pile perimeter.

The splicing of piles after initial driving has commenced is not recommended at this site. This constraint is due to pile-soil setup which occurs during stoppages required for pile splicing. It was demonstrated, in 2014, at the nearby Wonderland Road Interchange site, that soil-plugs formed between the pile flanges were dragged down once pile driving recommenced. The downward movement of the soil plug resulted in greater than 50% capacity loss within the top 20 m of the piles driven prior to splicing. Based on the above, the maximum pile lengths should be limited to length of pile which can be driven at the site without splicing on-site/during driving operations which is understood to be about 30 m.

#### **7.6.2.1 Geotechnical Axial Resistance in Compression**

The axial resistances at Ultimate Limit State (ULS) for driven steel 310x110 and 360x108 H-piles was assessed using both the American Petroleum Institute (API) design method using the program APILE developed by Ensoft (Ensoft, 2007) and the Laboratoire Central de Pont et Chaussée (LCPC) method which uses the direct cone penetration tip resistance to estimate the geotechnical resistance (the resistances estimated by this method were only made to the maximum depth of the CPTs advanced at the site). The geotechnical model outlined in Table 7.1 and on Figures D1 to D3 was used as input to these analyses. Given that friction piles are being proposed, the geotechnical resistances calculated for HP310x110 piles would also apply to HP310x94 and HP310x79 pile sizes. In this regard, the selection of the pile size should be based on the structural capacity of the pile.

Figures D3 and D4 in Appendix D provide profiles of the calculated factored geotechnical axial resistances at ULS in compression for HP310x110/HP310x79 and HP360x108 piles, respectively, estimated by both analysis methods; a resistance factor of 0.4 has been applied to the calculated ultimate capacities.

As shown on these figures, there is generally good agreement between the two design methods, however, below elevation 258 m (i.e. within the Middle Clayey Silt till unit), the value calculated from the CPT 16-06 data tends to be lower than calculated from the API method and the CPT

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16-07 data. There is typically more uncertainty when assessing the capacities of friction piles compared to piles driven to tip refusal. In particular, the adhesion developed in very stiff to hard cohesive soils, such as those present at this site, can be highly variable and is treated differently in various design methods. This may be reflected in the varying capacities estimated by the API and LCPC methods in the Middle Till unit.

Based on the results presented on Figure D3, the constraint against splicing piles to allow driving to greater depths, and the uncertainty in the adhesion values that will develop in the Middle Till, the recommended factored geotechnical resistance at ultimate limit state (ULS<sub>f</sub>) of an HP 310x79 or 310x110 pile is 1,350 kN for a pile length of 27 m (corresponding to a tip elevation of about 251 m). The geotechnical reaction at SLS of 1080 kN was estimated for this pile length. This SLS reaction was assumed to be 80% of the ULS<sub>f</sub>; the estimated geotechnical reaction at SLS for a 25 mm vertical settlement exceeds the geotechnical reaction at ULS<sub>f</sub> given above.

Consideration could also be given to the use of larger piles (e.g. HP360 pile types) to achieve higher capacities for the same length of pile; Figure D4 in Appendix D provides a ULS<sub>f</sub> versus elevation profile for HP360 piles. Further input should be provided by the foundations engineer if this pile size is considered.

### Soil Setup and Testing

The geotechnical axial resistance of driven steel H-Piles within a very stiff/hard silty clay deposit can be quite variable depending on non-measurable site conditions. Therefore, Dynamic Pile Testing (PDA) must be carried out on the piles driven to support the bridge abutment.

Piles driven in cohesive soils general gain capacity after driving has been completed and excess pore pressures have dissipated (i.e. the capacity of friction piles in clayey soils increases with time). The ULS<sub>f</sub> capacities identified above represent the 'long-term' capacities of the piles. Capacities determined by static pile testing or restriking of piles at the time of, or shortly following, driving would not be expected to equal the long-term capacities. To determine the actual, long-term pile capacities the following procedures are recommended to be carried out.

- At one abutment, two of the production piles will be driven to the targeted tip elevation while full time Pile Driving Analysis (PDA) testing is carried out to obtain the initial drive resistances.
- These 'test piles' shall remain in place for two weeks to allow for 14 days of soil set-up to occur.
- PDA testing of the piles shall be carried out on day 14.
- The result of the day-zero and day-14 results will be used to project one year capacities using the following relationship.

$$Q_t = Q_o \left( A \log \left( \frac{t}{t_o} \right) + 1 \right) \quad \text{Skov and Denver, 1988}$$

The A constant will be determined based on the zero and 14 day setup, followed by calculation of Q<sub>365</sub> which will be considered the long-term capacity of the piles.

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At the second abutment two of the production piles will be driven to the targeted tip elevation while full time PDA testing is carried out. The Skov and Denver equation and the A constant determined from the first abutment will be used to calculate the long-term  $Q_{365}$  capacity of the piles driven at the second abutment.

As discussed further in Section 8.5, the Hiley Formula as defined on Structural Drawing SS103-11, should be applied to each driven pile to provide a relative comparison between piles where PDA testing is carried out and the remaining piles. The "Hiley Formula Pile Resistance" for all piles shall be submitted to the geotechnical engineer for comparison with the PDA tested piles.

### Downdrag

The proposed underpass structure will be constructed in the same location as the existing bridge. The proposed grade raise in the vicinity of the new underpass is typically less than 1.5 m above existing site grades. In addition, the site soils consist predominantly of very stiff to hard clayey silt till with interbeds of compact to very dense granular soils. Based on these conditions, the piles are not anticipated to be subjected to significant downdrag loads.

### Relaxation of driven piles

For H-piles deriving their capacity predominantly from friction within the very stiff to hard silty clay / clayey silt till, relaxation and reduction of pile capacity is not a concern.

### Axial resistance in tension

For design against uplift, the tensile resistance provided in Table 7.4 is recommended. This value is based on a minimum pile length of 27 m (elevation of approximately 251 m or deeper).

**Table 7.4: Recommended Tensile Pile Resistance**

Pile Type	Minimum Pile Length(m)	Factored Geotechnical Resistance (Tension) at ULS <sub>r</sub> (kN)
HP310 x 110 or HP310 x 79	27	900

A resistance factor,  $\phi$ , of 0.3 has been applied to calculate the ULS<sub>r</sub> uplift resistance. The factored geotechnical resistance (tension) at ULS<sub>r</sub> provided above does not include the self weight of the pile. If shorter piles are found to be adequate, the axial capacity in tension will be correspondingly lower.

## **7.6.2.2 Geotechnical Lateral Resistance**

### ULS<sub>r</sub> and SLS Resistances

The geotechnical resistance of the pile against lateral loads is mobilized due to the passive resistance of the surrounding soil. Assessed values for horizontal passive resistance and geotechnical resistances at SLS for the proposed pile can be generated from information provided in Table C6.4 of the Commentaries to the Canadian Highway Bridge Design Code (CHBDC, 2006). A value at ULS<sub>r</sub> of 200 kN and a value at SLS of 110 kN may be used for an

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HP 310 x 79 pile in very stiff cohesive material. A horizontal displacement of 10 mm is assumed at ground surface for the SLS condition.

Group Action

Group action of piles (pile interaction) for lateral loading should be considered if centerline spacing of piles is less than 7 pile diameters/widths parallel to the direction of lateral load. Group action reductions would not apply to the leading pile in a row (load parallel to direction of lateral load) or less than 4 pile diameters/widths for loads perpendicular to the piles; we understand that these load conditions would not apply to this project.

The effect of interaction between piles can be considered by applying a reduction factor to the soil resistance (p-multiplier). The following reduction factors may be used with ULS and SLS values, as well as with p-y curves to account for pile group action:

**Table 7.5: Recommended Reduction Factors for Trailing Piles in a Row**

Ratio of pile spacing to pile diameter/width	Reduction Factor (p-multiplier)
<b>Load Parallel to Pile Spacing</b>	
7	1.0
4	0.8
3	0.7
2	0.6

### 7.6.3 Centre Pier Foundations

Concrete caisson (drilled pier) foundations are being considered as the preferred option to support the centre pier of the proposed Underpass structure. The caissons will tie into the pier columns and as such would act as partially embedded piles. No pile caps would be required at the ground surface for the centre pier which will reduce depths, and associated durations, of excavations within the Highway median.

#### 7.6.3.1 Geotechnical Axial Resistance in Compression

Figure D5 provides the anticipated geotechnical resistance at ULS (static analysis) versus depth for concrete caissons with diameters of 1.5 m to 1.8 m. It is recommended that the base of the caissons be founded near the surface of but within the Upper Sand deposit encountered at elevations of between approximately 263.5 m (CPT16-8 near east end of pier) to 262.5 m (BH13-03 near the west end of the pier). Advancement of the caissons significantly below Elevation 262.5 m is not recommended in order to limit potential difficulties associated with caisson installation below the water table within the sand and reduction in capacity due to the presence of the underlying till deposit.

Figure D5 indicates that a 1.5 m diameter caisson within the sand at an elevation 262.5 m will have a factored geotechnical resistance at ULS of 7,000 kN and a 1.8 m diameter caisson would have a factored ULS capacity of 10,000 kN.

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The geotechnical resistance at SLS was considered using the approach recommended in FHWA-NHI-10-16 Drilled Shafts Manual (2010) which suggest for a cohesionless soil a deformation equivalent to 1% of the caisson diameter would be anticipated when using 40% of the static design calculated resistance. An upperbound SLS design value of 80% of the factored ULS value is recommended by Stantec which is equivalent to 32% of the static design calculated resistance. For this load, the Drilled Shaft Manual suggests that less than 0.5% of the caisson diameter would be expected as settlement. Therefore the following geotechnical resistances at SLS are recommended for caissons founded within the sand layer discussed above.

Caisson Diameter	Geotechnical Resistance at SLS	Associated Settlement
1.5 m	5,600	< 20 mm
1.8 m	8,000	< 20 mm

### 7.6.3.2 Geotechnical Horizontal Resistance to Lateral Loads

The FHWA-NHI-10-016 Drilled Shafts Manual recommends the “p-y method” for computing the response of a drilled shaft to lateral loading.

The actual geotechnical resistance at ULS and SLS will depend on the stiffness of the caisson sections, the loading condition at the caisson head, and the actual projection of the caisson head above ground surface.

For the purpose of developing “non-linear springs” or “p-y curves” to be used for structural analysis, the following caisson geometry was considered.

- 1.2 m diameter, 5 m pier projections above ground;
- 1.8 m diameter caisson extending ~13 m below ground; and
- 24, 35 M bars for each diameter

The non-linear springs recommended as input parameters to the structural design are provided in Table D1 in Appendix D. The spring profiles are a function of the caisson width and the soil strength. If the caisson section below ground is to have a diameter other than 1.8 m, the non-linear springs will need to be recalculated to reflect the actual caisson diameter.

For preliminary design purposes, the following geotechnical resistances have been calculated for the above caisson section dimensions.

Condition	Free Head Condition	Fixed Head Condition
ULS, $\phi = 0.5$	500 kN	1000 kN
SLS, 25 mm	500 kN*	1000 kN*
SLS, 50 mm	700 kN*	1500 kN*
SLS, 80% ULS	400 kN	800 kN

Note: \*Exceed 80% of factored ULS value.

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The above geotechnical resistances at  $ULS_r$  consider the vertical axial load within the caisson to be 7000 kN corresponding to the anticipated SLS load. As well, the ULS lateral is considered to be applied at the top of the 5 m pier projection.

The maximum SLS value recommended for design purposes is 80% of the  $ULS_r$  values.

### **7.6.3.3 Caisson Installation Considerations**

The supply and installation of the caissons should be according to the OPSS 903 Construction Specification for Deep Foundations.

The new central pier is located at the same location as the existing pier which is supported on a pad foundation. Advancement of the caissons will require either coring through the existing pier foundation or removal of the foundation prior to installation. It is understood that coring through the footing is the preferred approach in order to reduce excavations within the highway median.

The CPT profiles indicate that the clayey silt deposit contains interlayers of silt/sandy silt/silty sand. The presence of wet silty/sandy zones within the upper till will necessitate the use of liners during installation of drilled piers/caissons central bridge pier foundations to minimize the potential for loss of ground into the drilled piers.

The water levels measured within the Upper Sand deposit during the current investigation varied from approximately 262 m (based on CPT dissipation tests) to 262.5 m (in monitoring wells in boreholes located north and south of the highway). Drilling of caissons into the saturated portion of the sand deposit could lead to disturbance of the foundation subgrade soils unless suitable construction procedures are adopted. As the measured water levels are at or very close to the design caisson tip elevation of 262.5 m, the following construction procedures are recommended to minimize disturbance of the subgrade and provide acceptable foundation performance:

- The water levels in the sand deposit should be determined immediately prior to caisson installation by taking measurements in the monitoring wells installed in the sand deposit.
- If water levels are above the caisson founding levels, the lower portions of the caissons will need to be advanced using drilling mud. Alternatively, dewatering measures could be implemented to lower the water level within sand deposit to below the founding level.
- The caissons must be founded within the sand deposit to achieve the design loads. In this regard, the caisson installations must be inspected by a QVE qualified in geotechnical engineering to confirm that the caissons are founded within the sand deposit and the foundation subgrade has not been disturbed as a result of caisson installation.

The perched water level within the upper clayey silt till may infill the caisson. If the caisson opening cannot be made dry, concrete placement should be carried out using tremie techniques.

## 7.7 LATERAL EARTH PRESSURES

### 7.7.1 Abutment and Culvert Backfill

Ontario Provincial Standard Drawing (OPSD) 3101.150 outlines the required extent of the granular backfill zone at the bridge abutments. The materials used as backfill behind the proposed bridge abutments and around the culvert should consist of free-draining granular fill placed and compacted using methods and equipment appropriate to the type of structure. For the purpose of this report, it is assumed that backfill materials meeting the requirements of OPSS Granular B (Type I or Type II) or Granular A materials will be used.

Excavation and backfill for the new bridge structure should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures. Backfill materials should be placed and compacted in accordance with the requirements of OPSS.PROV 206 and OPSS.PROV 501, respectively.

### 7.7.2 Static Lateral Earth Pressures

Static lateral earth pressures will need to be considered in the design of abutments, retaining walls (wingwalls) and RSS walls.

Earth pressures acting on structures should be computed in accordance with Section 6.9 of the CHBDC. For retaining walls that are designed to allow rotation, active earth pressures may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. The unfactored soil parameters provided in Table 7.7 may be used for design of the abutment and culvert walls. The surface of the backfill at locations has been assumed to be horizontal. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.6 of the CHBDC.

The total active (PA) and passive (PP) thrusts can be calculated using the following equations:

$$P_A = \frac{1}{2} K_a \gamma H^2$$

$$P_O = \frac{1}{2} K_o \gamma H^2$$

$$P_P = \frac{1}{2} K_p \gamma H^2$$

where H is the height of the wall and  $\gamma$  is the unit weight of the backfill soil. To calculate the earth pressure against the culvert, H should include the height of the culvert wall plus the depth of material overlying the culvert. Values for  $K_a$ ,  $K_p$ ,  $K_o$  and  $\gamma$  are provided in Table 7.6 for horizontal and sloping (2H:1V) backfill conditions, respectively. The thrust acts at a point one third up the height of the wall.



**Table 7.6: Recommended Design Earth Pressure Parameters – Abutments and Culvert**

Parameter	OPSS Gran B Type I		OPSS Gran A and Gran B Type II	
	Horizontal	2H:1V	Horizontal	2H:1V
Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	21		22.0	
Effective Friction Angle	32°		35°	
Coefficient of Earth Pressure at Rest ( $K_o$ )	0.47	0.68	0.43	0.62
Coefficient of Active Earth Pressure ( $K_a$ )	0.31	0.47	0.27	0.39
Coefficient of Passive Earth Pressure ( $K_p$ )	3.2	---	3.7	---

The backfill behind the reinforced zone of the RSS walls at the culvert site may consist of earth embankment fill with sloping (3H:1V) backfill present above and behind the walls. The unfactored soil parameters provided in Table 7.7 may be used for design these walls.

**Table 7.7: Recommended Design Earth Pressure Parameters – Culvert RSS Wingwalls**

Parameter	Compacted Earth Borrow Fill	
	Horizontal	3H:1V
Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	21	
Effective Friction Angle	32°	
Coefficient of Earth Pressure at Rest ( $K_o$ )	0.47	0.62
Coefficient of Active Earth Pressure ( $K_a$ )	0.31	0.39

## 7.8 EMBANKMENT DESIGN

New embankments and widening of existing embankments are required for the ramps of the modified interchange.

The subsurface investigations conducted at the site indicate the existing roadway embankments were constructed predominantly using clayey silt fill materials. The new roadway embankments are planned to be constructed using earth borrow fill except within the reinforced soil zones where granular fill materials are required.

Typically, the new and widened roadway embankments will have maximum embankment heights in the order of 7 m and embankment sideslopes of 2H:1V. However, due to property constraints, there is insufficient room to construct the embankments using the conventional sideslopes on the east side of the VMP from approximately Station 9+800 to 9+950 and on the west side of the N-W ramp from approximately Station 9+390 to Station 9+440 and Station 9+500 to 9+560. On the east side of the VMP, the maximum RSS slope inclination is 1.5H:1V. At the N-W ramp, the reinforced soil slopes have sideslopes varying between 1.125H:1V to 1.5H:1V.

An assessment of the global stability of the new roadway embankments and RSS slopes is provided in the following section of this report. The assessment of other failure mechanisms for the RSS slopes is the responsibility of the RSS slope designer.



### **7.8.1 Slope Stability Evaluation**

Slope stability evaluations were carried out at critical sections of the new roadway embankments (i.e. sections where the embankments are highest or where the steepest slopes are proposed) using the commercially available slope stability analysis software, SLOPE/W (GEO-SLOPE, 2007). The analyses included allowance for dynamic loading due to traffic by considering a static surcharge load equivalent to 0.8 m of additional fill, as per Section 6.9.5 of the CHBDC. A minimum factor of safety of 1.3 is considered acceptable against static, deep-seated embankment instability. Seismic analyses were not conducted as the ZAR is equal to 0 at this site.

The results of a slope stability analysis of a conventional (i.e. non-reinforced) embankment with a slope height of 7 m and 2H:1V sideslopes under traffic loads are presented on Figure D6 in Appendix D. The results of the stability analyses indicate that the proposed embankment with a slope angle of about 2H:1V would provide a factor of safety against instability of greater than 1.3.

Stability analyses were also completed for critical sections of the RSS slopes at the following locations:

- Station 9+550 on the N-W Ramp where an approximately 7 m slope with a reinforced sideslope of 1.125H:1V is proposed; and
- Station 9+950 on the east side of the VMP where the existing embankment will be widened using RSS and the new, widened embankment will have reinforced sideslopes constructed at inclinations of up to 1.5H:1V.

The results of the slope stability analyses at these locations under traffic loads are presented on Figures D7 and D8 in Appendix D. The results of the stability analyses indicate that the proposed reinforced embankments with slope angles of between 1.125 H:1V and 1.5H:1V would provide a factor of safety against global instability of greater than 1.3 provided that a minimum 5 m wide geogrid reinforced zone is established adjacent to the slope face.

### **7.8.2 Evaluation of Potential Ground Settlement due to Embankment Construction**

Settlement of the underlying soil due to the construction of the embankments was evaluated. The following assumptions were made in evaluating the settlement of the site soils under the proposed embankments:

- The typical soil profile given Table 7.1 was considered representative for the interchange site;
- The maximum height of the new embankments is approximately 7 m (in the area of the N-W ramp);
- The maximum height of new embankments for the full width of the VMP (i.e. approximately 35 m crest-to-crest width) is approximately 6 m near Station 9+800;
- The loads from the bridge abutments and central pier will be transferred to deep, competent strata by the piles and caissons and hence do not contribute substantially to the settlement of the site soil;

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- The measured preconsolidation pressures of the upper clayey silt till soils are typically higher than the anticipated post-construction stresses in this deposit. Further, the middle till is typically stronger than the upper till and, as such, would have higher preconsolidation pressures than the upper till. Therefore, substantial consolidation settlements of the cohesive native soils are not expected to occur and only immediate (elastic) settlement was considered in the analyses.
- Soil moduli used in the analyses were based on typical values identified in the CHBDC. The modulus of the upper till was also assessed by reviewing the load-deformation characteristics on the rebound and recompression curves from the consolidation test data ; and
- The maximum height of the embankment raising in the immediate vicinity of the bridge abutments is limited to about 1.5 m or less;

Evaluation of soil settlement due to the effects discussed above was performed using Settle3D (Rocscience, 2009) which is a three-dimensional computer program for the analysis of the immediate vertical settlement and consolidation of soil under surface loads such as embankments. Settlement evaluation was carried out for embankments constructed using earth borrow fill materials with 2H:1V slopes.

The results of the analyses analysis result indicate that the maximum total vertical settlement of the native soils beneath the approach embankments leading up to the new bridge is expected to be less than 25 mm due to the additional loading imposed by the proposed grade raise of 1.5 m in these areas. Settlements of less than 15 mm are at the proposed abutment locations. This settlement is anticipated to take place relatively rapidly and is expected to be completed during construction of the embankment.

Additional analyses were carried out to estimate the magnitude of settlement that would occur in new embankment areas with fill heights of approximately 7 m. These analysis result indicate that the maximum total vertical settlement of the native soils is approximately 110 mm. This settlement is anticipated to take place relatively rapidly and is expected to be completed during construction of the embankment.

The above settlement estimates do not include settlements associated with compression of the new fill materials used to construct or raise the embankments, which would occur during and after the construction of the embankment depending on the type of materials used. The anticipated potential 'self weight' settlement for a well compacted earth borrow is approximately 0.5% of the fill height (Goodger and Leach, 1990). For a 7 m fill height, up to 35 mm could be anticipated. Self weight settlement of general fills is assumed to occur over 10 years with more than 50% during the first year.

## 7.9 CRINKLAW DRAIN CULVERT REPLACEMENT

### 7.9.1 General

This section provides foundation recommendations for the proposed construction of a structural culvert where the southern extension of the VMP passes over the realigned Crinklaw Drain channel. The location of the proposed culvert, as displayed on Drawings 1C and 1D, is currently

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utilized as an agricultural field and has ground surface elevations varying between about 273 m and 273.5 m.

The proposed culvert is understood to consist of a precast concrete rigid frame box culvert with interior dimensions of approximately 3.96 m by 3.35 m that will have invert levels of about Elevation 271.6 m to 271.7 m. Wingwalls comprised of RSS retaining wall systems are planned to be constructed at both the inlet and outlet of the culvert. The wing walls have been classified as low appearance and low performance RSS walls. The embankments above the culvert will be constructed with side slopes of 3H:1V.

The box culvert should be founded on the native very stiff to hard clayey silt till or dense silt below any existing fill, topsoil or other soils containing organic matter. Any soft/loose or otherwise disturbed soils encountered at subgrade level should be subexcavated and replaced with structural fill consisting of compacted OPSS Granular A materials. As the site soils are highly susceptible disturbance as a result of construction activity, a 100 mm thick layer of lean concrete should be placed directly over the approved foundation subgrade to protect it from disturbance.

The bedding and leveling pad for the box culvert should be designed and constructed in accordance with Ontario Provincial Standard Specification (OPSS) 422 (*Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut*) for concrete box culverts. For the case of a rigid frame box culvert, a 200 mm layer of OPSS Granular A should be placed and compacted beneath the culvert for bedding purposes. A 75 mm layer of uncompacted OPSS Granular A should be placed between the compacted Granular A and the underside of the box for bedding purposes. The edges of the pad should extend at least 0.5 m horizontally away from the footing in all directions.

### 7.9.2 Geotechnical Vertical Resistance

The geotechnical resistances provided in Table 7.8 below may be used in the design provided the box culvert segments are placed on granular bedding materials overlying undisturbed native soils as described above.

**Table 7.8: Geotechnical Resistance for Crinklaw Drain Culvert**

Founding Element	Founding Elev. (m)	Footing Width(m)	Factored Geotechnical Resistance at ULSf (kPa)	Geotechnical Resistance at SLS (kPa)
Rigid frame box	~271.3	4.5	300	160

In accordance with Section 6.6.2 of the CHBDC, a resistance factor of 0.5 has been applied in calculating the factored geotechnical resistance at Ultimate Limit State (ULS<sub>f</sub>). The geotechnical resistance at Serviceability Limit State (SLS) corresponds to a maximum settlement of 25 mm.

The representative soil properties provided in Table 7.2 were used in the evaluation of the geotechnical resistance at ULS<sub>r</sub> and SLS given in Table 7.10,. The groundwater was conservatively assumed to be immediately beneath the proposed founding elevation.

### **7.9.3 Geotechnical Horizontal Resistance (Sliding)**

The unfactored horizontal resistance of the box culvert may be calculated using the following unfactored coefficients of friction:

- 0.55 between OPSS Granular A and precast concrete
- 0.45 between silty clay/clayey silt and precast concrete
- 0.45 between a precast concrete footings and a thin layer of uncompacted leveling sand

In accordance with Table 6.1 of the CHBDC, a resistance factor against sliding of 0.8 should be applied to obtain the resistance at ULS<sub>r</sub>.

### **7.9.4 Culvert Backfill and Lateral Earth Pressures**

The requirements for backfill and cover materials and the construction of a frost taper (backfill transition) for the box culvert are outlined in OPSS 422 and OPSD 803.010 (Backfill and Cover for Concrete Culverts). Backfill to culvert walls should consist of granular fill meeting the requirements of OPSS.PROV 1010 Granular A or Granular B Type II materials. The backfill should be placed and compacted in accordance with MTO's Special Provision SP105S21 (Amendment to OPSS 501).

A clay seal should be provided at the upstream and downstream ends of the culvert to act as a barrier to piping within the granular backfill outside of the culvert walls. Clay seal materials should be meet the requirements set out in OPSS.PROV 1205.

The culvert should be designed for the full overburden pressure and live loads using the at-rest earth pressure coefficients and soil unit weights provided in Section 7.7.2.

### **7.9.5 Scour Protection**

Scour and erosion protection measures (e.g. suitable non-woven geotextiles and/or rip-rap) should be provided at the culvert inlet and outlet if the flow velocities are sufficiently high. The requirements for, and design of, erosion protection measures for the culvert inlet and outlet should be assessed by the hydraulic design engineer.

## **7.10 RETAINED SOIL SYSTEM (RSS) WALLS**

### **7.10.1 General**

Consideration is being given to constructing the wingwalls at the inlet and outlet of the Crinklaw Drain culvert using low appearance and low performance Retained Soil System (RSS) wall systems. The RSS walls will have maximum heights of about 5.4 m and will extend outwards (i.e.

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towards the drain channel) from the edges the culvert at angles of 45 degrees from the orientation of the culvert. The proposed RSS wall lengths vary from 6 m to 9 m in length. The base elevations of the walls are planned to 'step up' away from the structure. The embankment sideslopes above the walls will be constructed with at an inclination of 3H:1V.

For the design of the RSS walls, the guidelines included in the following documents should be considered:

- RSS Design Guidelines (MTO, 2008);
- CHBDC Section 6.12 – MSE Structures (CHBDC, 2006); and
- CFEM Chapter 27 – Reinforced Soil Walls (CFEM, 2006)

Retained soil systems are listed in the MTO Designated Sources of Materials (DSM) and under Special Provisions 599S22 and 599S23.

The design of the internal stability of the RSS wall is the responsibility of the wall supplier.

### **7.10.2 Site Specific Geotechnical Considerations for RSS Walls**

A 200 mm thick Granular A Leveling Pad should be constructed beneath the facing or concrete elements of the RSS.

The factored geotechnical resistance at ULS for the Crinklaw Drain culvert RSS wingwalls is 250 kPa at the proposed foundation elevation of about 271 m to 273 m. This geotechnical resistance was evaluated based on assumed RSS dimensions (i.e., a reinforced zone having a width of 0.8H where H is the height of RSS wall (based on MTO's RSS Design Guidelines, MTO (2007)) and a length equal to the minimum length of the wingwalls (assumed to be 6 m). A wall height of 5.4 m was assumed; hence, the width of the reinforced zone behind the RSS wall was assumed to be approximately 4.5 m. The SLS resistance for 25 mm of total settlement was estimated to be 175 kPa.

The global stability of an RSS wingwall founded at an approximate elevation of about 271 m (~0.5 m below the proposed streambed elevation) was evaluated. The evaluation results are indicated in Figure D9 in Appendix D. The evaluated factor of safety against global instability is in excess of the required factor of safety of 1.5.

Unit weights, effective friction angles and lateral earth pressure coefficients provided in the preceding sections of this report may be used by the wall design in the design of the RSS wall.

## **7.11 OVERHEAD SIGN DESIGN**

### **7.11.1 General**

The interchange improvement project includes the installation of four new Overhead Sign Support Structures (designated as Sites 19-718-W-S, 19-719-W-S, 19-720-E-S, and 19-721-E-S). Table 7.9 summarizes the location and support type of each of the proposed overhead signs along

with information on the borehole advanced at the sign location as well as identifying the borehole advanced at each of the sign locations as part of the current investigation.

**Table 7.9: Overhead Sign Details**

Site Number	Proposed Sign Location Station / Highway 401 Direction	Sign Support Type	Borehole
19-718-W-S	30+210 / Eastbound	Cantilever – Ground Mounted	16-9
19-719-W-S	30+670 / Eastbound	Cantilever – Ground Mounted	16-10
19-720-E-S	31+667 / Westbound	Cantilever – Ground Mounted	16-11
19-721-E-S	32+111 / Westbound	Cantilever – Ground Mounted	16-12

### 7.11.2 Caisson Foundations for Overhead Signs

Overhead sign supports founded on caissons should be designed in accordance with the requirements in MTO's Sign Support Manual (MTO, 2015). The Sign Support Manual (SSM) includes standard foundation designs for ground-mounted (single or tri-chord) cantilever overhead signs (Section 3 of the SSM and Standard Drawing SS118-3).

The standard foundation designs provided in the SSM do not apply to sites where bedrock is at or near the surface, the footings will be located in rock fill or exceptionally soft or loose soils are present within the foundation zone. The standard sign foundations presented in the SSM for the ground-mounted, cantilever overhead sign supports have been developed for sites where the following minimum soil conditions are present within the foundation zone.

- Case 1 (Cohesionless Soils): Competent soils of uniform composition with a minimum internal friction angle of 28 degrees within the upper 2/3 of the caisson below the frost zone and 30 degrees within the lower third of the caisson below the frost zone.
- Case 2 (Cohesive Soils): Clay soil with a minimum undrained shear strength of 25 kPa within the upper 2/3 of the caisson below the frost zone and a minimum undrained shear strength of 50 kPa within the lower third of the caisson below the frost zone

A site-specific footing design is required for sites where soil conditions not meeting the minimum requirements outlined above are present.

Based on the results of the current investigation, the soils within the anticipated founding elevations of the overhead sign supports (OHSS) consist predominantly of very stiff clayey silt till with average undrained shear strengths of greater than 100 kPa that contain seams/interlayers of compact silt. Based on these conditions, the standard OHSS foundation designs are applicable to this project

### 7.11.3 Installation Considerations

Construction of the sign support foundations should be in accordance with OPSS PROV.915 (Construction Specification for Sign Support Structures).

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Seams and interlayers of water-bearing, non-cohesive soils (typically silt or sandy silt) were encountered at various locations within the clayey silt till deposits within the project study area. "Perched" groundwater may also develop within the surficial fill materials. Where wet, cohesionless soil deposits are encountered, these materials should be expected to run or flow into the holes drilled for the sign support foundations. Therefore, provision should be included for the use of temporary liners to reduce the potential for sidewall instability and ground loss during drilling and concrete placement.

Although not encountered at boreholes drilled at the sign support locations, cobbles or boulders were inferred to be encountered within the upper clayey silt till at other locations within the project study area. Appropriate construction equipment and procedures should be used to penetrate cobbles and boulders (if encountered) during the drilling of the holes for the foundations of the overhead sign supports.

### 7.12 CEMENT TYPE AND CORROSION POTENTIAL

One sample of the native soil from each of the overpass and culvert sites were submitted to Paracel Laboratories and Agat Laboratories, respectively, for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in Table 4.6.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The maximum soluble sulphate concentration for all the samples tested was 259 µg/g. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH ranged from 8.2 to 8.5 for both sites, which is within what is considered the normal range for soil pH of 5.5 to 9.0. However, the low resistivity values measured suggest a severe corrosion potential environment. The test results provided in Table 4.6 should be used by the designers in assessing the potential for corrosion of steel elements and may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.



## **8.0 CONSTRUCTION CONSIDERATIONS**

### **8.1 CONSTRUCTION STAGING AND DETOUR**

A local detour is anticipated to be required for the construction of the new underpass structure and interchange ramps as the VMP will be closed to traffic during the construction of the new bridge.

The construction of the foundations for the new central pier of the bridge is anticipated to involve staging and lane-reductions on Highway 401 using appropriate traffic control. The use of a temporary roadway protection system may be required near the centerline of existing Highway 401.

### **8.2 EXCAVATION AND BACKFILLING**

Excavation backfill for the new bridge structure should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures.

Any vegetation, fill, organic soils and other deleterious materials must be removed from beneath proposed the pile caps, retaining walls, RSS wall/slope areas, and the culvert. Where deleterious materials are encountered at foundation subgrade level, the materials should be excavated, removed and replaced with compacted granular fill materials. The lateral extent of the zone of subexcavation (and replacement) should include all deleterious material within the influence zone of the above foundation elements.

Grading work should be carried out in accordance with OPSS.PROV 206 Construction Specification for Grading and SP 206S03. Where existing embankments are to be widened, the new fill materials should be benched into the existing embankments in accordance with OPSD 208.010.

All side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations for Construction Projects (OHSA). The excavations required for the new abutments would be developed through the existing approach embankment fill and would extend to depths in the order of 4 m to 5 m below existing VMP road grade. The excavations for the central pier, the Crinklaw Drain culvert and other project components (e.g. the stormwater management pond to be constructed within the cloverleaf of the S-W ramp) are expected to encounter fill materials, the very stiff to hard clayey silt till and the compact to dense silt deposits. Where space permits, these excavations may be developed using open-cut methods. The clayey silt till would be classified as Type 2 soil while the fill materials and silt interlayers/deposits above the water table would be classified as Type 3 soils. OHSA indicates that temporary excavations made within these materials above the water table should be developed with side slopes no steeper than 1H:1V. Granular fill materials and the native silt deposits below the water table would be classified as Type 4 soil and excavations in these materials should be sloped no steeper than 3H:1V based on OSHA requirements.



## 8.3 TEMPORARY ROADWAY PROTECTION

Depending on the depth and geometry of the excavations needed to permit the construction of the central pier foundations, temporary protection systems may be required adjacent to the active highway lanes.

The contractor will ultimately be responsible to develop and implement a roadway protection system meeting the requirements of OPSS.PROV 539, including establishing appropriate geotechnical design parameters.

Based on the subsurface conditions at the site, the following table compares the available roadway protection options considered for the excavations for the central pier foundations within the Highway 401 median area:

**Table 8.1: Comparison of Roadway Protection Systems**

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
H-Piles with timber lagging; struts/rakers	<ul style="list-style-type: none"> <li>• simple installation</li> </ul>	<ul style="list-style-type: none"> <li>• not suitable beneath groundwater level without groundwater control</li> </ul>	Low	<ul style="list-style-type: none"> <li>• Potential for groundwater seepage and loss of ground unless groundwater control measures are implemented</li> </ul>
Steel sheet pile (SSP)	<ul style="list-style-type: none"> <li>• easier to install below watertable (no unwatering required during roadway protection installation)</li> </ul>	<ul style="list-style-type: none"> <li>• difficult to drive/install in very stiff to hard clayey silt till</li> </ul>	Medium	<ul style="list-style-type: none"> <li>• Possible damage to sheet pile walls during driving</li> </ul>

Both support systems are considered feasible for use at this site; however, the use of a steel sheet pile walls/enclosure presents itself as the more viable option for roadway protection as it reduces the potential for loss of ground if saturated fill or silt deposits are encountered. The sheet piling should be supported with struts or rakers from the construction side or tie-backs/ground anchors.

Roadway protection design should meet the requirements of Performance Level 1b as per OPSS.PROV 539 and should consider traffic loading. Performance Level 1b specifies a Maximum Angular Distortion of 1:1000 and a Maximum Horizontal Displacement of 10 mm. Strut, raker or tie-back spacing must be designed not to exceed these limits. Horizontal movement should be monitored throughout the culvert replacement process as described in OPSS.PROV 539. The monitoring requirements outlined in OPSS.PROV 539, including the milestone inspections to be completed by the Quality Verification Engineer, are considered to be appropriate for this project.

## **8.4 REUSE OF EXCAVATED MATERIAL**

The material near the ground surface in the vicinity of the project site typically consists of silty clay/clayey silt material. This material will not be suitable as backfill within and behind the structures for the proposed overpasses. The clayey silt materials can be re-used for embankment construction if they do not contain deleterious materials and is at a suitable water content for compaction at the time of construction.

## **8.5 PILE INSTALLATION**

It is essential that the compatibility of the pile driving equipment, the soil conditions, and the pile type being driven is properly accounted for in order to achieve the required pile penetration and a satisfactory pile foundation.

Piles shall have reinforced tips according to Ontario Provincial Standard Detail, OPSD 3000.100 Type I.

The pile driving equipment shall be appropriate to the driving conditions and capable of delivering a minimum specified energy of 500 J/cm<sup>2</sup> of steel cross section area. For a 310x110 pile, a driving energy of 70 kJ is recommended.

At least four production piles should be tested using a Pile Driving Analyser (PDA) to confirm that the ultimate pile capacities are greater than twice the design ULS<sub>r</sub> values. As indicated previously, two of the piles are recommended to be driven a minimum of 14 days prior to PDA testing to allow soil set-up to occur and to define the soil set-up model for the project.

MTO's principal pile driving control tool is the Hiley Formula as defined on Structural Drawing SS103-11. As noted on the structural drawing this approach is applicable to non-cohesive soils and to soils that provide sufficient rebound for the Hiley Formula to be effective. The soils at this site are cohesive and therefore the Hiley Formula is not the recommended method to provide final confirmation of the pile resistances. Nevertheless, Drawing SS103-11 shall be applied to each driven pile to provide a relative comparison between piles where PDA testing is carried out and the remaining piles. The "Hiley Formula Pile Resistance" for all piles shall be submitted to the geotechnical engineer for comparison with the PDA tested piles, however, it shall not be used as a pile capacity acceptance criteria.

## **8.6 TEMPORARY GROUNDWATER CONTROL**

Water levels measured in the near surface soils at the interchange site varied from depths of 0.2 m (Borehole 16-1 near N-W Ramp) to 3.3 m (Borehole 16-4 at proposed Crinklaw Drain Culvert) below existing ground surface.

Therefore, excavations in the area of the interchange site are expected to extend below the ground water level. Temporary unwatering, using conventional sump and pump techniques, is considered appropriate for excavations at the site developed predominantly within the clayey silt deposits.

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Increased groundwater inflow should be expected where excavations extend into or through thick silt/sandy silt interlayers present below the water table within the till (e.g. the silt deposit encountered near foundation level at the east end of the Crinklaw Drain culvert site). In these areas, pumping from filtered sumps established in the floor of the excavations is unlikely to effectively dewater the excavation due to the fine grained nature of the silt and alternative dewatering methods (e.g. pre-pumping from a series of sanded-in vacuum well-points or eductor wells installed into the silty subgrade soils) may be required to control groundwater inflows and limit the potential for disturbance of foundation subgrade materials.

Lowering of the water levels within the Upper Sand unit could be considered to reduce the potential for encountering groundwater, and associated installation difficulties, at the base of the caisson foundations for the central pier. The need for such dewatering activities would be dependent on the water levels at the time of construction.

Discussions relating to permit-to-take-water requirements and applications are presented in the hydrogeological report for this project.

## 9.0 SPECIFICATIONS

The following specifications are referenced in this report:

Table 9.1: Specifications Referenced in Report

Document	Title
OPSD 208.010	Benching of Earth Slopes
OPSD 803.010	Backfill and Cover for Concrete Culverts
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls – Abutment, Backfill – Minimum Granular Requirement
OPSD 803.010	Backfill and Cover for Concrete Culverts
OPSS422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut (Nov. 2015)
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specifications for Temporary Protection Systems
OPSS.PROV 915	Construction Specification for Sign Support Structures
OPSS.PROV 1010	Material Specification for Aggregates
OPSS.PROV 1205	Material Specification for Clay Seal
SP 206S03	Earth Excavation, Grading
SP 599S22	Retained Soil System, (Design and Construction Requirements)
SP 599S23	Retained Soil System (Requirements for Materials and QC/QA testing)

## 10.0 INSTRUMENTATION AND MONITORING

An Instrumentation and Monitoring Plan will be prepared three months prior to commencement of earthworks. The Plan will include discussion on the following items:

- Potential for ground movements and impacts to Highway 401;
- Potential impacts of proposed construction on other surrounding facilities;
- Identification of the need, if any, of measuring pore water pressures in cohesive soils beneath areas of new roadway embankment construction; and
- Discussion of settlement monitoring requirements during and following construction. As a minimum, monitoring is expected to include establishment of survey points along the road surface near the new structure locations. Immediately following paving, elevations at the centre line of each lane will be measured at all bridge abutments and above the culvert and at distances of 20 m and 50 m from these structures.

## 11.0 CLOSURE

The recommendations made in this report were made based on our our current understanding of the project. Stantec should be given the opportunity to review, and if necessary revise, the recommendations contained herein when the drawings and specifications are complete.

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, Stantec should be notified immediately in order to assess the additional information and its effects on the above recommendations.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Respectfully submitted,

**STANTEC CONSULTING LTD.**



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Designated Principal MTO Foundations Contact

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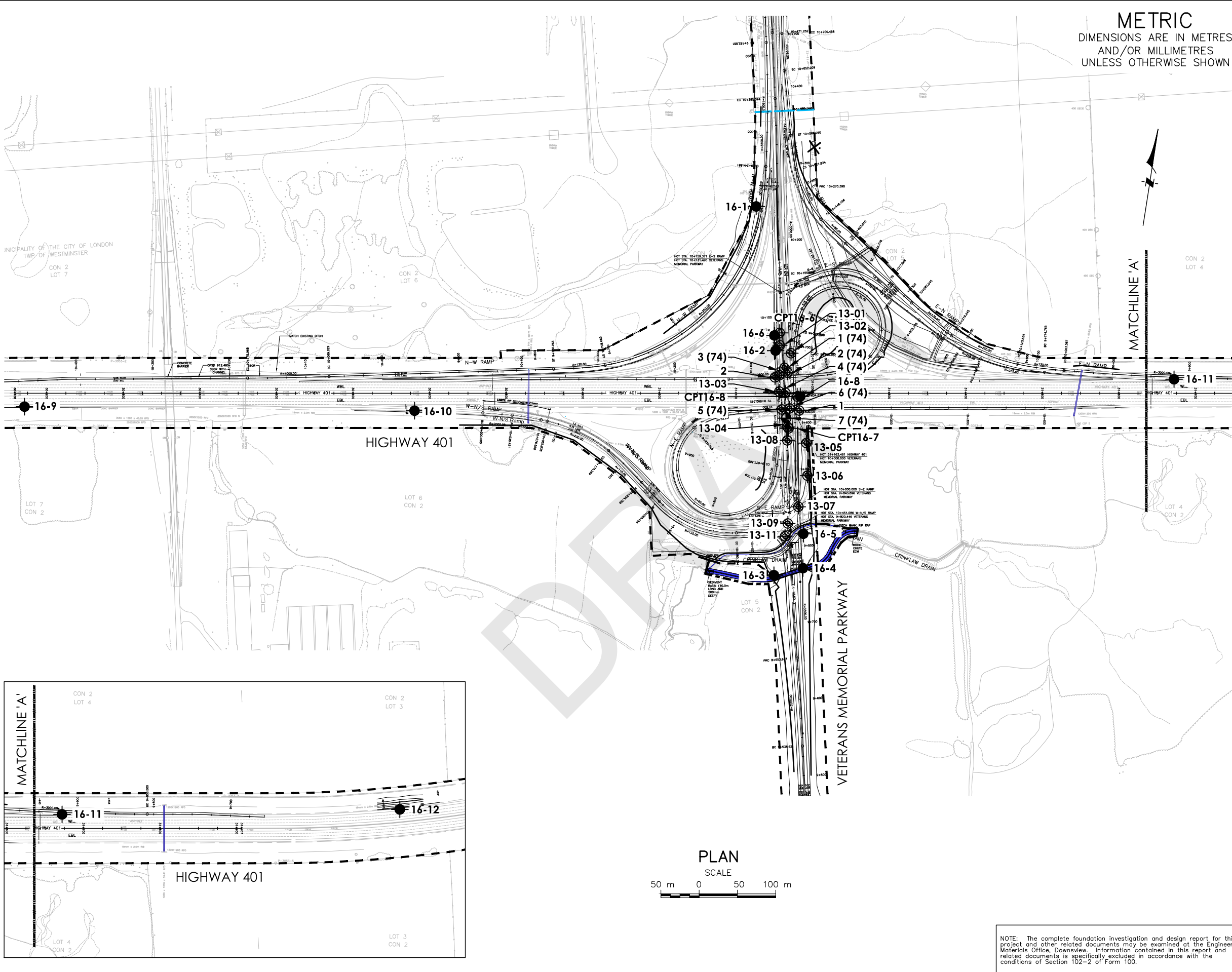
## 12.0 REFERENCES

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## APPENDIX A

Drawings Nos. 1A to 1D – Borehole Location Plans and Soil Strata Plots





METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

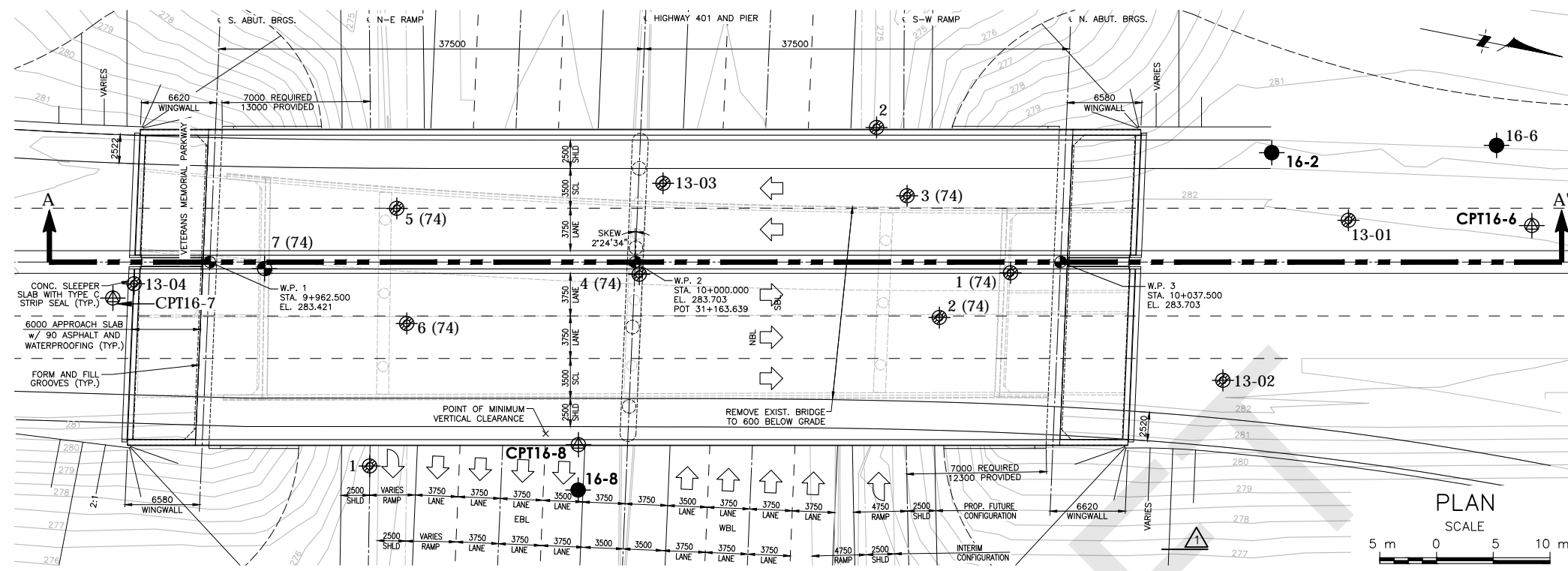
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VETERANS MEMORIAL PKWY  
INTERCHANGE IMPROVEMENTS  
BOREHOLE LOCATIONS

LEGEND				
	Borehole By Stantec			
	CPT By Stantec			
	Borehole By Others			
	Borehole & Cone By Others			
No	ELEV	MTM_ZONE 11 NORTH	COORDINATES EAST	
1	275.44	4 756 823.0	416 987.0	
2	274.87	4 756 861.0	416 949.0	
1(74)	274.09	4 756 875.1	416 959.3	
2(74)	274.15	4 756 869.7	416 964.4	
3(74)	273.75	4 756 864.8	416 954.4	
4(74)	274.91	4 756 843.0	416 965.8	
5(74)	274.42	4 756 820.9	416 964.3	
6(74)	274.76	4 756 823.8	416 974.0	
7(74)	275.18	4 756 810.5	416 971.7	
13-01	282.00	4 756 903.4	416 948.9	
13-02	282.00	4 756 895.3	416 964.9	
13-03	275.40	4 756 843.5	416 957.5	
13-04	281.90	4 756 799.5	416 975.3	
13-05	276.00	4 756 782.4	417 002.7	
13-06	275.20	4 756 740.6	417 010.6	
13-07	274.20	4 756 698.6	417 005.2	
13-08	281.30	4 756 782.1	416 977.6	
13-09	274.00	4 756 675.2	416 995.1	
13-11	273.80	4 756 657.9	416 993.9	
16-1	274.41	4 757 077.8	416 890.5	
16-2	281.38	4 756 895.6	416 944.4	
16-3	273.13	4 756 605.5	416 987.5	
16-4	273.52	4 756 620.5	417 023.5	
16-5	273.81	4 756 664.9	417 017.1	
16-6	281.25	4 756 914.9	416 939.9	
CPT16-6	281.95	4 756 919.3	416 946.2	
CPT16-7	281.81	4 756 797.9	416 976.9	
16-8	275.75	4 756 841.4	416 985.5	
CPT16-8	275.75	4 756 840.7	416 981.6	
16-9	270.74	4 756 672.4	415 986.7	
16-10	272.39	4 756 745.4	416 490.7	
16-11	278.34	4 756 938.7	417 464.3	
16-12	280.83	4 757 013.0	417 899.1	

REVISIONS				
DATE	BY	DESCRIPTION		
GEOCRES No				
HWY No	HWY	401	DIST	
SUBM'D	CHECKED	DATE 2016-05-26	SITE	
DRAWN	GBB	CHECKED	APPROVED	DWG 1A

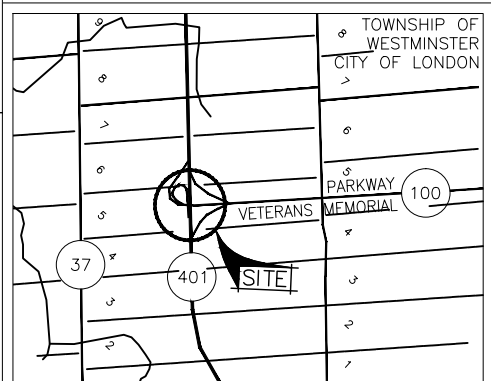
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.



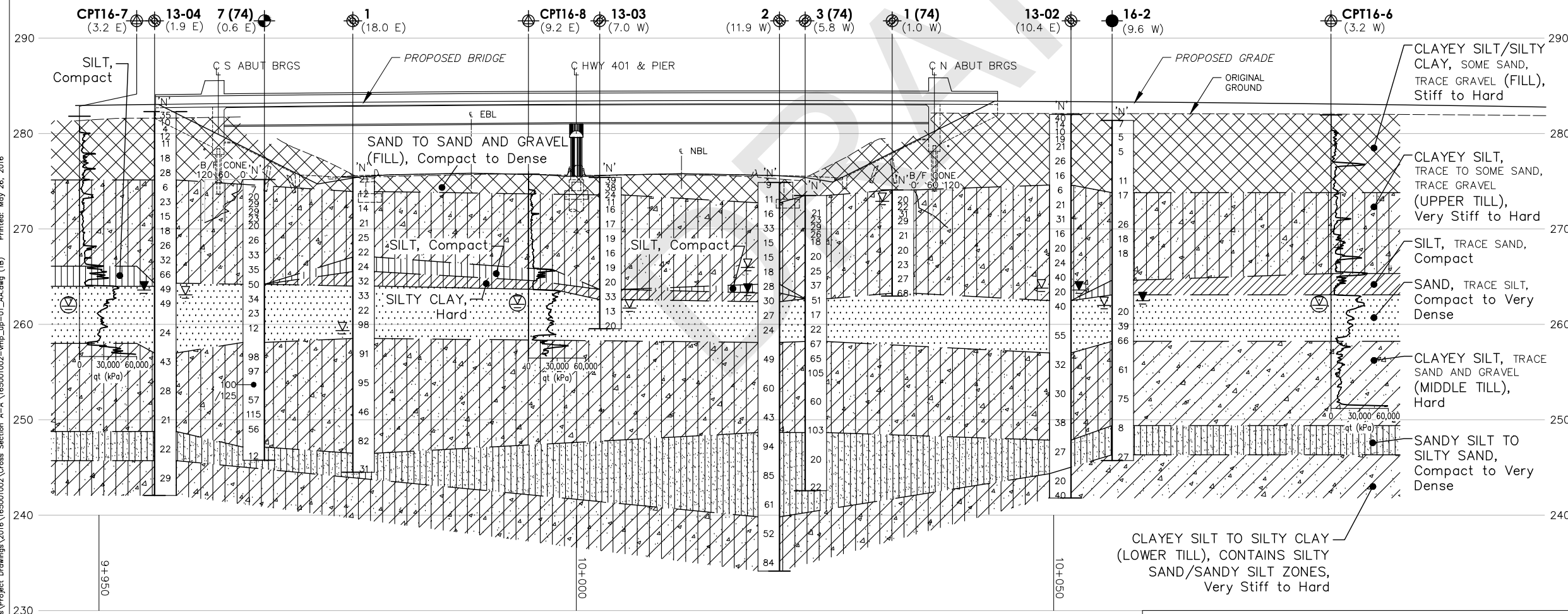
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VETERANS MEMORIAL PKWY  
INTERCHANGE IMPROVEMENTS  
BOREHOLE LOCATIONS & SOIL STRATA



KEY PLAN  
NOT TO SCALE



- LEGEND
- Borehole By Stantec
  - ⊕ CPT By Stantec
  - ⊗ Borehole By Others
  - ⊙ Borehole & Cone By Others
  - (x.x E) Offset East/West of Cross Section Line in meters
  - N Blows/0.3m (Std Pen Test, 475 J/blow)
  - ▽ WL at time of investigation, 2012-2016
  - ▬ WL Measured, 2012-2016
  - ⊖ WL - CPT Dissipation Test, 2016

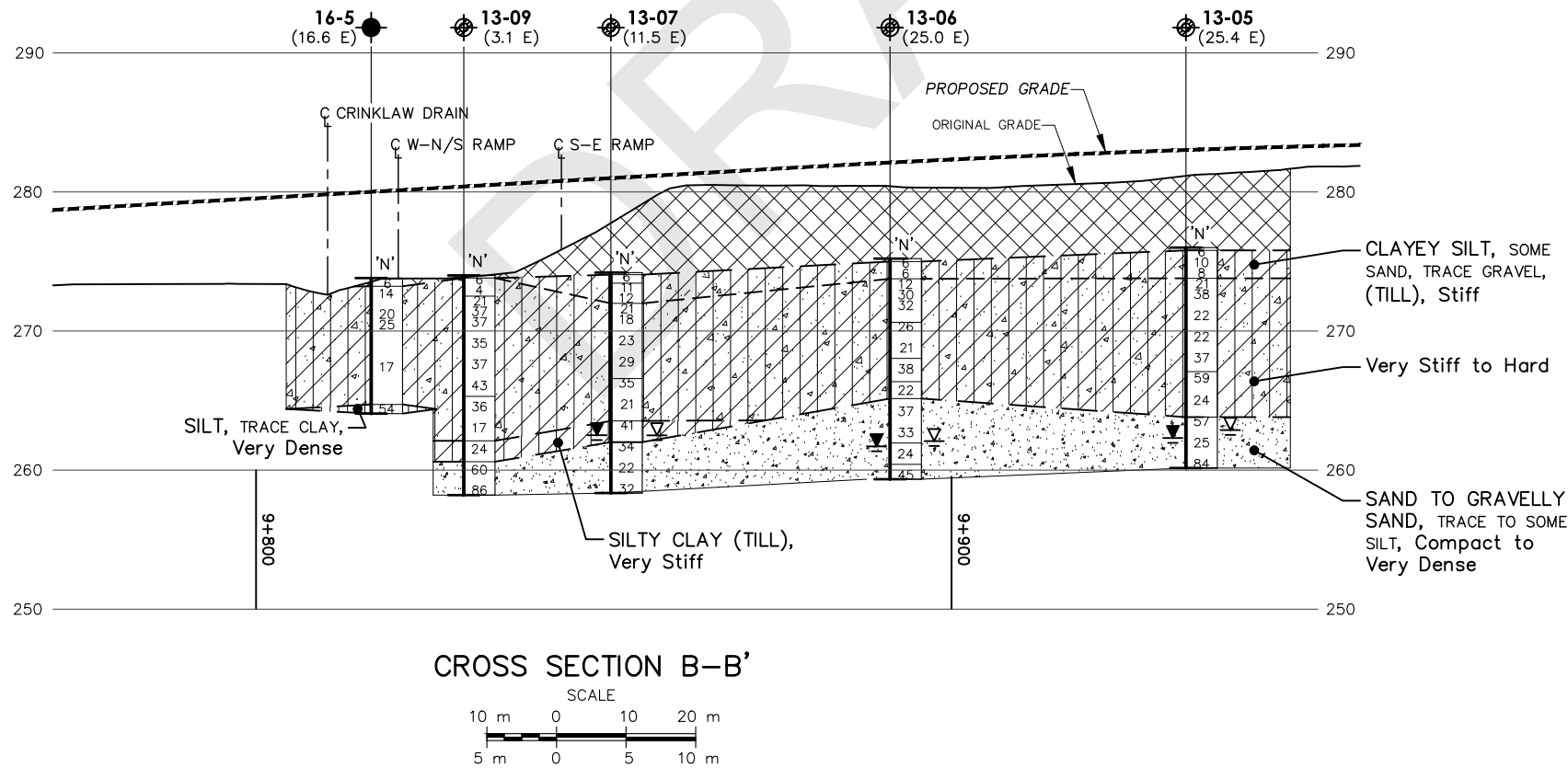
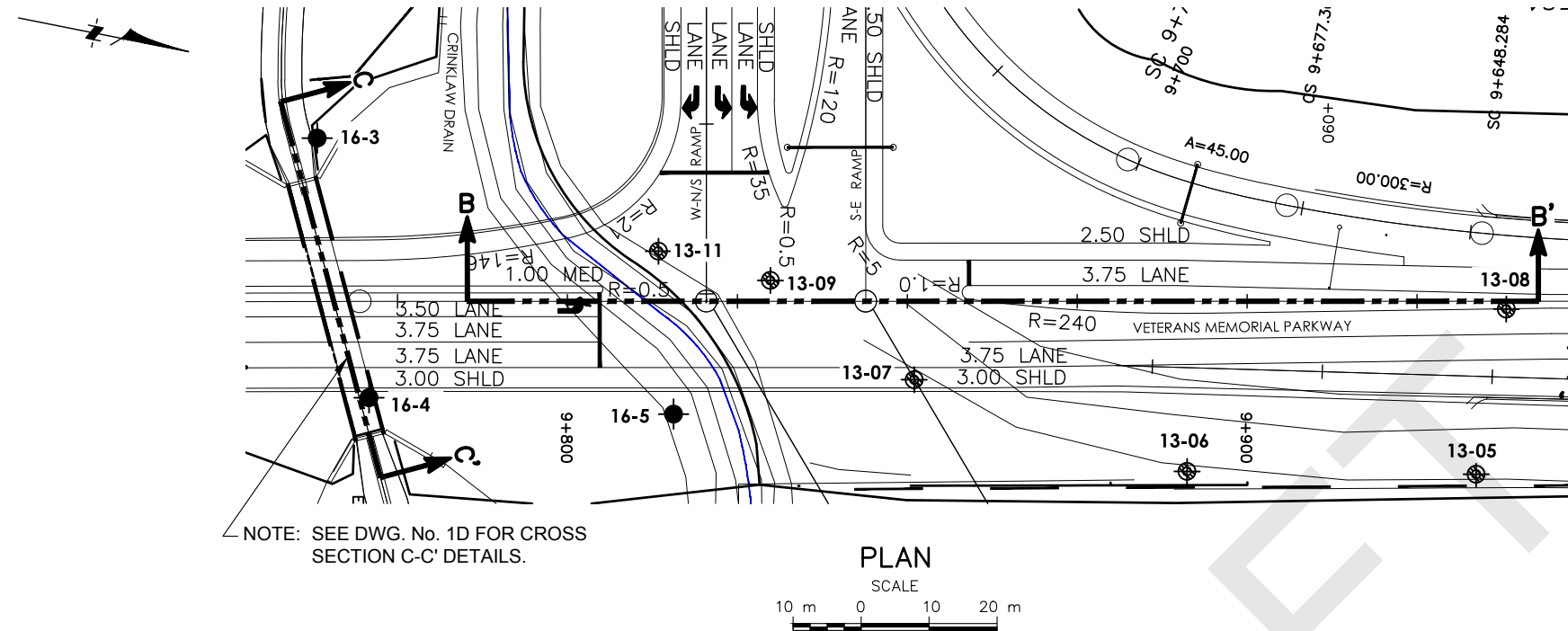
No	ELEV	MTM ZONE 11		COORDINATES	
		NORTH		EAST	
1	275.44	4 756	823.0	416	987.0
2	274.87	4 756	861.0	416	949.0
1(74)	274.09	4 756	875.1	416	959.3
2(74)	274.15	4 756	869.7	416	964.4
3(74)	273.75	4 756	864.8	416	954.4
4(74)	274.91	4 756	843.0	416	965.8
5(74)	274.42	4 756	820.9	416	964.3
6(74)	274.76	4 756	823.8	416	974.0
7(74)	275.18	4 756	810.5	416	971.7
13-01	282.00	4 756	903.4	416	948.9
13-02	282.00	4 756	895.3	416	964.9
13-03	275.40	4 756	843.5	416	957.5
13-04	281.90	4 756	799.5	416	975.3
16-1	275.41	4 757	077.7	416	890.5
16-2	281.38	4 756	895.6	416	944.4
16-6	281.25	4 756	914.9	416	939.9
CPT16-6	281.95	4 756	919.3	416	946.2
CPT16-7	281.81	4 756	797.9	416	976.9
16-8	275.75	4 756	841.4	416	985.5
CPT16-8	275.75	4 756	840.7	416	981.6

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REVISIONS		DATE		BY		DESCRIPTION	
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CROSS SECTION A-A'  
SCALE  
5 m 0 5 10 m

NOTES  
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.  
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.



METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

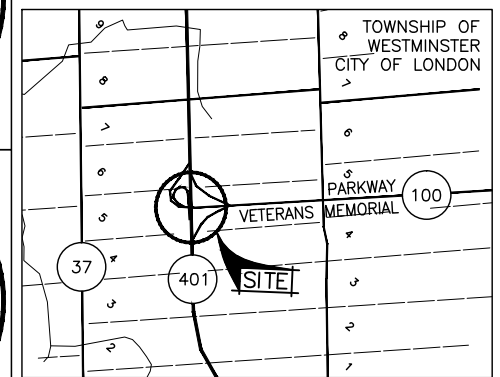


PLATE No  
**CONT**  
**WP** 3033-11-00

VETERANS MEMORIAL PKWY  
INTERCHANGE IMPROVEMENTS  
BOREHOLE LOCATIONS & SOIL STRATA



**SHEET**



KEY PLAN  
NOT TO SCALE

LEGEND

- Borehole By Stantec
- Borehole By Others
- (x.x E) Offset East/West/North of Cross Section Line in meters
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL at time of investigation Dec 2013
- WL Measured on Jan 2014

No	ELEVATION	MTM ZONE 11 NORTH	COORDINATES EAST
13-05	276.00	4 756 782.4	417 002.7
13-06	275.20	4 756 740.6	417 010.6
13-07	274.20	4 756 698.6	417 005.2
13-08	281.30	4 756 782.1	416 977.6
13-09	274.00	4 756 675.2	416 995.1
13-11	273.80	4 756 657.9	416 993.9
16-3	273.13	4 756 605.5	416 987.5
16-4	273.52	4 756 620.5	417 023.5
16-5	273.81	4 756 664.9	417 017.1

NOTES

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

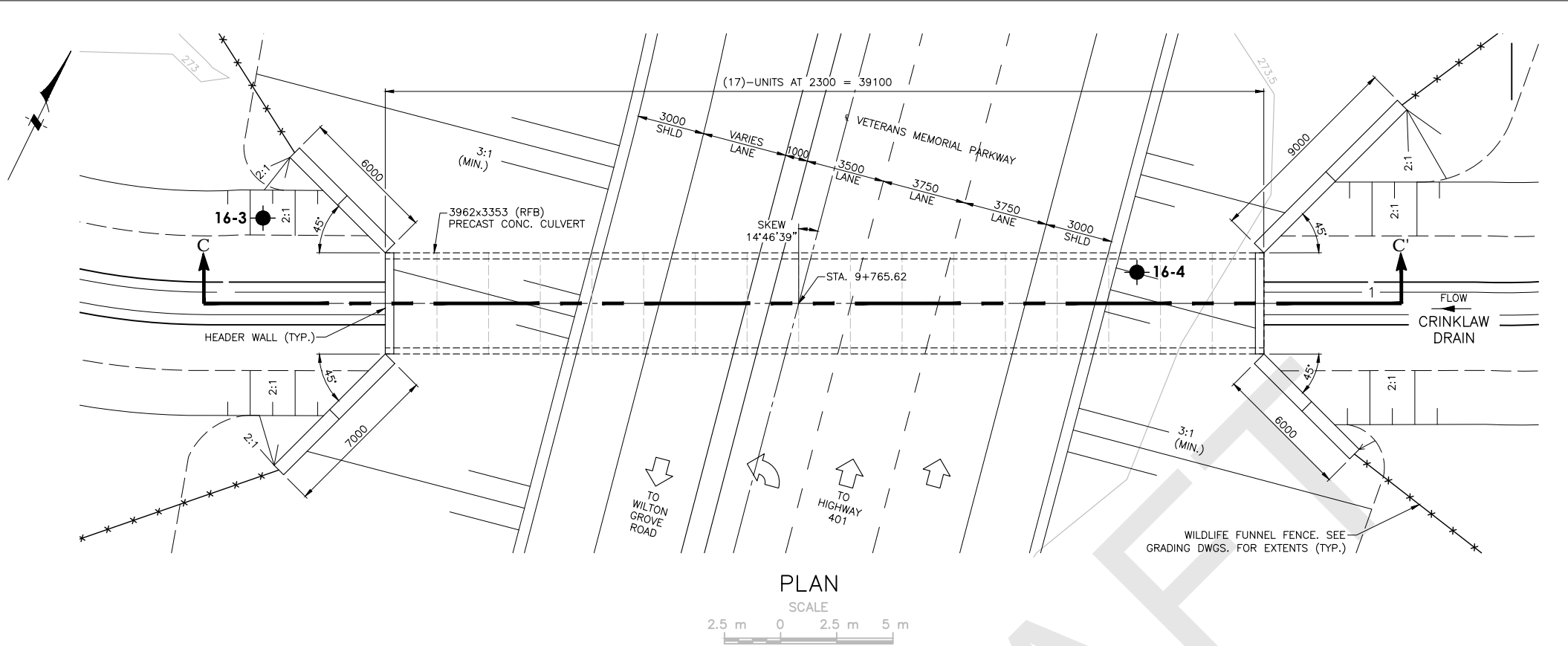
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No	
HWY No HWY	401
SUBM'D	CHECKED
DRAWN	GBB
DATE	2016-05-26
APPROVED	
DIST	
SITE	
DWG	1C





METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

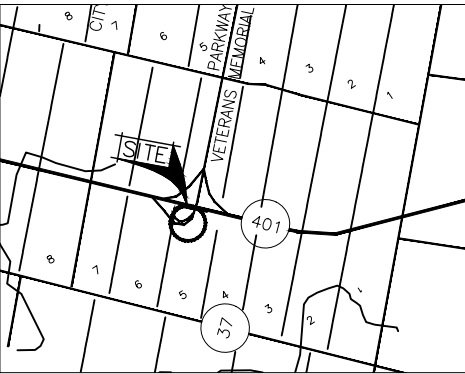


PLATE No  
CONT  
WP 3033-11-00

VETERANS MEMORIAL PKWY  
CRINKLAW DRAIN CULVERT  
BOREHOLE LOCATIONS & SOIL STRATA



SHEET



KEY PLAN  
NOT TO SCALE

LEGEND

- Borehole By Stantec
- (x.x E) Offset East/West/North of Cross Section Line in meters
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL at time of investigation April 2016
- WL Measured in May 2016

No	ELEVATION	MTM ZONE 11 NORTH	COORDINATES EAST
16-3	273.13	4 756 605.5	416 987.5
16-4	273.52	4 756 620.5	417 023.5

NOTES

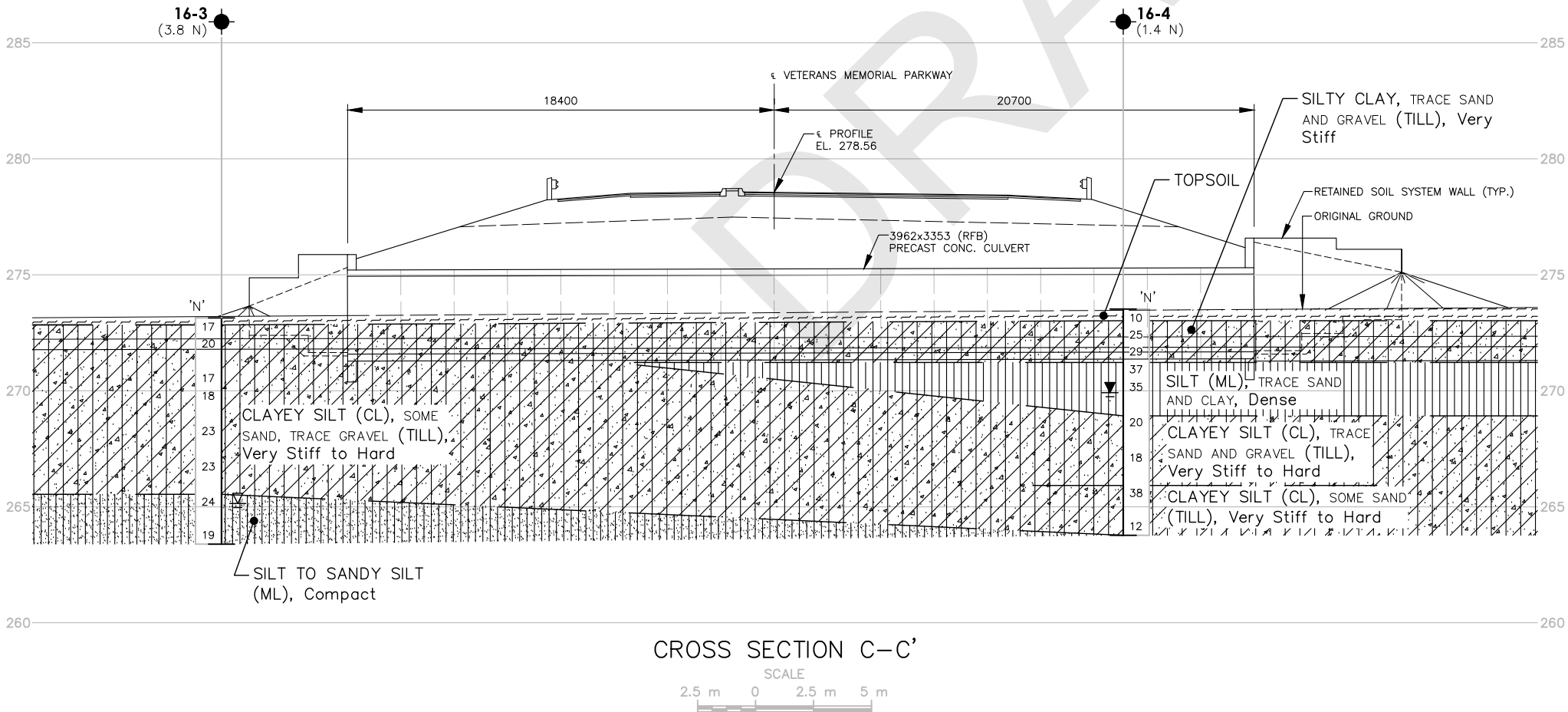
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No			
HWY No	HWY 401	DIST	
SUBM'D	CHECKED	DATE 2016-05-26	SITE
DRAWN	GGB	CHECKED	APPROVED
DWG			1D



CROSS SECTION C-C'

## APPENDIX B

Symbols and Terms Used on Borehole Records

Borehole Records (Current Investigation)

CPT 16-6 to 16-8 Results

Borehole Records from Previous Studies (Geocres Report Nos. 40I14-153, 40I14-147 and 40I14-107)

## SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

### SOIL DESCRIPTION

#### Terminology describing common soil genesis:

<i>Topsoil</i>	- mixture of soil and humus capable of supporting vegetative growth
<i>Peat</i>	- mixture of visible and invisible fragments of decayed organic matter
<i>Till</i>	- unstratified glacial deposit which may range from clay to boulders
<i>Fill</i>	- material below the surface identified as placed by humans (excluding buried services)

#### Terminology describing soil structure:

<i>Desiccated</i>	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	- having cracks, and hence a blocky structure
<i>Varved</i>	- composed of regular alternating layers of silt and clay
<i>Stratified</i>	- composed of alternating successions of different soil types, e.g. silt and sand
<i>Layer</i>	- > 75 mm in thickness
<i>Seam</i>	- 2 mm to 75 mm in thickness
<i>Parting</i>	- < 2 mm in thickness

#### Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488). The classification excludes particles larger than 76 mm (3 inches). The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

#### Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>Frequent</i>	> 20%

#### Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test N-Value (also known as N-Index). A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

#### Terminology describing consistency of cohesive soils:

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests.

Consistency	Undrained Shear Strength	
	kips/sq.ft.	kPa
<i>Very Soft</i>	<0.25	<12.5
<i>Soft</i>	0.25 - 0.5	12.5 - 25
<i>Firm</i>	0.5 - 1.0	25 - 50
<i>Stiff</i>	1.0 - 2.0	50 - 100
<i>Very Stiff</i>	2.0 - 4.0	100 - 200
<i>Hard</i>	>4.0	>200

## ROCK DESCRIPTION

### Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	<i>Very Poor</i>
25-50	<i>Poor</i>
50-75	<i>Fair</i>
75-90	<i>Good</i>
90-100	<i>Excellent</i>

Rock quality classification is based on a modified core recovery percentage (RQD) in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

### Terminology describing rock mass:

Spacing (mm)	Joint Classification	Bedding, Laminations, Bands
> 6000	<i>Extremely Wide</i>	-
2000-6000	<i>Very Wide</i>	<i>Very Thick</i>
600-2000	<i>Wide</i>	<i>Thick</i>
200-600	<i>Moderate</i>	<i>Medium</i>
60-200	<i>Close</i>	<i>Thin</i>
20-60	<i>Very Close</i>	<i>Very Thin</i>
<20	<i>Extremely Close</i>	<i>Laminated</i>
<6	-	<i>Thinly Laminated</i>

### Terminology describing rock strength:

Strength Classification	Unconfined Compressive Strength (MPa)
<i>Extremely Weak</i>	< 1
<i>Very Weak</i>	1 – 5
<i>Weak</i>	5 – 25
<i>Medium Strong</i>	25 – 50
<i>Strong</i>	50 – 100
<i>Very Strong</i>	100 – 250
<i>Extremely Strong</i>	> 250

### Terminology describing rock weathering:

Term	Description
<i>Fresh</i>	No visible signs of rock weathering. Slight discolouration along major discontinuities
<i>Slightly Weathered</i>	Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.
<i>Moderately Weathered</i>	Less than half the rock is decomposed and/or disintegrated into soil.
<i>Highly Weathered</i>	More than half the rock is decomposed and/or disintegrated into soil.
<i>Completely Weathered</i>	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

## STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders  
Cobbles  
Gravel



Sand



Silt



Clay



Organics



Asphalt



Concrete



Fill



Bedrock

## SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

## WATER LEVEL MEASUREMENT



measured in standpipe,  
piezometer, or well



inferred

## RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

## N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and N-values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N value corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

## DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to A size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (305 mm) into the soil. The DCPT is used as a probe to assess soil variability.

## OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
$\gamma$	Unit weight
$G_s$	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
$Q_u$	Unconfined compression
$I_p$	Point Load Index ( $I_p$ on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer





## RECORD OF BOREHOLE No 16-1

1 OF 2

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 757 078 E: 416 891 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 08 - 2016 04 08 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	✕ FIELD VANE						
						● QUICK TRIAXIAL	✕ LAB VANE								
274.4	Ground Surface							20 40 60 80 100							
274.9	Clayey silt (TOPSOIL)							20 40 60 80 100							
0.2	Silty clay, trace sand and gravel (FILL)		1	SS	12		274							2 19 57 22	
	Stiff Brown to grey Moist														
273.3			2	SS	9		273								
1.1	CLAYEY SILT (CL), trace sand and gravel (TILL)														
	Stiff to hard Brown to grey Moist		3	SS	20									Su >265 kPa	
	Sand seam at 2.3 m		4	ST			272								
	Wet sandy silt seam from 3.4 m to 3.7 m		5	SS	27		271							0 5 80 15 Non-Plastic	
	Grey below 4.8 m		6	ST			270								
														Consolidation	

Continued Next Page

 $\times^3, \times^3$ : Numbers refer to Sensitivity $\circ^3$  STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON.GPJ ONTARIO MOT.GDT 5/24/16



## RECORD OF BOREHOLE No 16-1

2 OF 2

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 757 078 E: 416 891 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 08 - 2016 04 08 CHECKED BY KN

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
							20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
	Monitoring Well Installed - Screen from depth 1.5 m to 4.6 m  Water level in monitoring well at 0.23 m depth (Elev. 274.18 m) on April 22, 2016, and May 3, 2016															



## RECORD OF BOREHOLE No 16-2

1 OF 4

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 896 E: 416 944 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 07 - 2016 04 08 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub>	W	W <sub>L</sub>		
								○ UNCONFINED      ✕ FIELD VANE ● QUICK TRIAXIAL    ✕ LAB VANE	WATER CONTENT (%)					
281.4	Ground Surface						20 40 60 80 100							
280.4	Silty Clay, trace sand, gravel and rootlets (TOPSOIL)		1	SS	7									
	Clayey silt to silty clay, some sand, trace gravel (FILL)													
	Stiff to hard Brown to grey Moist to wet													
			2	SS	5									
			3	SS	5									
			4	ST										
			5	SS	11									

Continued Next Page

$\times^3, \times^3$ : Numbers refer to Sensitivity  $\circ^3$  STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON.GPJ ONTARIO MOT.GDT 5/24/16



## RECORD OF BOREHOLE No 16-2

2 OF 4

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 896 E: 416 944 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 07 - 2016 04 08 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub>	W	W <sub>L</sub>			
								○ UNCONFINED ● QUICK TRIAXIAL	✕ FIELD VANE ✕ LAB VANE						
								20 40 60 80 100	20 40 60 80 100	WATER CONTENT (%)			kN/m <sup>3</sup>	GR SA SI CL	
10.0	<i>continued</i> CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Contains cobbles or boulders.  Very stiff to hard Grey to brown Moist to wet          - Auger grinding on inferred cobble or boulder						271							5 15 56 24 w <sub>c</sub> = 13.9%	
			8	SS	26		270								
			9	SS	18		269								
			10	SS	18		268								
							267								
							266								
			11	ST			266								
							265								
264.6							265								
16.8	SILT (ML), trace to some clay, trace sand  Dense Light grey Moist		12	ST			264								
								264							
263.1							263								
18.3	SAND (SP), trace silt  Compact to very dense Grey Moist  Wet below 19.4 m		13	ST			263								
								262							

Continued Next Page

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

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON GP J ONTARIO MOT.GDT 5/24/16

W.P.	<u>3033-11-00</u>	LOCATION	<u>Veterans Memorial Parkway &amp; Highway 401, London, ON N: 4 756 896 E: 416 944</u>	ORIGINATED BY	<u>RC</u>
DIST	<u>West</u>	HWY	<u>401</u>	BOREHOLE TYPE	<u>Hollow Stem Augers, Split Spoon Sampler</u>
DATUM	<u>Geodetic</u>	DATE	<u>2016 04 07 - 2016 04 08</u>	COMPILED BY	<u>KF</u>
				CHECKED BY	<u>KN</u>

[illegible]

Continued Next Page


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

○ <sup>3%</sup> STRAIN AT FAILURE

W.P.	3033-11-00	LOCATION	Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 896 E: 416 944	ORIGINATED BY	RC
DIST	West HWY 401	BOREHOLE TYPE	Hollow Stem Augers, Split Spoon Sampler	COMPILED BY	KF
DATUM	Geodetic	DATE	2016 04 07 - 2016 04 08	CHECKED BY	KN

[illegible]

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON.GPJ ONTARIO MOT.GDT 5/24/16

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



## RECORD OF BOREHOLE No 16-3

1 OF 2

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 606 E: 416 987 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 11 - 2016 04 11 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub>	W	W <sub>L</sub>		WATER CONTENT (%)				
								○ UNCONFINED      × FIELD VANE										
273.1	Ground Surface							20   40   60   80   100							GR	SA	SI	CL
0.0	Clayey Silt (TOPSOIL)						273											
272.8																		
0.3	CLAYEY SILT (CL), some sand, trace gravel (TILL)		1	SS	17													
	Very stiff to hard																	
	Brown		2	SS	20		272											
	Moist																	
			3	ST														
							271											
	Grey below 2.3 m		4	SS	17													
							270											
			5	SS	18													
							269											
			6	SS	23		268											
							267											
			7	SS	23													
							266											
265.5																		
7.6	SILT to SANDY SILT (ML)		8	SS	24		265											
	Compact																	
	Grey																	
	Wet						264											
			9	SS	19													
263.4																		
9.8	End of Borehole																	

Continued Next Page

$\times^3, \times^3$ : Numbers refer to Sensitivity  $\circ^3$  STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON.GPJ ONTARIO MOT.GDT 5/24/16







## RECORD OF BOREHOLE No 16-4

1 OF 2

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 620 E: 417 023 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 11 - 2016 04 11 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub>	W	W <sub>L</sub>			WATER CONTENT (%)
								○ UNCONFINED      ✕ FIELD VANE ● QUICK TRIAXIAL    ✕ LAB VANE							
273.5	Ground Surface							20 40 60 80 100					kN/m <sup>3</sup>	GR SA SI CL	
0.0	Clayey silt (TOPSOIL)														
	Dark brown to black														
273.0							273								
0.5	SILTY CLAY, trace sand and gravel (TILL)		1	SS	10										
	Very stiff		2	SS	25										
	Brown, moist						272								
			3	SS	29										
271.2															
2.3	SILT (ML), trace sand and clay						271								
	Dense		4	SS	37									0 4 91 5	
	Grey													Non-Plastic	
	Moist														
	Wet below 3 m		5	SS	35		270							0 6 91 3	
														Non-Plastic	
268.9							269								
4.6	CLAYEY SILT (CL), trace sand and gravel (TILL)		6	SS	20		268								
	Very stiff to hard														
	Grey														
	Moist		7	SS	18		267							2 12 55 31	
265.9							266								
7.6	CLAYEY SILT (CL), some sand (TILL)		8	SS	38		265								
	Very stiff to hard														
	Grey		9	SS	12		264								
	Wet														
263.8															
9.8	End of Borehole														

Continued Next Page

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

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON.GPJ ONTARIO MOT.GDT 5/24/16



## RECORD OF BOREHOLE No 16-4

2 OF 2

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 620 E: 417 023 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 11 - 2016 04 11 CHECKED BY KN

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
							20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
	Monitoring Well Installed - Screen from depth 4.3 m to 5.9 m  Water level in monitoring well at 3.3 m depth (Elev 270.22 m) on April 22, 2016  Water level in monitoring well at 3.6 m depth (Elev 269.92 m) on May 3, 2016															



## RECORD OF BOREHOLE No 16-5

1 OF 2

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 665 E: 417 017 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 11 - 2016 04 11 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub>	W	W <sub>L</sub>		
273.8	Ground Surface							20 40 60 80 100						
0.0	SILTY CLAY, trace sand and gravel (TOPSOIL)							20 40 60 80 100						
273.2	Firm Brown to Black		1	SS	6		273							
0.6	CLAYEY SILT (CL), some sand, trace gravel (TILL)													
	Very stiff to hard Brown Moist		2	SS	14									
			3	ST			272			15.0%				Su = 237 kPa Consolidation
			4	SS	20		271							2 16 57 25
	Grey below 3 m		5	SS	25		270							
			6	ST			269			23.3%				Su = 231 kPa
			7	SS	17		268							
			8	ST			266			22.4%				Su = 217 kPa
264.7	SILT (ML), trace clay		9	SS	54		265							0 0 96 4 Non-Plastic
9.1	Very dense Grey Wet													
264.1	End of Borehole													
9.8														

Continued Next Page

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

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON.GPJ ONTARIO MOT.GDT 5/24/16



W.P.	3033-11-00	LOCATION	Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 915 E: 416 940	ORIGINATED BY	RC
DIST	West HWY 401	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	KF
DATUM	Geodetic	DATE	2016 04 08 - 2016 04 08	CHECKED BY	KN

[illegible]

<sup>3</sup>,  $\times^3$ : Numbers refer to Sensitivity
 <sup>3%</sup> STRAIN AT FAILURE



## RECORD OF BOREHOLE No 16-8

1 OF 1

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, CPT COMPILED BY KF  
DATUM Geodetic DATE 2016 04 05 - 2016 04 05 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
275.7	Ground Surface							20	40	60	80	100		
0.0	275 mm ASPHALT													
275.4														
0.3	Sand and gravel (FILL)													
274.5							275							
1.2	CLAYEY SILT (CL), trace sand and gravel (TILL)  Hard Brown Moist						274							Su > 267 kPa
			1	ST			273							Su = 118 kPa
272.5														Su > 267 kPa
3.2	End of Borehole (CPT Hole)													

$\times^3, \times^3$ : Numbers refer to Sensitivity  $\circ^3$  STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON.GPJ ONTARIO MOT.GDT 5/24/16



# RECORD OF BOREHOLE No 16-9

1 OF 1

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 672 E: 415 987 ORIGINATED BY RC  
 DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
 DATUM Geodetic DATE 2016 04 04 - 2016 04 04 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	✕ FIELD VANE	● QUICK TRIAXIAL	✕ LAB VANE									
270.8	Ground Surface							20	40	60	80	100								
270.0	125 mm ASPHALT							20	40	60	80	100								
0.1	Sand and gravel, trace silt (FILL)																			
	Compact Brown																			
270.0	Silty clay, some sand, trace gravel (FILL)						270													
0.8	Firm to Stiff Grey Moist to wet		1	SS	6															
			2	SS	11		269													
268.5	CLAYEY SILT (CL) to SANDY SILT (ML), trace sand																			
2.3	Stiff to very stiff / compact Brown to grey Wet		3	SS	6		268										0 5 79 16			
			4	SS	11															
							267													
	Trace gravel below 4.5 m																			
			5	SS	21		266										0 20 69 11 Non-Plastic			
							265													
			6	SS	26		264													
263.8	End of Borehole																			
7.0	Water level in open borehole at 2.7 m depth (Elev. ) on completion of drilling																			

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON GPJ ONTARIO MOT.GDT 5/24/16

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



## RECORD OF BOREHOLE No 16-10

1 OF 1

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 745 E: 416 491 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 04 - 2016 04 04 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	✕ FIELD VANE	● QUICK TRIAXIAL	✕ LAB VANE	WATER CONTENT (%)					
272.4	Ground Surface						20	40	60	80	100						
272.0	115 mm ASPHALT						20	40	60	80	100						
0.1	Sand and gravel (FILL)																
	Dense Brown Moist		1	SS	37												
271.6	Silty sand and gravel (FILL)																
0.8	Compact Brown Moist		2	SS	22												21 42 (37)
270.9	CLAYEY SILT (CL), trace sand and gravel (TILL)																
1.5	Very stiff to hard Brown to grey Moist		3	SS	31												
			4	SS	28												
			5	SS	34												
			6	SS	17												2 14 54 30
																	Su = 240 kPa

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

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON GPJ ONTARIO MOT.GDT 5/24/16





## RECORD OF BOREHOLE No 16-11

1 OF 1

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 756 939 E: 417 464 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 05 - 2016 04 05 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	✕ FIELD VANE	● QUICK TRIAXIAL	✕ LAB VANE									
278.4	Ground Surface						20	40	60	80	100									
0.0	Sand and gravel (FILL)		1	SS	55															
	Very dense (frozen) Brown																			
277.5																				
0.9	CLAYEY SILT (CL), trace sand and gravel (TILL)		2	SS	14															
	Very stiff to hard Brown Moist																			
			3	SS	16															
			4	SS	51															
			5	SS	50															
			6	SS	20															
	Becomes grey below 5 m depth																			
			7	ST																
			</																	

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

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON GPJ ONTARIO MOT.GDT 5/24/16



## RECORD OF BOREHOLE No 16-12

1 OF 1

METRIC

W.P. 3033-11-00 LOCATION Veterans Memorial Parkway & Highway 401, London, ON N: 4 757 013 E: 417 899 ORIGINATED BY RC  
DIST West HWY 401 BOREHOLE TYPE Hollow Stem Augers, Split Spoon Sampler COMPILED BY KF  
DATUM Geodetic DATE 2016 04 04 - 2016 04 04 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								○ UNCONFINED	● QUICK TRIAXIAL	✕ FIELD VANE	✕ LAB VANE	WATER CONTENT (%)			
280.8	Ground Surface						20	40	60	80	100	PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	
280.0	115 mm ASPHALT						20	40	60	80	100				
0.1	Sand and gravel, trace silt (FILL)														
	Dense Brown Moist		1	SS	41										
280.0															
0.8	CLAYEY SILT (CL), some sand and gravel (TILL)		2	SS	13		280								
	Stiff to hard Brown Moist														
			3	SS	3		279								
			4	SS	6		278								1 14 56 29
															Su = 128 kPa
			5	SS	11										
							277								
			6	SS	22		276								
	Grey below 5.5 m						275								
			7	SS	20		274								
273.8															
7.0	End of Borehole														
	Water level in open borehole at 5.8 m depth on completion of drilling.														

STN13-ONTARIO MTO STANTEC 165001002 - VMP LONDON GPJ ONTARIO MOT.GDT 5/24/16

$\times^3, \times^3$ : Numbers refer to Sensitivity  $\circ^3$  STRAIN AT FAILURE

# PRESENTATION OF SITE INVESTIGATION RESULTS

## Veterans Memorial Parkway Expansion

*Prepared for:*

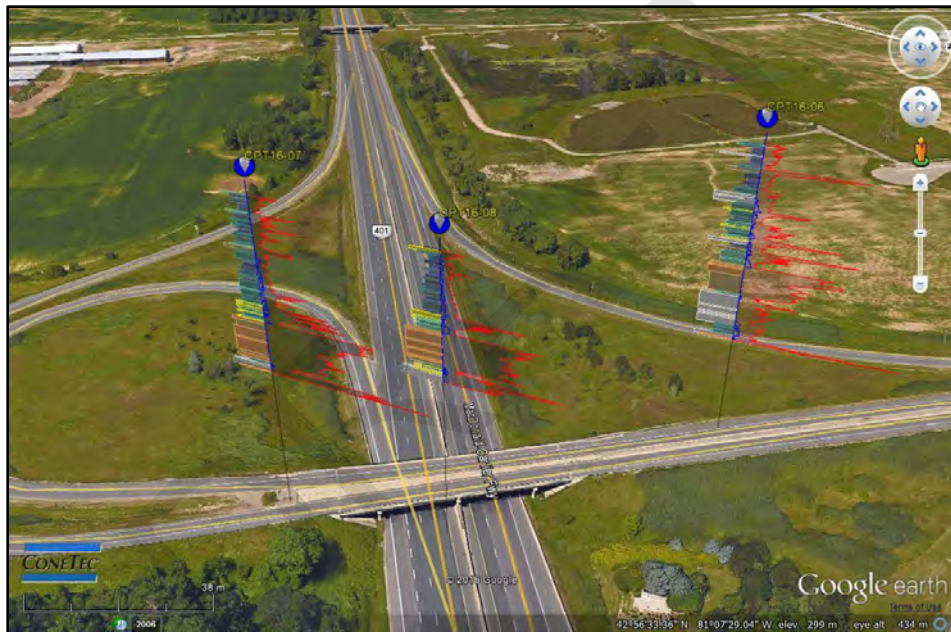
Stantec Consulting Ltd.

ConeTec Job No: 16-05007

Project Start Date: 05-Apr-2016

Project End Date: 05-Apr-2016

Report Date: 12-Apr-2016



*Prepared by:*

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### Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Stantec Consulting Ltd. at the Macdonald-Cartier Freeway and Veterans Memorial Parkway interchange in London, ON. The program consisted of three cone penetration tests.

### Project Information

Project	
Client	Stantec Consulting Ltd.
Project	Veterans Memorial Parkway Expansion
ConeTec project number	16-05007

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



Rig Description	Deployment System	Test Type
CPT truck rig (C5)	Rig Cylinder	CPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT	Consumer grade GPS	32617

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Advanced plots with Su(Nkt)

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm <sup>2</sup> )	Sleeve Area (cm <sup>2</sup> )	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
419:T1500F15U500	AD419	15	225	1500	15	500
Cone AD419 was used for all CPT soundings.						

Interpretation Tables	
Additional information	<p>The Soil Behaviour Type (SBT) classification chart (Robertson et al., 1986) was used to classify the soil for this project. A detailed set of CPT interpretations were generated and are provided in Excel format files in the release folder. The CPT interpretations are based on values of corrected tip (<math>q_t</math>), sleeve friction (<math>f_s</math>) and pore pressure (<math>u_2</math>) averaged over a user specified interval of 20 cm.</p> <p>Soils were classified as either drained or undrained based on the Soil Behaviour Type (SBT) classification chart (Robertson et al., 1986). Calculations for both drained and undrained parameters were included for materials that classified as silt (zone 6).</p>

## Limitations

This report has been prepared for the exclusive use of Stantec Consulting Ltd. (Client) for the project titled "Veterans Memorial Parkway Expansion". The report's contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

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

DRAFT



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

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

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

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

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

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



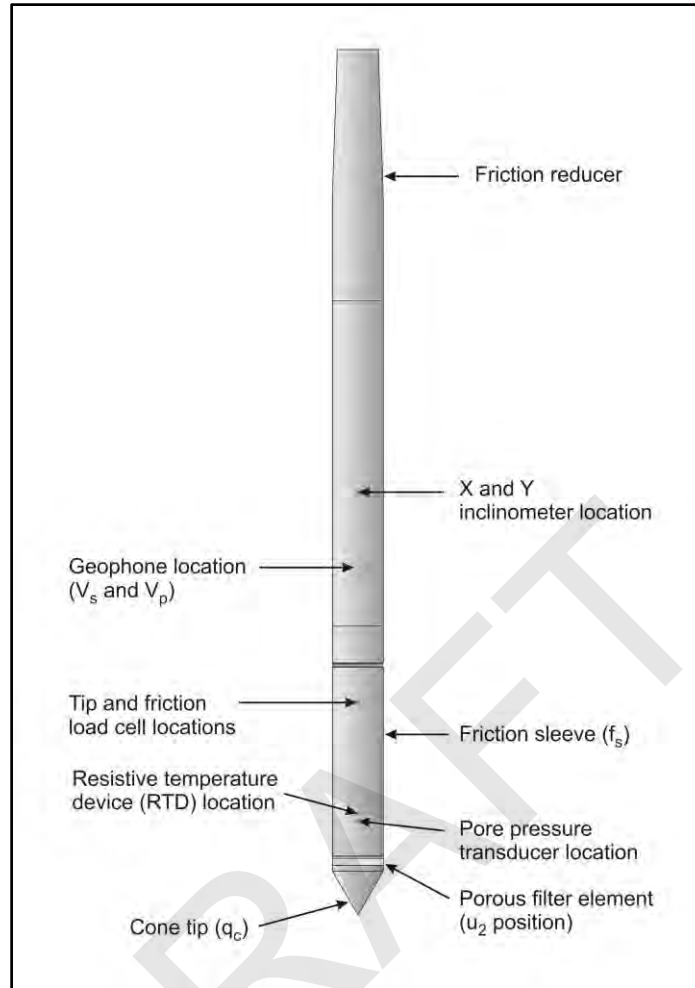


Figure CPTu. Piezocone Penetrometer (15 cm<sup>2</sup>)

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

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

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



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

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

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

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

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

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

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

where:  $q_t$  is the corrected tip resistance

$q_c$  is the recorded tip resistance

$u_2$  is the recorded dynamic pore pressure behind the tip ( $u_2$  position)

$a$  is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

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

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

The friction ratio ( $R_f$ ) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high

friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

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

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

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

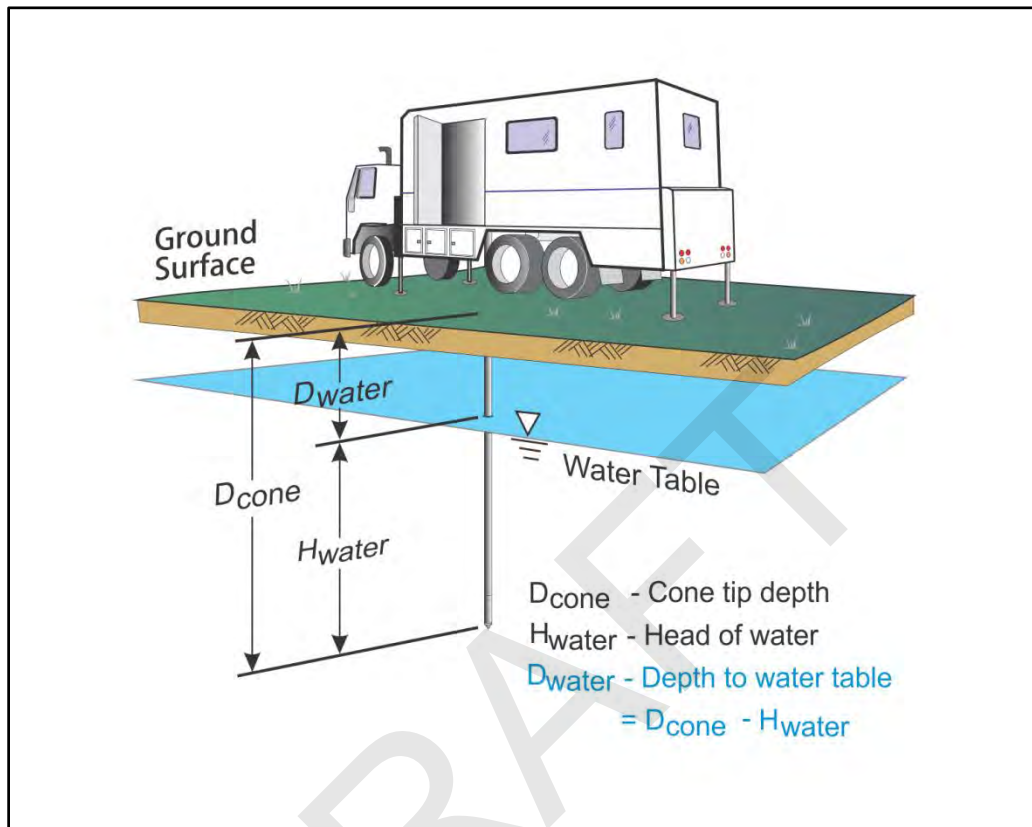


Figure PPD-1. Pore pressure dissipation test setup

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

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

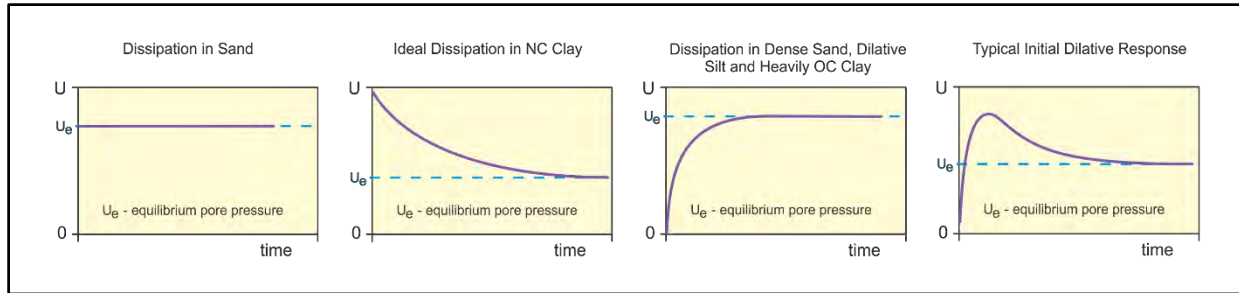


Figure PPD-2. Pore pressure dissipation curve examples

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

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

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

Where:

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

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

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

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

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

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

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

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

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

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The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with Undrained Shear Strength (Nkt)
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

DRAFT

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

DRAFT





Job No: 16-05007  
Client: Stantec Consulting Ltd.  
Project: Veterans Memorial Parkway Expansion  
Start Date: 05-Apr-2016  
End Date: 05-Apr-2016

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

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface <sup>1</sup> (m)	Final Depth (m)	Northing <sup>2</sup> (m)	Easting (m)	Refer to Notation Number
CPT16-06	16-05007_CP06	05-Apr-2016	419:T1500F15U500	20.0	30.55	4754588	489739	
CPT16-07	16-05007_CP07	05-Apr-2016	419:T1500F15U500	19.9	24.95	4754462	489767	
CPT16-08	16-05007_CP08	05-Apr-2016	419:T1500F15U500	13.6	19.00	4754504	489774	

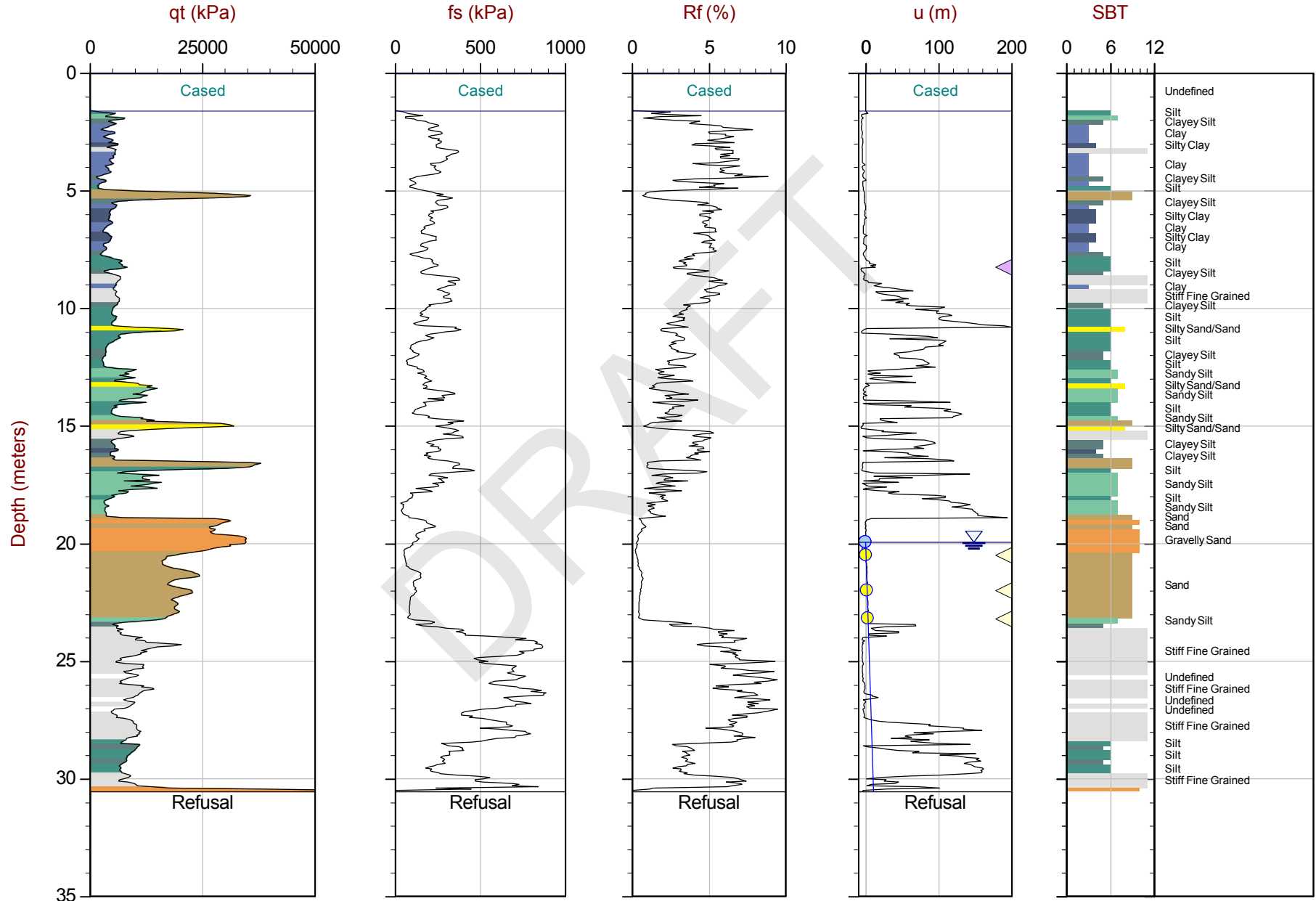
1. The assumed phreatic surface was based on pore pressure dissipation tests. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device with datum WGS84/UTM Zone 17 North.



# Stantec Consulting Ltd.

Job No: 16-05007  
Date: 04:05:16 12:39  
Site: V.M.P. Expansion

Sounding: CPT16-06  
Cone: 419:T1500F15U500  
Surface el. 282.0 m (Stantec)



Max Depth: 30.550 m / 100.23 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.200 m

File: 16-05007\_CP06.COR  
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986  
Coords: UTM 17N N: 4754588m E: 489739m  
Sheet No: 1 of 1

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

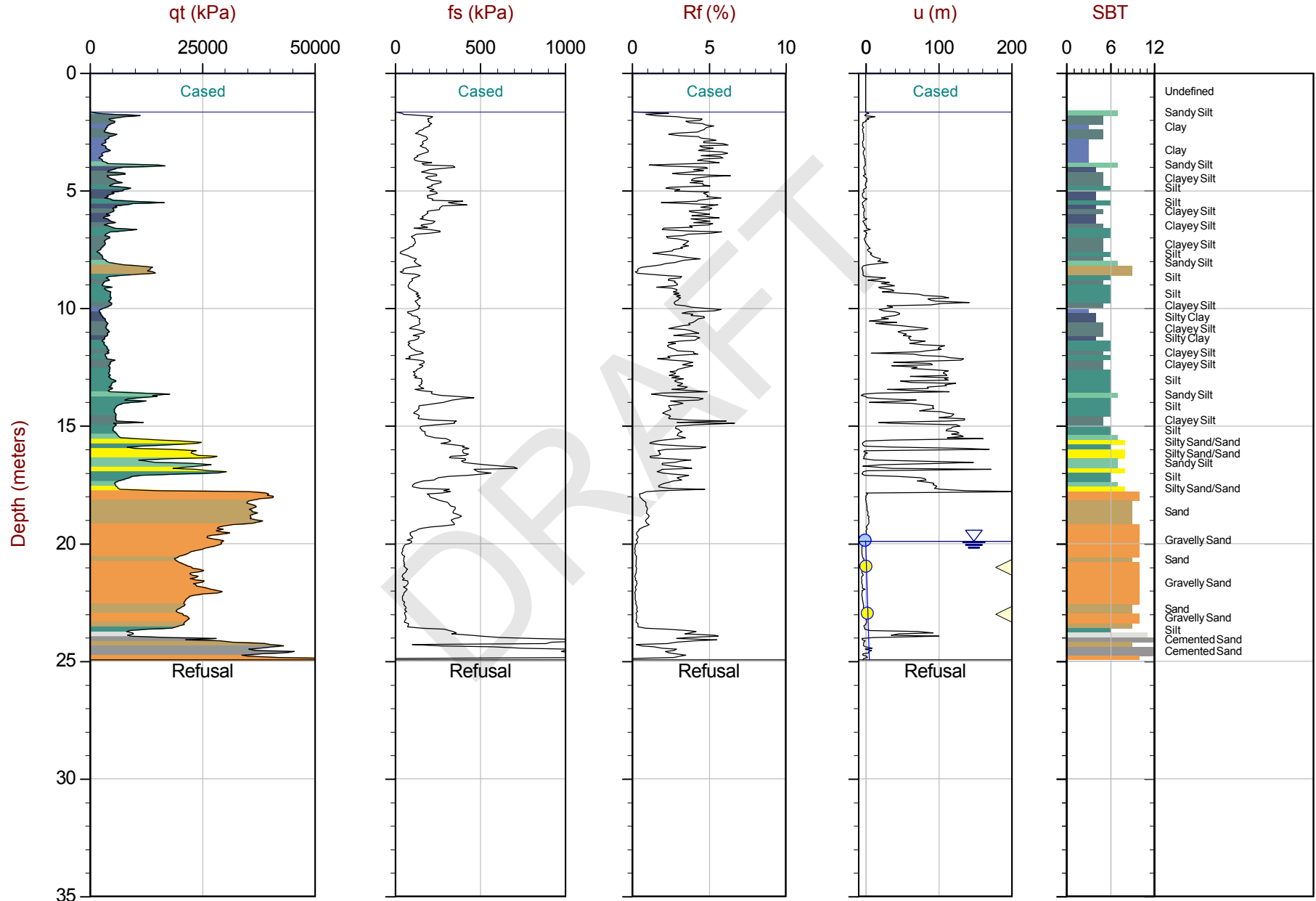
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



# Stantec Consulting Ltd.

Job No: 16-05007  
Date: 04:05:16 14:55  
Site: V.M.P. Expansion

Sounding: CPT16-07  
Cone: 419:T1500F15U500  
Surface el. 281.8 m (Stantec)



Max Depth: 24.950 m / 81.86 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.200 m

File: 16-05007\_CP07.COR  
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986  
Coords: UTM 17N: 4754462m E: 489767m  
Sheet No: 1 of 1

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

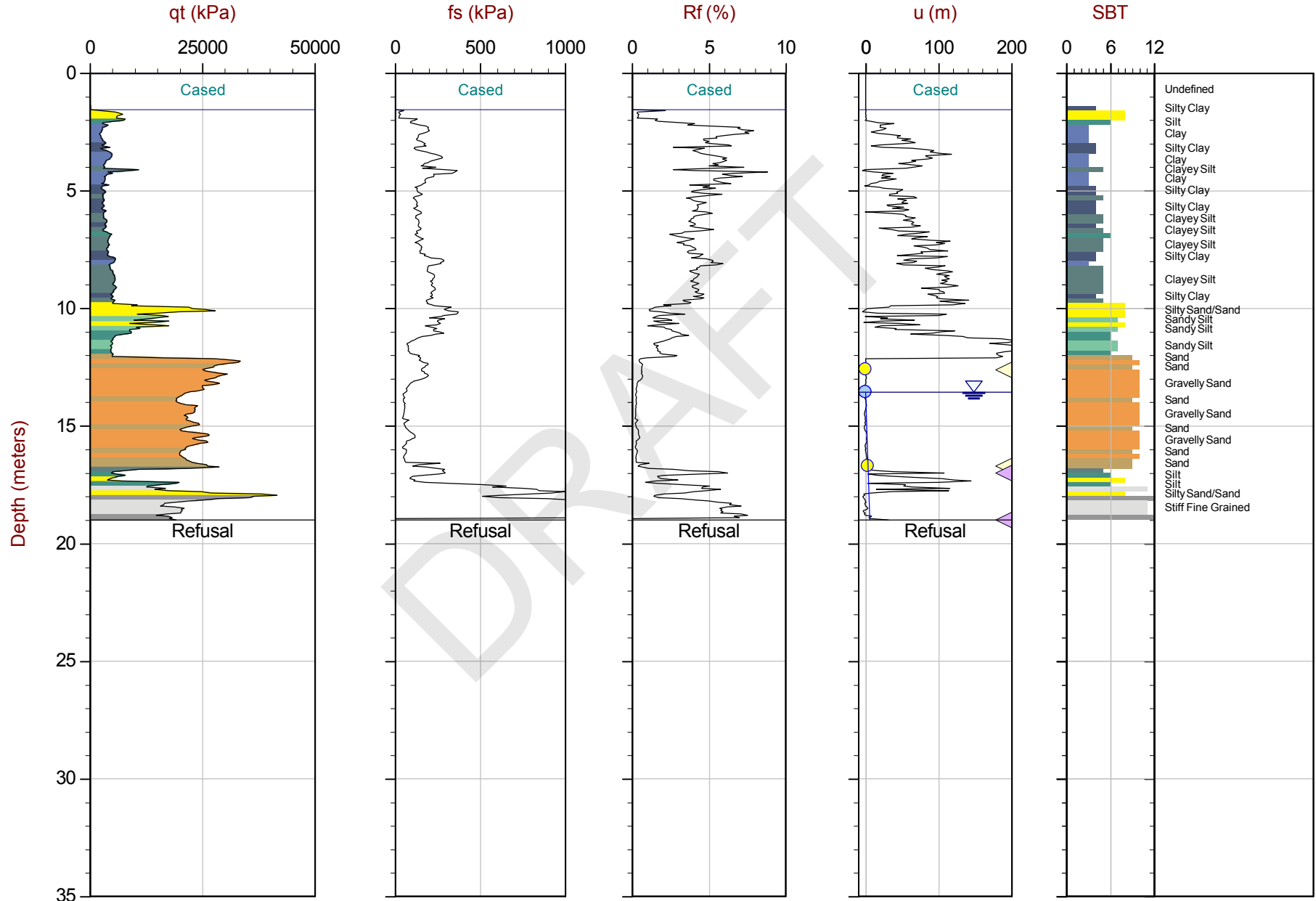
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



# Stantec Consulting Ltd.

Job No: 16-05007  
Date: 04:05:16 09:23  
Site: V.M.P. Expansion

Sounding: CPT16-08  
Cone: 419:T1500F15U500  
Surface el. 275.8 m (Stantec)



Max Depth: 19.000 m / 62.34 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.200 m

File: 16-05007\_CP08.COR  
Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986  
Coords: UTM 17N: 4754504m E: 489774m  
Sheet No: 1 of 1

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

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

Advanced Cone Penetration Test Plots with  
Undrained Shear Strength (Nkt)

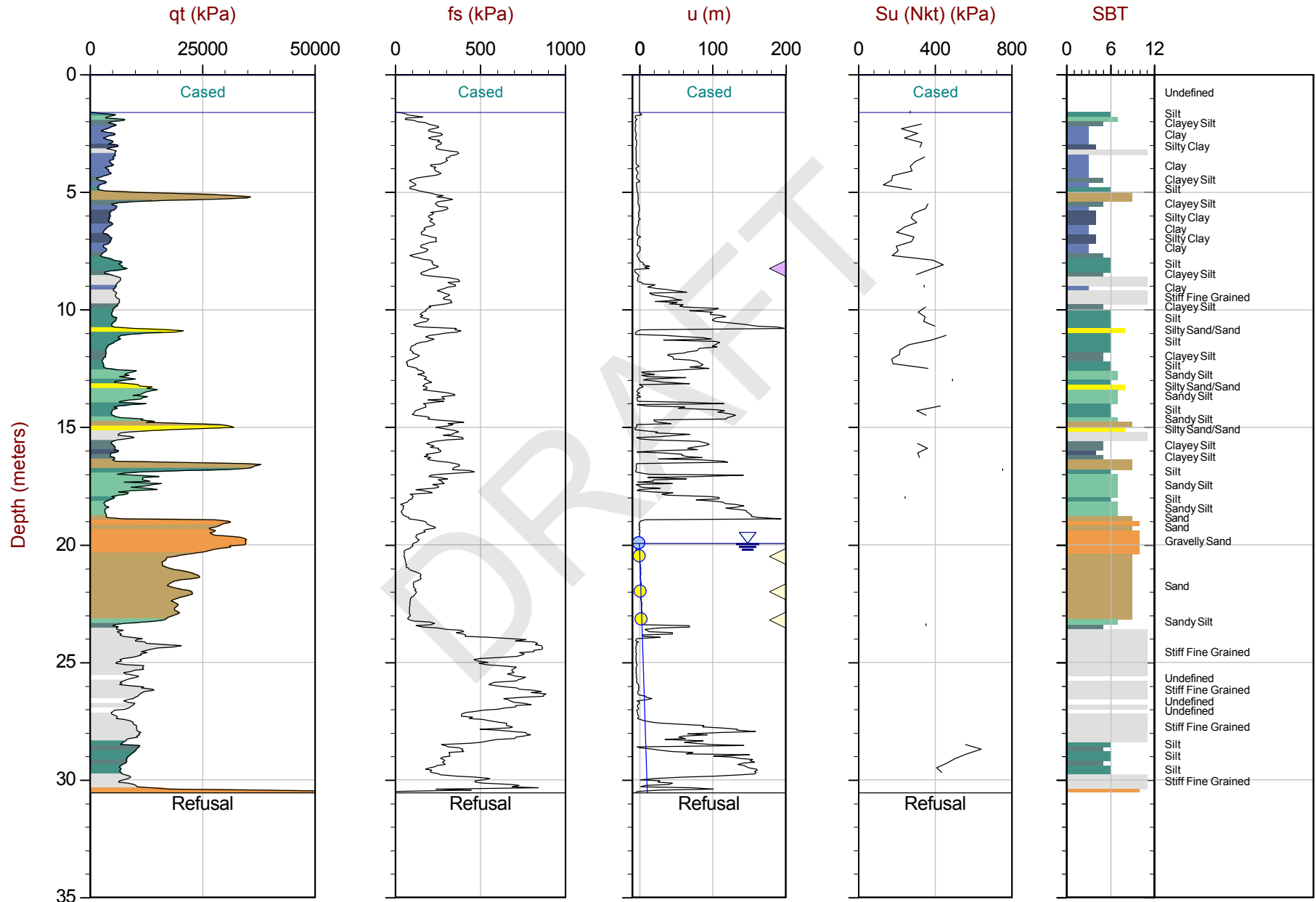
DRAFT



# Stantec Consulting Ltd.

Job No: 16-05007  
Date: 04:05:16 12:39  
Site: V.M.P. Expansion

Sounding: CPT16-06  
Cone: 419:T1500F15U500  
Surface el. 282.0 m (Stantec)



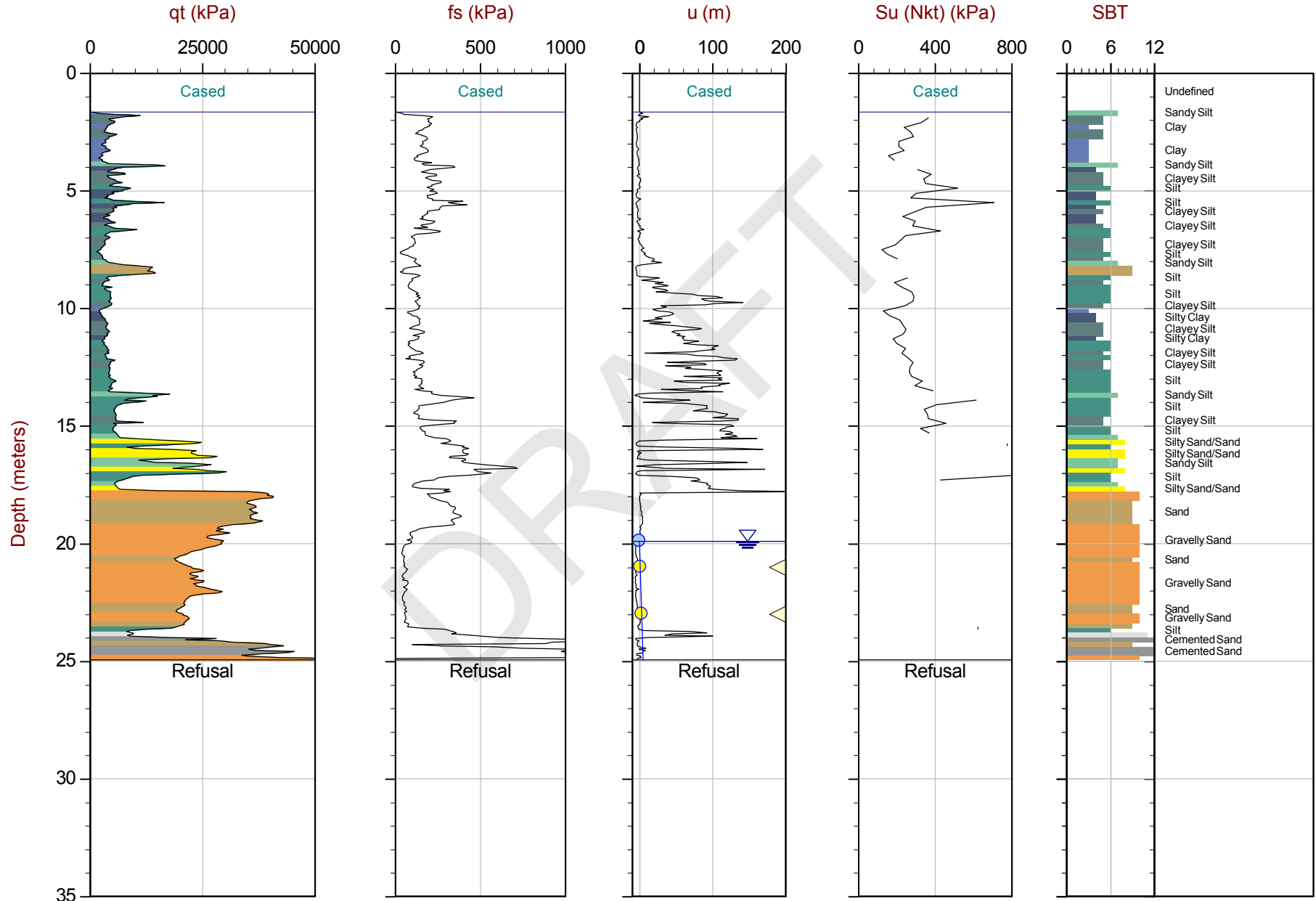
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



# Stantec Consulting Ltd.

Job No: 16-05007  
Date: 04:05:16 14:55  
Site: V.M.P. Expansion

Sounding: CPT16-07  
Cone: 419:T1500F15U500  
Surface el. 281.8 m (Stantec)



Max Depth: 24.950 m / 81.86 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.200 m

File: 16-05007\_CP07.COR  
Unit Wt: SBT Zones  
Su Nkt: 15.0

SBT: Robertson and Campanella, 1986  
Coords: UTM 17N: 4754462m E: 489767m  
Sheet No: 1 of 1

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

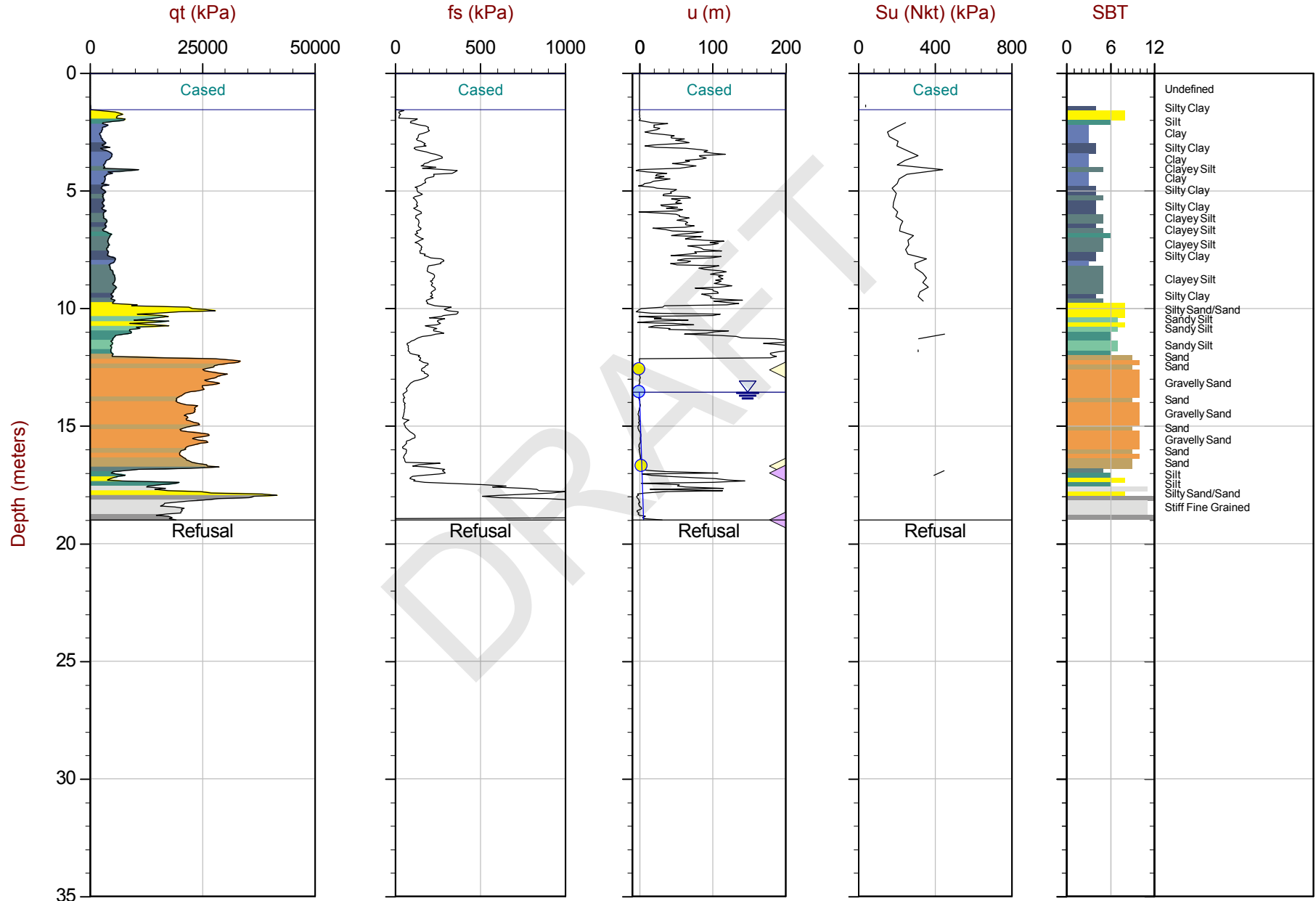
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



# Stantec Consulting Ltd.

Job No: 16-05007  
Date: 04:05:16 09:23  
Site: V.M.P. Expansion

Sounding: CPT16-08  
Cone: 419:T1500F15U500  
Surface el. 275.8 m (Stantec)



Max Depth: 19.000 m / 62.34 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.200 m

File: 16-05007\_CP08.COR  
Unit Wt: SBT Zones  
Su Nkt: 15.0

SBT: Robertson and Campanella, 1986  
Coords: UTM 17N: 4754504m E: 489774m  
Sheet No: 1 of 1

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

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



## Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

DRAFT



Job No: 16-05007  
Client: Stantec Consulting Ltd.  
Project: Veterans Memorial Parkway Expansion  
Start Date: 05-Apr-2016  
End Date: 05-Apr-2016

### ***CPT<sub>u</sub> PORE PRESSURE DISSIPATION SUMMARY***

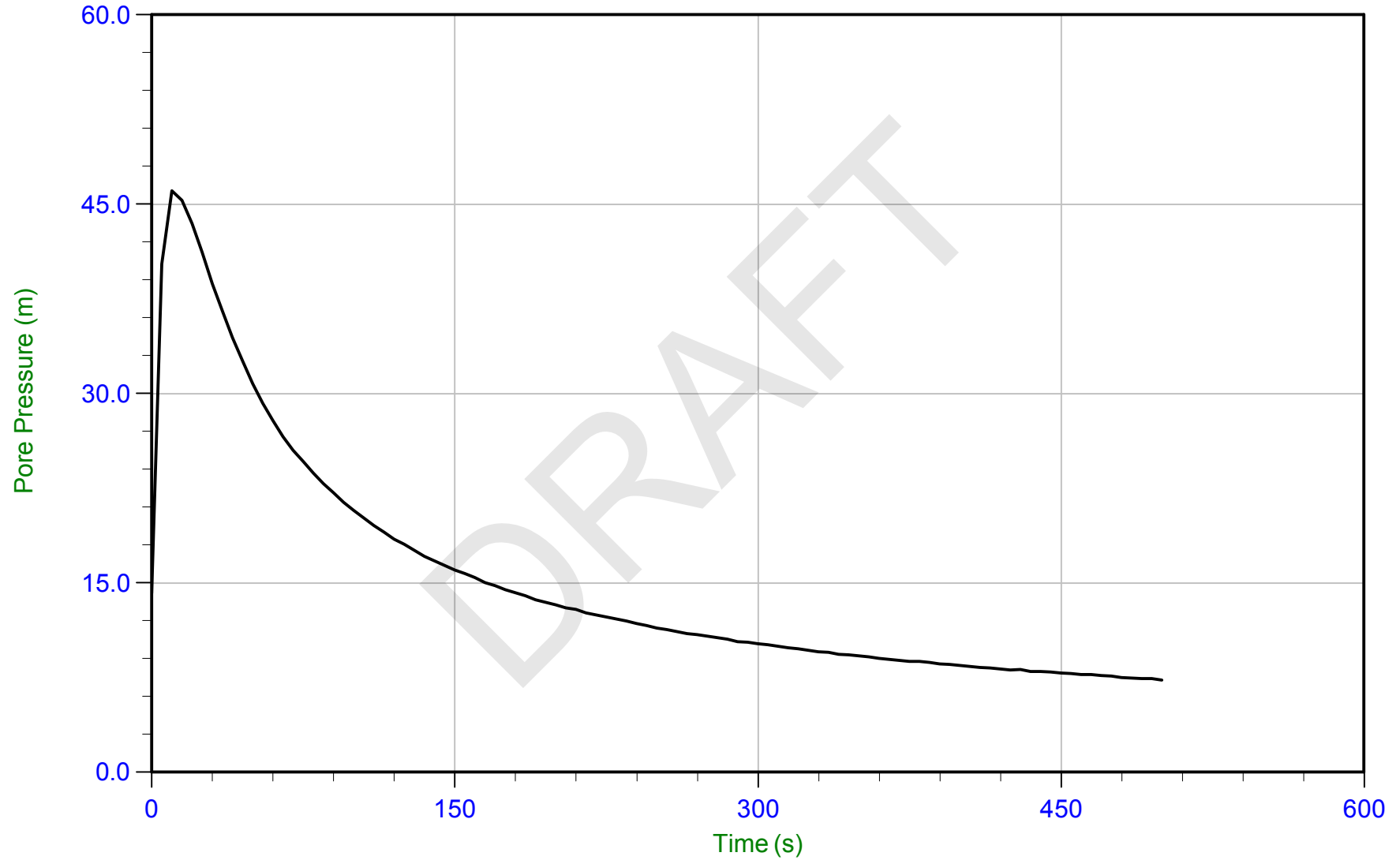
Sounding ID	File Name	Cone Area (cm <sup>2</sup> )	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U <sub>eq</sub> (m)	Calculated Phreatic Surface (m)
CPT16-06	16-05007_CP06	15	500	8.25	Not Achieved	
CPT16-06	16-05007_CP06	15	500	20.50	0.6	20.0
CPT16-06	16-05007_CP06	15	300	22.00	2.0	20.0
CPT16-06	16-05007_CP06	15	300	23.20	3.2	20.0
CPT16-07	16-05007_CP07	15	650	21.00	1.1	19.9
CPT16-07	16-05007_CP07	15	700	23.00	3.1	19.9
CPT16-08	16-05007_CP08	15	200	12.60	0.0	
CPT16-08	16-05007_CP08	15	475	16.70	3.1	13.6
CPT16-08	16-05007_CP08	15	950	17.00	Not Achieved	
CPT16-08	16-05007_CP08	15	430	19.00	Not Achieved	



*Stantec Consulting Ltd.*

Job No: 16-05007  
Date: 04/05/2016 12:39  
Site: V.M.P. Expansion

Sounding: CPT16-06  
Cone: 419:T1500F15U500  
Cone Area: 15 sq cm



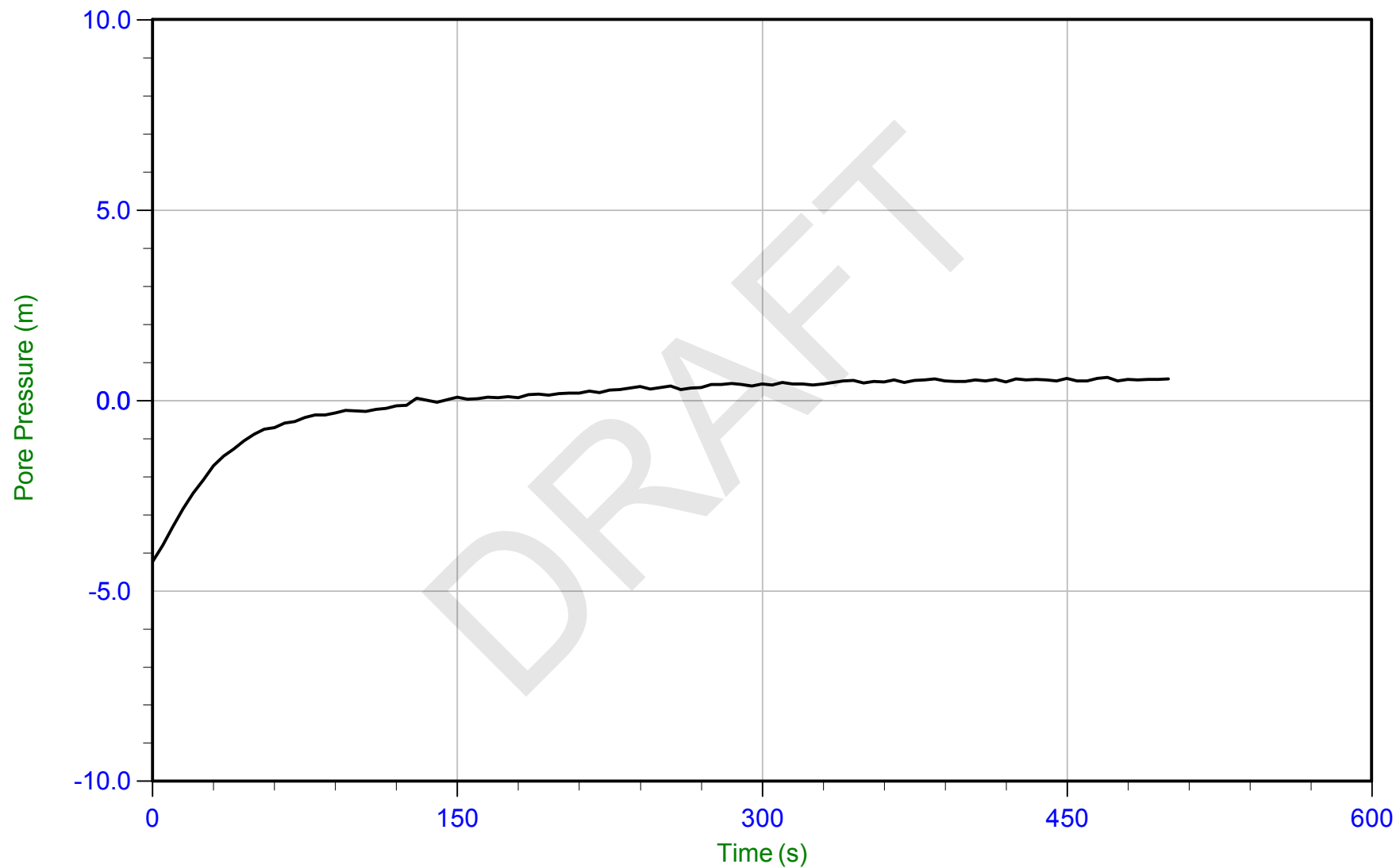
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Duration: 500.0 s  
U Min: 7.3 m  
U Max: 46.1 m



*Stantec Consulting Ltd.*

Job No: 16-05007  
Date: 04/05/2016 12:39  
Site: V.M.P. Expansion

Sounding: CPT16-06  
Cone: 419:T1500F15U500  
Cone Area: 15 sq cm



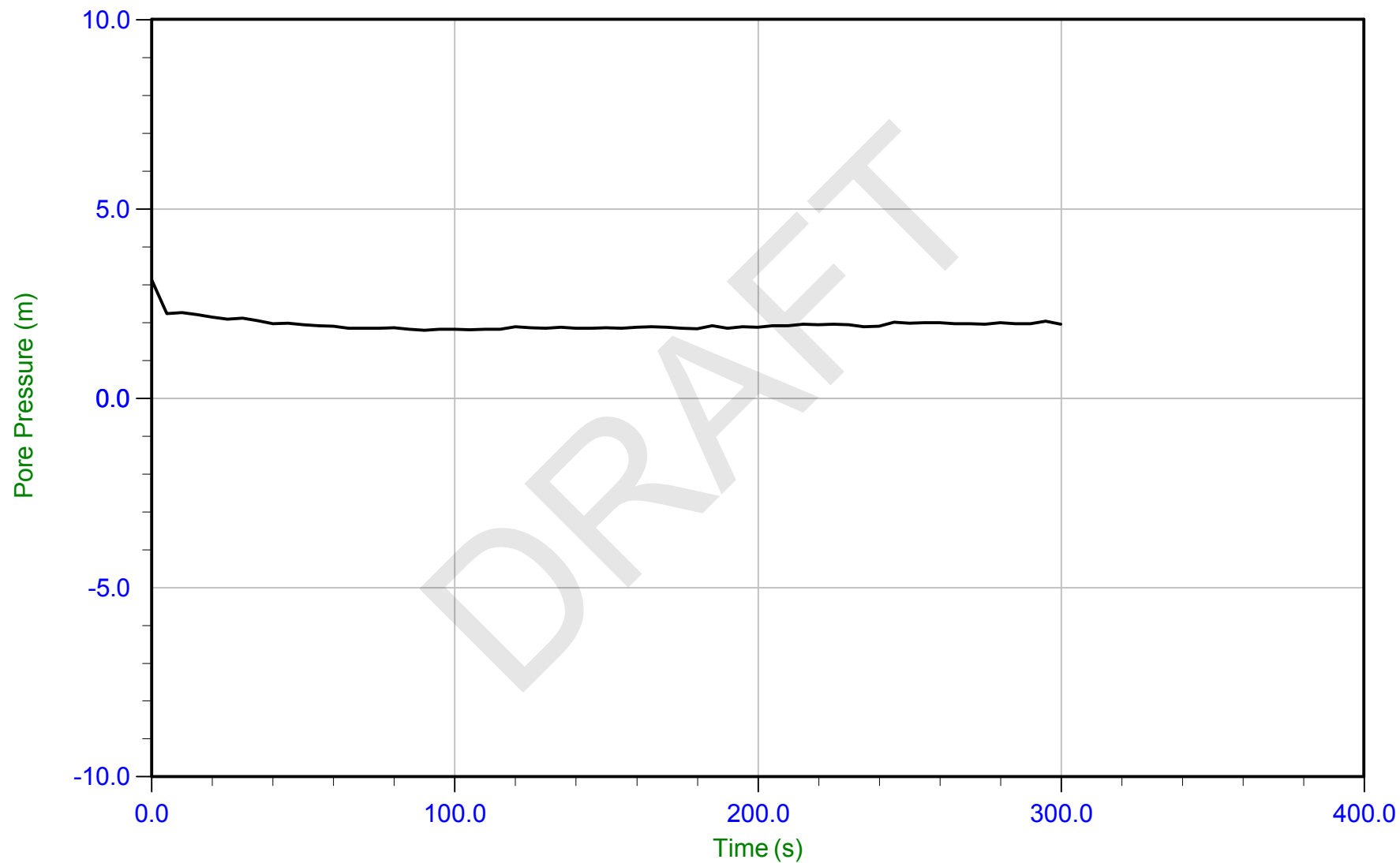
Trace Summary: Filename: 16-05007\_CP06.PPF      U Min: -4.2 m      WT: 19.949 m / 65.449 ft  
Depth: 20.500 m / 67.256 ft      U Max: 0.6 m      Ueq: 0.6 m  
Duration: 500.0 s



*Stantec Consulting Ltd.*

Job No: 16-05007  
Date: 04/05/2016 12:39  
Site: V.M.P. Expansion

Sounding: CPT16-06  
Cone: 419:T1500F15U500  
Cone Area: 15 sq cm



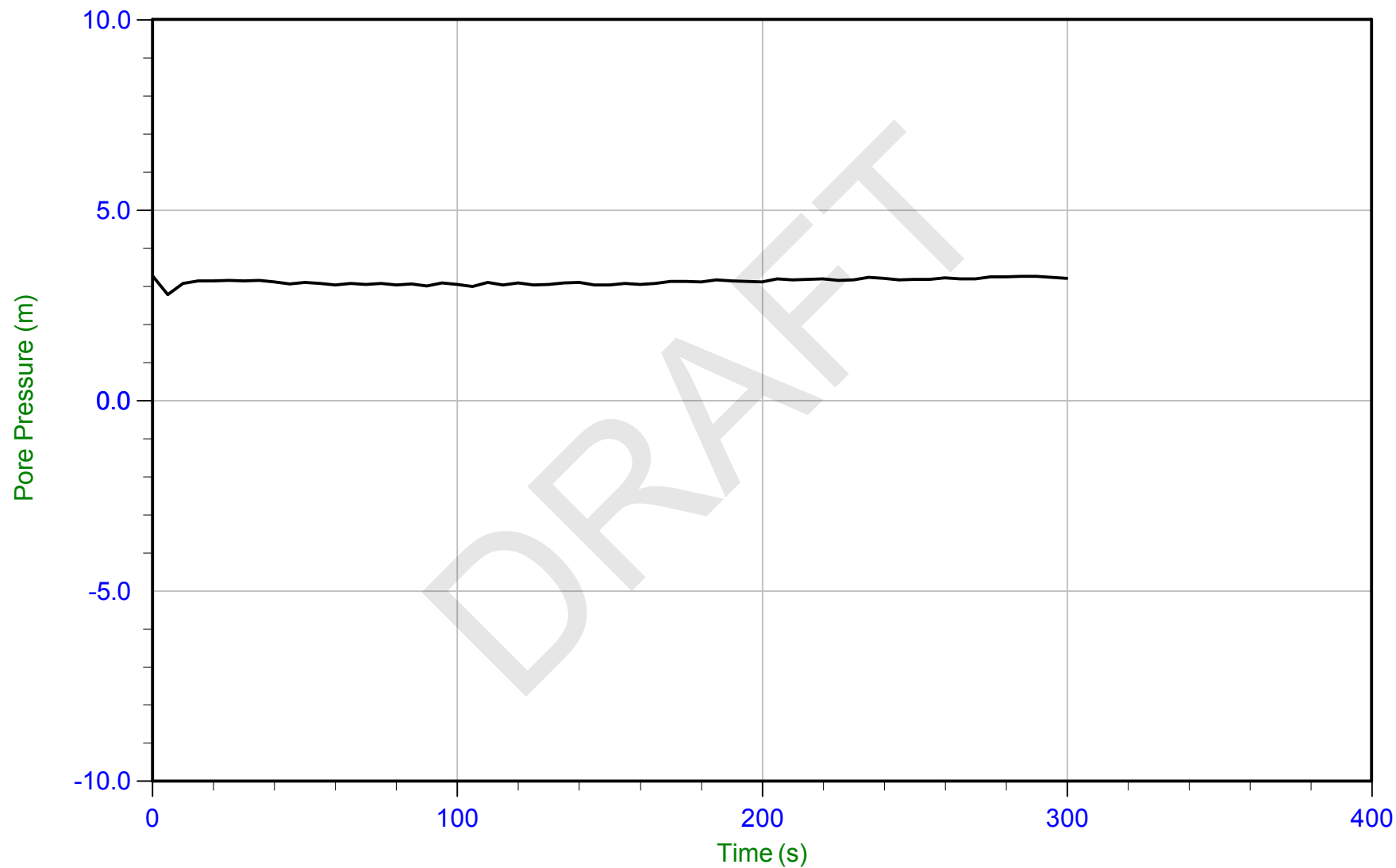
Trace Summary: Filename: 16-05007\_CP06.PPF      U Min: 1.8 m      WT: 20.008 m / 65.642 ft  
Depth: 22.000 m / 72.178 ft      U Max: 3.1 m      Ueq: 2.0 m  
Duration: 300.0 s



*Stantec Consulting Ltd.*

Job No: 16-05007  
Date: 04/05/2016 12:39  
Site: V.M.P. Expansion

Sounding: CPT16-06  
Cone: 419:T1500F15U500  
Cone Area: 15 sq cm



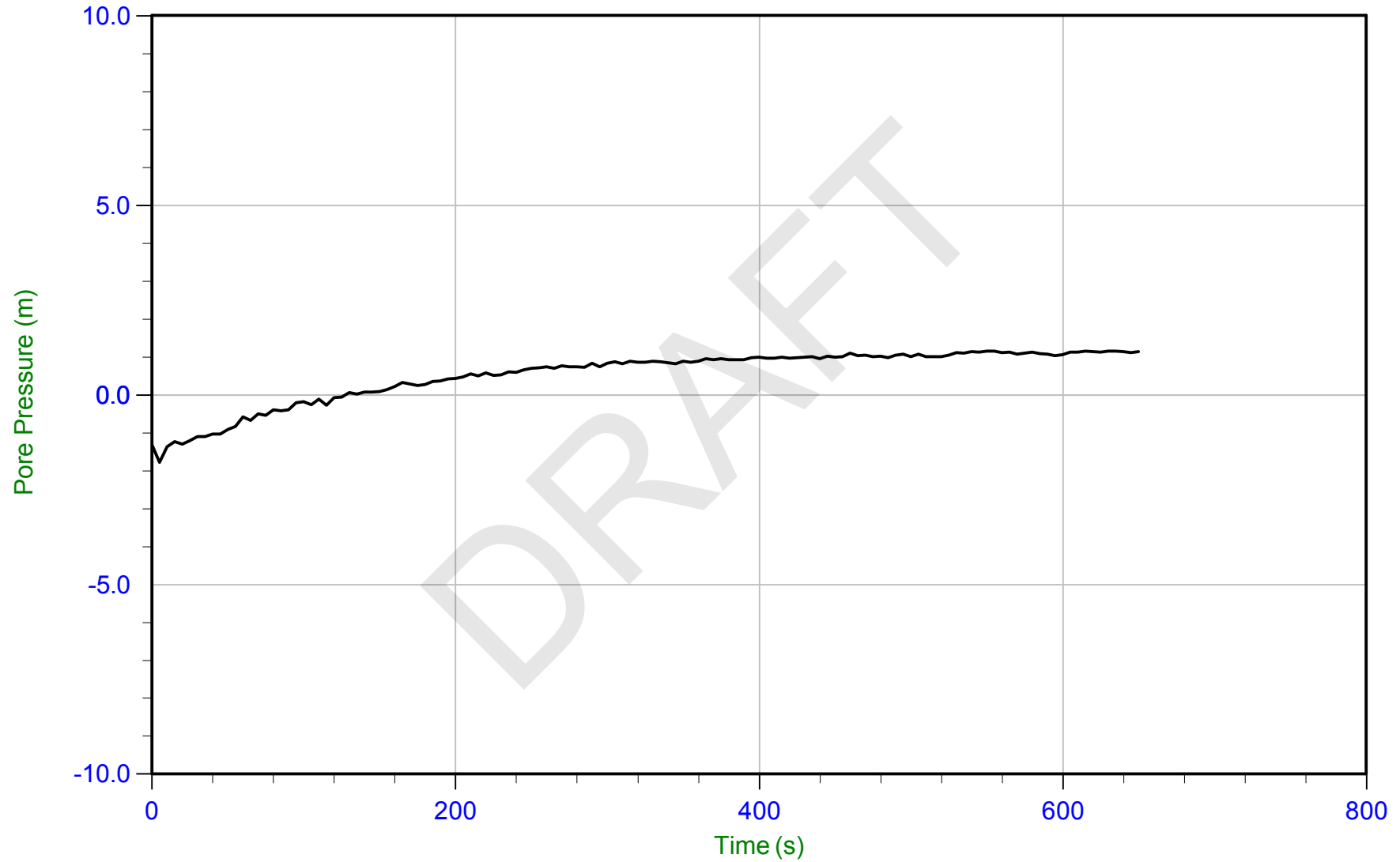
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	Depth: 23.200 m / 76.115 ft	U Max: 3.3 m	Ueq: 3.2 m
	Duration: 300.0 s		



*Stantec Consulting Ltd.*

Job No: 16-05007  
Date: 04/05/2016 14:55  
Site: V.M.P. Expansion

Sounding: CPT16-07  
Cone: 419:T1500F15U500  
Cone Area: 15 sq cm



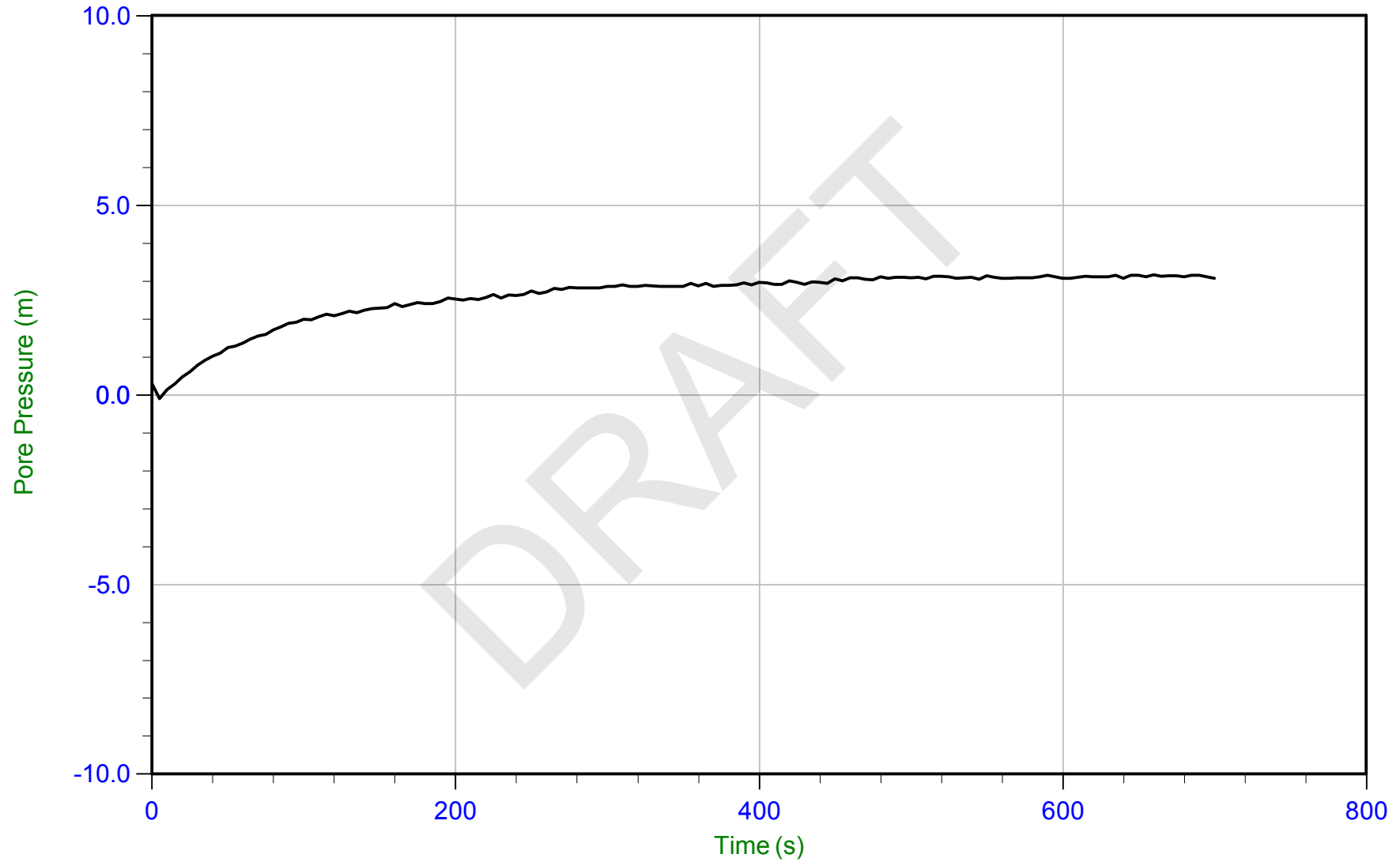
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	Depth: 21.000 m / 68.897 ft	U Max: 1.2 m	Ueq: 1.1 m
	Duration: 650.0 s		



*Stantec Consulting Ltd.*

Job No: 16-05007  
Date: 04/05/2016 14:55  
Site: V.M.P. Expansion

Sounding: CPT16-07  
Cone: 419:T1500F15U500  
Cone Area: 15 sq cm



Trace Summary: Filename: 16-05007\_CP07.PPF  
Depth: 23.000 m / 75.458 ft  
Duration: 700.0 s

U Min: -0.1 m  
U Max: 3.2 m

WT: 19.907 m / 65.311 ft  
Ueq: 3.1 m

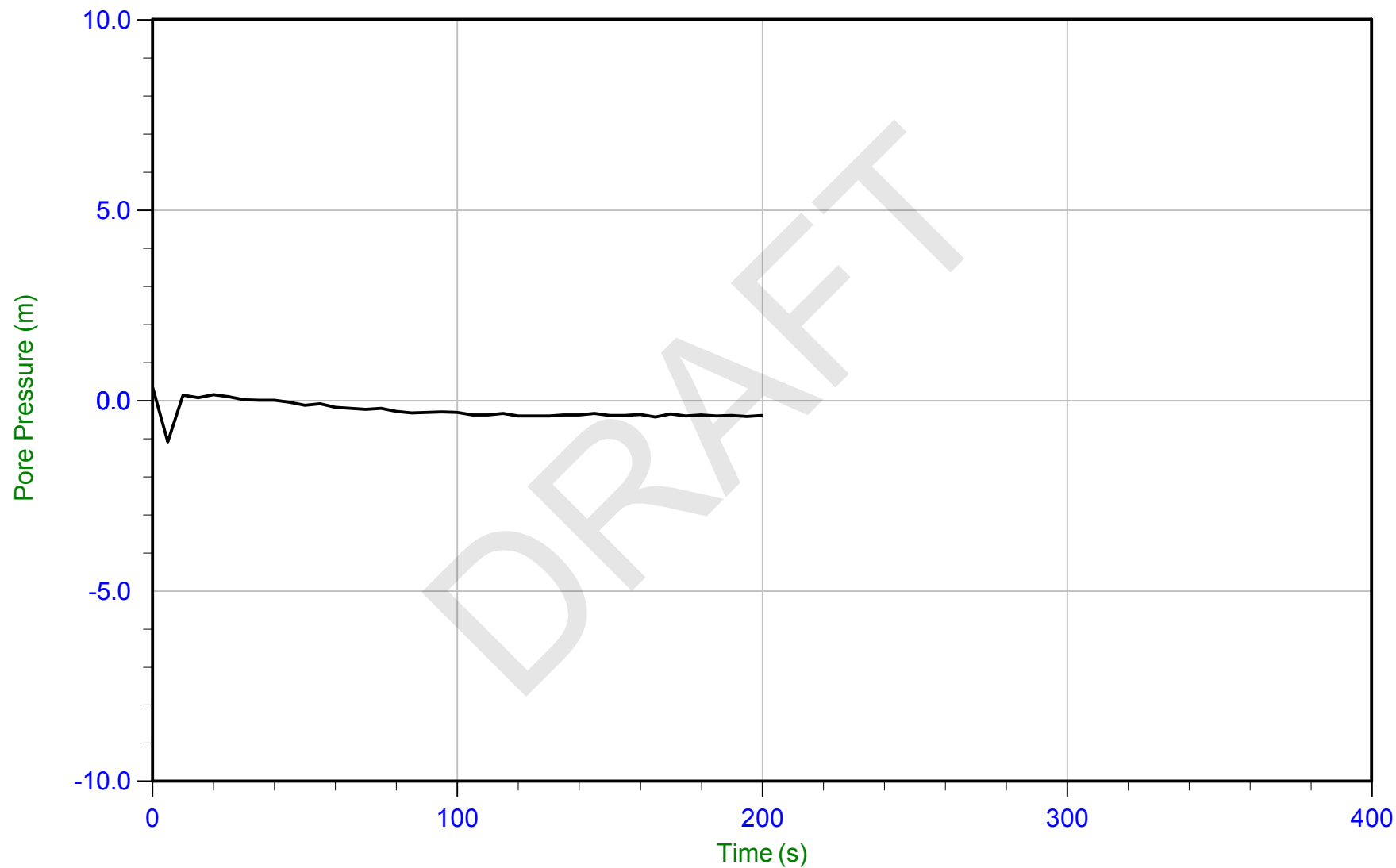




*Stantec Consulting Ltd.*

Job No: 16-05007  
Date: 04/05/2016 09:23  
Site: V.M.P. Expansion

Sounding: CPT16-08  
Cone: 419:T1500F15U500  
Cone Area: 15 sq cm



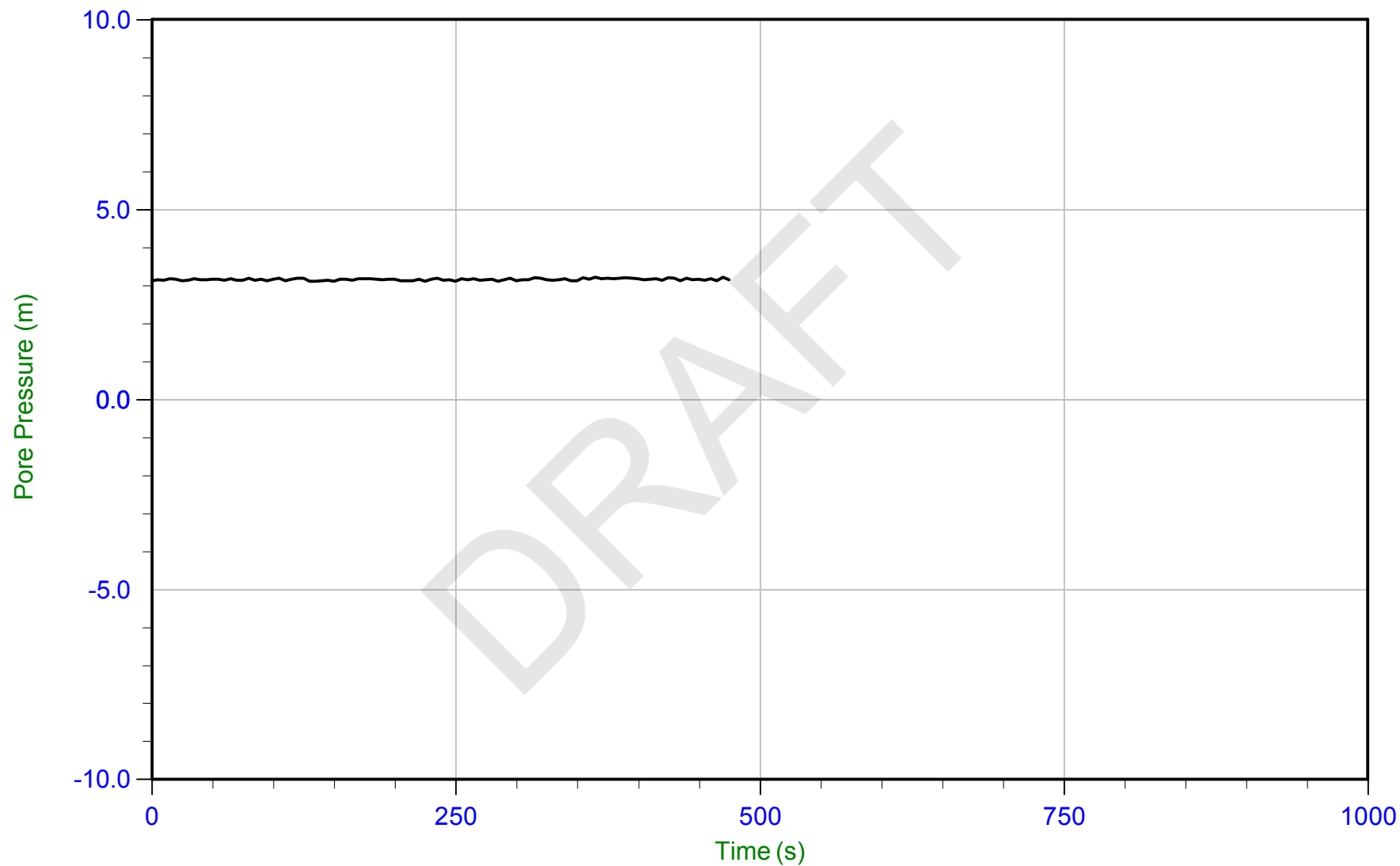
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	Depth: 12.600 m / 41.338 ft	U Max: 0.4 m	Ueq: 0.0 m
	Duration: 200.0 s		



*Stantec Consulting Ltd.*

Job No: 16-05007  
Date: 04/05/2016 09:23  
Site: V.M.P. Expansion

Sounding: CPT16-08  
Cone: 419:T1500F15U500  
Cone Area: 15 sq cm



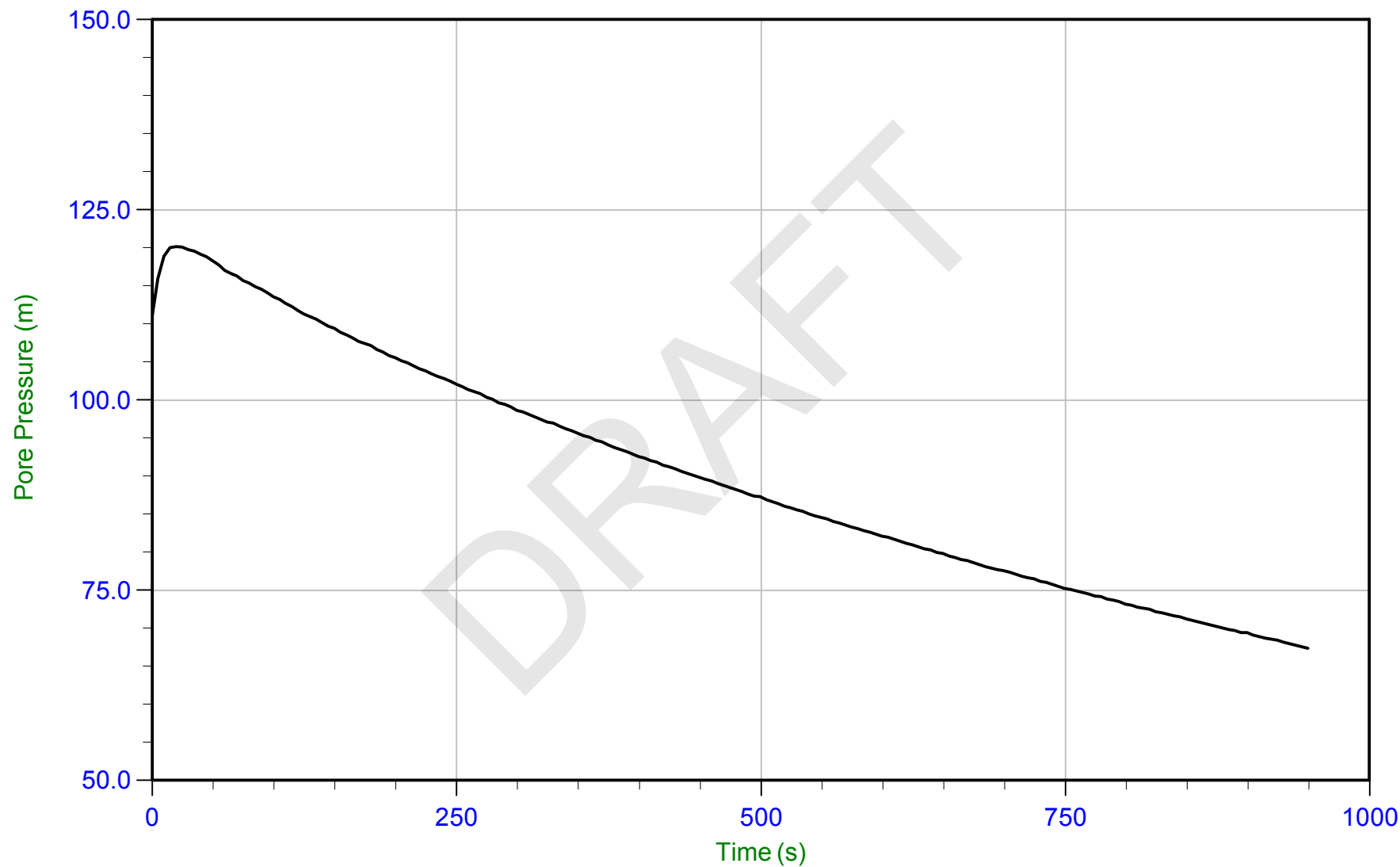
Trace Summary:	Filename: 16-05007_CP08.PPF	U Min: 3.1 m	WT: 13.564 m / 44.501 ft
	Depth: 16.700 m / 54.789 ft	U Max: 3.2 m	Ueq: 3.1 m
	Duration: 475.0 s		



*Stantec Consulting Ltd.*

Job No: 16-05007  
Date: 04/05/2016 09:23  
Site: V.M.P. Expansion

Sounding: CPT16-08  
Cone: 419:T1500F15U500  
Cone Area: 15 sq cm



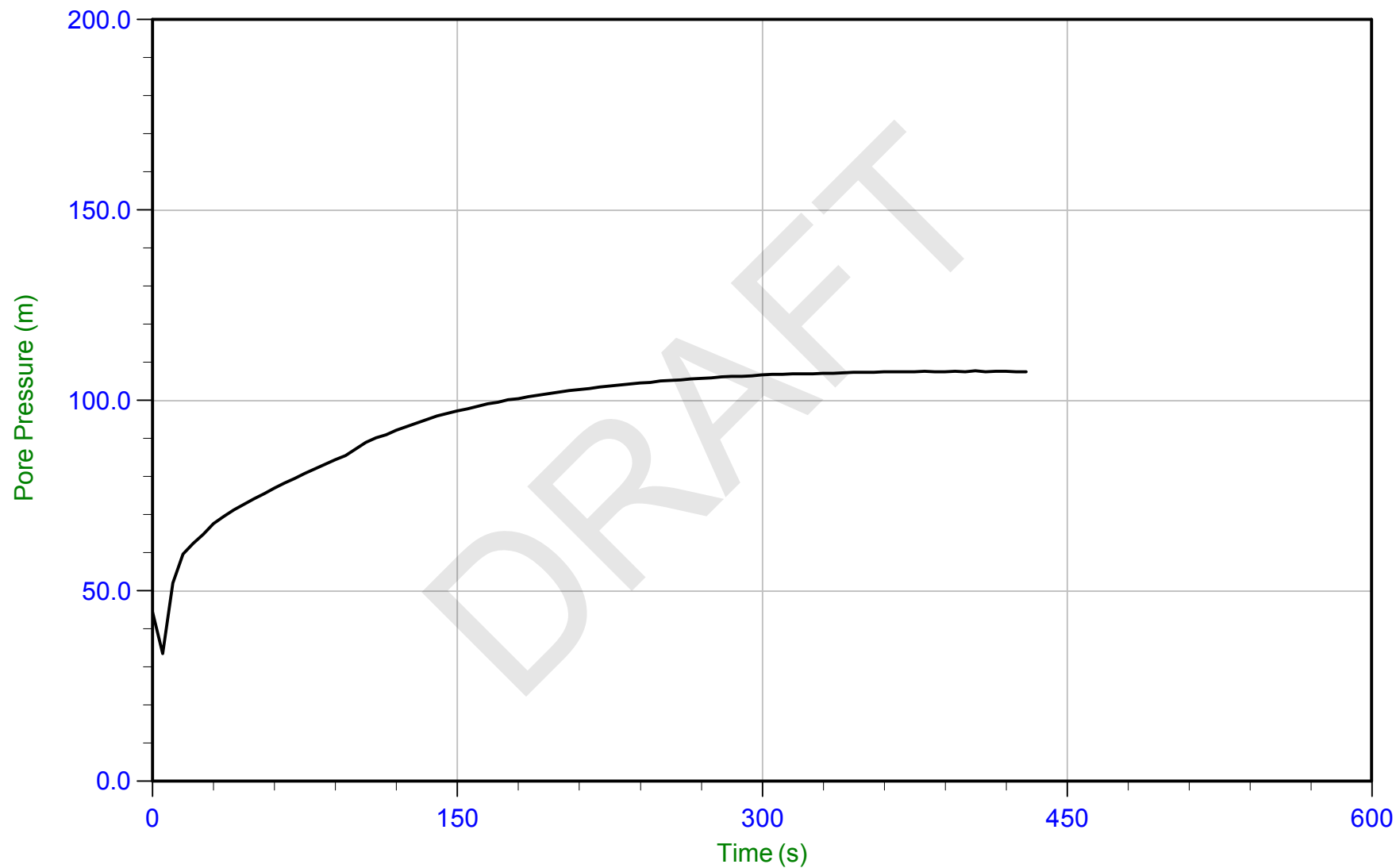
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Depth: 17.000 m / 55.774 ft  
Duration: 950.0 s  
U Min: 67.4 m  
U Max: 120.2 m



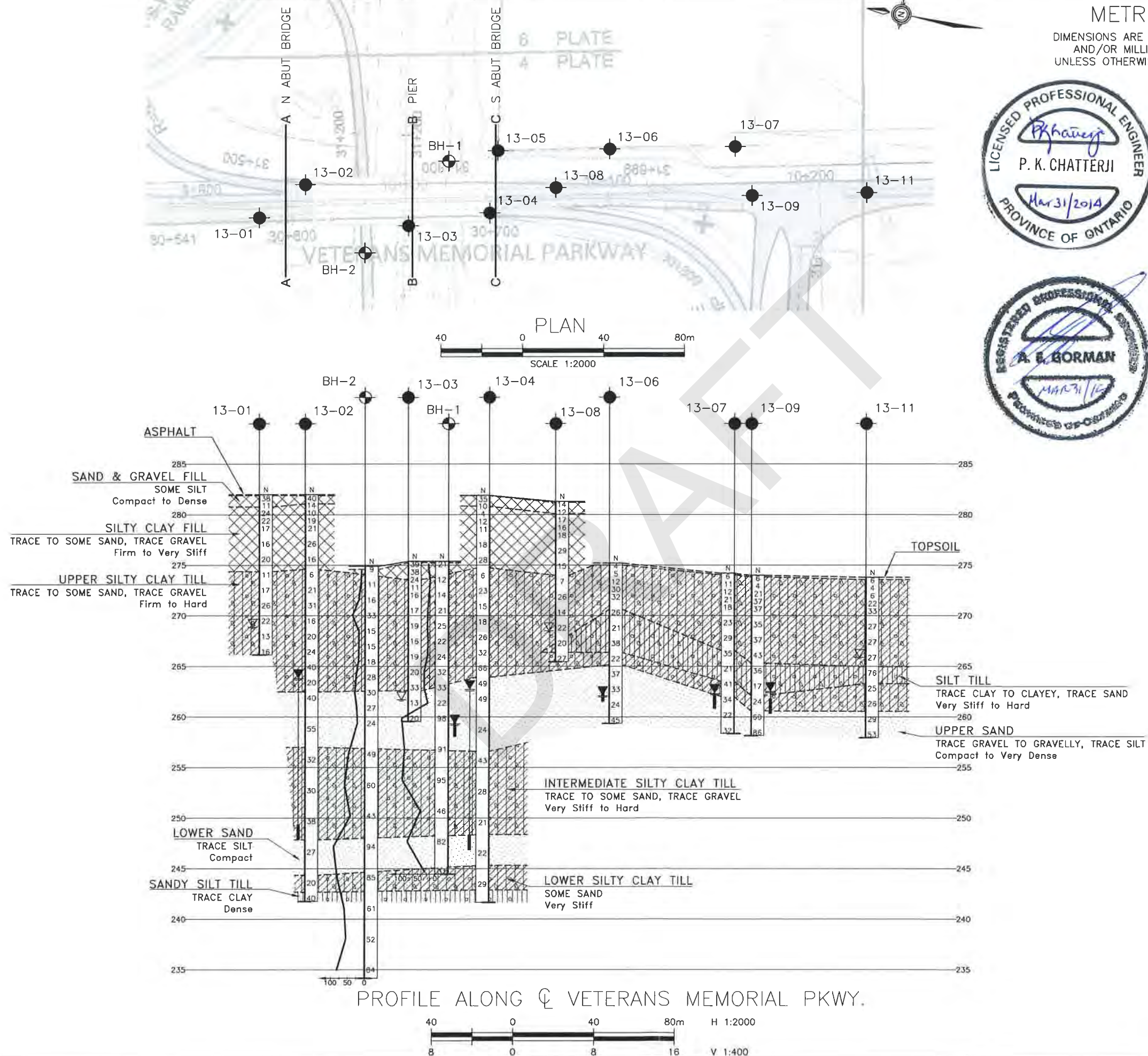
*Stantec Consulting Ltd.*

Job No: 16-05007  
Date: 04/05/2016 09:23  
Site: V.M.P. Expansion

Sounding: CPT16-08  
Cone: 419:T1500F15U500  
Cone Area: 15 sq cm



Trace Summary: Filename: 16-05007\_CP08.PPF U Min: 33.6 m  
Depth: 19.000 m / 62.335 ft U Max: 107.7 m  
Duration: 430.0 s

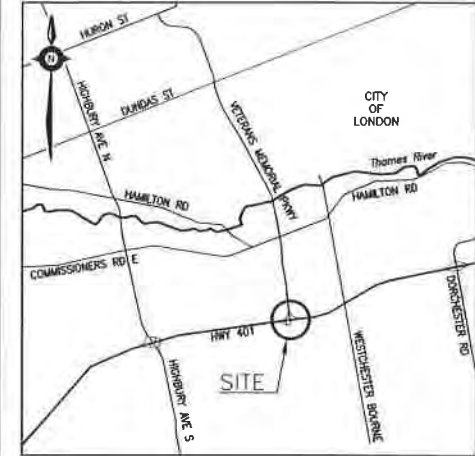


METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN



CONT No  
GWP No 3033-11-00

VETERANS MEMORIAL PARKWAY  
INTERCHANGE IMPROVEMENTS  
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

LEGEND

- Borehole (Current Investigation)
- Borehole (Previous Investigation)
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- W Water Level
- HA Head Artesian Water
- PZ Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
13-01	282.0	4 756 903.0	416 949.0
13-02	282.0	4 756 895.0	416 965.0
13-03	275.0	4 756 843.0	416 958.0
13-04	281.9	4 756 799.0	416 975.0
13-05	276.0	4 756 782.0	417 003.0
13-06	275.0	4 756 741.0	417 011.0
13-07	274.2	4 756 699.0	417 005.0
13-08	281.3	4 756 782.0	416 978.0
13-09	274.0	4 756 675.0	416 995.0
13-11	273.8	4 756 658.0	416 994.0
BH-1	275.4	4 756 823.0	416 987.0
BH-2	274.9	4 756 861.0	416 949.0

NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- BH-1 and BH-2 from Infrastructure Engineering Group shown for reference. Current sampling prevails for interpretation.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 40114-153

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	KMY	CHK KMY	CODE
DRAWN	AN	CHK AEG	SITE
			LOAD
			STRUCT
			DWG 1
			DATE MAR 2014




RECORD OF BOREHOLE No 13-01

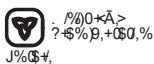
1 OF 2

METRIC

TI KI BIBBa@@a!! 3J8 Q?2JP =. K2Q+d\$%5(Ä0N- <Q@PÄFEÄBI CÄÄC@EÄGI JO 2R2R?L' Ä M RQ  
NT M C!@ \*J OLNJ 3LÄ?MKL N, 77-Ä; Q" ÄU5+ 8J . K2L'Ä \*M QP  
'Q? S. R(&0 /: 'Q? L A!@BÄCÄÄÄ!@BÄCÄB 8NL8VL'Ä \*M VM

;J 2ÄKOJ 123L			;Q.K3 L;			ROJSP 'ÄTQ ?LO 8JP' 2?2P'	L3L=Q?2JPÄ;80 3L MPQ 2 8ÄJPL ÄKLPL?Q?2J P OL; 2;?QP8 LÄKJ? 	K3Q?28 32 2' PQ?S0C3 J 2 ?SOL 8JP? LP? 3ZS2 32 2'			SP2? TL 2RN? y	OL.Q OV; ] ROQ2FÄ;2L '2; ?02 S?2J P Ä'#
L3L= L K?N	'L : 802K?2J P	?Q? ÄK3J?	PS: LO	?MKL	bPbÄ=Q3S			A! C! E! G! @!!	- K - 3	A! C! E!		
AGÄI	ASPHALT:ÄÄÄ " " #						AGA	A! C! E! G! @!!			[PÄ " B	RO :Q ;2 83
II I !! @	SANDÄ%8ÄRAVEL ' (%) *+, - % ./ )0 Ä123#		@	::	BG							
AG@Ä II G	SAND4ÄÄ;" (Ä556( 7ÄÄ)" (Ä)Ä 8," 0\$; 0 *+, - % ./ )0 Ä123#		A	::	@@		AG@					@F EG @D Ä;28 3#
AGII G @Ä	ÄÄÄCLAY4Ä0(ÄÄ0Ä)" (Ä)ÄÄÄÄ0(Ä 5+ÄÄ7 =( +ÄÄ;0Ä> *+, - % ./ )0 Ä123#		B	::	AC		AGI					! Ä! CD BD
			C	::	AA		AF					
			D	::	@F		AFC					
			E	::	@E		AFF					C @G CB BD
			F	::	ÄI		AFE					
							AFD					
AFCC FIE	ÄÄÄCLAY4ÄÄ)" (Ä)ÄÄÄÄ0(ÄÄ556( 7 ;0/;>>ÄÄÄ=(+ÄÄ;0Ä> *+, - % ./ )0 ÄÄ23#		G	::	@@		AFC					
							AFB					

JP? ? C:ÄÄECAFIRKÄÄÄ!@K?3Q?LÄ? J#IR? ÄÄB0@@C



## 2 OF 2

METRIC

TI KI	BIBBa@@a!!	3J8 Q?2IP	=. K29+&\$%5(A0N- <Q@P A0FDEÄBIÖÄÄ@EÄGI	JO 2R2R?L' Ä M	RQ
NT M	C!@	*J OLNJ 3LÄMKL	N, 77-Ä; Q" ÄU5(+	8J . K2L'Ä *M	QP
'Q? S.	R(&0 f:	'Q? L	ÄI@BÄÄÄÄÄÄI@BÄÄÄ	8NL8VL'Ä *M	VM

[illegible]

JP? ? C;ÄÆCAFIRKcÄÄA!@A?3Q?LÄ.? J#IR'? ÄÄBQ@C

+ B<sub>4</sub> × B<sub>W</sub> PU' X( +Äf > (Ä0,  
; (% /06/G



## METRIC

[illegible]

À # ? O Q P A ? A Q 2 SOL

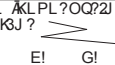


RECORD OF BOREHOLE No 13-02

2 OF 5

METRIC

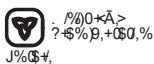
TI KI BIBBa@@a!! 3J8 Q?2JP =. K2Q+d\$%5(A0N- <Q@P A C A F E S D I B A A C @ E C I JO 2R2R?L' A M RQ  
NT M C!@ \*J OLNJ 3L?MKL N, 77-A; Q" AU5(+) 8J . K2L'A \*M QP  
'Q? S. R(&0 t: 'Q? L A!@B0A0A!@B0A0A 8NL8VL'A \*M VM

;J 2A KOJ 12BL			;Q.K3 L;			ROJSP 'ATQ ?LO 8JP' 2?2P';	L3L = Q?2JP A 8Q 3L	MPQ 2 8A8JPL AKLPL?OQ?2J P OL; 2;?QP8 LA1KJ ? 					K3Q?28 32 2' PQ?SOQB J 2 ?SOL 8JP? LP? 3Z S2 32 2'			SP2? TL 2RN? Y	OL.Q OV; ] ROQ2FA;2L ' 2;?O2'S?2JP A'#
L3L= L K?N	'L ; 8O2K?2J P	?OQ ?AK3J?	PS: LO	?MKL	bPbA=Q3B			A! C! E! G! @!!	- K - - 3								
	8.%0%U(8A+;" AK+(6/U) AK\$5( :/ 7A CLAY4A0+(SA0A)," (A)S%84A0+(S 5+\$87 =(+A;0?>A8N\$& *+, - % ./ )0 A?23#							!NLQOA;? OLPR?NA1K\$ o SP8JP1 2PL' + 12L3 A=QPL ● ZS28VA02Q?2Q3 x 3Q'A=QPL A! C! E! G! @!!									
			@! ::		B@		AFA										
							AF@										
							AF!										
			@@ ::		@E		AE										
			@A ::		A!		AEG										
							AEP										
			@B ::		AC		AEE										
			@C ::		C!		AED										
							AEC										
			@D ::		A!		AEB										
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@ D	SAND4A0+(SA)D4A0+(S56(7 8," 9S: 0A0A=(+A' ( %( R+(<																

JP? ? C;AECFAFIRKcAA@K'3Q?LA? J#R? AAB00C

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+ B4x B W PU' X( A+>+A0,  
; (%/060;

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@!



## 3 OF 5

METRIC

TI KI	BIBBa@@a!!	3J8 Q?2IP	=. K2Pq+d\$%\$&0N- <Q@P0CfDE5DIBAAAC@EACI	JO 2R2R?L' Å M	RQ
NT M	C!@	*J OLNJ 3LPMKL	N, 77Å; " ÅU5(+	8J . K3L'Å *M	QP
'Q? S.	R(&0 f:	'Q? L	A!@B0A0A!@B0A0A	8NL8VL'Å *M	VM

;J 23AKOJ 123L			;Q.K3 L;			;MPQ.2 8A6JPL AKL PL?OQ?2J P OL; 2;7 QP8 LA6J?			K3Q.228 32 2' PQ?SOQ8 32'S2 32 2' J 2 7SOL 8JP? LP?			OL.Q OV; ] ROQ2FA;2L ' 2;? O2*S?2JP Ä#				
L3L= L K?N	'L ; 8O2K?2J P		?OQ 7AK3J?	PS* LO	?MKL	bPbA=Q3S	ROJSP ÄTQ 7LO 8JP' 272P'	L3L = Q22JPÄ;8Q3L	A! C! E! G! @!! ;NLQOÄ? OLPR?NÄK\$ ○ SP8JP1 2PL' + 12L3 Ä=QPL ● ZS28VÄD2Q?2Q3 × 3Q*Ä=QPL A! C! E! G! @!!			K - - - 3 T Q?LOÄBJP?LP?Ä Ä # A! C! E!			SP2? TL 2RN?	OL.Q OV; ] ROQ2FA;2L ' 2;? O2*S?2JP Ä#
	8.%0%U(8Ä+." ÄK+(8/U) ÄK\$5( SAND4Ä0+(SÄ)D4Ä0+Ä\$56( 7 8, " 9S: 0ÄÄ=(+Ä' ( %( R+(< T( 0			@E	::	CI		AEA								! D D Ä;283#
								AE@								
								AEI								
				@F	::	DD		AD								
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ADF@ ADI	:/ DÄCLAY4Ä0+(\$Ä0Ä)," (Ä)8/84Ä0+(\$ 5+\$67 NS+8 R+(< T( 0 Ä?23#							ADF								
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				@	::	BI		ADE								! ! CD DD

8,%0%U(8FA(YOK\$5(

+ B<sub>4</sub> X B<sub>W</sub> PU' X (+A†) > (+A0,  
; (%/06/G

$\begin{array}{c} A! \\ @B \bullet D \\ @! \end{array}$ 
 $\ddot{A} \# \ddot{A} ? O \ddot{A} ? \ddot{A} O \ddot{A} SOL$

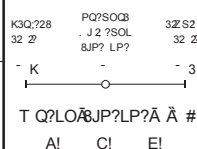
JP? ? C;ÄÆCAFIRKcÄÄA!@A?3Q?LÄ.? J#IR'? ÄÄBQ@C

RECORD OF BOREHOLE No 13-02

4 OF 5

METRIC

TI KI B1BBa@@a!! 3J8 Q?2JP =. K2Q+d\$%5(00N- <0@PACFDESDIBALAC@EACI JO 2R2R?L' A M RQ  
NT M C!@ \*J OLNJ 3LÄ?MKL N.77-Ä; Q" ÄU5+ 8J . K2L'Ä \*M QP  
'Q? S. R(&0 t: 'Q? L A!@BDA@ÄA!@BDA@ 8NL8VL'Ä \*M VM

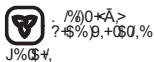
;J 2ÄKOJ 123L			;Q.K3 L;			MPQ 2 8ÄJPL AKLPL?OQ?2J P OL; 2;?QP8LÄKJ? 			OL.Q OV; ] ROQ2Ä;2L ' 2;?O2*S?2JP Ä#		
L3L= L K?N	L : 8O2K?2J P	?OQ ?ÄK3J?	PS: LO	?MKL	bPbÄ=Q3S	ROJSP 'ÄTQ ?LO 8JP' 2?2P'	L3L=Q?2JPÄ8O3L	A! C! E! G! @!!	SP2? TL 2RN?	RO :Q ;2 83	
	8,%0%U(8Ä+;" ÄK+(G/U) ÄK\$5( :/ DÄCLAY4Ä0(Ä\$Ä0Ä)," (Ä)8Ä4Ä0(Ä\$ 5+\$87 N\$+8 R+(< T( 0 Ä?23#						ADA				
							AD@				
			A!	::	BG		AD!				
							AC				
ACFI							ACC				
BC@	SAND4Ä0(Ä\$Ä)Ä 8," 9S: 0 R+(< T( 0						ACF				
			A@	::	AF		ACE				
							ACD				
ACOC							ACC				
BFE	:/ DÄCLAY4Ä)," (Ä)8Ä =( +Ä,0#> R+(< T( 0 Ä?23#		AA	::	A!		ACE				
ACAF											
B1B	SILT4Ä)," (Ä)8Ä4Ä0(Ä\$Ä:8< ' (%)( R+(< T( 0		AB	::	C!						

8,%0%U(8Ä(Y0K\$5(

+ B4×B W PU' X(Ä+>(Ä0,  
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A!  
@B D Ä #Ä ?OQPAQ?Ä Q2SOL  
@!

JP? ? C:ÄECAFIRKcÄÄA!@K?3Q?LÄ? J#R? ÄÄB@C



## 5 OF 5

METRIC

TI KI	BIBBa@@a!!	3J8 Q?2IP	=. K29+&\$%5(A0N- <Q@P0CfDE5DIBAA@EACI	JO 2R2R?L' Å M	RQ
NT M	C!@	*J OLNJ 3LAÏMKL	N, 77-Ä; Q" ÅU5(+	8J . K2L'Å *M	QP
'Q? S.	R(&0 f:	'Q? L	AI@BDAÄÄÄI@BDAÄÄ	8NL8VL'Å *M	VM

[illegible]

JP? ? C;ÄÆCAFIRKcÄÄA!@Ä?3Q?LÄ.? J#IR'? ÄÄBQ@C

+ B<sub>4</sub> × B<sub>W</sub> PU' X( +Ä† > (Ä0,  
; (% /06/G



## METRIC

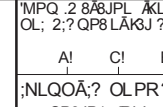
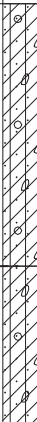




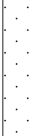

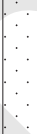

A!  
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 @!  
 Ä #Ä ?002PÄQ?Ä Q2SOL

RECORD OF BOREHOLE No 13-03

2 OF 2

METRIC

TI KI BIBBa@@a!! 3J8 Q?2JP =. K2Q +d\$%5(A0N- <Q@P A C A F E A C B D A A C @ E A F I D JO 2R2R?L' A M RQ  
NT M C!@ \*J OLNJ 3L A?MKL N.77-A; Q" A U5 + 8J . K2L'A \*M QP  
'Q? S. R(&0 t: 'Q? L A!@B D A C A A!@B D A C 8NL8VL'A \*M VM

;J 23AKOJ 123L			;Q.K3 L;			MPQ 2 8AJPL AKLPL?OQ?2J P OL; 2;?QP8LAKJ? 			K3Q?28 32 2' PQ?S0B J 2 ?SOL 8JP? LP? 3Z S2 32 2'			SP2? TL 2RN? 7	OL.Q OV; ] ROQ2FA;2L ' 2;?O2"S?2JP A#
L3L= L K?N	'L : 8O2K?2J P	:?OQ ?AK3J?	PS: LO	?MKL	bPbA=Q3B	ROJSP 'ATQ ?LO 8JP' 2?2P'	L3L = Q?2JP A;8O 3L	A! C! E! G! @!! ;NLQO A;? OLPR?N A K\$ o SP8JP1 2PL' + 12L3 A=CPL ● Z S28V A D2Q?2Q3 x 3Q'A=CPL A! C! E! G! @!!	- K - 3 T Q?LO AJP?LP?A A # A! C! E!	[P\ " B	RO ;Q ;2 83		
	8,%0%U(&A+" AK+(6/U) AK\$5( :/ D A CLAY4A0+(S A0A)," (A)S&4A0+(S 5+\$87 =(+A;0+> *+, - % ./ )0 A?23#		@!	::	A!		AED				! E DD B		
AEBF @@	NS+&						AEC						
AEAE @A			@@	::	BB		AEE						
	SAND4A0+(S A)D 8," 9\$: 0 R+(< T( 0		@A	::	@B		AEA				! D D A;2+3#		
AD IE @D			@B	::	A!		AEI						
	LP' AJ1A*J OLNJ 3L A Q A @ C A "I *J OLNJ 3L A J K L P A ? A @ C A "A Q P TQ ?L O A L=L 3 A Q ? A @ C A I *J OLNJ 3L A *Q8V123L' AT2?N *L P?J P2L A N I 3L K3SR A ? A I B A "4 8JP8OL ?L A J A I A A 4A ?N P Q; KNQ3?AK Q8 N A ? A;S O1Q8L I												

JP? ? C;A E C A F I R K C A A A @ K' 3 Q ? L A ? J # I R ? A A B D @ C



## 1 OF 5

METRIC

L3L= L K?N		;J 23AKOJ 123L		;Q.K3 L;		ROJSP ATQ ?LO 8JP 272P;		L3L = Q72JPÄ8Q3L		MPO. 2 8A6JPL AKLPL?OQ?2J P OL; 2;?QP8 LÄKJ ?		K3Q?28 32 2? PQ?SOQ3 J 2 ?SOL 8JP? LP? 32 S2 32 2?		SP2? TL 2RN?		OL.Q OV; ] ROQ2FÄ;2L ' 2;?O2*S?22JF Ä#	
AG@I		"L ; 8O2K?2J P		?Q?ÄKJ?		PS* LO ?MKL bPÄ=Q3S		L3L = Q72JPÄ8Q3L		AI CI EI GI @!!		- K - 3		7		RO ;Q ;2 8:	
AG@I		ASPHALT:ÄADÄ" #		X		@ :: BD		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		SAND\$%8GRAVEL4Ä)" (Ä)Ä		X		A :: @!		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		; / DÄCLAY4Ä)" (Ä)S\$24ÄOÄ\$Ä556( 7 1/4" ÄOÄ;0Ä>		X		A :: @!		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		*+, - %		X		B :: C		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
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AGIIG		Ä123#		X		D :: @@		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		; / DÄCLAY4Ä)" (Ä)S\$24ÄOÄ\$Ä556( 7 1/4" ÄOÄ;0Ä>		X		E :: @G		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		*+, - %		X		F :: AG		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		.. / ) 0		X		G :: E		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		Ä123#		X		H :: @H		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		; / DÄCLAY4Ä)" (Ä)S\$24ÄOÄ\$Ä556( 7 1/4" ÄOÄ;0Ä>		X		I :: @I		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		*+, - %		X		J :: @J		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		.. / ) 0		X		K :: @K		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		Ä123#		X		L :: @L		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		; / DÄCLAY4Ä)" (Ä)S\$24ÄOÄ\$Ä556( 7 1/4" ÄOÄ;0Ä>		X		M :: @M		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		*+, - %		X		N :: @N		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		.. / ) 0		X		O :: @O		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		Ä123#		X		P :: @P		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		; / DÄCLAY4Ä)" (Ä)S\$24ÄOÄ\$Ä556( 7 1/4" ÄOÄ;0Ä>		X		Q :: @Q		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		*+, - %		X		R :: @R		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		.. / ) 0		X		S :: @S		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		Ä123#		X		T :: @T		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		; / DÄCLAY4Ä)" (Ä)S\$24ÄOÄ\$Ä556( 7 1/4" ÄOÄ;0Ä>		X		U :: @U		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		*+, - %		X		V :: @V		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		.. / ) 0		X		W :: @W		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		Ä123#		X		X :: @X		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		; / DÄCLAY4Ä)" (Ä)S\$24ÄOÄ\$Ä556( 7 1/4" ÄOÄ;0Ä>		X		Y :: @Y		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
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AGIIG		.. / ) 0		X		AA :: @AA		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		Ä123#		X		AB :: @AB		AG@		AI CI EI GI @!!		AI CI EI		[P\ " B		CC CC @Ä;28:	
AGIIG		; / DÄCLAY4Ä)" (Ä)S\$24ÄOÄ\$Ä556( 7 1/4" ÄOÄ;0Ä>		X		AC :: @AC		AG@		AI CI EI GI @!!		AI CI EI					

JP? ? C;ÄÆCAFIRKcÄÄA!@A?3Q?LÄ.? J#IR? ÄÄBQ@@C

8,%0%U(8FA(YOK\$5(

+ B<sub>4</sub> X B<sub>W</sub> PU' X (+A†) > (+A0,  
; (%/06/G

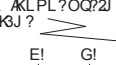
$\begin{array}{c} A! \\ @B \bullet D \\ @! \end{array}$ 
 $\ddot{A} \# \ddot{A} ? O \ddot{A} ? \ddot{A} O \ddot{A} SOL$

RECORD OF BOREHOLE No 13-04

2 OF 5

METRIC

TI KI B1BBa@@a!! 3J8 Q?2JP =. K2Q+d\$%5(00N- <Q@PACFEA I DA AC@EADIB JO 2R2R?L' A M RQ  
NT M C!@ \*J OLNJ 3LA?MKL N.77-A; Q" AU5(+) 8J . K2L'A \*M QP  
'Q? S. R(&0 t: 'Q? L A!@BDAIEA\@BDA@ 8NL8VL'A \*M VM

;J 2AKOJ 12BL			;Q.K3 L;			ROJSP 'ATQ ?LO 8JP' 2?2P'	L3L = Q?2JPÄ:8O 3L	MPQ 2 8A8JPL AKLPL?OQ?2J P OL; 2;?QP8 LÄKJ? 	K3Q:228 32 2'	PQ?SOB J 2 ?SOL 8JP? LP?	3ZS2 32 2'	SP2? TL 2RN?	OL.Q OV; ] ROQ2FA;2L ' 2;?O2*S?2JP Ä'#
L3L= L K?N	'L : 8O2K?2J P	?OQ ?AK3J?	PS: LO	?MKL	bPbÄ=Q3B			A! C! E! G! @!! ;NLQOÄ? OLPR?NÄK\$ ○ SP8JP1 2PL' + 12L3 Ä=QPL ● ZS28VÄD2Q?2Q3 × 3Q'A=QPL A! C! E! G! @!!	- K - 3				
	8,%0%U(&A+ " AK+(G/U) AK\$5( :/ D-CLAY4Ä)" (Ä)\$%&4Ä0+Ä\$56(7 :0/>> R+(< \$ " 9 Ä?23#												
AFIIA @@	=( +Ä,0+>		@!	::	@D		AF@						
							AFI						
			@@	::	@G		AE						
							AEC						
AEF@ @@	N\$-8		@A	::	AE		AEP						
			@B	::	BA		AEB						
			@C	::	EE		AED						
AEBI @GI	SAND4Ä0+SA)D4Ä0+Ä\$56( 7 ' (% (ÄQÄ=(+Ä') (%( R+(< ./ )0		@D	::	C		AEC						
							AEB						
							AFA						

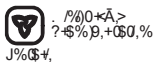
JP? ? C:ÄECAFIRKcÄÄI@K'3Q?LÄ? J#IR? ÄÄB@C

8,%0%U(8Ä)Y0K\$5(

+ B4× B W PU' X( +Ä+>(+Ä0,  
; (%/06/0

A!  
@B D Ä #Ä ?OQPÄQ?Ä Q2SOL  
@!





## 3 OF 5

METRIC

TI KI	BIBBa@@a!!	3J8 Q?2IP	=. K2Pq+d\$%\$&0N- <Q@P A C F E A I D A A C @ F A D I B	JO 2R2R?L' Ä M	RQ
NT M	C!@	*J OLNJ 3LÄMKL	N, 77Ä; Q" ÄU5(+	8J . K3L'Ä *M	QP
'Q? S.	R(&0 f:	'Q? L	A!@B@AIEÄ!@B@A@	8NL8VL'Ä *M	VM

;J 23AKOJ 123L					
L3L= L K7N	"L ; 80ZK?2I P  8,%0%U(8A:" AK+(6/U) AK\$5( SAND4Ä0:(SA)D4Ä0+8\$56( 7 ' (% ( Ä0Ä=(+Ä' (% ( R+(< ./ )0	?Q?ÄK3J?	PS.* LO ? MKL	bPbÄ=Q3S	ROJSP ÄTQ ?LO 8JP 222P:
AEIIB A@E	8," 9S; 0	@E ::	C		
ADEE ADB	/ DÄCLAY4Ä0+(SA0Ä)," (Ä)8?&4Ä0+\$ 5+\$67 =( +Ä;0>>ÄÄ1\$-8 R+(< ./ )0 Ä?23#	@F ::	AC		
		@G ::	CB		
		@	::	AG	

JP? ? C;ÄÆCAFIRKcÄÄA!@A?3Q?LÄ.? J#IR'? ÄÄBQ@C

8,%0%U(8FA(YOK\$5(

+ B<sub>4</sub> X B<sub>W</sub> PU' X (+) A (+) A<sub>0</sub>,  
; (%/06/0

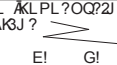
$\begin{array}{c} A! \\ @B \bullet D \\ @! \end{array}$ 
 $\ddot{A} \# \ddot{A} ? O \ddot{A} ? \ddot{A} O \ddot{A} SOL$

RECORD OF BOREHOLE No 13-04

4 OF 5

METRIC

TI KI B1BBa@@a!! 3J8 Q?2JP =. K2Q+d\$%5(30N- <Q@PACFEA I DA AC@EADIB JO 2R2R?L' A M RQ  
NT M C!@ \*J OLNJ 3L?MKL N.77-A; Q" AU5+ 8J . K2L'A \*M QP  
'Q? S. R(&0 t: 'Q? L A!@BDAIEAA!@BDA@ 8NL8VL'A \*M VM

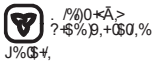
;J 23AKOJ 123L			;Q.K3 L;			MPQ 2 8A8JPL AKLPL?OQ?2J P OL; 2;?QP8 LAKJ? 			K3Q?28 32 2P PQ?SOB J 2 ?SOL 8JP? LP? 3ZS2 32 2P			SP2? TL 2RN? 7			OL.Q OV; ] ROQ2FA;2L ' 2;?O2*S?2JP A'#		
L3L= L K?N	L : 8O2K?2J P	?OQ ?AK3J?	PS: LO	?MKL	bPbA=Q3S	ROJSP 'ATQ ?LO 8JP' 2?2P'	L3L=Q?2JP?8Q3L	A! C! E! G! @!!	- K - 3	T Q?LO8JP?LP?A A #	A! C! E!	[P" B	RO :Q :2 83				
	8,%0%U(&A+ " AK+(6,U) AK\$5( :/ D#CLAY4A)," (A)\$%84AO{SA\$56(7 =( +<A;0>A0(\$-8 R+(< T( 0 A?23#																
ACGC	BBD SAND4AO{SA)D 8," 9S: 0 R+(< T( 0		A!	::	A@												
ACDB	BEE ;/ D#CLAY4A)," (A)\$%& =( +<A;0> R+(< T( 0 A?23#		A@	::	AA												
ACAI	B!! : \$%&A\$ILT4AO{SA:\$< ' (%)( R+(< T( 0 A?23#		AA	::	A												

JP?.? C:AAECAFIRKcAA@K?3Q?LA.? J#IR? AAB@C

8,%0%U(&A(YOK\$5(

+ B4x B W PU' X!+A+>+A0,  
; (%/060

A!  
@B D A #A ?OQPAD?A Q2SOL  
@!



5 OF 5

METRIC

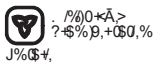
TI KI	BIBBa@Qa!!	3J8 Q?2IP	=. K22q +d5%5(50N- <Q@PFAFEA I DA AC@EADIB	JO 2R2R?L' Ä M RQ
NT M	C!@	*J OLNJ 3LÄMKL	N. 77-Ä; q" ÄQU5 +)	8J . K2LÄ *M QP
'Q? S.	R(&/0 f.	'Q? L	A!@BDAIEÄA!@BDAÄ	8NL8VL'Ä *M VM

[illegible]

JP? ? C;ÄÆCAFIRKcÄÄA!@Ä?3Q?LÄ.? J#IR'? ÄÄBQ@C

+ B<sub>4</sub> × B<sub>W</sub> PU' X( +Ä† > (+Ä0,  
; (%/06/G

À # ? O O P A ? A Q 2 SOL



## 1 OF 2

METRIC

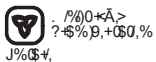
TI KI	BIBBa@@a!!	3J8 Q?2IP	=. K2Q +:s%5(A0N- <Q@P0CDEAGACÄÄC@#ÄI F	JO 2R2R?L' Ä M	RQ
NT M	C!@	*J OLNJ 3LÄMKL	N, 77-Ä; Q" ÄU5 (+	8J . K2L'Ä *M	QP
'Q? S.	R(&/0 f:	'Q? L	ÄI@BÄÄIBÄÄI@BÄÄIB	8NL8VL'Ä *M	VM

;J 23AKOJ 123L		;Q.K3 L;		ROJSP 'ATQ 7LO 8JP' 272P;		L3L = Q72JPÄ8Q3L		MPQ 2 8A8JPL AKLPL?OQ?2J P OL; 2;? QP8 LÄKJ ? A! C! E! G! @!! ;NLQOÄ? OLPR?NÄKS ○ SP8JP1 2PL' + 12L3 Ä=QPL ● Z S28VÄD2Q2Q3 × 3QÄ=CPL A! C! E! G! @!! K3Q:728 32 Z' PQ7SOQ3 J 2 ?SOL 3Z'S2 32 Z' 8JP? LP? 8JP? LP? - K - 3 T Q7LOÄBJP?LP?Ä Ä # A! C! E!		SP2? TL 2RN? 7 [P\ " B		OL.Q OV; ] ROQ2Ä;2L ' 2;? Q2"S?2JP Ä# RO ;Q ;2 83	
L3L= L K?N	'L ; 8O2K?2J P	PO?ÄKJ?	PS* LO	?MKL	bPÄ=C3S								
AFEI	!! !	TOPSOIL:ÄÄ!!Ä" " #	@	::	E								
II A	;/ D-CLAY4Ä);" (Ä)\$24Ä0{SÄ556( 7 1/# ÄQÄ;0P> \$ Ä +,-% ./ ) 0 Ä?23#		A	::	@!								
AFBG	AIA	=( +Ä;0P>ÄÄN\$-Ä	B	::	G								
			C	::	A@								
			D	::	BG								
			E	::	AA								
			F	::	AA								
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AEF@	GI	87S< <ÄILT4Ä0{SÄ)\$& N\$+& R+< T( 0 Ä?23#		::	D								

JP? ? C;ÄÆCAFIRKcÄÄA!@A?3Q?LÄ.? J#IR'? ÄÄBQ@C

+ B<sub>4</sub> X B<sub>W</sub> PU' X (+A) > (+A0,  
; (%/06/G

A!  
 @B-D    Ä # ? OQ P Q ? Ä Q2 SOL  
 @!  
 1



## 2 OF 2

METRIC

TI KI BIBBa@@a!! 3J8 Q?2IP =. K2Q +d\$%5(80N- <C@PÄFDEAGACÄÄC@PÄI F JO 2R2R?L' Ä M RQ

NT M    CI@                      \*J OLNJ 3LÃMKL   N,77-Ä; Q" ÅQU5 (+)                      8J . KZBL'Ä \*M    QP

'Q? S. R,&(0 !: 'Q? L A!@B@AIB!A!@B@AIB 8NL8VL'Ä \*M VM

[illegible]

JP? ? C;ÄÆCAFIRKcÄÄA!@A?3Q?LÄ.? J#IR'? ÄÄBQ@C

+ B<sub>4</sub> × B<sub>W</sub> PU' X( +Ä† > (Ä0,  
; (% /06/G



## 1 OF 2

METRIC

[illegible]

JP? ? C;ÄÆCAFIRKcÄÄA!@A?3Q?LÄ.? J#IR'? ÄÄBQ@C

8,%0%U(8FA(Y0AK\$5(

+ B<sub>4</sub> X B<sub>W</sub> PU' X (+A) > (+A0,  
; (%/06/0

$\begin{matrix} A! \\ @B \bullet D \\ @! \end{matrix}$ 
 $\ddot{A} \# \ddot{A} ? O \ddot{A} ? \ddot{A} O \ddot{A} SOL$



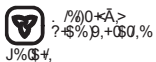
## 2 OF 2

METRIC

[illegible]

JP? ? C;ÄÆCAFIRKcÄÄA!@A?3Q?LÄ.? J#IR'? ÄÄBQ@C


+ B<sub>4</sub> × B<sub>W</sub> PU' X( +Ä† > (Ä0,  
; (% /06/0



## 1 OF 2

METRIC

TI KI	BIBBa@@a!!	3J8 Q?2IP	=. K25+&\$%5(A0N- <Q@P AIDEA GIEA AC@PDI A	JO 2R2R?L' Å M	RQ
NT M	C!@	*J OLNJ 3LA ÅMKL	N, 77-Å; Q" ÅU5 +)	8J . K2L'Å *M	QP
'Q? S.	R(&0 f:	'Q? L	AI@BDAICAAI@BDAIC	8NL8VL'Å *M	VM

;J 2ÄKOJ 123L			;Q.K3 L;			ROJSP ÄTQ 7LO 8JP 272P;			L3L=Q72JPÄ;8Q3L			MPQ.2 8Ä6JPL ÄKL PL?OQ?2J P OL; 2;?QP8 LÄKJ? 			K3Q.728 32 2' PQ7SOQB J 2 ?SOL 8JP? LP? 3Z'S2 32 2'			OL.Q OV; ] ROQ2Ä;2L ' 2;? O2'S?2JP Ä#		
L3L= L K?N			'L ; 8O2K?2J P			?Q?ÄK3J?			PS* LO ?MKL bPÄ-Q3S;			Ä! C! E! G! @!!			- K - 3			SP2? TL 2RN?		
AFCA												Ä! C! E! G! @!!			T Q?LOÄBJP?LP?Ä Ä #			P\ " B		
!!			TOPSOIL:ÄQDÄ" #																	
!! A			:/ 7ÄCLAY4Ä);" ( Ä)SÄÄ0(ÄÄ56( 7 1/+"			@ :: E												B @F DC AE		
AFBD			*+, - % ./ ) 0																	
!! G			Ä?23# ;0/>>			A :: @@														
						B :: @A														
AFAI						C :: A@												C @D C BA		
AIA			=( +Ä;0>>			D :: @G														
						E :: AB														
						F :: A												C @E CA BC		
AEEE																				
FIE			87S< <ÄSILT4Ä0(ÄÄ)SÄ)SÄ& 8,*" 9S: 0ÄÄ'(%( R+(< T( 0 Ä?23#			G :: BD														
						A@														

JP? ? C;ÄÆCAFIRKcÄÄA!@Ä?3Q?LÄ.? J#IR'? ÄÄBQ@C

8,%0%U(8FA(YOK\$5(

+ B<sub>4</sub> X B<sub>W</sub> PU' X (+A†) > (+A0,  
; (%/06/G

$\begin{matrix} A! \\ @B \bullet D \\ @! \end{matrix}$ 
 $\ddot{A} \# \ddot{A} ? O \ddot{A} ? \ddot{A} O \ddot{A} SOL$





## 2 OF 2

METRIC

[illegible]

JP? ? C;ÄÆCAFIRKcÄÄA!@A?3Q?LÄ.? J#IR'? ÄÄBQ@C

+ B<sub>4</sub> × B<sub>W</sub> PU' X( +Ä† > (Ä0,  
; (% /06/G



## METRIC

$\begin{array}{c} A! \\ @B \bullet D \\ @! \end{array}$ 
 $\ddot{A} \# \ddot{A} ? O \ddot{A} ? \ddot{A} O \ddot{A} SOL$

RECORD OF BOREHOLE No 13-08

2 OF 2

METRIC

TI KI BIBBa@@a!! 3J8 Q?2JP =. K2Q +d\$%5(Ä0N- <0@PÄCDEAGAÄÄC@BÄFIE JO 2R2R?L' Ä M RQ  
NT M C!@ \*J OLNJ 3LÄ?MKL N, 77-Ä; Q" ÄU5+ 8J . K2L'Ä \*M QP  
'Q? S. R(&0 t: 'Q? L A!@B0ÄIEÄÄ!@B0ÄIE 8NL8VL'Ä \*M VM

;J 2ÄKOJ 123L			;Q.K3 L;			ROJSP ÄTQ ?LO 8JP' 2?2P';	L3L = Q?2JPÄ;8Q 3L	MPQ 2 8Ä8JPL ÄKLPL ?OQ?2J P OL; 2;?QP8 LÄKJ ? Ä! C! E! G! @!! ;NLQOÄ;? OLPR?NÄK\$ ○ SP8JP1 2PL' + 12L3 Ä=QPL ● ZS28VÄD2Q?2Q3 × 3QÄ=QPL Ä! C! E! G! @!!					K3Q?28 32 2' PQ?SOQB J 2 ?SOL 8JP? LP? 3ZS2 32 2' - K - 3 T Q?LOÄ8JP?LP?Ä Ä # Ä! C! E!			SP2? TL 2RN? Y P" B	OL.Q OV; ] ROQ2Ä;2L ' 2;?O2'S?2JP Ä'#	
L3L= L K?N	'L : 8O2K?2J P	?OQ ?ÄK3J?	PS: LO	?MKL	bPbÄ=Q3B			Ä! C! E! G! @!!	Ä! C! E!	Ä! C! E!	Ä! C! E!	RO ;Q ;2 83						
	8, %0%U(&Ä+;" ÄK+(G/U) ÄK\$5( :/ 7ÄCLAY4Ä)" (Ä) \$&4Ä0(Ä\$Ä56(7 1Ä" ÄOÄ=(+Ä;0Ä> *+, - % ./ ) 0 Ä?23#		@!	::	@C		AF@											
							AF!											
			@@	::	AA		AE											
AEGD @B	87\$(<ÄSILT4Ä)" (Ä) \$&4Ä0(Ä\$Ä56(7 =(+Ä;0Ä> R(< T( 0 Ä?23#		@A	::	A!		AEG											
							AER											
			@B	::	AF		AEE											
AEDD @D	LP' ÄJ1Ä"J OLNJ 3LÄQÄ@GÄ" I *J OLNJ 3LÄJKLPÄ? Ä@GÄ"ÄQP TQ ?LOÄ8L=L 3ÄQ?Ä@GÄ I *J OLNJ 3LÄ"Q8V123L' ÄT2?N *L P?J P21ÄNI 3LK3SRÄ? ;S O1Q8LI																	

JP? ? C:ÄECAFIRKÄÄÄ!@K' 3Q?LÄ? J#R? ÄÄB0@C



## 1 OF 2

METRIC

[illegible]

JP? ? C;ÄÆCAFIRKcÄÄA!@Ä?3Q?LÄ.? J#IR'? ÄÄBQ@@C

8,%0%U(8FA(YOK\$5(

+ B<sub>4</sub> X B<sub>W</sub> PU' X (+A†) > (+A0,  
; (%/06/G

A!  
 @B@D  
 @!  
 Ä #Ä ?OQPAQ?Ä Q2SOL

RECORD OF BOREHOLE No 13-09

2 OF 2

METRIC

TI KI BIBBa@@a!! 3J8 Q?2JP =. K2Q+d\$%5(00N- <0@PACAFEDFAA AC@EBI @ JO 2R2R?L' A M RQ  
NT M C!@ \*J OLNJ 3LA?MKL N.77-A; Q" AU5+ 8J . K2L'A \*M .1Q  
'Q? S. R(&0 t: 'Q? L A!@BDAIAAA!@BDAIA 8NL8VL'A \*M VM

;J 23AKOJ 12BL			;Q.K3 L;			MPQ 2 8AJPL AKLPL?OQ?2J P OL; 2;?QP8LAKJ?			K3Q?28 32 2P PQ?SOQB J 2 ?SOL 8JP? LP? 3ZS2 32 2P			SP2? TL 2RN?			OL.Q OV; ] ROQ2FA;2L ' 2;?O2'S?2JP A#		
L3L= L K?N	L : 8O2K?2J P	?OQ ?AK3J?	PS: LO	?MKL	bPbA=Q3S	ROJSP 'ATQ ?LO 8JP' 2?2P;	L3L = Q?2JP?8Q3L	A! C! E! G! @!!	- K - 3	T Q?LO8JP?LP?A A #	A! C! E!	[P" B	RO ;Q ;2 83				
	87\$(< <SILT4A0+(\$A)3\$& =(+<A)0+>AA\$& R+< ./ )0 A?23#		@!	::	@F		AEC										
AEA@	:/ TACLAY =(+<A;0+> *+, - % T( 0 A?23#		@@	::	AC		AEB										
AEIE	SAND4A);" (A)D4A0+ \$56(7 =(+<A' (%( *+, - % T( 0		@A	::	E!		AE@										
ADG@	LP' AJ1A"JOLNJ 3LAQACAI K(e, " (0(+99 0\$%0/ %A: %9/0)A>A@ " " A/S" (0(+A: d(&U7(AIAK=8 A/9( - /0A5QAA A 70Q &A: -(%)  T Q?LO8L=L 3LOQ' 2PR; W ' Q?LAAAAAKAA#AAIA-IAA# ' (:IAA VA@AAW@AAWAAAD c\$AA@CAAA@AAWAAAD		@B	::	GE		AEI										
							AD										

JP? ? C:AECAFIRKcAA@K'3Q?LA? J#IR? AAB@C



## 1 OF 2

METRIC

[illegible]

8,%0%U(8FA(YOK\$5(

+ B<sub>4</sub> X B<sub>W</sub> PU' X (+A†) > (+A0,  
; (%/06/G

$\begin{array}{c} A! \\ @B \bullet D \\ @! \end{array}$ 
 $\ddot{A} \# \ddot{A} ? O \ddot{A} ? \ddot{A} O \ddot{A} SOL$



## METRIC

L3L=		J 2A KOJ 123L		:Q.K3 L;		MPO 2 8A6JPL AKLPL?OQ?2I P OL; 2;?QP8 LAKJ ?	K3Q?28 3Z 2P	PQ780G 3Z S2 3Z 2P	OL.Q OV; ] ROQ2FA;2L ' 2;?O2'S?2JF A#
L K?N		L : 8OK?2I P	?QO QAKJ?	PS* LO	?MKL	A' E' G' @!! :NLQOA? OLPR?NAKS o SP8JP1 PL' + 12L3 A=CPL ● ZS28VA'D2Q?2Q3 X 3Q'A=CPL	- K - 3	T Q?LOA6JP?LP?A A #	SP2? TL 2RN?
		8.%0%U(8A+* AK+(6/U) AK\$5(		bPhA-Q9S	ROJSP ATQ ?LO 8JP 272P;	A' C' E' G' @!! T Q?LOA6JP?LP?A A #	A' C' E'	[P " B	RO : Q ; 2 8

[illegible]

JP? ? C;ÄÆCAFIRKcÄÄA!@A?3Q?LÄ.? J#IR'? ÄÄBQ@C

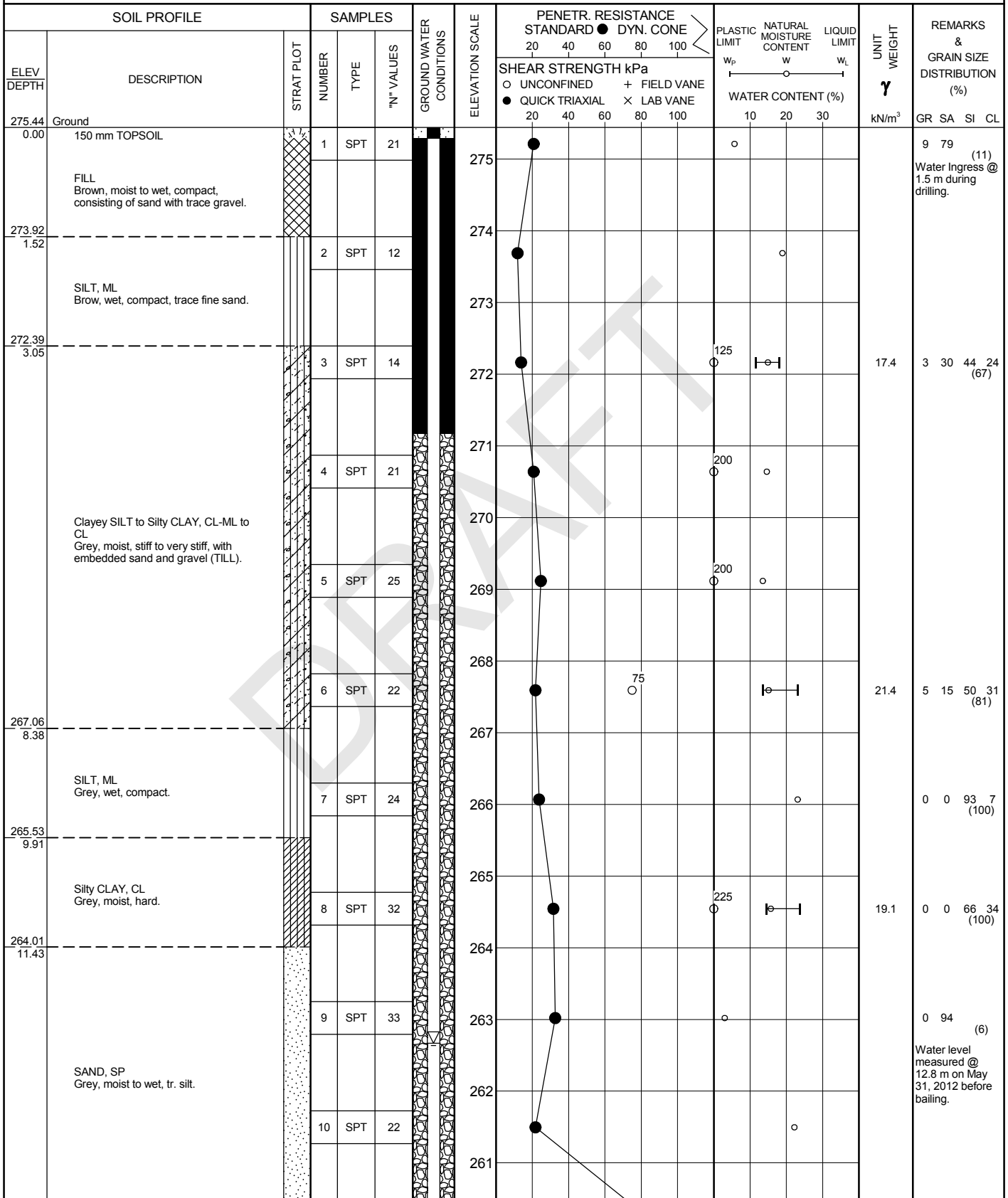
+ B<sub>4</sub>X<sup>B</sup>W<sup>PU'</sup>X(+A<sup>+</sup>)+(+A<sup>0</sup>,  
; (%/06G<sup>@B<sup>+</sup>D</sup> A<sup>A</sup>?00P<sup>A</sup>?A<sup>Q2</sup>SOL  
@!

# RECORD OF BOREHOLE No 1

1 OF 3

METRIC

W.P. GWP 3033-11-00 LOCATION Veterans Memorial Parkway SE Northing - 4756823, Easting - 416987 ORIGINATED BY JL  
 DIST London HWY 401 BOREHOLE TYPE 110 mm ID H/S Auger COMPILED BY JL  
 DATUM Geodetic DATE 21.2.12 - 22.2.12 CHECKED BY EC



Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

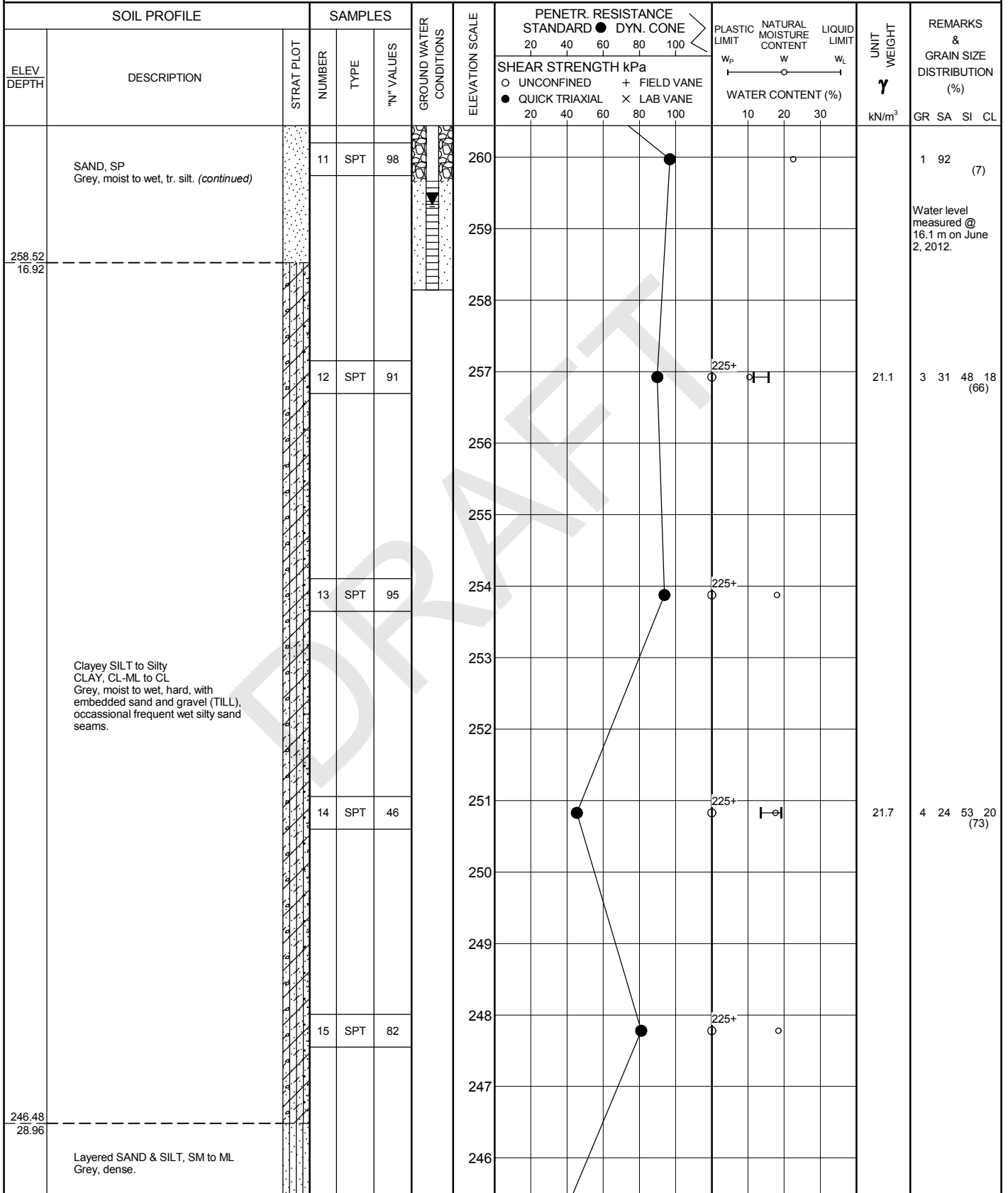


# RECORD OF BOREHOLE No 1

2 OF 3

METRIC

W.P. GWO-3033-11-00 LOCATION Veterans Memorial Parkway SE Northing - 4756823, Easting - 416987 ORIGINATED BY JL  
 DIST London HWY 401 BOREHOLE TYPE 110 mm ID H/S Auger COMPILED BY JL  
 DATUM Geodetic DATE 21.2.12 - 22.2.12 CHECKED BY EC



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 150 UNCONFINE SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 1										3 OF 3		METRIC		
W.P. <u>GWP 3033-11-00</u>			LOCATION <u>Veterans Memorial Parkway SE</u> Northing - 4756823, Easting - 416987				ORIGINATED BY <u>JL</u>							
DIST <u>London</u> HWY <u>401</u>			BOREHOLE TYPE <u>110 mm ID H/S Auger</u>				COMPILED BY <u>JL</u>							
DATUM <u>Geodetic</u>			DATE <u>21.2.12 - 22.2.12</u>				CHECKED BY <u>EC</u>							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE STANDARD ● DYN. CONE		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE						
244.50 30.94	Layered SAND & SILT, SM to ML Grey, dense. (continued)		16	SPT	31		245							0 51 43 7 (49)
	End of borehole.													

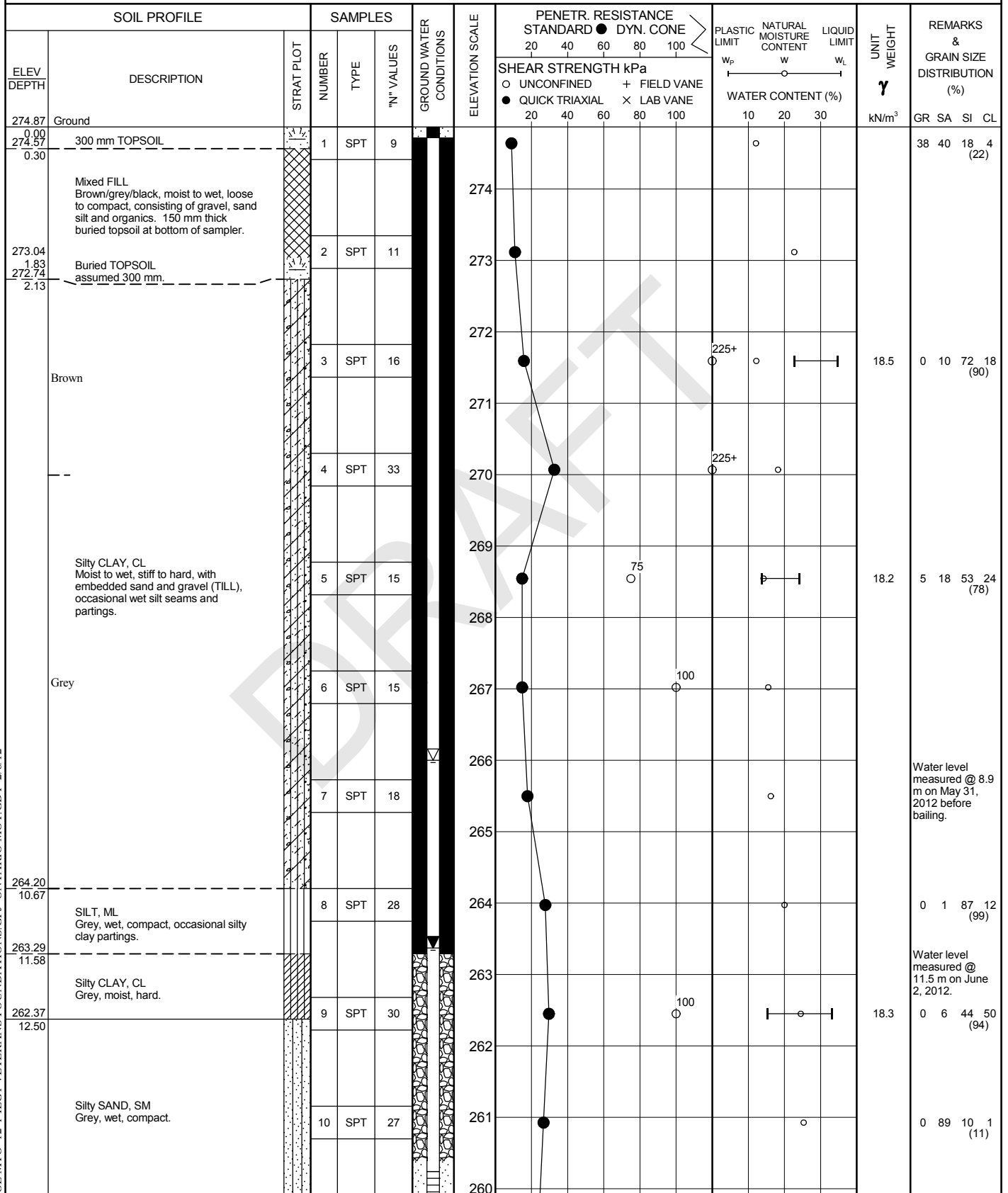
JOE MTO 12-1-IEG1 VETERANS FOUNDATIONS.GPJ ONTARIO MOT.GDT 2/6/12

# RECORD OF BOREHOLE No 2

1 OF 3

METRIC

W.P. GWP 3033-11-00 LOCATION Veterans Memorial Parkway NW Northing - 4756861, Easting - 416949 ORIGINATED BY JL  
 DIST London HWY 401 BOREHOLE TYPE 110 mm ID H/S Auger COMPILED BY JL  
 DATUM Geodetic DATE 23.2.12 - 24.2.12 CHECKED BY EC



Continued Next Page

+ 3, X 3: Numbers refer to  
Sensitivity

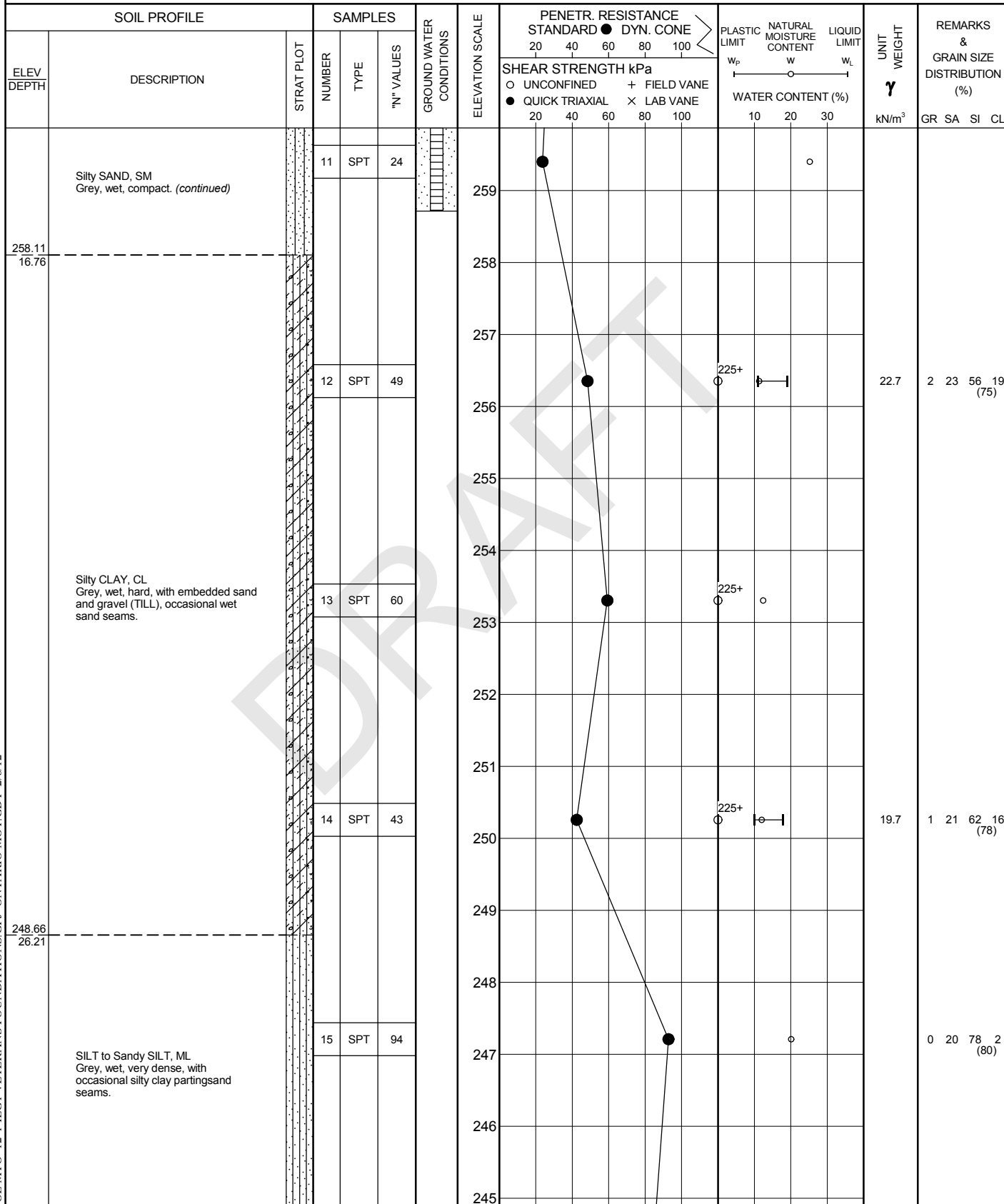
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

# RECORD OF BOREHOLE No 2

2 OF 3

METRIC

W.P. GWP 3033-11-00 LOCATION Veterans Memorial Parkway NW Northing - 4756861, Easting - 416949 ORIGINATED BY JL  
DIST London HWY 401 BOREHOLE TYPE 110 mm ID H/S Auger COMPILED BY JL  
DATUM Geodetic DATE 23.2.12 - 24.2.12 CHECKED BY EC



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

## METRIC

[illegible]

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 1

W.P. 32-73-02 LOCATION Co-ords. 15,605,824 N: 1,367,952 E. ORIGINATED BY PJS  
DIST 2 HWY. Local BORING DATE October 31, 1974 COMPILED BY PJS  
DATUM Geodetic BOREHOLE TYPE Solid Auger and Cone Test CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$ P.C.F.	REMARKS % GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
899.0	Ground Level															
0.0			1	SS	20											
			2	SS	22											5 18 58 19
			3	SS	31											
			4	SS	29											
	Brown Grey		5	SS	21											
	Clayey silt, some sand, traces of gravel (Glacial Till)		6	SS	20											2 18 49 31
			7	SS	23											
	Very Stiff to Hard		8	SS	27											
863.5			9	SS	68											0 94 (6)
862.5	F. Sand Very Dense															
36.5	End of Borehole															

OFFICE REPORT ON SOIL EXPLORATION

20  
15 5 % STRAIN AT FAILURE  
10



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 2


W.P. 32-73-02 LOCATION Co-ords. 15,605,799 N: 1,367,970 E. ORIGINATED BY RJS  
DIST. 2 HWY. Local BORING DATE October 31, 1974 COMPILED BY RJS  
DATUM Geodetic BOREHOLE TYPE Solid Auger & Cone Test CHECKED BY [Signature]

SOIL PROFILE		SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT				LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	VALUES	20	40	60	80	100	$w_p$	$w$	$w_L$	
899.2	Ground Level													
0.0			1	SS	13									
			2	SS	20									
			3	SS	24									
	Brown Grey		4	SS	23									
			5	SS	23									
	Clayey silt, some sand, traces of gravel (Glacial Till)		6	SS	24									1 19 54 26
			7	SS	21									
	Stiff to Hard		8	SS	26									
			9	SS	36									
862.8	silly clay		10	SS	20									0 0 45 55
36.4	Fine sand. Grey													
859.2														
40.0	End of Borehole													

20  
15 5 % STRAIN AT FAILURE  
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 3

W.P. 32-73-02 LOCATION Co-ords. 16,605,789 N: 1,367,936 E. ORIGINATED BY PJS  
DIST. 2 HWY. Local BORING DATE November 6, 1974 COMPILED BY PJS  
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY 

SOIL PROFILE		STRAT PLOT	SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV DEPTH	DESCRIPTION		NUMBER	TYPE	VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
897.9	Ground Level															GR SA SI CL
0.0																W.L. not established
			1	SS	21											3 15 70 12
			2	SS	23											3 21 52 24
			3	SS	24											
			4	SS	26											
			5	SS	18											
			6	SS	20											
			7	SS	25											0 4 69 27
			8	SS	37											
862.4			9	SS	51											
35.5	Fine Sand		10	SS	17											0 96 (4)
	Dense to Compact		11	SS	22											
	Grey		12	SS	67											
847.7			13	SS	95											
50.2	Clayey silt, some		14	SS	105											3 24 61 12
	sand, traces of															
	gravel															
	(Glacial Till)															
	Hard															
	Grey		15	SS	60											5 22 58 15
816.4			16	SS	103											
81.5	Fine to medium sand		17	SS	20											
	with silt.															
	Compact Grey															
796.4			18	SS	22											0 72 (28)
101.5	End of Borehole															

20  
15  $\phi$  5 % STRAIN AT FAILURE  
10



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 4

W.P. 32-73-02 LOCATION Co-ords. 16,605,718 N; 1,367,974 E. ORIGINATED BY PJS  
DIST. 2 HWY. Local BORING DATE November 6, 1974 COMPILED BY PJS  
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger & Cone CHECKED BY J.P.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
901.7	Ground Level															
0.0	Clayey silt, some sand, traces of gravel (Glacial Till) Very Stiff to Hard		1	SS	28	900										5 21 58 16
			2	SS	28											
			3	SS	25											
			4	SS	26	890										
	Brown silty sand Grey		5	SS	22											3 57 31 9
			6	SS	28	880										
			7	SS	34											1 21 47 31
870.2			8	SS	32											
31.5	End of Borehole															
	Note: Water level not established.															

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

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEER- G SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 5

W.P. 32-73-02 LOCATION Co-ords. 15,605,644 N: 1,367,970 E. ORIGINATED BY PJS  
DIST. 2 HWY. Local BORING DATE November 4, 1974 COMPILED BY PJS  
DATUM Geodetic BOREHOLE TYPE Solid Auger and Cone CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
900.1	Ground Level															
0.0			1	SS	13											
			2	SS	26											
			3	SS	12											
			4	SS	18											
			5	SS	26											
			6	SS	10											
			7	SS	31											
			8	SS	25											
			9	SS	35											
868.6																
31.5	End of Borehole															

20  
15  $\diamond$  5 % STRAIN AT FAILURE  
10



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 6

W.P. 32-73-02 LOCATION Co-ords. 16,605,655N: 1,368,002 E. ORIGINATED BY PJS  
DIST 2 HWY. Local BORING DATE November 5, 1974 COMPILED BY PJS  
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT $w_L$ PLASTIC LIMIT $w_p$ WATER CONTENT $w$			UNIT WEIGHT $\gamma$	REMARKS % GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	VALUES		20	40	60	80	100	$w_p$	$w$	$w_L$		
901.2	Ground Level															
0.0			1	SS	23	900										
			2	SS	32											
			3	SS	34											
			4	SS	41	890										2 18 50 30
			5	SS	24											
			6	SS	32	880										1 15 59 25
			7	SS	28											
869.7			8	SS	40	870										
31.5	End of Borehole															

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 7

W.P. 32-73-02 LOCATION Co-ords. 16,605,610 N: 1,367,995 E. ORIGINATED BY PJS  
DIST. 2 HWY. Local BORING DATE November 1, 1974 COMPILED BY PJS  
DATUM Geodetic BOREHOLE TYPE Follow Stem Auger and Cone CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT				LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	$W_P$	$W$	$W_L$	
902.6	Ground Level														
0.0	Brown Grey  Clayey silt, some sand, traces of gravel (Glacial Till)  Stiff to Hard		1	SS	7	900									1 18 58 23
			2	SS	20										
			3	SS	29										
			4	SS	29										
			5	SS	23	890									
			6	SS	20										
			7	SS	26										
			8	SS	33	880									
			9	SS	35										4 11 58 27
866.6	Fine Sand  Dense to Compact  Grey		10	SS	50	870									
36.0			11	SS	34										
			12	SS	23	860									1 98 ( 1 )
			13	SS	12										
846.6	Clayey silt, some sand, traces of gravel (Glacial Till)  Hard Grey					850									
56.0			14	SS	98										
			15	SS	97	840									1 19 59 21
			16	SS	100/5"										
			17	SS	57	830									3 16 61 20
			18	SS	115										
			19	SS	56	820									
810.6	Fine sand with silt, Compact					810									
92.0			20	SS	12										0 79 ( 21 )
806.1	End of Borehole  Note: Water level not established.														
96.5															

20  
15  $\phi$  5 % STRAIN AT FAILURE  
10

OFFICE REPORT ON SOIL EXPLORATION

## APPENDIX C

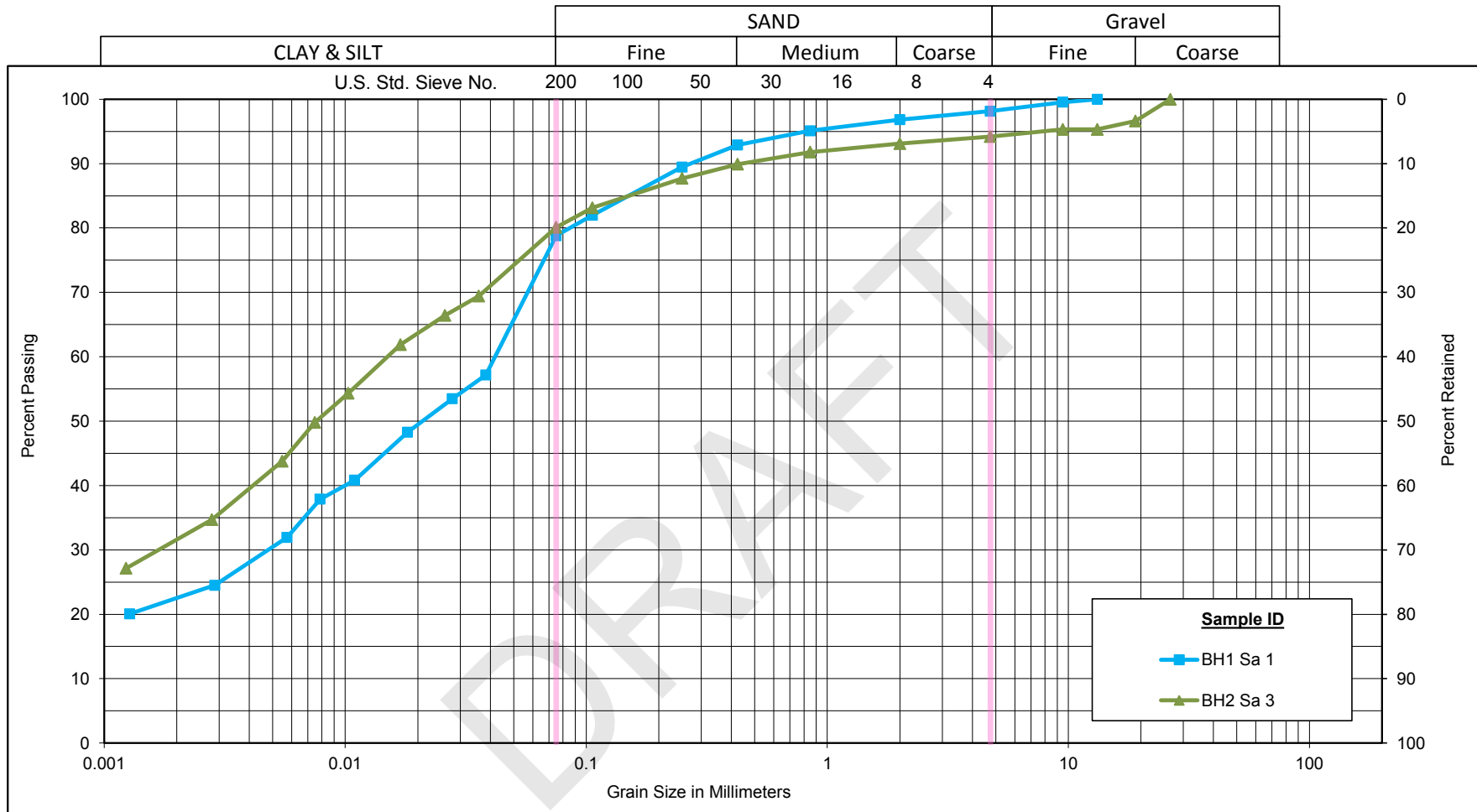
Laboratory Test Results

Figures C1 to C22

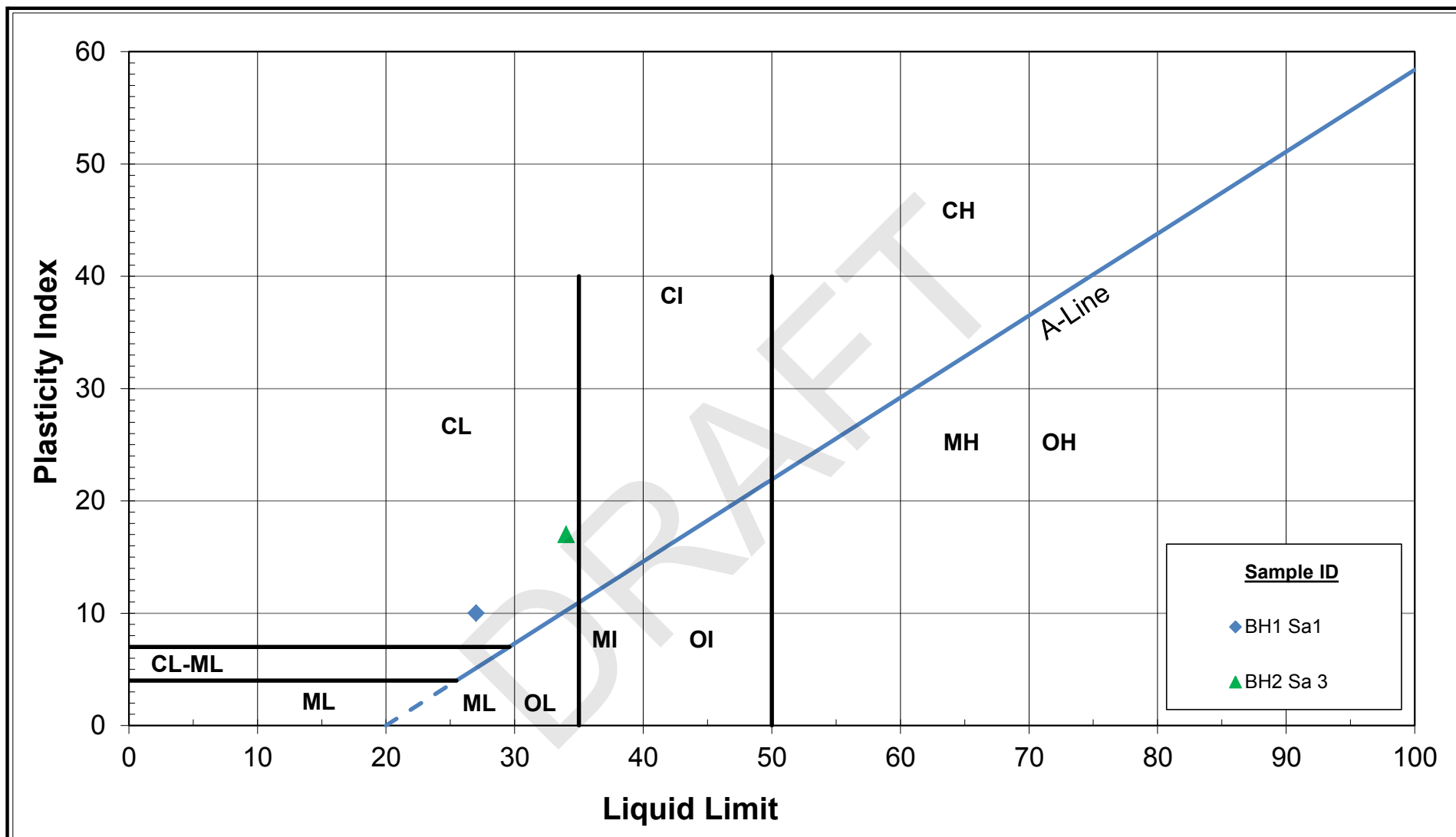
Corrosivity Testing Results



# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH1 Sa 1	0.0-0.6	2.0	19.2	56.8	22.0
BH2 Sa 3	3.0-3.7	6.0	13.9	49.1	31.0



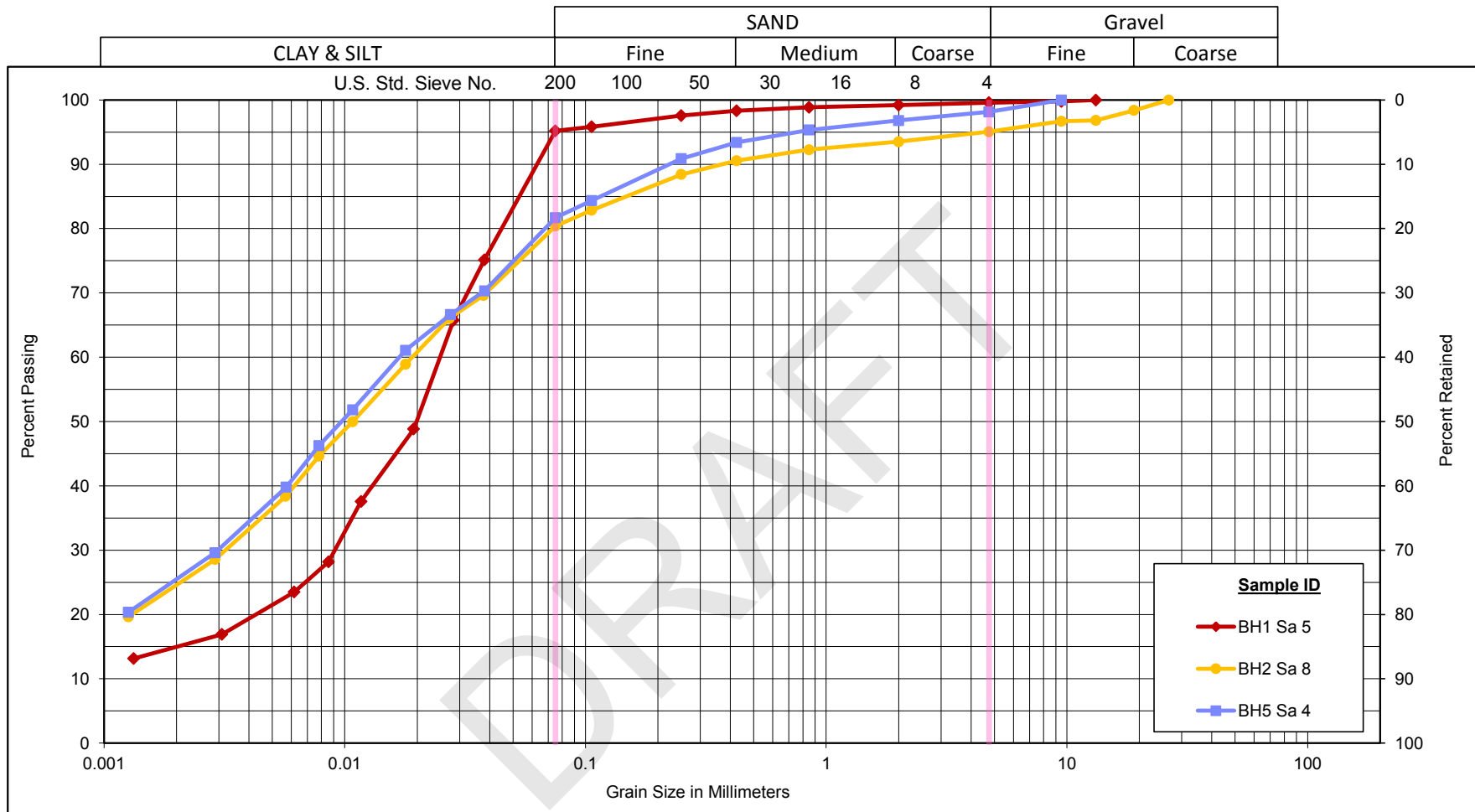
# PLASTICITY CHART

## FILL: Clayey Silt (CL)

Figure No. C2

Project No. 165001002.26

# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH1 Sa 5	3.0-3.4	0.0	4.8	80.2	15.0
BH2 Sa 8	10.7-11.3	5.0	14.6	56.4	24.0
BH5 Sa 4	2.3-2.9	2.0	16.3	56.7	25.0

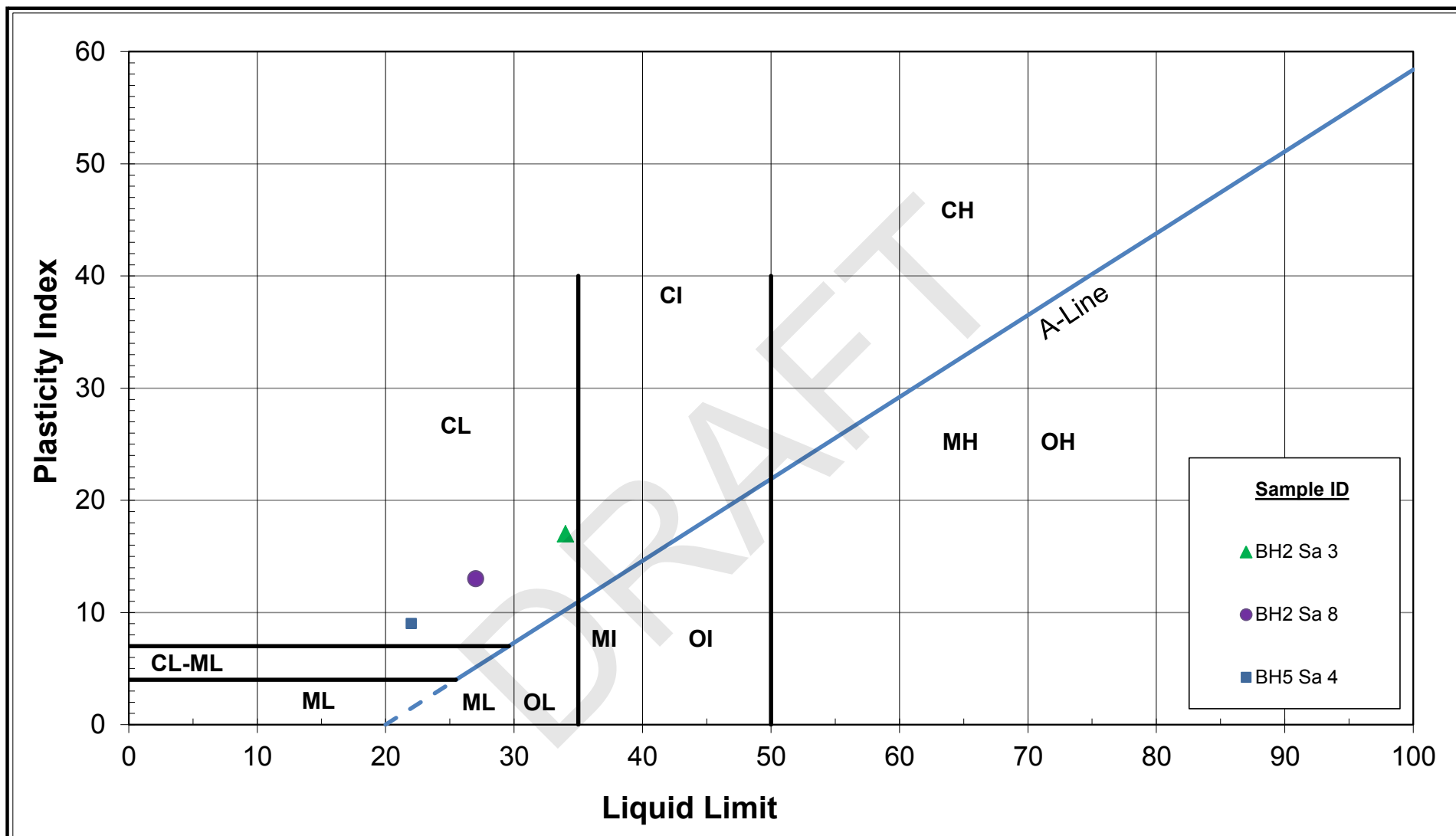


GRAIN SIZE DISTRIBUTION  
Upper Clayey Silt TILL (CL)

Figure No. C3

Project No. 165001002.260





# PLASTICITY CHART

## Upper Clayey Silt TILL (CL)

Figure No. C4

Project No. 165001002.26

**Project**  
**Project No.**  
**Borehole No.**  
**Sample No.**  
**Sample Depth**

**Design-Build HWY 401/VMP**  
**165001002.260**  
**BH 1**  
**ST6**  
**15-17 ft**

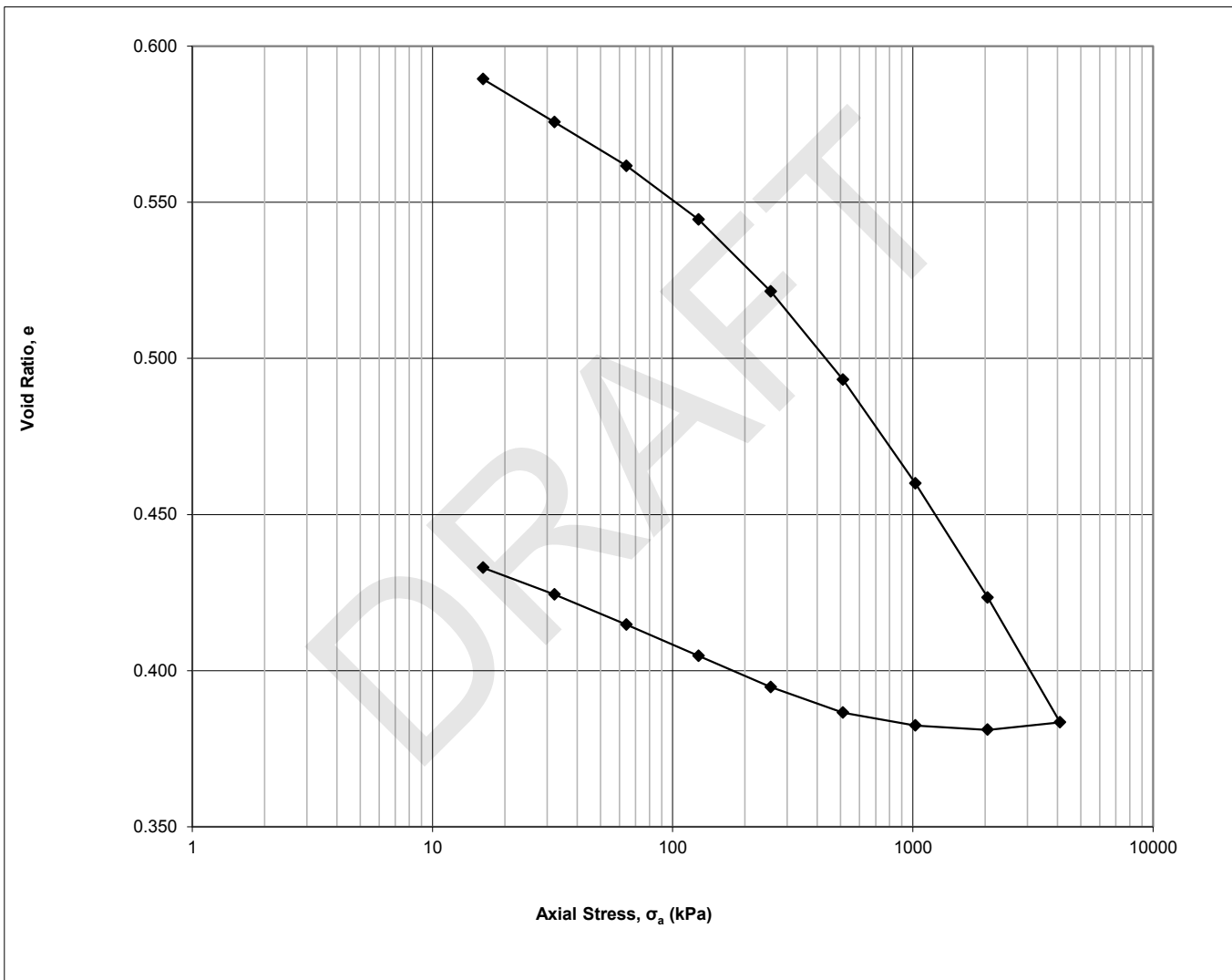


Figure No. C5-A

**One-Dimensional Consolidation Test using Incremental Loading**  
**ASTM D2435/D2435M - 11**

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	HWY 401
Borehole	BH 1
Sample No.	ST6
Depth	15-17 ft
Sample Date	08/04/2016
Test Number	One
Technician Name	Daniel Boateng

**Soil Description & Classification**

Silt	
Specific Gravity of Solids	2.749
Average water content of trimmings %	20
<b>Additional Notes (information source, occurrence and size of large isolated particles etc.)</b>	

**Initial Specimen Conditions**

Height	mm	20.00
Diameter	mm	50.00
Area	mm <sup>2</sup>	1963
Volume	mm <sup>3</sup>	39270
Mass	g	81.25
Dry Mass	g	67.51
Density	Mg/m <sup>3</sup>	2.069
Dry Density	Mg/m <sup>3</sup>	1.719
Water Content	%	20.35
Degree of Saturation	%	93.4
Height of Solids	mm	12.51
Initial Void Ratio		0.599

**Final Specimen Conditions**

Water Content	%	17.33
Final Void Ratio		0.433
Degree of Saturation	%	110.0
Differential Height	mm	1.92
Estimated Preconsolidation Stress	kPa	

Figure No. C5-B

## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	HWY 401
Borehole	BH 1
Sample No.	ST6
Depth	15-17 ft
Sample Date	08/04/2016
Test Number	One
Technician Name	Daniel Boateng

**Test Procedure**

Date Started	15/04/2016
Date Finished	16/04/2016
Machine Number	C
Cell Number	C
Ring Number	C
Trimming Procedure	Turntable
Moisture Condition	Inundated
Axial Stress at Inundation	8 kPa
Water Used	Distilled
Test Method	B
Interpretation Procedure for $c_v$	2

**All Departures from Outlined ASTM D2435/D2435M-11 Procedure**
**Calculations**

Load Increment	Increment Duration	Axial Stress	Corrected Deformation	Specimen Height	Axial Strain	Void Ratio
	min	$\sigma_a$ kPa	$\Delta H$ mm	H mm	$\epsilon_a$ %	e
Seating	73.0	8	0.0000	20.0000	0.00	0.599
1	71.5	16	0.1205	19.8795	0.60	0.589
2	76.5	32	0.2936	19.7064	1.47	0.576
3	78.3	64	0.4682	19.5318	2.34	0.562
4	78.3	128	0.6832	19.3168	3.42	0.544
5	83.3	256	0.9710	19.0290	4.86	0.521
6	85.0	512	1.3248	18.6752	6.62	0.493
7	93.5	1024	1.7402	18.2598	8.70	0.460
8	98.5	2048	2.1969	17.8031	10.98	0.423
9	107.0	4096	2.6965	17.3035	13.48	0.383
10	70.0	2048	2.7267	17.2733	13.63	0.381
11	70.0	1024	2.7098	17.2902	13.55	0.382
12	70.0	512	2.6577	17.3423	13.29	0.387
13	71.8	256	2.5558	17.4442	12.78	0.395
14	75.0	128	2.4308	17.5692	12.15	0.405
15	80.3	64	2.3055	17.6945	11.53	0.415
16	85.3	32	2.1846	17.8154	10.92	0.424
17	87.0	16	2.0776	17.9224	10.39	0.433
18						
19						

Figure No. C5-C

## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	HWY 401
Borehole	BH 1
Sample No.	ST6
Depth	15-17 ft
Sample Date	08/04/2016
Test Number	One
Technician Name	Daniel Boateng

**Calculations**

Load Increment	Axial Stress $\sigma_a$ , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation $\Delta H_{50}$ mm	Specimen Height $H_{50}$ mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio $e_{50}$	Time $t_{50}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s	Time $t_{90}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s
Seating	8								
1	16	0.0764	19.9236	0.38	0.593			150	5.60E-01
2	24	0.2395	19.7605	1.20	0.580			116	7.11E-01
3	48	0.4102	19.5898	2.05	0.566			110	7.37E-01
4	96	0.6125	19.3875	3.06	0.550			101	7.92E-01
5	192	0.8772	19.1228	4.39	0.529			118	6.59E-01
6	384	1.2102	18.7898	6.05	0.502			107	7.00E-01
7	768	1.5667	18.4333	7.83	0.474			66	1.09E+00
8	1536	2.0136	17.9864	10.07	0.438			114	6.03E-01
9	3072	2.4292	17.5708	12.15	0.405			65	1.01E+00
10	3072	2.7253	17.2747	13.63	0.381				
11	1536	2.7126	17.2874	13.56	0.382				
12	768	2.6763	17.3237	13.38	0.385				
13	384	2.5771	17.4229	12.89	0.393				
14	192	2.4625	17.5375	12.31	0.402				
15	96	2.3421	17.6579	11.71	0.412				
16	48	2.2266	17.7734	11.13	0.421				
17	24	2.1636	17.8364	10.82	0.426				
18									
19									

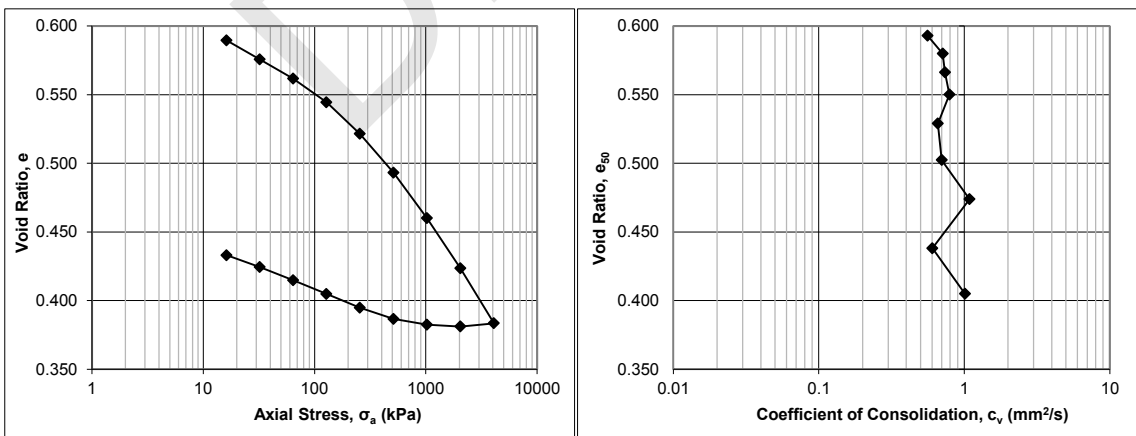


Figure No. C5-D

Project  
Project No.  
Borehole No.  
Sample No.  
Sample Depth

Design-Build HWY 401/VMP  
165001002.260  
BH 2  
ST7  
30-32 ft

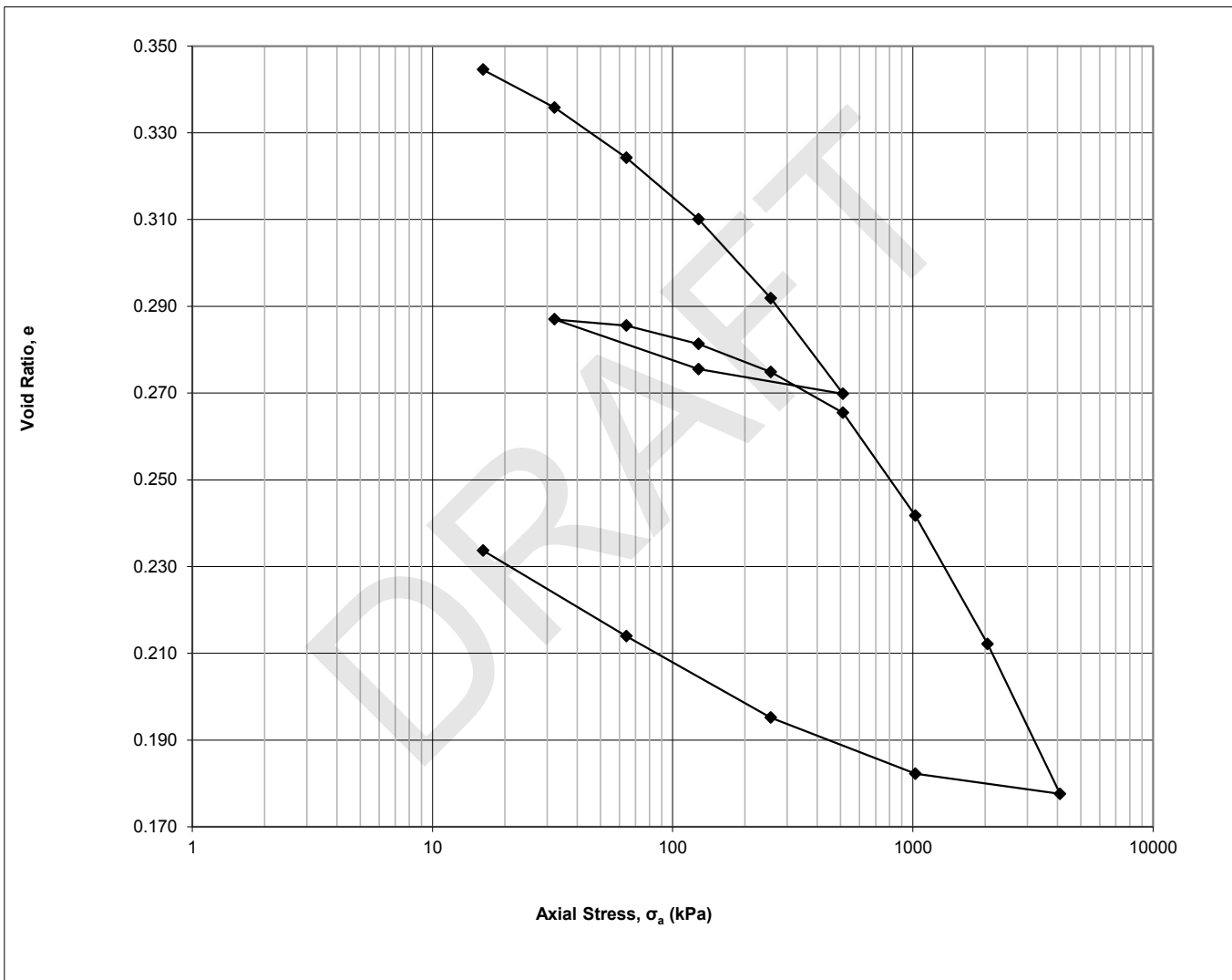


Figure No. C6-A

**One-Dimensional Consolidation Test using Incremental Loading**  
**ASTM D2435/D2435M - 11**

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	HWY 401
Borehole	BH 2
Sample No.	ST7
Depth	30-32 ft
Sample Date	07/04/2016
Test Number	Four
Technician Name	Daniel Boateng

**Soil Description & Classification**

Silty Clay Till	
Specific Gravity of Solids	2.735
<b>Additional Notes (information source, occurrence and size of large isolated particles etc.)</b>	

**Initial Specimen Conditions**

Height	mm	20.00
Diameter	mm	50.00
Area	mm <sup>2</sup>	1963
Volume	mm <sup>3</sup>	39270
Mass	g	88.89
Dry Mass	g	78.12
Density	Mg/m <sup>3</sup>	2.264
Dry Density	Mg/m <sup>3</sup>	1.989
Water Content	%	13.79
Degree of Saturation	%	100.6
Height of Solids	mm	14.55
Initial Void Ratio		0.375

**Final Specimen Conditions**

Water Content	%	12.58
Final Void Ratio		0.234
Degree of Saturation	%	147.3
Differential Height	mm	1.95
Estimated Preconsolidation Stress	kPa	

Figure No. C6-B

## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	HWY 401
Borehole	BH 2
Sample No.	ST7
Depth	30-32 ft
Sample Date	07/04/2016
Test Number	Four
Technician Name	Daniel Boateng

**Test Procedure**

Date Started	22/04/2016
Date Finished	23/04/2016
Machine Number	C
Cell Number	C
Ring Number	C
Trimming Procedure	Turntable
Moisture Condition	Inundated
Axial Stress at Inundation	16 kPa
Water Used	Distilled
Test Method	B
Interpretation Procedure for $c_v$	2

**All Departures from Outlined ASTM D2435/D2435M-11 Procedure**
**Calculations**

Load Increment	Increment Duration	Axial Stress	Corrected Deformation	Specimen Height	Axial Strain	Void Ratio
	min	$\sigma_a$ kPa	$\Delta H$ mm	H mm	$\epsilon_a$ %	e
Seating	0.0	16	0.0000	20.0000	0.00	0.375
1	71.5	16	0.4408	19.5592	2.20	0.345
2	73.3	32	0.5681	19.4319	2.84	0.336
3	76.5	64	0.7367	19.2633	3.68	0.324
4	78.3	128	0.9424	19.0576	4.71	0.310
5	80.0	256	1.2072	18.7928	6.04	0.292
6	83.3	512	1.5280	18.4720	7.64	0.270
7	70.0	128	1.4451	18.5549	7.23	0.276
8	80.0	32	1.2782	18.7218	6.39	0.287
9	70.0	64	1.2992	18.7008	6.50	0.286
10	70.0	128	1.3605	18.6395	6.80	0.281
11	70.0	256	1.4547	18.5453	7.27	0.275
12	75.0	512	1.5913	18.4087	7.96	0.265
13	92.0	1024	1.9365	18.0635	9.68	0.242
14	92.0	2048	2.3674	17.6326	11.84	0.212
15	102.3	4096	2.8696	17.1304	14.35	0.178
16	70.0	1024	2.8023	17.1977	14.01	0.182
17	77.0	256	2.6135	17.3865	13.07	0.195
18	99.0	64	2.3411	17.6589	11.71	0.214
19	131.3	16	2.0536	17.9464	10.27	0.234

Figure No. C6-C



## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	HWY 401
Borehole	BH 2
Sample No.	ST7
Depth	30-32 ft
Sample Date	07/04/2016
Test Number	Four
Technician Name	Daniel Boateng

**Calculations**

Load Increment	Axial Stress $\sigma_a$ , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation $\Delta H_{50}$ mm	Specimen Height $H_{50}$ mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio $e_{50}$	Time $t_{50}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s	Time $t_{90}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s
Seating	8								
1	16	0.3958	19.6042	1.98	0.348			134	6.08E-01
2	24	0.5217	19.4783	2.61	0.339			139	5.80E-01
3	48	0.6818	19.3182	3.41	0.328			175	4.53E-01
4	96	0.8764	19.1236	4.38	0.315			148	5.25E-01
5	192	1.1282	18.8718	5.64	0.297			150	5.03E-01
6	384	1.4285	18.5715	7.14	0.277			169	4.32E-01
7	320	1.4600	18.5400	7.30	0.274				
8	80	1.3175	18.6825	6.59	0.284				
9	48	1.2917	18.7083	6.46	0.286			276	1.01E-04
10	96	1.3407	18.6593	6.70	0.283			223	3.31E-01
11	192	1.4296	18.5704	7.15	0.277			180	4.06E-01
12	384	1.5490	18.4510	7.74	0.268			215	3.36E-01
13	768	1.7948	18.2052	8.97	0.251			148	4.76E-01
14	1536	2.1550	17.8450	10.78	0.227			142	4.76E-01
15	3072	2.6080	17.3920	13.04	0.196			149	4.30E-01
16	2560	2.8137	17.1863	14.07	0.181				
17	640	2.6653	17.3347	13.33	0.192				
18	160	2.4318	17.5682	12.16	0.208				
19	40	2.1669	17.8331	10.83	0.226				

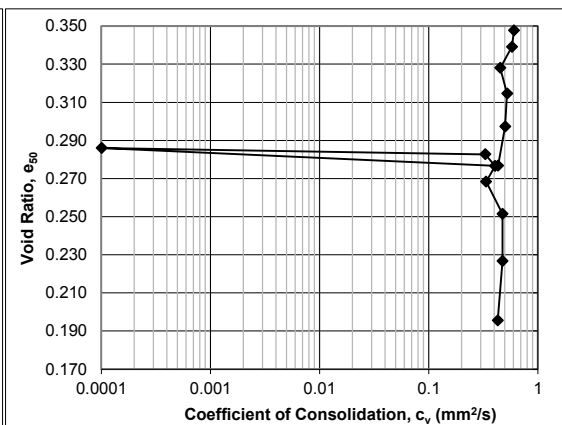
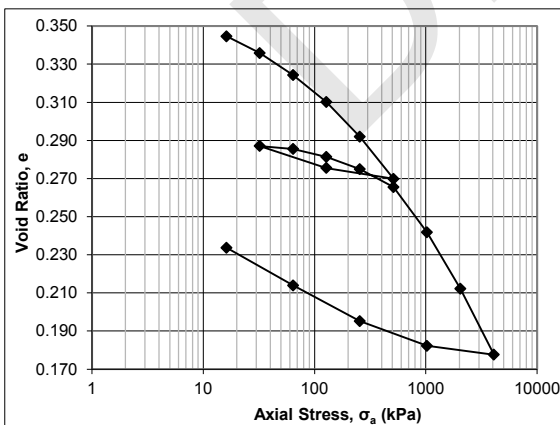


Figure No. C6-D

Project  
Project No.  
Borehole No.  
Sample No.  
Sample Depth

Design-Build HWY 401/VMP  
165001002.260  
BH 5  
ST3  
5-7 ft

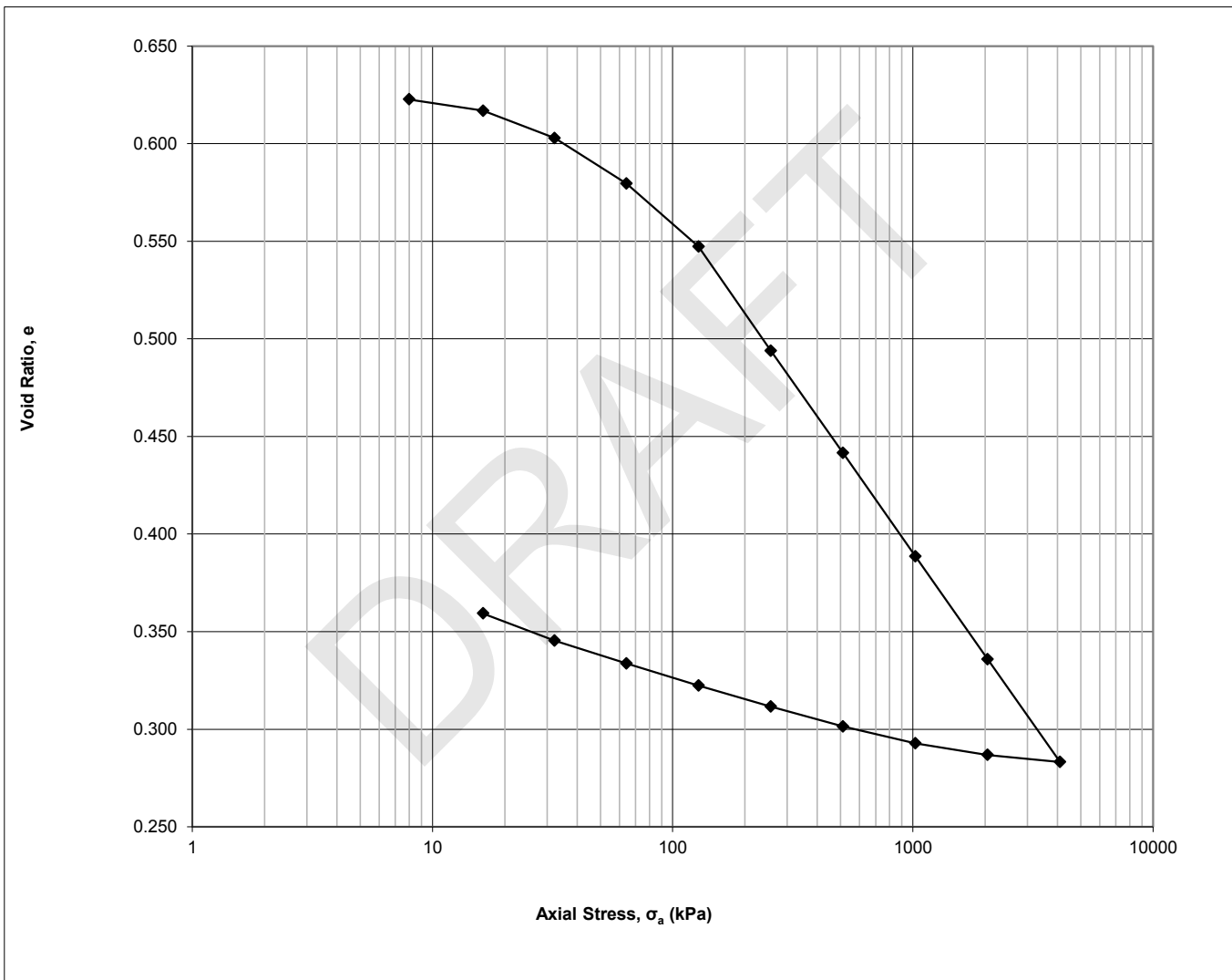


Figure No. C7-A

**One-Dimensional Consolidation Test using Incremental Loading**  
**ASTM D2435/D2435M - 11**

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	Hwy 401/VMP
Borehole	BH 5
Sample No.	ST3
Depth	5-7 ft
Sample Date	11/04/2016
Test Number	Two
Technician Name	Daniel Boateng

**Soil Description & Classification**

Silty Clay Till	
Specific Gravity of Solids	2.735
Average water content of trimmings %	14
<b>Additional Notes (information source, occurrence and size of large isolated particles etc.)</b>	

**Initial Specimen Conditions**

Height	mm	20.00
Diameter	mm	50.00
Area	mm <sup>2</sup>	1963
Volume	mm <sup>3</sup>	39270
Mass	g	75.22
Dry Mass	g	66.19
Density	Mg/m <sup>3</sup>	1.915
Dry Density	Mg/m <sup>3</sup>	1.686
Water Content	%	13.64
Degree of Saturation	%	59.9
Height of Solids	mm	12.33
Initial Void Ratio		0.623

**Final Specimen Conditions**

Water Content	%	9.58
Final Void Ratio		0.359
Degree of Saturation	%	72.9
Differential Height	mm	0.75
Estimated Preconsolidation Stress	kPa	

Figure No. C7-B

## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	Hwy 401/VMP
Borehole	BH 5
Sample No.	ST3
Depth	5-7 ft
Sample Date	11/04/2016
Test Number	Two
Technician Name	Daniel Boateng

**Test Procedure**

Date Started	15/04/2016
Date Finished	16/04/2016
Machine Number	D
Cell Number	D
Ring Number	D
Trimming Procedure	Turntable
Moisture Condition	Inundated
Axial Stress at Inundation	8 kPa
Water Used	Distilled
Test Method	B
Interpretation Procedure for $c_v$	2

**All Departures from Outlined ASTM D2435/D2435M-11 Procedure**
**Calculations**

Load Increment	Increment Duration	Axial Stress	Corrected Deformation	Specimen Height	Axial Strain	Void Ratio
	min	$\sigma_a$ kPa	$\Delta H$ mm	H mm	$\epsilon_a$ %	e
Seating	70.0	8	0.0000	20.0000	0.00	0.623
1	73.3	16	0.0715	19.9285	0.36	0.617
2	93.0	32	0.2441	19.7559	1.22	0.603
3	93.0	64	0.5306	19.4694	2.65	0.580
4	105.0	128	0.9293	19.0707	4.65	0.547
5	113.5	256	1.5873	18.4127	7.94	0.494
6	113.5	512	2.2328	17.7672	11.16	0.441
7	122.0	1024	2.8851	17.1149	14.43	0.389
8	122.0	2048	3.5348	16.4652	17.67	0.336
9	119.0	4096	4.1841	15.8159	20.92	0.283
10	70.0	2048	4.1393	15.8607	20.70	0.287
11	71.8	1024	4.0662	15.9338	20.33	0.293
12	76.8	512	3.9600	16.0400	19.80	0.301
13	92.0	256	3.8345	16.1655	19.17	0.312
14	102.0	128	3.7017	16.2983	18.51	0.322
15	115.5	64	3.5614	16.4386	17.81	0.334
16	131.3	32	3.4184	16.5816	17.09	0.345
17	162.0	16	3.2458	16.7542	16.23	0.359
18						
19						

Figure No. C7-C

## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	Hwy 401/VMP
Borehole	BH 5
Sample No.	ST3
Depth	5-7 ft
Sample Date	11/04/2016
Test Number	Two
Technician Name	Daniel Boateng

**Calculations**

Load Increment	Axial Stress $\sigma_a$ , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation $\Delta H_{50}$ mm	Specimen Height $H_{50}$ mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio $e_{50}$	Time $t_{50}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s	Time $t_{90}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s
Seating	8								
1	16	0.0311	19.9689	0.16	0.620			148	5.71E-01
2	24	0.1571	19.8429	0.79	0.610			241	3.46E-01
3	48	0.3855	19.6145	1.93	0.591			173	4.71E-01
4	96	0.7333	19.2667	3.67	0.563			134	5.89E-01
5	192	1.2962	18.7038	6.48	0.517			171	4.35E-01
6	384	1.9284	18.0716	9.64	0.466			299	2.32E-01
7	768	2.5381	17.4619	12.69	0.417			334	1.94E-01
8	1536	3.1803	16.8197	15.90	0.365			371	1.62E-01
9	3072	3.8368	16.1632	19.18	0.311			415	1.33E-01
10	3072	4.1454	15.8546	20.73	0.286				
11	1536	4.0832	15.9168	20.42	0.291				
12	768	3.9868	16.0132	19.93	0.299				
13	384	3.8747	16.1253	19.37	0.308				
14	192	3.8149	16.1851	19.07	0.313				
15	96	3.6903	16.3097	18.45	0.323				
16	48	3.5391	16.4609	17.70	0.336				
17	24	3.3828	16.6172	16.91	0.348				
18									
19									

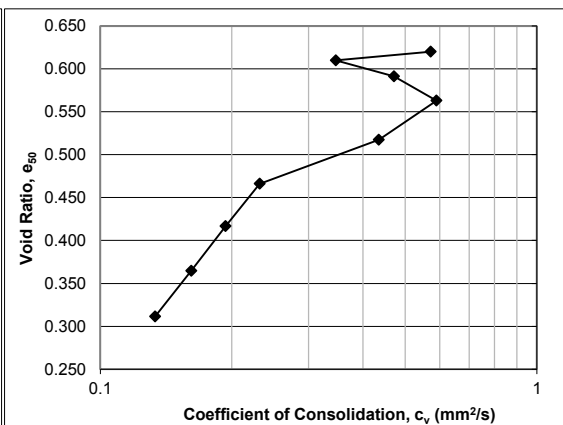
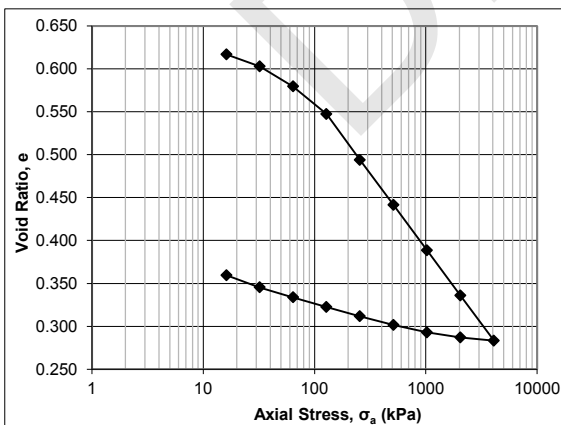
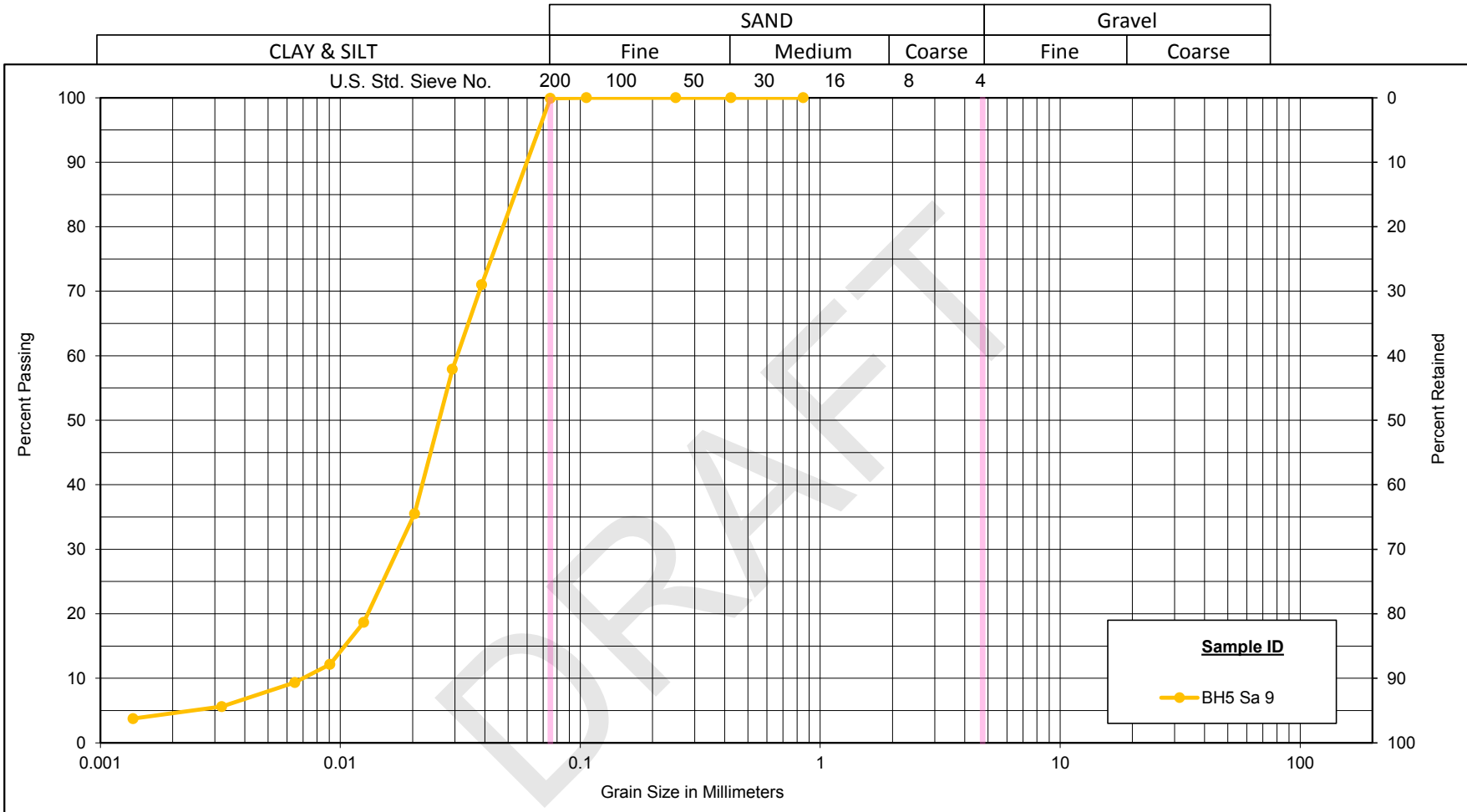


Figure No. C7-D

# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH5 Sa 9	9.1-9.8	0.0	0.1	95.9	4.0



## GRAIN SIZE DISTRIBUTION

### Upper Silt (ML)

Figure No. C8

Project No. 165001002.260

Project  
Project No.  
Borehole No.  
Sample No.  
Sample Depth

Design-Build HWY 401/VMP  
165001002.260  
BH 2  
ST12  
55-57 ft

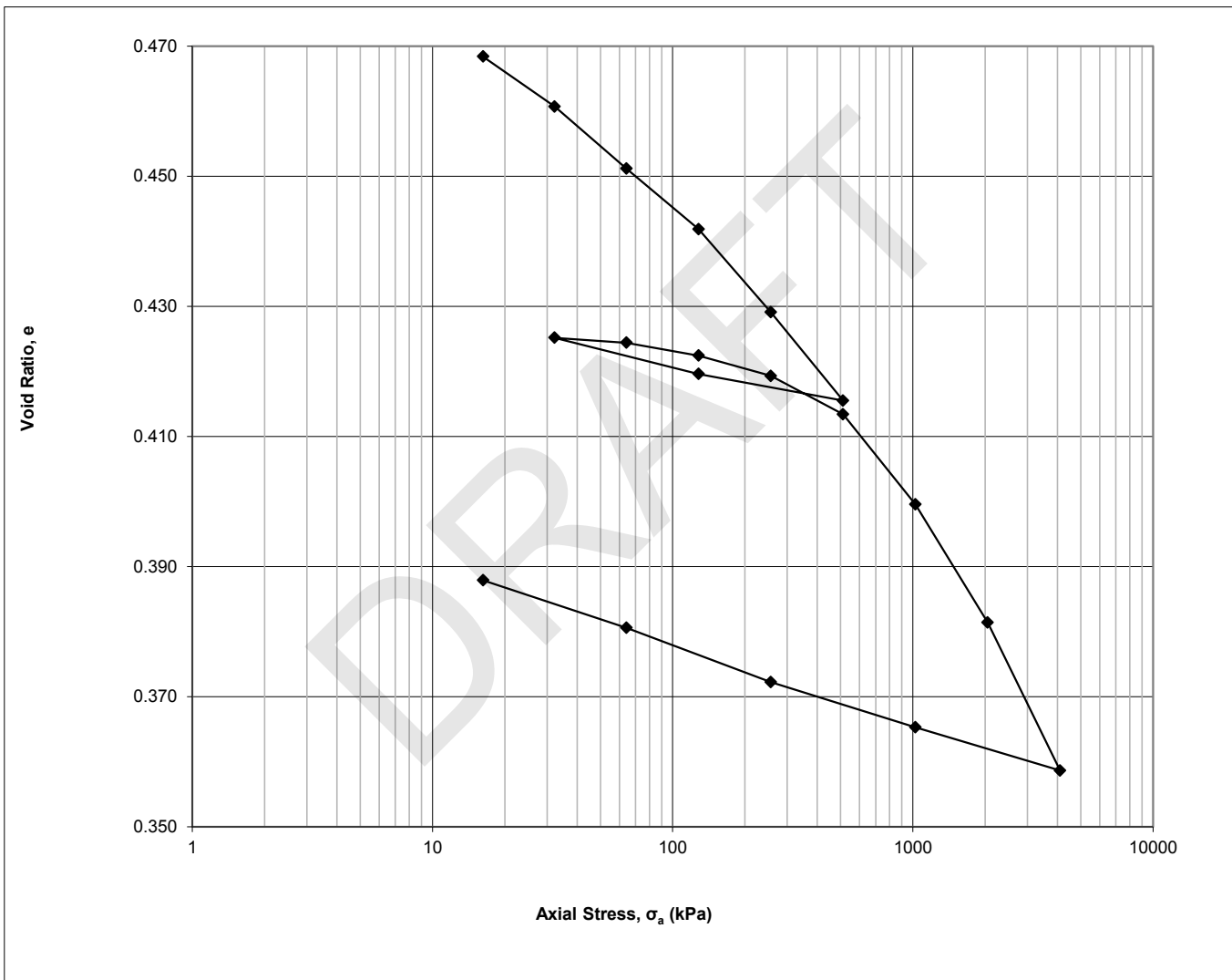


Figure No. C9-A

**One-Dimensional Consolidation Test using Incremental Loading**  
**ASTM D2435/D2435M - 11**

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	HWY 401
Borehole	BH 2
Sample No.	ST12
Depth	55-57 ft
Sample Date	07/04/2016
Test Number	Three
Technician Name	Daniel Boateng

**Soil Description & Classification**

Silt	
Specific Gravity of Solids	2.758
Average water content of trimmings %	16
<b>Additional Notes (information source, occurrence and size of large isolated particles etc.)</b>	

**Grain Size Distribution**

Gravel, retained on No. 4 Sieve	%	
Coarse Sand, retained on No. 10 Sieve	%	
Medium Sand, retained on No. 40 Sieve	%	
Fine Sand, retained on No. 200 Sieve	%	
Silt, 0.074 to 0.005 mm	%	
Clay, smaller than 0.005 mm	%	
Colloids, smaller than 0.001 mm	%	

**Initial Specimen Conditions**

Height	mm	20.00
Diameter	mm	50.00
Area	mm <sup>2</sup>	1963
Volume	mm <sup>3</sup>	39270
Mass	g	84.08
Dry Mass	g	72.75
Density	Mg/m <sup>3</sup>	2.141
Dry Density	Mg/m <sup>3</sup>	1.853
Water Content	%	15.57
Degree of Saturation	%	87.9
Height of Solids	mm	13.43
Initial Void Ratio		0.489

**Final Specimen Conditions**

Water Content	%	15.18
Final Void Ratio		0.388
Degree of Saturation	%	107.9
Differential Height	mm	2.64
Estimated Preconsolidation Stress	kPa	

Figure No. C9-B



## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	HWY 401
Borehole	BH 2
Sample No.	ST12
Depth	55-57 ft
Sample Date	07/04/2016
Test Number	Three
Technician Name	Daniel Boateng

**Test Procedure**

Date Started	19/04/2016
Date Finished	20/04/2016
Machine Number	C
Cell Number	C
Ring Number	C
Trimming Procedure	Tunable
Moisture Condition	Inundated
Axial Stress at Inundation	16 kPa
Water Used	Distilled
Test Method	B
Interpretation Procedure for $c_v$	2

**All Departures from Outlined ASTM D2435/D2435M-11 Procedure**
**Calculations**

Load Increment	Increment Duration	Axial Stress	Corrected Deformation	Specimen Height	Axial Strain	Void Ratio
	min	$\sigma_a$ kPa	$\Delta H$ mm	H mm	$\epsilon_a$ %	e
Seating	0.0	16	0.0000	20.0000	0.00	0.489
1	70.0	16	0.2735	19.7265	1.37	0.468
2	73.0	32	0.3768	19.6232	1.88	0.461
3	74.8	64	0.5048	19.4952	2.52	0.451
4	73.0	128	0.6300	19.3700	3.15	0.442
5	76.5	256	0.8016	19.1984	4.01	0.429
6	79.8	512	0.9839	19.0161	4.92	0.416
7	70.0	128	0.9290	19.0710	4.65	0.420
8	70.0	32	0.8541	19.1459	4.27	0.425
9	70.0	64	0.8644	19.1356	4.32	0.424
10	70.0	128	0.8909	19.1091	4.45	0.422
11	69.8	256	0.9330	19.0670	4.67	0.419
12	71.5	512	1.0125	18.9875	5.06	0.413
13	79.8	1024	1.1984	18.8016	5.99	0.400
14	83.0	2048	1.4421	18.5579	7.21	0.381
15	91.5	4096	1.7478	18.2522	8.74	0.359
16	70.0	1024	1.6587	18.3413	8.29	0.365
17	70.0	256	1.5651	18.4349	7.83	0.372
18	70.0	64	1.4533	18.5467	7.27	0.381
19	70.0	16	1.3551	18.6449	6.78	0.388

Figure No. C9-C

## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Design-Build HWY 401/VMP
Project Location	HWY 401
Borehole	BH 2
Sample No.	ST12
Depth	55-57 ft
Sample Date	07/04/2016
Test Number	Three
Technician Name	Daniel Boateng

**Calculations**

Load Increment	Axial Stress $\sigma_a$ , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation $\Delta H_{50}$ mm	Specimen Height $H_{50}$ mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio $e_{50}$	Time $t_{50}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s	Time $t_{90}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s
Seating	8								
1	16	0.2619	19.7381	1.31	0.469			60	1.38E+00
2	24	0.3356	19.6644	1.68	0.464			112	7.32E-01
3	48	0.4657	19.5343	2.33	0.454			152	5.34E-01
4	96	0.5881	19.4119	2.94	0.445			155	5.16E-01
5	192	0.7437	19.2563	3.72	0.433			124	6.32E-01
6	384	0.9244	19.0756	4.62	0.420			126	6.14E-01
7	320	0.9325	19.0675	4.66	0.419				
8	80	0.8617	19.1383	4.31	0.425				
9	48	0.8620	19.1380	4.31	0.425			384	2.02E-01
10	96	0.8859	19.1141	4.43	0.423			240	3.22E-01
11	192	0.9271	19.0729	4.64	0.420			278	1.12E-05
12	384	0.9891	19.0109	4.95	0.415			325	2.36E-01
13	768	1.1389	18.8611	5.69	0.404			149	5.05E-01
14	1536	1.3743	18.6257	6.87	0.386			371	1.98E-01
15	3072	1.6483	18.3517	8.24	0.366			194	3.68E-01
16	2560	1.6647	18.3353	8.32	0.365				
17	640	1.5798	18.4202	7.90	0.371				
18	160	1.4661	18.5339	7.33	0.380				
19	40	1.3723	18.6277	6.86	0.387				

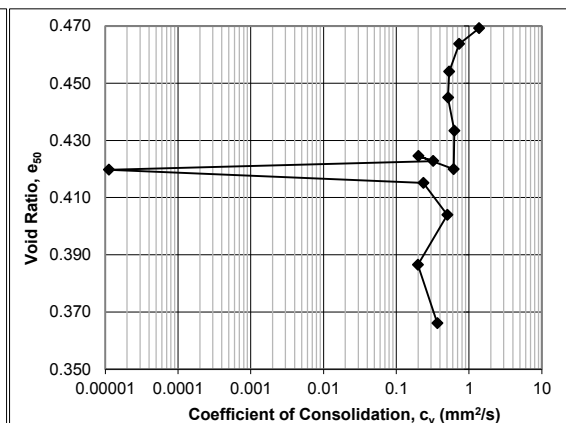
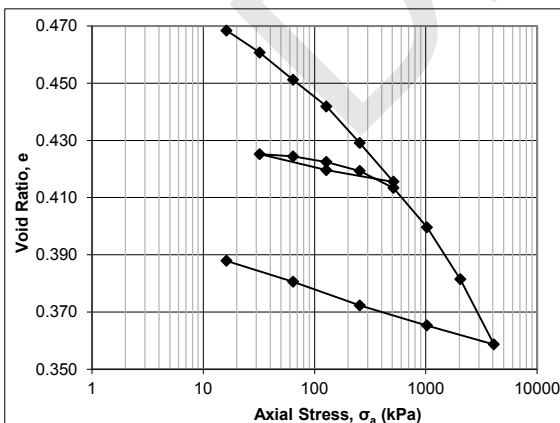
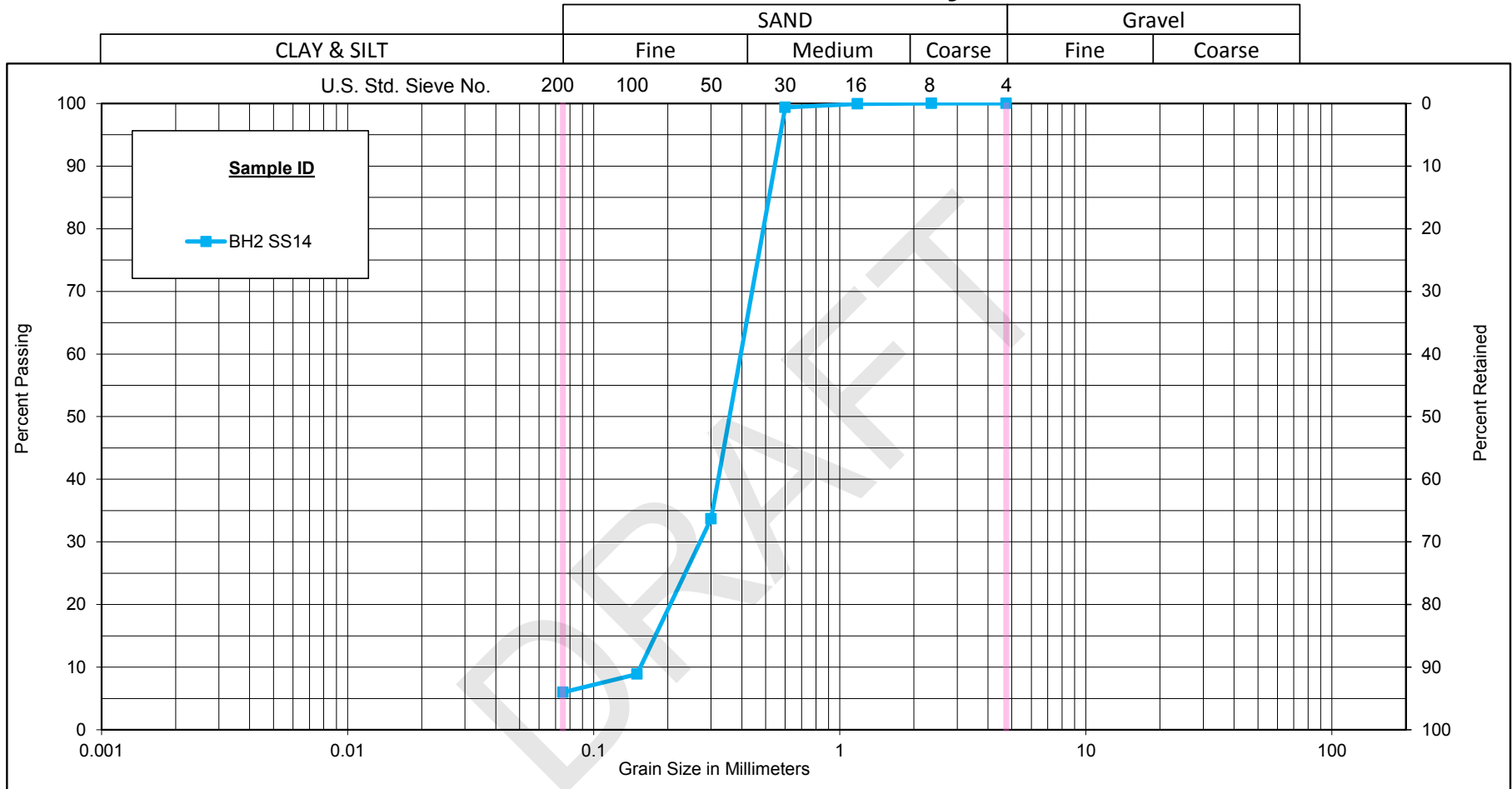


Figure No. C9-D

# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt & Clay
BH2 SS14	19.8-20.4	0.0	94.0	6.0

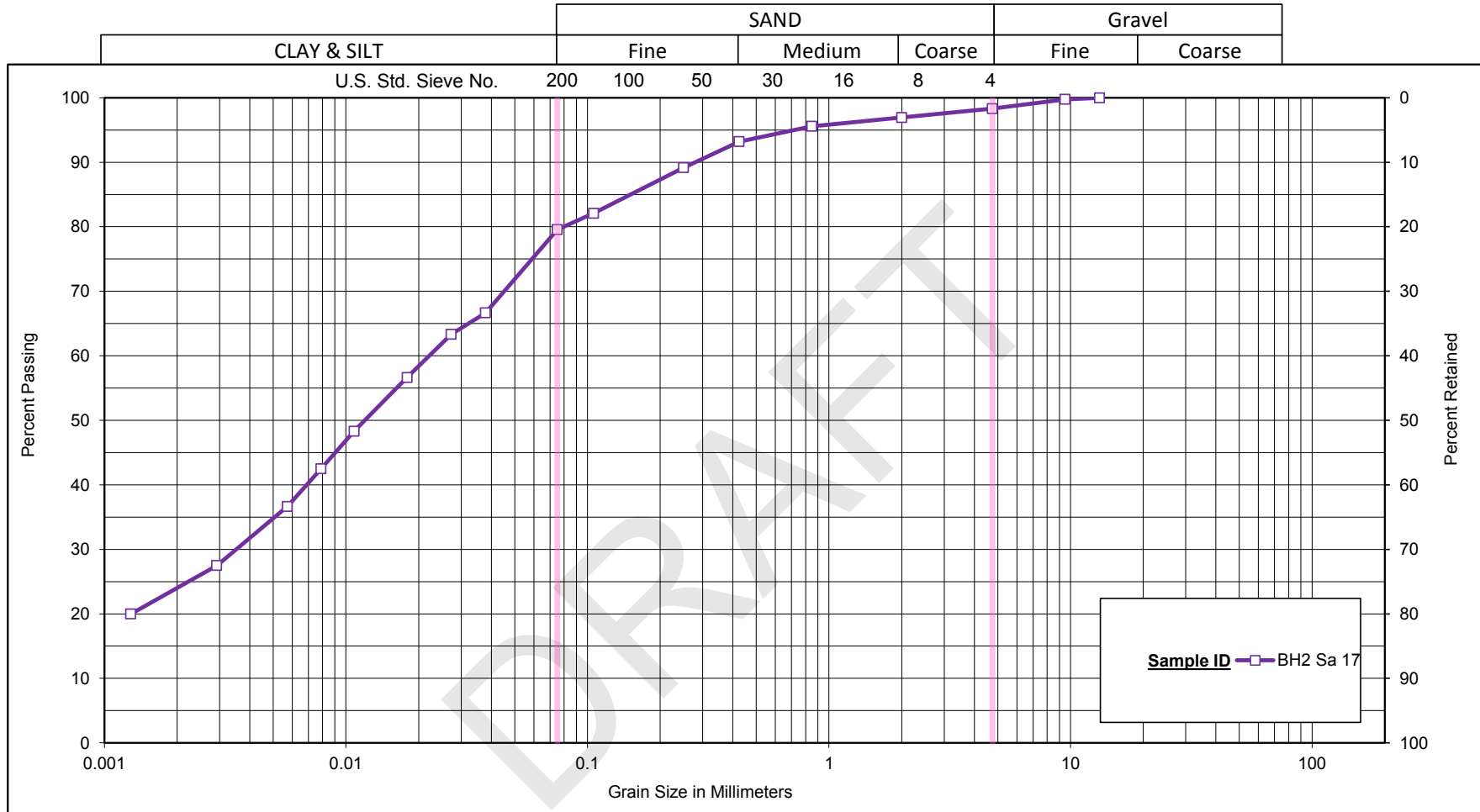


GRAIN SIZE DISTRIBUTION  
Upper Sand (SP)

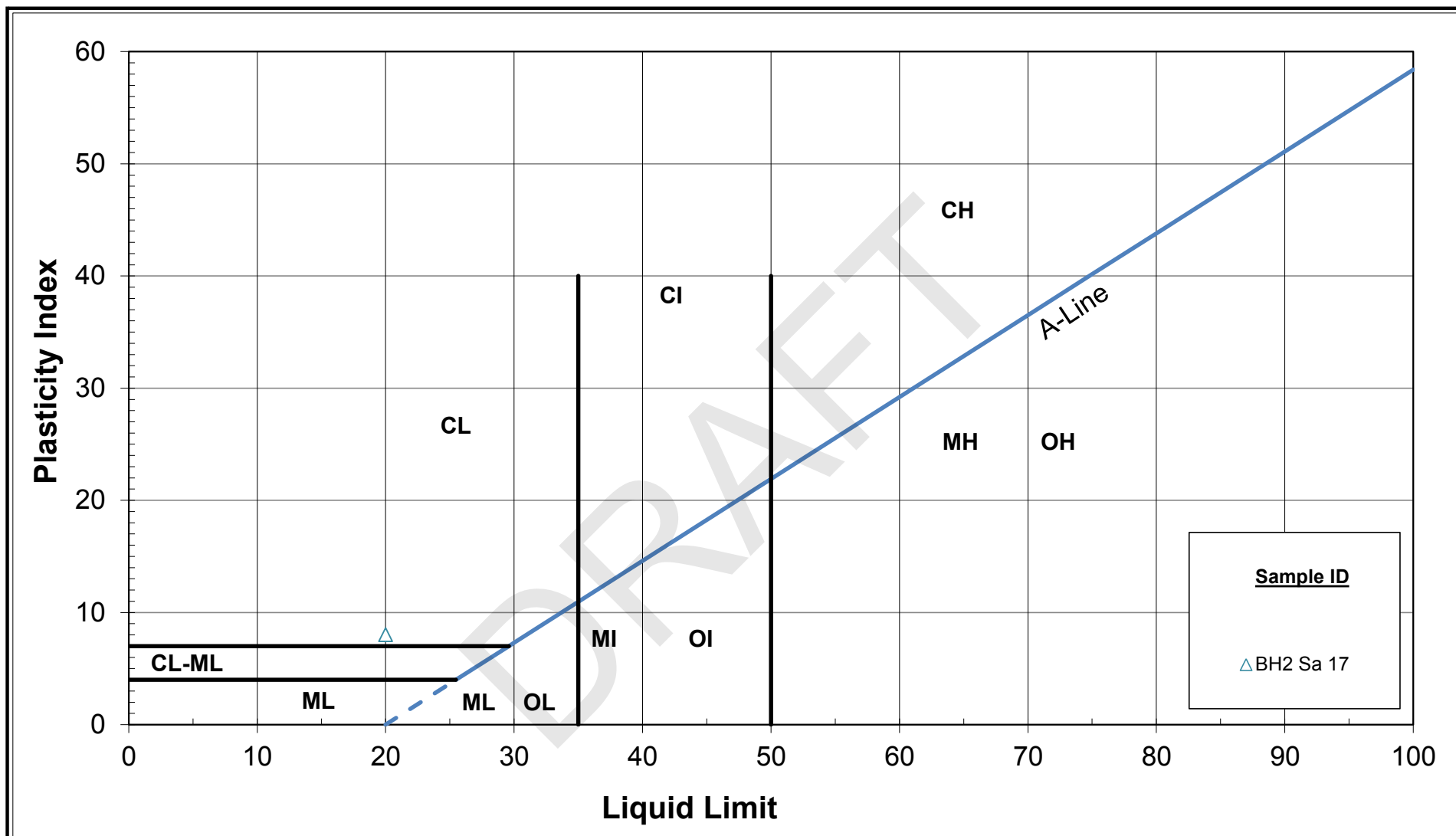
Figure No. C10

Project No. 165001002.260

# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH2 Sa 17	25.9-26.5	2.0	18.4	56.6	23.0



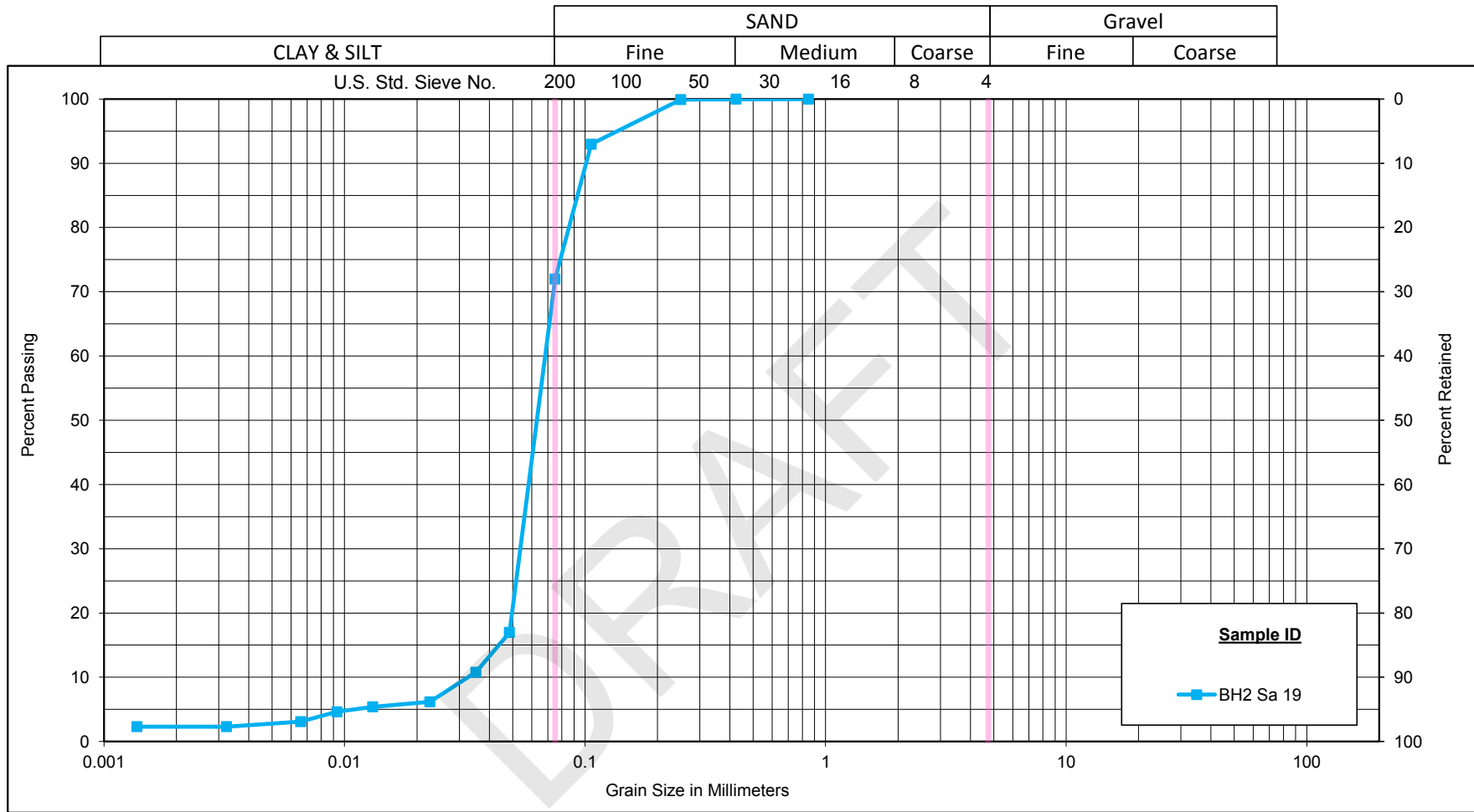
# PLASTICITY CHART

Middle Clayey Silt TILL (CL)

Figure No. C12

Project No. 165001002.26

# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH2 Sa 19	32.0-32.6	0.0	28.0	70.0	2.0

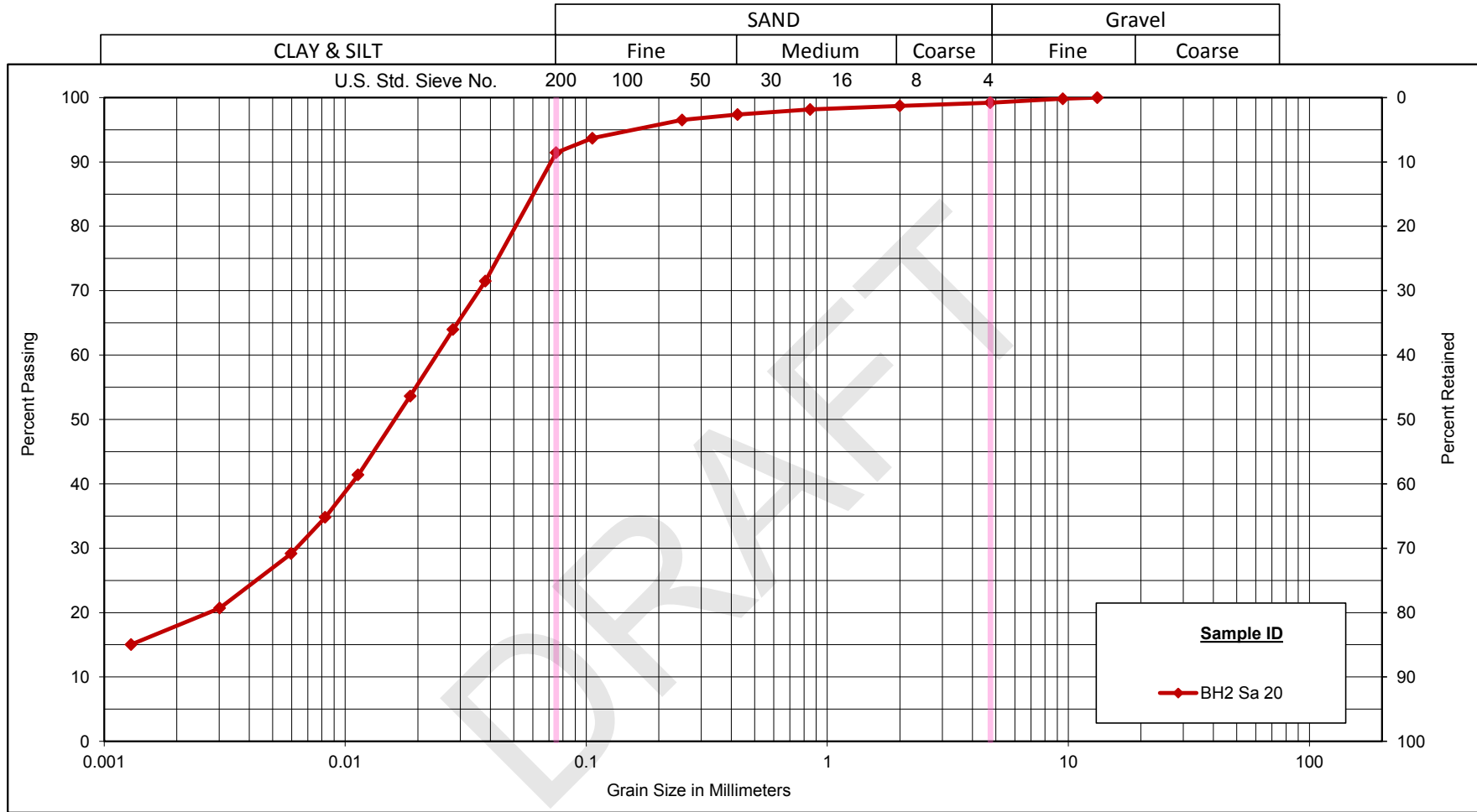


**GRAIN SIZE DISTRIBUTION**  
Sandy Silt (ML)

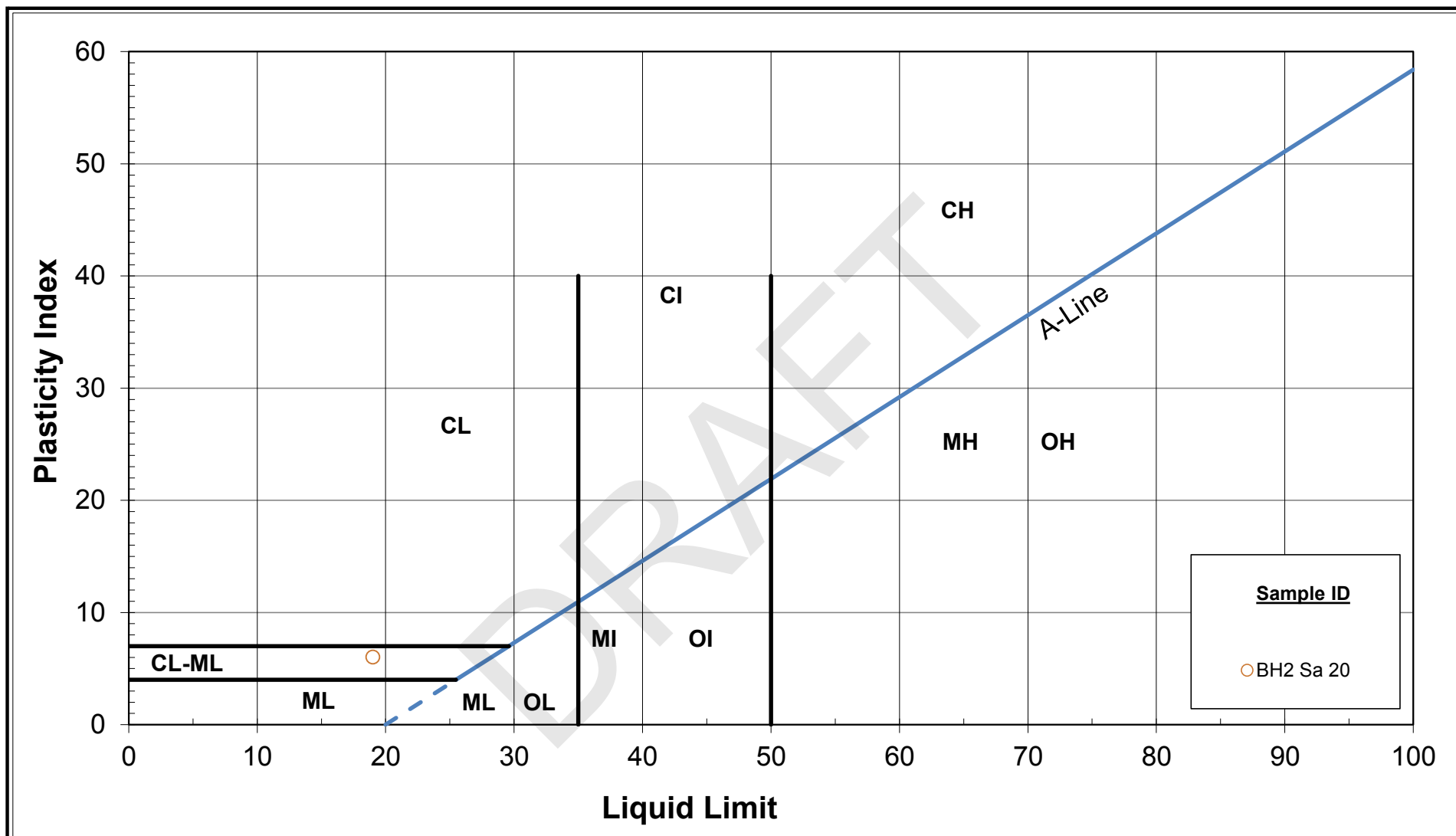
Figure No. C13

Project No. 165001002.260

# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH2 Sa 20	35.0-35.7	1.0	7.6	74.4	17.0



# PLASTICITY CHART

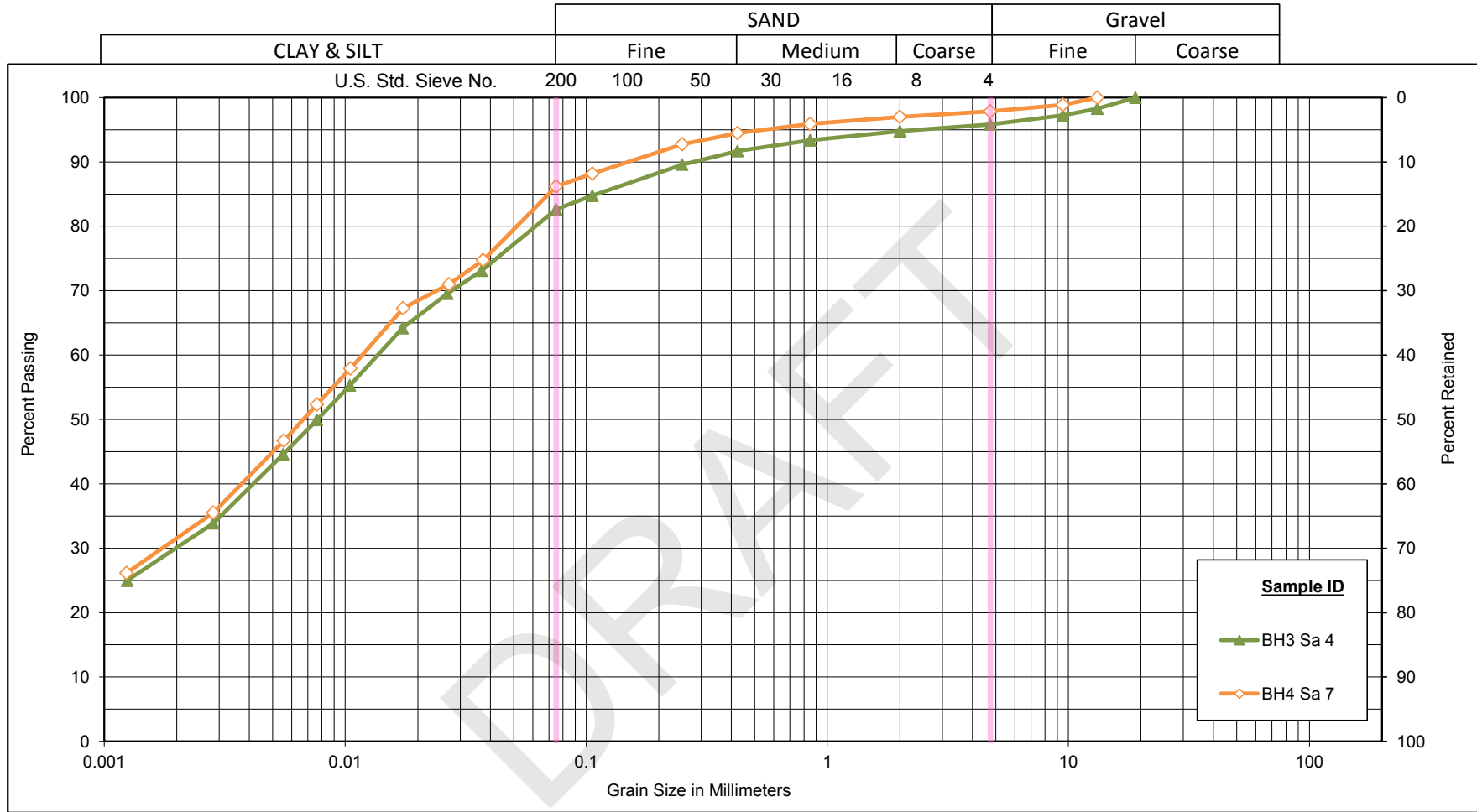
## Lower Clayey Silt TILL (CL)

Figure No. C15

Project No. 165001002.26



# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH3 Sa 4	2.3-2.9	4.0	13.4	53.6	29.0
BH4 Sa 7	6.1-6.7	2.0	11.8	55.2	31.0

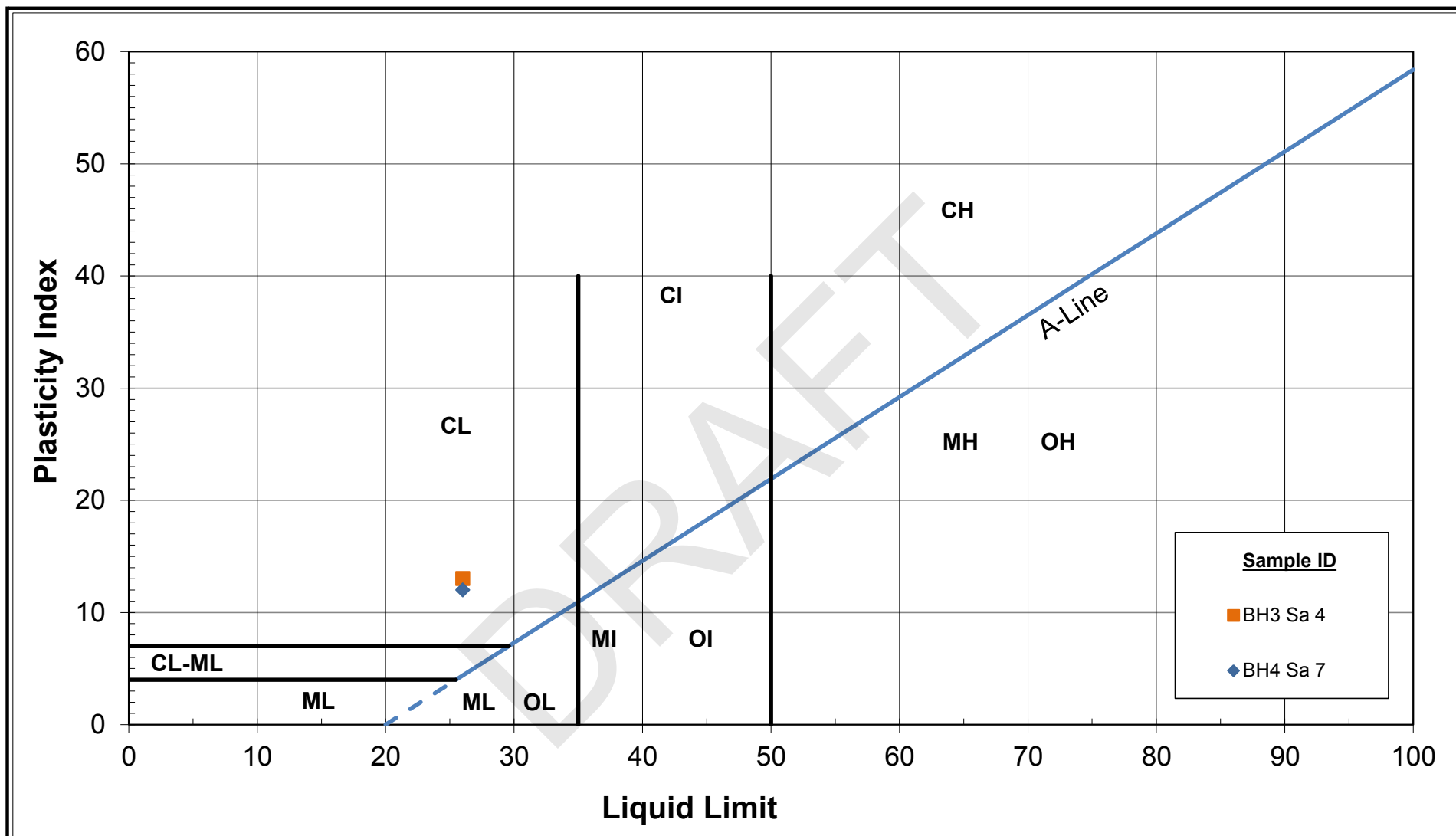


## GRAIN SIZE DISTRIBUTION

Clayey Silt TILL (CL)  
Culvert Site

Figure No. C16

Project No. 165001002.260



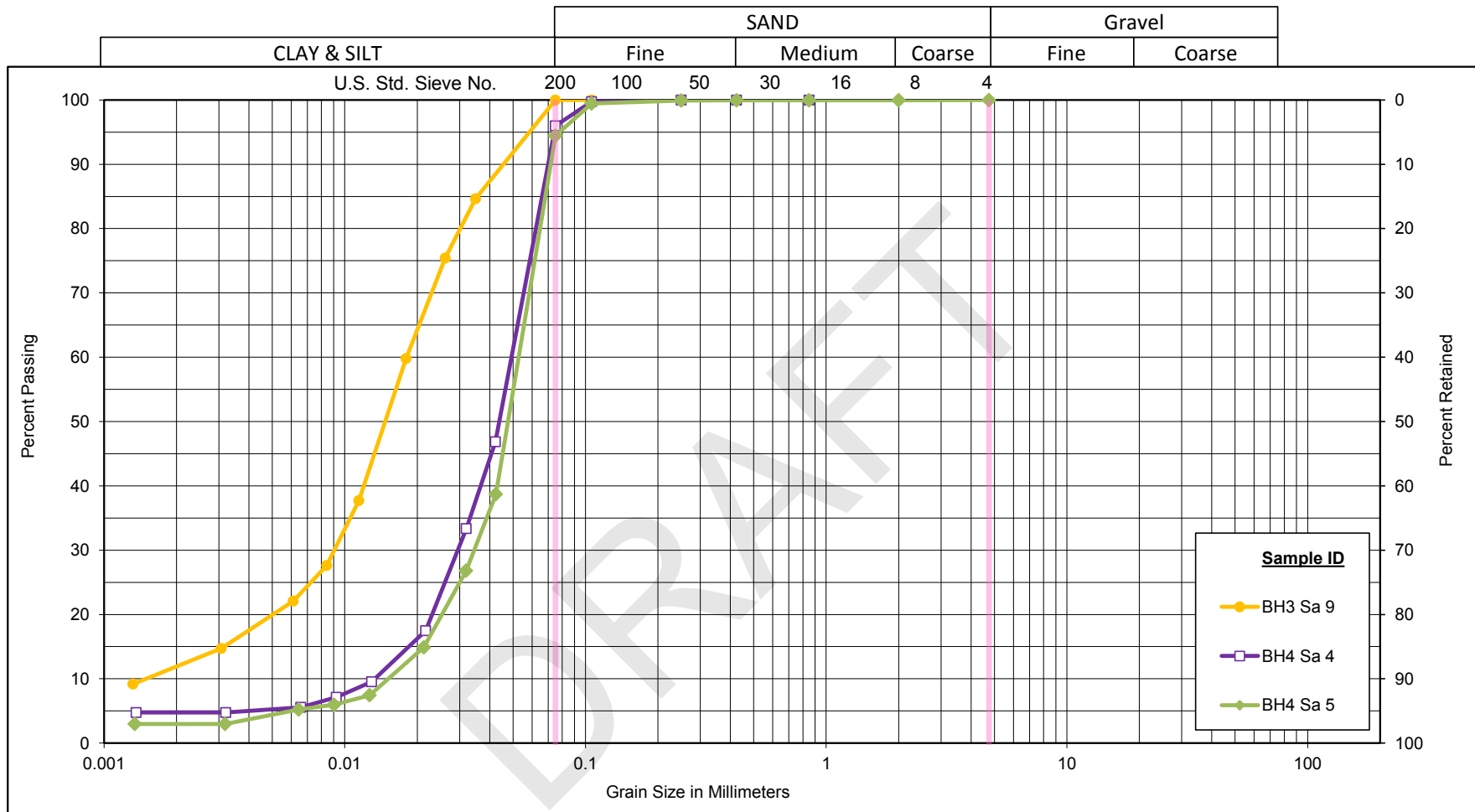
# PLASTICITY CHART

Clayey Silt TILL (CL)  
Culvert Site

Figure No. C17

Project No. 165001002.26

# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH3 Sa 9	9.1-9.8	0.0	0.0	89.0	11.0
BH4 Sa 4	2.3-2.9	0.0	4.0	91.0	5.0
BH4 Sa 5	3.0-3.7	0.0	5.6	91.4	3.0



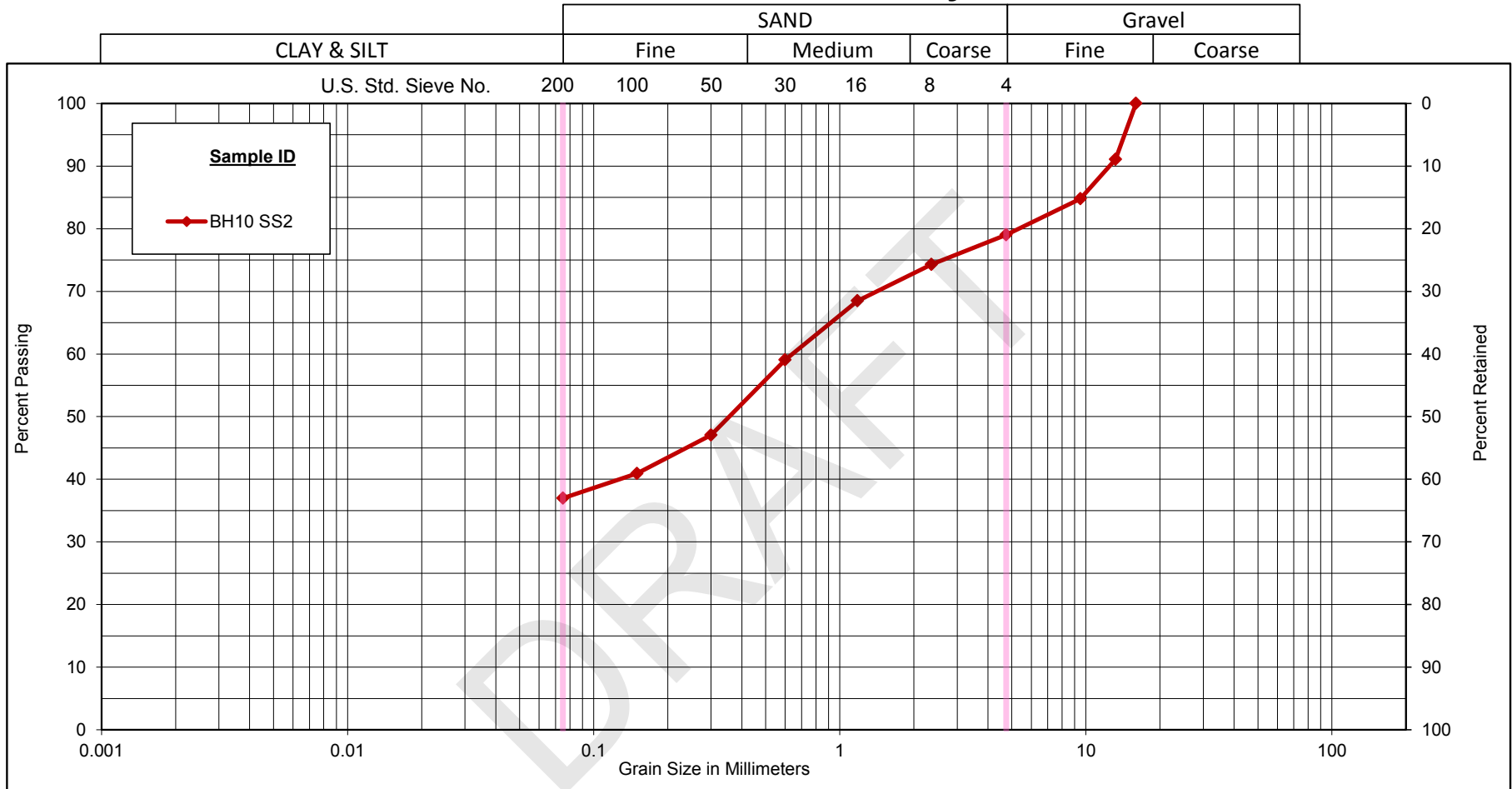
## GRAIN SIZE DISTRIBUTION

Silt (ML)  
Culvert Site

Figure No. C18

Project No. 165001002.260

# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt & Clay
BH10 SS2	0.6-1.2	21.0	42.0	37.0



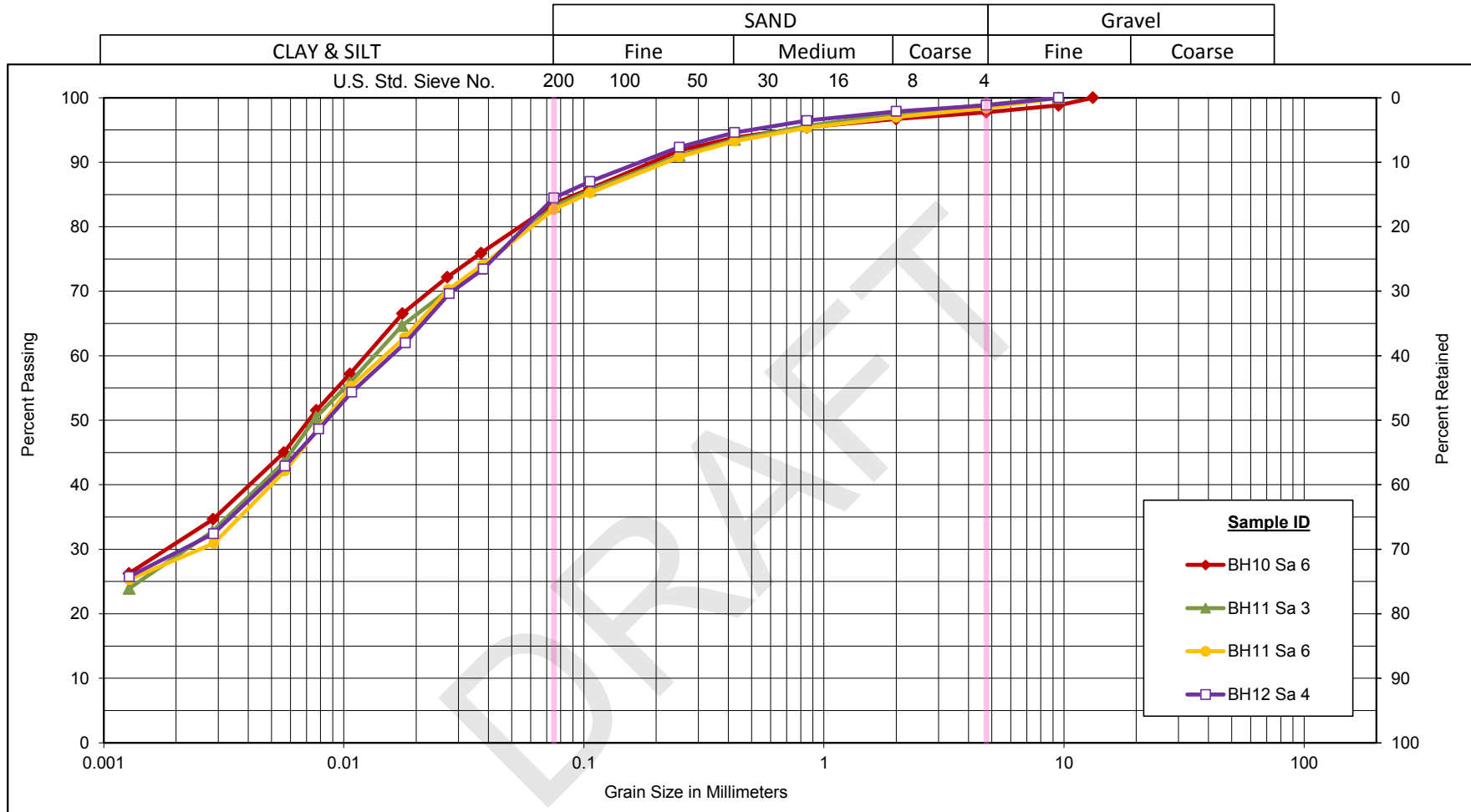
## GRAIN SIZE DISTRIBUTION

FILL: Sand and Gravel (SM)  
Overhead Sign Sites

Figure No. C19

Project No. 165001002.260

# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH10 Sa 6	4.6-5.2	2.0	14.4	53.6	30.0
BH11 Sa 3	1.5-2.1	1.0	15.9	55.1	28.0
BH11 Sa 6	4.6-5.2	2.0	15.3	54.7	28.0
BH12 Sa 4	2.3-2.9	1.0	14.5	55.5	29.0

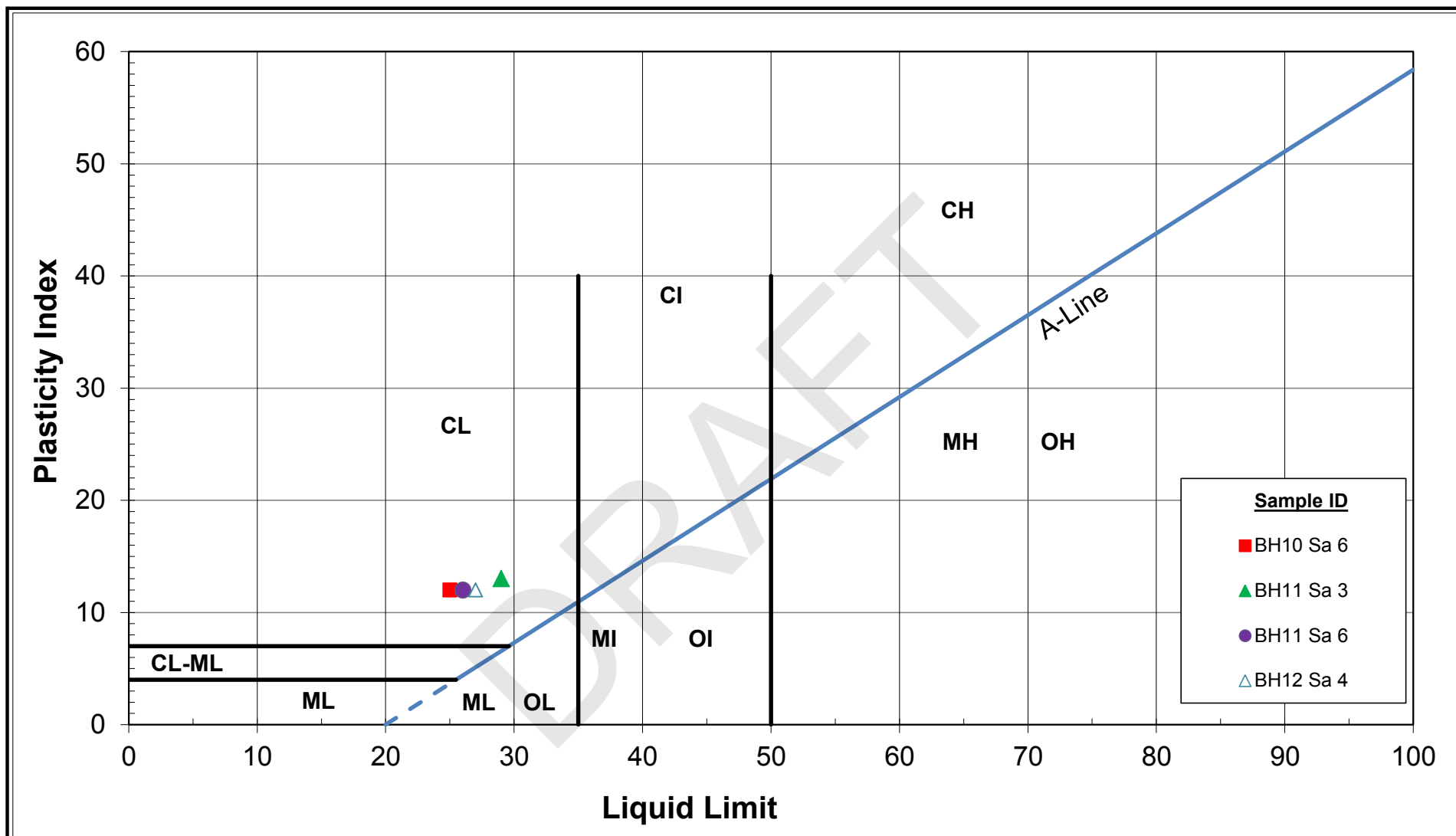


## GRAIN SIZE DISTRIBUTION

Clayey Silt TILL (CL)  
Overhead Sign Sites

Figure No. C20

Project No. 165001002.260



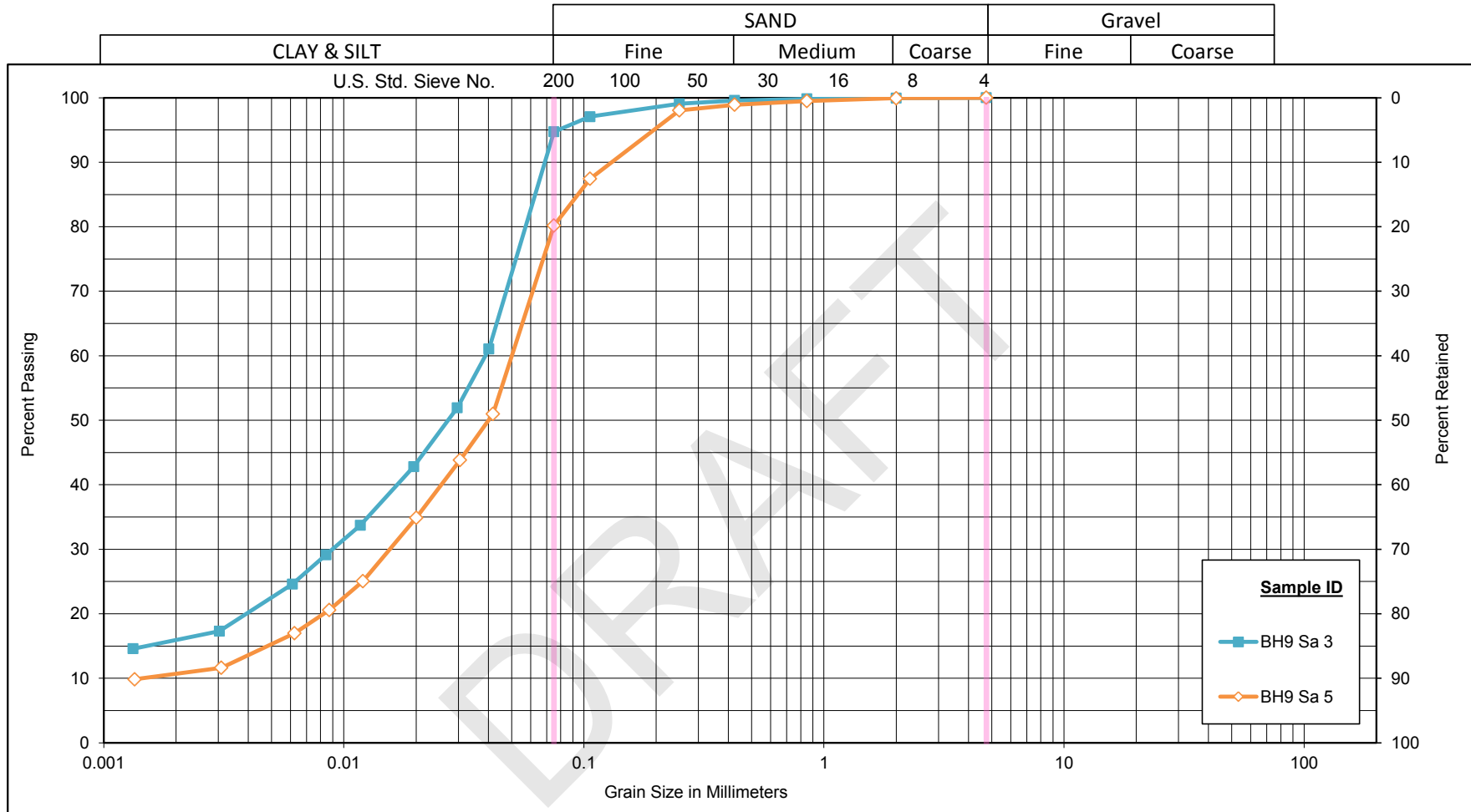
# PLASTICITY CHART

Clayey Silt TILL (CL)  
Overhead Sign Sites

Figure No. C21

Project No. 165001002.26

# Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH9 Sa 3	2.3-2.9	0.0	5.3	78.7	16.0
BH9 Sa 5	4.6-5.2	0.0	19.8	69.2	11.0

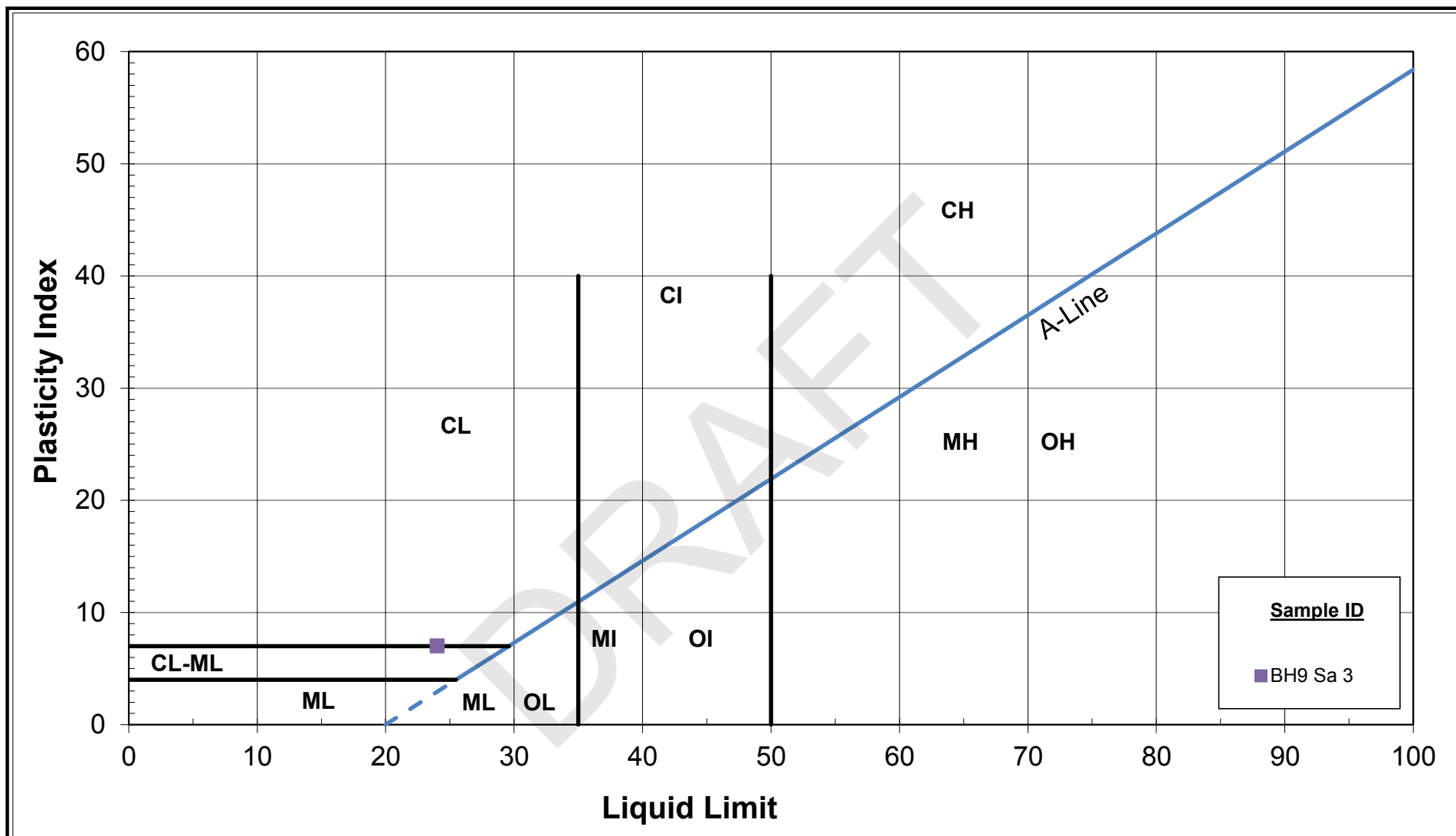


## GRAIN SIZE DISTRIBUTION

Sandy Silt (ML)  
Overhead Sign Sites

Figure No. C22

Project No. 165001002.260



# PLASTICITY CHART

Clayey Silt (CL)  
Overhead Sign Sites

Figure No. C23

Project No. 165001002.26





## Certificate of Analysis

AGAT WORK ORDER: 16W085374

PROJECT: Veteran Memorial Parkway IC

5835 COOPERS AVENUE  
MISSISSAUGA, ONTARIO  
CANADA L4Z 1Y2  
TEL (905)712-5100  
FAX (905)712-5122  
<http://www.agatlabs.com>

CLIENT NAME: STANTEC CONSULTING LTD.

SAMPLING SITE: HWY 401/VMP

ATTENTION TO: ALLEN MACGARVIE

SAMPLED BY: R. Cluthe

### Corrosivity Package

DATE RECEIVED: 2016-04-14

DATE REPORTED: 2016-04-21

SAMPLE DESCRIPTION: Borehole 4 SS4

SAMPLE TYPE: Soil

DATE SAMPLED: 4/11/2016

Parameter	Unit	G / S	RDL	7490981
Sulphide	%		0.05	0.06
Chloride (2:1)	µg/g		2	9
Sulphate (2:1)	µg/g		2	254
pH (2:1)	pH Units		NA	8.51
Electrical Conductivity (2:1)	mS/cm		0.005	0.333
Resistivity (2:1)	ohm.cm		1	3000
Redox Potential (2:1)	mV		5	243

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

7490981 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).

Certified By:

Amanjot Bhela

Certificate of Analysis  
**Client: Stantec Consulting Ltd. (Ottawa)**  
**Client PO:**

Report Date: 12-May-2016

Order Date: 10-May-2016

**Project Description: 165001002.260**

<b>Client ID:</b>	BH 8 7'-8'9"	-	-	-
<b>Sample Date:</b>	05-Apr-16	-	-	-
<b>Sample ID:</b>	1620143-01	-	-	-
<b>MDL/Units</b>	Soil	-	-	-

**Physical Characteristics**

% Solids	0.1 % by Wt.	86.0	-	-	-
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**General Inorganics**

pH	0.05 pH Units	8.23 [1]	-	-	-
Resistivity	0.10 Ohm.m	7.26	-	-	-

**Anions**

Chloride	5 ug/g dry	699 [1]	-	-	-
Sulphate	5 ug/g dry	29 [1]	-	-	-

DRAFT

## APPENDIX D

Figure D1 to D2 - Geotechnical Model

Figure D3 - Factored Axial Capacity of HP310x110

Figure D4 - Factored Axial Capacity of HP360x108

Figure D5 - Factored Axial Capacity of 1500 mm and 1800 mm Caissons

Table D1 - Load Intensity  $p$  (kN/m) vs Lateral Deflection  $y$  (m) of 1800 mm

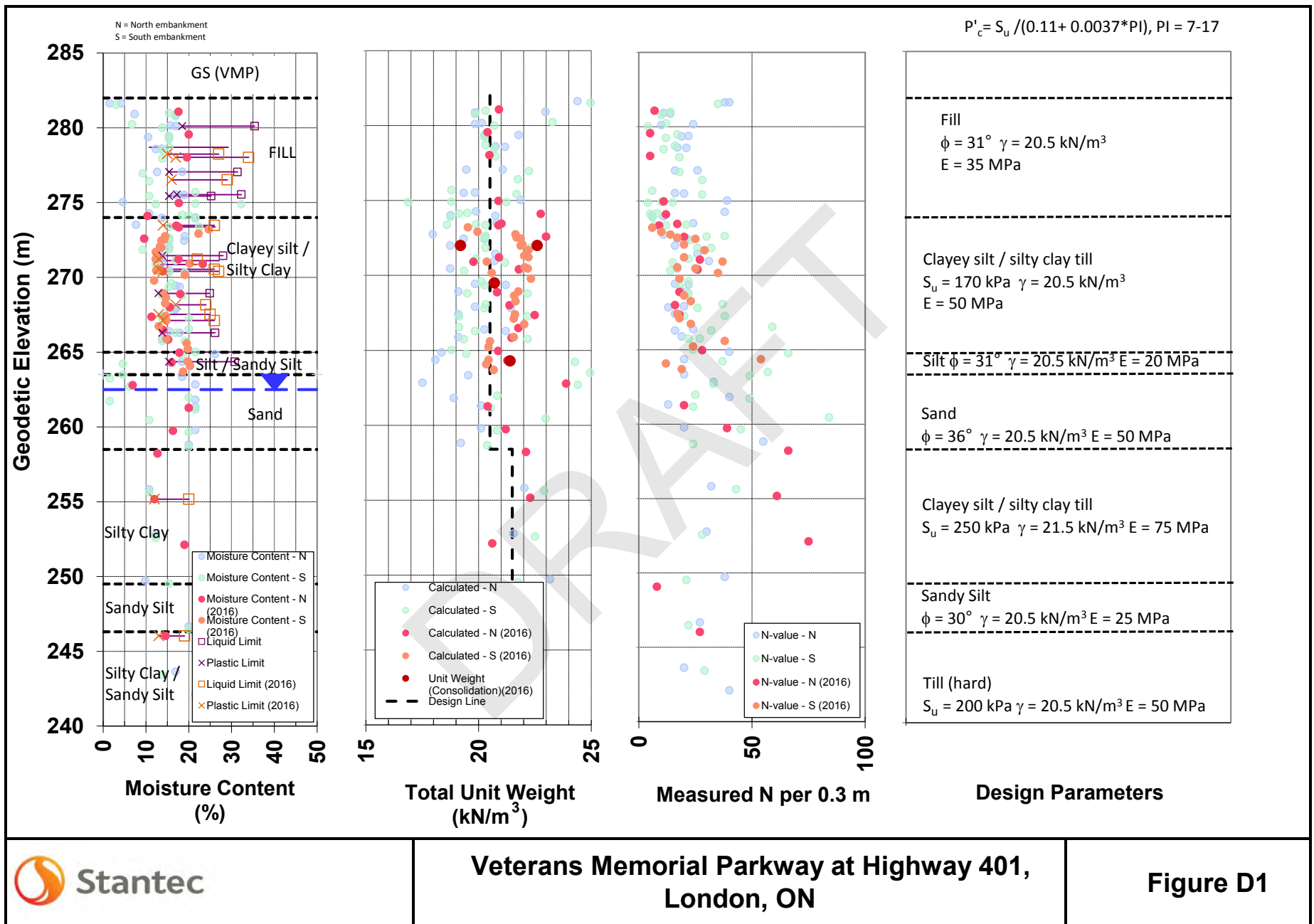
Caisson at Various Depths below Pile Head

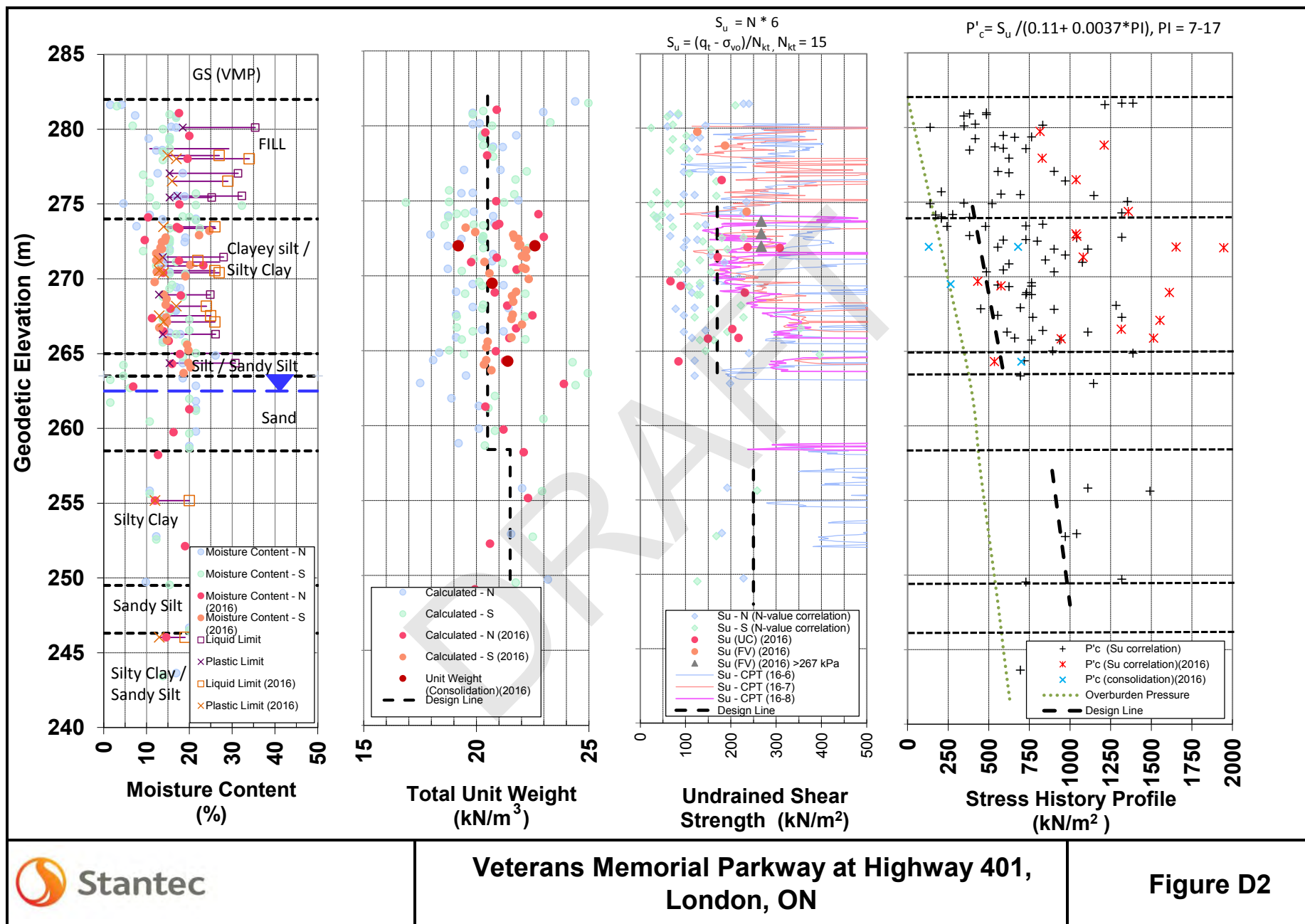
Figure D6 - Static Slope Stability Analysis (Drained) NW Ramp

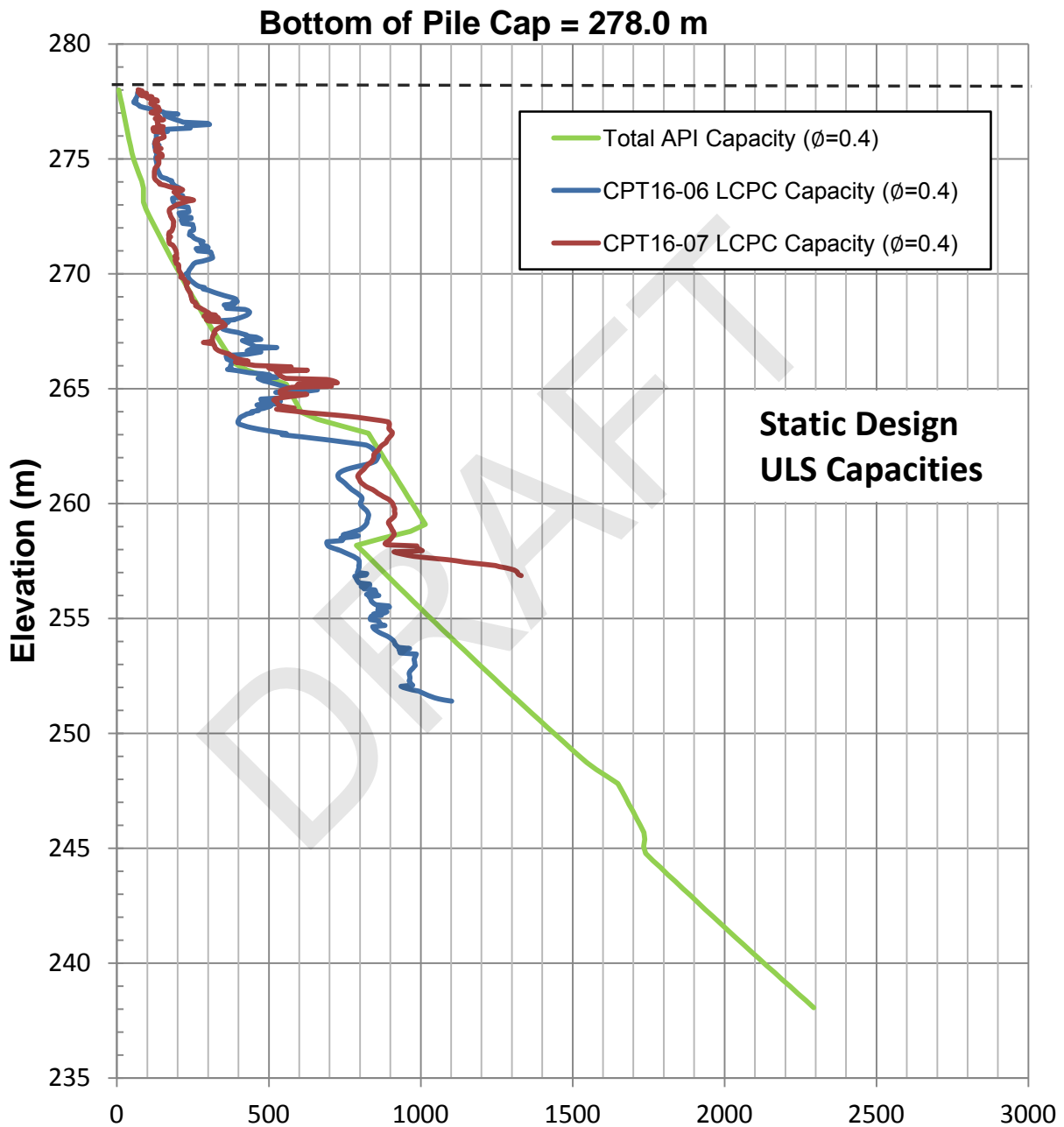
Figure D7 - Static Slope Stability Analysis (Drained) NW Ramp – RSS

Figure D8 - Static Slope Stability Analysis (Drained) VMP - RSS

Figure D9 - Static Slope Stability Analysis (Drained) Crinklaw Drain Culvert





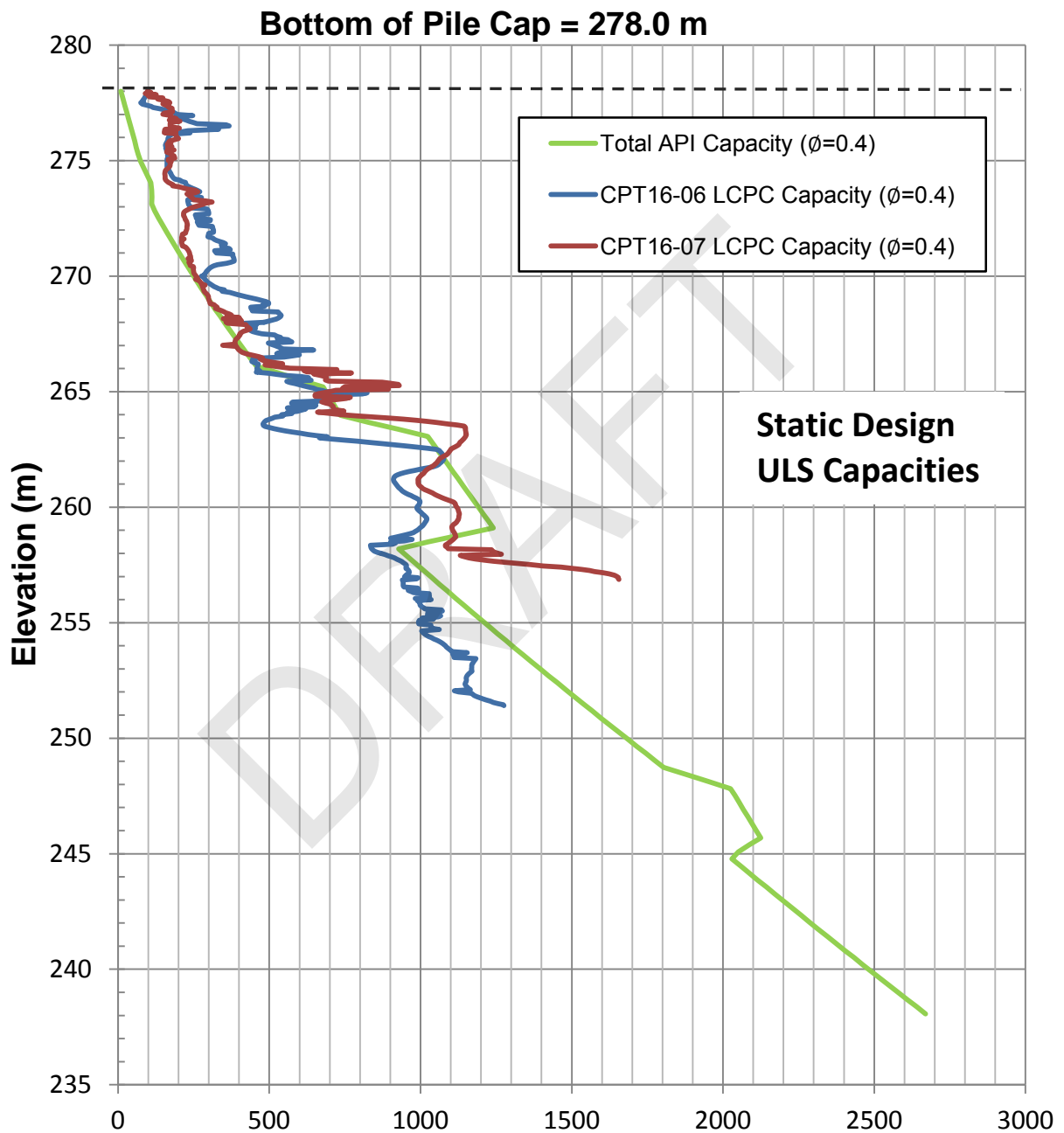


**Axial Capacity (kN)**

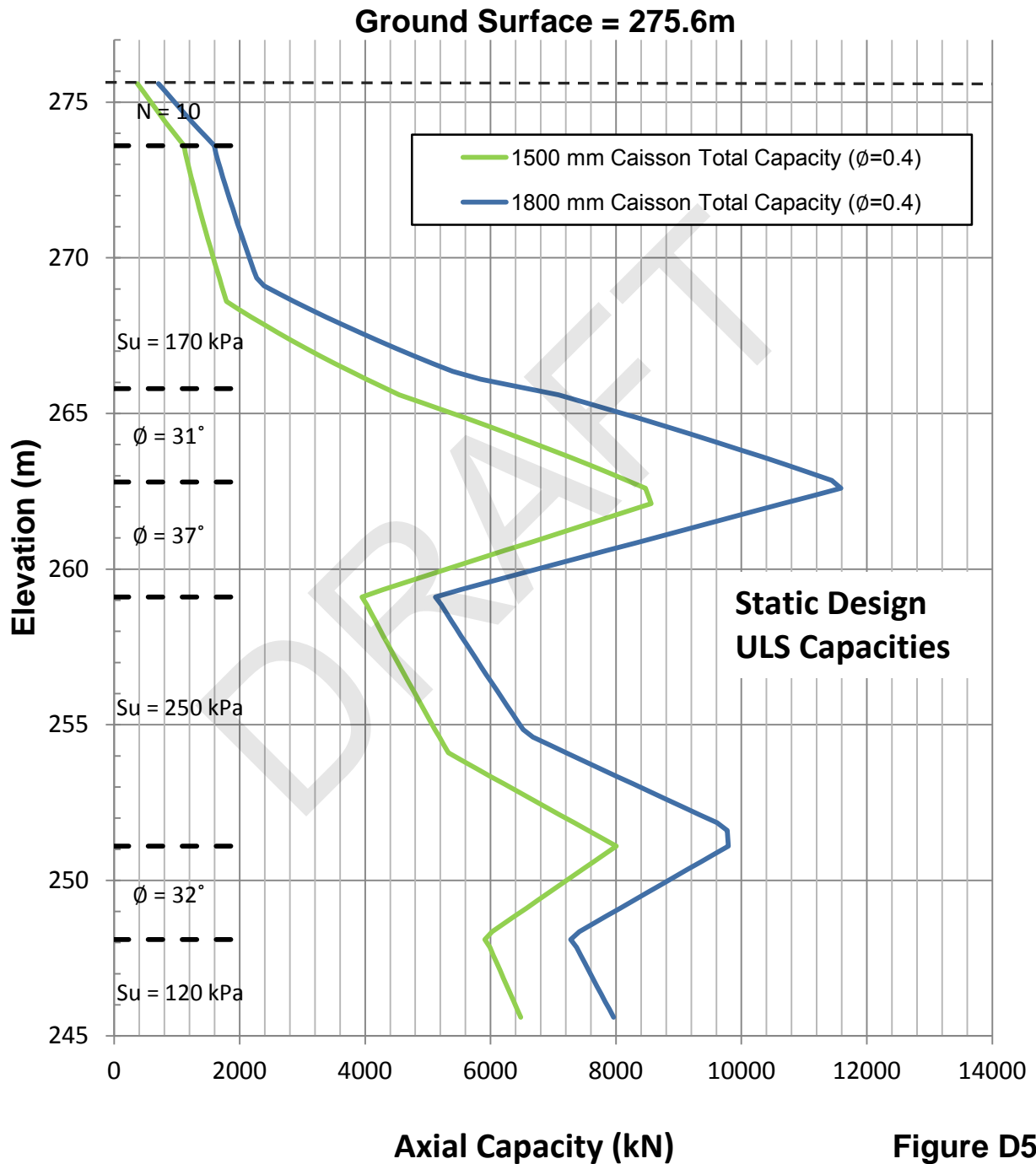
**Figure D3**

**Factored Axial Capacity of HP 310×110**

**HP 310×79**



**Figure D4**  
**Factored Axial Capacity of HP 360×108**



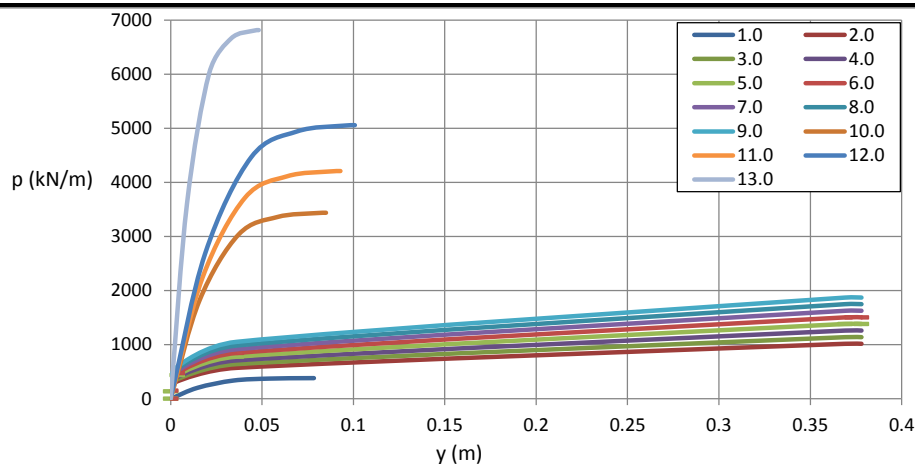
**Figure D5**

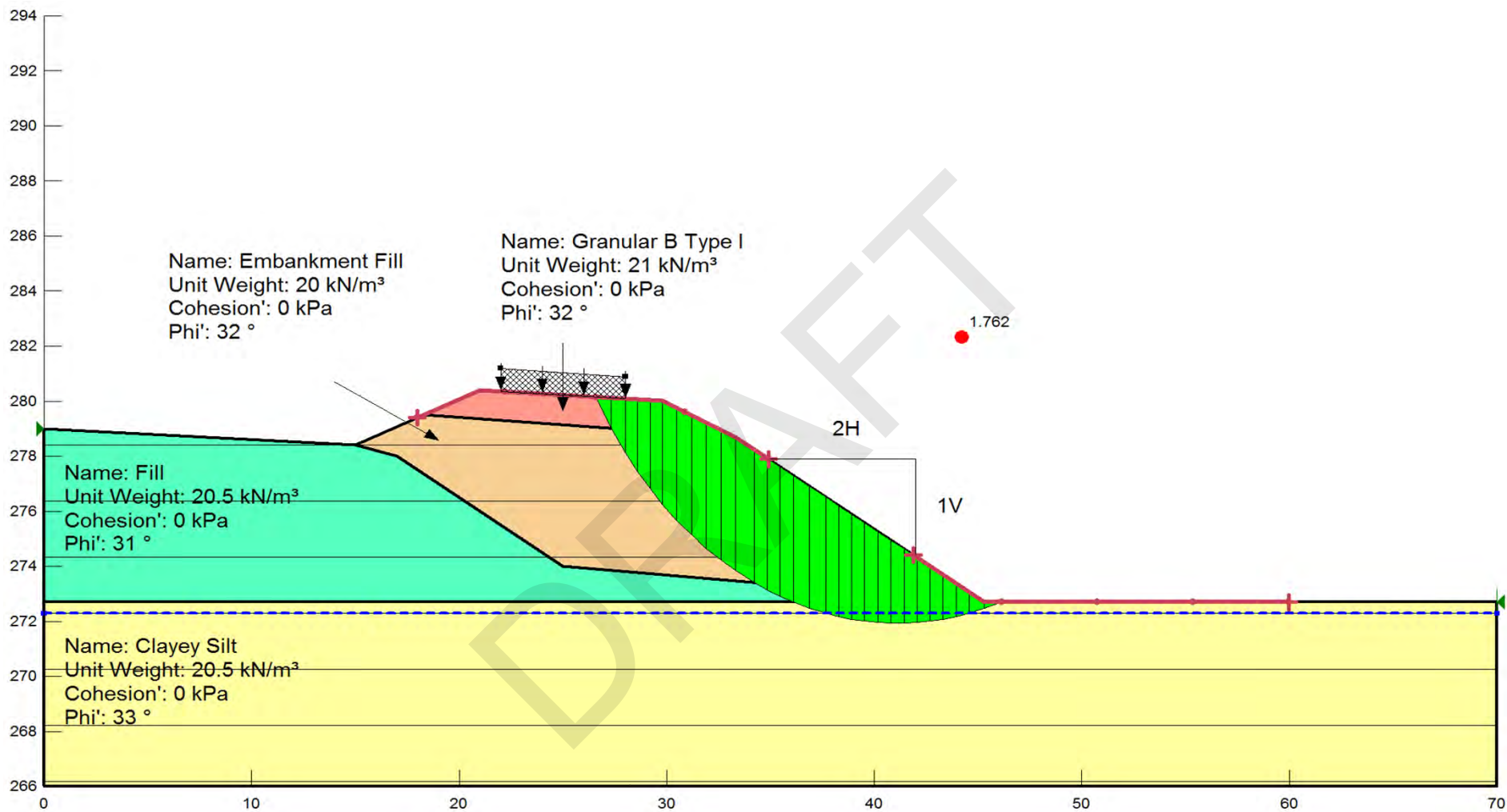
**Factored Axial Capacity of 1500 mm and 1800 mm Caissons**



Load Intensity $p$ (kN/m) vs Lateral Deflection $y$ (m) of 1800 mm Caisson at Various Depths below Pile Head								
Depth Below Pile Head (m)	Curve Points							
		1	2	3	4	5	6	7
1.0	Y	0	0.0147	0.0343	0.0539	0.0735	0.0784	0.1568
	P	0	202.662	336.883	372.896	380.839	381.524	381.524
2.0	Y	0	0.000036	0.0036	0.036	0.369	0.378	0.756
	P	0	101.6403	321.415	571.5656	1016.403	1016.403	1016.403
3.0	Y	0	0.000036	0.0036	0.036	0.369	0.378	0.756
	P	0	113.8663	360.0768	640.3173	1138.663	1138.663	1138.663
4.0	Y	0	0.000036	0.0036	0.036	0.369	0.378	0.756
	P	0	126.0743	398.6819	708.9678	1260.743	1260.743	1260.743
5.0	Y	0	0.000036	0.0036	0.036	0.369	0.378	0.756
	P	0	138.2751	437.2642	777.5778	1382.751	1382.751	1382.751
6.0	Y	0	0.000036	0.0036	0.036	0.369	0.378	0.756
	P	0	150.4723	475.8351	846.1677	1504.723	1504.723	1504.723
7.0	Y	0	0.000036	0.0036	0.036	0.369	0.378	0.756
	P	0	162.6674	514.3995	914.746	1626.674	1626.674	1626.674
8.0	Y	0	0.000036	0.0036	0.036	0.369	0.378	0.756
	P	0	174.8613	552.9598	983.3171	1748.612	1748.612	1748.612
9.0	Y	0	0.000036	0.0036	0.036	0.369	0.378	0.756
	P	0	187.0542	591.5175	1051.883	1870.542	1870.542	1870.542
10.0	Y	0	0.0159	0.0372	0.0585	0.0797	0.085	0.17
	P	0	1826.434	3036.062	3360.615	3432.203	3438.374	3438.374
11.0	Y	0	0.0174	0.0407	0.0639	0.0871	0.0929	0.1858
	P	0	2236.029	3716.926	4114.263	4201.905	4209.461	4209.461
12.0	Y	0	0.0189	0.0441	0.0693	0.0945	0.1008	0.2016
	P	0	2686.408	4465.587	4942.956	5048.251	5057.327	5057.327
13.0	Y	0	0.009	0.0211	0.0331	0.0452	0.0482	0.0964
	P	0	3619.837	6017.216	6660.453	6802.334	6814.565	6814.565

The response of a pile to lateral loads is a nonlinear relationship. The  $p$ - $y$  geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The  $p$ - $y$  curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The Table D-1 presents the Load Intensity per unit length of pile  $p$  (kN/m) vs Lateral Deflection  $y$  (m) of an 1800 mm Caisson. The  $p$ - $y$  points can be used for the structural design of the pile in response to lateral loads. In the case where non-linear springs are spaced at 250 mm, the above "p" values are to be multiplied by 0.25 to obtain the correct spring stiffnesses.





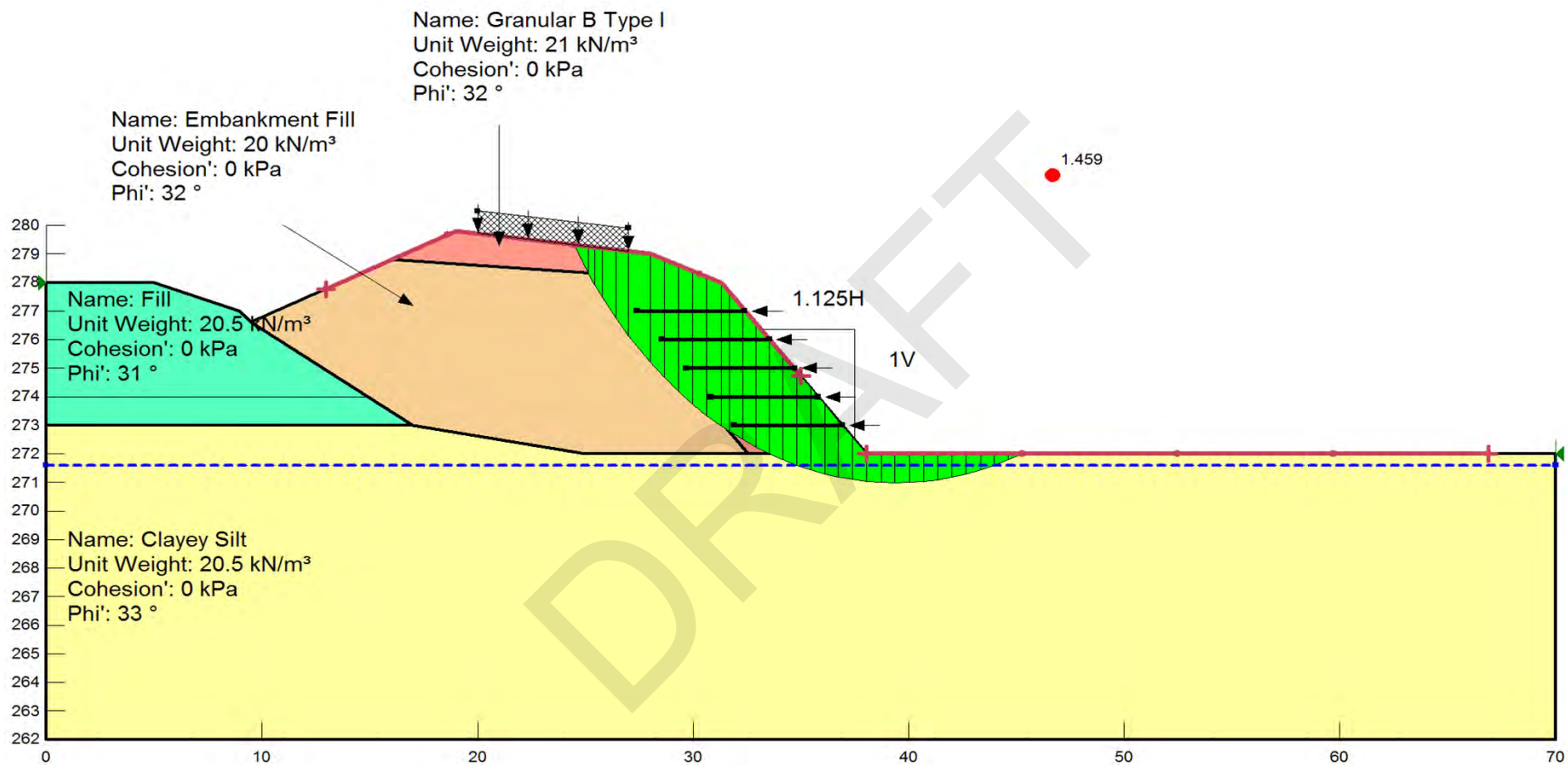
# Static Slope Stability Analysis (Drained) NW Ramp

Highway 401 & Veterans Memorial Parkway Interchange

Figure D6

Project No. 165001002

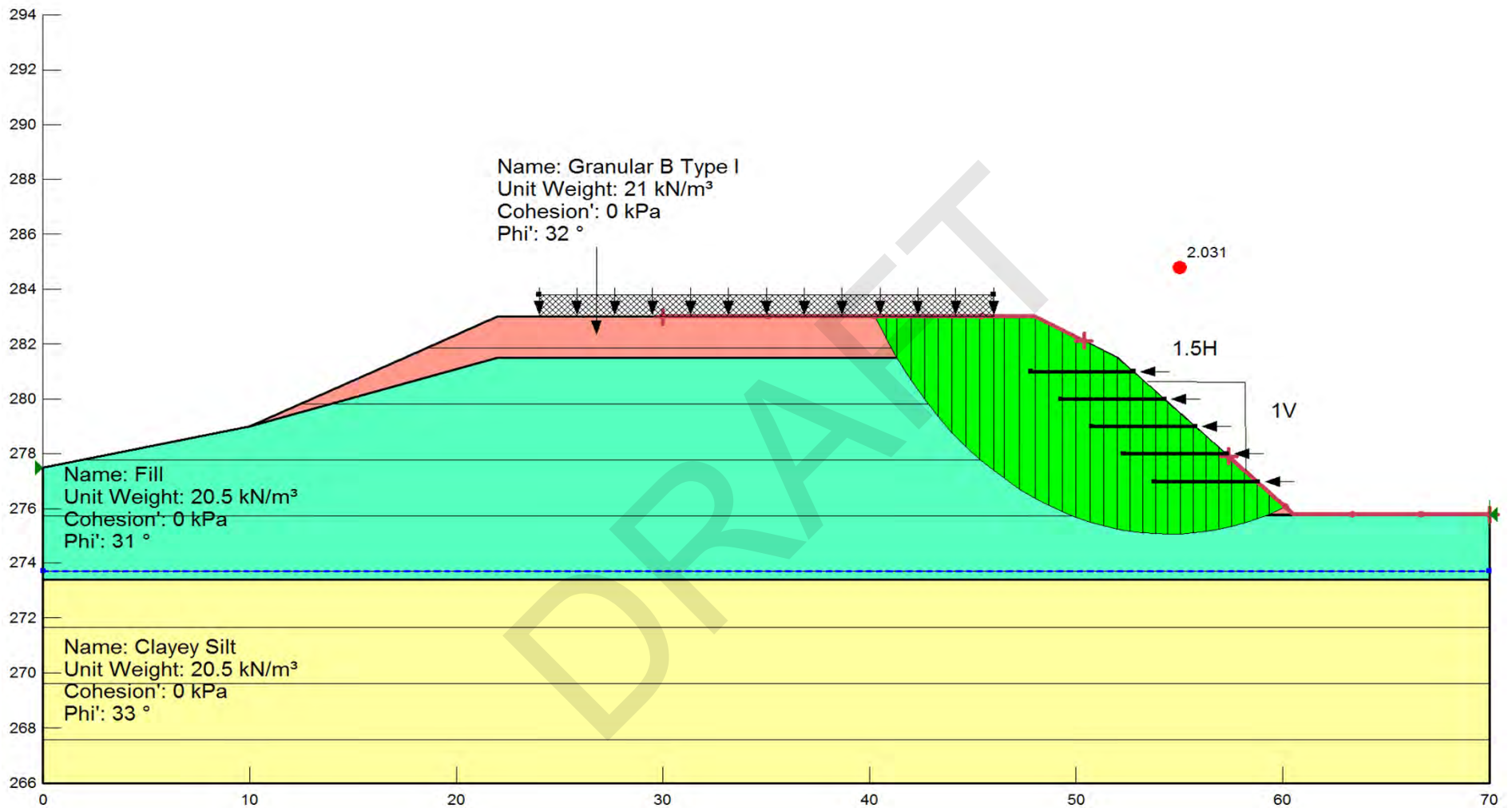
GWP No. 3033-11-00



# Static Slope Stability Analysis (Drained) NW Ramp - RSS Highway 401 & Veterans Memorial Parkway Interchange

Figure D7

Project No. 165001002  
 GWP No. 3033-11-00



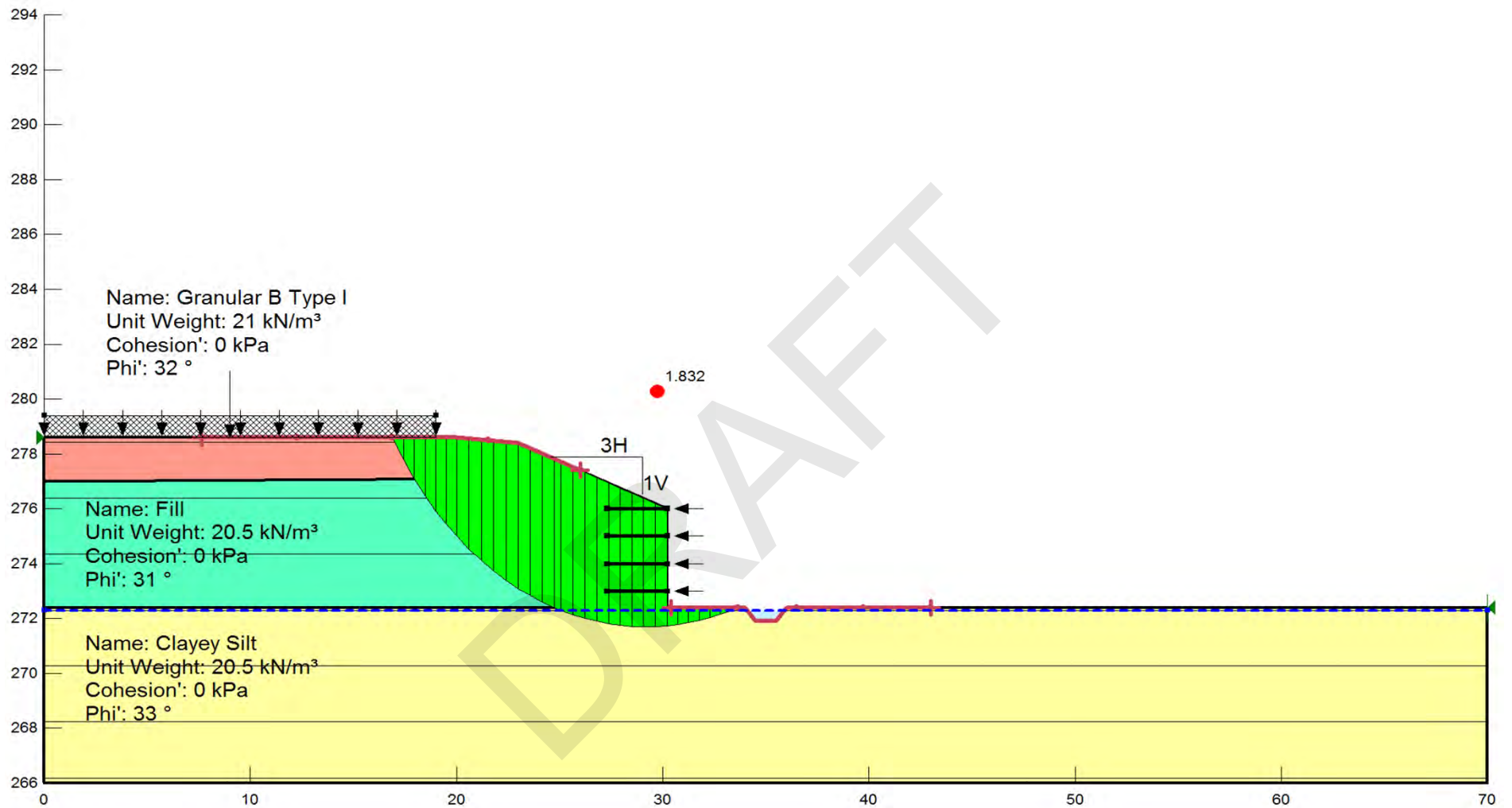
# Static Slope Stability Analysis (Drained) VMP - RSS

Highway 401 & Veterans Memorial Parkway Interchange

Figure D8

Project No. 165001002

GWP No. 3033-11-00



# Static Slope Stability Analysis (Drained) Crinklaw Drain Culvert

Highway 401 & Veterans Memorial Parkway Interchange

Figure D9

Project No. 165001002

GWP No. 3033-11-00