



## Foundation Investigation and Design Report

*Highway 9 - Holland Drainage Canal Bridge (Site No. 37-31) Replacement, Schomberg, Ontario, Assignment No. 2016-E-0029-07 and -17, G.W.P. 2266-18-00*

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# PART A

**FOUNDATION INVESTIGATION REPORT  
HIGHWAY 9 - HOLLAND DRAINAGE CANAL BRIDGE (SITE NO. 37-31)  
SCHOMBERG, ONTARIO  
ASSIGNMENT NO. 2016-E-0029-07 AND -17  
G.W.P. 2266-18-00**



## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the replacement of the Highway 9 – Holland Drainage Canal Bridge (Site No. 37-31) in Schomberg, Ontario, as shown on the Key Plan on Drawing 1.

This report addresses the foundation investigation carried out to support the replacement of the existing Highway 9 – Holland Drainage Canal Bridge. This report was developed based on information from the current investigation, supplemented with information from a 1965 foundation investigation carried out at the site (by others) and reported in the following:

- **MTO GEOCRES No. 31D-025:** “Foundation Investigation Report for Proposed Holland Marsh Drainage Canal Bridge, Lot 35, Con. VI, Twp of King, Co. of York, Hwy #9, Line ‘K’, District #6 (Toronto). W.J. 65-F-118 - - W.P. 171-65”, prepared by Department of Highway Ontario, Foundation Section – Materials and Testing Division, dated December 13, 1965.

The Terms of Reference for the foundation engineering services are outlined in MTO’s Work Item Order No. 2016-E-0029-007, dated January 2, 2018 and MTO’s Work Item Order No. 2016-E-0029-0017, dated November 20, 2018, which form part of the Consultant’s Assignment for the Central Region Large Value Retainer under Agreement No. 2016-E-0029-007 and 2016-E-0029-0017 for this project.

## 2.0 SITE DESCRIPTION

The Highway 9 – Holland Drainage Canal Bridge (Site No. 37-31) is located about 3.2 km west of Highway 400 along Highway 9, near West Canal Bank Road in Schomberg, Ontario. The site is surrounded by marshland to the south and northwest with farmland to the northeast. The surrounding area is generally flat and contains distributary channels of the Holland Canal. The existing road surface along Highway 9 rises slightly from west to east and is at approximately Elevations 220 m to 221 m at the Holland Drainage Canal Bridge. The ground surface south of the bridge at the embankment toe is at about Elevation 219 m. The existing embankment side slopes are inclined at about 2 Horizontal and 1 Vertical (2H:1V). At the time of the 2018-2019 investigation, visual observations suggested no evidence of settlement nor surficial/global instability of the existing approach embankments. The water level within the Holland Canal was measured at approximately Elevations 217 m to 219 m between December 2018 and March 2019.

Based on GEOCRES No. 31D-025, the DHO Foundation Investigation Report dated December 13<sup>th</sup>, 1965, the original ground surface at the site in 1965 prior to the construction of the existing Holland Drainage Canal Bridge was between Elevations 218 m and 220 m. It is understood that approximately 3 m of peat was removed from the Holland Drainage Canal Bridge and roadway footprint and replaced with fill as part of the construction of the existing bridge and approach embankments. Based on visual observations at the site and anecdotal evidence, it is possible that a portion of the originally excavated peat material may have been stockpiled adjacent to the roadway embankment (within the MTO Right-of-Way) and within the footprint of the new proposed widening(s).



## 3.0 INVESTIGATION PROCEDURES

### 3.1 1965 Investigation

A total of four boreholes (Boreholes 1 and 3 to 5) and two Dynamic Cone Penetration Tests (DCPTs) (Boreholes 2 and 6) were advanced as part of the 1965 investigation (GEOCRE No. 31D-025) for the Highway 9 – Holland Drainage Canal. The boreholes and DCPTs are located within or immediately adjacent to the footprint of the existing abutment foundations, as shown on Drawing 1. These borehole locations have been estimated based on plotting the coordinates and offsets shown on the 1965 drawings, and converting these to MTM NAD83 (Zone 10) coordinates. The Record of Borehole sheets for the four boreholes and two DCPTs from the 1965 investigation are presented in Appendix A. For the purposes of this report, these boreholes have been renumbered to reference the last digits of the GEOCRE No. followed by the original borehole number (e.g., Borehole No.1 renumbered to BH 25-1).

The boreholes were advanced using “NX” and “BX” casing and wash-boring methods. The Standard Penetration Test (SPT) “N” values in the 1965 investigation were obtained using a manual (i.e. rope and cathead) hammer.

### 3.2 2018-2019 Investigation

The field work for the current (2018-2019) investigation was carried out between November 12, 2018 and March 13, 2019, during which time six boreholes (designated as Boreholes A1-1, A1-2, A2-1, A2-2, AP-1 and AP-2), one Cone Penetration Test (CPT) (designated as CPT18-01B), one Seismic CPT (SCPT) (designated as SCPT19-02), and one monitoring well hole (designated as Borehole MW-01) were advanced at the site, at the locations shown on Drawing 1. Boreholes A1-1 and A2-1 were advanced within the existing westbound lane of Highway 9 at the west and east abutments, respectively. Boreholes A1-2 and A2-2 were advanced near the existing toe of the Highway 9 embankment, south of the existing west and east abutments, respectively. Boreholes AP-1 and MW-01 were advanced at the existing south toe of the west approach embankment. Borehole AP-2 was advanced within the existing eastbound lane of Highway 9, within the east approach embankment. CPT18-01B was advanced within the existing westbound lane on the east side of the existing bridge, and SCPT19-02 was advanced near the existing south toe of the west approach embankment. Traffic protection consisted of a lane closure for boreholes advanced from the roadways platforms, and shoulder/lane closures to aid in the loading and unloading of the track-mounted drill rig for the borehole advanced off of the roadway, all consistent with Book 7 requirements.

All boreholes were advanced with 210 mm outside diameter (O.D.) hollow-stem augers and/or 127 mm O.D. flush-joint casing by a CME 55 track-mounted drill supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. In general, the hollow-stem augers were only used to drill the upper 5 m to 10 m of the boreholes; once the boreholes reached the clay stratum, the flush-joint casing was lowered through the hollow-stem augers and wash-rotary drilling methods (with the casing filled with water/drilling mud) were used to maintain pressure at the bottom of the borehole so as to reduce disturbance at the sampling and testing depth. Geo-Environmental Drilling transported the required quantity of fresh water to site daily for use in the wash-rotary drilling activities. The table below outlines the depth intervals in which hollow-stem augers and/or casing were used to advance each borehole drilled at the site.



Borehole No.	Depth Interval of Borehole Advancement Method (m)	
	210 mm O.D. Hollow-Stem Augers	127 mm O.D. Flush-Joint Casing
A1-1	0 - 10.2	10.2 - 50.9
A1-2	0 - 5.2	5.2 - 61.6
A2-1	0 - 10.2	10.2 - 52.4
A2-2	N/A	0 - 50.9
AP-1	0 - 6.1	6.1 - 31.7
AP-2	0 - 31.7	N/A
MW-01	0 - 8.5	N/A

Soil samples were obtained at 0.75 m, 1.5 m, 3.0 m and 5.0 m intervals of depth using a 50 mm O.D. split-spoon sampler driven with an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)<sup>1</sup>, and using 76 mm O.D. thin-walled 'Shelby' Tube samplers (ASTM D1587)<sup>2</sup> to obtain relatively undisturbed samples in the cohesive soils. Given the consistency of the clay strata at this site (i.e. generally firm to stiff), the use of a Piston-sampler, which is typically used to achieve better recovery in soft to very soft soils, was not considered necessary at this site. In-situ field vane shear tests were carried-out primarily using an MTO 'N'-vane and in some cases using an MTO 'B'-vane in cohesive soils for assessing undrained shear strengths (ASTM D2573)<sup>3</sup>.

Monitoring Well MW-01 was installed in a borehole advanced using only 210 mm O.D. hollow-stem augers by an AMS Power Probe 9580 rubber track-mounted drill supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. No soil samples were obtained during the drilling of this borehole for the well installation.

The CPT and SCPT holes (CPT18-01B and SCPT19-02) were advanced using a truck-mounted CPT rig and a track-mounted CPT rig, respectively. Both rigs were equipped with 30-ton rig cylinders and were operated by ConeTec Investigations Ltd. (ConeTec) of Richmond Hill, Ontario

The groundwater conditions in the open boreholes were observed during and immediately following the drilling operations. A standpipe piezometer was installed in Boreholes AP-1, AP-2, and MW-01 to allow monitoring of the water level(s). The installed piezometers consist of a 50 mm diameter PVC pipe, with a 1.5 m to 3 m long slotted screen within a filter sand pack. The borehole and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to near ground surface with bentonite pellets. The upper 200 mm of the borehole was capped with a flush-mount well casing or above grade casing. Boreholes A1-1, A2-1, A1-2 and A2-2 were backfilled to

<sup>1</sup> ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

<sup>2</sup> ASTM D1587 - Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.

<sup>3</sup> ASTM D2573 - Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.



ground surface with bentonite pellets and/or a cementitious grout, in accordance with Ontario Regulation 903, Wells (as amended). Boreholes A1-1 and A2-1 were sealed at the roadway surface with cold patch asphalt upon completion. The piezometers and monitoring wells installed at the site during the 2018-2019 investigation have not been decommissioned. It is understood that they will be decommissioned at the time of construction by the contractor.

The field work was monitored on a full-time basis by a member of Golder's engineering staff who located the boreholes and CPTs in the field, directed the sampling and in-situ testing operations, logged the boreholes and examined the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further visual review and geotechnical laboratory testing. Index and classification testing consisting of natural moisture content, Atterberg limits and grain size distribution were conducted on selected samples. In addition, specialized strength and deformation testing including direct shear, triaxial, and one-dimensional consolidation (oedometer) testing were carried out on selected soil specimens obtained from the Shelby tube samples. All laboratory tests were carried out in accordance with MTO and / or ASTM Standards, as applicable.

One soil sample obtained from each of Boreholes A1-1 and A2-1, using appropriate sampling protocols, was submitted to a specialist analytical laboratory under chain of custody procedures for testing of conductivity / resistivity, pH, chemical analysis of sulphate and chloride content, to assess the potential for the soil to cause deterioration to buried concrete and corrosion to steel.

The borehole locations were surveyed in the field by Golder personnel using a Trimble Geo 7X Global Positioning System (GPS) unit. The Trimble Geo 7X achieved a horizontal accuracy of 1.1 cm to 3 cm and a vertical accuracy of 1.4 cm to 5.8 cm while in use at the site. The locations of the CPT and SCPT holes were surveyed in the field by ConeTec. The locations given on the Record of Borehole sheets and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, including the geographic (Latitude / Longitude) coordinates, the ground surface elevations, and borehole drilled depths are summarized below.

Borehole / CPT No.	MTM NAD83		Ground Surface Elevation (m)	Borehole / CPT Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
A1-1	4,875,633.5 (44.020618)	294,019.4 (-79.634475)	220.7	50.9
A1-2	4,875,612.3 (44.020427)	294,032.6 (-79.634310)	219.3	61.6
A2-1	4,875,646.1 (44.020732)	294,058.4 (-79.633989)	220.6	52.4
A2-2	4,875,623.4 (44.020527)	294,063.6 (-79.633924)	218.8	50.9
AP-1	4,875,610.1 (44.020407)	294,009.4 (-79.634599)	219.6	31.7
AP-2	4,875,648.5	294,077.6	220.7	31.7



Borehole / CPT No.	MTM NAD83		Ground Surface Elevation (m)	Borehole / CPT Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
	(44.020754)	(-79.633749)		
MW-01	4,875,610.1 (44.020407)	294,007.8 (-79.634619)	219.7	8.5
CPT18-01B	4,875,646.5 (44.020727)	294,060.1 (-79.633963)	220.6 <sup>1</sup>	50.0
SCPT19-02	4,875,613.1 (44.020426)	294,028.5 (-79.634357)	219.3 <sup>2</sup>	60.0

**Notes:**

1. Elevation obtained from BH A2-1 (approximately 3 m away).
2. Elevation obtained from BH A1-2 (approximately 2 m away).

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

This section of Highway 9 is located in an area defined as 'Peat and Muck' within the Simcoe Lowlands physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)<sup>4</sup>.

The Simcoe Lowlands physiographic region covers the central portion of the County of Simcoe. Following the retreat of the last glacial ice sheet, the lowland was flooded by the now extinct post-glacial Lake Algonquin. This past post-glacial lacustrine environment is marked by deep sand, silt and clay beds overlying glacial ground moraine material. The Holland River valley, which crosses Highway 400 just north of Highway 9 and South Canal Road, is located within the Simcoe Lowlands region. This valley extends to the southwest from Cook Bay at the south end of Lake Simcoe, and was once a shallow extension of the lake. The floor of the valley consists of peat, soft clays and loose sands. It is understood that during initial construction of Highway 400 through this area, a layer of peat about 2 m to 3 m thick was removed to allow construction of the road upon the underlying sand and clay strata; towards Lake Simcoe, the peat deposit is considerably thicker.

### 4.2 General Overview of Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes of the current investigation including piezometer installation details and water level readings, and the results of the in-situ and laboratory tests are provided on the Record of Borehole Sheets in Appendix B. The results of the in-situ field tests (i.e., SPT "N"-values and shear strengths from the field vanes) as presented on the borehole records and in Section 4 are uncorrected. The SPT "N"-values from the 1965 investigation are based on use of a manual hammer, while those in the 2018-2019 investigation are based on use of an automatic hammer and the values are reported with no adjustment in this report, although it is recognized that SPT "N"-values obtained using a manual hammer are

<sup>4</sup>Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.



frequently higher than those obtained using an automatic hammer (CFEM, 2006)<sup>5</sup>. The results of the CPT and SCPT testing are also provided in Appendix B. The results of the geotechnical laboratory testing on soil samples are presented on the laboratory test figures in Appendix C. The results of the analytical testing are provided in Appendix D.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profiles on Drawings 2 to 4 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected, however, the factual data presented on the borehole records governs any interpretation of the site conditions.

In general, the subsurface soils encountered at the site consist of surficial layers of topsoil and/or asphalt, underlain by fill material(s) and peat / organic silt. The fill and / or organic materials are underlain by a relatively thin, upper soft to very stiff clayey silt deposit, further underlain by a loose to dense silt and sand to silty sand to sand deposit. The non-cohesive deposit is underlain by a thick, lower soft to very stiff clayey silt to silty clay deposit.

Detailed descriptions of the subsurface conditions are provided in the following sections of this report.

#### 4.2.1 Topsoil

An approximately 130 mm thick layer of topsoil was encountered immediately at ground surface in Borehole A1-2, which was advanced near the existing south toe of the Highway 9 embankment.

#### 4.2.2 Asphalt

An approximately 170 mm to 300 mm thick layer of asphalt was encountered immediately at ground surface in Boreholes A1-1, A2-1, and AP-2, which were advanced through the existing Highway 9 roadway.

#### 4.2.3 Fill

Fill was encountered in all of the boreholes advanced at the site.

A silt and sand to silty sand fill, approximately 1.3 m to 3.8 m thick, was encountered at ground surface in Borehole AP-1, below the topsoil in Borehole A1-2, and below the asphalt in Boreholes A1-1, A2-1, and AP-2. The base of this fill layer extended to between Elevations 219.3 m and 216.6 m.

A clayey silt to sandy clayey silt fill, approximately 1.4 m and 3.2 m thick, was encountered at ground surface in Borehole A2-2 and underlying the silty sand fill in Borehole AP-2, respectively. The base of this fill layer extended to Elevations 216.0 m and 217.4 m in Boreholes AP-2 and A2-2, respectively.

The SPT “N” values measured within the non-cohesive portions of the fill generally range between 2 blows and 24 blows per 0.3 m of penetration, however one SPT “N” value of 50 blows per 0.04 m of penetration was recorded in Borehole AP-2 which may be due to the presence of gravel within the fill at this location. The SPT “N” values indicate the granular fills generally have a very loose to compact level of compactness. The SPT “N”-values measured within the cohesive portions of the fill range from 4 blows to 10 blows per 0.3 m of penetration, suggesting a firm to stiff consistency.

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<sup>5</sup> Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition.



A grain size distribution test was carried out on one sample of the silt and sand fill and the results are shown on Figure C1 in Appendix C. An Atterberg limits test was carried out on one sample of the silt and sand fill and measured a liquid limit of 14 per cent, a plastic limit of 12 per cent, and a corresponding plasticity index of 2 per cent. These results, which are plotted on a plasticity chart on Figure C2 in Appendix C, indicate the presence of low plasticity silt within the silt and sand fill. The natural water content measured on samples of the non-cohesive fill range between 11 and 16 per cent.

Grain size distribution tests were carried out on two samples of the clayey silt with sand to sandy clayey silt fill and the results are shown on Figure C3 in Appendix C. An Atterberg limits test was carried out on one sample of the cohesive fill and measured a liquid limit of 19 per cent, a plastic limit of 13 per cent, and a corresponding plasticity index of 6 per cent. These results, which are plotted on a plasticity chart on Figure C4 in Appendix C, indicate the fill consists of clayey silt of low plasticity. The natural water content measured on selected samples of the cohesive fill ranges from about 11 to 18 per cent.

#### 4.2.4 Peat / “Muck”

A deposit of peat was encountered in Boreholes A1-2, A2-2, and AP-1, which were all advanced near the south toe of the existing Highway 9 roadway embankment. The thickness of the peat ranged from approximately 0.8 m to 2.7 m thick and the surface of the peat layer was encountered between Elevations 218.2 m and 217.4 m, and extended to Elevations 216.6 m to 215.4 m. An approximately 0.3 m thick layer of peat was encountered below the silty sand fill at Elevation 216.6 m in Borehole A1-1, which was advanced through the existing roadway embankment.

All of the boreholes from the 1965 investigation (i.e. Borehole 25-1 and 25-3 to 25-5) encountered a layer of black “muck” at ground surface or below water ranging in thickness from 0.8 m to 2.9 m thick; these boreholes were advanced in the then-existing canal or on the banks of the canal. The “muck” material is described as comprising of silt and clay with decayed organic matter and extended to depths between 0.8 m and 3 m, corresponding to between Elevations 216.8 m and 217.0 m.

The SPT “N”-values measured within the peat layer during the 2018-2019 investigation range between 2 blows and 5 blows per 0.3 m of penetration. One SPT “N”-value measured within the “muck” during the 1965 investigation was 0 blows per 0.3 m of penetration. These results suggest a very soft to soft consistency.

Organic content testing was carried out on a selected sample of the peat from Borehole A1-1 and the result was an organic content of about 66 per cent. The natural water content measured on selected samples of the peat obtained during the 2018-2019 investigation ranges from about 151 to 333 per cent. The natural water content measured on a selected sample of the silt and clay “muck” obtained during the 1965 investigation was approximately 25 per cent.

#### 4.2.5 Upper Clayey Silt

A relatively thin (approximately 0.6 m to 3.4 m thick) deposit of clayey silt was encountered underlying the fill in Borehole AP-2, underlying the peat / “muck” in Boreholes A1-1, A1-2, A2-2, AP-1, 25-3, 25-4, and 25-5 and interlayered within the silty sand to sand deposit (described below) in Borehole A2-1. The surface of the deposit was encountered between Elevations 217.0 m and 215.4 m. In the 1965 investigation, the deposit was classified as silt; however, the laboratory testing conducted at the time of the investigation suggests the deposit is comprised of clayey silt.



The SPT “N”-values measured within the upper clayey silt deposit during the 2018-2019 investigation range between weight of hammer and 7 blows per 0.3 m of penetration, suggesting a soft to firm consistency. The SPT “N”-values measured within the clayey silt deposit during the 1965 investigation range between 3 blows and 29 blows per 0.3 m of penetration, suggesting a soft to very stiff consistency. In-situ field vane shear strength testing carried out within the upper clayey silt deposit measured undrained shear strengths ranging between 29 kPa to greater than 96 kPa, indicating that the deposit has a firm to very stiff consistency. The sensitivity values calculated from the field vane tests are 4.3 and 11.8, indicating the deposit ranges from medium to extra sensitive.

Grain size distribution tests were carried out on three samples of the upper clayey silt deposit during the 2018-2019 investigation, and the results are shown on Figure C5 in Appendix C. A grain size distribution test was carried out on one sample of the silt deposit recovered from the 1965 investigation, and the results are presented on the borehole records included in Appendix A.

Atterberg limits testing was carried out on three selected samples of the upper clayey silt deposit during the 2018-2019 investigation, and measured liquid limits ranging between 26 and 31 per cent, plastic limits ranging between 18 and 20 per cent, and plasticity indices ranging between 8 and 11 per cent. These results, which are plotted on a plasticity chart on Figure C6 in Appendix C, indicate that the deposit consists of clayey silt of low plasticity. Atterberg limits testing was carried out on two selected samples of the silt deposit from the 1965 investigation and measured liquid limits of about 25 and 26 per cent, plastic limits of about 19 per cent, and plasticity indices of about 6 and 7 per cent. These test results also indicate that the deposit is comprised of clayey silt of low plasticity.

The natural water content measured on selected samples of the upper clayey silt to silty clay deposit obtained during the 2018/2019 and 1965 investigations ranges between about 26 and 28 per cent.

#### **4.2.6 Silt and Sand to Silty Sand to Sand**

An approximately 3.0 m to 4.7 m thick deposit of silt and sand to silty sand to sand was encountered underlying the fill (and interlayered with the upper clayey silt deposit) in Borehole A2-1, underlying the upper clayey silt deposit in Boreholes A1-1, A1-2, A2-2, AP-1, AP-2, 25-3, 25-4, and 25-5 and underlying the peat layer in Borehole 25-1. The surface of the deposit was encountered between Elevations 215.9m and 213.2 m.

The SPT “N”-values measured within the silt and sand to silty sand to sand deposit during the 2018-2019 investigation range between 7 blows and 40 blows per 0.3 m of penetration. The SPT “N”-values measured within the sand deposit during the 1965 investigation range between 22 blows and 40 blows per 0.3 m of penetration with one value of 71 blows per 0.3 m of penetration in Borehole 25-4. These results suggest that the deposit generally has a loose to dense level of compactness.

Grain size distribution tests were carried out on six samples of the silt and sand to silty sand to sand deposit during the 2018-2019 investigation, and the results are shown on Figure C7 in Appendix C. Grain size distribution tests were carried out on seven samples of the sand deposit recovered from the 1965 investigation, and the results are presented on the borehole records and the grain size distribution test figure included in Appendix A.

The natural water content measured on selected samples of the silt and sand to silty sand to sand deposit obtained during the 2018-2019 and 1965 investigations ranges from about 10 to 42 per cent.

A laboratory consolidated, drained direct shear (DS) test was carried out on a selected sample of the silty sand to sand deposit from Borehole A1-2. The details of the test results are shown Figures C8A to C8C in Appendix C. The results of the direct shear test are summarized below.



Borehole / Sample Number	Depth (m)	Effective Cohesion Intercept, $c'$ (kPa)	Effective Angle of Internal Friction, $\phi'$ (Degrees)
A1-2 / SA No. 7	4.9	0	42

**Note:**

1. Assessed shear strength parameters are only valid over the of stress conditions in test.

A laboratory consolidated, drained (CID) triaxial compression test was carried out on one reconstituted sample of the sand deposit. In total, one combined specimen was tested. The sample was obtained from a 50 mm O.D. split-spoon sampler and reconstituted by compaction at a 15% water content and a dry density of 1.74 g/cm<sup>3</sup> (for a bulk density of about 19.6 kN/m<sup>3</sup>). The details of the test results are shown Figures C9A to C9D in Appendix C. The results of the CID triaxial test are summarized below.

Borehole / Sample Number	Average Depth of Combined Samples (m)	Effective Cohesion Intercept, $c'$ (kPa)	Effective Angle of Internal Friction, $\phi'$ (Degrees)
AP-2 / SS9 – SS10	8.7	0	38

**Note:**

1. Assessed shear strength parameters are only valid over the of stress conditions in test.

#### 4.2.7 Lower Silty Clay to Clayey Silt

An extensive lower deposit of silty clay to clayey silt was encountered underlying the silt and sand to silty sand to sand in all of the boreholes advanced at the site. The top of the lower silty clay to clayey silt stratum was encountered at elevations ranging from 212.3 m to 208.6 m. All boreholes were terminated within this deposit, penetrating it for a thickness ranging from 21.5 m to 52.9 m for the 2018-2019 investigation and 6.1 m to 17.5 m for the 1965 investigation. A clayey silt crust is present within the upper portion of this deposit and extends to about Elevation 205.0 m, for a thickness of about 3.6 m to 7.3 m.

The SPT “N” values measured in the deposit generally range from weight of rods to 19 blows per 0.3 m of penetration (with one value of 44 blows per 0.3 m of penetration in Borehole A2-1) in the 2018-2019 investigation and 12 to 32 blows per 0.3 m of penetration in the 1965 investigation, suggesting a very soft to hard consistency. In-situ field vane shear strength testing carried out within the lower silty clay to clayey silt deposit measured undrained shear strengths ranging between 13 kPa to greater than 96 kPa, indicating that the deposit has a soft to very stiff consistency. The results of the CPT testing indicate that the undrained shear strength within the deposit generally ranges from about 40 kPa to 140 kPa, and a corresponding firm to very stiff consistency. The sensitivity values calculated from the field vane tests range between 1.1 and 5.8 with a mean value of 2.4, indicating the deposit is of medium sensitivity.

Grain size distribution tests were carried out on 24 samples of the lower silty clay to clayey silt deposit during the 2018-2019 investigation, and the results are shown on Figures C10A to C10D in Appendix C. Grain size distribution tests were carried out on four samples of the lower silty clay to clayey silt deposit recovered from the 1965 investigation, and the results are presented on the borehole records included in Appendix A.

Atterberg limits testing was carried out on 25 selected samples of the lower clayey silt to silty clay deposit during the 2018-2019 investigation, and measured liquid limits ranging between 21 and 46 per cent, plastic limits ranging between 14 and 19 per cent, and plasticity indices ranging between 5 and 28 per cent. These results, which are



plotted on a plasticity chart on Figures C11A and C11B in Appendix C, indicate that the deposit consists of clayey silt of low plasticity to silty clay of intermediate plasticity, but the deposit is predominantly classified as a clayey silt of low plasticity. Atterberg limits testing was carried out on six selected samples of the 'silt' deposit from the 1965 investigation and measured liquid limits between about 26 and 41 per cent, plastic limits between about 19 and 21 per cent, and plasticity indices between about 7 and 20 per cent. These test results indicate that the deposit is comprised of clayey silt of low plasticity to silty clay of intermediate plasticity. The natural water content measured on selected samples of the upper clayey silt to silty clay deposit obtained during the 2018/2019 and 1965 investigations ranges from about 6 to 40 per cent. Liquidity indices calculated from the results of the Atterberg limits testing carried out on the deposit range from about 0 to as high as 1.94 with a mean value of 0.86. Samples with liquidity indices of about 1 are generally located between Elevations 202 m and 186 m.

Laboratory consolidated, drained (CID) triaxial compression tests and consolidated, undrained (CIU) triaxial compression tests with pore pressure measurements were carried out on two samples of the silty clay to clayey silt deposit. In total, one set of two samples and one set of three samples were tested. All specimens tested were obtained from relatively "undisturbed" Shelby tube samples. The details of the CID and CIU triaxial test results are shown Figures C12A to C12D and C13A to C12D, respectively in Appendix C. The results of the triaxial tests are summarized below.

Borehole / Sample Number	Depth (m)	Elevation (m)	Test Type	Effective Cohesion Intercept, $c'$ (kPa)	Effective Angle of Internal Friction, $\phi'$ (Degrees)	Note(s)
A2-2 / TO-1	12.5	206.3	CID	0	32	Specimen obtained from within stiff crust
A1-2 / TO-1	20.1	199.2	CIU	0	20	Specimen obtained from firm silty clay to clayey silt below crust

**Note:**

1. Assessed shear strength parameters are only valid over the of stress conditions in test.

Laboratory consolidation (oedometer) testing (in accordance with ASTM D2435) was carried out on ten horizontally trimmed (HTO) specimens and three vertically trimmed (VTO) specimens of the silty clay to clayey silt deposit obtained from Shelby tube samples in Boreholes A1-2, A2-1, A2-2, and AP-1 to assess the compressibility characteristics of the deposit. While the conventional Horizontally Trimmed Orientation (HTO) specimens are tested to assess the vertical compressibility characteristics of the clayey soils (including  $c_v$ ), the Vertically Trimmed Orientation (VTO) specimens are tested to assess the horizontal consolidation characteristics of the clayey soils (such as  $c_h$ ). Each test was conducted using a load increment ratio of one. The samples are located throughout the deposit between the depths of 12.5 m and 40.0 m. The details of the test results which present the laboratory  $e-\log(\sigma'_v)$  curves are shown on Figures C14A - C14D to C26A - C26D and are summarized below.



Borehole / Sample Number	Test Type <sup>1</sup>	Sample Depth / Elevation (m)	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p'$ (Avg) - $\sigma_{vo}'$ (kPa)	OCR (Avg)	$C_c$	$C_r$	$e_o$	$C_{v(HTO)}$ or $C_{h(VTO)}$ (cm <sup>2</sup> /s)
A2-2 / TO-1	HTO	12.5 / 206.3	135	410 to 450	295	3.2	0.188	0.013	0.672	$5.0 \times 10^{-3}$
AP-1 / TO2	HTO	18.6 / 201.0	230	245 to 250	20	1.1	0.077	0.008	0.622	$5.2 \times 10^{-3}$
A2-1 / TO4	HTO	24.6 / 196.0	185	80 to 300	5	1.0	0.335	0.011	0.874	$4.4 \times 10^{-3}$
AP-1 / TO3	QHTO	24.7 / 194.9	240	370 to 420	155	1.6	0.090	0.010	0.522	$1.1 \times 10^{-1}$
A2-2 / TO3	QHTO	24.8 / 194.0	245	190 to 205	-50	0.8	0.307	0.032	0.883	$2.8 \times 10^{-3}$
A2-2 / TO3	VTO	24.8 / 194.0	245	-	-	-	-	-	-	$1.0 \times 10^{-2}$
AP-1 / TO4 <sup>4</sup>	QHTO	27.7 / 191.9	265	285 to 320	40	1.1	0.090	0.011	0.618	$3.5 \times 10^{-2}$
A2-1 / TO6	HTO	30.7 / 189.9	285	170 to 180	-110	0.6	0.196	0.024	0.843	$2.3 \times 10^{-3}$
A1-2 / TO2	HTO	32.3 / 186.9	315	270 to 310	-25	0.9	0.467	0.033	1.040	$5.3 \times 10^{-3}$
A1-2 / TO2	VTO	32.3 / 186.9	315	-	-	-	-	-	-	$9.4 \times 10^{-3}$
A2-1 TO7	QHTO	41.5 / 179.1	385	250 to 300	-110	0.7	0.083	0.005	0.550	$2.3 \times 10^{-3}$
A2-1 TO7	VTO	41.5 / 179.1	385	-	-	-	-	-	-	$2.8 \times 10^{-3}$
A2-2 / TO5	HTO	40.0 / 178.8	340	215 to 240	-110	0.7	0.197	0.016	0.714	$1.3 \times 10^{-3}$

Note(s): 1. HTO implies Horizontally Trimmed Orientation of specimen (i.e. conventional set-up) to test vertical consolidation characteristics.  
2. VTO implies Vertically Trimmed Orientation of specimen to test horizontal consolidation characteristics (in particular  $c_h$ ).  
3. QHTO implies a 'Quick' consolidation test was carried out on a HTO specimen in accordance with Method B or ASTM D2435.  
4. Results of consolidation test suggest that the specimen may have been disturbed. Results not considered in the analysis and design.

where  $\sigma_p'$  Estimated preconsolidation stress (using Casagrande construction and Work interpretation methods)

$\sigma_{vo}'$  Calculated existing vertical effective stress

$C_c$  Compression index

$C_r$  Recompression index

$e_o$  Initial void ratio

OCR Overconsolidation ratio

$C_{v/h}$  Coefficient of consolidation in the normally consolidated range, for approximate stress range  $100 \text{ kPa} \leq \sigma_v' \leq 400 \text{ kPa}$



Two laboratory flexible-wall permeameter tests were carried out on specimens of the lower silty clay to clayey silt deposit obtained from Shelby tube samples TO-3 in Borehole A2-2 and TO-7 in Borehole A2-1 to assess the hydraulic conductivity characteristics of the deposit. The results of the testing indicate a hydraulic conductivity of  $7.8 \times 10^{-10}$  m/s and  $1.7 \times 10^{-10}$  m/s for sample TO-3 in Borehole A2-2 and sample TO-7 in Borehole A2-1, respectively. The details of the test results are shown of Figures C27 and C28.

The two CPTs carried out at the site were terminated within the lower silty clay to clayey silt deposit. The pore pressure response measured during the cone penetration testing indicates that small layers of higher drainage capacity exist within the deposit. These higher permeability layers were generally encountered near the top of the deposit (i.e. at Elevations such as 207 m, 205 m, 198 m, 194 m) and near the bottom of the deposit (i.e. at Elevations such as 179 m, 174 m, 172 m, 170 m).

The preconsolidation stress within the deposit was also evaluated from the results of the CPT testing using the equation presented below after Demers and Leroueil (2002). Based on this empirical correlation, the results of the CPT testing suggest that the preconsolidation stress within the deposit generally ranges from about 175 kPa to 500 kPa.

$$\sigma_p' = \frac{q_t - \sigma_{vo}}{3.4}$$

Where:

$\sigma_p'$  = Estimated preconsolidation stress

$q_t$  = corrected tip resistance

$\sigma_{vo}$  = vertical stress

The results of the laboratory consolidation tests and the CPT tests suggest that two regions of higher strength and higher overconsolidation are present within the lower silty clay to clayey silt deposit. These two regions (or crusts) are both more overconsolidated at the top of the crust and trend to lower overconsolidation (or to near normal consolidated) at the bottom of the crust. The characteristics of the two higher strength regions within the deposit are shown below. The remaining portions of the deposit outside the elevations of these crusts are generally normally consolidated.

Crust	Approx. Elevation Range	Range of $\sigma_{vo}'$ (kPa)	Range of $\sigma_p'$ (kPa)	Approx. Range of OCR
Upper crust	210 m to 205 m	100 to 160	500 to 200	5 to 1.2
Lower crust	198 m to 194 m	220 to 260	300 to 255	1.4 to 1.0

### 4.3 Groundwater Conditions

The groundwater levels measured in the open boreholes during the 1965 investigation are included on the borehole records in Appendix A. The groundwater levels in the open boreholes during the 2018-2019 were measured, where possible, prior to the addition of drilling fluid/flush water, and also upon completion of drilling operations and are



included on the borehole records in Appendix B. The water levels measured in the open boreholes at the time of the investigation are not considered representative of the water levels at the site due to the addition of drilling fluids.

A standpipe piezometer was installed in Boreholes AP-1, AP-2 and MW-01 to allow monitoring of the groundwater level at this site. The groundwater levels recorded in the piezometers are shown on the borehole records in Appendix B and are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date
AP-1	219.6	0.0	219.6	January 4, 2019
		1.9	217.7	March 8, 2018
		2.4	217.2	March 11, 2019
		2.0	217.6	March 14, 2019
AP-2	220.7	2.1	218.6	March 21, 2019
MW-01	219.7	2.1	217.6	March 8, 2019
		2.6	217.1	March 11, 2019
		2.0	217.7	March 14, 2019

Based on the water level readings measured in the piezometers, the groundwater level at the site is at a depth of about 2 m below ground surface, about Elevations 217.6 m to 218.6 m. The 50-year flood level for the site is estimated at Elevation 220.28 m, as provided by AECOM.

The groundwater level observations at this site will be subject to seasonal fluctuations and precipitation events; the water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation and snow melt.

#### 4.4 Analytical Testing Results

Two soil samples were submitted for analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. Detailed analytical test results are included in Appendix D and the test results are summarized below:

Borehole No. - Sample No.	pH	Resistivity (ohm-cm)	Electrical Conductivity (µmho/cm)	Soluble Chlorides (µg/g)	Soluble Sulphates (µg/g)
A1-1 - 3	8.03	230	4,330	2,400	99
A2-1 - 3	8.19	670	1,490	660	120



## 5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Carter Comish and Ms. Nikol Kochmanová, P.Eng., and reviewed by Mr. Kevin Bentley, P.Eng. Mr. Paul Dittrich, P.Eng., a Principal with Golder and MTO Foundations Designated Contact, conducted an independent technical and quality control review of this report.

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# PART B

FOUNDATION DESIGN REPORT  
HIGHWAY 9 - HOLLAND DRAINAGE CANAL BRIDGE (SITE NO. 37-31)  
SCHOMBERG, ONTARIO  
ASSIGNMENT NO. 2016-E-0029-07 AND -17  
G.W.P. 2266-18-00



## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides detail foundation design recommendations for the proposed replacement of the Highway 9 – Holland Drainage Canal Bridge (Site No. 37-31) and the associated widening and raising of the approach embankments in Schomberg, Ontario. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 1965 and 2018-2019 subsurface investigation at this site.

This Foundation Investigation and Design Report containing interpretations and recommendations is intended for the use of the Ministry of Transportation, Ontario only and shall not be used or relied upon for any other purpose or by any other parties including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of this report. Where comments are made on construction, they are provided only in order to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

### 6.2 Project Understanding

The existing bridge at the site consists of a three-span structure with the abutments and piers supported on timber piles reportedly driven to about Elevation 204.7 m which would be near the bottom of the crust of the lower silty clay to clayey silt stratum. The existing bridge deck is approximately 24.2 m in length and 12.5 m in width. The bridge accommodates three lanes of traffic, one westbound lane and two eastbound lanes, one of which is a left-turning lane. Based on Drawing D-5869-1, included as part of GEOCREC No. 31D-025, the bottom of the drainage canal is at Elevation 216.9 m, with the canal side slopes at 2H:1V.

Based on the General Arrangement Drawing and the Pile Layout and Details Drawing as provided by AECOM, it is understood that the existing bridge is to be replaced by a single-span bridge with integral abutments supported on driven steel H-Piles. The drawings indicate that each abutment of the replacement bridge will be supported on 14 H-piles, and that the design includes the construction of the foundations (comprising an additional 6 H-piles at each abutment) to support a future widening of the bridge (to the north). The above noted drawings indicate that the proposed new H-piles are to be located immediately adjacent to the front, battered row of the existing abutment pile group(s). Based on the geometry and pile layout shown on the above referenced drawings, there is a risk that the new piles may come in contact with the existing abutment piles during installation/driving. Such interaction could result in the new piles being deflected out of the design alignment. The risks associated with the potential for pile conflicts occurring during the construction of this bridge configuration have been discussed with AECOM and MTO, and mitigation options have been identified in a Risk Register presented to MTO (and included in Appendix E).

In order to reduce the future drag loads and lateral loads on the six H-piles which will support the future widening to the north (as a result of the fill placement for the future northerly embankment widening), it has been recommended by Golder that the first 20 m of the proposed future widening to the north be constructed as part of this contract. This will allow settlement mitigation techniques to be implemented for the north embankment widening prior to the installation of the H-piles which will help to reduce the drag loads and lateral loads which might otherwise occur on the piles in the future.



Through discussions between Golder and MTO foundations, it is understood that the site has been identified as a well-suited location to carry-out evaluation and verification of the actual downdrag forces (i.e. drag loads) acting on the new H-Piles installed adjacent to the widened portion of the approach embankment.

As part of the bridge replacement, the existing Highway 9 will be re-aligned to the south, with the centerline of the new bridge/highway shifted to the south by about 9 m and the bridge widened accordingly to accommodate this change to the alignment. The road grade of the existing Highway 9 was initially proposed to be raised by about 0.75 m with the new fill for the highway widening(s) requiring a corresponding thickness of approximately 2.5 m. Following discussions between Golder, AECOM and MTO, considering the challenges associated with limiting the settlements of the new widened and raised embankments founded on the thick clayey strata at the site, AECOM revised the new roadway vertical profile to limit the grade raise to about 0.3 m, thereby reducing the required thickness of the new fills to about 2.0 m.

A 100 mm (4 inch) diameter, steel cased, high pressure Enbridge Gas Line is located on the site approximately 15 m south of the south crest of the existing highway embankments. Settlement analysis along the alignment of the existing gas line was carried out to estimate the settlements due to the construction of the new embankment widenings. As a result of this settlement assessment, it is our understanding the gas line is to be re-aligned to the north side of Highway 9 in advance of the new bridge and embankment construction.

It is understood that the construction for the bridge replacement and highway widening will be carried out in stages. Stage 1 will involve construction of the southerly, and a portion of the northerly, embankment widenings (including associated settlement mitigation measures) followed by construction of the southern half of the new abutments and bridge while maintaining traffic flow on the existing Highway 9. In Stage 2, the traffic will be shifted onto the new southern half of the bridge and embankments, while the northern half of the new abutments and bridge is constructed and the existing Highway 9 embankment is re-constructed and grade raised.

## 6.3 General Foundation Design Context

### 6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code* CAN/CSA S6-14 (CHBDC (2014)) and its *Commentary*, the bridge structure and its foundation system may be classified as having large traffic volumes and their performance as having potential impacts on other transportation corridors, resulting in a “Typical Consequence Level” associated with exceeding limit states design.

For proposed temporary works where the design life is less than two years, such as the temporary condition associated with the settlement mitigation option that would require the placement of surcharge loads on the approach fills (in order to accelerate consolidation and the dissipation of pore pressures), a “Low Consequence Level” could be used as per Section C6.4.1.1 of the *Commentary* to the CHBDC (2014).

Based on the level of foundation investigations completed to date at this location in comparison to the degree of site understanding in Section 6.5 of CHBDC (2014), the level of confidence for design for the Highway 9 - Holland Drainage Canal Bridge has been assessed as a “Typical degree of site and prediction model understanding” based on having boreholes at / near each foundation element and through the approach embankments.

The corresponding consequence factor(s),  $\Psi$ , and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$ , from Tables 6.1 and 6.2 of the CHBDC (2014) have been used for the appropriate aspects of the foundation design.



### 6.3.2 Correlation of Automatic and Manual Hammer for SPT “N” Values

The results of the 2018-2019 investigation generally demonstrate that lower Standard Penetration Test (SPT) “N”-values were measured within the various soil strata at this site than those recorded in the boreholes from the 1965 investigation (GEOCRE No. 31D-025). The differences are anticipated to be largely due to the use of an automatic hammer with higher efficiency in the 2018-2019 investigation as compared to a manually operated (i.e., rope-and-cathead) hammer that would most likely have been used in the 1965 investigation. The 2018-2019 SPT “N”-values correlate reasonably well with the 1965 data when corrected (as suggested in CFEM 2016) to a 60% efficiency of hammer energy transfer. The foundation options and recommendations presented below are based on the correlated “N<sub>60</sub>”-values, where applicable.

### 6.3.3 Seismic Design

#### 6.3.3.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation and laboratory testing. The measured shear wave velocity of soils up to 30 m below founding level (based on the results from SCPT19-02) were used to define the seismic site classification in accordance with Table 4.1 of the CHBDC (2014). Based on this methodology, it is considered that a Site Class D would be applicable for the design of the new Highway 9 – Holland Drainage Canal Bridge.

#### 6.3.3.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the CHBDC (2014) and as obtained from the NRC (2017) website, the peak ground acceleration (PGA) and peak ground velocity (PGV) values and design spectral acceleration (Sa) values for Site Class D (adjusted from the reference Site Class C in accordance with Section 4.4.3.3 of the CHBDC (2014)) are presented below.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.040	0.062	0.102
PGV (m/s)	0.040	0.062	0.100
Sa (0.2) (g)	0.067	0.102	0.160
Sa (0.5) (g)	0.054	0.079	0.122
Sa (1.0) (g)	0.033	0.050	0.076
Sa (2.0) (g)	0.016	0.025	0.039
Sa (5.0) (g)	0.003	0.006	0.009
Sa (10.0) (g)	0.001	0.003	0.004

#### 6.3.3.3 Soil Liquefaction

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface deformations)



and under undrained conditions generate excess pore water pressures. The excess pore water pressures may also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction of slopes or banks along rivers and other shorelines.

The liquefaction susceptibility of the cohesive and non-cohesive soils at this site was evaluated by comparing the cyclic stress ratio (based on penetration resistance) required to trigger liquefaction with the available cyclic resistance ratio (based on penetration resistance). Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required.

The methodology used to assess liquefaction potential at the site is consistent with that presented in the *Commentary to the CHBDC, 2014*. It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction analysis was carried out using in-situ testing data collected at the borehole, CPT and SCPT locations. The results of the liquefaction assessment indicate that the soils at the site are not considered susceptible to liquefaction or cyclic mobility/softening during the 2,475-year design earthquake.

## 6.4 Foundation Options

Based on the proposed bridge geometry and the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments for the new Holland Drainage Canal Bridge structure. A comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip or spread footings:** Although the abutments could be founded on the silty sand to sand deposit at about Elevations 213 m to 215 m, the spread / strip footings would have to be relatively large to provide a suitable geotechnical resistance. In addition, excessive settlements are anticipated due to the underlying extensive silty clay to clayey silt deposit. As such, spread / strip footings are not considered feasible at this site, and are not discussed further in this report.
- **Steel H-piles or pipe piles driven into/terminating within the lower clayey silt to silty clay deposit:** Refusal (as defined by 3 m of material having an SPT “N” value >100 blows per 0.3 m) was not encountered within the depth of investigation (up to about 60 m), and as such, end-bearing driven piles are not feasible at this site. Friction piles that terminate within the lower portion of the lower clayey silt to silty clay stratum are considered feasible to support the new abutments at the site and several different pile sizes and lengths (and the associated geotechnical resistances) are provided in Section 6.5.

It is understood that the existing bridge is supported on relatively short (i.e. approximately 14 m long), timber piles driven into the upper portion of the lower clayey silt to silty clay deposit. Although there may be a desire to consider adopting a similar (i.e. short-pile) foundation design, it should be realized that the existing bridge is a narrow, relatively light, 3-span structure founded on 74 timber piles. In order to support the new, significantly wider and heavier, single-span bridge on similar short-piles, it is estimated that more than twice as many piles as the long pile option (being considered for the current design) would be required. This significantly greater number of piles could not be accommodated in a single row, integral abutment design and it would be necessary to use a two-row pile group and a semi-integral abutment design which is less desirable.



In addition, the settlement performance of a group of short piles terminated in the softest portion of the clayey stratum would be a design challenge and may result in settlements in excess of 25 mm.

- **Drilled shafts (caissons):** Drilled shafts (caissons) founded on an end-bearing stratum are not considered feasible for support of the new abutments as refusal conditions was not encountered within the depth of investigation (up to about 60 m). Friction caissons, while technically feasible, would require a relatively long shaft to obtain adequate geotechnical resistances while limiting settlements to less than 25 mm. Such long drilled shafts would be expensive to construct in the firm to stiff clayey soils while maintaining the integrity of the foundation soils that are currently supporting the existing structure (i.e. deep liners and careful construction methods to balance pressure and avoid soil squeeze would be required). In addition, a conventional drilled shaft design would not allow an integral abutment design for the structure.

Consideration could be given to adopting a modified drilled shaft that incorporates an H-pile section in the upper portion of the pile to provide the flexibility required for an integral abutment design. However, such a design would still require the drilled shafts to be terminated within the lower portion of the lower clayey silt to silty clay stratum in order to limit the settlements of the larger diameter piles to less than 25 mm. In addition, the larger diameter drilled shafts would be more affected by drag loads which could exacerbate the settlements. Finally, with a larger diameter drilled shaft, there would be a higher risk of conflict with the existing timber piles during installation (as discussed below). As such, drilled shafts (conventional or hybrid) and are not considered practical at this site and are not discussed further in this report.

Due to the risk of the new piles coming in contact with the existing piles during installation and driving, it is recommended that the new foundations (and associated piles) be located behind the existing abutments. However, we understand that the design preference for the structure is to locate the new abutments in front of the existing abutments. If the new piles are installed in front of the existing piles, the appropriate risk mitigation strategies outlined in the Risk Register submitted to MTO by AECOM (included in Appendix E), and summarized in Section 6.4.1, should be implemented. It is noted that the existing timber piles are battered inward (towards the canal) and reportedly spaced at 1,270 mm center-to-center. As such, the larger the pile section adopted in the design, the higher the risk of coming in contact with an existing pile during the construction. In this regard, adopting the smaller cross-section steel H-pile option is preferred from a foundations perspective over the use of a larger diameter drilled shaft. Further, the capacity of the driven steel H-pile option also has the advantage of being checked during construction by high-strain dynamic (PDA) testing (which is more difficult for the larger diameter drilled shaft). If conflicts between the existing and new piles require the removal and re-installation during construction, and if as a result the effect on the capacity of the new piles is questioned, the capacity of the new pile(s) could be checked by testing during construction.

Based on the considerations outlined above and discussed in Table 1, the preferred option from a geotechnical / foundations perspective is to support the abutments for the proposed bridge on driven steel H-piles acting as friction piles founded within the lower silty clay to clayey silt stratum. In addition, as noted above, from a foundations perspective, it is also recommended that the new abutment piles be located behind the existing abutments to avoid conflicts with the existing timber piles.

#### 6.4.1 Risk Mitigation Strategies

It is understood that the preferred alternative from a structural perspective is to locate the new abutments, and install the new abutment piles, in front of the existing abutment foundations as this would result in a shorter bridge span (i.e. about 23 m in length rather than up to 31 m in length for the alternative of locating the new abutments



behind the existing abutments). Additionally, the longer spanned bridge (i.e., 31 m), would require a higher grade raise on the approach embankments (greater than 0.75 m), which would induce additional settlements. As noted above, there is a risk that the new piles may come in contact with the existing piles if the shorter spanned bridge option is selected. To address these risks, a Risk Register (see Appendix E) was submitted to MTO by AECOM; the foundations related risk mitigation strategies for the construction of the abutment foundations for the shorter spanned bridge are outlined below.

Risk	Foundations Risk Response Strategy
Based on the design information, the front row of the existing timber piles are battered inward (toward the canal) and spaced at 1,270 mm centre-to-centre. The new piles would need to fit between the existing piles but there is a potential for conflict if the existing piles were installed at different spacings than shown on the design drawings.	<p>The size (width or diameter) of the piles will need to be restricted to one that can easily accommodate the existing pile spacing with as much spacing flexibility as possible. As such, narrower driven steel H-piles are preferred over larger diameter drilled shafts.</p> <p>The existing abutment pile cap will need to be sub-excavated and removed in order to expose the tops of existing piles to accurately locate the new piles and reduce the chance of interference between the new and the old piles during installation. A Non-Standard Special Provision notifying the contractor of the need to survey the locations of the existing piles after removing the existing pile caps and provide this information to the designer to be included in the Contract Documents. A sample NSSP (Interference with Existing Piles – Pre-Augering and Pile Extraction) is included in Appendix F.</p>
The actual as-built pile spacing (of the new foundations) may impact pile capacity due to group effects if the location(s) of the new piles are required to be adjusted to avoid conflict with the existing piles.	The current pile group design could be modified to include conservatism in the capacity to account for potential closer pile spacing; cannot anticipate all scenarios but will reduce time for re-design during construction.
The existing piles are spaced such that three or more piles are too close to allow new piles to fit between the existing; the Contractor would need to remove one or more of the existing pile(s).	Guidance on the removal of existing piles (if necessary, but as a last resort) to be compiled and included in the Contract Documents in case piles are required to be removed. A Non-Standard Special Provision notifying the contractor of the process for considering and obtaining approval for the removal of an existing pile to be included in the Contract Documents. A sample NSSP (Interference with Existing Piles – Pre-Augering and Pile Extraction) is included in Appendix F.
The new piles may come in contact with the existing piles, even if the existing pile spacing is known, potentially resulting in a reduced pile capacity or a non-uniform pile spacing or out of alignment piles.	Assessment of the new pile capacity to be calculated (or checked by PDA testing) during construction, if required.
The new piles may come in contact with the existing pile and deflect beyond the tolerable range and will need to be removed and driven again.	Guidance on the process to allow removal of the new pile and re-installation to be compiled and included in the Contract Documents in case piles are required to



Risk	Foundations Risk Response Strategy
	<p>be removed and re-driven. A Non-Standard Special Provision notifying the contractor of the process for considering and obtaining approval for the removal and reinstallation of a new pile to be included in the Contract Documents. A sample NSSP (Interference with Existing Piles – Pre-Augering and Pile Extraction) is included in Appendix F.</p> <p>Abutment thickness to be over-sized to allow a greater tolerance in the final pile position.</p> <p>Assessment of the new pile capacity to be calculated (or checked by PDA testing) during construction, if necessary.</p>

At the 30 % executive presentation meeting with MTO, the following foundation-related design considerations were included:

- Pre-augering to a depth of 5 m prior to installation of the new steel H-piles to check that conflicts with the existing piles will not occur. Pile spacing to be adjusted if necessary after pre-augering;
- Elimination of the Corrugated Steel Pipe (CSP) in the integral abutments to save space and allow more flexibility in the adjustment of the new pile spacing, if necessary; and,
- Modification of the pile group design to include conservatism in the capacity to account for potential closer pile spacing.

The design considerations outlined in the bullet points above have been incorporated into the foundation recommendations provided below.

## 6.5 Deep Foundations

### 6.5.1 Steel H-Pile Foundations

Driven steel-H-piles founded within the lower silty clay to clayey silt deposit are considered feasible for the support of the new abutments. Due to the lack of refusal conditions encountered at this site, the majority of the pile resistance will be derived from the shaft resistance and as such, relatively long piles will be required to achieve the required design resistances. It is recommended that pile be driven to Elevation 173 m or lower, i.e., 45 m or longer piles, in order to found the piles within the very stiff portion of the lower silty clay to clayey silt deposit.

The factored ultimate and serviceability geotechnical axial resistances for three different pile sizes and four different pile lengths (with corresponding pile tip elevations) are provided below to allow the structural designer to optimize the foundation design from a structural and economical perspective. The pile tip elevations have been calculated based an assumed underside of pile cap at Elevation 218.0 m.



Pile Type	Pile Length (m)	Estimated Design Pile Tip Elevation (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance, for 25 mm of Settlement (kN)
HP310 x 110	30	188	500	>500
	40	178	700	>700
	45	173	800	>800
	50	168	900	>900
HP310 x 132	30	188	500	>500
	40	178	700	>700
	45	173	800	>800
	50	168	900	>900
HP 360 x 132	30	188	600	>600
	40	178	800	>800
	45	173	900	>900
	50	168	1,000	>1,000

Based on discussions with the structural designer, we understand that a factored geotechnical resistance of 900 kN/pile is required for the design of the replacement bridge foundations. Given this and the information summarized above, it is our understanding that the HP360x132 pile (45 m in length) will be adopted for the design.

As discussed in Section 6.5.2 (below) it is anticipated that drag loads will be imposed on the steel H-piles given the nature of the very deep clayey strata at this site. The drag loads will have to be considered in the assessment of the structural resistance of the piles.

The following note (Note 7 from the MTO *Structural Manual*, Section 3.3.3 (MTO, 2016)), should be shown on the Contract Drawing:

**“PILES TO BE DRIVEN TO ELEVATION 173.00”**

Pile installation should be in accordance with Ontario Provincial Standard Specification (OPSS).PROV 903 (*Deep Foundations*) as amended by the Special Provision (SP) for Dynamic Testing (included in Appendix F).

Pile dynamic analyzer (PDA) testing should be completed on at least 10% of piles or two piles (whichever is greater) at each foundation element in each stage of construction. The Contract Documents must include the SP for Dynamic Testing (as included in Appendix F) that has been developed to amend OPSS.PROV 903 to address PDA testing, including specifying the minimum number of piles to be tested by PDA. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile and results of the PDA testing; the criteria must therefore be established at the time of construction after the piling equipment is known to ensure that the piles are not overdriven and to avoid possible damage to the piles.

Assessment of the ultimate geotechnical resistance by Pile Dynamic Analyzer (PDA) testing should commence once the pile reaches a depth within about 1.5 m above the design pile tip elevation shown above and at 0.5 m



intervals of depth until the ultimate axial resistance is achieved or the design pile tip elevation is reached. If the ultimate capacity as determined by PDA testing is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for no less than two weeks (CFEM, 2006) and PDA testing should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the two week wait period, the Contract Administrator should be notified and authorization given prior to driving the pile below the design pile tip elevation. The two week wait period should be included in a note on the Contract Drawings.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design, the CSPs should be backfilled with loose, fine to medium sand. A Non-Standard Special Provision (NSSP) detailing the installation method and gradation of this sand should be included in the contract documents if CSPs are to be included as part of the design; an example is included in Appendix F. The annulus between the pre-augered hole and the CSP should be backfilled with OPSS.PROV 1010 (*Aggregates*) Granular B Type II material. Alternatively, given the subsurface conditions at the site, consideration could be given to loosening the soils below the pile cap by pre-augering to a depth of about 5 m below the elevation of the underside of pile cap. However, the integral abutment design will have to be checked using the lateral resistances indicated for this option (in Section 6.5.3) to confirm that this approach will provide the flexibility required.

Pile caps for the abutments should be provided with a minimum of 1.5 m of soil cover to provide adequate protection against frost penetration as interpreted from Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Depths for Southern Ontario*).

### 6.5.2 Downdrag and Drag Loads

As a result of the loading from the new widened approach embankments (including the up to 4.5 m deep subexcavation and replacement of the organic peats with granular backfill), long-term consolidation settlement and creep settlement of the underlying cohesive deposit will occur. It is estimated that the immediate settlement of the non-cohesive / granular deposit(s) under the full height embankment loading at the abutment will be up to about 35 mm and the consolidation settlement will be up to about 75 mm and that it will take about 20 years for about 90 per cent of the consolidation settlement to occur. With this magnitude of settlement, downdrag loads are expected to be imposed on the piles. The difference in the vertical movement between the thick overburden (i.e., from the consolidation settlement and creep of the cohesive deposits) and the long piles (i.e., from the elastic deformation of the piles under the load from the bridge structure and from the punching of the piles into the soil deposit below the pile tip) will result in the development of negative skin friction, downdrag and drag loads on the piles.

Analyses to estimate the drag loads acting on the recommended pile foundation option at the abutments was carried out in accordance with Section 6.11.4.10 of *CHBDC and its Commentary* using the method proposed by Briaud and Tucker (1996). It is noted that the method used to assess the deformation of the pile and the associated drag load is dependent on a number of factors including the pile length, foundation conditions at the pile tip, the unfactored dead load on the pile and the anticipated post-construction settlement profile of the foundation soils following installation of the piles. If any of these factors is different from those assumed in the analysis, the estimated drag loads would need to be reassessed.

For the piles installed below/adjacent to the new widened portion of the approach embankments following the construction of the widened embankment including the removal/replacement of any peat encountered, the position of the neutral plane for each of the proposed pile types, assuming a pile tip elevation of 173 m (pile length of 45 m), and the associated unfactored drag loads are shown below.



Pile Type	Elevation of Neutral Plane (m)	Unfactored Drag Load (kN)
HP 310x110	191.0	1,300
HP 310x132	190.8	1,350
HP 360x132	190.5	1,600

For the piles installed below/adjacent to the existing embankment, the position of the neutral plane and the unfactored drag loads as a result of the proposed 0.30 m grade raise are shown below.

Pile Type	Elevation of Neutral Plane (m)	Unfactored Drag Load (kN)
HP 310x110	194.5	1,100
HP 310x132	194.0	1,150
HP 360x132	193.4	1,350

For this project, it is recommended that installation of wick drains in conjunction with preloading and surcharging be carried out for the widened portions of the new approach embankments (prior to the installation of the piles) to reduce the long-term post-construction settlements at the site. In addition, it is recommended that lightweight fill (cellular concrete) be utilized within the grade raise portions of the approach embankments (below the existing roadway) to reduce the post-construction settlements. Further details on settlement mitigation options are provided in Section 6.7.4. Although these mitigation options are predicted to reduce the settlements to an amount tolerable for the design of the approach embankments (i.e. to about 25 mm at the abutments), based on the calculated position of the neutral plane and the trend of the estimated remaining settlement with depth, the mitigation measures are not anticipated to have a significant effect on reducing the drag loads on the piles. This is in part due to the fact that the settlement mitigation measures (i.e. wick drains) do not extend to the full depth of the clayey stratum and in part due to the fact that in theory, only a relatively small magnitude of differential movement (i.e. about 5 mm to 10 mm) between the pile and soil is required to fully mobilize the drag loads. It is for this reason that as noted in Section 6.2, the site has been identified as an ideal location to install instrumentation to monitor and verify the actual drag loads acting on one the new abutment piles following completion of the new approach embankments.

In accordance with the requirements of CFEM (2006), an assessment is required to be carried out to check if the structural capacity of the steel H-piles can accommodate the factored dead load on the piles combined with the factored drag load indicated above.

The pile settlements due to the drag loads are anticipated to be less than 25 mm.

### 6.5.3 Resistance to Lateral Loads

Resistance to lateral loading may be derived using vertical piles, with enhanced support offered by inclined (battered) piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in



front of the piles, whereas inclined piles derive lateral resistance from the soil in front of the piles as well as from the horizontal component of the axial load present in the inclined pile. For integral abutment design, the steel H-piles may be required to be installed within a 3 m long corrugated steel pipe (CSP) filled with sand in accordance with the NSSP for integral abutments (a copy of which is included in Appendix F), if deemed necessary by the structural designer.

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory (as outlined below). However, it should be noted that the response of a pile to lateral loads is highly non-linear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1 per cent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behavior of the soil should be considered by the use of P-y curves.

The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory as suggested in CHBDC (2014) Commentary (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction,  $k_h$ , (kPa/m) is based on the equation(s) given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (CFEM, 1992).

For non-cohesive soils:

$$k_h = \frac{n_h z}{B} \quad \text{Where} \quad \begin{array}{l} n_h \text{ is the constant of subgrade reaction (kPa/m);} \\ z \text{ is the depth (m); and} \\ B \text{ is the pile diameter or width (m).} \end{array}$$

For cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{Where} \quad \begin{array}{l} s_u \text{ is the undrained shear strength of the soil (kPa); and} \\ B \text{ is the pile diameter or width (m).} \end{array}$$

The following values of  $n_h$  (Terzaghi, 1955) and  $s_u$  may be incorporated into the calculations of horizontal subgrade reaction ( $k_h$ ) for the structural analyses for a single vertical pile, based on the interpreted stratigraphic profiles shown on Drawings 2 to 4. The ranges in values reflect the variability in the subsurface conditions, the soil properties, and the approximate nature of the linear-elastic subgrade reaction analysis.

Foundation Element	Soil Unit		Elevation (m)	$n_h$ (kPa/m)	$s_u$ (kPa)
West Abutment	Assuming CSP is used	Loose sand within CSP	218.5 – 215.5	2,000	-
		Soft to firm clayey silt	215.5 – 215.0	-	40
		Compact to dense silt and sand to sand (below groundwater level)	215.0 – 210.5	8,000	-
	Assuming pre-augering to a	Very loose to compact silt and sand to silty sand fill	218.5 - 216	2,000	-



Foundation Element	Soil Unit		Elevation (m)	$n_h$ (kPa/m)	$s_u$ (kPa)
	depth of 5 m prior to installation of the steel H-piles to avoid conflicts with the existing piles	Soft to firm clayey silt	215.5 – 215.0	-	25
		Compact to dense silt and sand to sand (below groundwater level)	215.0 – 213.5	4,000	-
		Compact to dense silt and sand to sand (below groundwater level)	213.5 – 210.5	8,000	-
	Stiff clayey silt		210.5 – 205.0	-	60 (top) to 40 (bottom)
	Firm silty clay to clayey silt		205.0 – 201.0	-	40
	Firm to stiff silty clay to clayey silt		201.0 – 176.0	-	40 (top) to 90 (bottom)
	Stiff silty clay to clayey silt		176.0 – 168.0	-	120
East Abutment	Assuming CSP is used	Loose sand within CSP	218.5 – 215.5	2,000	-
		Soft to firm clayey silt	215.5 – 213.0	-	40
	Assuming pre-augering to a depth of 5 m prior to installation of the steel H-piles to avoid conflicts with the existing piles	Soft to firm clayey silt fill	218.5 – 215.5	-	25
		Very loose to compact silt and sand fill	218.5 – 215.5	2,000	-
		Soft to firm clayey silt	215.5 – 213.5	-	25
		Soft to firm clayey silt	213.5 – 213.0	-	40
	Loose to dense silt and sand to sand (below groundwater level)		213.0 – 209.0	8,000	-
	Stiff clayey silt		209.0 – 205.0	-	60 (top) to 40 (bottom)
	Firm silty clay to clayey silt		205.0 – 201.0	-	40
	Stiff silty clay to clayey silt		201.0 – 176.0	-	40 (top) to 90 (bottom)
	Stiff silty clay to clayey silt		176.0 – 168.0	-	120



As noted above, the response of a pile to lateral loads is highly non-linear which the conservative subgrade reaction approach outlined above cannot accurately capture. In order to model the non-linear nature of the soil behavior, the P-y curves included in Appendix G can be utilized.

For a single driven HP 360x132 pile, the estimated factored serviceability geotechnical resistance for 10 mm of horizontal deflection at the pile cap is estimated to be 40 kN assuming the upper 5 m of soil below the pile cap has been pre-augered. This value is based on analyses carried out using the commercially available program LPILE Plus (Version 5.0), developed by Ensoft Inc.

Based on the above, both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at Ultimate Limit States (ULS). At Serviceability Limit States (SLS), the horizontal reaction of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction ( $k_h$ ) of the soil as discussed above. The SLS reaction should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (CHBDC (2014) Commentary Section 6.11.2.2).

Group action for lateral loading should also be evaluated by reducing the coefficient of horizontal subgrade reaction either in the direction of loading or perpendicular to the direction of loading by relevant group pile efficiency factors as outlined in Section C6.11.3.4 of the *Commentary to the CHBDC* (2014).

## 6.6 Lateral Earth Pressures for Design of Abutments and Wingwalls

The lateral earth pressures acting on the abutment walls and any associated wingwalls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Depending on the Seismic Performance Category for the proposed bridge structure, seismic (earthquake) loading may also have to be taken into account in the design.

The following recommendations are made concerning the design of the abutment/wing walls:

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular B Type II, should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*).
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill in accordance with OPSD 3102.100 (Walls, Abutment, Backfill, Drain) and OPSD 3190.100 (Wall, Retaining and Abutment, Wall Drain). Where cellular concrete is used as backfill immediately behind the abutment wall, a prefabricated, vertical drainage composite sheet shall be installed between the cellular concrete and the back of the abutment wall which connects with the longitudinal drains and weep holes described above.
- Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement) and OPSD 3121.150 (Walls, retaining, Backfill, Minimum Granular Requirement).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Hand-operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design, as required.



- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.5 m behind the back of the wall as shown on Figure C6.20(a) of the Commentary to the CHBDC (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line flatter than 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing or pile cap as shown on Figure C6.20(b) of the Commentary to the CHBDC (2014).

### 6.6.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the assessment of lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat. Where there is sloping ground behind the wall, the coefficient(s) of lateral earth pressure will need to be adjusted to account for the slope as per the Commentary to the CHBDC (2014) Section C6.12.1.

- For a restrained wall, the pressures are based on the fill behind the granular backfill zone, and the following parameters (unfactored) may be used assuming the use of earth fill:

Material	Earth Fill
Soil Unit Weight:	20 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure: Active, $K_a$ At rest, $K_o$	0.36 0.53

- For an unrestrained wall, the pressures are based on the engineered granular fill within the backfill zone, and the following parameters (unfactored) may be used:

Material	Granular 'A'	Granular 'B' Type II	Cellular Concrete
Soil Unit Weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>	5 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure: Active, $K_a$ At rest, $K_o$	0.27 0.43	0.27 0.43	0.27 0.43

- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.
- If the wall support and superstructure allow lateral yielding (i.e., unrestrained structure), active earth pressures may be used in the geotechnical design of the structure.

The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the CHBDC (2014).



## 6.6.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading may have to be taken into account in the design of abutment / wingwalls in accordance with Section 4.6.5 of the CHBDC (2014). In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and/or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC (2014) and its Commentary, for structures which allow lateral yielding, the horizontal seismic coefficient,  $k_h$ , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA. For structures that do not allow lateral yielding,  $k_h$  is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient  $k_v$  is taken as zero.
- The following seismic active pressure coefficients ( $K_{AE}$ ) may be used in design; these coefficients reflect the maximum  $K_{AE}$  obtained for each of the earthquake design periods and backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

	Design Earthquake	Site PGA	Seismic Active Pressure Coefficients, $K_{AE}$			
			Granular A	Granular B Type II	Earth Fill	Cellular Concrete
<b>Yielding Wall (Unrestrained)</b>	<b>475-Yr</b>	0.040g	0.26	0.26	0.29	0.26
	<b>975-Yr</b>	0.062g	0.26	0.26	0.30	0.26
	<b>2,475 Yr</b>	0.102g	0.27	0.27	0.31	0.27
<b>Non-Yielding Wall (Restrained)</b>	<b>475-Yr</b>	0.040g	0.27	0.27	0.30	0.27
	<b>975-Yr</b>	0.062g	0.28	0.28	0.32	0.28
	<b>2,475 Yr</b>	0.102g	0.31	0.31	0.34	0.31

- The  $K_{AE}$  value for a yielding wall is applicable provided that the wall can move up to  $250k_h$  mm, where  $k_h$  is the site-specific PGA as given in the table above. This corresponds to displacements of 10 mm, 15 mm, and 25 mm for the 475-year, 975-year, and 2,475-year design earthquakes, respectively at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be calculated per Section C4.6.5 of the *Commentary* to CHBDC (2014).



## 6.7 Approach Embankments

The proposed construction of the replacement of the Highway 9 – Holland Drainage Canal Bridge requires an embankment grade raise and widening / alignment shift to the south of the existing Highway 9. The original design by AECOM (for the bridge design having the new abutments located in front of the existing abutments), required the existing Highway 9 road grade to be raised by about 0.75 m with new fills approximately 2.5 m thick required in the area of the embankment widening(s). However, as noted in Section 2.0, after discussions between Golder, AECOM and MTO, in order to reduce the settlements of the new widened and raised embankments founded on the thick clayey strata at the site, in December 2019 AECOM revised the new roadway vertical profile to limit the grade raise to about 0.3 m, thereby reducing the required thickness of the new fills to about 2.0 m.

All of the analyses below assume that prior to construction of the new approach embankments, all topsoil, peat/organic soil, and existing surficial fill materials will be stripped from the footprint of the new widened embankments. Based on the boreholes at the abutments and in the approach embankment areas, subexcavation of peat/organic soil will be required to as deep as about Elevation 215.0 m within the footprint of the widened approach embankments. The existing ground surface elevation within the footprint of the areas to be subexcavated ranges from about Elevation 219 m to Elevation 221 m and as such, excavations up to as deep as 6 m may be required. Additional recommendations for the subgrade preparation and embankment construction are provided in Section 6.10.1.

### 6.7.1 Global Stability

The following subsections outline the method used to evaluate static global stability of the proposed approach embankments. The geotechnical soil parameters used in the analyses are also presented. The results of the stability analyses are summarized in Section 6.7.3 where they are discussed together with the results of the settlement analyses along with recommendations regarding possible design and construction alternatives to mitigate post-construction settlement.

#### 6.7.1.1 Method of Analysis

Two-dimensional limit equilibrium slope stability analyses were performed using the commercially available program Slide 2018 (Version 8.014), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. Morgenstern-Price is a general method of slices which is based on equilibrium of forces and moments acting on each slice of soil mass above the potential failure surface. For all analyses, the Factors of Safety (FoS) of numerous potential failure surfaces were computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor,  $\Psi$ , and the geotechnical resistance factor,  $\phi_{gu}$ . (i.e.,  $FoS = 1/(\Psi \cdot \phi_{gu})$ ). Accordingly, a minimum FoS of 1.33 and 1.54 has been used for the design of the embankments and retaining walls for consideration of the global stability under short-term/temporary and long-term/permanent static conditions, respectively as per Table 6.2 of CHBDC (2014). For the temporary construction works where the design life is less than two years (i.e., the temporary condition associated with the placement of the surcharge on the approach fills as discussed further in Section 6.7.4.2), a target FoS of 1.16 has been used for the short-term / temporary condition considering that the consequence of failure of the surcharge fill (in the widening area where traffic is not present) is considered low.

Stability analyses have been completed for various representative sections of the proposed approach embankments, using the geometry shown in the typical cross sections provided by AECOM. The soil stratigraphy was simplified to represent the subsurface conditions encountered in the nearest boreholes. The critical sections



for analysis were selected based on the greatest embankment height and the “poorest” (i.e., “weakest”) soil conditions encountered within the area.

In the analyses, the groundwater level was interpreted to be at Elevation 217.7 m based on the groundwater conditions encountered within the monitoring wells from the recent borehole investigation.

### 6.7.1.2 Soil Shear Strength Parameters

For the non-cohesive soils present at the site, the effective stress parameters employed in the analyses were estimated from the results of the of the laboratory CID triaxial test and direct shear test as well as from correlations based on the in-situ Standard Penetration Tests (SPT) as proposed by Peck et al (1974) and U.S. Navy (1986). The parameters as estimated from the lab tests and from the correlations were adjusted, if necessary, using engineering judgment based on precedent experience in similar soil conditions, where appropriate.

For the cohesive deposits, total stress parameters were employed in the analyses of the short-term, undrained conditions (i.e., temporary conditions). The total stress parameters (i.e., average mobilized undrained shear strength –  $s_u$ ) for the cohesive soils were estimated from corrected field vane tests (based on using Bjerrum’s correction method), from the laboratory oedometer tests (following the correlation proposed by Mesri, (1975)), and from the CPT results (based on Mesri (1975) and by Demers and Leroueil (2002)). A plot of the undrained shear strength versus elevation for the cohesive soils encountered at the site during the geotechnical investigation are shown on Figure 1. Effective stress parameters were also assigned to the cohesive deposits to evaluate the stability for the long-term, drained conditions (i.e., permanent conditions). The effective stress parameters (i.e., effective cohesion ( $c'$ ) and effective friction angle ( $\phi'$ )) for the cohesive deposits were estimated based on the results of the laboratory CIU and CID triaxial tests, as well as from empirical correlations based on the plasticity index. The empirical correlations proposed by Mitchell (1993), Kulhawy and Mayne (1990), and Ladd et al. (1977) were employed to check the parameters estimated from the triaxial tests.

For the existing silt and sand to silty sand embankment fill, effective stress parameters were employed in the analysis for both the short-term and long-term conditions. The effective stress parameters (i.e., effective cohesion ( $c'$ ) and effective friction angle ( $\phi'$ )) for the cohesive fill were estimated based on the in-situ Standard Penetration Tests (SPT) using the correlations proposed by Peck et al (1974) and U.S. Navy (1986). The parameters as estimated from the correlations were adjusted, if necessary, using engineering judgment based on precedent experience in similar soil conditions, where appropriate.

Summarized below is the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types at the site. The results of the stability analyses are discussed in Section 6.7.3.1.

Soil Deposits used in Analyses at Simplified Representative Sections	Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	Effective Cohesion, $c'$ (kPa)	Effective Friction Angle, $\phi'$ (°)	Undrained Shear Strength, $s_u$ (kPa)
New embankment fill (compacted OPSS Granular A or Granular B Type II)	21	-	35 to 45	-
Existing silt and sand to silty sand embankment fill	20	-	28	-
Upper clayey silt	20	0	32	40



Soil Deposits used in Analyses at Simplified Representative Sections	Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	Effective Cohesion, $c'$ (kPa)	Effective Friction Angle, $\phi'$ (°)	Undrained Shear Strength, $s_u$ (kPa)
Silt and sand to sand	19.5	-	35	-
Clayey silt crust	20	0	32	66 to 44 <sup>1,2</sup>
Lower silty clay to clayey silt - Layer 1	20	0	20	44 to 41.5 <sup>2</sup>
Lower silty clay to clayey silt - Layer 2	20	0	20	41.5 to 90.8 <sup>2</sup>
Lower silty clay to clayey silt - Layer 3	20	0	35	-

**Notes:**

1. Soil unit is expected to remain in the over-consolidated state under new embankment loading.
2. Undrained shear strength decreases / increases linearly from the top of the deposit to the bottom of the deposit as shown on Figure 1.

The following is a summary of the new embankment side slope inclination, unit weight and effective friction angle(s) used for the new granular fill modelled in the slope stability analyses.

Fill Type	Recommended Slope Inclination	Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	Effective Friction Angle, $\phi'$ (°)	Effective Cohesion, $c'$ (kPa)
Granular Fill (main embankment)	2H:1V	21	35 to 45	0
Granular Fill <sup>1</sup> (surcharge)	1H:1V (for slopes facing existing Hwy 9 and at abutments) 2H:1V (for slopes facing away from existing Hwy 9)	21	35 to 45	0

**Note:**

1. In order to limit the size of the footprint and volume of fill required for the surcharge, it is recommended that consideration be given to constructing the side slopes of the temporary surcharge fill at 1H:1V for the slopes facing the existing Highway 9 roadway. In addition, a higher friction angle (i.e.  $\phi'=45^\circ$ ) has been considered for the OPSS Granular fill in some of the stability analyses to better assess the potential for surficial/localized side slope instability which may require maintenance during the surcharge period.

## 6.7.2 Settlement

The following subsections outline the methods used to carry out the settlement analyses at the proposed approach embankments for the realigned highway. The analyses include the assumption that all existing peat/organic layer(s) are removed and replaced with granular fill as part of the new construction (including consideration of the heavier unit weight for the replacement granular material). The results of the analyses are presented in Section 6.7.3 where they are discussed together with the results of the stability analyses along with recommendations regarding potential



design and construction alternatives to mitigate stability issues and/or post-construction settlement, where applicable.

### 6.7.2.1 Method of Analysis

To estimate the magnitude of expected settlement, analyses were carried out at the west and east approach embankments. Settlement analyses were carried out using the commercially available program *Settle*<sup>3D</sup> (Version 4.0), developed by Rocscience Inc. The stress distribution calculations used in the settlement analyses were based on the Westergaard (1938) solution.

For the purpose of the settlement analysis, a geotechnical resistance factor ( $\phi_{gu} = 0.8$ ) has been used for the design of the approach embankments when evaluating the settlement, as per Table 6.2 of CHBDC (2014).

Settlement of the approach embankments, which are considered to extend to 20 m behind the abutments, will occur as a result of the grade raise and embankment widening(s). The design and consideration of the performance of the highway embankment beyond the approaches is beyond the current scope of this work. The total settlement of the approach embankments will be comprised of:

- compression of the existing and new embankment fill(s);
- short-term (immediate) compression of the non-cohesive/granular soils; and,
- long-term consolidation and creep settlement of the firm to very stiff silty clay to clayey silt.

### 6.7.2.2 Settlement Performance Requirements

The settlement performance criterion for design of highway embankment widenings is outlined in MTO's Guideline titled, "Embankment Settlement Criteria for Design", dated July 2010. The allowable total post-construction settlement limits for freeway embankment widenings vary as a function of distance from the bridge abutment. For a King's Highway over a 20 year period following construction, the settlement limits are as outlined below.

Distance from Bridge Abutment (m)	Allowable Settlement (mm)
0 to 20	25
20 to 50	50
50 to 75	100
>75	200

The allowable differential settlement across the widened and existing highway embankment for a non-freeway is 100H:1V.

### 6.7.2.3 Settlement Parameter Selection

The majority of the soils encountered at the site consist of firm to very stiff cohesive soils. The settlement parameters associated with the extensive cohesive deposit encountered at the site are shown as the design lines as presented on Figure 1 and are based on the graphical presentation of this data. The lower cohesive deposit has



been sub-divided into multiple layers based on consideration of all of the in-situ and laboratory test data to develop the design lines for the preconsolidation stress, void ratio and compression index/recompression index.

The immediate compression of the non-cohesive silty sand to sand deposit was modelled by estimating an elastic modulus of deformation based on the SPT “N”-values and using correlations proposed by Bowles (1984), Kulhawy and Mayne (1990), and Peck et al. (1974) as well engineering judgement from experience with similar soils in this region of Ontario. These estimated values were also compared with the typical range of expected values for similar soil types, as outlined in Section C6.9.3.6 of the *Commentary to the CHBDC* (2014) and adjusted, if necessary. The results of the in-situ CPT and SCPT, and triaxial testing carried out were also used to refine the deformation parameters (i.e., modulus of elasticity or Young’s modulus,  $E'$ ).

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation tests, along with the results of the in-situ field vane tests to estimate the stress history and deformation parameters for the cohesive deposits. In addition, the results of the laboratory index tests were employed to further assess deformation parameters (i.e., compression and recompression indices) using empirical correlations proposed in literature by Hough (1957), Bowles (1984), Azzouz et al. (1976), Koppula (1986), Kulhawy and Mayne (1990) and Terzaghi and Peck (1967).

The coefficient of consolidation,  $c_v$  and  $c_h$  ( $\text{cm}^2/\text{s}$ ), required in the time-rate settlement analysis was established using the results of the laboratory consolidation tests (on HTO and VTO specimens), the results of the flexible wall permeameter tests, and the results from the in-situ pore pressure dissipation testing carried out during the CPT investigation in accordance with ASTM D6067. The results from these different testing methods were also checked with the correlation from the U.S. Navy (1986) with liquid limit assuming normally consolidated or over-consolidated soils, as applicable.

In addition to primary consolidation within the cohesive deposits, secondary compression is also expected occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after dissipation of the majority of the excess pore pressures under a constant stress.

A summary of the parameters used for carrying out the settlement analyses for the simplified representative stratigraphy under the approach embankments are summarized below.

Soil Deposits (for simplified subsurface model)	Bulk Unit Weight ( $\text{kN}/\text{m}^3$ )	Elastic Modulus (MPa)	$C_c^1$	$C_r^1$	$\sigma'_p$ (kPa) <sup>1</sup>	$e_o^1$	$m_v^2$ ( $\text{m}^2/\text{kN}$ )	$c_v^3$ ( $\text{cm}^2/\text{sec}$ )
Existing silty sand fill (Elev. 219 to 216 m)	19	50	-	-	-	-	-	-
Upper clayey silt (Elev. 216 to 215 m)	18.75	-	0.2	0.01	187.5	0.7	-	$3.2 \times 10^{-3}$
Silt and sand to sand (Elev. 215 to 210.5 m)	19.5	75	-	-	-	-	-	-
Clayey silt crust I (Elev. 210.5 to 205 m)	20	-	-	-	-	-	$2.2 \times 10^{-5}$	$5.0 \times 10^{-3}$
Lower silty clay to clayey silt – Layer 1	19	-	-	-	-	-	$1.5 \times 10^{-5}$	$5.0 \times 10^{-3}$



Soil Deposits (for simplified subsurface model)	Bulk Unit Weight (kN/m <sup>3</sup> )	Elastic Modulus (MPa)	C <sub>c</sub> <sup>1</sup>	C <sub>r</sub> <sup>1</sup>	σ' <sub>p</sub> (kPa) <sup>1</sup>	e <sub>o</sub> <sup>1</sup>	m <sub>v</sub> <sup>2</sup> (m <sup>2</sup> /kN)	c <sub>v</sub> <sup>3</sup> (cm <sup>2</sup> /sec)
(Elev. 205 to 201 m)								
Lower silty clay to clayey silt – Layer 2 (Elev. 201 to 198.5 m)	19	-	0.35 to 0.2	0.02 to 0.01	188.5 to 296.0	1 to 0.75	-	3.2 x10 <sup>-3</sup>
Clayey silt crust II (Elev. 198.5 to 194 m)	20.5	-	-	-	-	-	3.5 x 10 <sup>-5</sup>	6.2 x10 <sup>-3</sup>
Lower silty clay to clayey silt – Layer 3 (Elev. 194 to 176 m)	19	-	0.4 to 0.15	0.03 to 0.013	341.0 to 413.0	1 to 0.65	-	3.2 x10 <sup>-3</sup>
Lower silty clay to clayey silt – Layer 4	19	100	-	-	-	-	-	-

**Note:**

1. Values of C<sub>c</sub>, C<sub>r</sub>, σ'<sub>p</sub>, and e<sub>o</sub> vary with depth as shown on the Summary Plot of Engineering Parameters for Cohesive Deposits on Figure 1.
2. Values of m<sub>v</sub> used for linear analysis in stiff portions clayey crust(s).
3. Where c<sub>h</sub> required (i.e. for wick drain analysis) a ratio of c<sub>h</sub>/c<sub>v</sub> = 2.7 was used based on assessment of all of the laboratory consolidation (HTO and VTO) and permeameter testing, and in-situ CPT pore pressure dissipation testing.

## 6.7.3 Results of Analyses

### 6.7.3.1 Global Stability

Based on the results of the analysis for deep seated global failure surfaces that penetrate the native foundation soils, the FoS for the new widened portion of the embankments as well as for the portion of the embankment with the grade raise, for side slopes constructed at an inclination of 2H:1V or flatter, for the short-term (undrained) and long-term (drained) cases are greater than 1.33 and 1.54, respectively. An example of the static global stability results is provided on Figures 2 and 3 for the short-term (undrained) and long-term (drained) cases, respectively.

### 6.7.3.2 Settlement

Assuming that all existing peat/organic layer(s) are removed and replaced with granular fill as part of the new construction, the total factored settlements below the widened portion of the new embankments and under the existing embankment due to the proposed grade raise are estimated be as follows:

Settlement Type	Factored Settlements under Widened Portion of New Embankment (mm)	Factored Settlements under Existing Embankment due to Grade Raise (mm)
Compression of the existing/new embankment fill (during construction)	35 - 70	5 - 10
Short-term (immediate) compression of the non- cohesive/granular soils	<10	<5



Settlement Type	Factored Settlements under Widened Portion of New Embankment (mm)	Factored Settlements under Existing Embankment due to Grade Raise (mm)
Long-term primary consolidation settlement of the firm to stiff silty clay to clayey silt stratum	55 - 80	40 - 65

In addition to the settlements above, the long-term, factored secondary (creep) compression settlement of the firm to stiff silty clay to clayey silt stratum is estimated to be 25 mm to 35 mm per log-cycle of time (following completion of the primary consolidation) under both the widened portion of the new embankments and under the existing embankment due to the proposed grade raise.

It is estimated that the time required to complete 90% of the primary consolidation settlement will be approximately 20 years following completion of fill placement. In addition, after the primary consolidation period, secondary (creep) compression will occur, the magnitude of which depends on the period of interest.

#### 6.7.4 Settlement Mitigation Options

Based on the proposed approach and highway embankment geometry and the subsurface conditions at this site, the post-construction settlements (primary and secondary) can be mitigated or reduced to satisfy the MTO's performance requirements as outlined in Section 6.7.2.2, using one or a combination of the following mitigation options:

- **Preloading of the widened approach embankment areas:** It is estimated that the time required to complete 90% of the primary consolidation settlement will be approximately 20 years following completion of fill placement, which is not considered practical from a construction perspective.
- **Preloading of the widening approach embankment areas in conjunction with a surcharge:** Preloading in conjunction with surcharging would accelerate the rate of settlement over the use of preloading alone. It is estimated that in order to reduce the post-construction settlements to within the performance requirements, it would be necessary to construct a surcharge at least 3 m high (over a distance from 0 m to 60 m back from the abutments) on top of the new design embankment widening(s) on the south side(s), and at least 2 m high (over a distance from 0 m to 20 m back from the abutments) on top of the new design embankment widening(s) on the north side(s). The surcharge fills would have to remain in place for approximately 40 months.
- **Use of lightweight fill such as cellular concrete for construction of the grade raise and widened portions of the embankments:** While the use of cellular concrete for the construction of the grade raise and widened embankments would reduce the new loading on the foundation soils and the associated settlements, based on the 50-year flood elevation as provided by AECOM, the maximum cellular concrete thickness feasible at this site is 2.5 m to satisfy an adequate FoS against uplift (buoyancy). If 2.5 m of cellular concrete was utilized in the construction of the new embankment widenings and grade raises, it is estimated that the post construction settlements in the widening areas would still exceed the performance requirements, assuming no other mitigation options were adopted. As such, the use of cellular concrete on its own is not feasible; however, it could be used in combination with one of the other mitigation options.



- **Use of wick drains (in conjunction with preloading and surcharging):** Installation of wick drains, in combination with the construction of a 3 m high surcharge (south side widenings) and a 2 m high surcharge (north side widenings), would shorten the length of preload/surcharge period. Section 6.7.4.4 outlines the specific details of this settlement mitigation such as wick drain depth, wick drain spacing and surcharge dimensions. Following a preload/surcharge period of about 8 months, the total settlement occurring at the widened portions of the new embankments following the 20 year design life would satisfy the performance requirements. However, the settlements estimated to occur within the grade raise portion of the new embankment would not satisfy the performance requirements. In addition, the differential settlements across the new embankment platform (i.e. from widening area to grade raise to widening area) would exceed the differential settlement guideline noted in Section 6.7.2.2. As such, the use of wick drains and surcharging on their own is not feasible; however, it could be used in combination with one of the other mitigation options.
- **Use of wick drains, preloading, surcharging and cellular concrete:** Through an iterative 3D modelling process that included the time dependency of the primary consolidation and secondary compression settlements, a hybrid settlement mitigation option was developed. The design consists of the installation of wick drains in combination with the construction of a 2.5 m high surcharge (south side widenings) and a 2 m high surcharge (north side widenings) with the specific details of the wick drain depths, wick drain spacings and surcharge dimensions included in Section 6.7.4.5. The surcharges would have to remain in place for about 8 months, after which time a 1.5 m thickness of cellular concrete would be installed within the central grade raise section of the existing highway embankment. With this combination of the mitigation measures, the total and differential settlement tolerances outlined in Section 6.7.2.2 can be satisfied.
- **Do Nothing (no mitigation; long-term maintenance required):** The allowable total post-construction settlement for freeway embankment approach widenings is 25 mm over a 20-year period following completion of construction (per MTO's embankment settlement criteria and Section 6.7.2.2). However, if the approach embankments and approach slabs to the new bridge structure could tolerate up to about 80 mm and 65 mm of post-construction settlement in the widening and grade raise areas, respectively, and if MTO is willing to accept the additional maintenance associated with these settlements, then constructing the new approach embankments with no settlement mitigation measures could be considered.

If the 'Do Nothing' option is not acceptable to MTO and the magnitudes of post-construction settlement (and associated maintenance costs) cannot be tolerated, then based on the options outlined above, and discussed in more detail below, the preferred option to mitigate the post-construction settlements of the new approach embankments is a combination of wick drains, preloading, surcharging, and lightweight (cellular concrete) fill.

A comparison of the alternative settlement mitigation options based on advantages, disadvantages, risks and relative costs is provided in Table 2 following the text of this report. Each of these mitigation options is discussed in more detail in the following sections.

#### 6.7.4.1 Preloading

Preloading could be considered for reducing post-construction settlements of the subsoils under the proposed embankment widenings. Preloading refers to the placement of fill up to the proposed profile grade of the highway, in one stage, in advance of the embankment completion and final pavement construction, in order to consolidate the underlying compressible soils. Preloading reduces the magnitude of long-term, post-construction settlements by promoting such settlements to occur under the fill loads, during the construction period, and in advance of final grading of the embankment and completion of the pavement structure.



Given the thick nature of the clayey strata at this site, it is estimated that the time required to complete 90% of the primary consolidation settlement will be approximately 20 years following completion of fill placement. Following the completion of majority of the primary consolidation, it is estimated that additional secondary compression settlement would occur over the following 20-year period to the assumed design life for the roadway equal to the values shown in the table below.

Distance from Bridge Abutment (m)	Estimated Settlement in Widened Portion (mm)	Estimated Settlement in Grade Raise Portion (mm)
0 to 20	15	15
20 to 50	15	15
>50	15	15

Based on the estimated length of time required to complete 90% of the primary consolidation settlement, preloading alone is not considered a practical or feasible settlement mitigation option for this site.

#### 6.7.4.2 Surcharging

Similar to preloading, surcharging refers to the placement of embankment fill in advance of final pavement construction to reduce long-term, post-construction settlements. The difference between preloading and surcharging is the amount of fill placed and the time required for consolidation to be achieved. With surcharging, the fill is placed to the full embankment design height (i.e., preloading), followed by an additional lift of fill (the surcharge) above that required to construct the final embankment geometry. The additional lift of fill applies greater stress to the underlying cohesive deposits and reduces the time for consolidation over that achieved by preloading alone, resulting in “overconsolidation” of the underlying compressible foundation soils relative to the final design embankment height. At the end of the surcharge period, the portion of the surcharge fill remaining above the required embankment height is removed (and could be re-used, if possible, in other areas of the project). Surcharging is considered a viable option at this site as the construction schedule will not allow for sufficient time for the consolidation settlements to occur under preloading only.

It is estimated that it would be necessary to construct a 3 m high surcharge (for the south side widenings) and a 2 m high surcharge (for the north side widenings) and that the surcharge fills would have to remain in place for about 60 months (subject to confirmation by monitoring) in order to satisfy the MTO’s embankment settlement performance criteria. The estimated remaining long-term, post-construction settlement of the final embankment (at 20 years after completion of construction) are summarized below.

Distance from Bridge Abutment (m)	Estimated Settlement in Widened Portion (mm)	Estimated Settlement in Grade Raise Portion (mm)	MTO’s Settlement Criteria (July, 2010) (mm)
0 to 20	<25	<20	<25
20 to 50	<35	<25	<50



Distance from Bridge Abutment (m)	Estimated Settlement in Widened Portion (mm)	Estimated Settlement in Grade Raise Portion (mm)	MTO's Settlement Criteria (July, 2010) (mm)
>50	<45	<40	<100

The table below provides details of the surcharge dimensions that would be required to achieve the post-construction settlements indicated above.

Surcharge Layout <sup>1</sup>	Distance from Abutments (m)	
	0 to 20	20 to 60
	Surcharge Dimensions	Surcharge Dimensions
North Widening	2 m high 11 m wide	N/A
South Widening	3 m high 17 m wide	3 m high 16 m wide

Note(s): the height of surcharge is measured relative to the existing road grade.

The slope of the surcharge fill facing the existing Highway 9 shall be constructed at 1H:1V profile and the slope that faces away from the existing Highway 9 shall be constructed at a profile of 2H:1V. All surcharge slopes that face the proposed abutment locations shall be constructed at a profile of 1H:1V with the toe of the surcharge fill extending to the location of the proposed new bridge abutments.

Slope stability analyses were carried out to assess the FoS of the proposed new embankment widenings under the loading from the 3 m high surcharge discussed above assuming 1H:1V side slopes (for the slope facing the existing Highway 9 and the slope at the abutment) and a 2H:1V side slope (for the slope facing away from existing Highway 9). As shown on Figures 4 and 5, the minimum target FoS of 1.16 (for the short-term / temporary condition for global slip surfaces that extend into the foundation soils) is satisfied assuming a friction angle ( $\phi'$ ) of 35 degrees for the granular surcharge fill material. It is noted that the calculated FoS will be greater than 1.3 if a friction angle of 45 degrees is assumed for the granular surcharge fill material. Shallow, surficial failures within the granular surcharge fill may occur where the fill slopes at 1H:1V (depending on precipitation/snow melt), however it is anticipated that these types of surficial sloughs could be addressed by periodic maintenance during the temporary/surcharge period.

#### 6.7.4.3 Lightweight Fill

Lightweight fill, such as cellular concrete could be used for construction of a portion of the embankment widening(s) and grade raises, to reduce the new loading imposed on the underlying compressible foundation soils which in turn would reduce the magnitude of the post-construction settlements.

It is estimated that it would be necessary to construct a large portion of the new embankment fills (in both the widening and grade raise areas) with cellular concrete in order to reduce the long-term, post-construction settlement of the final embankment (at 20 years after completion of construction) and satisfy the MTO's settlement performance



criteria. However, based on a 50-year flood elevation of 220.28 m (as provided by AECOM), and an assumed 1 m thickness of required pavement structure on top of the lightweight fill, for a FoS of 1.5 for against uplift (buoyancy), the maximum cellular concrete thickness feasible at this site is 2.5 m (assuming the base of the cellular fill is located at or above Elevation 217.5 m).

If 2.5 m of cellular concrete is utilized in the construction of the new embankment widening and grade raise area areas at the site, it is estimated that the post construction settlements (at 20 years post-construction) would be reduced to the values shown in the table below.

Distance from Bridge Abutment (m)	Estimated Settlement in Widened Portion (mm)	Estimated Settlement in Grade Raise Portion (mm)	MTO's Settlement Criteria (July, 2010) (mm)
0 to 20	<35	<20	<25
20 to 50	<65	<50	<50
>50	<70	<55	<100

Implementing cellular concrete as described above would reduce the estimated total and differential settlements in the grade raise areas to the tolerances outlined in Section 6.7.2.2; however, the total settlements in the widening area(s) from 0 m to 20 m behind the abutments would not be satisfied.

If cellular concrete is used as a settlement mitigation at this site, it is recommend that the cellular concrete be specified to have an unconfined compressive strength of 1 MPa at 28 days and a unit weight of 5 kN/m<sup>3</sup>. An NSSP titled Cellular Concrete is included in Appendix F which addresses the requirements for cellular concrete at the site.

#### 6.7.4.4 Wick Drains and Surcharges

Where the time required to achieve an adequate degree of consolidation is considered too long, consideration may be given to installing wick drains in conjunction with preloading and/or surcharging to accelerate the rate of primary consolidation. Wick drains are prefabricated geotextile drains installed vertically from ground surface into or through soft to firm, compressible soils to increase the rate of excess porewater pressure dissipation. Typically, wick drains are installed on a 1 m to 3 m equilateral triangular grid spacing over the footprint of the embankment widening. Pre-drilling can be required in some instances due to the presence of very stiff to hard clayey crust layers or due to the presence of dense silt and sand layers. Wick drains should be installed in accordance with OPSS 220 which includes the requirements for pre-augering, if necessary.

It is estimated that it would be necessary to install wick drains at spacings of 2 m to 2.5 m and to depths ranging from about 25 m to 30 m, in combination with the surcharges described in Section 6.7.4.2 and left in place for about 8 months, (subject to confirmation by monitoring) in order to satisfy the MTO's embankment settlement performance criteria. The estimated remaining long-term, post-construction settlement of the final embankment (at 20 years after completion of construction) are summarized below.



Distance from Bridge Abutment (m)	Estimated Settlement in Widened Portion (mm)	Estimated Settlement in Grade Raise Portion (mm)	MTO's Settlement Criteria (July, 2010) (mm)
0 to 20	<20	<35	<25
20 to 50	<35	<35	<50
>50	<60	<50	<100

The table below provides details of the wick drain layout dimensions and depths that would be required to achieve the post-construction settlements indicated above.

Wick Drain Spacing <sup>2</sup> Dimensions and Installation Depths						
Location	Distance from Abutments (m)					
	0 to 25 <sup>1</sup>		25 to 45		45 to 55	
	Spacing (m)	Depth (m)	Spacing (m)	Depth (m)	Spacing (m)	Depth (m)
North Widening <sup>1</sup>	2.5	45	N/A	N/A	N/A	N/A
South Widening	2	45	2.5	30	2.5	25

Note(s): 1. The wick drains installed at the northern widening will only extend from 0 m to 20 m from the abutment(s).

2. The wick drain spacing pattern is to be an equilateral triangle.

The installation of wick drains and placement of surcharge fill would not be practical in the grade raise portion of the roadway at this site and would potentially require an additional stage of construction thereby significantly lengthening the overall schedule. Since the estimated total settlements in the grade raise portion for this settlement mitigation option (as summarized above) do not satisfy the performance limits (at 20 years after completion of construction) as outlined in Section 6.7.2.2, some additional mitigation measure is required. In addition, implementation of this settlement mitigation will yield differential settlements at the site to a maximum of approximately 30 mm across the new embankment section, corresponding to a rate of 1:7.5 which exceed the differential settlement guidelines outlined in Section 6.7.2.2.

#### 6.7.4.5 Wick Drains, Surcharges and Cellular Concrete

To further reduce the post construction settlement in the grade raise area to satisfy the MTO's performance criteria, consideration should be given to installing cellular concrete within this section of the embankment in conjunction with the installation of wick drains and surcharging in the widening areas.

In order to assess the most effective combination of these three settlement mitigation techniques, a three-dimensional, iterative design process was undertaken using the commercially available software Settle3D (Version 4.0) by Rocscience Inc. Consideration was given to the following variables to arrive at an efficient



settlement mitigation strategy that satisfies the MTO's criteria for total and differential settlements, while also considering the surcharge time and installation costs:

- Wick drain spacing and wick drain length;
- Surcharge height, footprint area in plan, and side slopes geometry; and,
- Cellular concrete thickness and footprint area in plan.

Multiple scenarios were analysed and a comparison of the predicted settlements (with distance from the abutment in the grade raise and widening areas, and across the embankment section) associated with a select number of potential settlement mitigation combinations is shown in Figures 6 to 8. The details of the settlement mitigation design which is believed to provide the most favorable combination of differential settlements, total settlements, surcharging time, and installation cost, and is recommended for the site, is described in the following tables.

Recommended Wick Drain Spacing <sup>2</sup> Dimensions and Installation Depths						
Location	Distance from Abutments (m)					
	0 to 25 <sup>1</sup>		25 to 45		45 to 55	
	Spacing (m)	Depth (m)	Spacing (m)	Depth (m)	Spacing (m)	Depth (m)
North Widening <sup>1</sup>	2.5	30	N/A	N/A	N/A	N/A
South Widening	2	30	2.5	30	2.5	25

- Note(s):
1. The wick drains installed at the northern widening will only extend from 0 m to 20 m from the abutment(s).
  2. The wick drain spacing pattern is to be an equilateral triangle.
  3. Wick drain installation footprint to extend to at least the limits of the surcharge footprint; wick drains should be installed 2 rows beyond the surcharge footprint where possible.

Recommended Surcharge Layout <sup>1</sup>	Distance from Abutments (m)	
	0 to 20	20 to 60
	Surcharge Dimensions <sup>1,2</sup>	Surcharge Dimensions <sup>1,2</sup>
North Widening	2 m high <sup>1</sup> 11 m wide	N/A
South Widening	2.5 m high <sup>1</sup> 16 m wide	2.5 m high <sup>1</sup> 15 m wide

- Note(s):
1. The height of the surcharge is measured relative to the existing road grade.
  2. Surcharge side slopes facing existing Highway 9 and facing towards bridge abutments to be constructed at 1H:1V. Side slopes facing away from existing Highway 9 to be constructed at 2H:1V.



Recommended Cellular Concrete Layout	Details
Grade Raise Area	1.5 m thickness, 12 m width of Cellular Concrete from 0 m to 25 m from the abutment(s). The limits of the cellular concrete in the north-south direction should be approximately +/- 6 m from the centreline of the existing Highway 9.
South Side Widening Area	"Wedge" of Cellular Concrete defined by the abutment wall, the front slope of the surcharge embankment (at 1H:1V) and the bottom of the pavement structure. The approximate dimensions of the upside down, right angled, triangular wedge of Cellular Concrete are 3 m length (E/W direction), 10 m width (N/S direction), and 1.5 m height.

Note(s): An NSSP outlining the requirements for Cellular Concrete placement is included in Appendix F and should be included in the Contract Package.

As noted in Section 6.7.4.2, slope stability analyses showed an adequate FoS for the proposed new embankment widenings under a 3 m high surcharge fill. As such, the FoS for the 2.5 m high surcharge fills recommended for the preferred settlement mitigation scheme described above will also satisfy the minimum requirements. Shallow, surficial failures within the granular surcharge fill may occur where the fill slopes at 1H:1V (depending on precipitation/snow melt), however it is anticipated that these types of surficial sloughs could be addressed by periodic maintenance during the temporary/surcharge period.

Following the installation of the wick drains, and construction of the surcharges (and surcharging for about 8 months, subject to confirmation by monitoring), and then placement of cellular concrete in the grade raise area, the estimated long-term, post-construction settlement of the final embankment (at 20 years after completion of construction) is summarized below.

Distance from Bridge Abutment (m)	Estimated Settlement in Widened Portion (mm)	Estimated Settlement in Grade Raise Portion (mm)	MTO's Settlement Criteria (July, 2010) (mm)
0 to 20	<20	<25	<25
20 to 50	<35	<40	<50
>50	<65	<50	<100

The estimated total settlements within the grade raise and widening portions of final embankment (at 20 years after completion of construction) will satisfy the settlement tolerances outlined in Section 6.7.2.2 for this mitigation strategy. In addition, the differential settlement tolerances will also be satisfied in the east-west direction, along the profile parallel to the centreline of the roadway. However, differential settlements of approximately 20 mm across the new embankment section (in the north-south direction) which corresponds to a rate of 10:1 are predicted to occur which exceeds the MTO's performance guidelines. Further reduction of this differential



settlement would require extreme mitigation measures (i.e. wick draining and surcharging within the grade raise portion/along existing highway) and add cost and time to the construction schedule. It is possible that the actual differential settlements may not be as severe as predicted by the model, and it is anticipated that the regularly scheduled maintenance to the asphalt (i.e. milling and grinding) over the 20 years post-construction can even out much of this differential settlement if it occurs. As such, the combination of mitigation measures outlined above is recommended for the Highway 9 reconstruction.

It is noted that settlements of up to about 25 mm are predicted to occur within the existing roadway at the existing bridge abutments during the surcharging period as a result of the loading from the adjacent surcharge fills. However, larger settlements of up to 35 mm (north side) and 50 mm (south side) are predicted to occur immediately adjacent to sides of the existing bridge abutments at the limits of the embankment widening/wick drain areas. These settlements may affect the performance of the existing bridge, as well as the transition along the existing roadway approach to the existing bridge. As such, it is recommended that the existing bridge be instrumented (for settlement and tilt) and monitored during the Stage 1 construction (i.e. during the surcharge period and pile driving ) to check the safety/stability of the existing bridge while it continues to be used to carry traffic along Highway 9.

## 6.8 Enbridge Gas Utility

It is our understanding that a 100 mm diameter Enbridge gas pipeline is located parallel to the southern toe of the existing Highway 9 roadway embankment(s). The gas line is located about 15 m south of the guardrail on the west side of the bridge, running eastward until it crosses Highway 9 from south to north about 42 m east of the east abutment. It is understood that depending on the anticipated settlements due to the new construction, the gas line may need to be relocated prior to the start of construction.

Based on the settlement analysis, the maximum factored total settlement along the Enbridge gas utility is estimated to be up to about 135 mm as a result of the embankment widening and surcharge construction required as part of the settlement mitigation strategy for the site.

All of the settlement mitigation options outlined in Section 6.7.4 will result in settlement occurring along the existing gas pipeline. Although the lightweight fill option would result in the least amount of new settlement on the pipeline, as discussed in Section 6.7.4.3, due to the high-water level, the maximum cellular concrete thickness feasible at the site is 2.5 m and this thickness is not sufficient to reduce the settlement in the embankment widening areas to a level that would satisfy the MTO's Embankment Settlement Criteria Guideline. As such, the use of lightweight fill to reducing the loading on, and settlement of, the existing gas pipeline is not feasible. Further, the settlement mitigation strategy recommended for the site requires the installation of wick drains in the embankment widening areas; the presence of the existing gas pipeline will interfere with the installation of the wick drains.

Based on the above considerations, after discussions between AECOM, Golder, T2 Utility Engineers (T2), MTO and Enbridge, it was decided that the magnitude of predicted settlement is unacceptable and that the gas pipeline will be relocated to the north side of the Highway 9 embankment and beyond the footprint of the wick drain areas.

Analysis has been carried out to estimate the settlements along the proposed new alignment of the Enbridge gas line on the north side of Highway 9. The details of the proposed new alignment of the gas line were provided to Golder by T2 on January 7, 2020. The proposed location of the gas line re-alignment runs slightly askew of parallel to Highway 9 and ranges from approximately 2.5 m north of the north embankment toe west of the bridge structure to approximately 11.5 m north of the north embankment east of the bridge structure. The proposed gas line location does not intercept the footprint of the proposed northerly widenings, wick drains, or surcharges however the gas



line may intercept the footprint of the backslope of the organic subexcavation. It is recommended that the gas line be installed below the deepest portion of the organics at the site to avoid conflicts that might otherwise occur during the excavation and construction.

Based on the results of the settlement analysis, which are shown on Figure 9 the anticipated post-construction settlements along the new gas line alignment are summarized below:

Location	Approx. Range of Settlements (mm)	Approx. Deflection Ratio (Settlement/Length)
Westerly of surcharge on west side of canal	20 to 30	1/1750 to 1/5250
Adjacent to surcharge on west side of canal	30 to 40	
Between surcharges	25 to 30	
Adjacent to surcharge on east side of canal	25 to 35	

Enbridge indicated to AECOM, T2 and Golder on February 12, 2020 that the above settlement magnitudes and deflection rates were acceptable and tolerable for the re-aligned pipeline. It is understood that the pipeline will be relocated in early 2020 prior to any of the new construction associated bridge structure replacement.

## 6.9 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete foundations and reinforcement steel and other concrete or steel elements buried in the soil. The long-term performance and durability of the foundations are directly related to their respective corrosion resistance. Generally, the corrosivity potential to a structure depends on the soil resistivity / electrical conductivity, hydrogen ion concentration, and salts (chloride and sulphate) concentrations. The analytical results for the samples submitted for testing are summarized in Section 4.4 and the analytical laboratory test reports are included in Appendix D.

### 6.9.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 ("*Additional requirements for concrete subjected to sulphate attack*") for potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, based on the two samples of soil tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

### 6.9.2 Potential for Corrosion

The test results indicate a pH of 8.0 and 8.2 and a resistivity of 230 ohm-cm and 670 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not considered detrimental to concrete durability. However, the resistivity indicates that the soil corrosiveness is Severe (2,000 ohm-cm > R), as per Table 3.2 of the Gravity



Pipe Design Guidelines (MTO, 2014), and corrosion protection should be applied to the foundation element / materials. Further, given that the foundations are located adjacent to the roadway shoulder and will be exposed to de-icing salt, consideration should be given to selection of a “C” type exposure class as defined by CSA A23.1 Table 1.

It is ultimately up to the structural designer to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

## 6.10 Construction Considerations

### 6.10.1 Subgrade Preparation and Embankment Construction

Prior to construction of the new approach embankments, it is recommended that all topsoil, peat/organic soil, and existing surficial fill materials be stripped from the footprint of the embankment footprint and be replaced with OPSS Granular B Type II. Based on the borehole results at the abutments and approach embankment areas, subexcavation of peat/organic soil will be required down to about Elevation 215.0 m within the footprint of the approach embankments. A Notice to Contractor titled Restriction on Construction Operations - Excavation of Organic Materials is included in Appendix F which addresses the organic excavation requirements and specifies the maximum open excavation width allowable during the subexcavation.

Fill for construction of the new embankments should consist of OPSS.PROV 1010 (*Aggregates*) granular materials (i.e. Granular A or Granular B Type II). The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*) and OPSS.PROV 206 (*Grading*). Permanent embankment side slopes should be constructed no steeper than 2 horizontal to 1 vertical (2H:1V) in granular fill.

For the temporary surcharge fills, a granular fill material such as OPSS Granular B Type II will be required to satisfy the target factor of safety associated with the global stability for a 1H:1V temporary side slope, as discussed in Section 6.7.4.2.

In accordance with MTO’s standard practice, a minimum 2 m wide bench should be provided where embankment slopes are greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m, consistent with OPSD 202.010 (*Slope Flattening*).

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (*Topsoil*) and OPSS.PROV 804 (*Seed and Cover*) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS 511 (*Rip Rap, Rock Protection and Granular Sheeting*), and OPSS.PROV 1004 (*Aggregates – Miscellaneous*) will be required to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

### 6.10.2 Excavation and Control of Groundwater and Surface Water

During the removal of the topsoil, peat/organic soil, and existing surficial fills, excavations extending down to Elevation 215.0 m (i.e., to a depth of up to about 6 m) will be required within the footprint of the widened approach embankments.

In addition, due to the proximity of the new piles to the existing piles, and in order to reduce the risk of conflicts during the new pile installation, excavations will be required to expose the existing pile caps to depths of up to about 4 m to 5 m below the present Highway 9 road surface, through the existing approach embankment fill.



Open-cut excavations must be carried out in accordance with the guidelines outlined in the most recent version of the Occupational Health and Safety Act and Regulation for Construction Activities (Ontario Regulation 213). The peat material and soft portions of the clayey silt deposit are classified as Type 4 soils, the existing fill materials, silt and sand to sand deposit and the firm to stiff portions of the clayey silt and silty clay to clayey silt deposits are classified as Type 3 soils, according to the OHSA. Temporary excavations (i.e., those that are open for a relatively short time period) within Type 4 soils should be made with side slopes no steeper than 3H:1V, while those within Type 3 soils should be made with side slopes no steeper than 1H:1V.

It is expected that for construction staging, temporary protection systems will be required along Highway 9 to facilitate the staged preloading and surcharging, and removal of the existing bridge structure. Recommendations for temporary protection systems are provided in Section 6.10.3.

Excavations for the abutment foundations and the new approach embankments will be below the groundwater level, which has been interpreted to be at approximately Elevation 217.7 m, with the potential for seasonally higher water levels. Excavation of the organic soils to approximately Elevation 215.0 m will require excavation below the groundwater table. A NSSP titled Restriction on Construction Operations - Excavation of Organic Materials is included in Appendix F which specifies that the maximum length of open excavation should be a maximum of 3 m in any direction at any given time. All organic material removed below the groundwater table should be replaced with Granular B Type II (OPSS 1010). Excavation slopes for subexcavation of organic material should be carried out in accordance with OPSS 203. The Granular B Type II backfill material should be placed up to 0.3 m above the groundwater table.

Dewatering will be required to maintain the excavations for the abutment foundations dry during construction; is it recommended that the water level be lowered to below the base of the proposed CSPs if included as part of the design. Dewatering should be carried out in accordance with OPSS.PROV 517 (*Dewatering*) as referenced in OPSS 902 (*Excavating and Backfilling Structures*) as amended by SP FOUN0003; a copy of which is provided in Appendix F. It is noted that the designer will need to fill in the data for “return period” in this SP.

The existing standpipe piezometers (installed in Boreholes AP-1, AP-2 and MW-1) should be maintained operational to allow for continued monitoring of the groundwater level at the site up to the construction, at which time the piezometers should be decommissioned by the Contractor in accordance with Ontario Regulation 903 (as amended).

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations.

### 6.10.3 Temporary Protection Systems

Temporary protection systems will be required to facilitate the construction of the widened approach embankments and new bridge foundations. Based on the General Arrangement drawing, temporary protection systems are proposed between the south side of the existing Highway 9 and the proposed widened approach embankments, as well as in front of the proposed abutments and at the toe of the new approach embankments.

The temporary protection systems shall be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the temporary protections systems along Highway 9 shall meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent structures or utilities can tolerate this magnitude of deformation.



Consideration could be given to either partial or full removal of the protection system(s) upon completion of construction or each stage of construction (as required). Where possible, full removal of the protection system should be considered to mitigate potential impediments to future rehabilitation/reconstruction work at the underpass sites, or to the road structure above. An NSSP titled Temporary Protection Systems (which amends OPSS 539) is included in Appendix F to address the removal or cut-off of the protection system.

Although the selection of the protection systems will be the responsibility of the contractor, it is considered that shoring could consist of either driven steel sheet-piling or soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds. Although groundwater seepage is anticipated to be minor through the cohesive deposits, it may be more extensive from the non-cohesive deposits; it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards. The sheet piles or soldier piles would have to be driven or socketted to sufficient depth to provide the necessary passive resistance for the retained soil height, including any surcharge loads placed behind the protection system within at least a 1H:1V (and up to 1.5H:1V) zone relative to the base of the excavation. Lateral support to the sheet piles or soldier piles could be provided in the form of struts, rakers or temporary anchors.

Although the design of the temporary protection system is the responsibility of the Contractor, the following parameters are provided to enable the structural designer to develop a conceptual design and assess the approximate construction costs for the protection system(s) at this site:

Soil Type	Unit Weight ( $\gamma$ , kN/m <sup>3</sup> )	Internal Angle of Friction ( $\phi$ , degrees)	Undrained Shear Strength ( $s_u$ , kPa)	Coefficient of Earth Pressure <sup>(2)</sup>		
				Active, $K_a$	At Rest, $K_o$	Passive, $K_p$ <sup>(3)</sup>
New embankment fill (compacted OPSS Granular A or Granular B Type II)	21	35	-	0.27	0.43	3.69
Existing silt and sand to silty sand embankment fill	20	28	-	0.36	0.53	2.77
Upper clayey silt	20	32	40	0.31	0.47	3.25
Silt and sand to sand	19.5	35	-	0.27	0.43	3.69
Clayey silt crust	20	32	66 to 44 <sup>4</sup>	0.31	0.47	3.25
Lower silty clay to clayey silt - Layer 1	20	20	44 to 41.5 <sup>4</sup>	0.49	0.66	2.04
Lower silty clay to clayey silt – Layer 2	20	20	41.5 to 90.8 <sup>4</sup>	0.49	0.66	2.04
Lower silty clay to clayey silt – Layer 3	20	35	-	0.27	0.43	3.69



- Note(s):
1. The temporary shoring design should be assessed for both the drained ( $\phi'$ ) and undrained ( $s_u$ ) cases and the design should be based on the more conservative earth pressure conditions.
  2. The lateral earth pressure coefficients presented above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are expected, the coefficients should be corrected accordingly.
  3. The total passive resistance below the base of the excavation adjacent to the temporary protection system may be calculated based on the values of  $K_p$  indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.
  4. Undrained shear strength decreases / increases linearly from the top to the bottom of the deposit as shown on Figure 1.

## 6.10.4 Instrumentation and Monitoring

### 6.10.4.1 Embankment Settlement Monitoring

As discussed in Sections 6.7.4, wick drains plus preloading and surcharging of the widened approach embankments is recommended as the preferred settlement mitigation strategy for the site. For these mitigation options it is recommended that settlement monitoring and measurement of the dissipation of pore water pressures in the lower cohesive deposit be carried out as the approach embankments are constructed. The proposed embankment monitoring instrumentation program is outlined in detail in the NSSP titled Supply and Installation of Embankment Monitoring Equipment in Appendix F. The embankment monitoring instrumentation comprises the following:

- A total of 10 multi-level vibrating wire piezometers each with four VWP tips.
- A total of 22 settlement plates (SPs) installed at the base of the embankment fill and/or top of the wick drain drainage blanket; and,
- A total of 16 settlement points (Ss) installed at the crest(s) of the surcharge embankment and preload areas.

The location and details of the instrumentation monitoring equipment are shown on Drawings 5 to 11. An instrumentation monitoring procedure that addresses the recommended frequency of monitoring as well as Review and Alert levels for the instruments can be prepared if required, but is beyond the scope of the current assignment.

### 6.10.4.2 Downdrag Forces Acting on Production Piles

As noted in Section 6.2, the site has been identified as a well-suited location to install instrumentation to monitor and verify the actual downdrag forces (i.e. drag loads) acting on the new H-piles located adjacent to the widened portion of the embankment following construction of the new bridge. The southerly widening of the east abutment has been selected for H-pile instrumentation as the downdrag forces are predicted to be largest in this area of the bridge foundation plus the risk of the new H-pile coming in contact with an existing timber pile during installation of the instrumented pile is reduced. The proposed pile instrumentation program is outlined in detail in the NSSP titled Supply and Installation of Downdrag Monitoring Equipment in Appendix F. The downdrag monitoring instrumentation comprises the following:

- 1 multi-level vibrating wire piezometer with six (6) VWP tips.
- 1 multi-point borehole extensometer with nine (9) monitoring points (anchors).
- 1 multi-point pile extensometer with four (4) monitoring points.
- A total of thirty-eight (38) strain gauges installed on the instrumented production H-pile.



The location and details of the downdrag monitoring equipment are shown on Drawings 12 and 13. An instrumentation monitoring procedure can be prepared if required, but is beyond the scope of the current assignment.

#### **6.10.4.3 Post Construction Monitoring of Differential Settlement Across Embankment**

Through discussions between MTO and Golder, it has been recommended that long-term monitoring of total and differential settlements across the completed cross-section of the east approach embankment be carried out. In this regard, it is proposed that a Shape Acceleration Array (SAA) affixed to a Temporary Benchmark (TBM) be used to measure the total and differential settlements across the embankment. The SAA will consist of a series of MEMS accelerometers (gravity sensors) spaced at 500 mm over a length of approximately 39 m. The SAA will be oriented perpendicular to the Highway 9 centerline and offset approximately 6 m east of the east abutment near the end of the approach slab. The SAA is to be installed near the bottom of the new embankments and below the cellular concrete. The details of the SAA instrumentation are as described in the NSSP titled Supply and Installation of Shape Accel Array in Appendix F. The location and details of the SAA instrumentation are shown on Drawings 14 and 15.

An instrumentation monitoring procedure can be prepared if required, but is beyond the scope of the current assignment.

#### **6.10.5 Vibration Monitoring During Construction**

Vibration monitoring should be carried out during pile driving, during installation of protection system(s), and during operation of vibratory compaction equipment to check that the vibration levels at nearby residential/commercial/agricultural structures and on utilities are maintained below tolerable levels.

A maximum peak particle velocity (PPV) of 50 mm/s is generally considered applicable for bridge structures in good condition. In this regard, it is recommended that vibration monitoring be carried out on the existing and new bridge structures during the different stages of construction.

Residential and agricultural buildings are located as close as about 65 m from the proposed abutment locations. A PPV threshold of 25 mm/s is generally considered applicable for vibration impacts on private structures and wells.

For utilities in the vicinity of the site (including the gas pipeline), a PPV threshold of 10 mm/s is recommended.

Pre- and post-construction condition surveys and vibration monitoring are recommended at and near structures located within a 250 m radius of the piling operations (including the gas pipeline, if applicable), and it would be prudent to carry out such monitoring during critical stages of the construction, such as during pile driving operations and during installation of temporary shoring. A NSSP titled Vibration Monitoring, which describes the requirements for vibration monitoring is presented in Appendix F.



## 7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Carter Comish and Ms. Nikol Kochmanová, P.Eng. and reviewed by Mr. Kevin Bentley, P.Eng. Mr. Paul Dittrich, P.Eng., a Principal with Golder and MTO Foundations Designated Contact, conducted an independent technical and quality control review of this report.

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### ASTM International

- |            |   |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.   |
| ASTM D1587 | Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.  |
| ASTM D2573 | Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.   |
| ASTM D6067 | Standard Practice for Using the Electronic Piezocone Penetrometer Tests for Environmental Site Characterization and Estimation of Hydraulic Conductivity. |



**Canadian Standards Association (CSA):**

CAN/CSA A23.1-14 Concrete Materials and Methods of Concrete Construction

**Commercial Software:**

LPILE Plus (Version 5.0)

Settle3D (Version 4.0) by Rocscience Inc.

Slide (Version 2018) by Rocscience Inc.

**Ontario Provisional Standard Drawing:**

OPSD 202.010 Slope Flattening

OPSD 3090.101 Foundation Frost Penetration Depths for Southern Ontario

**Ontario Provincial Standard Specifications (OPSS)**

OPSS.PROV 206 Construction Specification for Grading

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 511 Construction Specification for Rip-Rap, Rock Protection and Granular Sheet piling

OPSS.PROV 517 Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation

OPSS.PROV 539 Construction Specification for Temporary Protection Systems

OPSS 802 Construction Specification for Topsoil

OPSS.PROV 804 Construction Specification for Seed and Cover

OPSS.PROV 902 Construction Specification for Excavating and Backfilling Structures

OPSS.PROV 903 Construction Specification for Deep Foundations

OPSS.PROV 1004 Material Specification for Aggregates – Miscellaneous

OPSS.PROV 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

**Special Provisions**

SP FOUN0003 Dewatering Structure Excavations

SP109F57 Amendment to OPSS 903, April 2016

SP903506 High-strain Dynamic Testing, Deep Foundations, October 2017

**Ontario Water Resources Act**

Ontario Regulation 903/90 Wells: O. Reg. 468/10 Amendment to Ontario Regulation 903

**Ontario Occupational Health and Safety Act**

Ontario Regulation 213 (Construction Projects)

**Ministry of Transportation, Ontario**

*Gravity Pipe Design Guidelines*, Ministry of Transportation Circular Culverts and Storm Sewers, April 2014.



*Structural Manual*, Provincial Highways Management Division, Highway Standards Branch, Bridge Office, August 2016.

Structural Drawing SS103-11 (2018). Pile Driving Control

MTO Foundations Guideline, Embankment Settlement Criteria for Design, July 2010.



TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES – HIGHWAY 9 – HOLLAND DRAINAGE CANAL BRIDGE

Foundation Option	Feasibility	Advantages	Disadvantages	Risk / Consequences	Estimated Costs
Spread/strip footings founded on native soils	Although the abutments could be founded on the silty sand to sand deposit at about Elevation 213 m to 215 m, the spread / strip footings would have to be relatively large to provide a suitable geotechnical resistance. In addition, excessive settlements are anticipated due to the underlying extensive silty clay to clayey silt deposit. As such, spread / strip footings are not considered feasible at this site.	Conventional excavation and construction techniques.	<ul style="list-style-type: none"><li>Excavations of up to 8 m required to find a suitable founding stratum.</li><li>Likely very large footings required given the low geotechnical resistances available.</li><li>Would require sub-excavation of weak soil deposits below high water table adjacent to existing roadway.</li><li>Not feasible to mitigate long-term settlements to acceptable levels, i.e., pre-loading not practical.</li><li>Requirement for extensive dewatering measures during construction.</li><li>Extensive temporary protection systems will be required along Highway 9 lanes to facilitate excavation to founding elevation.</li><li>Precludes use of integral abutments; potentially greater maintenance required at abutments.</li></ul>	<ul style="list-style-type: none"><li>Low geotechnical resistances to satisfy SLS.</li><li>Higher total and differential settlement compared to foundation elements founded on deep foundations.</li><li>Footings must be constructed after the preferred settlement mitigation measure has been implemented to reduce long-term, post-construction settlements.</li></ul>	<ul style="list-style-type: none"><li>Lower relative cost than deep foundations; however, would require additional costs associated with temporary protection systems and dewatering measures for deep excavations.</li></ul>
Steel H-piles – 30 m to 50 m long friction piles with new pile cap(s) located in front of existing abutment(s)  HP310x110 or HP310x132 or HP360x132	Feasible for support of the abutments but not recommended from a foundations perspective due to the high risk that the new piles will come in contact with the existing piles during installation/driving.	<ul style="list-style-type: none"><li>Abutment pile caps located at a higher elevation (i.e., within the approach embankment fill) as compared with shallow foundation option founded on native sand soils thereby reducing excavation depth and associated protection system requirements; however, excavation of the peat layer would still be required for the embankment widening(s).</li><li>High geotechnical resistances can be obtained (depending on pile size and length).</li><li>Allows for integral abutment construction, assuming that the pile capacities are sufficient to allow for one row of piles.</li></ul>	<ul style="list-style-type: none"><li>New piles may come in contact with the existing piles during installation/driving; such interaction may result in the new piles being deflected out of the design alignment requiring re-driving or re-design during construction.</li><li>Very long piles (30 m to 50 m) required to achieve the required design resistances – requirement for two to three splices.</li><li>Temporary protection systems will be required along Highway 9 lanes to facilitate excavation to pile cap level.</li><li>Pile load testing (i.e. PDA) recommended to confirm capacity of long friction piles during construction.</li></ul>	<ul style="list-style-type: none"><li>High risk that the new piles will come in contact with the existing piles during installation/driving; such interaction may result in the new piles being deflected out of the design alignment requiring work stoppage, re-driving and/or re-design during construction.</li><li>Piles should be driven after the preferred embankment settlement mitigation measure has been implemented for some reduction in downdrag and drag loads. However, theoretical analysis indicates relatively large drag loads will still occur and so drag loads must be considered in the pile design.</li></ul>	<ul style="list-style-type: none"><li>Higher relative cost than spread / strip footings.</li><li>Lower relative cost than drilled shaft (caissons).</li><li>Additional cost for pile load tests (PDA) during construction.</li></ul>



Foundation Option	Feasibility	Advantages	Disadvantages	Risk / Consequences	Estimated Costs
<p>Steel H-piles – 30 m to 50 m long friction piles with new pile cap(s) located behind existing abutment(s)</p> <p>HP310x110 or HP310x132 or HP360x132</p>	<p>Feasible and recommended option for support of the abutments.</p>	<ul style="list-style-type: none"> <li>With the new piles located outside/behind the existing abutments, the risk of the new piles interfering with the existing piles is considered low.</li> <li>Abutment pile caps located at a higher elevation (i.e., within the approach embankment fill) as compared with shallow foundation option founded on native sand soils thereby reducing excavation depth and associated protection system requirements; however, excavation of the peat layer would still be required for the embankment widening(s).</li> <li>Higher geotechnical resistances can be obtained (depending on pile size and length) compared to shallow foundations.</li> <li>Allows for integral abutment construction, assuming that the pile capacities are sufficient to allow for one row of piles.</li> </ul>	<ul style="list-style-type: none"> <li>Very long piles (30 m to 50 m) required to achieve the required design resistances – requirement for two to three splices.</li> <li>Temporary protection systems will be required along Highway 9 lanes to facilitate excavation to pile cap level.</li> <li>Pile load testing (i.e. PDA) recommended to confirm capacity of long friction piles during construction.</li> </ul>	<ul style="list-style-type: none"> <li>Piles should be driven after the preferred embankment settlement mitigation measure has been implemented for some reduction in downdrag and drag loads. However, theoretical analysis indicates relatively large drag loads will still occur and so drag loads must be considered in the pile design.</li> </ul>	<ul style="list-style-type: none"> <li>Higher relative cost than spread / strip footings.</li> <li>Higher relative cost than for piles located inside the existing abutments due to the requirements for a longer span bridge, including longer girders and a higher volume of fill material.</li> <li>Lower relative cost than drilled shaft (caissons).</li> <li>Additional cost for pile load tests (PDA) during construction.</li> </ul>
<p>Drilled shafts / caissons</p>	<ul style="list-style-type: none"> <li>Not considered practical for support of abutments due to lack of a refusal layer (i.e. 3 m of “100-blow” soil) within 60 m below the road grade.</li> <li>Friction caissons require a long shaft to obtain adequate geotechnical resistances for SLS (to limit abutment settlement) and would be expensive to construct in the soft to stiff clayey soils.</li> </ul>		<ul style="list-style-type: none"> <li>Presence of relatively weak (firm) and thick clay deposit creates potential for construction problems associated with soil squeeze and need for likely permanent liners.</li> <li>Temporary or permanent liners would be required plus special measures such as use of drilling mud or head of water inside casing, and tremie placement of concrete; likely not possible to inspect caisson base.</li> <li>Proper cleaning of the base would still be required even though base resistance not a significant component of caisson capacity for friction caissons – difficult cleaning operation below the water table.</li> <li>Precludes use of integral abutments, unless a hybrid pile design adopted which requires a larger diameter increasing risk of conflicts with existing piles during installation.</li> <li>Generation of soil cuttings / spoils during drilled shaft advancement.</li> </ul>	<ul style="list-style-type: none"> <li>May not be possible to inspect base of the drilled shaft due to length of foundation element and potential need for bentonite slurry inside the liners which may affect shaft resistance.</li> <li>Drilled shafts should be installed after the preferred embankment settlement mitigation measure has been implemented to reduce drag loads; otherwise full drag loads must be considered in the pile design.</li> <li>Not possible to easily load test drilled shaft so geotechnical resistance cannot be confirmed during construction.</li> </ul>	<ul style="list-style-type: none"> <li>Higher relative cost than spread / strip footings and driven steel H-piles.</li> </ul>



TABLE 2 – COMPARISON OF SETTLEMENT MITIGATION OPTIONS– HIGHWAY 9 – HOLLAND DRAINAGE CANAL BRIDGE

Foundation Option	Feasibility	Advantages	Disadvantages	Risk / Consequences	Estimated Costs
Do Nothing	<ul style="list-style-type: none"><li>May be feasible depending on MTO's willingness to accept post-construction settlements in excess of MTO's Embankment Settlement Criteria Guideline (July 2010).</li></ul>	<ul style="list-style-type: none"><li>No additional fill required beyond embankment widening design geometry.</li><li>No delay in schedule as preload or surcharge period is not required.</li><li>Settlement monitoring program is not necessarily required (but some instrumentation maybe advisable in order to track the rate and magnitude of settlement and plan rehabilitation/maintenance activities accordingly).</li></ul>	<ul style="list-style-type: none"><li>Does not reduce post-construction settlement to within limits of MTO Embankment Settlement Criteria Guideline.</li><li>Maintenance will be required, likely at more than one time over the 20 year design life, to mitigate differential settlements at the bridge structure and between the embankment grade raise and widening areas.</li><li>Enbridge gas line will need to be relocated due to the settlements caused by the embankment widening.</li></ul>	<ul style="list-style-type: none"><li>Risks of poor roadway performance across new embankment section (between existing and widened embankment) and between approach embankment and bridge structure; will require asphalt padding and possibly installation of lightweight fill within the first 20 years after construction.</li><li>Risk of distress to structural approach slab; may require maintenance or replacement..</li><li>Long-term monitoring recommended and maintenance required to maintain highway safety.</li></ul>	<ul style="list-style-type: none"><li>Least expensive construction option, but greater long-term maintenance costs.</li></ul>
Preloading	<ul style="list-style-type: none"><li>Not considered practical as preloading period is estimated to be approximately 20 years, exceeding any practical construction time period.</li></ul>	<ul style="list-style-type: none"><li>Reduces post-construction settlement (but not as much as for surcharge option); some creep settlement will still occur.</li><li>No additional fill required beyond embankment widening design geometry.</li></ul>	<ul style="list-style-type: none"><li>Long delay in schedule to accommodate preload period unless early works contract for preload fill construction adopted.</li><li>Preloading period is estimated to be approximately 20 years in duration, exceeding any practical construction time period.</li><li>Potential for negative impact to the existing piles and approach embankments with existing bridge in operation during construction/preload period.</li><li>Would require embankment settlement monitoring program.</li><li>Enbridge gas line will likely need to be relocated due to the settlements caused by the embankment widening.</li></ul>	<ul style="list-style-type: none"><li>Risks of impact to performance of the existing bridge and road embankments (i.e., settlement due to preload fill and associated downdrag loads on existing piles) due to close proximity of embankment widening to existing bridge; monitoring of existing structure is recommended.</li><li>Long-term monitoring and maintenance required to maintain highway safety during the preload period.</li></ul>	<ul style="list-style-type: none"><li>Additional costs required for instrumentation and monitoring.</li></ul>

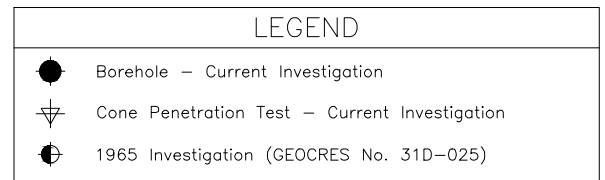
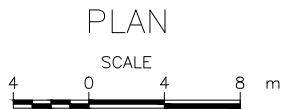


Foundation Option	Feasibility	Advantages	Disadvantages	Risk / Consequences	Estimated Costs
Preloading and Surcharging	<ul style="list-style-type: none"><li>Feasible to mitigate post construction settlement to within MTO's Embankment Settlement Criteria Guideline; however, not considered practical as the surcharging period would be approximately 60 months, which would exceed the typical construction time period.</li></ul>	<ul style="list-style-type: none"><li>Reduces post-construction settlement to within limits of MTO Embankment Settlement Criteria Guideline.</li><li>Requires shorter construction preload/surcharge duration than that required for preloading alone.</li><li>Reduces post-construction settlement as compared to preloading alone.</li></ul>	<ul style="list-style-type: none"><li>Delay in schedule to accommodate preload and surcharge period unless early works contract for preload fill construction adopted.</li><li>Surcharge period is estimated to be 60 months in duration, exceeding the typical construction time period.</li><li>Potential for negative impact to the existing piles and approach embankments with existing bridge in operation during surcharge period.</li><li>Would require settlement monitoring program.</li><li>Enbridge gas line will need to be relocated due to the settlements caused by the embankment widening.</li></ul>	<ul style="list-style-type: none"><li>Risks of impact to performance of the existing bridge and road embankments (i.e. settlement due to surcharge fill and associated downdrag loads on existing piles) due to close proximity of embankment widening and surcharge fill to existing bridge; monitoring of existing structure is recommended.</li><li>Surcharge load must be constructed in such a way to maintain stability of the surcharge and existing embankment.</li><li>Long-term monitoring and maintenance required to maintain highway safety during the surcharge period.</li></ul>	<ul style="list-style-type: none"><li>Additional costs for placement and removal of surcharge.</li><li>Additional costs required for instrumentation and monitoring.</li></ul>
Lightweight Fill – Cellular Concrete	<ul style="list-style-type: none"><li>Feasible at this site, but maximum thickness of cellular concrete restricted to 2.5 m due to uplift (buoyancy) considerations as a result of the high water level.</li></ul>	<ul style="list-style-type: none"><li>Reduces post-construction settlement to within limits of MTO Embankment Settlement Criteria Guideline within embankment grade raise areas only; long-term settlement in embankment widening areas within the approach area (0 m to 20 m from abutment) still exceeds criteria.</li><li>Reduces potential for differential settlement between widened and existing embankments.</li><li>Some reduction in downdrag loads on new piles.</li><li>No delay in schedule as preload or surcharge period is not required.</li><li>No settlement monitoring program required.</li></ul>	<ul style="list-style-type: none"><li>High cost of cellular concrete material.</li><li>Due to buoyancy as a result of the high-water level, only about 2.5 m thickness of cellular concrete is feasible at the site; as such post-construction settlement not reduced to within MTO Embankment Settlement Criteria Guideline limits within embankment widening areas.</li><li>Enbridge gas line will need to be relocated due to the settlements caused by the embankment widening.</li></ul>	<ul style="list-style-type: none"><li>Buoyancy of lightweight fill may cause floatation of embankment depending on layout of lightweight fill and actual groundwater elevation at time of high-water/50-year storm event.</li><li>Reduced risk of impacts to existing embankment and bridge structure.</li><li>Reduced risk of differential settlements between existing embankments and bridge abutments (in grade raise areas), but differential settlements will remain between widened embankment areas, grade raise areas and new bridge abutments.</li></ul>	<ul style="list-style-type: none"><li>Lightweight fill materials cost is up to an order of magnitude higher than other fill materials; on the order of \$250/m³.</li></ul>



Foundation Option	Feasibility	Advantages	Disadvantages	Risk / Consequences	Estimated Costs
Wick Drains (on 2 m to 2.5 m triangular grid spacing), plus Surcharging, and Preloading	<ul style="list-style-type: none"> <li>Feasible from a geotechnical / foundation perspective.</li> <li>This option would mitigate post construction settlement to within MTO's Embankment Settlement Criteria Guideline in the widened portion of the embankment but would not meet criteria in grade raise area.</li> <li>Time required for surcharge period is anticipated to be reduced to about 8 months (monitoring required for confirmation).</li> </ul>	<ul style="list-style-type: none"> <li>Reduces post-construction settlement to within MTO Embankment Settlement Criteria Guideline in the widening area of the embankment only.</li> <li>Duration of preload/surcharge reduced to about 8 months by installation of wick drains which speed up rate of consolidation.</li> <li>Some reduction in downdrag loads on new piles.</li> </ul>	<ul style="list-style-type: none"> <li>Differential settlement across embankment section will be greater than specified in MTO Settlement Criteria Guideline.</li> <li>Delay in schedule to install wick drains and to accommodate preload and surcharge period unless early works contract for preload fill construction adopted.</li> <li>Deep wick drain installation (up to 45 m depth) required through an upper deposit of silts and sands.</li> <li>Would require settlement monitoring program.</li> <li>A portion of the existing embankment fill and/or upper sands and silts may need to be pre-drilled through, to access treatment areas and install wick drains.</li> <li>Enbridge gas line will need to be relocated to avoid conflicts with wick drain installation and due to the settlements caused by the embankment widening/surcharging.</li> </ul>	<ul style="list-style-type: none"> <li>Risk that preload/surcharge time takes longer than estimated; assessment based on monitoring data required.</li> <li>Surcharge load must be constructed in such a way to maintain stability of the surcharge and existing embankment. Some risk of surficial instability of 1H:1V surcharge side slopes during surcharge period.</li> </ul>	<ul style="list-style-type: none"> <li>Higher cost than preload only or surcharge only options due to additional costs for installation of deep wick drains and possibly due to requirements for pre-drilling.</li> <li>Additional costs required for instrumentation and monitoring.</li> </ul>
Wick Drains (on 2 m to 2.5 m triangular grid spacing), plus Surcharging, and Preloading, and Cellular Concrete	<ul style="list-style-type: none"> <li>Feasible and preferred option from a geotechnical / foundation perspective as this option would mitigate post construction settlement to within MTO's Embankment Settlement Criteria Guideline in both the widened and grade raise portions of the embankment.</li> <li>Time required for surcharge period is anticipated to be reduced to about 8 months (monitoring required for confirmation).</li> </ul>	<ul style="list-style-type: none"> <li>Reduces post-construction settlement to within MTO Embankment Settlement Criteria Guideline in the widening areas and in the grade raise areas of the new embankments.</li> <li>Differential settlements across embankment section (between widening and grade raise areas) are reduced when compared to wick drain and surcharge option without use of cellular concrete.</li> <li>Duration of preload/surcharge period reduced to about 8 months by installation of wick drains which speed up rate of consolidation.</li> </ul>	<ul style="list-style-type: none"> <li>Differential settlement across embankment section is greater than specified in MTO Embankment Settlement Criteria Guideline.</li> <li>Delay in schedule to install wick drains and to accommodate preload and surcharge period unless early works contract for preload fill construction adopted.</li> <li>Deep wick drain installation (up to 30 m depth) required through an upper deposit of silts and sands.</li> <li>Would require settlement monitoring program.</li> <li>A portion of the existing embankment fill and/or upper sands and silts may need to be pre-drilled through, to access treatment areas and install wick drains.</li> <li>Enbridge gas line will need to be relocated to avoid conflicts with wick drain installation and due to the settlements caused by the embankment widening/surcharging</li> <li>Theoretically, little reduction in drag loads on piles as a result of geometry of mitigation measures required to minimize differential settlements on embankments.</li> </ul>	<ul style="list-style-type: none"> <li>Risk that preload/surcharge time takes longer than estimated; assessment based on monitoring data required.</li> <li>Surcharge load must be constructed in such a way to maintain stability of the surcharge and existing embankment. Some risk of surficial instability of 1H:1V surcharge side slopes during surcharge period.</li> <li>Risk of floatation of lightweight fill if groundwater levels significantly exceed 50-year storm water levels.</li> </ul>	<ul style="list-style-type: none"> <li>Higher cost than surcharge and wick drain option due to additional costs for installation of cellular concrete.</li> <li>Additional costs required for instrumentation and monitoring.</li> </ul>





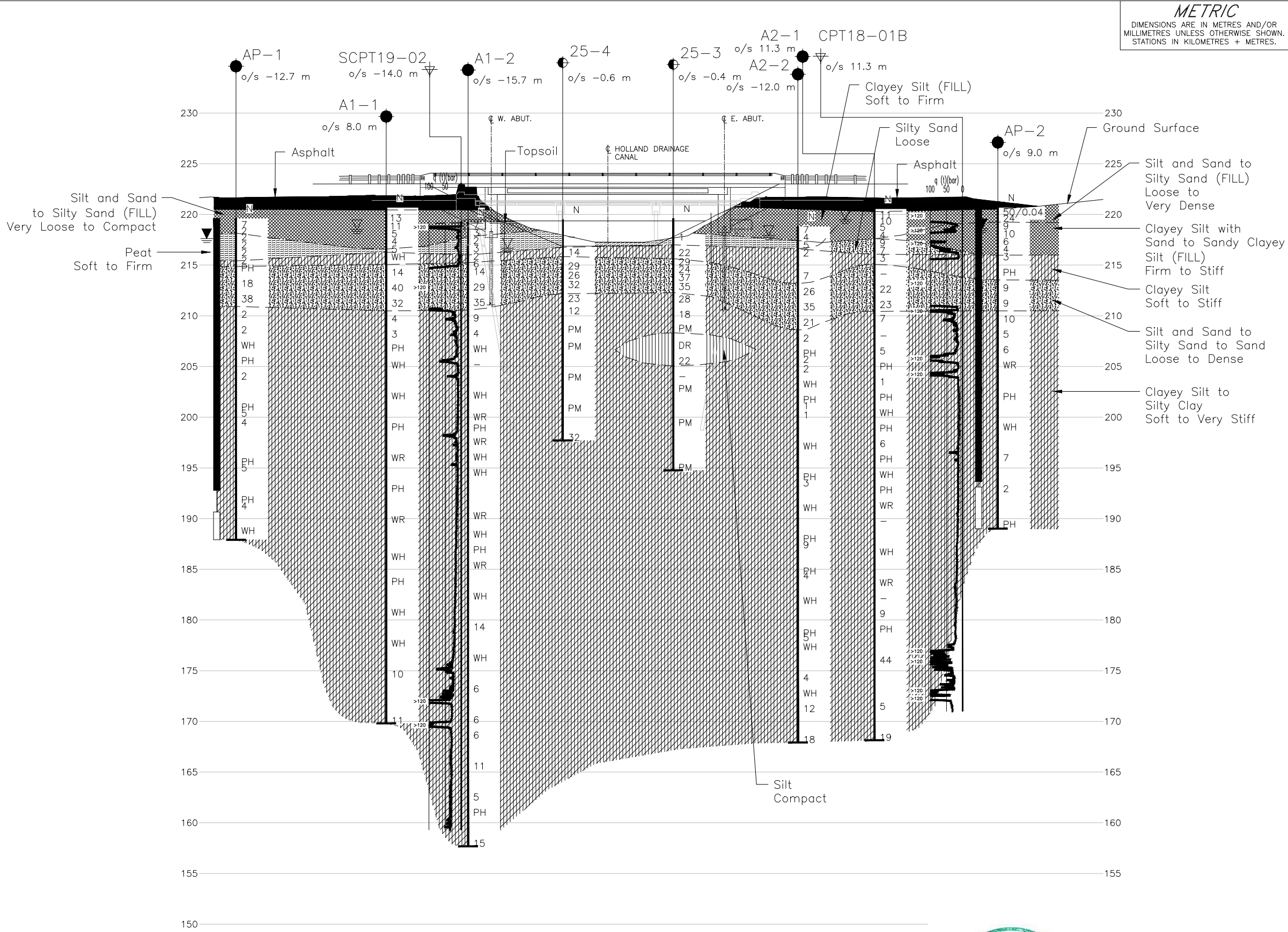
**NOTES**

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

NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9		PROJECT NO. 1671430	DIST. CENTRAL
SUBM'D. NK	CHKD. NK	DATE: 08/19/2019	SITE: 37-31
DRAWN: DD	CHKD. JPD	APPD. JPD	DWG. 1

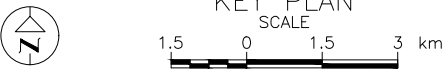
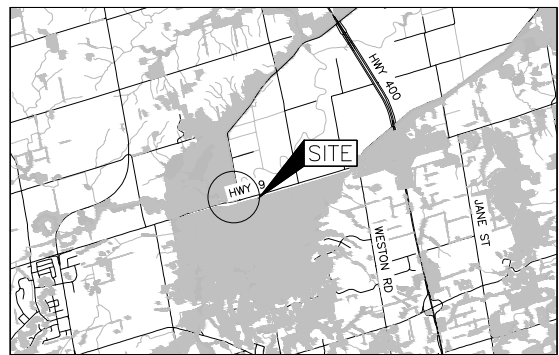






**METRIC**  
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.	SHEET
GWP No. 2266-18-00	82
HIGHWAY 9 HOLLAND DRAINAGE CANAL BRIDGE (SITE NO. 37-31) SOIL STRATA 1	



LEGEND

- Borehole - Current Investigation
- Cone Penetration Test - Current Investigation
- 1965 Investigation (GEOCRES No. 31D-025)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- q(t) CPT Corrected Tip Resistance (bar)
- WL in piezometer, measured on March 14 and 21, 2019
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
25-3	219.6	4875631.8	294048.8
25-4	219.5	4875629.1	294038.2
A1-1	220.7	4875633.5	294019.4
A1-2	219.3	4875612.3	294032.6
A2-1	220.6	4875646.1	294058.4
A2-2	218.8	4875623.4	294063.6
AP-1	219.6	4875610.1	294009.4
AP-2	220.7	4875648.5	294077.6
CPT18-01B	220.6	4875646.5	294060.1
SCPT19-02	219.3	4875613.1	294028.5

NOTES

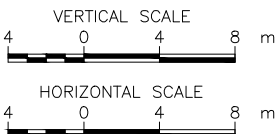
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

REFERENCE

Base plans and general arrangement provided in digital format by Aecom, drawing file nos. X-60570685-C-HWY9-BASE.dwg, C3D\_60570685-Holland Canal Bridge\_37-31-1-SPAN\_B700.dwg, 60570685-C-AL1-HWY 9.dwg, b-074-009-1.dwg, OG 37-31.dwg, received May 24, 2019 and 39 GENERAL ARRANGEMENT.dwg, X-60570685-C2-CONSTR-HWY 9.dwg, received July 22, 2019.

A-A PROFILE



NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9	PROJECT NO. 1671430	DIST.	
SUBM'D. NK	CHKD. NK	DATE: 08/19/2019	SITE:
DRAWN: DD	CHKD. JPD	APPD. JPD	DWG. 2

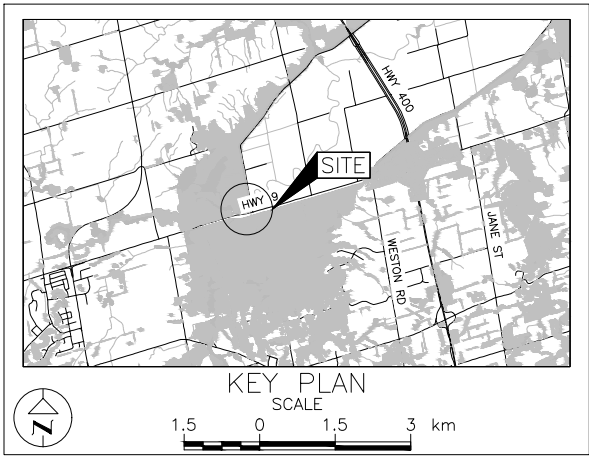


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
GWP No. 2266-18-00

HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE (SITE NO. 37-31)  
SOIL STRATA 2

SHEET  
83



LEGEND

- Borehole - Current Investigation
- ▽ Cone Penetration Test - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- q (t) CPT Corrected Tip Resistance (bar)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
A1-1	220.7	4875633.5	294019.4
A1-2	219.3	4875612.3	294032.6
SCPT19-02	219.3	4875613.1	294028.5

NOTES

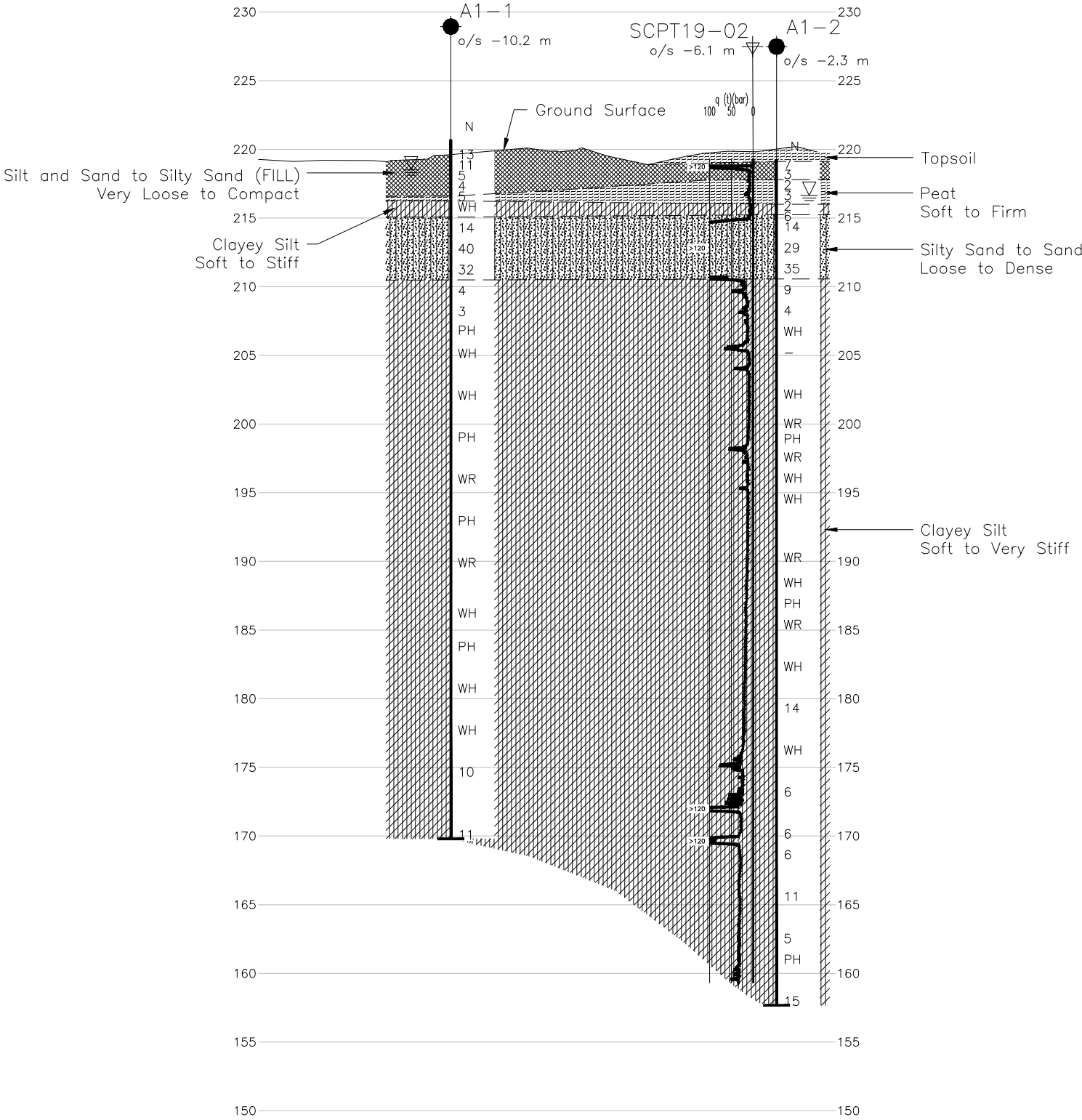
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

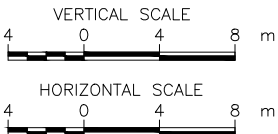
REFERENCE

Base plans and general arrangement provided in digital format by Aecom, drawing file nos. X-60570685-C-HWY9-BASE.dwg, CSD\_60570685-Holland Canal Bridge\_37-31-OPT\_1-SSPAN\_B700.dwg, 60570685-C-ALI-HWY 9.dwg, b-074-009-1.dwg, OG 37-31.dwg, received May 24, 2019 and 39 GENERAL ARRANGEMENT.dwg, X-60570685-C2-CONSTR-HWY 9.dwg, received July 22, 2019.

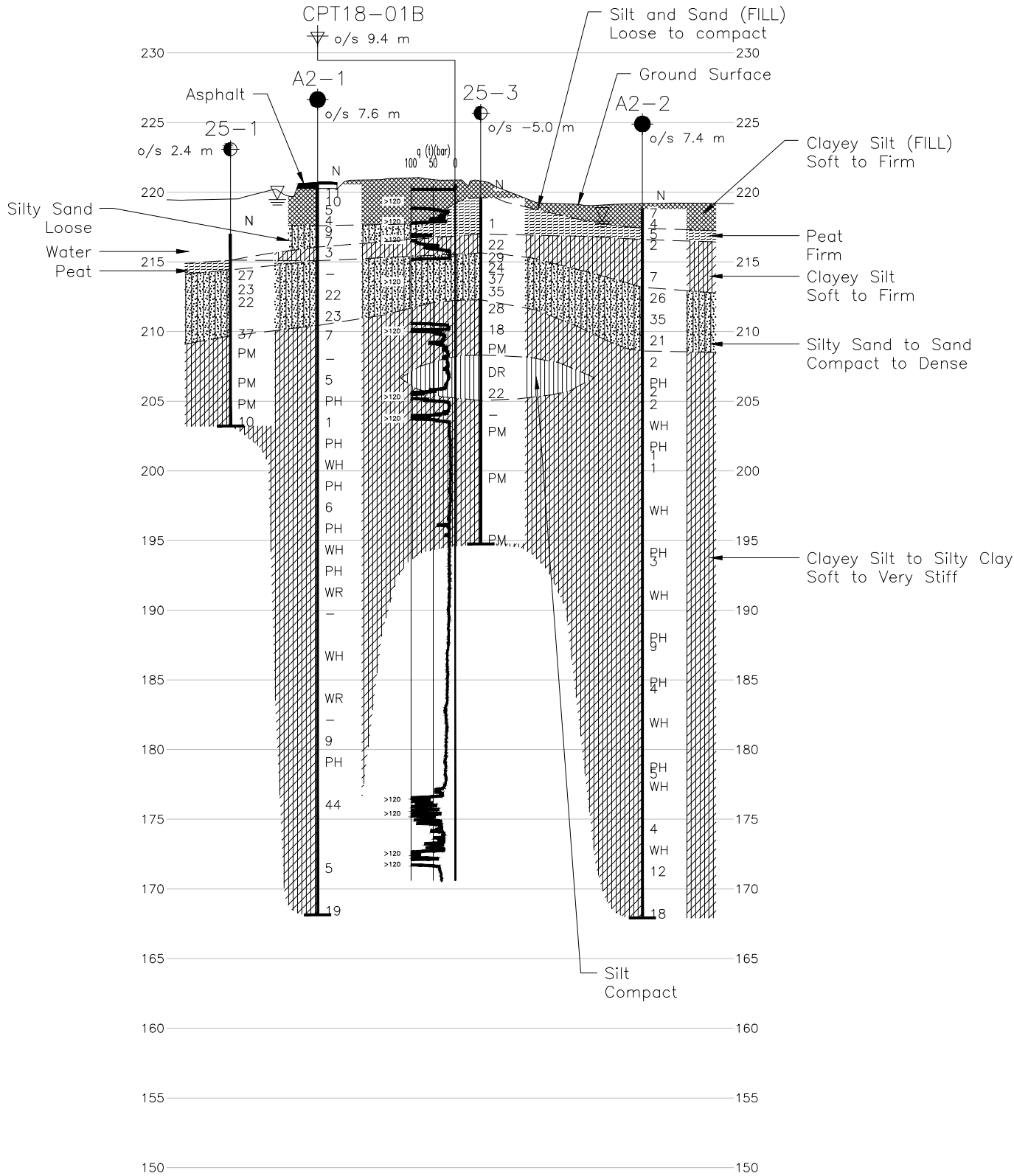
NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9	PROJECT NO. 1671430		DIST. .
SUBM'D. NK	CHKD. NK	DATE: 08/19/2019	SITE: .
DRAWN: DD	CHKD. JPD	APPD. JPD	DWG. 3



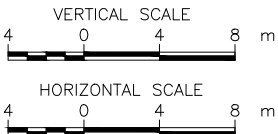
B-B WEST ABUTMENT  
1







C-C EAST ABUTMENT  
1

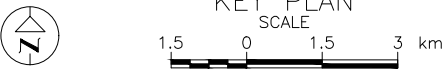


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No. \_\_\_\_\_  
GWP No. 2266-18-00

HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE (SITE NO. 37-31)  
SOIL STRATA 3

SHEET  
84



LEGEND

- Borehole - Current Investigation
- ▽ Cone Penetration Test - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- q(t) CPT Corrected Tip Resistance (bar)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
25-1	218.9	4875651.0	294051.8
25-3	219.6	4875631.8	294048.8
A2-1	220.6	4875646.1	294058.4
A2-2	218.8	4875623.4	294063.6
CPT18-01B	220.6	4875646.5	294060.1

NOTES

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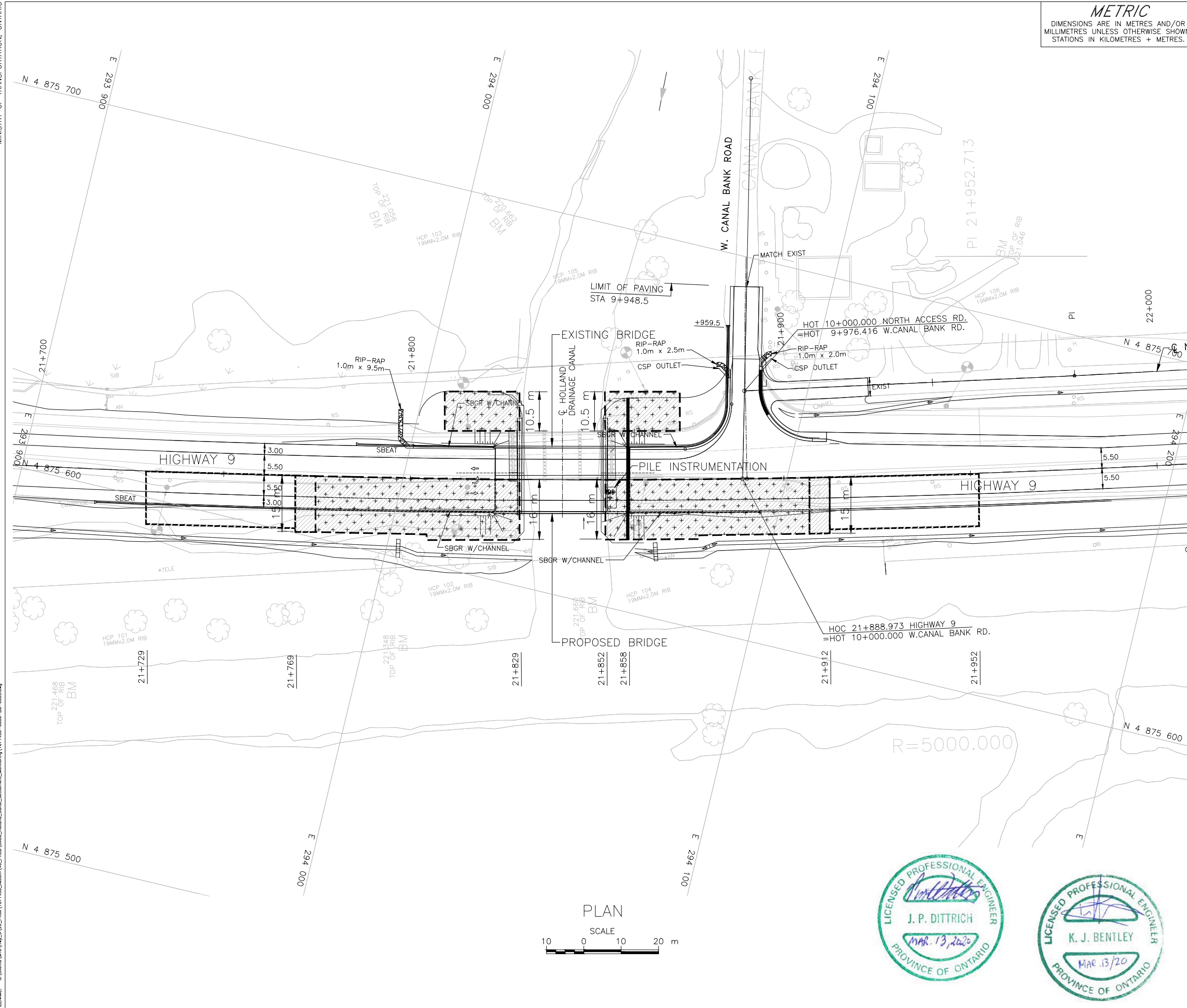
REFERENCE

Base plans and general arrangement provided in digital format by Aecom, drawing file nos. X-60570685-C-HWY9-BASE.dwg, CSD\_60570685-Holland Canal Bridge\_37-31-OPT\_1-SSPAN\_B700.dwg, 60570685-C-ALI-HWY 9.dwg, b-074-009-1.dwg, OG 37-31.dwg, received May 24, 2019 and 39 GENERAL ARRANGEMENT.dwg, X-60570685-C2-CONSTR-HWY 9.dwg, received July 22, 2019.

NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9	PROJECT NO. 1671430		DIST. .
SUBM'D. NK	CHKD. NK	DATE: 08/19/2019	SITE: .
DRAWN: DD	CHKD. JPD	APPD. JPD	DWG. 4







CONT No. 2020-2015  
GWP No. 2268-18-00

HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT  
WICK DRAIN, SURCHARGE AND  
INSTRUMENTATION LOCATIONS

**GOLDER**

KEY PLAN  
SCALE  
1.5 0 1.5 3 km

LEGEND

- Approximate Wick Drain and Surcharge Area
- Approximate Surcharge Only Area
- Approximate Preload Area
- Approximate Location of Pile Instrumentation (Refer to Drawings 12 and 13)
- Approximate Location of Shape Acceleration Array (SAA) (Refer to Drawings 14 and 15)

NOTES

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

REFERENCE

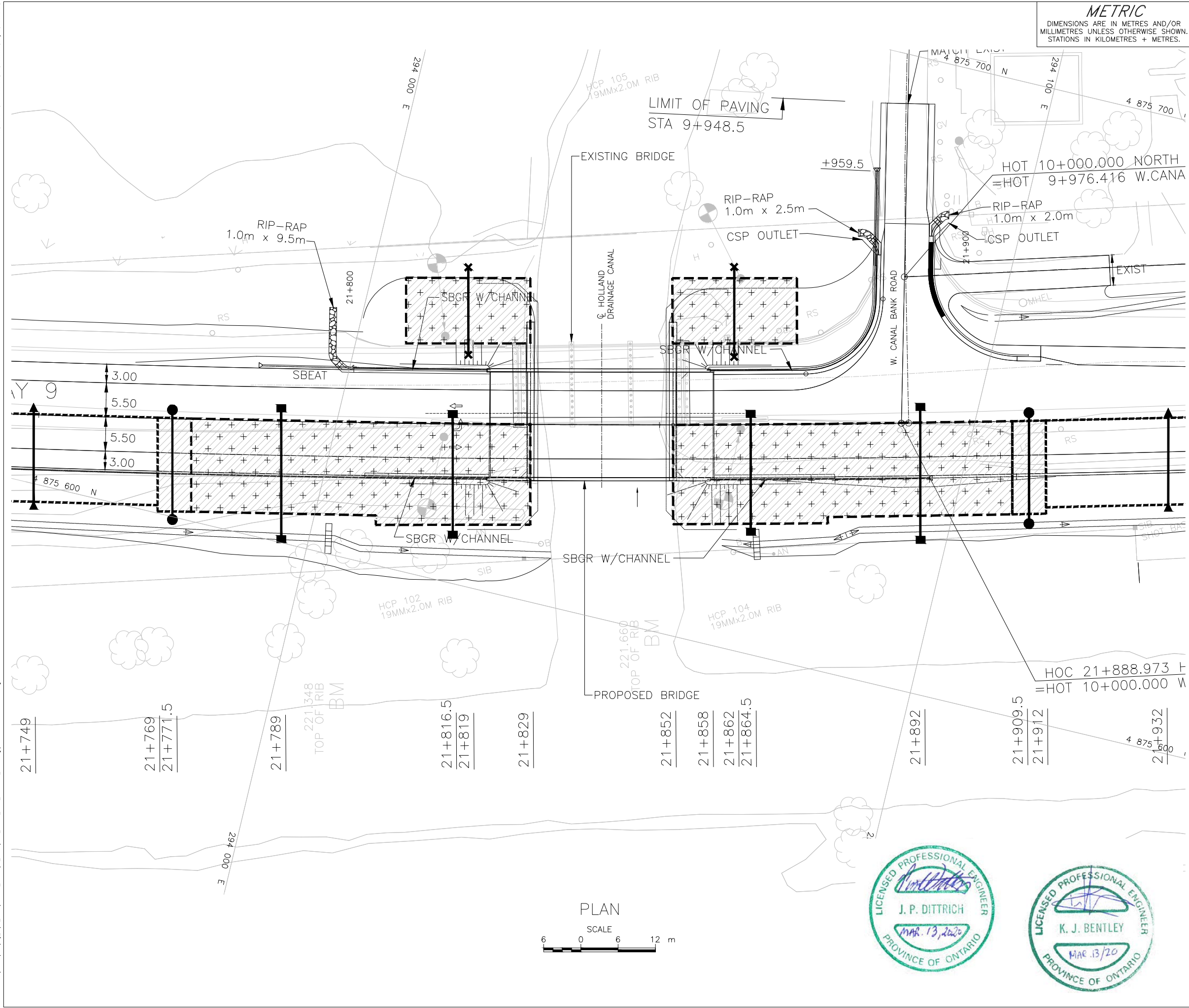
Base plans and general arrangement provided in digital format by Aecom, drawing file nos. X-60570685-C-HWY9-BASE.dwg, C3D\_60570685-Holland Canal Bridge\_37-31-OPT\_1-SSPAN\_B700.dwg, 60570685-C-ALI-HWY 9.dwg, b-074-009-1.dwg, OG 37-31.dwg, received May 24, 2019 and 39 GENERAL ARRANGEMENT.dwg, received July 22, 2019. Base plans provided in digital format by Aecom, drawing file no. X-60570685-C2-CONSTR-HWY 9.dwg, received December 09, 2019.

NO.	DATE	BY	REVISION

Geocres No. 31D-734

HWY. 9	PROJECT NO. 1671430	DIST.
SUBM'D. MA	CHKD. MA	DATE: 12/13/2019
DRAWN: DD	CHKD. MA/JMAC	APPD. JPD



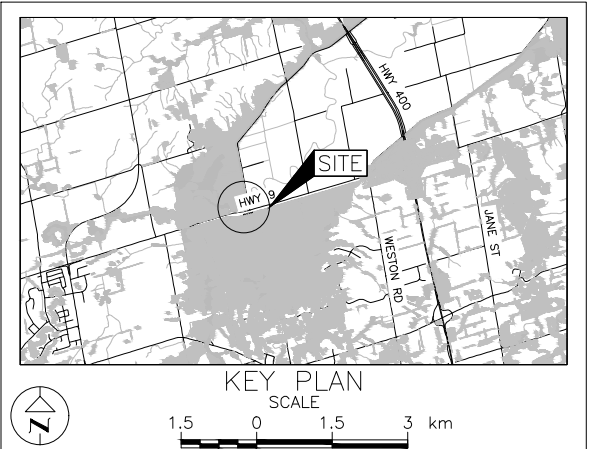


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No. 2020-2015  
GWP No. 2268-18-00

HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT  
SETTLEMENT MONITORING  
INSTRUMENTATION LOCATIONS

SHEET  
58



LEGEND

Approximate Wick Drain and Surge Area

Approximate Surge Only Area

Approximate Preload Area

Monitoring Section - Type A  
(Refer to Drawings 7 and 11)

Monitoring Section - Type B  
(Refer to Drawings 8 and 11)

Monitoring Section - Type C  
(Refer to Drawings 9 and 11)

Monitoring Section - Type D  
(Refer to Drawings 10 and 11)

**NOTES**  
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**REFERENCE**  
Base plans and general arrangement provided in digital format by Aecom, drawing file nos. X-60570685-C-HWY9-BASE.dwg, C3D\_60570685-Holland Canal Bridge\_37-31-OPT\_1-SSPAN\_B700.dwg, 60570685-C-ALL-HWY 9.dwg, b-074-009-1.dwg, OG 37-31.dwg, received May 24, 2019 and 39 GENERAL ARRANGEMENT.dwg, received July 22, 2019.  
Base plans provided in digital format by Aecom, drawing file no. X-60570685-C2-CONSTR-HWY 9.dwg, received December 09, 2019.

NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9		PROJECT NO. 1671430	DIST. .
SUBM'D. MA	CHKD. MA	DATE: 12/13/2019	SITE: .
DRAWN: DD	CHKD. MA/JMAC	APPD. JPD	DWG. 6



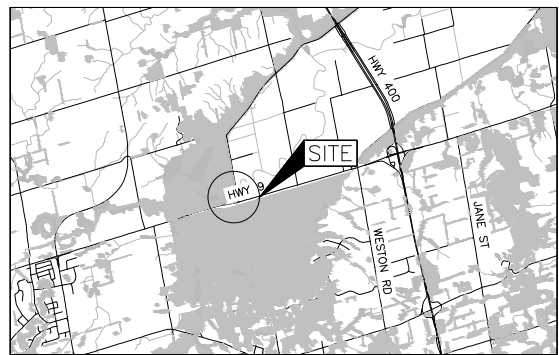


*METRIC*  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No. 2020-2015  
WP No. 2268-18-00

HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT  
EMBANKMENT MONITORING  
INSTRUMENTATION LOCATIONS - TYPE A

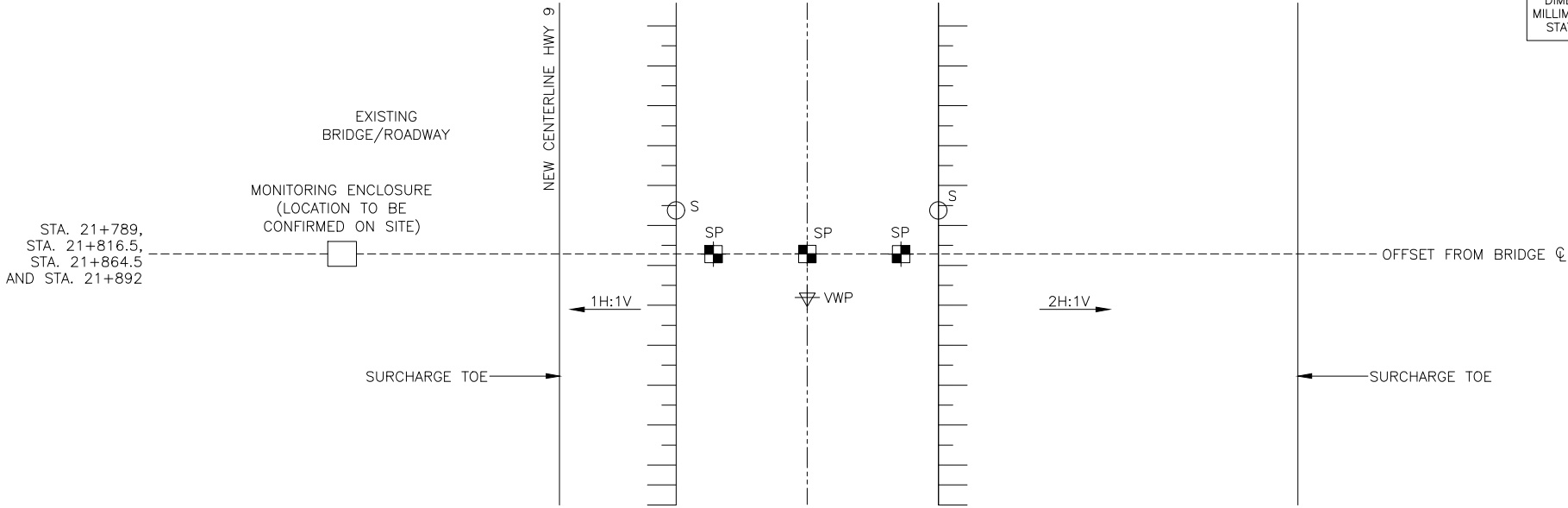
SHEET  
59



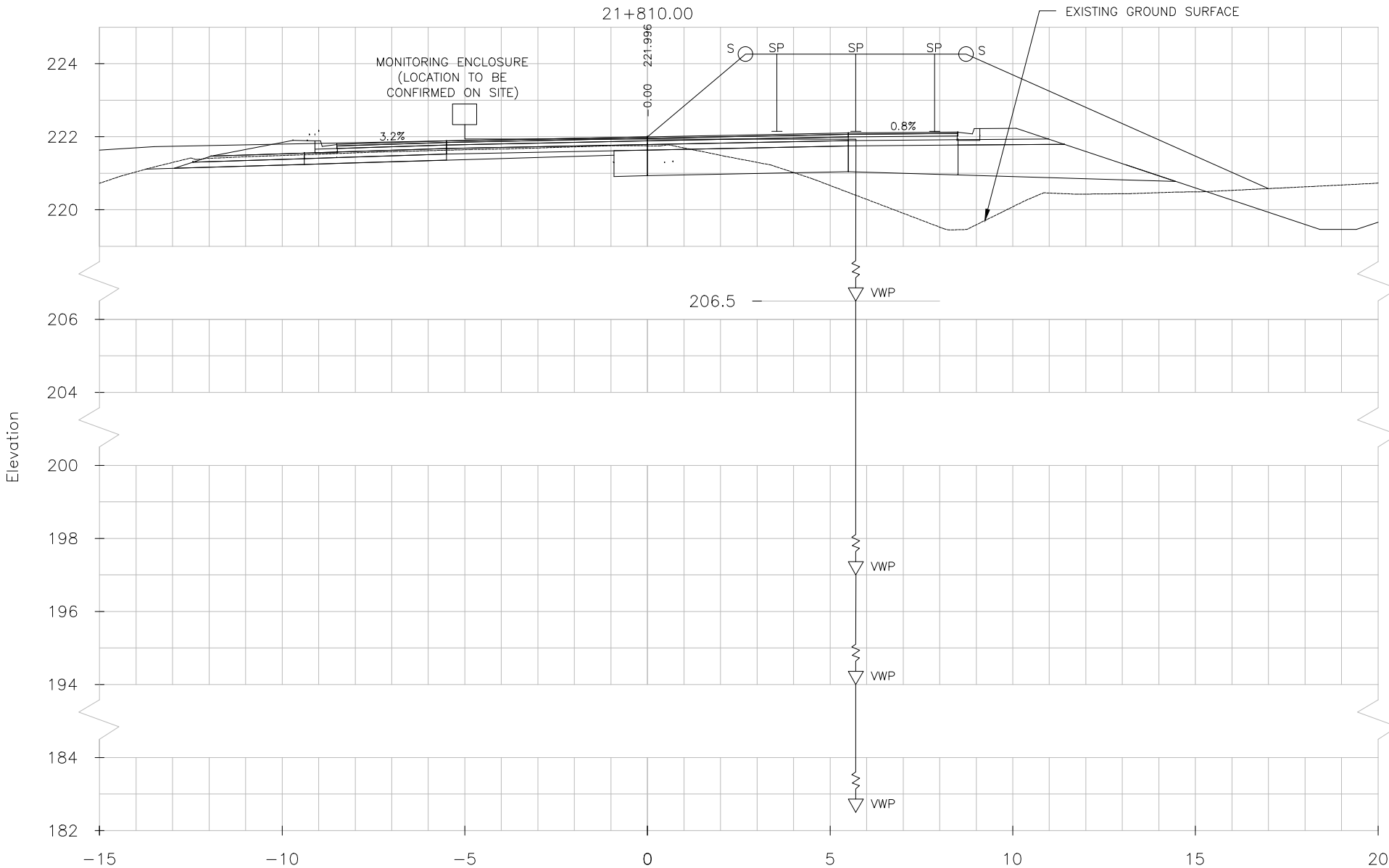
KEY PLAN  
SCALE  
1.5 0 1.5 3 km

LEGEND

- SP Settlement Plate
- VWP Multi-Level Vibrating Wire Piezometer (Plan)
- S Surface Settlement Point
- VWP Multi-Level Vibrating Wire Piezometer (Section)



MONITORING PLAN - INSTRUMENTATION TYPE A  
NOT TO SCALE



MONITORING SECTION - INSTRUMENTATION TYPE A  
NOT TO SCALE



NOTES

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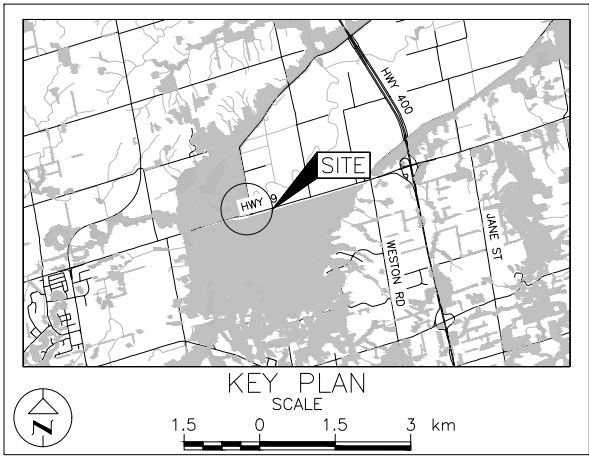
See Specifications for exact location, depth and number of instruments.

NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9		PROJECT NO. 1671430	
SUBM'D. MA		DATE: 12/13/2019	
DRAWN: DD		APPD. JPD	
CHKD. MA/JMAC		DIST. .	
		SITE: .	
		DWG. 7	

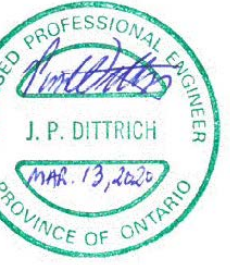


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No. 2020-2015 WP No. 2268-18-00	
HIGHWAY 9 HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT EMBANKMENT MONITORING INSTRUMENTATION LOCATIONS - TYPE B	
SHEET 60	

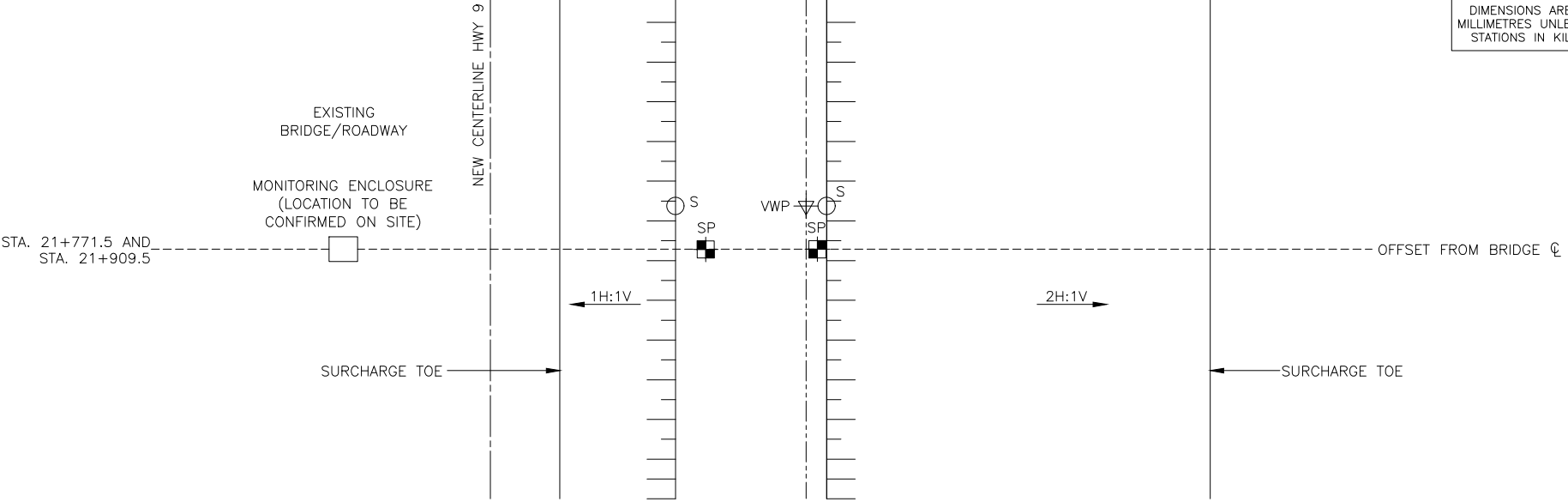


LEGEND	
	SP Settlement Plate
	VWP Multi-Level Vibrating Wire Piezometer (Plan)
	S Surface Settlement Point
	VWP Multi-Level vibrating Wire Piezometer (Section)

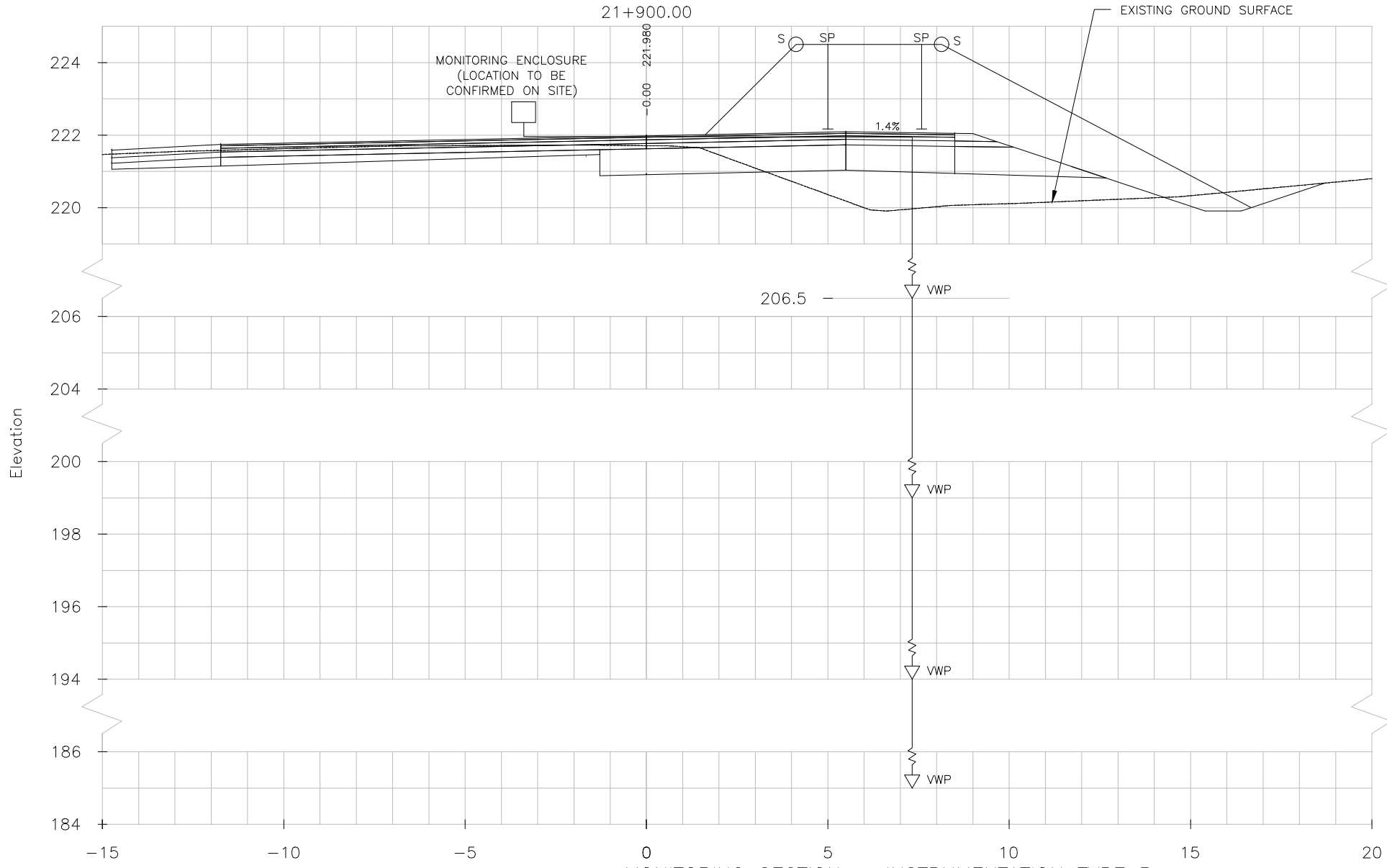


**NOTES**  
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See Specifications for exact location, depth and number of instruments.

NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9	PROJECT NO. 1671430		DIST. .
SUBM'D. MA	CHKD. MA	DATE: 12/13/2019	SITE: .
DRAWN: DD	CHKD. MA/JMAC	APPD. JPD	DWG. 8



MONITORING PLAN - INSTRUMENTATION TYPE B  
NOT TO SCALE



MONITORING SECTION - INSTRUMENTATION TYPE B  
NOT TO SCALE

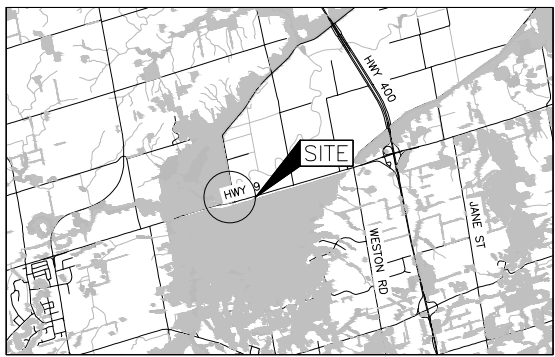


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No. 2020-2015  
WP No. 2268-18-00

HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT  
EMBANKMENT MONITORING  
INSTRUMENTATION LOCATIONS - TYPE C

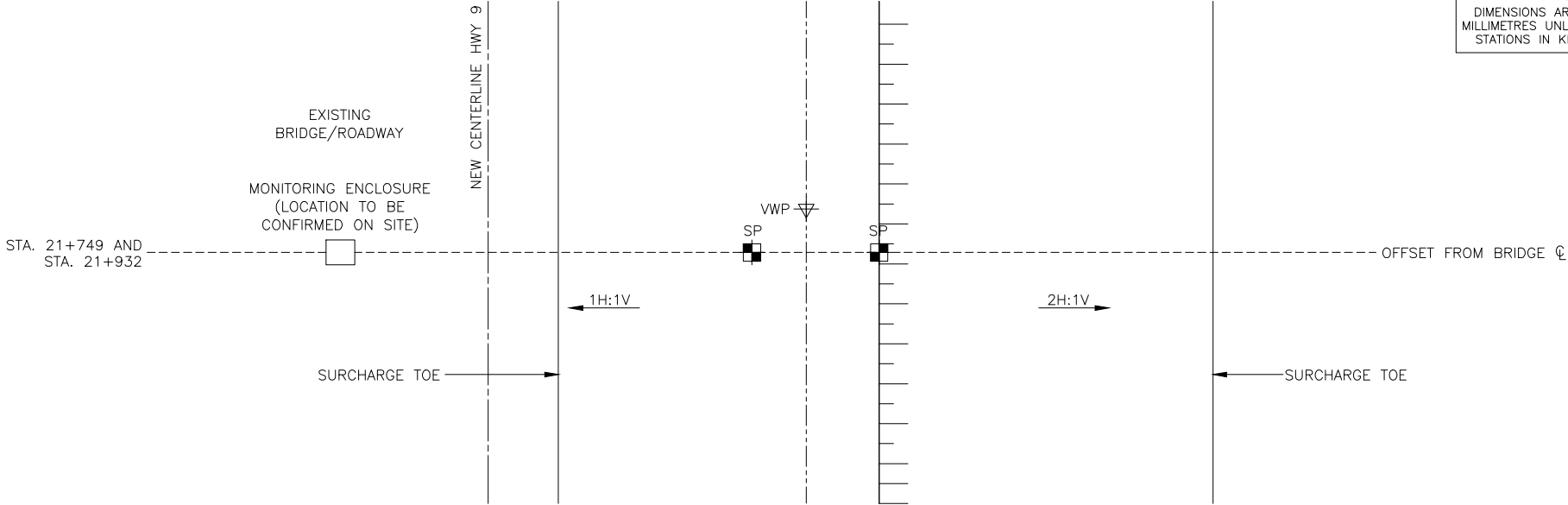
SHEET  
61



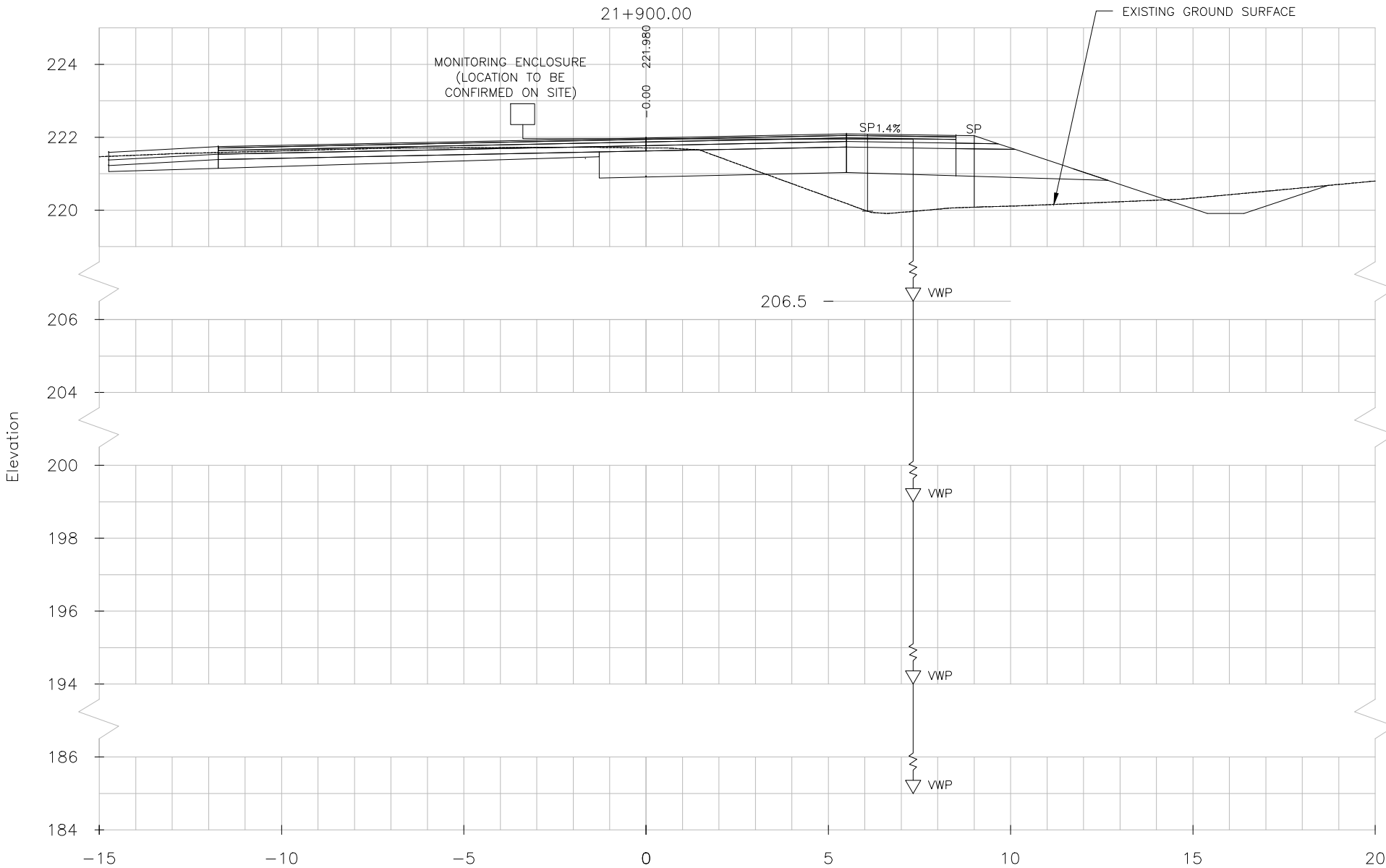
KEY PLAN  
SCALE  
1.5 0 1.5 3 km

LEGEND

- SP Settlement Plate
- VWP Multi-Level Vibrating Wire Piezometer (Plan)
- VWP Multi-Level vibrating Wire Piezometer (Section)



MONITORING PLAN - INSTRUMENTATION TYPE C  
NOT TO SCALE



MONITORING SECTION - INSTRUMENTATION TYPE C  
NOT TO SCALE



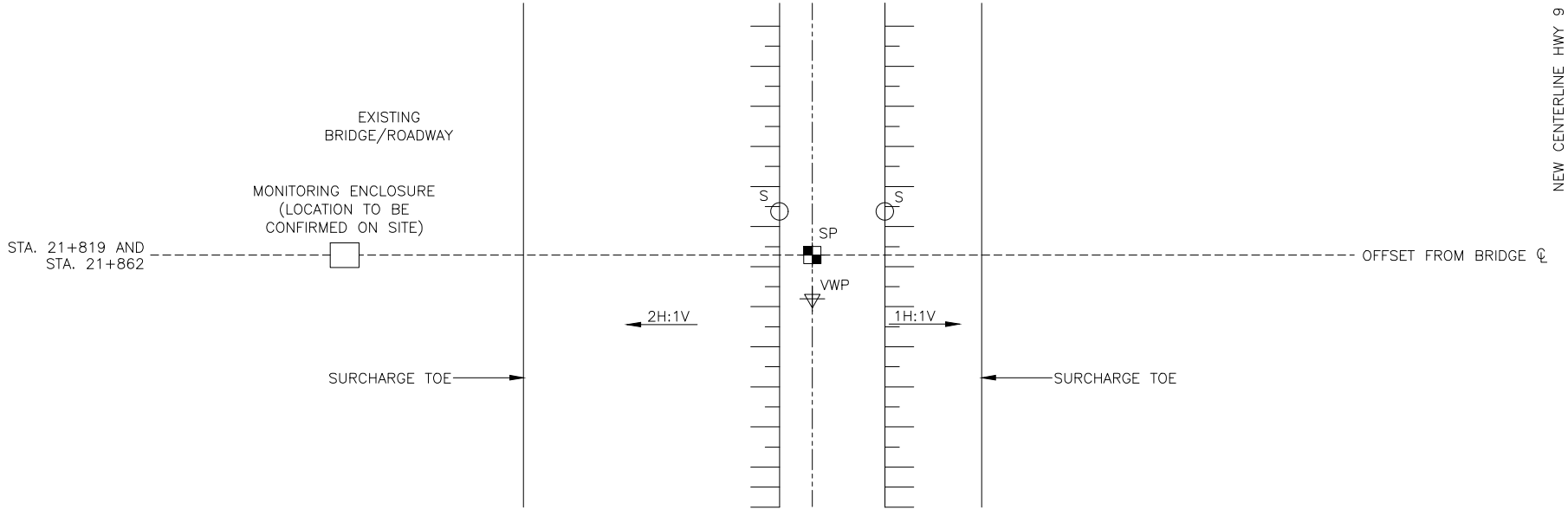
NOTES

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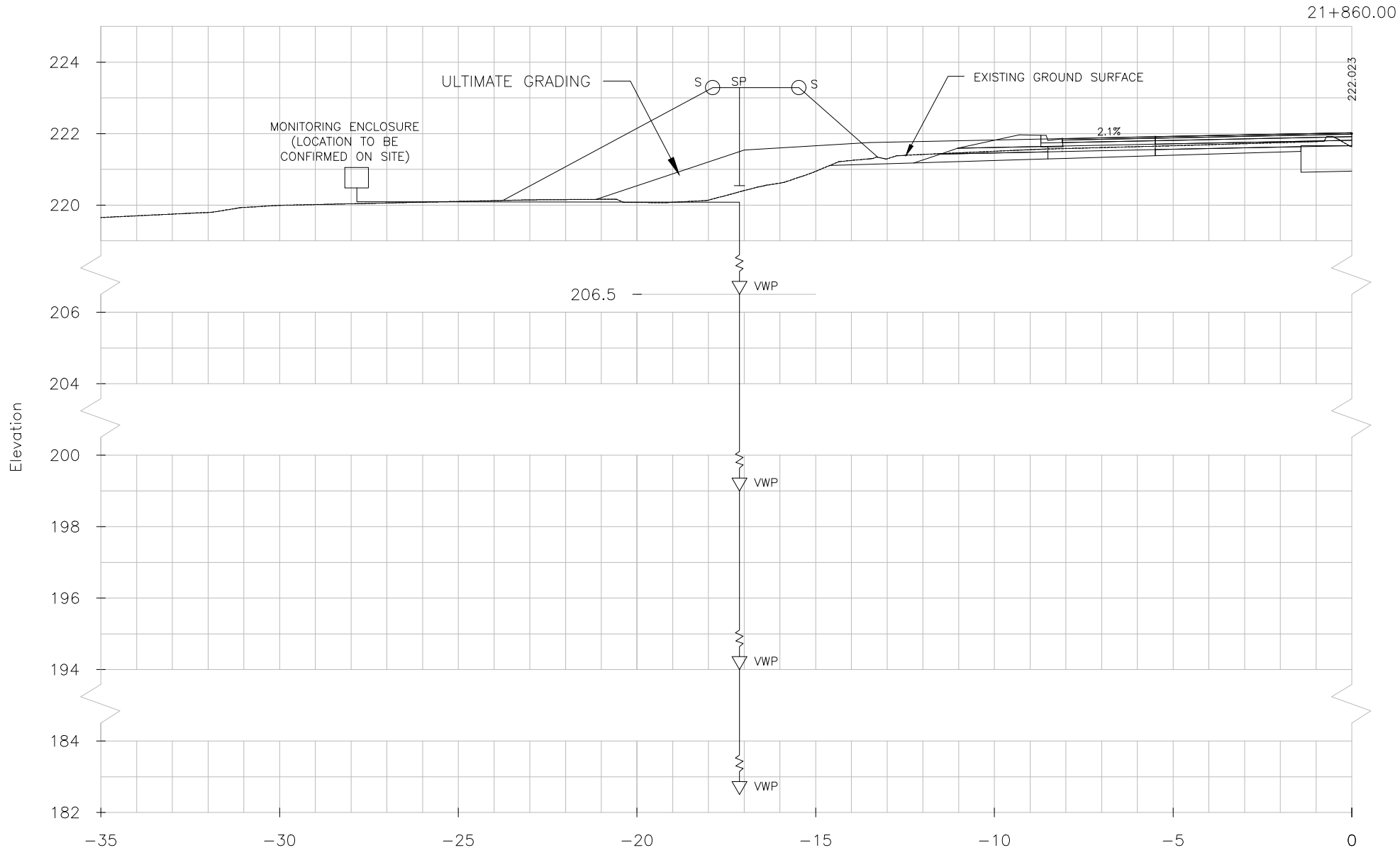
See Specifications for exact location, depth and number of instruments.

NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9	PROJECT NO. 1671430	DIST. .	
SUBM'D. MA	CHKD. MA	DATE: 12/13/2019	SITE: .
DRAWN: DD/JM	CHKD. MA/JMAC	APPD. JPD	DWG. 9





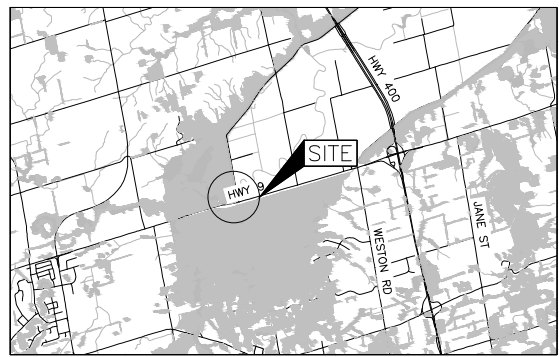
MONITORING PLAN — INSTRUMENTATION TYPE D  
NOT TO SCALE



MONITORING SECTION — INSTRUMENTATION TYPE D  
NOT TO SCALE

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

CONT No. 2020-2015 WP No. 2268-18-00	SHEET 62
HIGHWAY 9 HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT EMBANKMENT MONITORING INSTRUMENTATION LOCATIONS - TYPE D	



KEY PLAN  
SCALE  
1.5 0 1.5 3 km

LEGEND

- SP Settlement Plate
- VWP Multi-Level Vibrating Wire Piezometer (Plan)
- S Surface Settlement Point
- VWP Multi-Level Vibrating Wire Piezometer (Section)



NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.  
See Specifications for exact location, depth and number of instruments.

NO.	DATE	BY	REVISION
1	12/13/2019	DD	1
Geocres No. 31D-734			
HWY. 9	PROJECT NO. 1671430	DIST.	
SUBM'D. MA	CHKD. MA	DATE: 12/13/2019	SITE:
DRAWN: DD	CHKD. MA/JMAC	APPD. JPD	DWG. 10

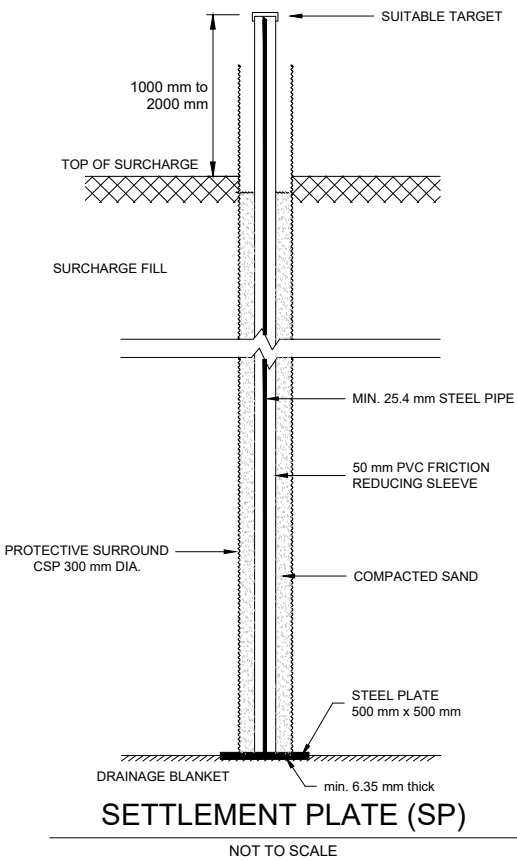
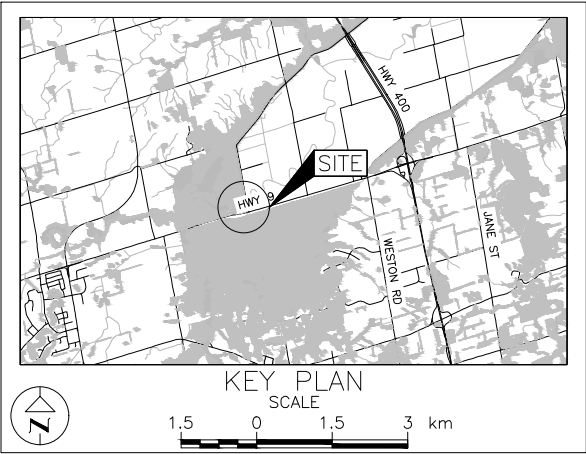


METRIC  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No. 2020-2015  
GWP No. 2268-18-00

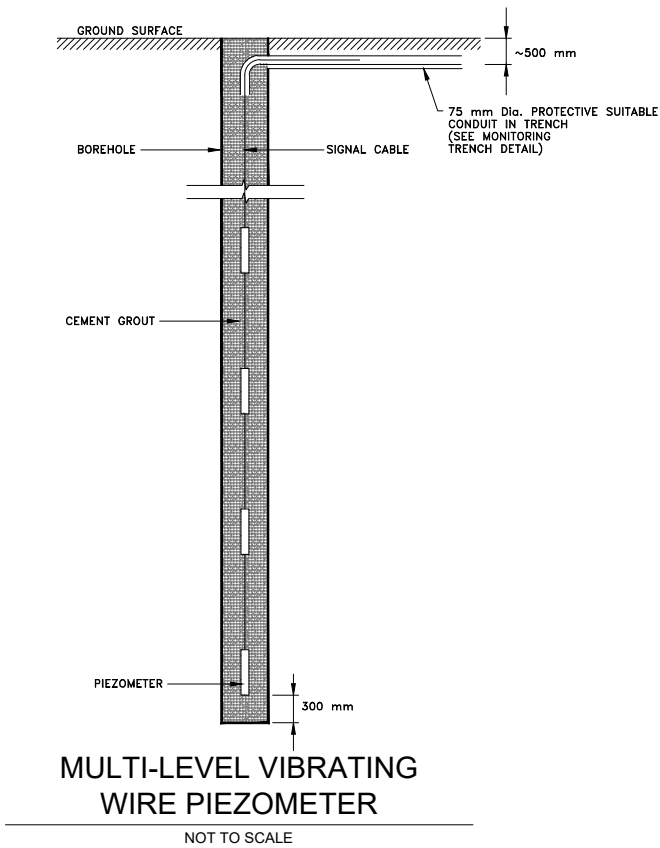
HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT  
TYPICAL EMBANKMENT MONITORING  
INSTRUMENTATION DETAILS

SHEET  
63



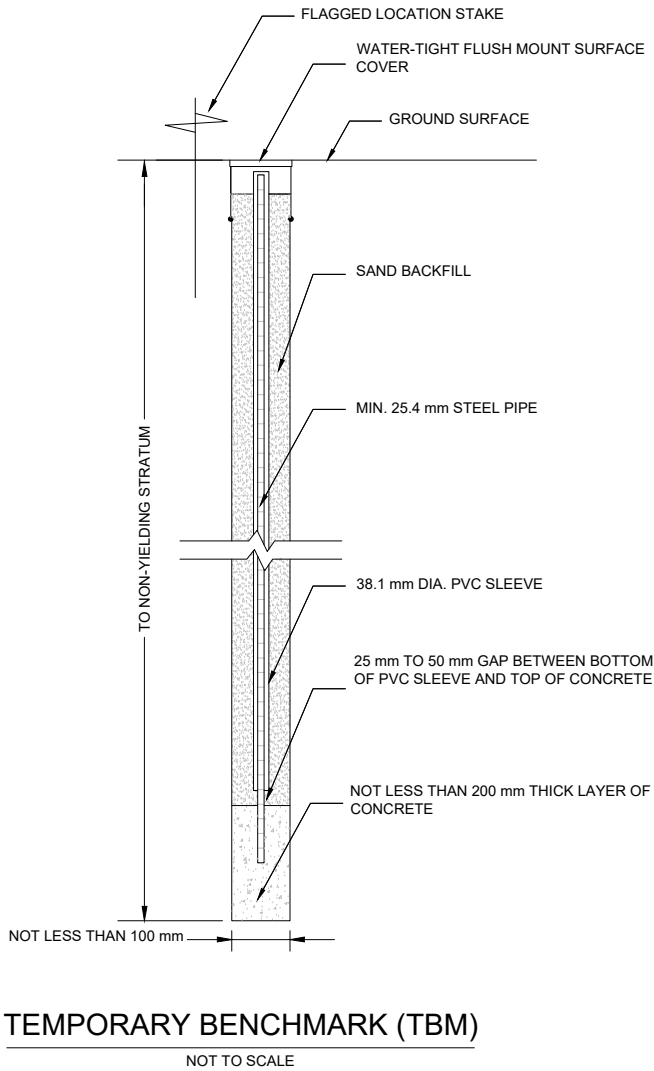
SETTLEMENT PLATE (SP)

NOT TO SCALE



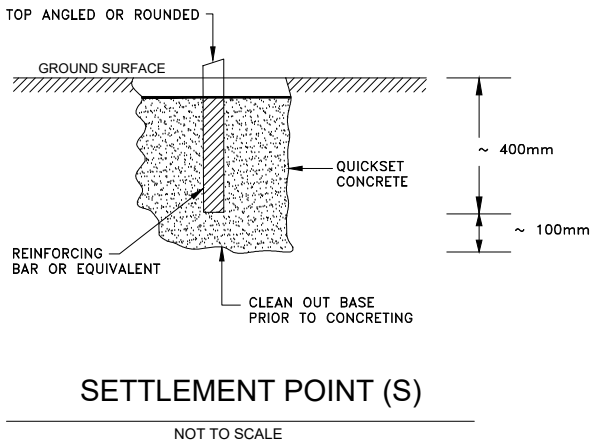
MULTI-LEVEL VIBRATING  
WIRE PIEZOMETER

NOT TO SCALE



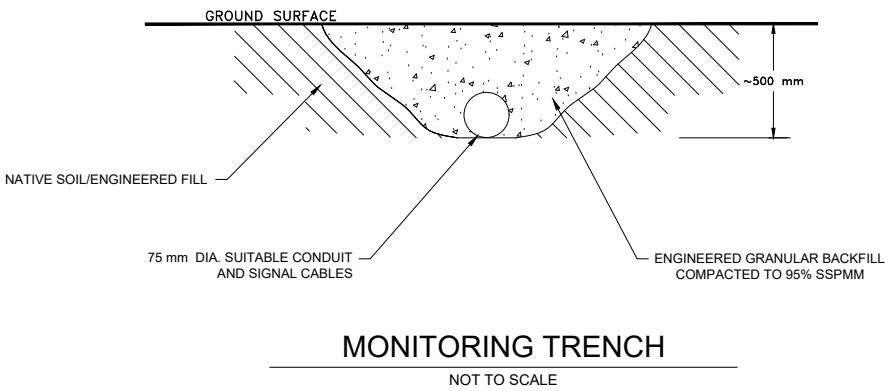
TEMPORARY BENCHMARK (TBM)

NOT TO SCALE



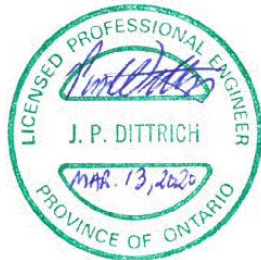
SETTLEMENT POINT (S)

NOT TO SCALE



MONITORING TRENCH

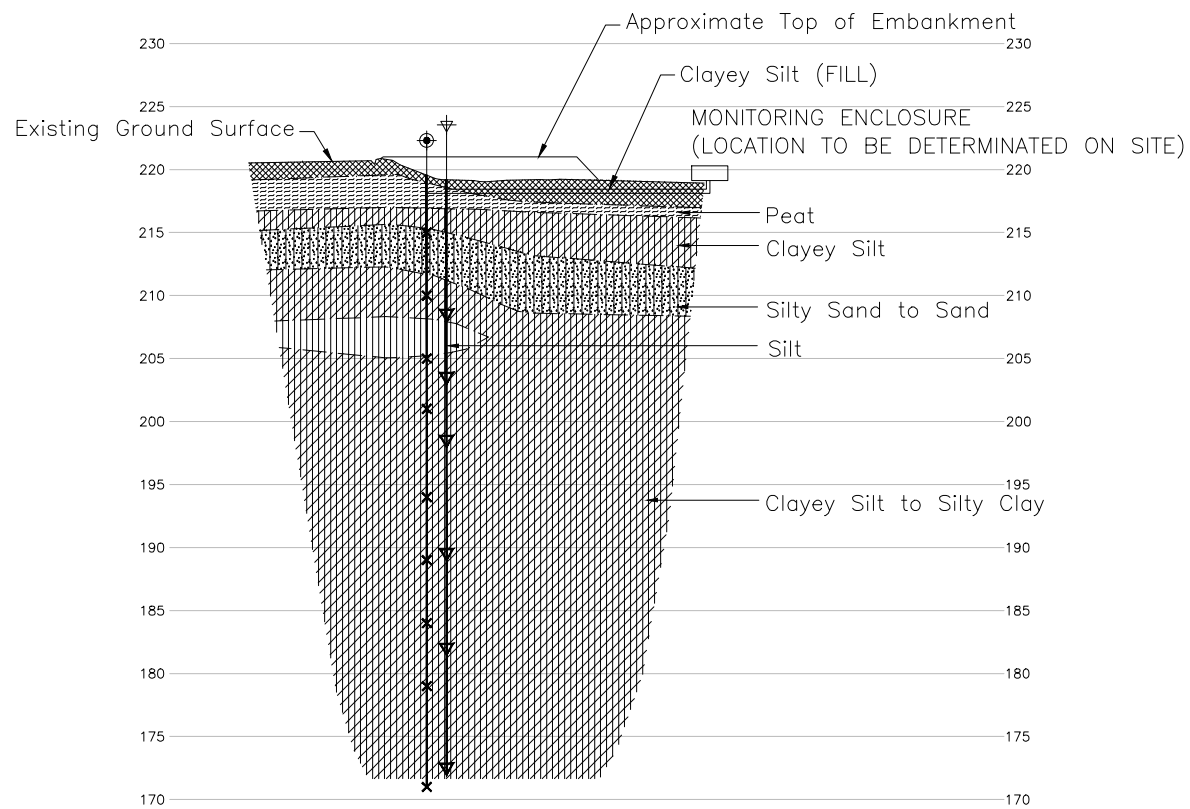
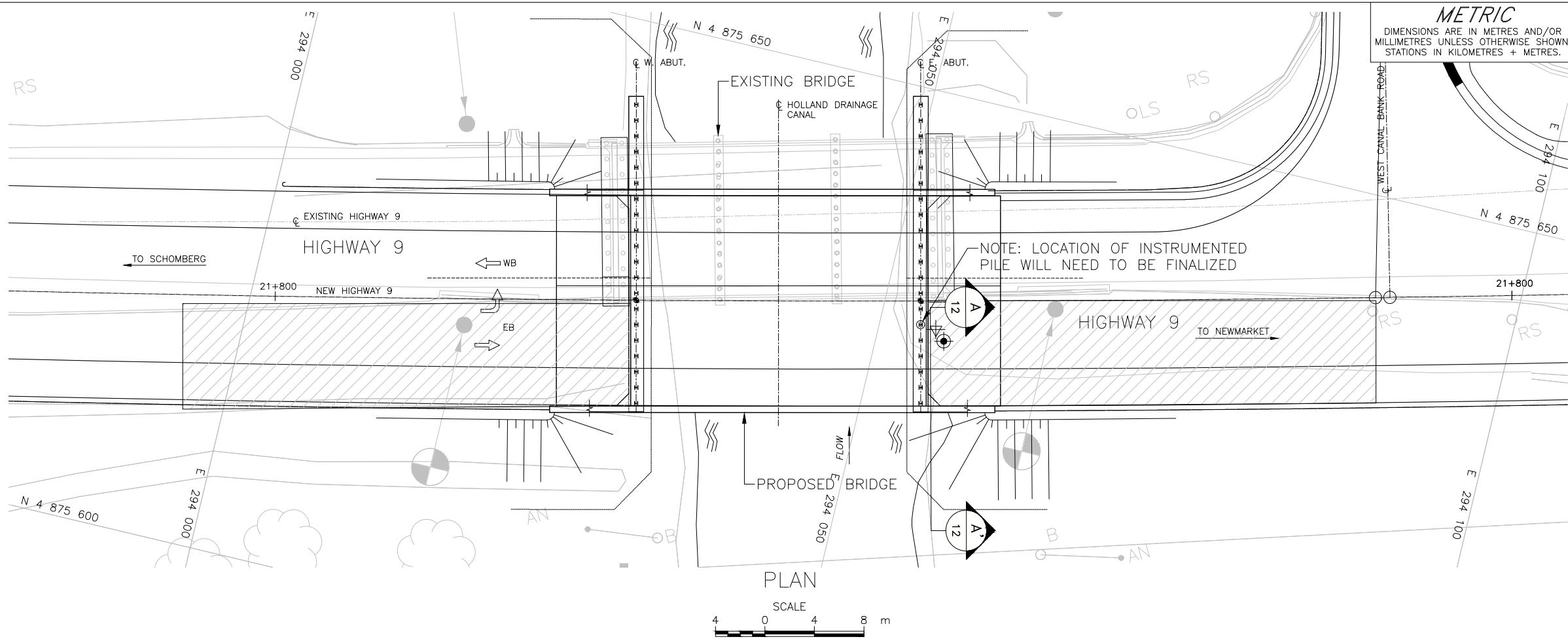
NOT TO SCALE



NOTES  
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NO.	DATE	BY	REVISION
1	12/13/2019	DD	1
Geocres No. 31D-734			
HWY. 9	PROJECT NO. 1671430	DIST.	
SUBM'D. MA	CHKD. MA	DATE: 12/13/2019	SITE:
DRAWN: DD	CHKD. MA/JMAC	APPD. JPD	DWG. 11





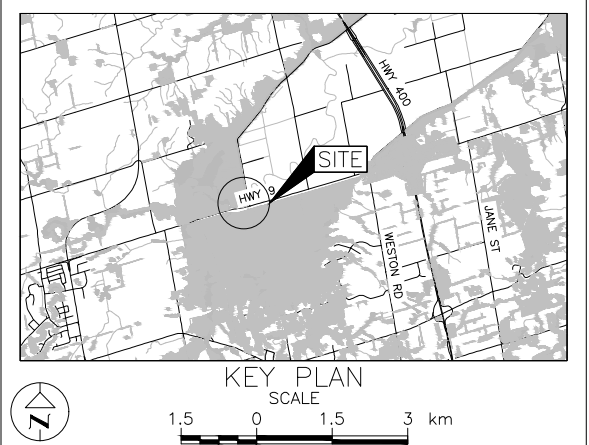
TYPICAL MONITORING SECTION  
A-A HIGHWAY 9 EAST ABUTMENT  
NOT TO SCALE

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

CONT No. 2020-2015  
GWP No. 2268-18-00

HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT  
EAST ABUTMENT H-PILE  
INSTRUMENTATION LOCATIONS

SHEET  
64



LEGEND

H

H-Pile Section 360 X 132

⊕

Instrumented Pile

⊙

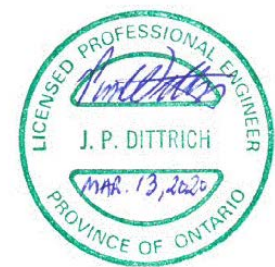
Multi-Point Borehole Extensometer

⊕

Multi-Level Vibrating Wire Piezometer

▨

Approximate Surcharge Area



**NOTES**

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

See Specifications for exact location, depth and number of instruments.

For subsurface information refer to Golder's Technical Memo dated April 30, 2019.

**REFERENCE**

Base plans and general arrangement provided in digital format by Aecom, drawing file nos. X-60570685-C-HWY9-BASE.dwg, C3D-60570685-Holland Canal Bridge\_37-31-OPT\_1-SSPAN\_B700.dwg, 60570685-C-ALI-HWY 9.dwg, b-074-009-1.dwg, OG 37-31.dwg, received May 24, 2019 and 39 GENERAL ARRANGEMENT.dwg, X-60570685-C2-CONSTR-HWY 9.dwg, received July 22, 2019.

Pile layout provided in digital format by Aecom, drawing file no. 04-PILE LAYOUT AND DETAILS.dwg, received December 09, 2019.

NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9		PROJECT NO. 1671430	DIST. .
SUBM'D. MA	CHKD. MA	DATE: 12/12/2019	SITE: .
DRAWN: DD	CHKD. MA/JMAC	APPD. JPD	DWG. 12

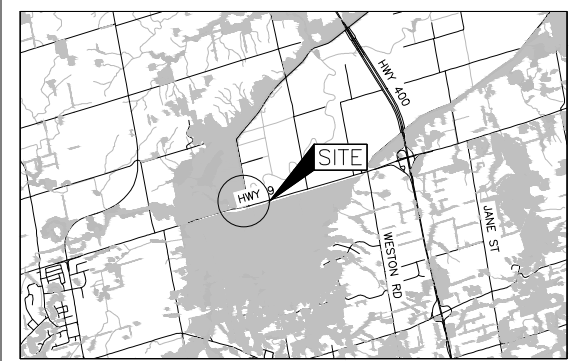


*METRIC*  
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MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

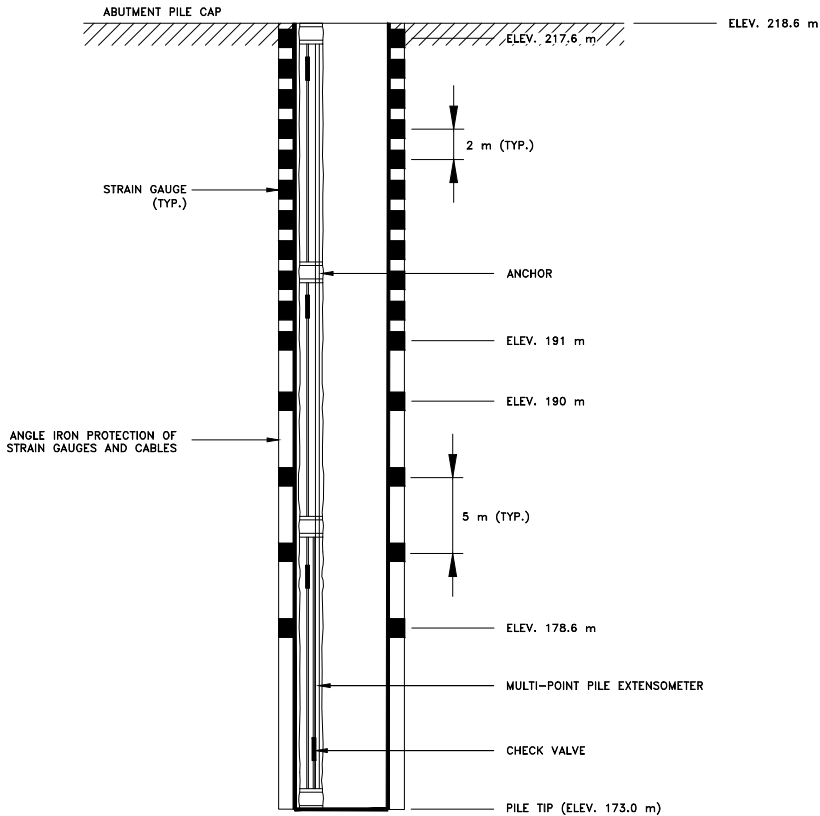
CONT No. 2020-2015  
GWP No. 2268-18-00

HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT  
TYPICAL H-PILE INSTRUMENTATION  
DETAILS

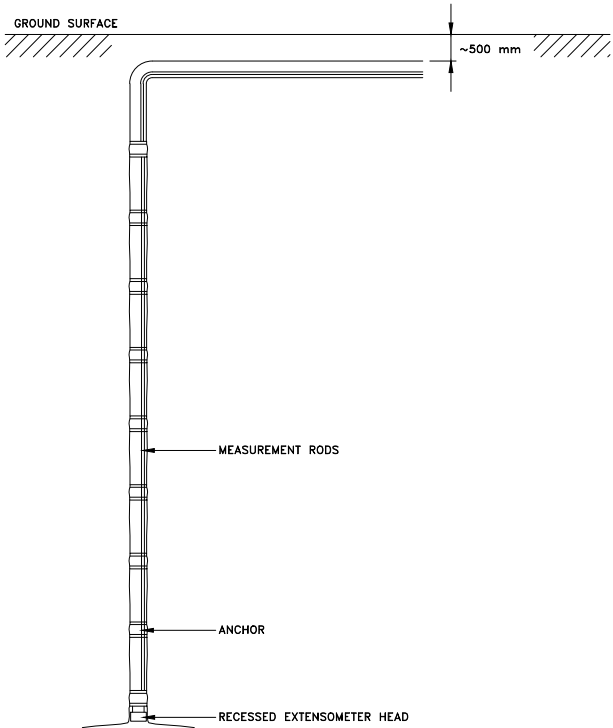
SHEET  
65



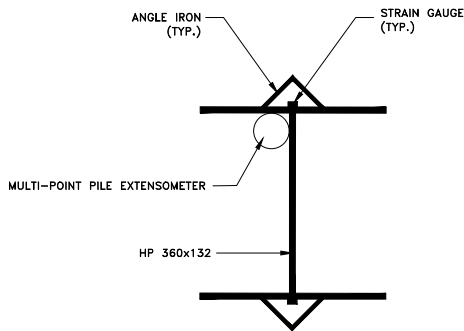
KEY PLAN  
SCALE  
1.5 0 1.5 3 km



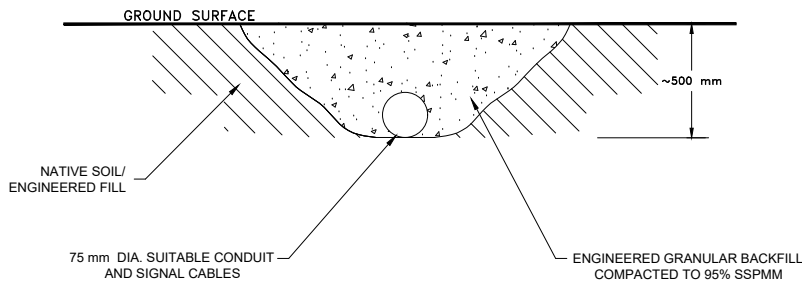
INSTRUMENTED PRODUCTION PILE  
NOT TO SCALE



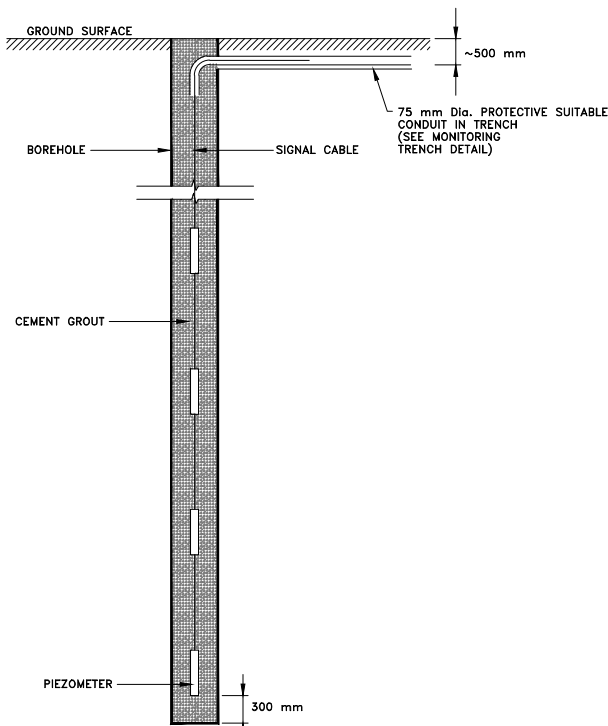
MULTI-POINT BOREHOLE EXTENSOMETER  
NOT TO SCALE



INSTRUMENTED PRODUCTION  
PILE SECTION (TYP.)  
NOT TO SCALE



MONITORING TRENCH  
NOT TO SCALE



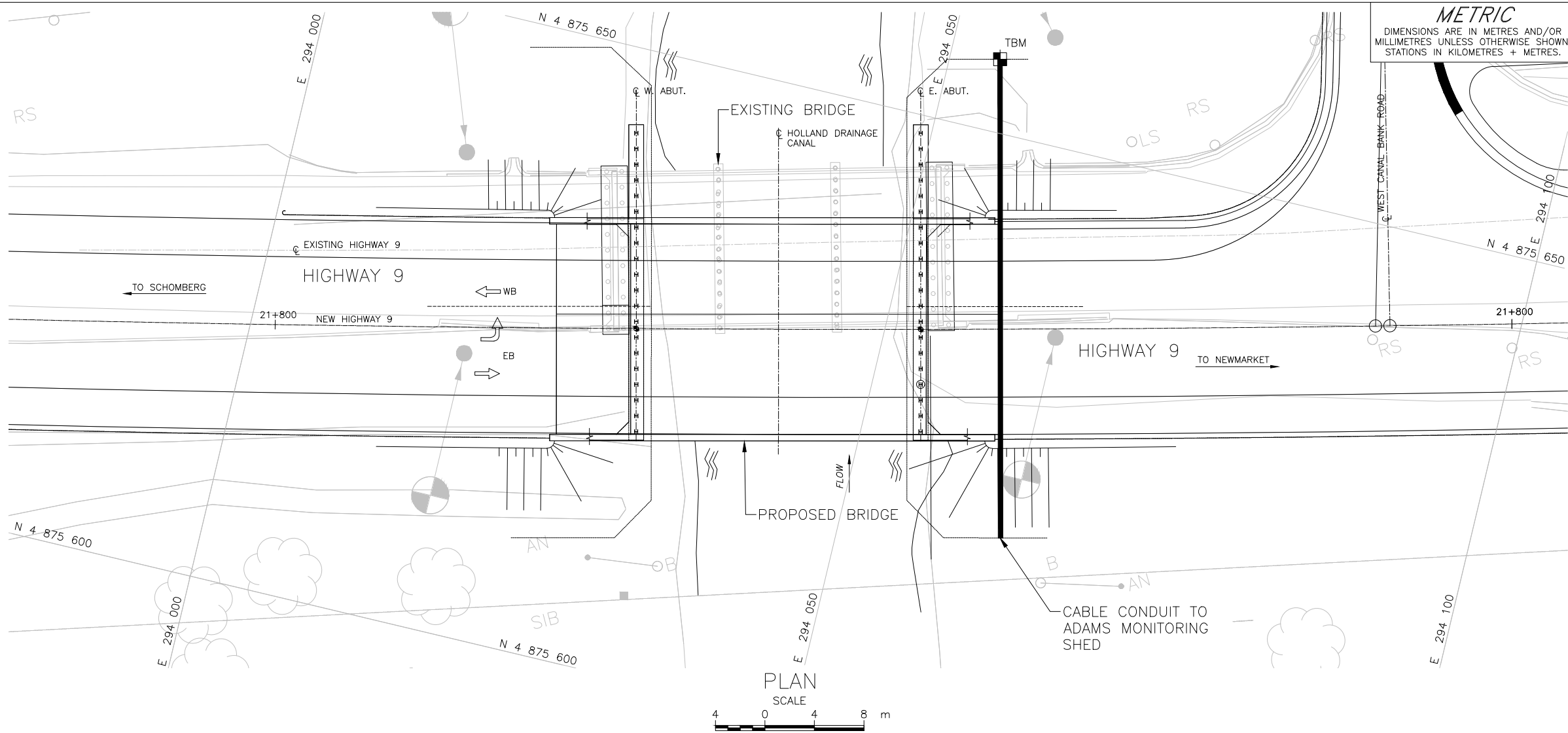
MULTI-LEVEL VIBRATING  
WIRE PIEZOMETER  
NOT TO SCALE



NOTES  
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NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9	PROJECT NO. 1671430	DIST. .	
SUBM'D. MA	CHKD. MA	DATE: 12/13/2019	SITE: .
DRAWN: DD	CHKD. MA/JMAC	APPD. JPD	DWG. 13



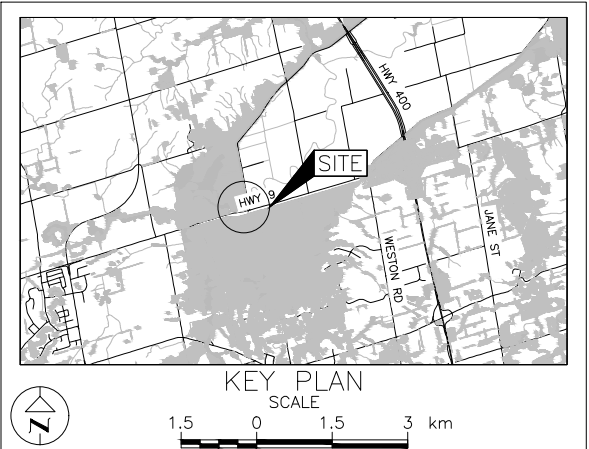


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No. 2020-2015  
GWP No. 2268-18-00

HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT  
DIFFERENTIAL SETTLEMENT MONITORING  
INSTRUMENTATION LOCATIONS

SHEET  
66



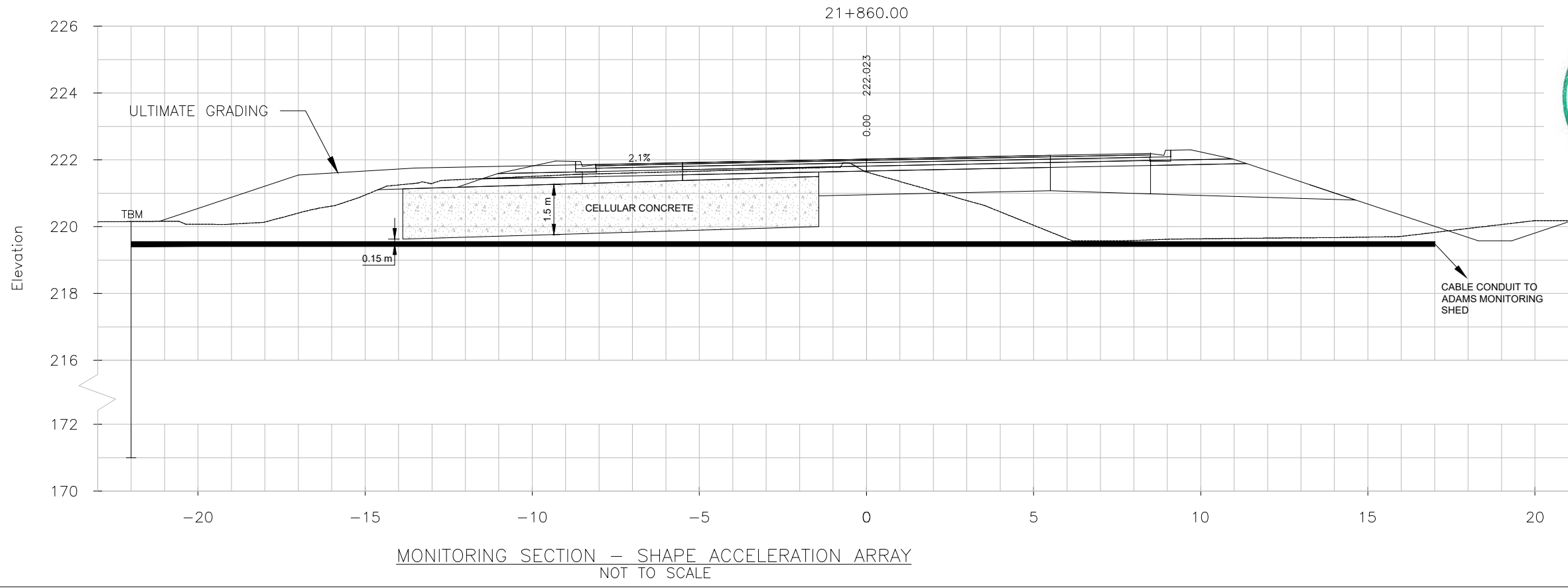
LEGEND

—

Approximate Location of Shape Acceleration Array (SAA)

■

TBM Temporary Benchmark



LICENSED PROFESSIONAL ENGINEER  
J. P. DITTRICH  
MAR. 13, 2020  
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER  
K. J. BENTLEY  
MAR. 13/20  
PROVINCE OF ONTARIO

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See Specifications for exact location, depth and number of instruments.

For subsurface information refer to Golder's Technical Memo dated April 30, 2019.

**REFERENCE**

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Pile layout provided in digital format by Aecom, drawing file no. 04-PILE LAYOUT AND DETAILS.dwg, received December 09, 2019.

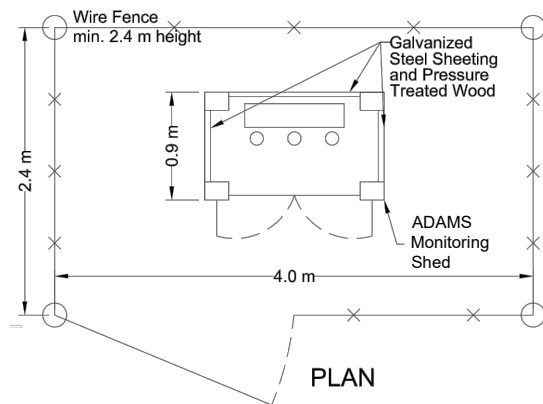
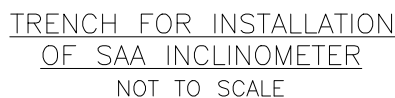
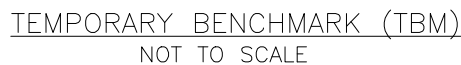
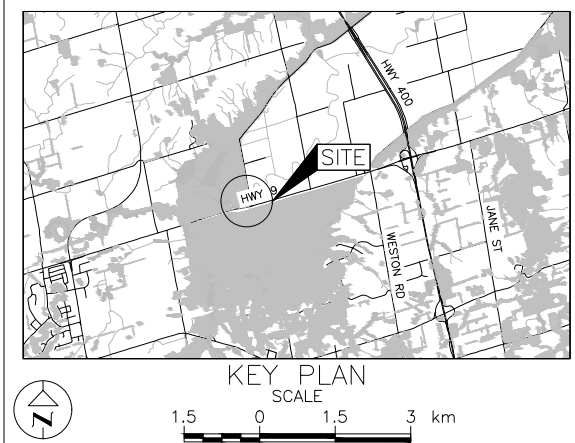
NO.	DATE	BY	REVISION
Geocres No. 31D-734			
HWY. 9	PROJECT NO. 1671430		DIST. .
SUBM'D. CC	CHKD. CC	DATE: 12/12/2019	SITE: .
DRAWN: JM	CHKD. JPD	APPD. JPD	DWG. 14



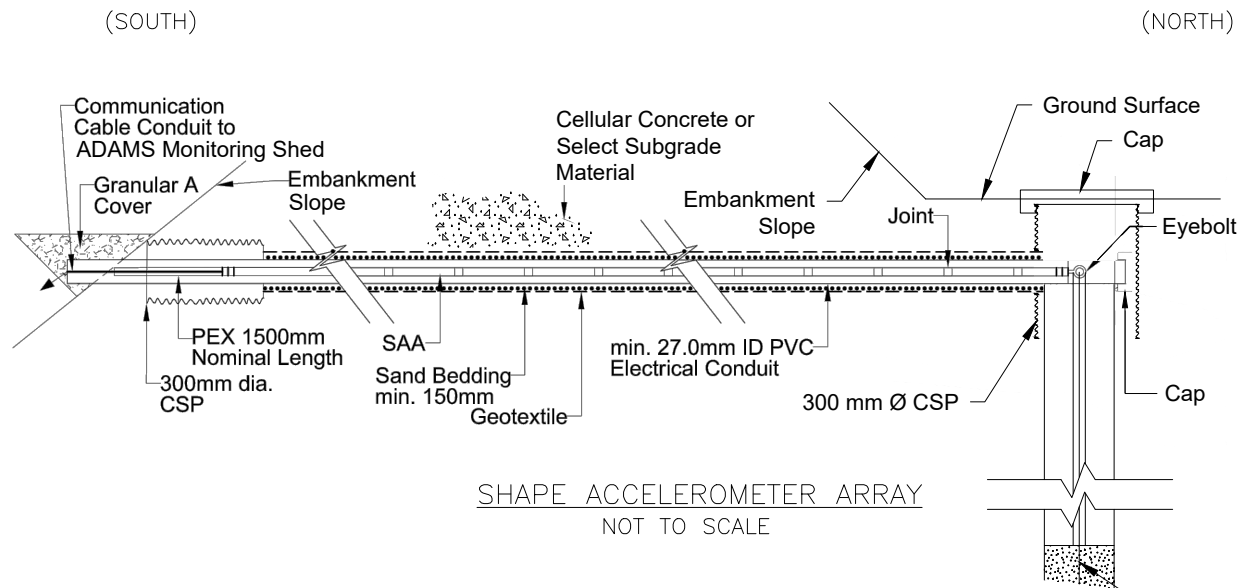
CONT No. 2020-2015  
GWP No. 2268-18-00

HIGHWAY 9  
HOLLAND DRAINAGE CANAL BRIDGE REPLACEMENT  
TYP. DIFFERENTIAL SETTLEMENT  
MONITORING INSTRUMENTATION DETAILS

SHEET  
67



MONITORING SHED FOR DATA MANAGEMENT  
AND WEB-BASED INTERFERENCE  
NOT TO SCALE



NOTES

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

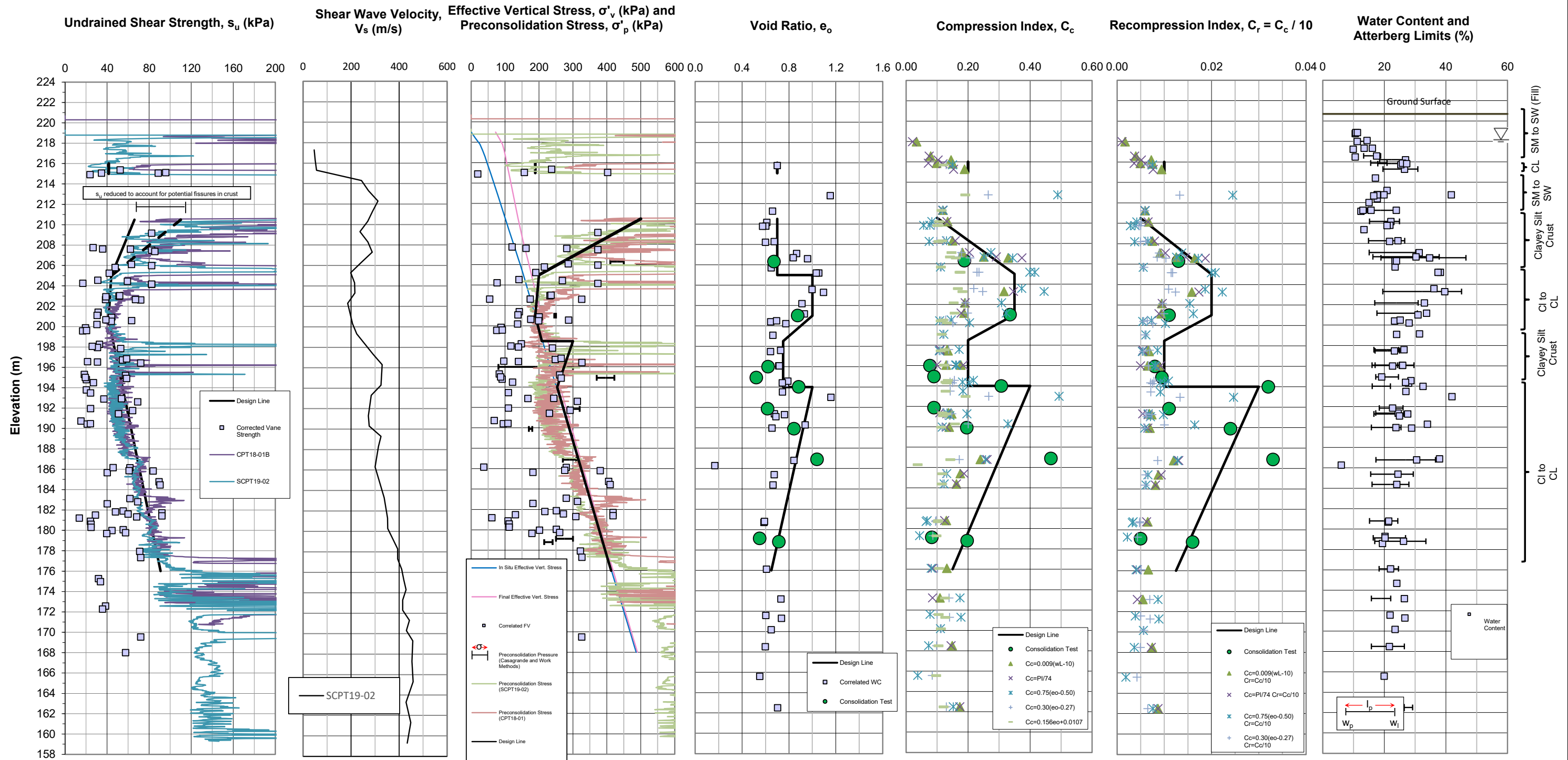
	-		-	
NO.	DATE	BY	REVISION	
Geocres No. 31D-734				
HWY. 9		PROJECT NO. 1671430		DIST. .
SUBM'D. CC	CHKD. CC	DATE: 12/13/2019	SITE:	
DRAWN: JM	CHKD. JPD	APPD. JPD	DWG. 15	



C:\Users\CCorish\Desktop\1671430 Highway 9 Plots of all parameters March 2020.xlsx All Plots - Final

SUMMARY PLOT OF ENGINEERING PARAMETERS FOR  
COHESIVE DEPOSITS  
Highway 9 - Holland Drainage Canal

FIGURE 1



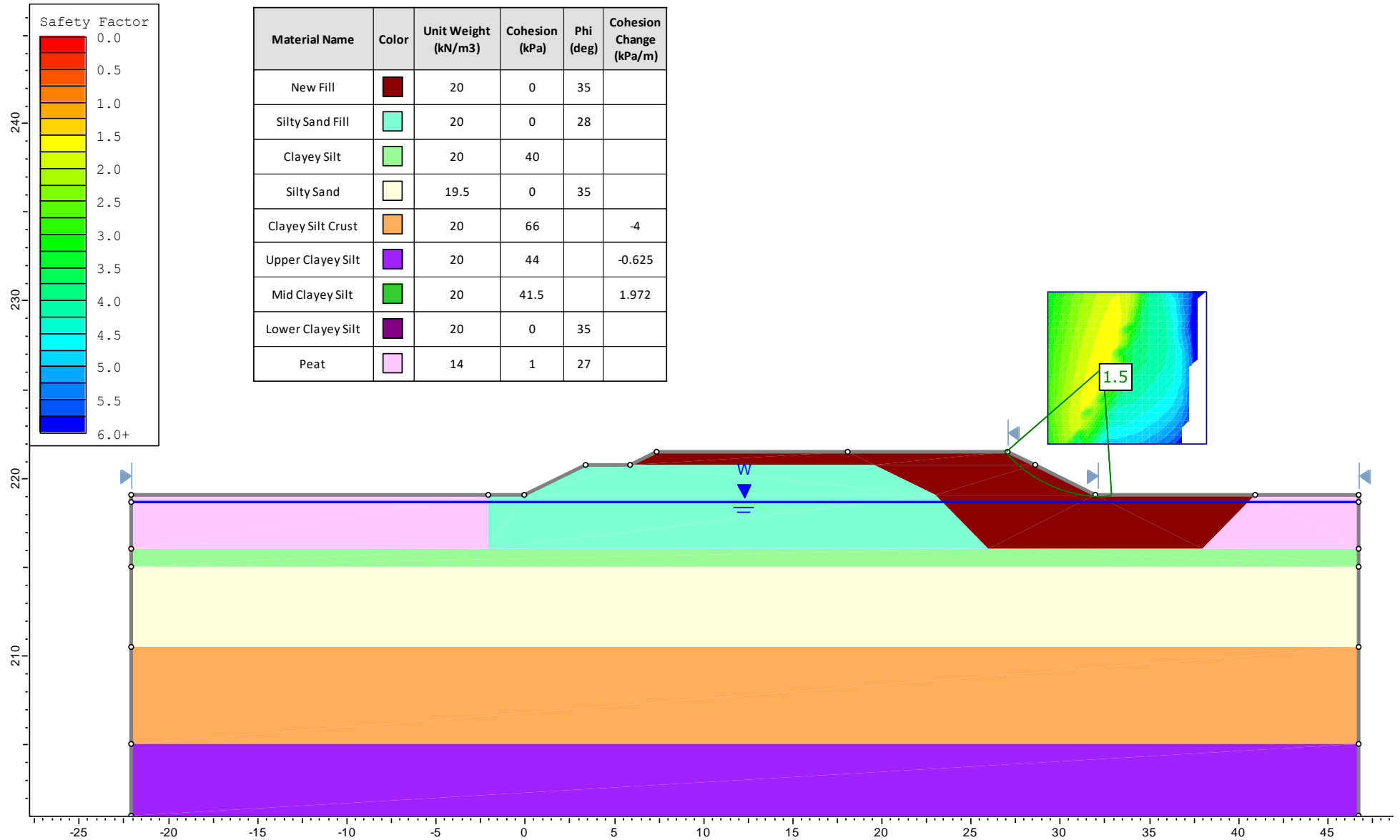




# Highway 9 – Holland Drainage Canal Site 37-31

## Static Slope Stability Analysis - Short Term (Undrained) Case

Figure 2



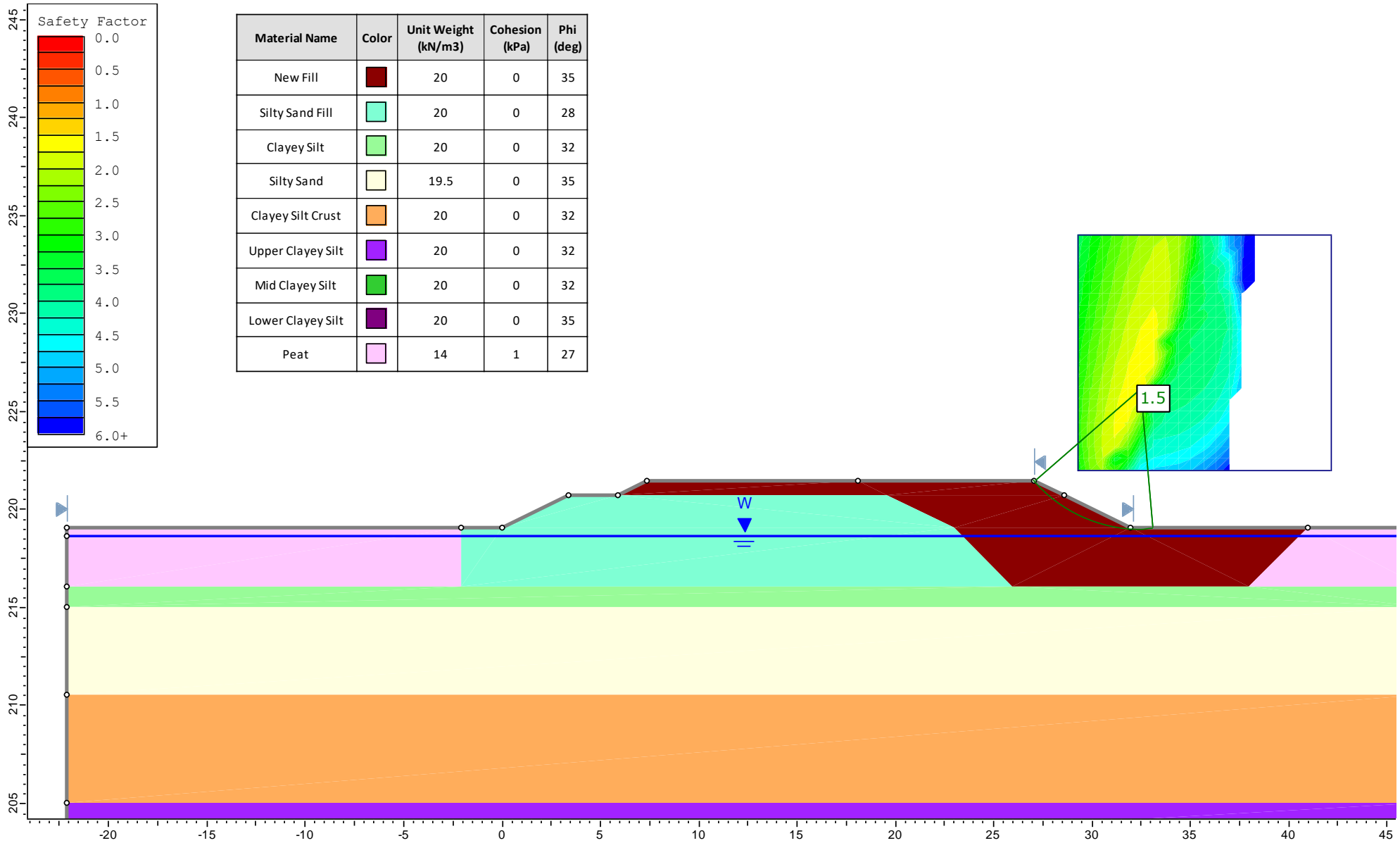




# Highway 9 – Holland Drainage Canal Site 37-31

## Static Slope Stability Analysis - Long Term (Drained) Case

Figure 3



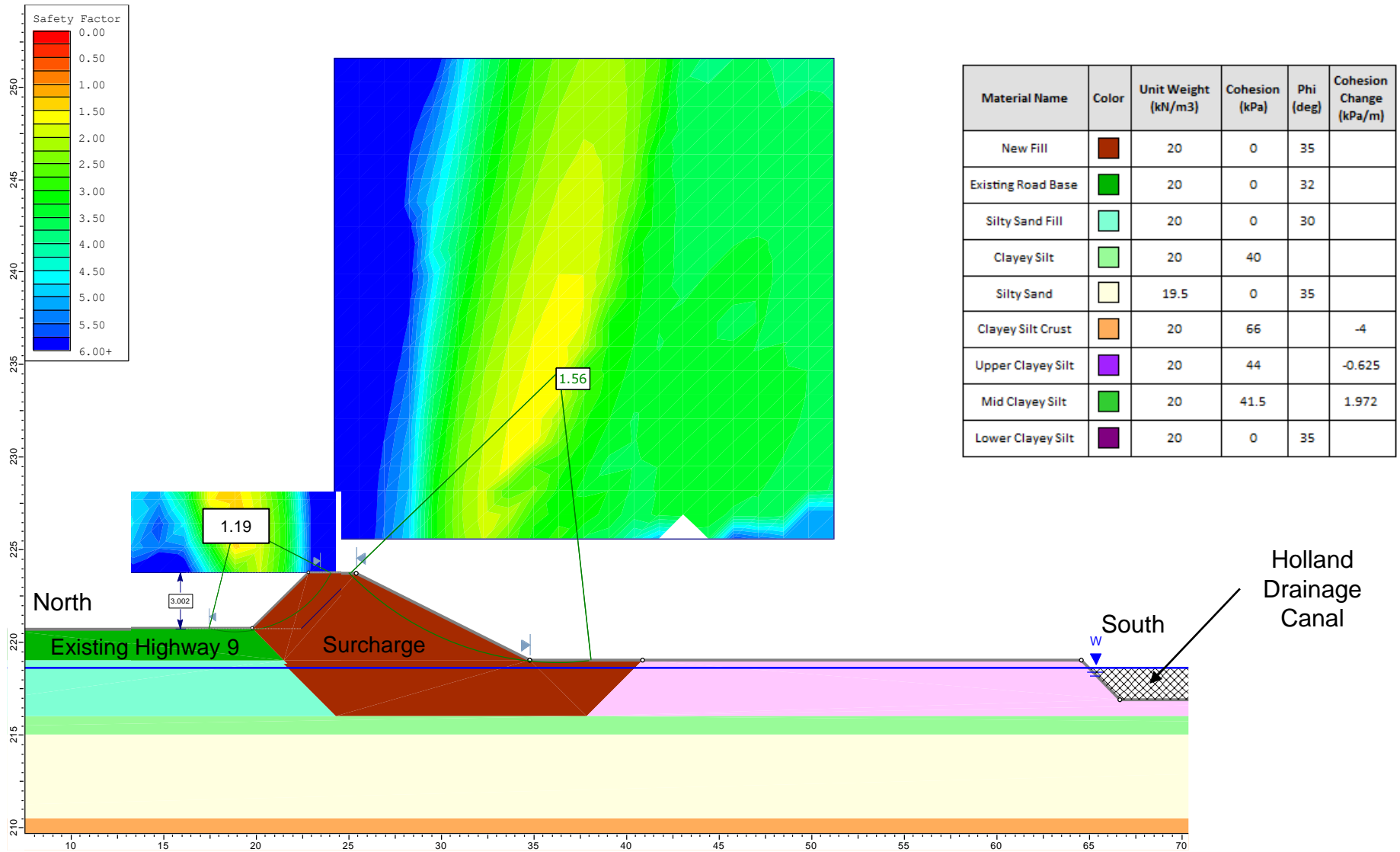




# Highway 9 – Holland Drainage Canal Site 37-31

Static Slope Stability Analysis - Temporary Surcharging - N/S Cross Section - Granular Fill  $\phi' = 35^\circ$

Figure 4



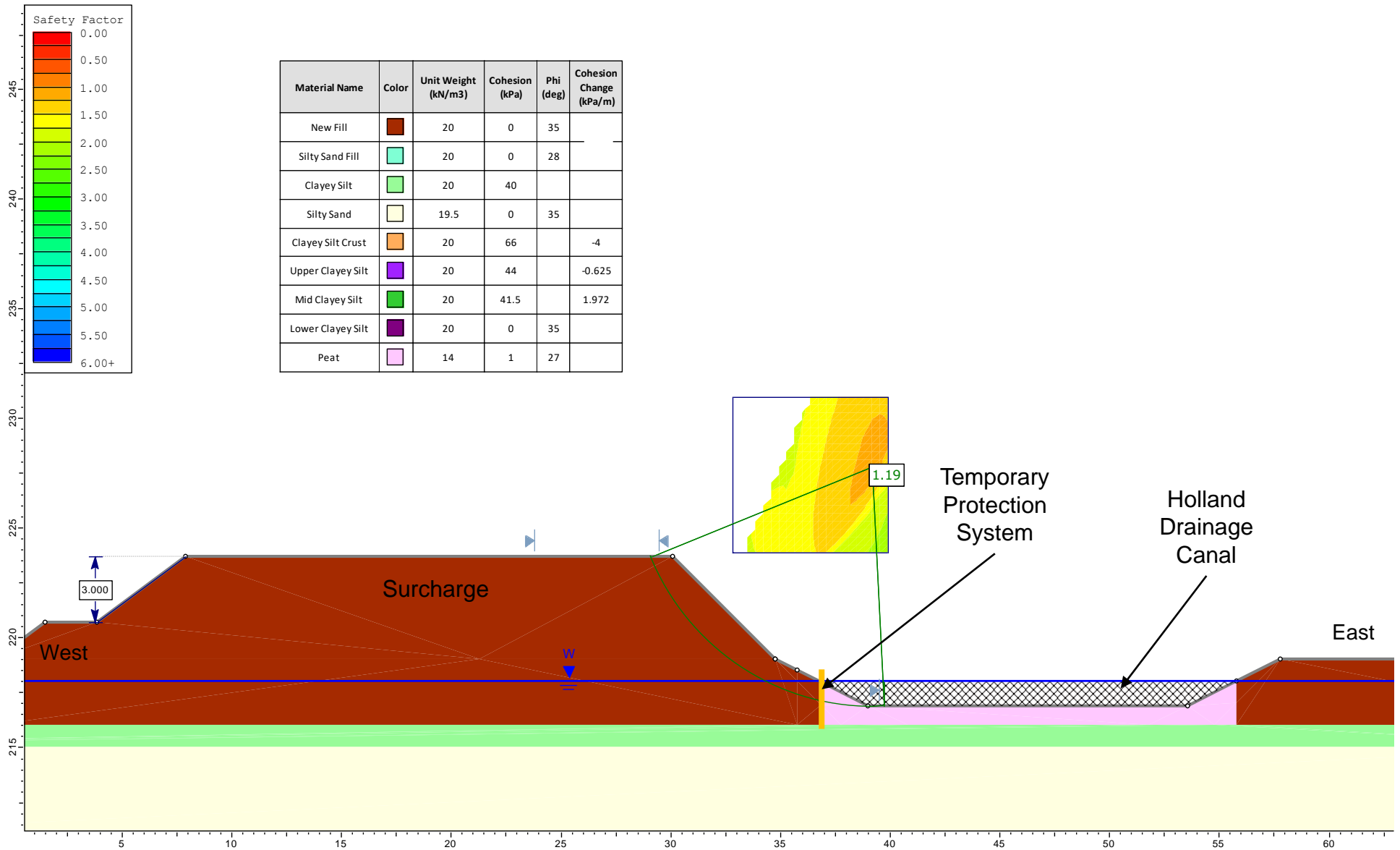




# Highway 9 – Holland Drainage Canal Site 37-31

Static Slope Stability Analysis - Temporary Surcharging - E/W Cross Section - Granular Fill  $\phi' = 35^\circ$

Figure 5



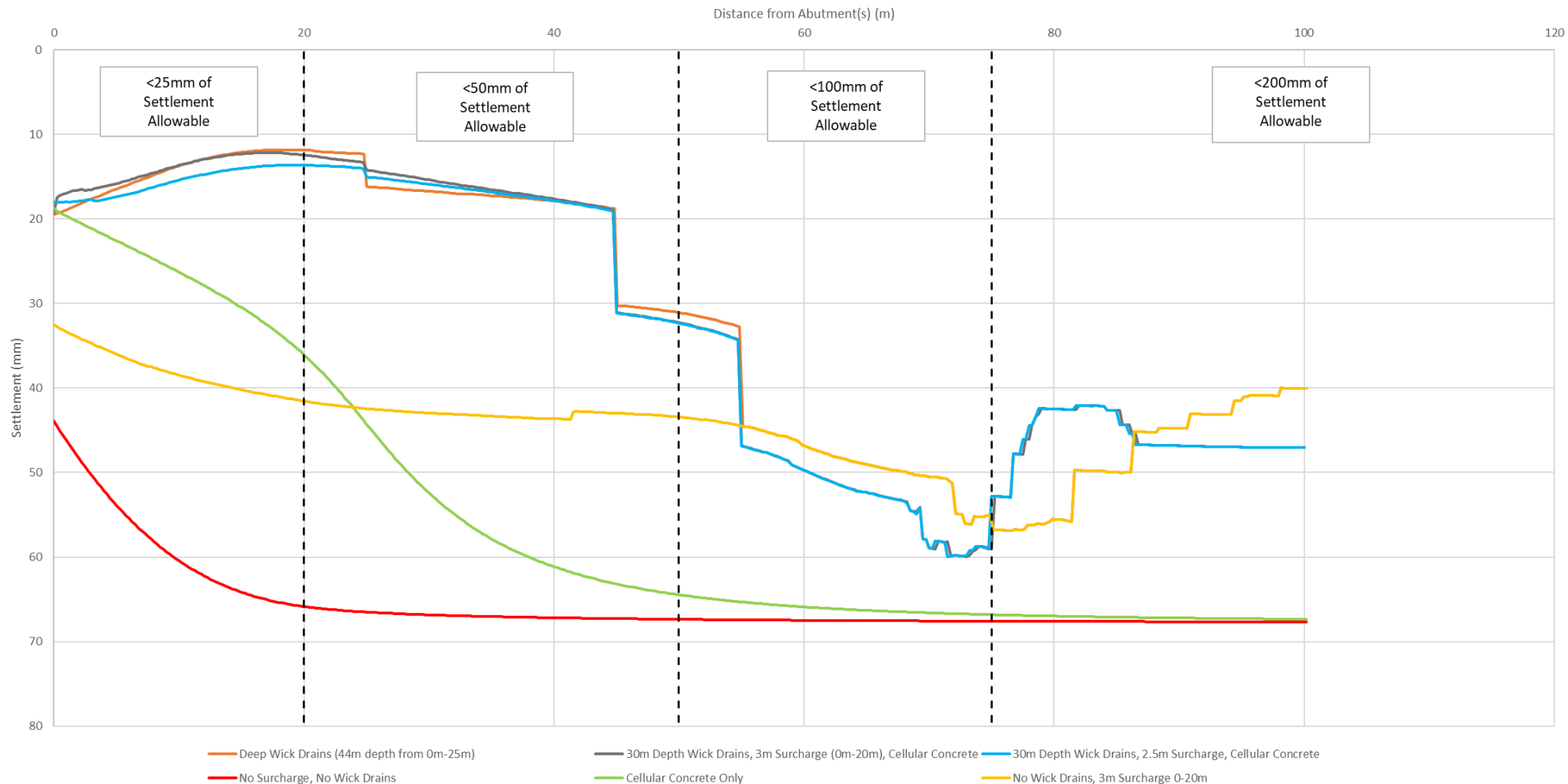




# Highway 9 – Holland Drainage Canal Site 32-31

## Total Estimated Settlement Of Various Settlement Mitigation Options in Embankment Widening Area

### Figure 6



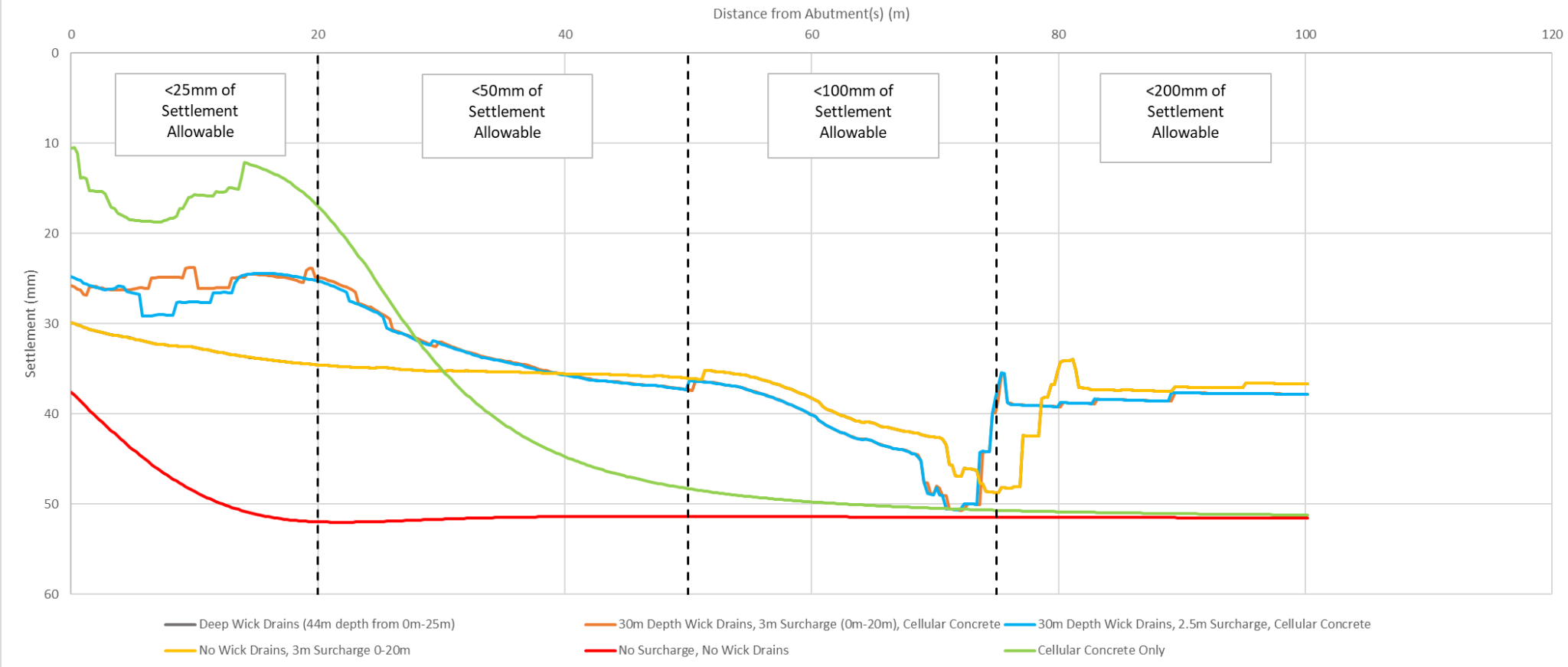




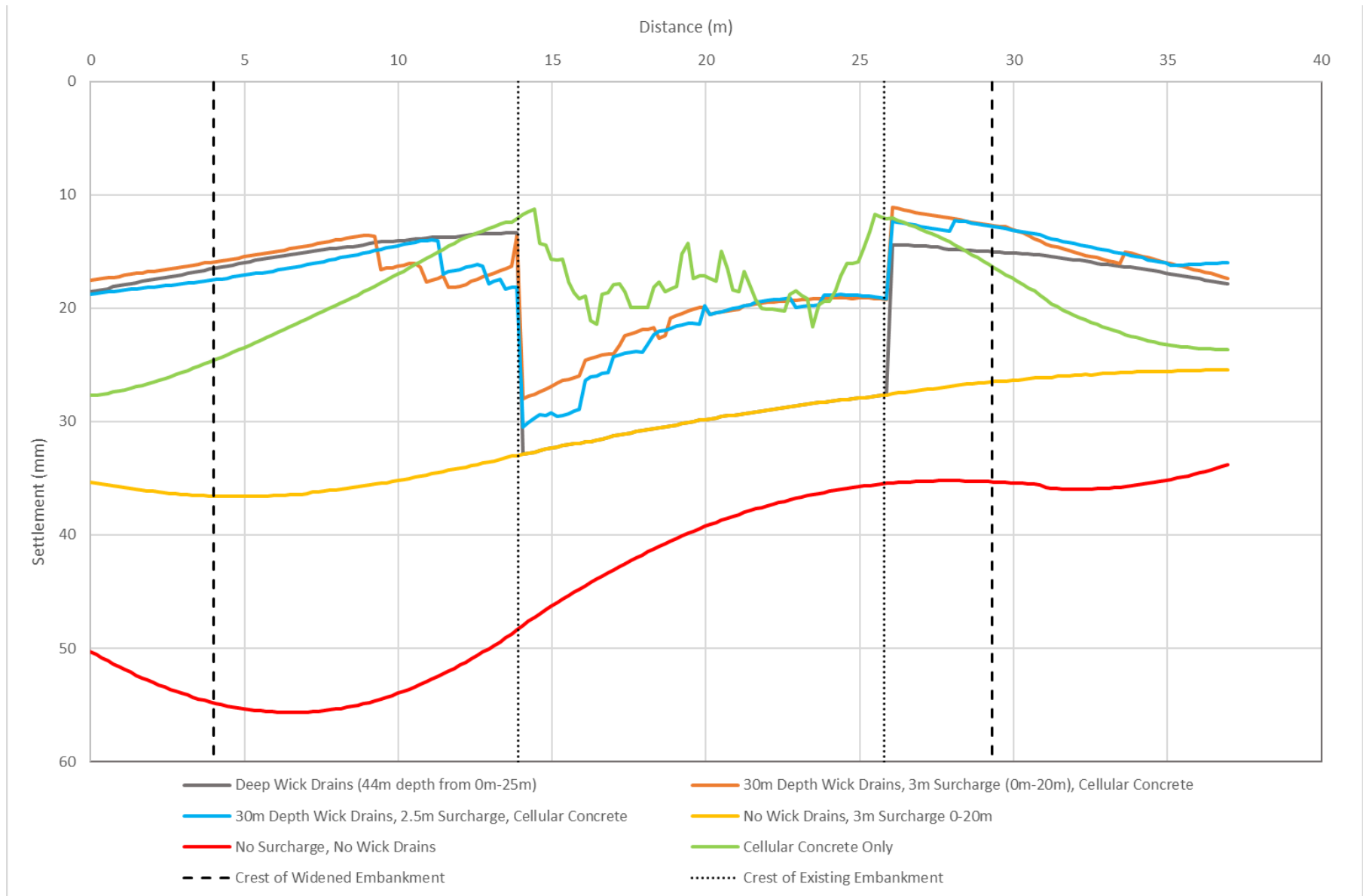
## Highway 9 – Holland Drainage Canal Site 32-31

### Total Estimated Settlement Of Various Settlement Mitigation Options in Embankment Grade Raise Area

Figure 7





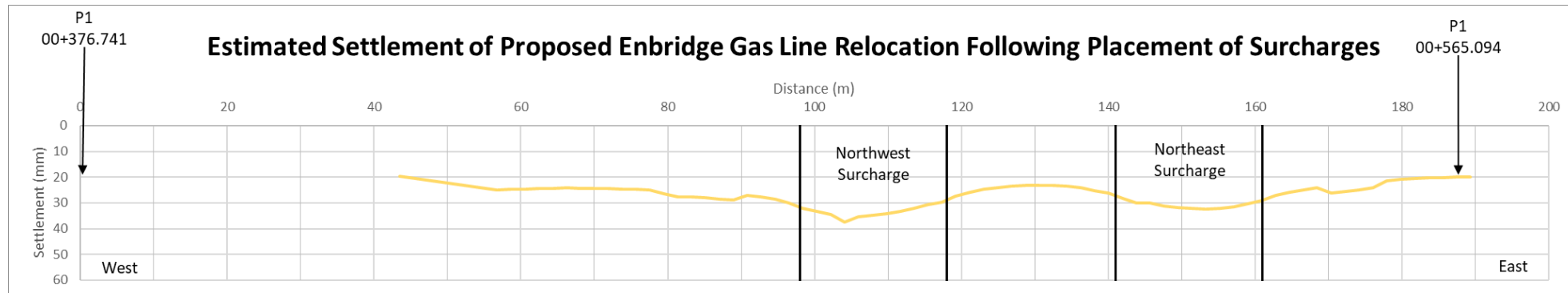






# Highway 9 – Holland Drainage Canal Site 32-31 Total Estimated Settlement Along Proposed Location of Enbridge Gas Line

Figure 9





**APPENDIX A**

**Borehole Records and Laboratory Test  
Results from 1965 Investigation  
(GEOCRES No. 31D-025)**



### FOUNDATION SECTION

CHECKED BY K. G.S. *gk*

[illegible]



DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 25-2

FOUNDATION SECTION

JOB 65-F-118 LOCATION East Pier - North Corner ORIGINATED BY P.P.  
W.P. 171-65 BORING DATE Nov. 1, 1965. COMPILED BY P.P.  
DATUM Geodetic BOREHOLE TYPE Cone. CHECKED BY K.G.S. *dl*

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BLOWS / FOOT	WATER CONTENT %			
218.9 0.0 m	718.1 0.0 Water level										
217.1 1.7 m	712.4 5.7 Water										
						710					
						700					
						690					
						680					
204.4 14.8 m	670.7 48.7 End of borehole.					670					



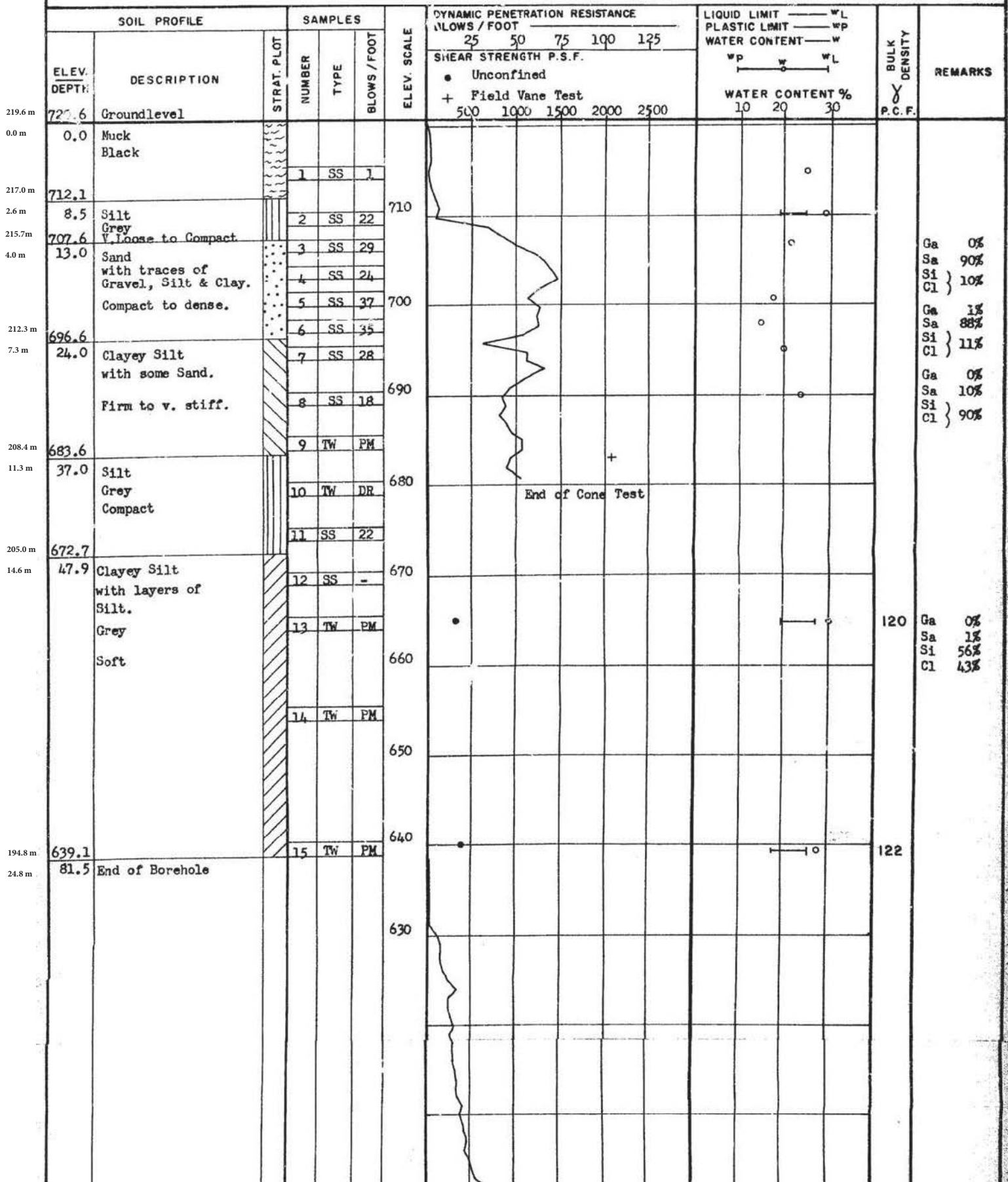
DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 25-3

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 65-E-118 LOCATION East Pier - South Corner ORIGINATED BY P.P.  
W.P. 171-65 BORING DATE Nov. 2 and 3, 1965. COMPILED BY P.P.  
DATUM Geodetic BOREHOLE TYPE Washbore - NX & BX Casings CHECKED BY K.G.S.





DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 25-4

FOUNDATION SECTION

JOB 65-R-118 LOCATION West Pier - South Corner ORIGINATED BY P.P.  
W.P. 171-65 BORING DATE November 4, 1965. COMPILED BY P.P.  
DATUM Geodetic BOREHOLE TYPE Washbore - NX Casing. CHECKED BY K.G.S. *AK*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		25	50	75	100	125	WP	WL	W		
219.5 m	720.2 Groundlevel															
0.0 m	0.0 Muck															
	Black		1	SS	-											
216.8 m	711.2															
2.7 m	9.0 Clayey silt to silt.		2	SS	14	710										Gr 0%
215.9 m	708.2 Grey.															Sa 7%
3.7 m	12.0 Sand with some gravel & traces of silt & clay. Brown. Compact.		3	SS	29											Si 75%
			4	SS	26											Cl 18%
			5	SS	32	700										Gr 0%
212.2 m	696.2															Sa 86%
7.3 m	24.0 Clayey silt		6	SS	23											Si) 14%
	Grey															Gr 19%
	Stiff		7	SS	12	690										Sa 71%
																Si) 10%
208.7 m	684.7		8	TW	PM										120.5	Gr 0%
10.8 m	35.5 Silty clay		9	TW	PM	680									113	Sa 1%
	Grey															Si 43%
	Soft		10	TW	PM	670										Cl 55%
			11	TW	PM	660										
197.7 m	648.7		12	SS	32	650										
21.8 m	71.5															

End of Cone Test



DEPARTMENT OF HIGHWAYS - ONTARIO		<b>RECORD OF BOREHOLE NO. 25-5</b>		FOUNDATION SECTION	
MATERIALS & TESTING DIVISION					
JOB <u>65-F-118</u>		LOCATION <u>West Abutment - North Corner</u>		ORIGINATED BY <u>P.P.</u>	
W.P. <u>171-65</u>		BORING DATE <u>Nov. 8 and 9, 1965</u>		COMPILED BY <u>P.P.</u>	
DATUM <u>Geodetic</u>		BOREHOLE TYPE <u>Washbore - NX Casing</u>		CHECKED BY <u>K.G.S.</u>	

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — WL PLASTIC LIMIT — wp WATER CONTENT — w			BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					WATER CONTENT %
							25	50	75	100	125	o Unconfined + Field Vane Test					
							500	1000	1500	2000	2500	wp	w	wL			
219.9 m	721.4	Groundlevel															
0.0 m	0.0	Muck				720											
		Black															
		V. soft		1	SS	-											
217.0 m	711.8																
2.9 m	9.6	Silt to Clayey Silt.		2	SS	3	710										
215.8 m	708.1	Grey		3	SS	24											
4.1 m	13.3	Sand															
		with some Silt and		4	SS	40											
		Clay, traces of		5	SS	35	700										
		Gravel.		6	SS	71											
		Greyish Brown		7	SS	23											
		Compact to v. Dense															
211.7 m	694.4																
8.2 m	27.0	Clayey Silt		8	SS	11	690										
		with seams of		9	TW	PM											
		Silt and Sand.															
		Grey															
		Stiff															
207.5 m	680.9			10	TW	PM	680										
12.3 m	40.5	Silt		11	TW	PM											
		with traces of															
205.2 m	673.1	Sand.															
14.7 m	48.3	Compact.		12	TW	PM	670										
		Silty Clay		13	TW	PM											
		Grey		14	SS	-	660										
		Soft to Firm															
198.1 m	649.9			15	SS	7	650										
21.8 m	71.5																

Ga 2%  
Sa 87%  
Si 11%  
Cl)

123

110



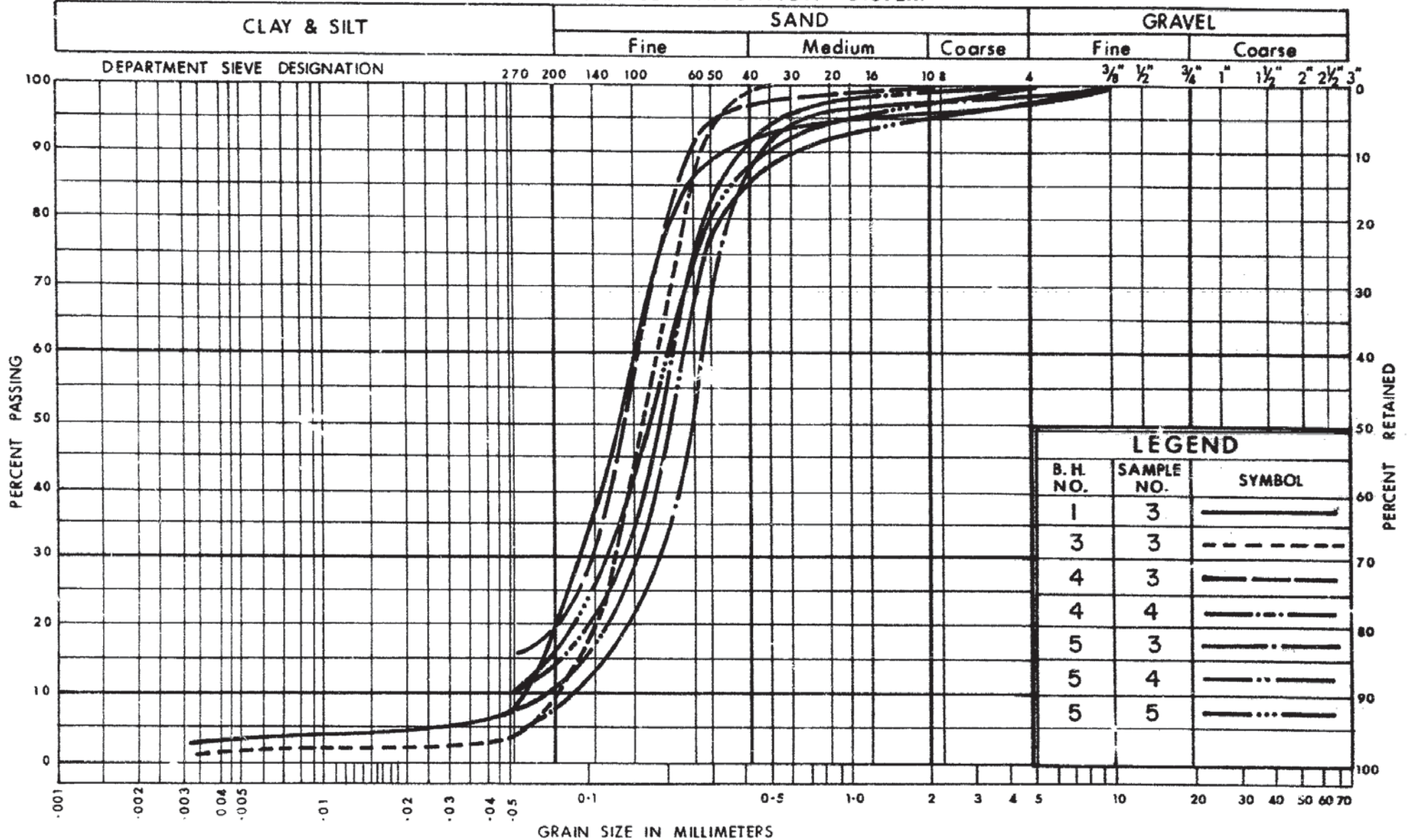
FOUNDATION SECTION

JOB 65-F-118 LOCATION East Abutment - South Corner. ORIGINATED BY P.P.  
W. P. 171-65 BORING DATE November 1, 1965. COMPILED BY P.P.  
DATUM Geodetic BOREHOLE TYPE Dynamic Cone Penetration CHECKED BY \_\_\_\_\_

[illegible]



# UNIFIED SOIL CLASSIFICATION SYSTEM



ONTARIO

DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

## GRAIN SIZE DISTRIBUTION

W.P. No. 171-65

JOB No. 65-F-118



**APPENDIX B**

**Borehole Records and CPT and SCPT Results  
from Current (2018-2019) Investigation**



## LIST OF SYMBOLS

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

### I. GENERAL

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

### II. STRESS AND STRAIN

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

### III. SOIL PROPERTIES

#### (a) Index Properties

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

#### (a) Index Properties (continued)

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

#### (b) Hydraulic Properties

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

#### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_{\alpha}$	secondary compression index
$m_v$	coefficient of volume change
$C_v$	coefficient of consolidation (vertical direction)
$C_h$	coefficient of consolidation (horizontal direction)
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

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

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

Notes: 1  
2

$\tau = c' + \sigma' \tan \phi'$   
shear strength = (compressive strength)/2



## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### Dynamic Cone Penetration Resistance; $N_d$ :

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

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

### III. SOIL DESCRIPTION

#### (a) Non-Cohesive (Cohesionless) Soils

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

#### (b) Cohesive Soils Consistency

	$C_u, S_u$	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

### V. MINOR SOIL CONSTITUENTS

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



## WEATHERINGS STATE

**Fresh:** no visible sign of weathering

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

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

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

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

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

## BEDDING THICKNESS

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

## JOINT OR FOLIATION SPACING

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

## GRAIN SIZE

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

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

## CORE CONDITION

### Total Core Recovery (TCR)

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

### Solid Core Recovery (SCR)

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

### Rock Quality Designation (RQD)

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

## DISCONTINUITY DATA

### Fracture Index

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

### Dip with Respect to Core Axis

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

### Description and Notes

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

### Abbreviations

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



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



PROJECT <u>1671430 (W0007)</u>		<b>RECORD OF BOREHOLE No A1-1</b>		SHEET 2 OF 4		<b>METRIC</b>	
G.W.P. <u>2266-18-00</u>		LOCATION <u>N 4875633.5; E 294019.4 MTM NAD 83 ZONE 10 (LAT. 44.020618; LONG. -79.634475)</u>		ORIGINATED BY <u>JP</u>			
DIST <u>CENTRAL</u> HWY <u>9</u>		BOREHOLE TYPE <u>CME 55 Track-mounted Drill Rig</u>		COMPILED BY <u>KN</u>			
DATUM <u>Geodetic</u>		DATE <u>January 21 to 24, February 26 and March 13, 2019</u>		CHECKED BY <u>NK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	W <sub>p</sub> W W <sub>L</sub>						
--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100	20 40 60 80 100	10 20 30						
<div>CLAYEY SILT, trace sand Soft to stiff Grey Moist to wet</div> <div>- No recovery in Shelby tube pushed from a depth of 21.3 m to 22.0 m</div>			12	SS	WH		205								
							204	3.2 + 2.5 +							
							203								
				13	SS	WH		202							
							201	5.8 + 3.7 +							
				-	TO	PH		200							
							199								
							198								
							197								
				15	SS	WR		196							
							195	4.2 + 3.5 +							
							194								
			TO1	TO	PH		193								
						192									
						191									


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PROJECT <u>1671430 (W0007)</u>		<b>RECORD OF BOREHOLE No A1-1</b>		SHEET 3 OF 4		<b>METRIC</b>	
G.W.P. <u>2266-18-00</u>		LOCATION <u>N 4875633.5; E 294019.4 MTM NAD 83 ZONE 10 (LAT. 44.020618; LONG. -79.634475)</u>		ORIGINATED BY <u>JP</u>			
DIST <u>CENTRAL</u> HWY <u>9</u>		BOREHOLE TYPE <u>CME 55 Track-mounted Drill Rig</u>		COMPILED BY <u>KN</u>			
DATUM <u>Geodetic</u>		DATE <u>January 21 to 24, February 26 and March 13, 2019</u>		CHECKED BY <u>NK</u>			

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							WATER CONTENT (%)	
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED							
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100		10 20 30						
	CLAYEY SILT, trace sand Soft to stiff Grey Moist to wet		16	SS	WR			190						0 2 64 34		
									189							
									188							
									187							
			18	SS	WH					186			○			
										185						
										184						
			TO2	TO	PH					183						
										182						
										181					0 2 64 34	
										180						
										179						
							178									
			21	SS	WH			177								
								176								

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



PROJECT		2266-18-00		LOCATION		N 4875612.3; E 294032.6 MTM NAD 83 ZONE 10 (LAT. 44.020427; LONG. -79.634310)		ORIGINATED BY		SK									
DIST		CENTRAL HWY 9		BOREHOLE TYPE		CME 55 Track-mounted Drill Rig		COMPILED BY		KN									
DATUM		Geodetic		DATE		November 12 to 15 and 19, 2018		CHECKED BY		NK									
SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	20			40	60	80	100						20	40
219.3	GROUND SURFACE																		
0.0	TOPSOIL (130 mm)																		
0.1	Silt and sand, some clay, trace gravel (FILL) Very loose to loose Brown Moist		1	SS	7														
			2	SS	3														
217.9	PEAT, some silt Soft Brown Moist		3	SS	2														
1.5			4	SS	3														
216.1	CLAYEY SILT, trace sand Soft to firm Grey Wet		5A	SS	2														
3.3			5B	SS	2														
215.3	Silty SAND, trace clay Loose to dense Grey Wet		6A	SS	6														
4.0			6B	SS	6														
			7	SS	14														
			8	SS	29														
			9	SS	35														
210.6	CLAYEY SILT, trace sand Firm to very stiff Grey Wet		10	SS	9														
8.7																			
			11	SS	4														
			12	SS	WH														
			13	SS	-														
	- No recovery in Shelby tube pushed from a depth of 13.7 m to 14.3 m. A split-spoon was pushed from a depth of 13.7 m to 14.3 m to retrieve a disturbed sample.																		

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PROJECT 1671430		<b>RECORD OF BOREHOLE No A1-2</b>		SHEET 2 OF 5		<b>METRIC</b>	
G.W.P. 2266-18-00		LOCATION N 4875612.3; E 294032.6 MTM NAD 83 ZONE 10 (LAT. 44.020427; LONG. -79.634310)		ORIGINATED BY SK			
DIST CENTRAL HWY 9		BOREHOLE TYPE CME 55 Track-mounted Drill Rig		COMPILED BY KN			
DATUM Geodetic		DATE November 12 to 15 and 19, 2018		CHECKED BY NK			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			W <sub>P</sub>	W	W <sub>L</sub>		GR	SA	SI	CL			
--- CONTINUED FROM PREVIOUS PAGE ---								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%)										
	CLAYEY SILT, trace sand Firm to very stiff Grey Wet						204														
							203			3.5 + 4.4 +											
			14	SS	WH		202										0	0	47	53	
							201														
							200			1.5 + 2.8 +											
							199														
			TO1	TO	PH		198			2.2 + 2.2 +											
			16	SS	WR		197											0	0	67	33
							196			2.8 + 2.0 +											
							195			1.7 + 1.4 +											
			18	SS	WH		194														
							193			2.3 + 2.0 +											
							192														
							191														
			19	SS	WR		190														

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PROJECT <u>1671430</u>		<b>RECORD OF BOREHOLE No A1-2</b>		SHEET 3 OF 5		<b>METRIC</b>	
G.W.P. <u>2266-18-00</u>		LOCATION <u>N 4875612.3; E 294032.6 MTM NAD 83 ZONE 10 (LAT. 44.020427; LONG. -79.634310)</u>		ORIGINATED BY <u>SK</u>			
DIST <u>CENTRAL</u> HWY <u>9</u>		BOREHOLE TYPE <u>CME 55 Track-mounted Drill Rig</u>		COMPILED BY <u>KN</u>			
DATUM <u>Geodetic</u>		DATE <u>November 12 to 15 and 19, 2018</u>		CHECKED BY <u>NK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	w <sub>p</sub>			w	w <sub>L</sub>	
	--- CONTINUED FROM PREVIOUS PAGE ---																
	CLAYEY SILT, trace sand Firm to very stiff Grey Wet																
			20	SS	WH												
			TO2	TO	PH												
			21	SS	WR												
			22	SS	WH												

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PROJECT		1671430		RECORD OF BOREHOLE No A1-2				SHEET 5 OF 5				METRIC					
G.W.P.		2266-18-00		LOCATION		N 4875612.3; E 294032.6 MTM NAD 83 ZONE 10 (LAT. 44.020427; LONG. -79.634310)				ORIGINATED BY SK							
DIST		CENTRAL HWY 9		BOREHOLE TYPE		CME 55 Track-mounted Drill Rig				COMPILED BY KN							
DATUM		Geodetic		DATE		November 12 to 15 and 19, 2018				CHECKED BY NK							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
								20	40	60	80	100					
157.7	CLAYEY SILT, trace sand Firm to very stiff Grey Wet		31	SS	15		159										
61.6	END OF BOREHOLE						158										
	NOTE:  1. Borehole was advanced using 203 mm O.D. hollow-stem augers to a depth of 5.2 m below ground surface and 127 mm O.D. casing below a depth of 5.2 m below ground surface to end of borehole.  2. Water level measured in open borehole at a depth of 2.6 m below ground surface (Elev. 216.7 m) when borehole was at a depth of 4.6 m below ground surface prior to adding drilling fluid.  3. Water level measured in open borehole at ground surface upon completion of drilling. The water level measurement is not considered to be representative of the ground water level due to introduction of drilling fluid during wash-boring operation.																






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PROJECT <u>1671430</u>		<b>RECORD OF BOREHOLE No A2-1</b>		SHEET 2 OF 4		<b>METRIC</b>	
G.W.P. <u>2266-18-00</u>		LOCATION <u>N 4875646.1; E 294058.4 MTM NAD 83 ZONE 10 (LAT. 44.020732; LONG. -79.633989)</u>		ORIGINATED BY <u>KN</u>			
DIST <u>CENTRAL</u> HWY <u>9</u>		BOREHOLE TYPE <u>CME 75 Track-mounted Drill Rig</u>		COMPILED BY <u>KN</u>			
DATUM <u>Geodetic</u>		DATE <u>November 20 to 23, December 19 to 20, 2018</u>		CHECKED BY <u>NK</u>			

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					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>					
--- CONTINUED FROM PREVIOUS PAGE ---																				
	SILTY CLAY to CLAYEY SILT, trace sand, trace gravel Soft to very stiff Grey Wet - No recovery in Shelby tube pushed from a depth of 15.2 m to 15.9 m		-	TO	PH		205													
									204				2.6 +							
					14	SS	1											○		
									203				3.4 +							
					TO2	TO	PH		202											
									201				2.6 + 2.4 +							
					16	SS	WH		200								○			
					TO3	TO	PH		199											
									198				3.1 +							
					18	SS	6		197								1	1		
																	69	29		
					TO4	TO	PH		196											
							195				1.4 +									
			20	SS	WH		194								○					
			TO5	TO	PH		193				1.5 + 1.8 +									
							192				3.2 +									
			22	SS	WR		191													

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PROJECT 1671430		RECORD OF BOREHOLE No A2-1		SHEET 3 OF 4		METRIC							
G.W.P. 2266-18-00		LOCATION N 4875646.1; E 294058.4 MTM NAD 83 ZONE 10 (LAT. 44.020732; LONG. -79.633989)		ORIGINATED BY KN									
DIST CENTRAL HWY 9		BOREHOLE TYPE CME 75 Track-mounted Drill Rig		COMPILED BY KN									
DATUM Geodetic		DATE November 20 to 23, December 19 to 20, 2018		CHECKED BY NK									
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES						SHEAR STRENGTH kPa
	--- CONTINUED FROM PREVIOUS PAGE ---					20 40 60 80 100	○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED	10 20 30				
	SILTY CLAY to CLAYEY SILT, trace sand, trace gravel Soft to very stiff Grey Wet		TO6	TO	-	190	2.8					C 18.9	
						189							
						188							
			24	SS	WH	187							0 0 42 58
						186	2.8	3.5					
						185							
			25	SS	WR	184							
						183	2.2	2.2					
	- No recovery in Shelby tube pushed from a depth of 38.1 m to 38.7 m		-	TO	-	182	1.4	1.8					
			26	SS	9	181							
						180	2.2	2.1					
			TO7	TO	PH	179						C 20.7 C(VTO) 20.9 Perm	
						178	2.1						
						177	2.3						
			27	SS	44	176							0 1 78 21

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PROJECT		RECORD OF BOREHOLE No A2-1				SHEET 4 OF 4		METRIC										
1671430																		
G.W.P. 2266-18-00		LOCATION N 4875646.1; E 294058.4 MTM NAD 83 ZONE 10 (LAT. 44.020732; LONG. -79.633989)				ORIGINATED BY KN												
DIST CENTRAL HWY 9		BOREHOLE TYPE CME 75 Track-mounted Drill Rig				COMPILED BY KN												
DATUM Geodetic		DATE November 20 to 23, December 19 to 20, 2018				CHECKED BY NK												
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										
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100					10 20 30 WATER CONTENT (%)						
	SILTY CLAY to CLAYEY SILT, trace sand, trace gravel Soft to very stiff Grey Wet					175												
						174												
						173												
						172												
				28	SS	5	171											
							170											
168.2	END OF BOREHOLE		29	SS	19	169										0 1 64 35		
52.4	NOTE:  1. Borehole was advanced using 203 mm O.D. hollow-stem augers to a depth of 10.2 m below ground surface and 127 mm O.D. casing below 10.2 m below ground surface to end of borehole.  2. Open borehole dry before adding drilling fluid at a depth of 4.6 m below ground surface.  3. Water level measured in open borehole at a depth of 1.1 m below ground surface (Elev. 219.5 m) upon completion of drilling. The water level measurement is not considered to be representative of the ground water level due to introduction of water during wash boring operations.																	

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



PROJECT <u>1671430</u>		<b>RECORD OF BOREHOLE No A2-2</b>		SHEET 2 OF 4		<b>METRIC</b>	
G.W.P. <u>2266-18-00</u>		LOCATION <u>N 4875623.4; E 294063.6 MTM NAD 83 ZONE 10 (LAT. 44.020527; LONG. -79.633924)</u>		ORIGINATED BY <u>JP</u>			
DIST <u>CENTRAL</u> HWY <u>9</u>		BOREHOLE TYPE <u>CME 55 Track-mounted Drill Rig</u>		COMPILED BY <u>KN</u>			
DATUM <u>Geodetic</u>		DATE <u>January 11, 14 to 16, 2019</u>		CHECKED BY <u>NK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	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>	GR	SA	SI	CL
								○ UNCONFINED      + FIELD VANE	● QUICK TRIAXIAL      × REMOULDED	WATER CONTENT (%)										
	--- CONTINUED FROM PREVIOUS PAGE ---																			
	SILTY CLAY to CLAYEY SILT, trace sand Soft to very stiff Grey Wet		12	SS	WH		203									0	1	29	70	
					TO2	TO	PH		202											
					13	SS	1		201								0	0	50	50
					14	SS	1		200											
					15	SS	WH		197											

Continued Next Page

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

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GTA-MTO 001 S:\CLIENTS\MTO\HWY 9\02 DATA\GIN\HWY 9.GPJ GAL-GTA.GDT 20-3-17

Continued Next Page

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



PROJECT		2266-18-00		LOCATION		N 4875623.4; E 294063.6 MTM NAD 83 ZONE 10 (LAT. 44.020527; LONG. -79.633924)		ORIGINATED BY		JP									
DIST		CENTRAL HWY 9		BOREHOLE TYPE		CME 55 Track-mounted Drill Rig		COMPILED BY		KN									
DATUM		Geodetic		DATE		January 11, 14 to 16, 2019		CHECKED BY		NK									
SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	20			40	60	80	100	20						40
	--- CONTINUED FROM PREVIOUS PAGE ---																		
	SILTY CLAY to CLAYEY SILT, trace sand Soft to very stiff Grey Wet		24	SS	WH														
			25	SS	12														
167.9			26	SS	18														
50.9	END OF BOREHOLE																		
	NOTE:  1. Borehole was advanced using 127 mm O.D. casing.  2. Water level measured in open borehole at a depth of 0.9 m below ground surface (Elev. 217.9 m) upon completion of drilling. The water level measurement is not considered to be representative of the ground water level due to introduction of water during wash boring operation.																		

GTA-MTO 001 S:\CLIENTS\MTOWHY\_9\02\_DATA\GINT\HWY\_9.GPJ GAL-GTA.GDT 20-3-17



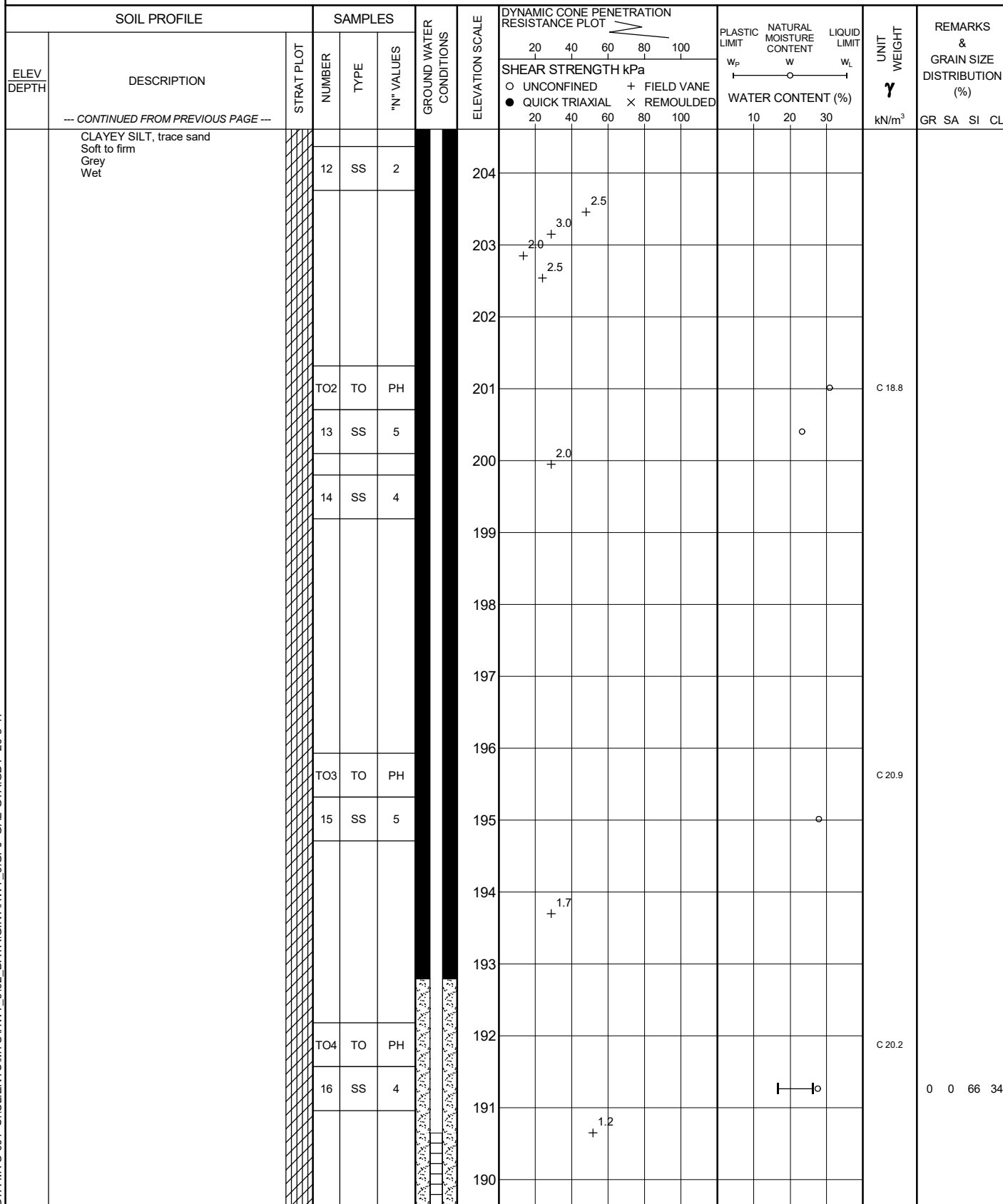


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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



PROJECT <u>1671430</u>		<b>RECORD OF BOREHOLE No AP-1</b>		SHEET 2 OF 3		<b>METRIC</b>	
G.W.P. <u>2266-18-00</u>		LOCATION <u>N 4875610.1; E 294009.4 MTM NAD 83 ZONE 10 (LAT. 44.020407; LONG. -79.634599)</u>		ORIGINATED BY <u>JP</u>			
DIST <u>CENTRAL</u> HWY <u>9</u>		BOREHOLE TYPE <u>CME 55 Track-mounted Drill Rig</u>		COMPILED BY <u>KN</u>			
DATUM <u>Geodetic</u>		DATE <u>December 21, 2018, January 3 and 4, 2019</u>		CHECKED BY <u>NK</u>			



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

GTA-MTO 001 S:\CLIENTS\MTOWHY\_9\02\_DATA\GINT\HWY\_9.GPJ GAL-GTA.GDT 20-3-17



PROJECT 1671430		RECORD OF BOREHOLE No AP-1				SHEET 3 OF 3		METRIC																					
G.W.P. 2266-18-00		LOCATION N 4875610.1; E 294009.4 MTM NAD 83 ZONE 10 (LAT. 44.020407; LONG. -79.634599)				ORIGINATED BY JP																							
DIST CENTRAL HWY 9		BOREHOLE TYPE CME 55 Track-mounted Drill Rig				COMPILED BY KN																							
DATUM Geodetic		DATE December 21, 2018, January 3 and 4, 2019				CHECKED BY NK																							
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 (%)												
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					10 20 30				GR SA SI CL													
187.9	CLAYEY SILT, trace sand Soft to firm Grey Wet		17	SS	WH																								
31.7	END OF BOREHOLE																												
	NOTES:  1. Borehole was advanced using 203 mm O.D. hollow-stem augers to a depth of 6.1 m below ground surface and 127 mm O.D. casing below 6.1 m below ground surface to end of borehole.  2. Water level measured in open borehole at ground surface upon completion of drilling. The water level measurement is not considered to be representative of the ground water level due to introduction of water during wash boring operations.  3. Water level measurements in standpipe piezometer:  <table border="1" style="margin-left: 20px;"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev. (m)</th> </tr> </thead> <tbody> <tr> <td>Jan. 4, 2019</td> <td>0.0</td> <td>219.6</td> </tr> <tr> <td>Mar. 8, 2019</td> <td>1.9</td> <td>217.7</td> </tr> <tr> <td>Mar. 11, 2019</td> <td>2.4</td> <td>217.2</td> </tr> <tr> <td>Mar. 14, 2019</td> <td>2.0</td> <td>217.6</td> </tr> </tbody> </table>	Date	Depth (m)	Elev. (m)	Jan. 4, 2019	0.0	219.6	Mar. 8, 2019	1.9	217.7	Mar. 11, 2019	2.4	217.2	Mar. 14, 2019	2.0	217.6													
Date	Depth (m)	Elev. (m)																											
Jan. 4, 2019	0.0	219.6																											
Mar. 8, 2019	1.9	217.7																											
Mar. 11, 2019	2.4	217.2																											
Mar. 14, 2019	2.0	217.6																											



PROJECT 1671430		RECORD OF BOREHOLE No AP-2		SHEET 1 OF 3		METRIC									
G.W.P. 2266-18-00		LOCATION N 4875648.5; E 294077.6 MTM NAD 83 ZONE 10 (LAT. 44.020754; LONG. -79.633749)		ORIGINATED BY SP											
DIST CENTRAL HWY 9		BOREHOLE TYPE CME 55 Track-mounted Drill Rig		COMPILED BY KN											
DATUM Geodetic		DATE January 30 and 31 and February 1, 2019		CHECKED BY NK											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		WATER CONTENT (%) W <sub>p</sub> — W — W <sub>L</sub>		γ		GR SA SI CL	
220.7	GROUND SURFACE							20 40 60 80 100		10 20 30		kN/m <sup>3</sup>			
0.0	ASPHALT (170 mm)							20 40 60 80 100		10 20 30					
0.2	Silty sand, trace gravel to gravelly (FILL) Compact to very dense Brown Moist		1	SS	50/0.04		220								
			2	SS	24										
219.3							219							3 31 53 13	
1.5	Clayey silt with sand to Sandy clayey silt, trace gravel (FILL) Firm to stiff Brown to grey below 3.8 m Moist		3	SS	9										
			4	SS	10		218								
			5	SS	6		217								
			6	SS	4									2 29 55 14	
216.0			7A	SS	3		216								
4.7	CLAYEY SILT Firm to very stiff Grey Moist		7B												
							215	4.3 +							
								>95.76 +							
			TO1	TO	PH		214								
213.5	- (1 mm to 2 mm) thick SILT and SAND seams encountered at depths of 6.2 m and 6.3 m														
7.2	SAND, trace to some silt, trace clay, trace gravel Loose Grey Moist to wet		9	SS	9		213								
							212					CID 19.6			
			10	SS	9		211							2 88 8 2	
210.5															
10.2	SILTY CLAY to CLAYEY SILT, trace sand Soft to stiff Grey Moist		11	SS	10		210								
							209								
			12	SS	5		208								
							207								
			13	SS	6									0 3 43 54	
							206								

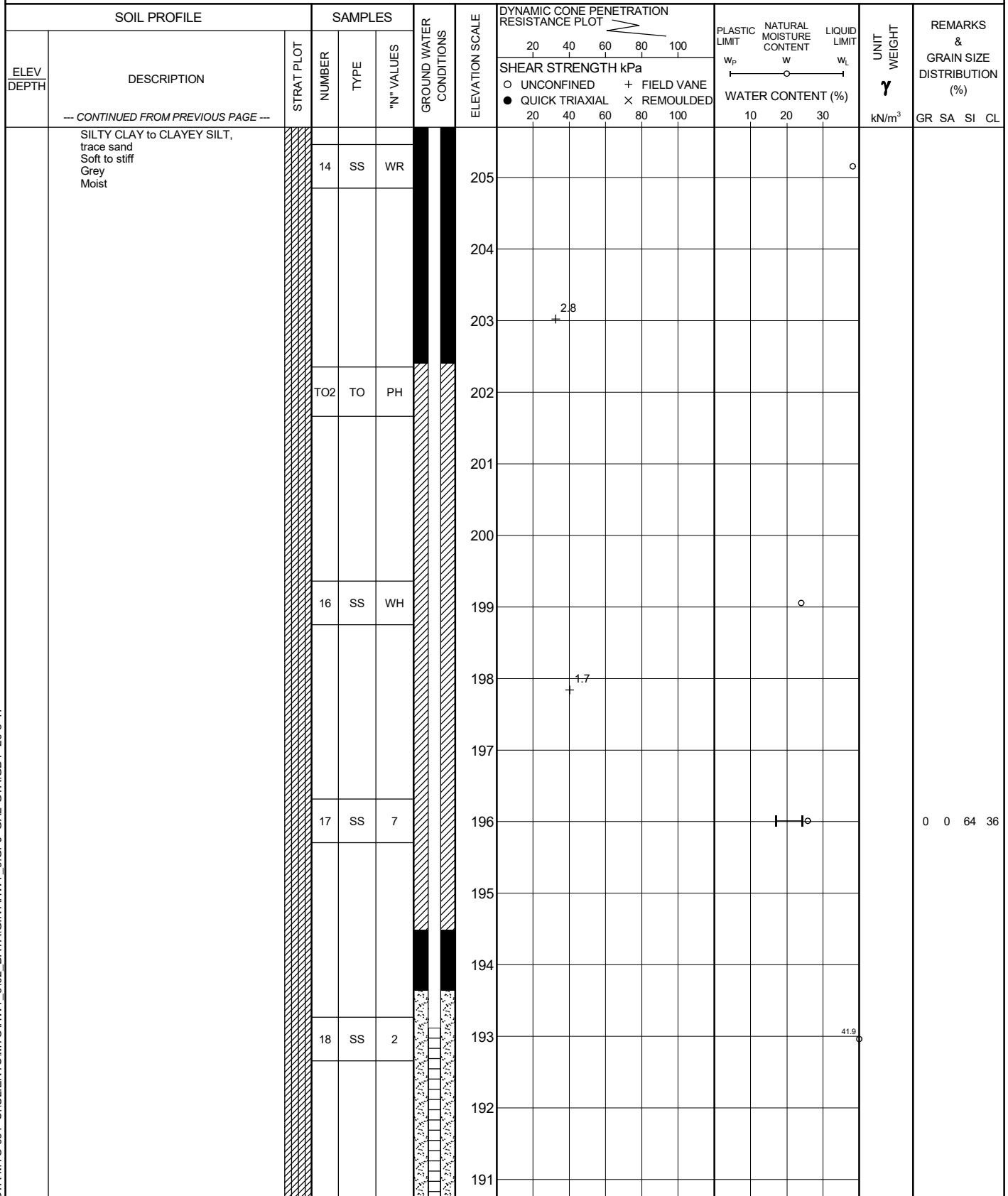
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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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PROJECT <u>1671430</u>		<b>RECORD OF BOREHOLE No AP-2</b>		SHEET 2 OF 3		<b>METRIC</b>	
G.W.P. <u>2266-18-00</u>		LOCATION <u>N 4875648.5; E 294077.6 MTM NAD 83 ZONE 10 (LAT. 44.020754; LONG. -79.633749)</u>		ORIGINATED BY <u>SP</u>			
DIST <u>CENTRAL</u> HWY <u>9</u>		BOREHOLE TYPE <u>CME 55 Track-mounted Drill Rig</u>		COMPILED BY <u>KN</u>			
DATUM <u>Geodetic</u>		DATE <u>January 30 and 31 and February 1, 2019</u>		CHECKED BY <u>NK</u>			



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

GTA-MTO 001 S:\CLIENTS\MTOWHY\_902\_DATA\GNT\HWY\_9.GPJ GAL-GTA.GDT 20-3-17



PROJECT <u>1671430</u>	<b>RECORD OF BOREHOLE No AP-2</b>	SHEET 3 OF 3	<b>METRIC</b>
G.W.P. <u>2266-18-00</u>	LOCATION <u>N 4875648.5; E 294077.6 MTM NAD 83 ZONE 10 (LAT. 44.020754; LONG. -79.633749)</u>	ORIGINATED BY <u>SP</u>	
DIST <u>CENTRAL</u> HWY <u>9</u>	BOREHOLE TYPE <u>CME 55 Track-mounted Drill Rig</u>	COMPILED BY <u>KN</u>	
DATUM <u>Geodetic</u>	DATE <u>January 30 and 31 and February 1, 2019</u>	CHECKED BY <u>NK</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						PLASTIC LIMIT   NATURAL MOISTURE   LIQUID CONTENT   LIMIT			UNIT WEIGHT  γ  kN/m³	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 (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
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PROJECT		2266-18-00		LOCATION		N 4875610.1; E 294007.8 MTM NAD ZONE 10 (LAT. 44.020407; LONG. -79.634619)		ORIGINATED BY		CC		DIST		CENTRAL HWY 9		BOREHOLE TYPE		AMS Power Probe 9580 Rubber Track Mount with 210mm O.D. Hollow Stem Augers		COMPILED BY		CC		DATUM		Geodetic		DATE		March 8, 2019		CHECKED BY		NK		RECORD OF BOREHOLE No MW-01		SHEET 1 OF 1		METRIC	
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)																								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W <sub>p</sub> W W <sub>L</sub>																												
219.7	GROUND SURFACE						20	40	60	80	100																														
0.0	Silt and sand to silty sand, trace to some gravel, trace clay, some organics to a depth of 0.6 m (FILL) Very loose to loose Brown to black Moist																																								
218.1	PEAT																																								
1.6	Soft Black Moist																																								
215.4	CLAYEY SILT																																								
4.3	Soft Grey Moist																																								
214.8	SILT and SAND to SILTY SAND, trace to some clay Compact to dense Grey/Brown Moist to wet																																								
4.9																																									
211.2	END OF BOREHOLE																																								
8.5	NOTE:  1. Soil profile for Borehole MW-01 obtained from information collected during the advancement of Borehole AP-1.  2. Groundwater level measurements in piezometer:  Date      Depth (m)      Elev. (m)  Mar. 8, 2019      2.1      217.6 Mar. 11, 2019      2.6      217.1 Mar. 14, 2019      2.0      217.7																																								



# PRESENTATION OF SITE INVESTIGATION RESULTS

## Holland Drainage Canal Bridge Site No. 37-31

*Prepared for:*

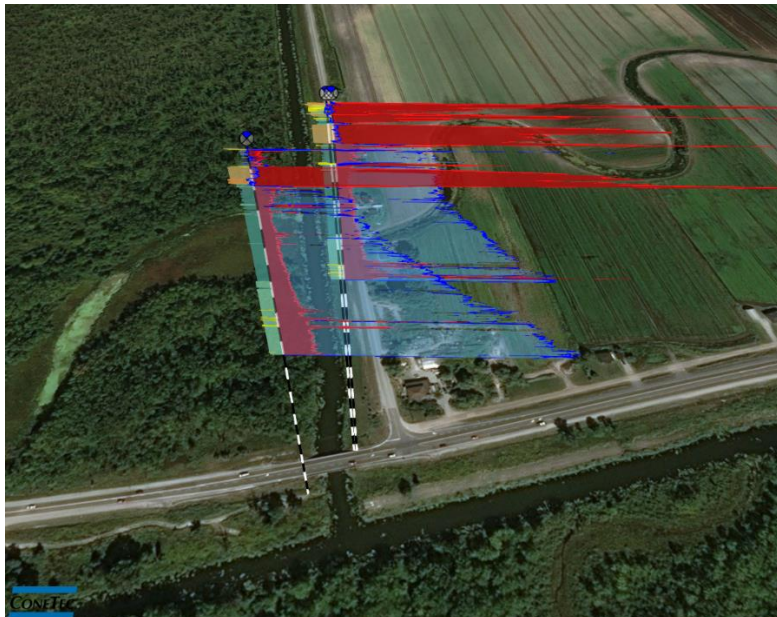
Golder Associates

ConeTec Job No: 19-05015

Project Start Date: 28-Feb-2019

Project End Date: 08-Mar-2019

Report Date: 12-Mar-2019



*Prepared by:*

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





## Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates at Highway 8 and Highway 9, Schomberg, ON. The program consisted of 2 cone penetration tests (CPT), and 1 seismic cone penetration test (SCPT).

## Project Information

Project	
Client	Golder Associates
Project	Holland Drainage Canal Bridge Site No. 37-31
ConeTec project number	19-05015

An image from Google Earth including the CPT and SCPT test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C3)	30 ton rig cylinder	CPT
CPT track rig (TC7)	30 ton rig cylinder	SCPT



Coordinates		
Test Type	Collection Method	EPSG Number
CPT, SCPT	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	Standard Plots, Advanced Plots, Soil Behaviour Type (SBT) Scatter Plots and Seismic Shear Wave (Vs) Plots are included in the release files.

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)
545:T1500F15U500	545	15	225	1500	15	500
560:T1500F15U500	560	15	225	1500	15	500
The CPT summary indicates which cone was used for each sounding.						

Calculated Geotechnical Parameter Tables	
Additional information	<p>The Normalized Soil Behaviour Type Chart based on <math>Q_{tn}</math> (SBT <math>Q_{tn}</math>) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip resistance (<math>q_t</math>) sleeve friction (<math>f_s</math>) and pore pressure (<math>u_2</math>). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile.</p> <p>Soils were classified as either drained or undrained based on the <math>Q_{tn}</math> Normalized Soil Behavior Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).</p>



## Limitations

This report has been prepared for the exclusive use of Golder Associates (Client) for the project titled “Holland Drainage Canal Bridge Site No. 37-31”. 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.



Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

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 5 cm<sup>2</sup>, 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.



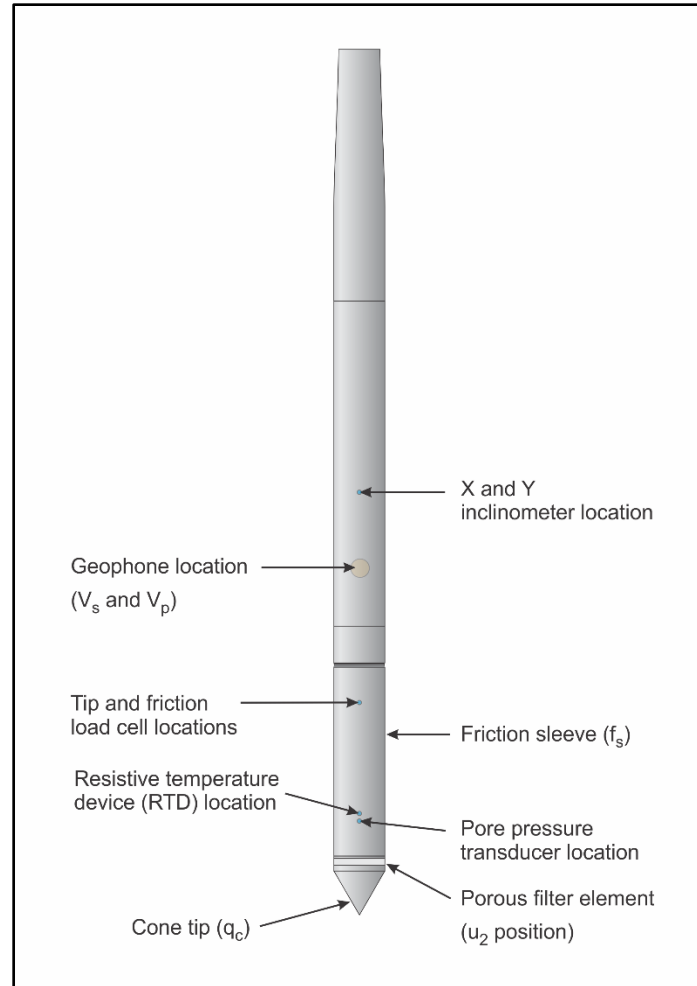


Figure CPTu. Piezocone Penetrometer (15 cm<sup>2</sup>)

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 interval is 2.5 cm; 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 38.1 mm 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 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 et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behaviour 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 files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

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



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

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

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

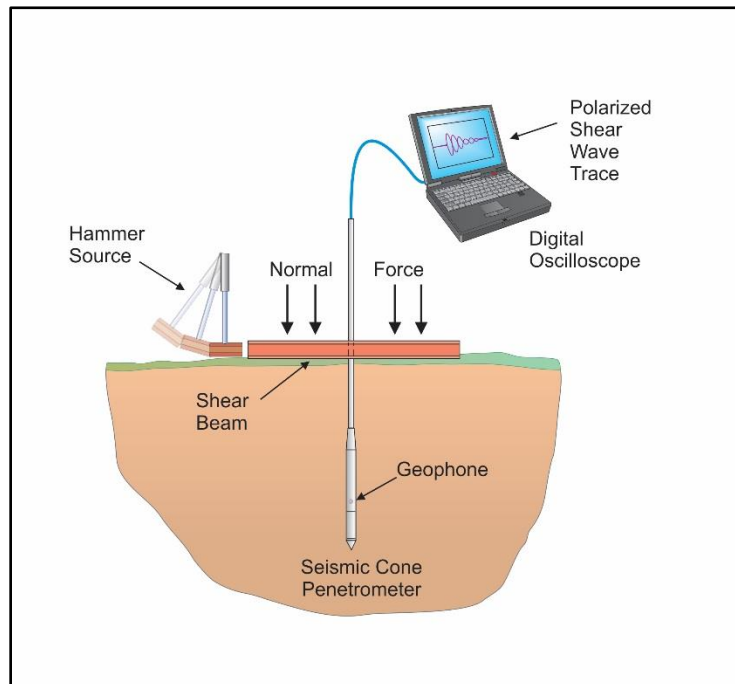


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM D5778 and ASTM D7400 standards.

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

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Typically, five wave traces for each orientation are recorded for quality control and uncertainty analysis purposes. After reviewing wave



traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

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

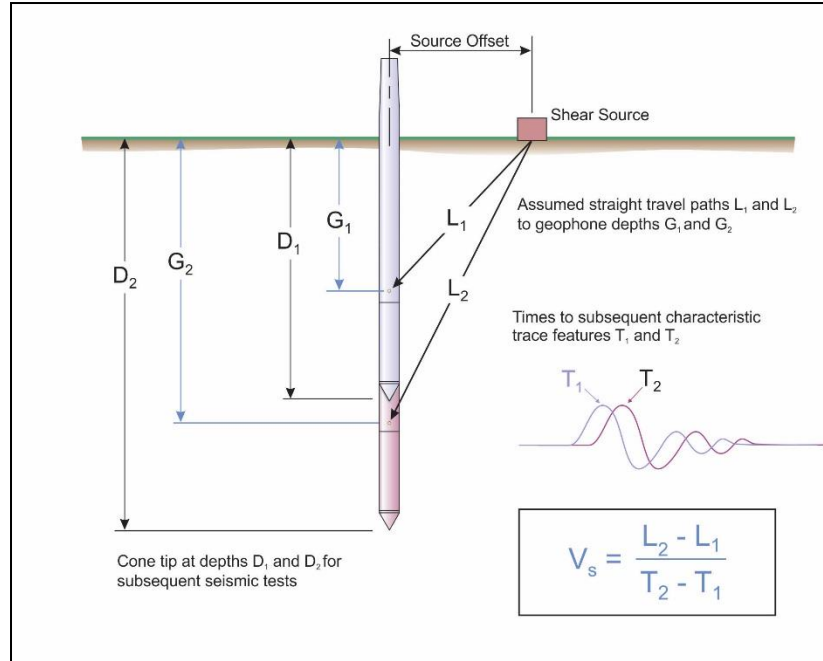


Figure SCPTu-2. Illustration of a seismic cone penetration test

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

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

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

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

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



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

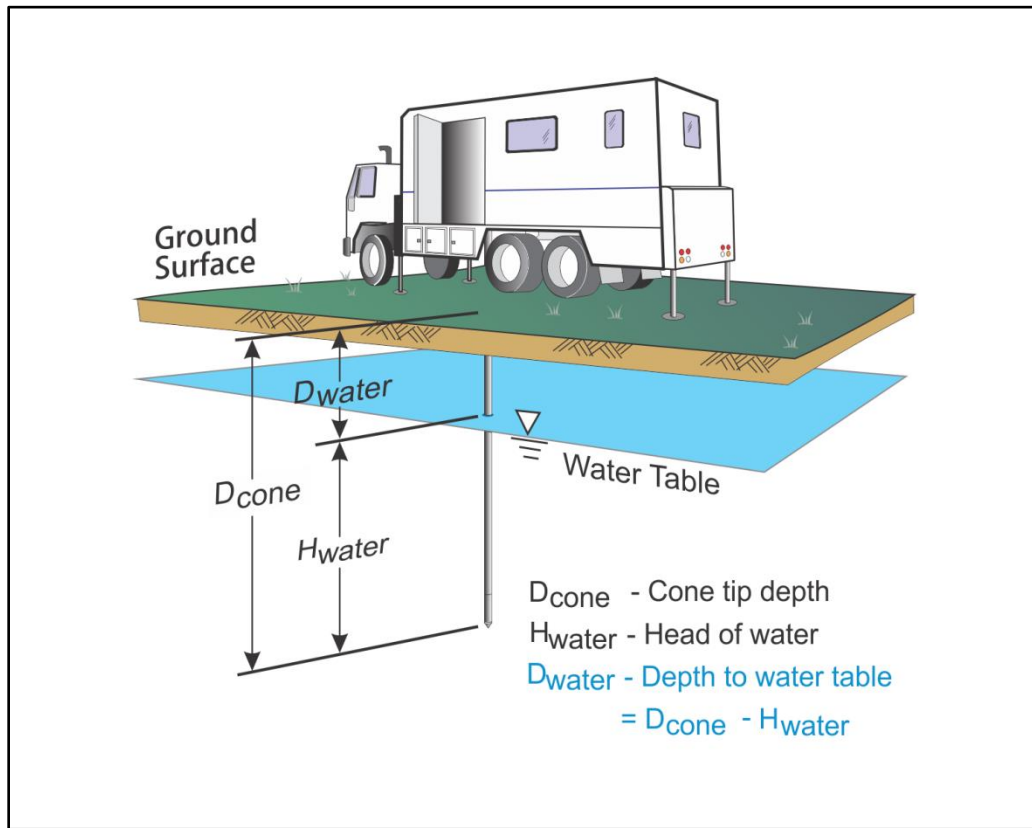


Figure PPD-1. Pore pressure dissipation test setup

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

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



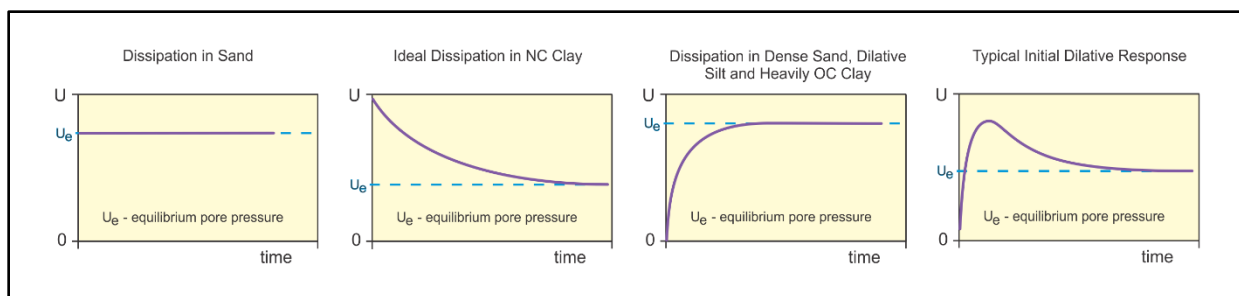


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure ( $u_{eq}$ ) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in 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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Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, "An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils", *Canadian Geotechnical Journal*, 36(2): 369-381.

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



The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with  $I_c$ ,  $S_u(N_{kt})$ ,  $\Phi$  and  $N1(60)I_c$
- Soil Behaviour Type (SBT) Scatter Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Traces
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



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





Job No: 19-05015  
Client: Golder Associates  
Project: Holland Drainage Canal Bridge Site No.37-31  
Start Date: 28-Feb-2019  
End Date: 08-Mar-2019

### ***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
CPT18-01	19-05015_CP01	28-Feb-2019	560:T1500F15U500	2.5	2.98	4875082	609479	
CPT18-01B	19-05015_CP01B	28-Feb-2019	560:T1500F15U500	2.5	50.00	4875082	609481	
SCPT19-02	19-05015_SP02	08-Mar-2019	545:T1500F15U500	0.3	60.00	4875048	609452	3

1. The assumed phreatic surface was based on the dynamic pore pressure response unless otherwise noted. 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.
3. The assumed phreatic surface was based on pore pressure dissipation tests.





# Golder Associates

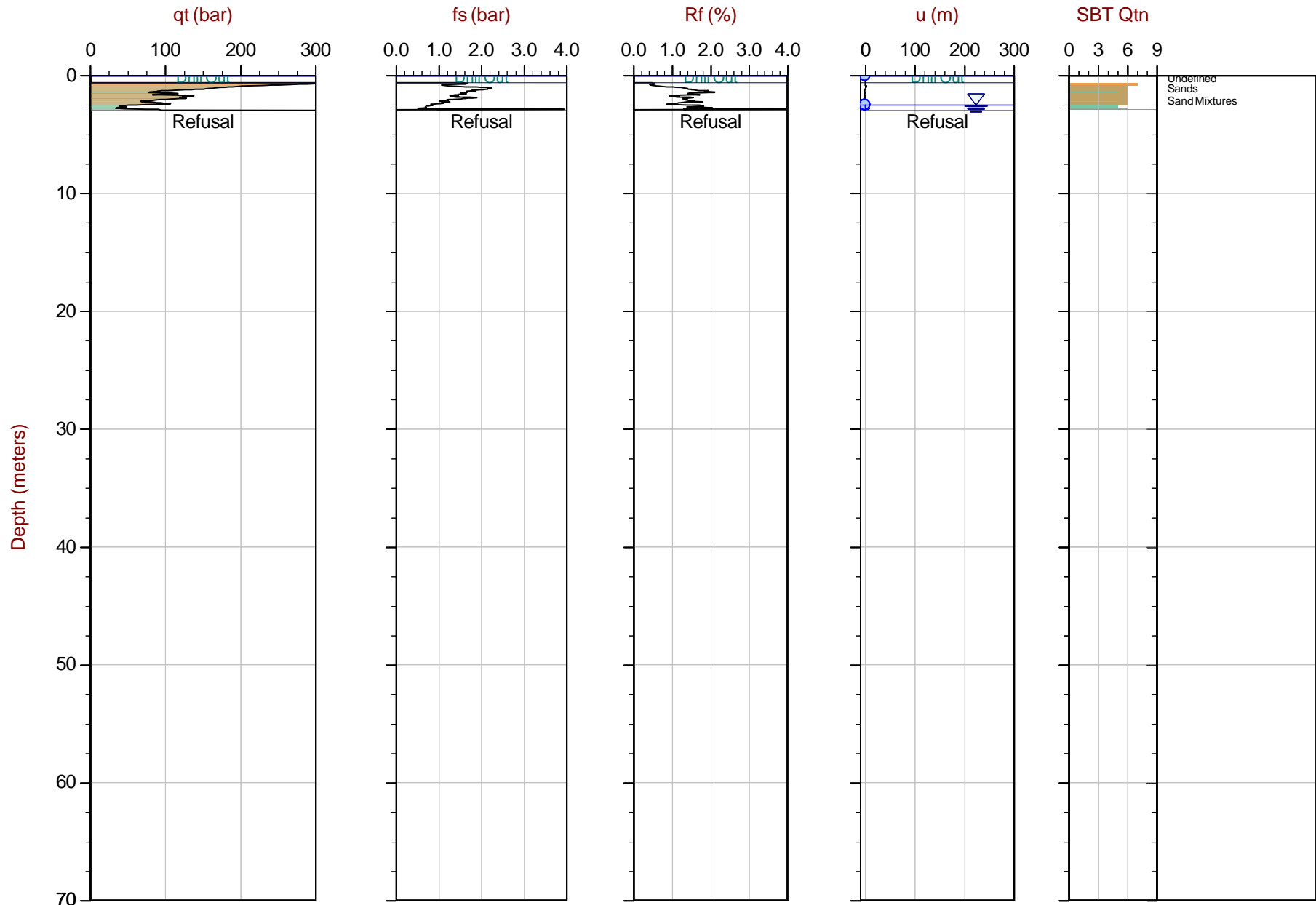
Job No: 19-05015

Date: 2019-02-28 10:25

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: CPT18-01

Cone: 560:T1500F15U500



Max Depth: 2.975 m / 9.76 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 19-05015\_CP01.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 17 NN: 4875082m E: 609479m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

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





# Golder Associates

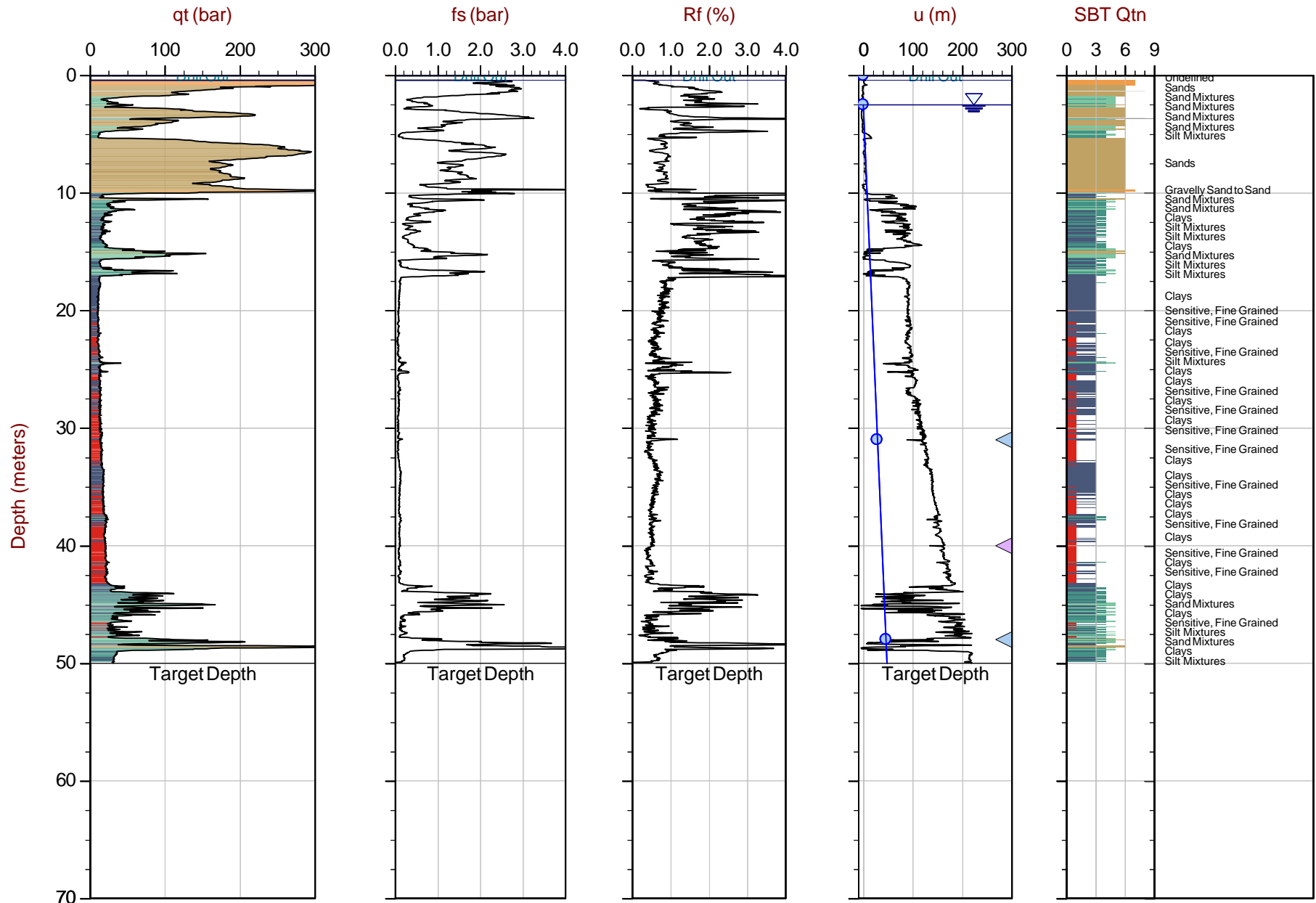
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Date: 2019-02-28 11:12

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: CPT18-01B

Cone: 560:T1500F15U500



Max Depth: 50.000 m / 164.04 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 19-05015\_CP01B.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 17 NN: 4875082m E: 609481m

Sheet No: 1 of 1

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

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





# Golder Associates

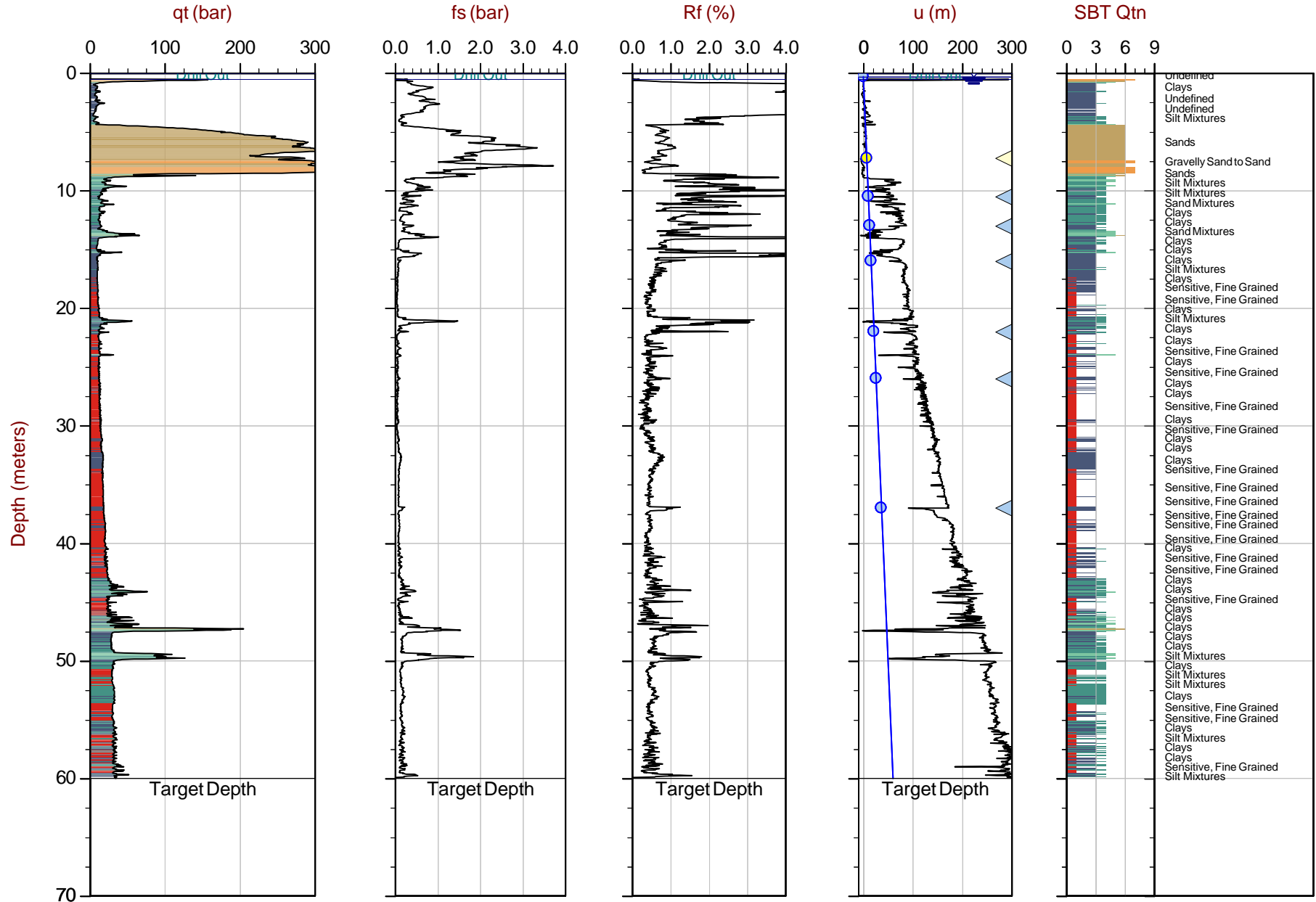
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Date: 2019-03-08 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500



Max Depth: 60.000 m / 196.85 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 19-05015\_SP02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 17 NN: 4875048m E: 609452m

Sheet No: 1 of 1

Overplot Item: ● 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  
Ic, Su(Nkt), OCR and N1(60)Ic





# Golder Associates

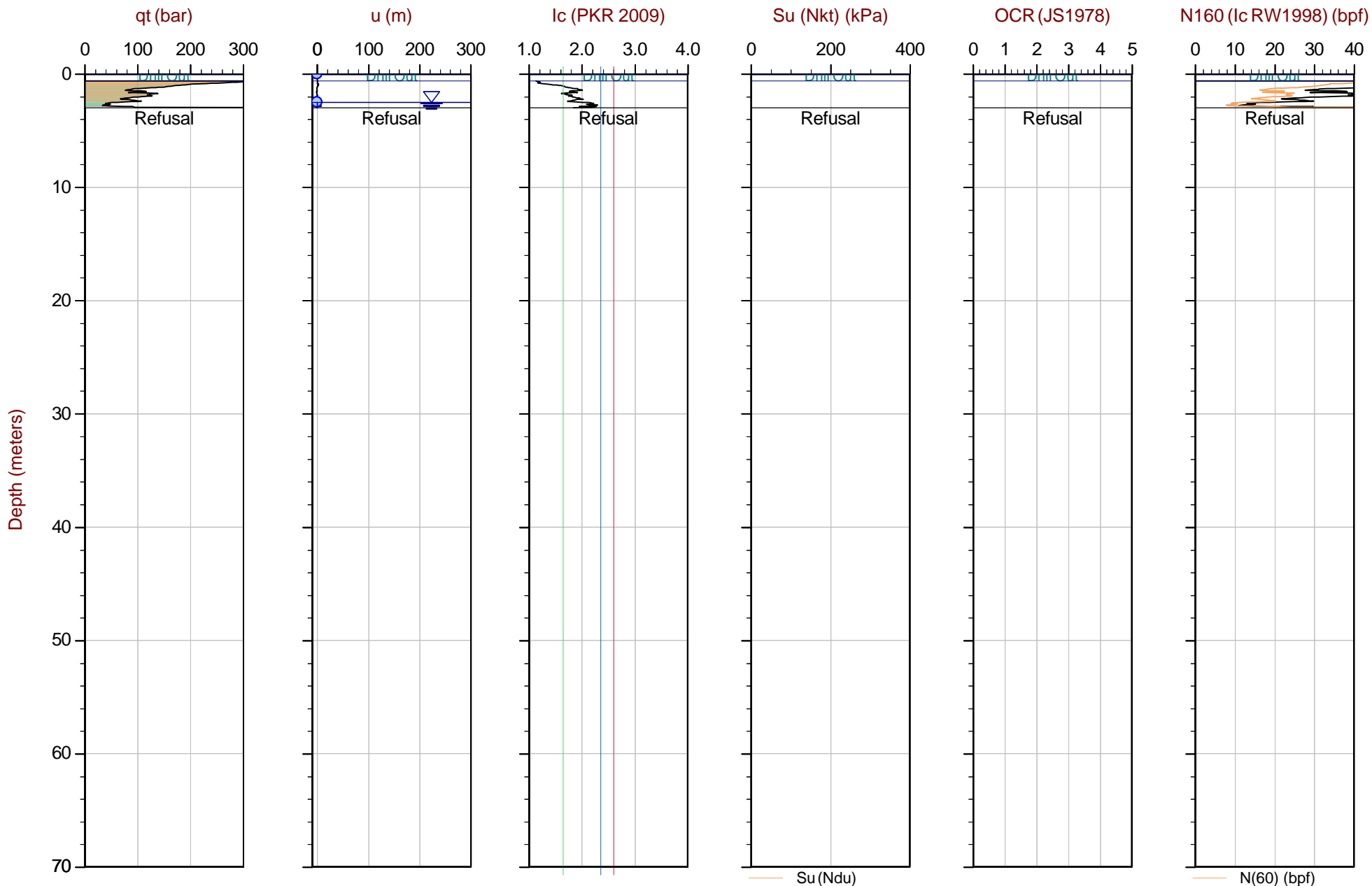
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Date: 2019-02-28 10:25

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: CPT18-01

Cone: 560:T1500F15U500



Max Depth: 2.975 m / 9.76 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 19-05015\_CP01.COR

Unit Wt: SBTQtn (PKR2009)

Su Nkt/Ndu: 12.5 / 11.0

SBT: Robertson, 2009 and 2010

Coords: UTM 17 NN: 4875082m E: 609479m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved

— Hydrostatic Line

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





# Golder Associates

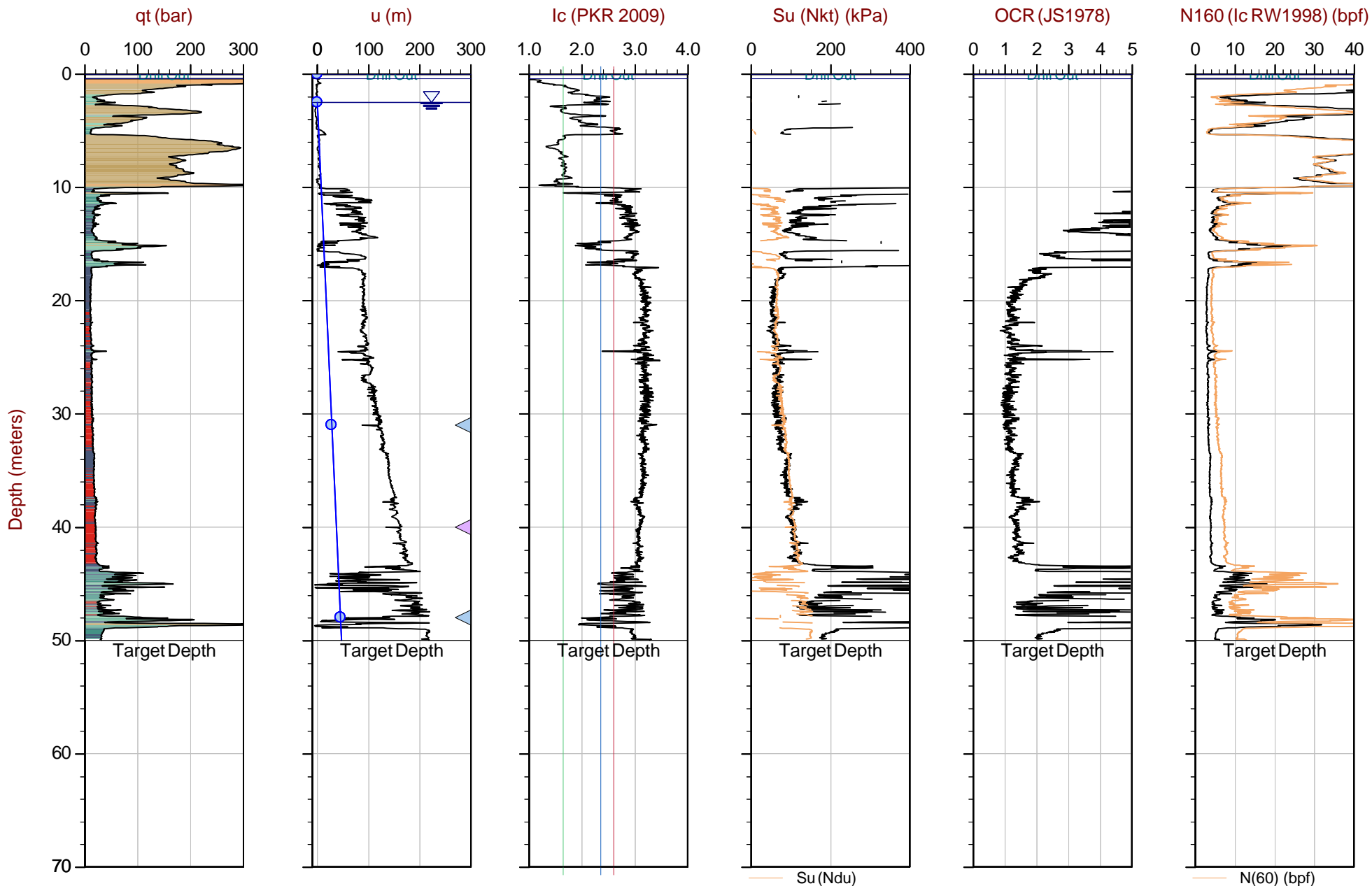
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Sounding: CPT18-01B

Cone: 560:T1500F15U500



Max Depth: 50.000 m / 164.04 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 19-05015\_CP01B.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt/Ndu: 12.5 / 11.0

SBT: Robertson, 2009 and 2010

Coords: UTM 17 NN: 4875082m E: 609481m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

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





# Golder Associates

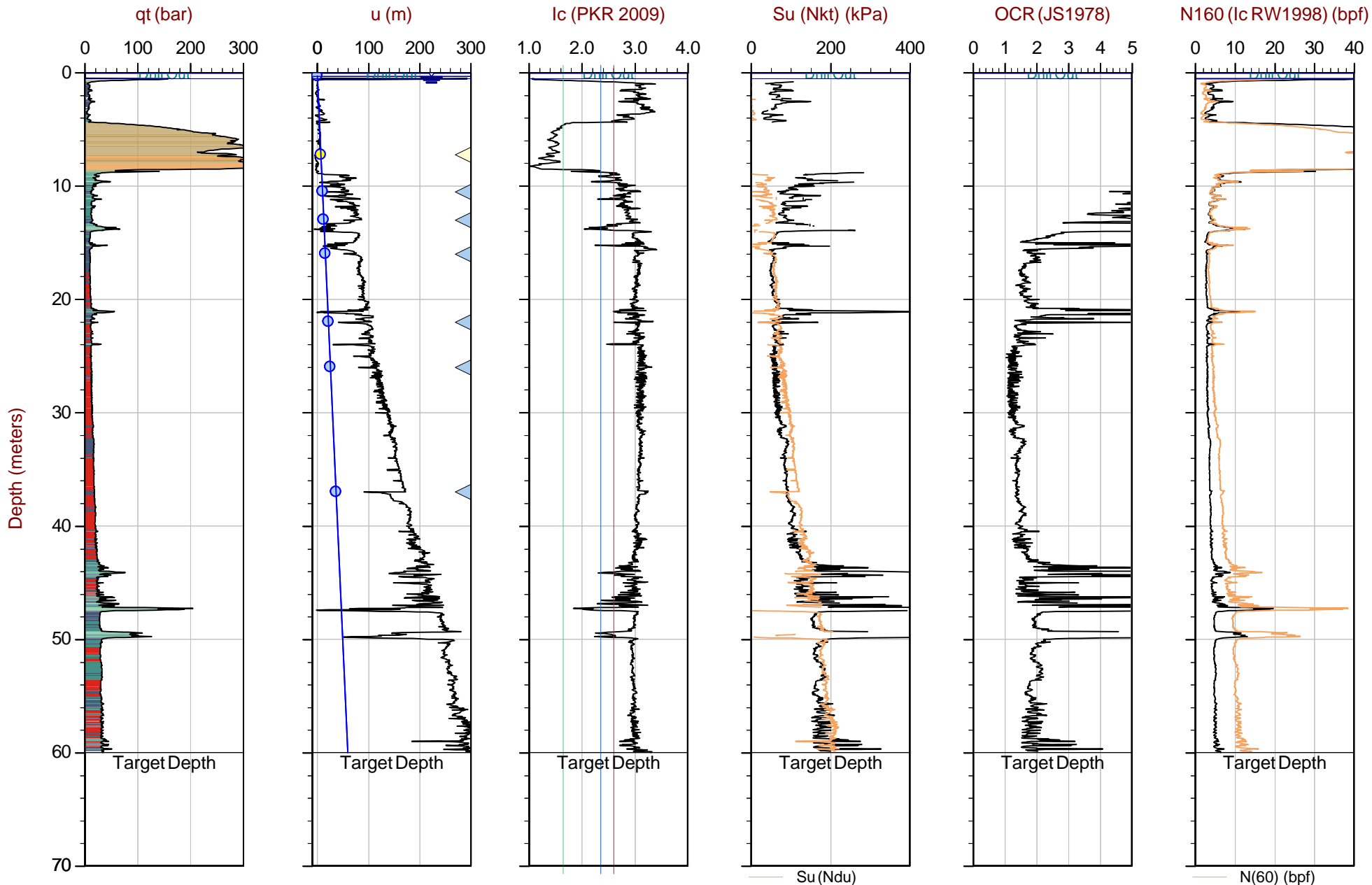
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Date: 2019-03-08 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500



Max Depth: 60.000 m / 196.85 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 19-05015\_SP02.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt/Ndu: 12.5 / 11.0

SBT: Robertson, 2009 and 2010

Coords: UTM 17 NN: 4875048m E: 609452m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq

▲ Dissipation, Ueq achieved

▲ Dissipation, Ueq not achieved

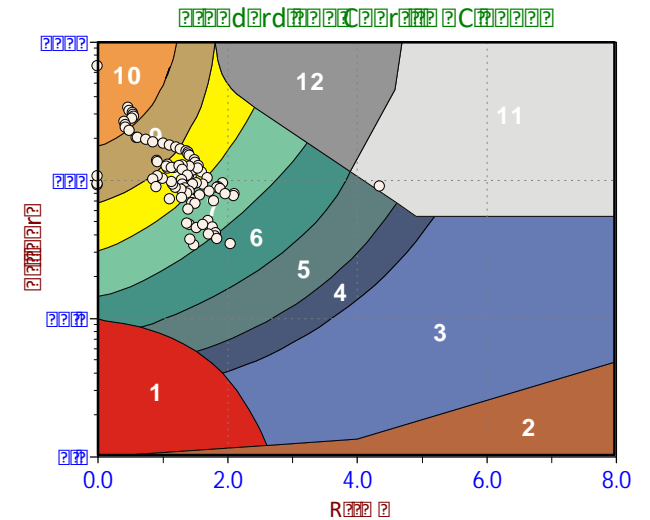
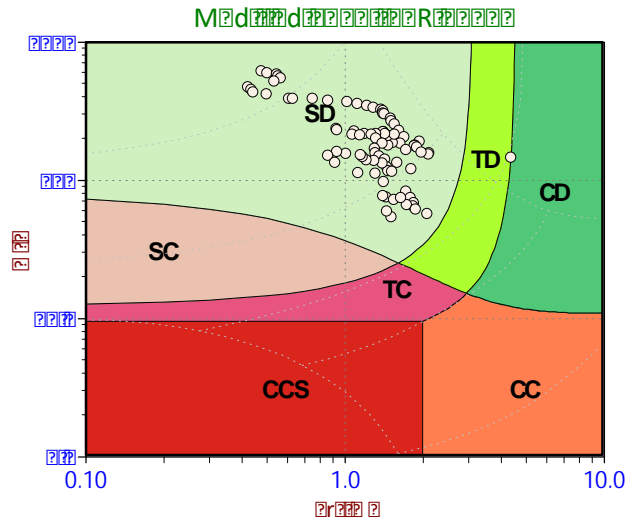
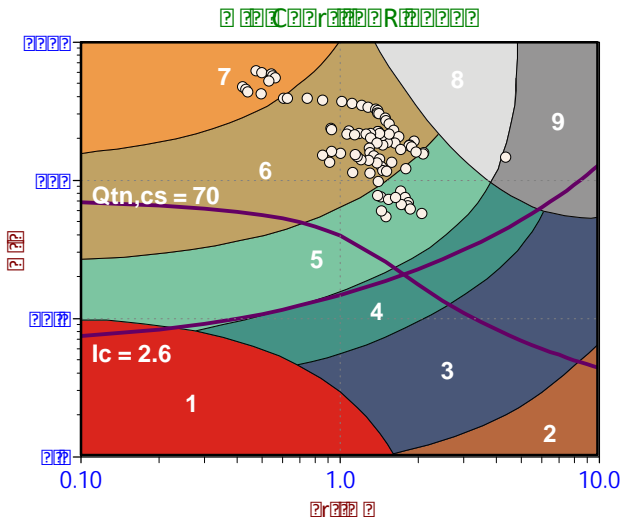
— Hydrostatic Line

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



## Soil Behaviour Type (SBT) Scatter Plots





## Depth Ranges

- >0.0 to 5.0 m
- >5.0 to 10.0 m
- >10.0 to 15.0 m
- >15.0 to 20.0 m
- >20.0 to 25.0 m
- >25.0 to 30.0 m
- >30.0 to 35.0 m
- >35.0 to 40.0 m
- >40.0 to 45.0 m
- >45.0 to 50.0 m
- >50.0 m

## Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

## Legend

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

## Legend

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





# Golder Associates

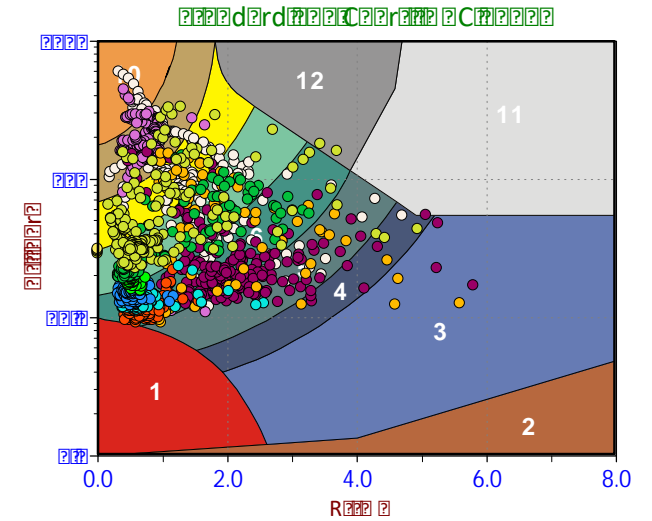
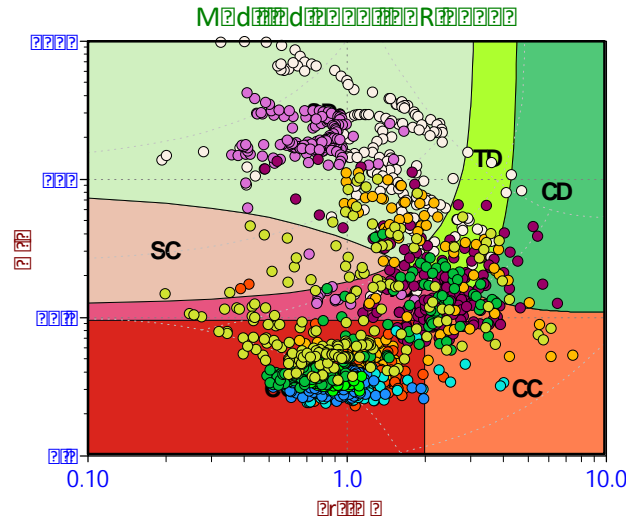
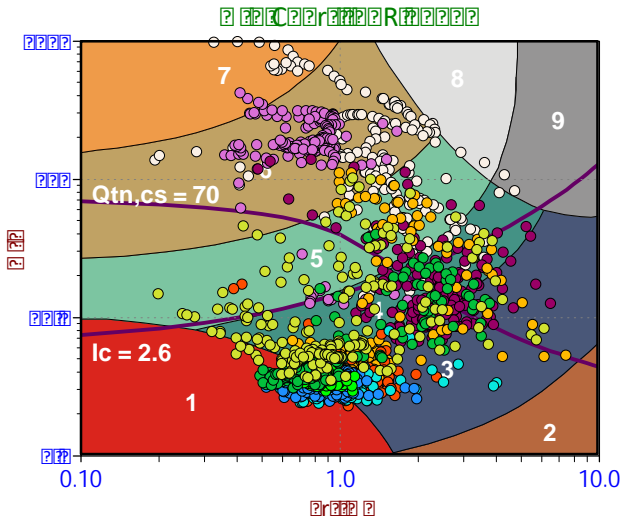
Job No: 19-05015

Date: 2019-02-28 11:12

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: CPT18-01B

Cone: 560:T1500F15U500



## Depth Ranges

- >0.0 to 5.0 m
- >5.0 to 10.0 m
- >10.0 to 15.0 m
- >15.0 to 20.0 m
- >20.0 to 25.0 m
- >25.0 to 30.0 m
- >30.0 to 35.0 m
- >35.0 to 40.0 m
- >40.0 to 45.0 m
- >45.0 to 50.0 m
- >50.0 m

## Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

## Legend

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

## Legend

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





# Golder Associates

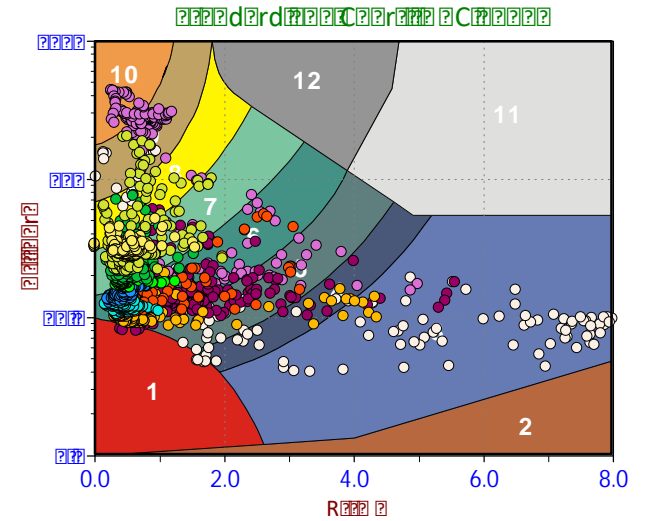
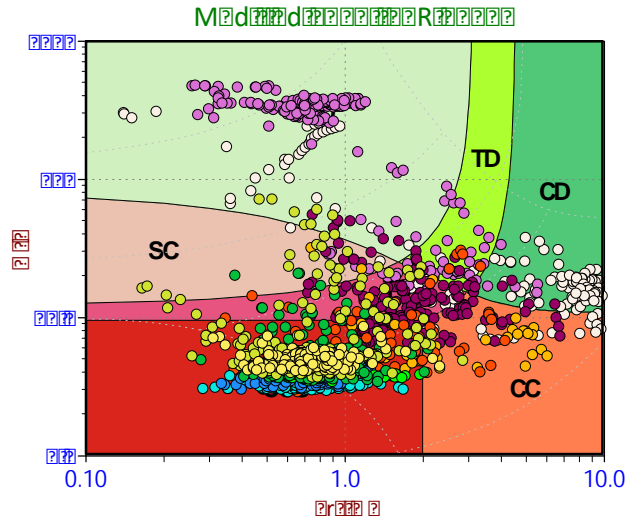
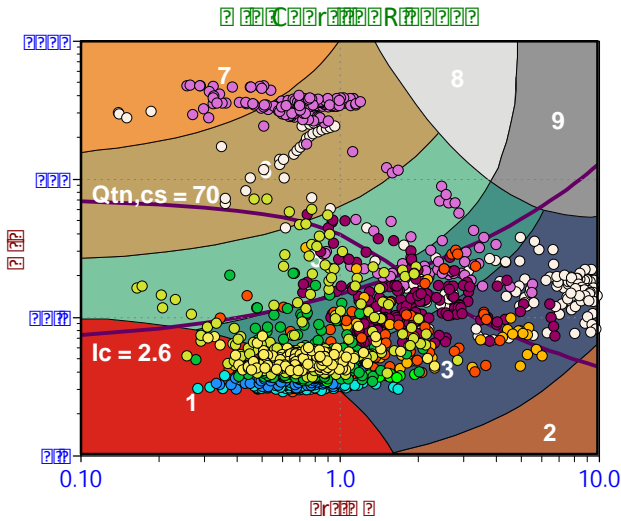
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Date: 2019-03-08 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500



## Depth Ranges

- >0.0 to 5.0 m
- >5.0 to 10.0 m
- >10.0 to 15.0 m
- >15.0 to 20.0 m
- >20.0 to 25.0 m
- >25.0 to 30.0 m
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- >35.0 to 40.0 m
- >40.0 to 45.0 m
- >45.0 to 50.0 m
- >50.0 m

## Legend

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- Organic Soils
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- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

## Legend

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



## Seismic Cone Penetration Test Plots





# Golder Associates

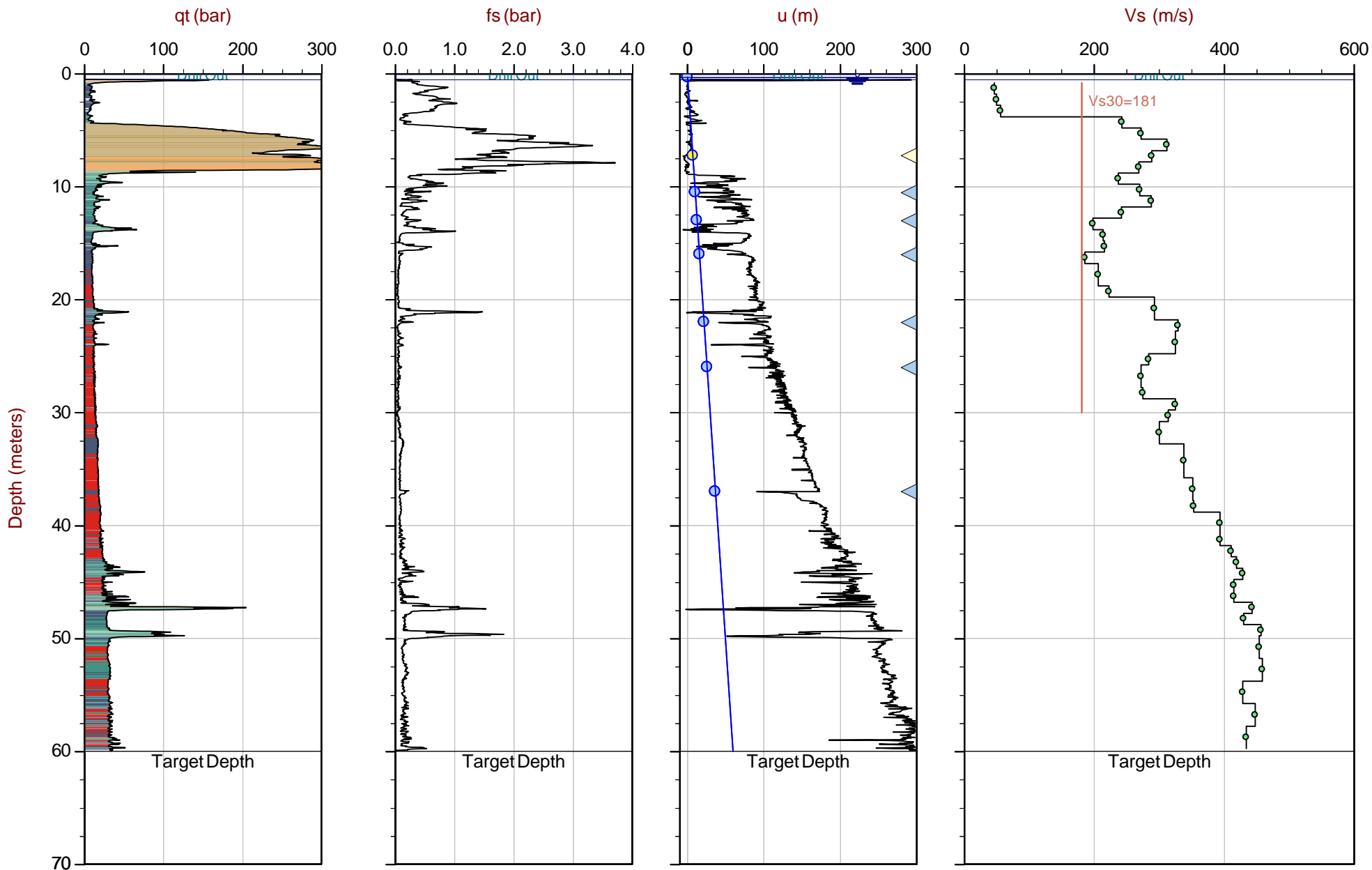
Job No: 19-05015

Date: 2019-03-08 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500



Max Depth: 60.000 m / 196.85 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 19-05015\_SP02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 17 NN: 4875048m E: 609452m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▲ Dissipation, Ueq not achieved

— Hydrostatic Line

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



## Seismic Cone Penetration Test Tabular Results





Job No: 19-05015  
Client: Golder Associates  
Project: Holland Drainage Canal Bridge Site No. 37-31  
Sounding ID: SCPT19-02  
Date: 08-Mar-2019

Seismic Source: Wedge  
Source Offset (m): 2.00  
Source Depth (m): 0.15  
Geophone Offset (m): 0.20

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

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.00	0.80	2.10			
2.00	1.80	2.59	0.49	10.62	46
3.00	2.80	3.32	0.73	14.42	50
4.00	3.80	4.16	0.84	14.97	56
5.00	4.80	5.06	0.90	3.70	243
6.00	5.80	5.99	0.93	3.42	272
7.00	6.80	6.94	0.95	3.05	312
8.00	7.80	7.91	0.96	3.33	289
9.00	8.80	8.88	0.97	3.60	269
10.00	9.80	9.86	0.98	4.12	237
11.00	10.80	10.84	0.98	3.64	270
12.00	11.80	11.82	0.98	3.41	288
13.00	12.80	12.81	0.99	4.07	242
14.00	13.80	13.80	0.99	4.99	198
15.00	14.80	14.79	0.99	4.62	214
16.00	15.80	15.78	0.99	4.60	216
17.00	16.80	16.77	0.99	5.34	186
19.00	18.80	18.76	1.99	9.65	206
20.00	19.80	19.75	0.99	4.45	223
22.00	21.80	21.74	1.99	6.80	293
23.00	22.80	22.74	1.00	3.02	329
25.00	24.80	24.73	1.99	6.14	325
26.00	25.80	25.73	1.00	3.51	284
28.00	27.80	27.72	1.99	7.33	272
29.00	28.80	28.72	1.00	3.62	275
30.00	29.80	29.72	1.00	3.07	325
31.00	30.80	30.72	1.00	3.18	314
33.00	32.80	32.71	2.00	6.66	300
36.00	35.80	35.71	2.99	8.86	338





Job No: 19-05015  
Client: Golder Associates  
Project: Holland Drainage Canal Bridge Site No. 37-31  
Sounding ID: SCPT19-02  
Date: 08-Mar-2019

Seismic Source: Wedge  
Source Offset (m): 2.00  
Source Depth (m): 0.15  
Geophone Offset (m): 0.20

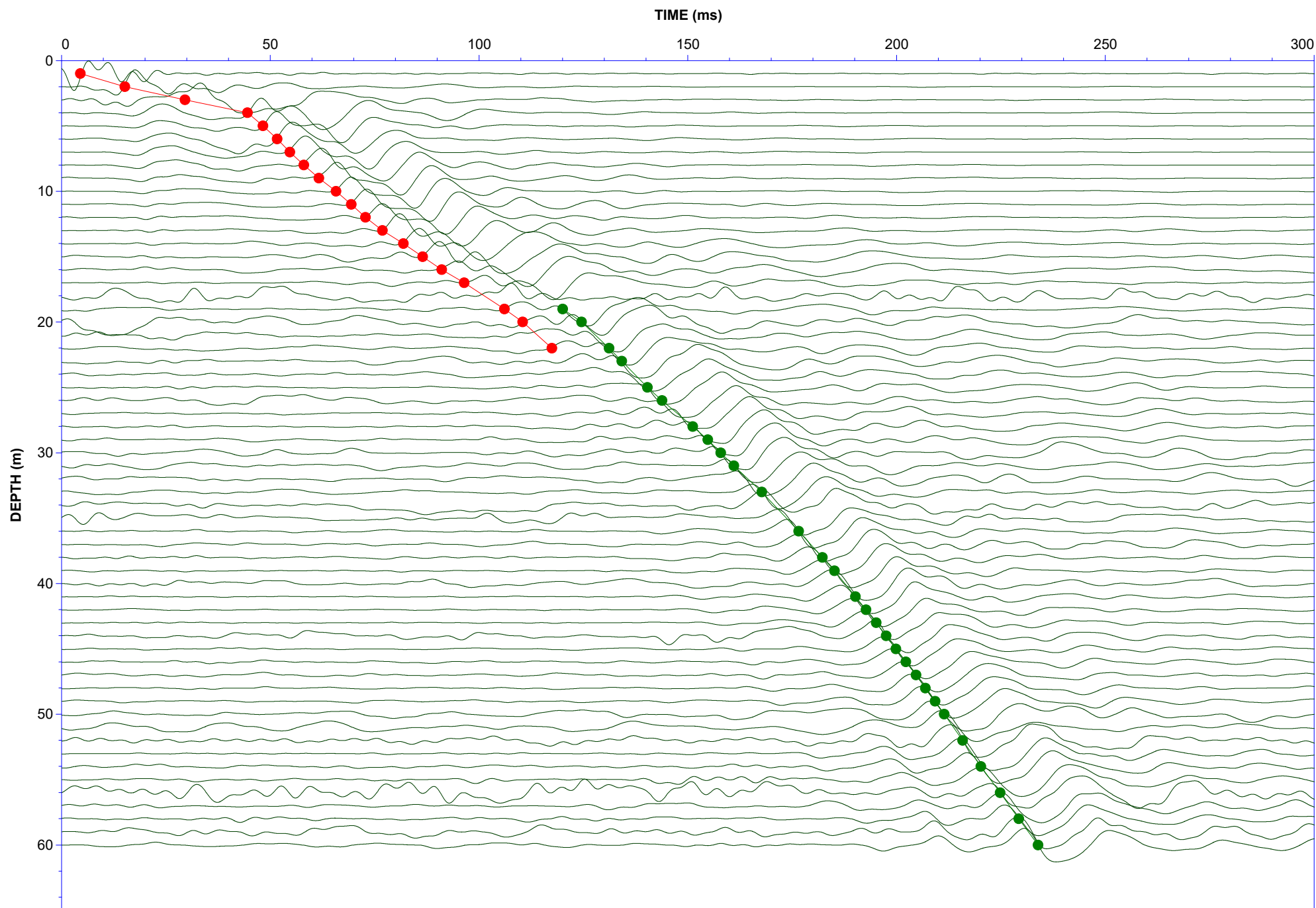
### **SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>**

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
38.00	37.80	37.70	2.00	5.68	352
39.02	38.82	38.72	1.02	2.89	353
41.00	40.80	40.70	1.98	5.02	394
42.00	41.80	41.70	1.00	2.53	394
43.00	42.80	42.70	1.00	2.43	411
44.00	43.80	43.70	1.00	2.38	419
45.00	44.80	44.69	1.00	2.33	429
46.00	45.80	45.69	1.00	2.41	415
47.00	46.80	46.69	1.00	2.41	415
48.00	47.80	47.69	1.00	2.25	443
49.00	48.80	48.69	1.00	2.32	430
50.00	49.80	49.69	1.00	2.18	457
52.00	51.80	51.69	2.00	4.41	454
54.00	53.80	53.69	2.00	4.36	459
56.00	55.80	55.69	2.00	4.66	429
58.00	57.80	57.68	2.00	4.46	448
60.00	59.80	59.68	2.00	4.61	434



## Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Traces







## Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots





Job No: 19-05015  
 Client: Golder Associates  
 Project: Holland Drainage Canal Bridge Site No.37-31  
 Start Date: 28-Feb-2019  
 End Date: 08-Mar-2019

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

Sounding ID	File Name	Cone Area (cm <sup>2</sup> )	Duration (s)	Test Depth (m)	Equilibrium Pore Pressure U <sub>eq</sub> (m)	Estimated Equilibrium Pore Pressure U <sub>eq</sub> (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)	t <sub>50</sub> <sup>a</sup> (s)	Assumed Rigidity Index (I <sub>r</sub> )	c <sub>h</sub> <sup>b</sup> (cm <sup>2</sup> /min)
CPT18-01B	CPT18-01B	15	550	31.000	Not Achieved	28.5		2.5	395	100	1.8
CPT18-01B	CPT18-01B	15	600	40.000	Not Achieved						
CPT18-01B	CPT18-01B	15	300	48.000	Not Achieved	45.5		2.5	38	100	18.4
SCPT19-02	SCPT19-02	15	500	7.250	6.9		0.3				
SCPT19-02	SCPT19-02	15	300	10.500	Not Achieved	10.2		0.3	93	100	7.5
SCPT19-02	SCPT19-02	15	1800	13.000	Not Achieved	12.7		0.3	870	100	0.8
SCPT19-02	SCPT19-02	15	4130	16.000	Not Achieved	15.7		0.3	3592	100	0.2
SCPT19-02	SCPT19-02	15	900	22.000	Not Achieved	21.7		0.3	698	100	1.0
SCPT19-02	SCPT19-02	15	2040	26.000	Not Achieved	25.7		0.3	1484	100	0.5
SCPT19-02	SCPT19-02	15	2500	37.000	Not Achieved	36.7		0.3	2135	100	0.3

a. Time is relative to where umax occurred.

b. Houlsby and Teh, 1991.





*Golder Associates*

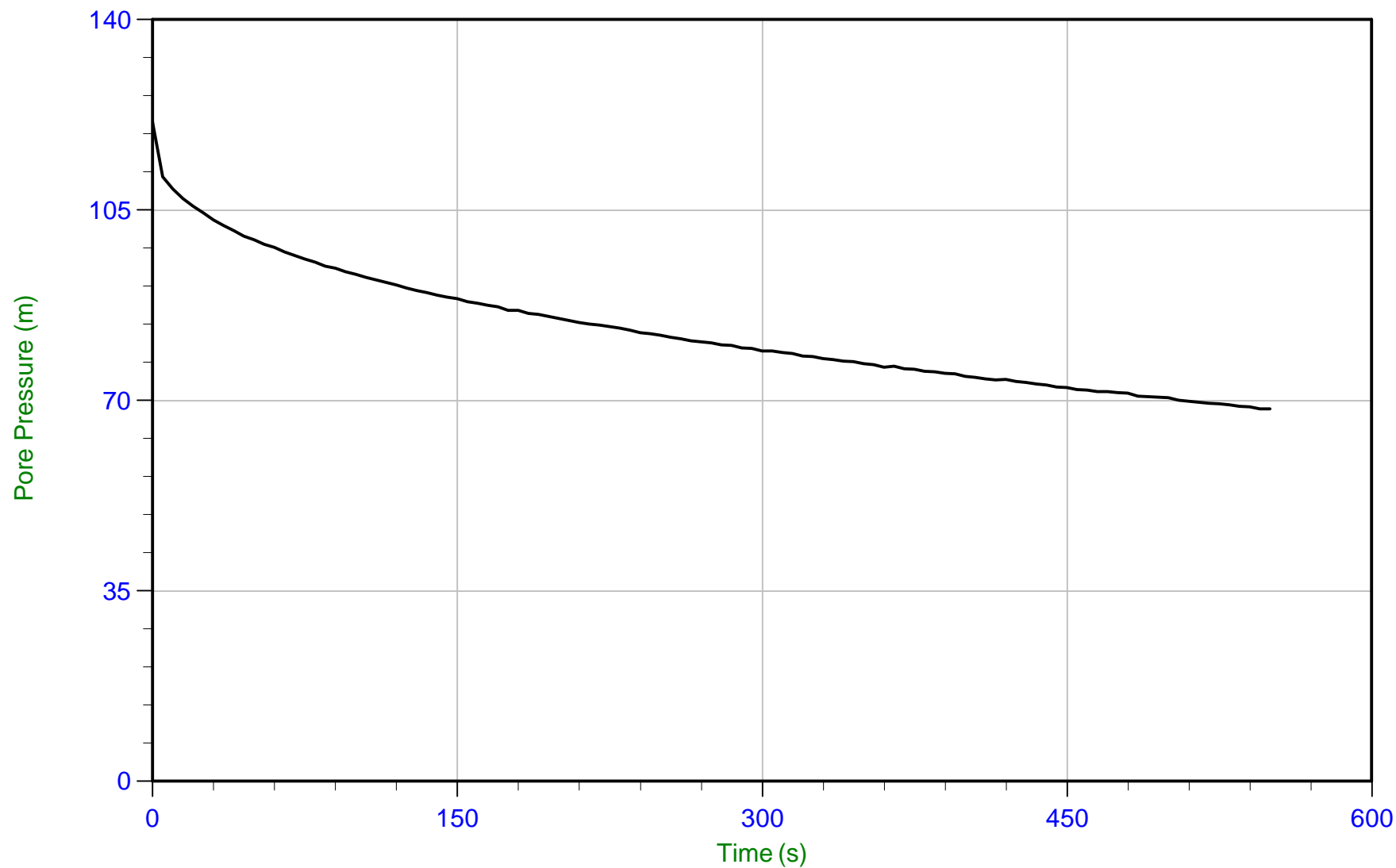
Job No: 19-05015

Date: 02/28/2019 11:12

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: CPT18-01B

Cone: 560:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 19-05015\_CP01B.PPF  
Depth: 31.000 m / 101.705 ft  
Duration: 550.0 s

u Min: 68.4 m  
u Max: 121.2 m  
u Final: 68.4 m

WT: 2.500 m / 8.202 ft  
Ueq: 28.5 m  
U(50): 74.84 m

T(50): 395.2 s  
Ir: 100  
Ch: 1.8 cm<sup>2</sup>/min





*Golder Associates*

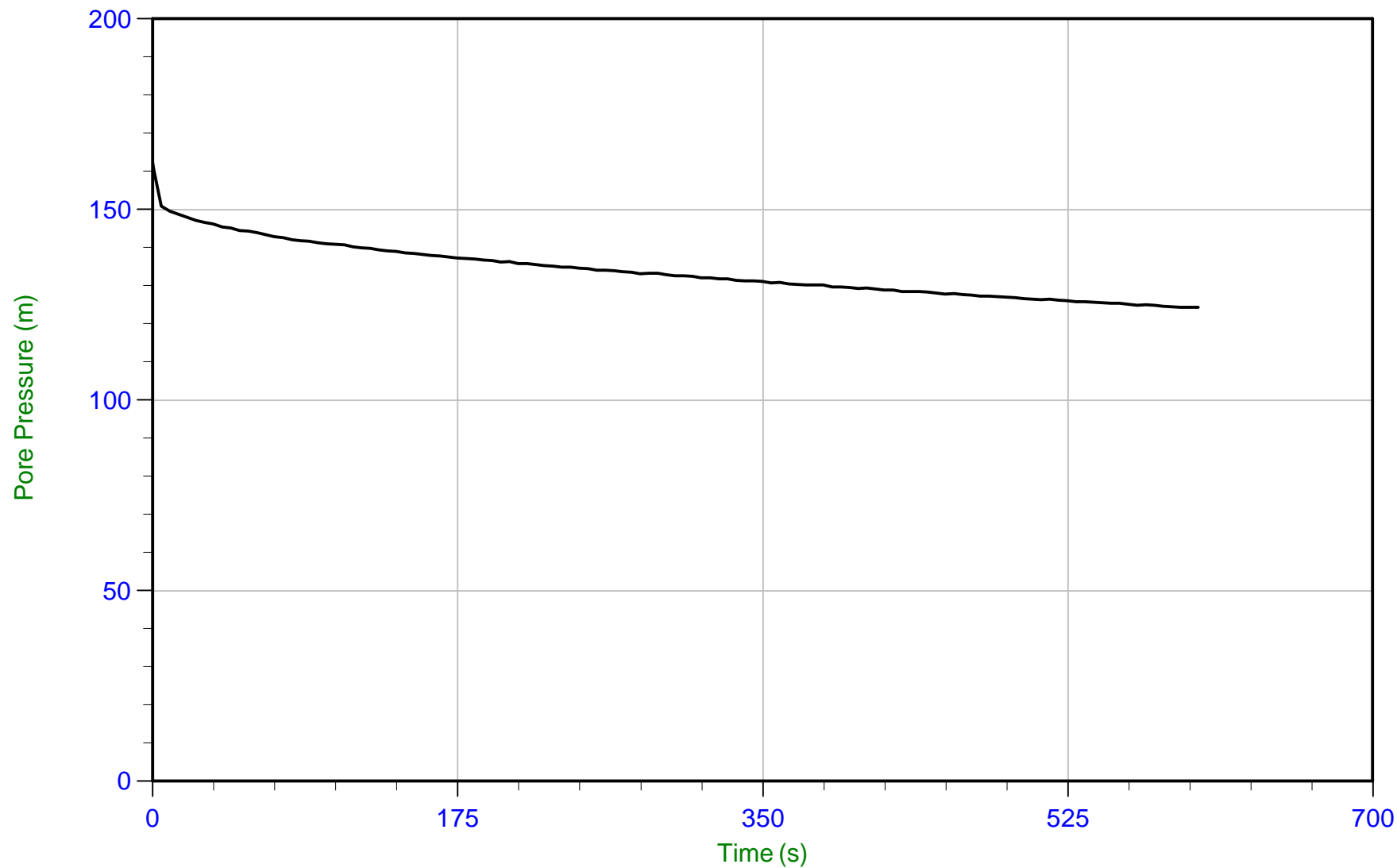
Job No: 19-05015

Date: 02/28/2019 11:12

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: CPT18-01B

Cone: 560:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 19-05015\_CP01B.PPF

Depth: 40.000 m / 131.232 ft

Duration: 600.0 s

u Min: 124.3 m

u Max: 162.2 m

u Final: 124.4 m





*Golder Associates*

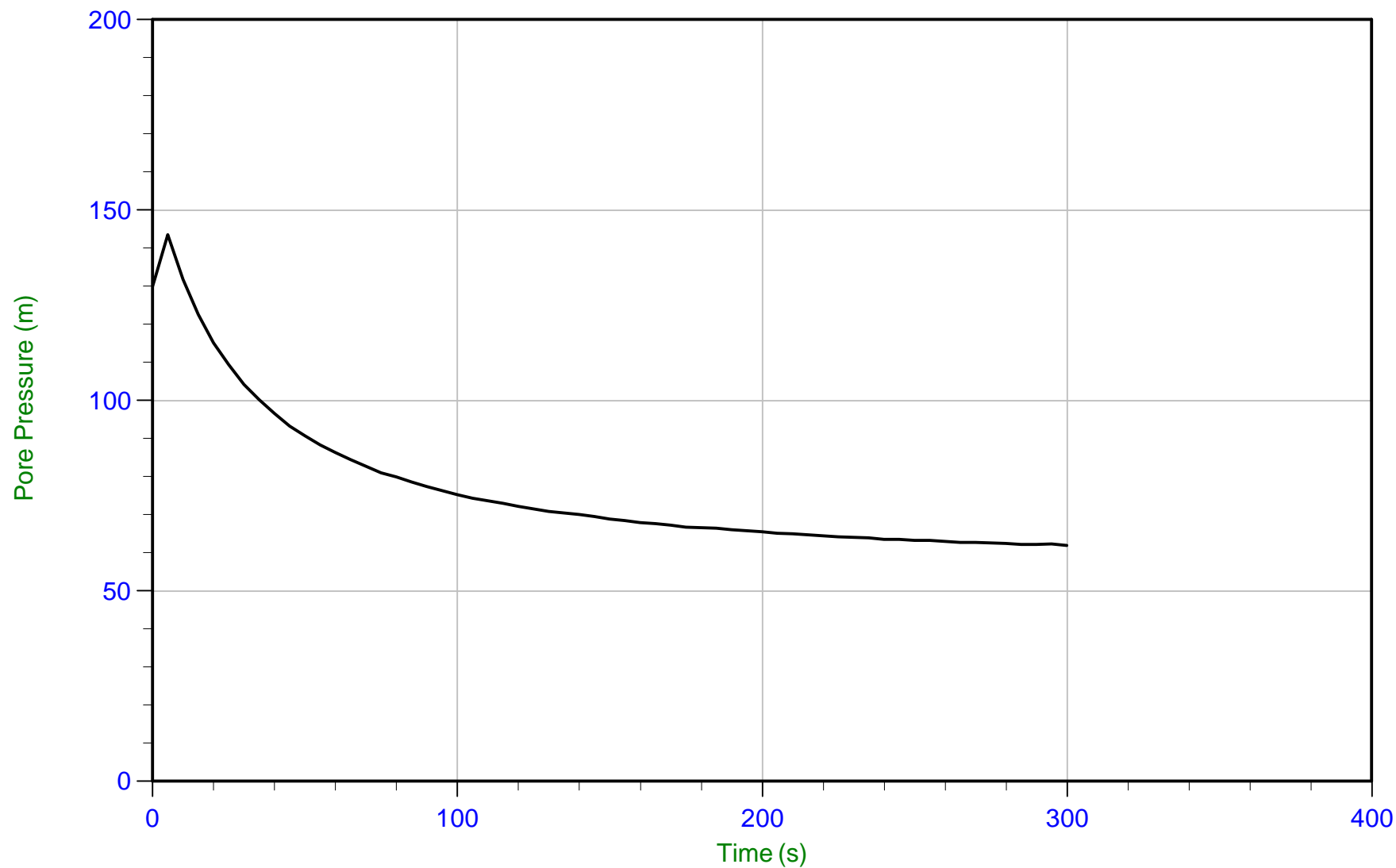
Job No: 19-05015

Date: 02/28/2019 11:12

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: CPT18-01B

Cone: 560:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 19-05015\_CP01B.PPF  
Depth: 48.000 m / 157.478 ft  
Duration: 300.0 s

u Min: 62.0 m  
u Max: 143.6 m  
u Final: 62.0 m

WT: 2.500 m / 8.202 ft  
Ueq: 45.5 m  
U(50): 94.55 m

T(50): 38.1 s  
Ir: 100  
Ch: 18.4 cm<sup>2</sup>/min





*Golder Associates*

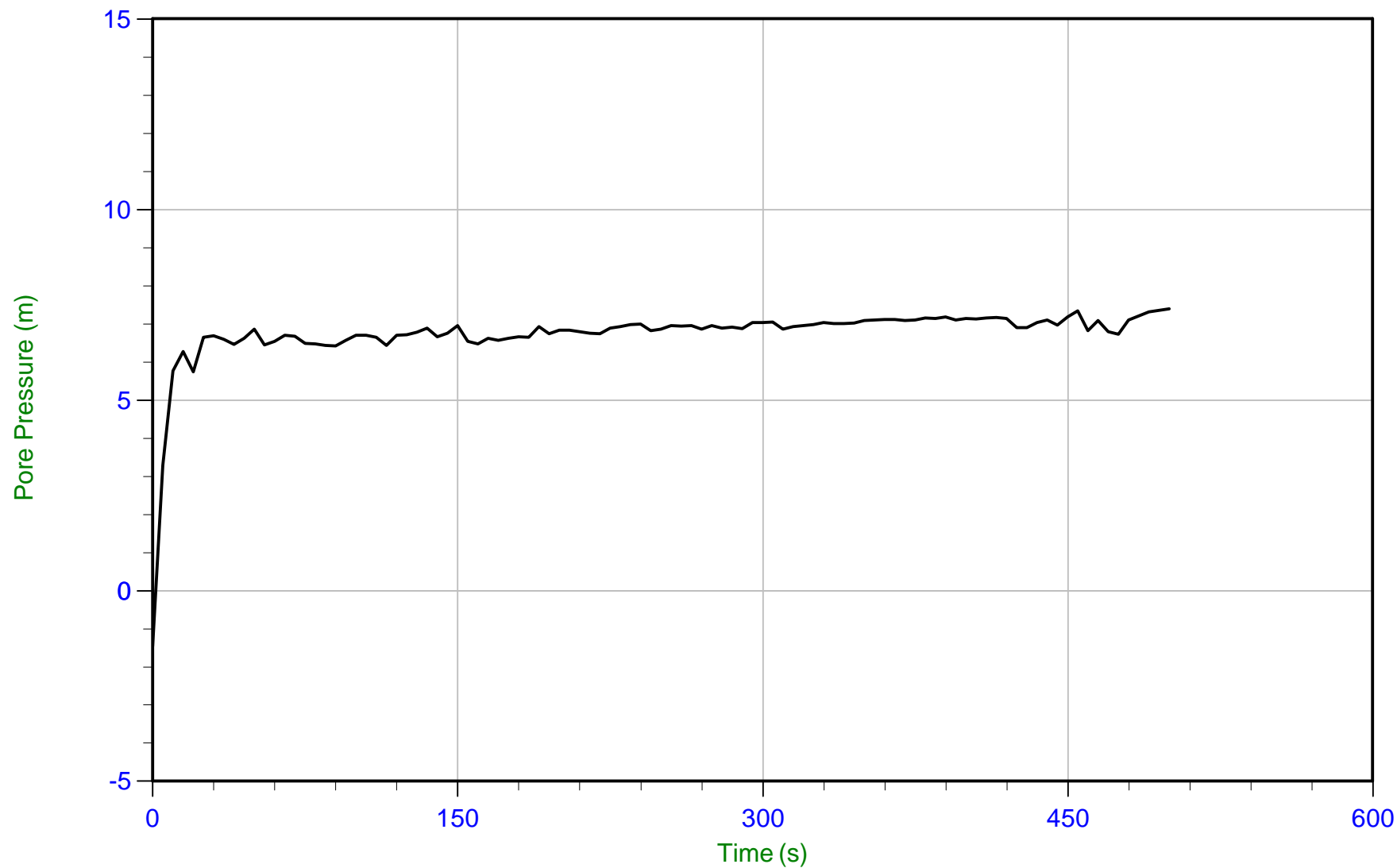
Job No: 19-05015

Date: 03/08/2019 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 19-05015\_SP02.PPF

Depth: 7.250 m / 23.786 ft

Duration: 500.0 s

u Min: -1.5 m

u Max: 7.4 m

u Final: 7.4 m

WT: 0.303 m / 0.994 ft

Ueq: 6.9 m





*Golder Associates*

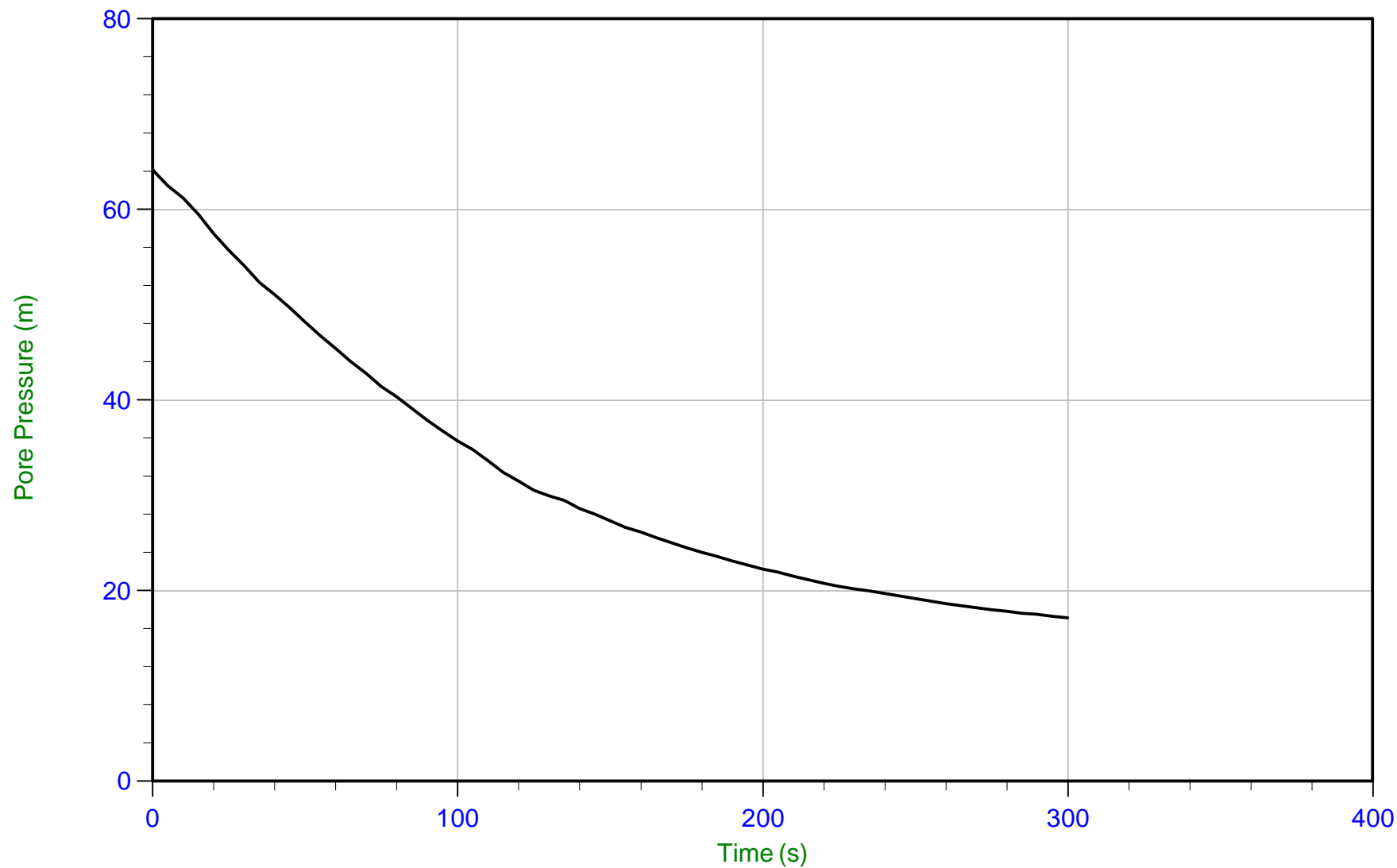
Job No: 19-05015

Date: 03/08/2019 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 19-05015\_SP02.PPF

Depth: 10.500 m / 34.448 ft

Duration: 300.0 s

u Min: 17.2 m

u Max: 64.2 m

u Final: 17.2 m

WT: 0.303 m / 0.994 ft

Ueq: 10.2 m

U(50): 37.19 m

T(50): 93.2 s

Ir: 100

Ch: 7.5 cm<sup>2</sup>/min





*Golder Associates*

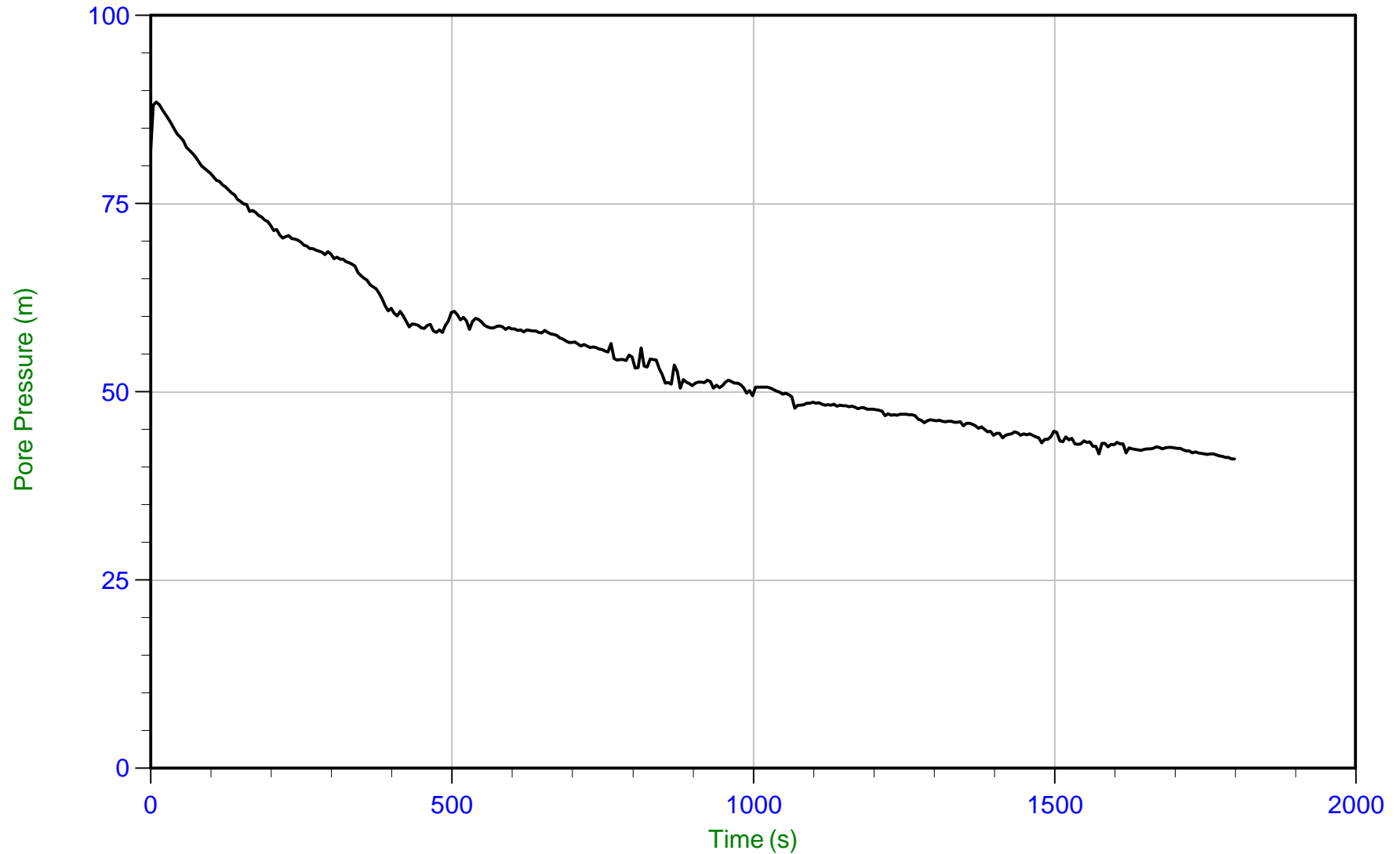
Job No: 19-05015

Date: 03/08/2019 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 19-05015\_SP02.PPF

Depth: 13.000 m / 42.650 ft

Duration: 1800.0 s

u Min: 41.1 m

u Max: 88.5 m

u Final: 41.1 m

WT: 0.303 m / 0.994 ft

Ueq: 12.7 m

U(50): 50.60 m

T(50): 869.9 s

Ir: 100

Ch: 0.8 cm<sup>2</sup>/min





*Golder Associates*

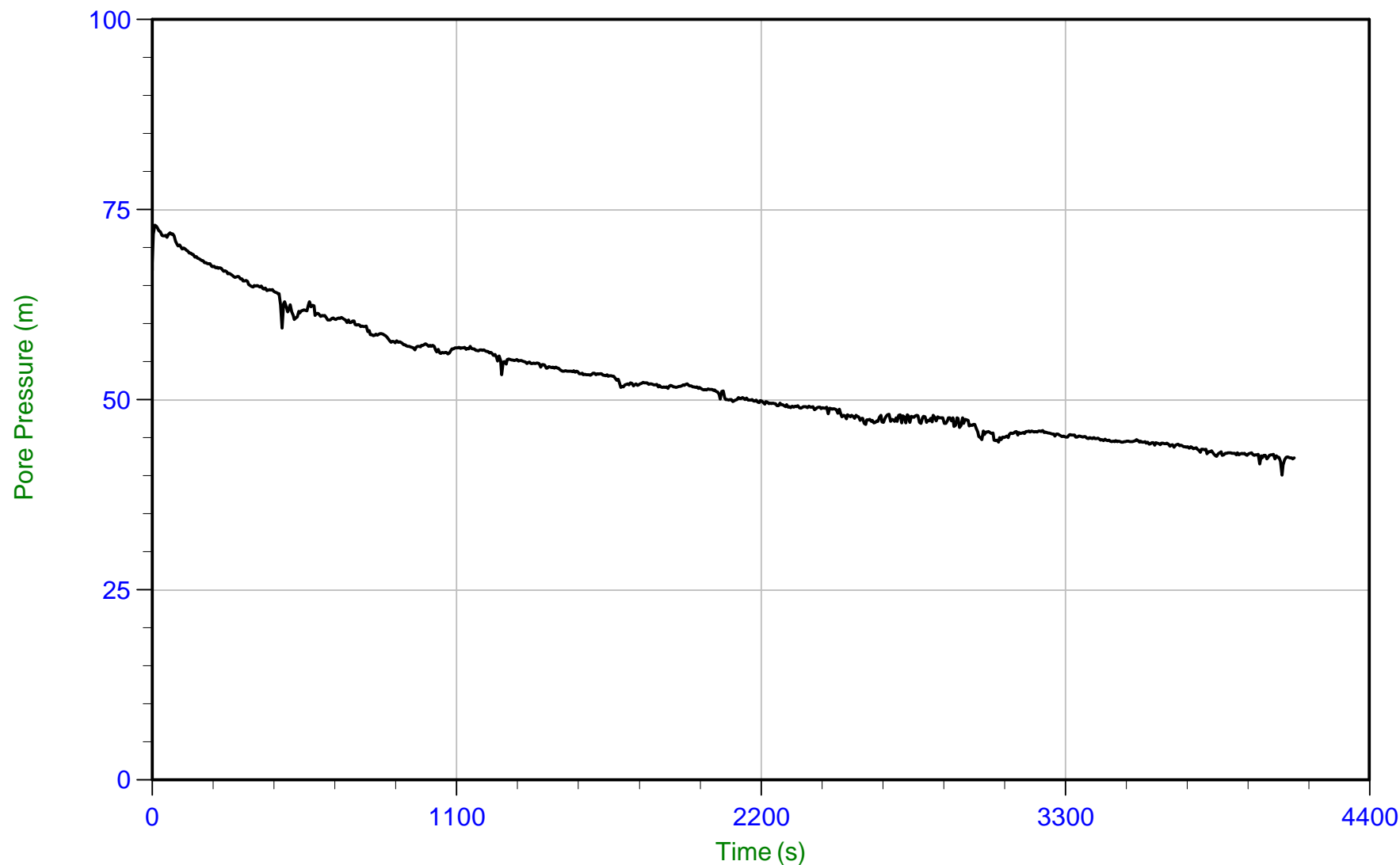
Job No: 19-05015

Date: 03/08/2019 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 19-05015\_SP02.PPF

Depth: 16.000 m / 52.493 ft

Duration: 4130.0 s

u Min: 40.1 m

u Max: 72.9 m

u Final: 42.3 m

WT: 0.303 m / 0.994 ft

Ueq: 15.7 m

U(50): 44.32 m

T(50): 3592.0 s

Ir: 100

Ch: 0.2 cm<sup>2</sup>/min





*Golder Associates*

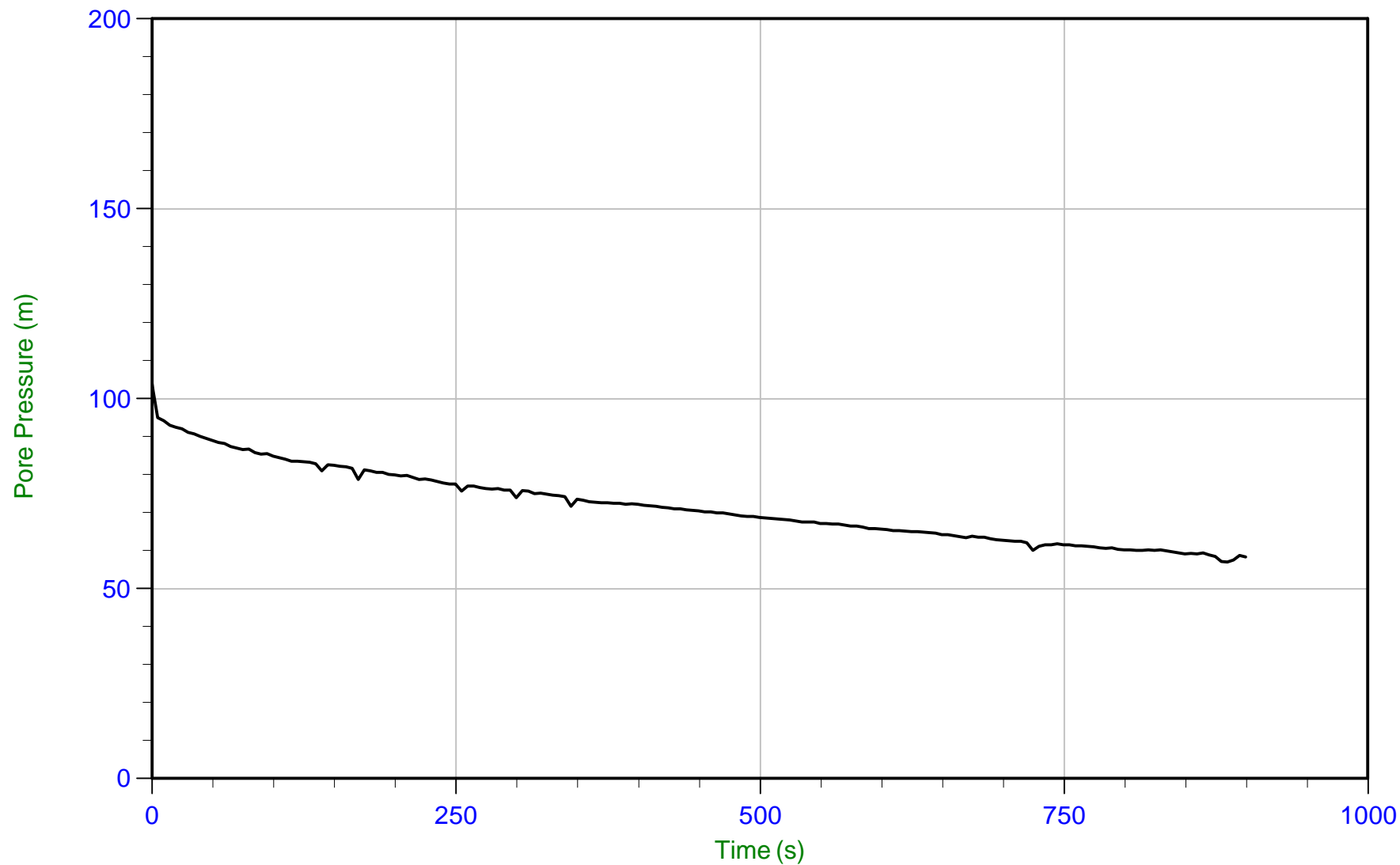
Job No: 19-05015

Date: 03/08/2019 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 19-05015\_SP02.PPF

Depth: 22.000 m / 72.178 ft

Duration: 900.0 s

u Min: 57.0 m

u Max: 103.9 m

u Final: 58.4 m

WT: 0.303 m / 0.994 ft

Ueq: 21.7 m

U(50): 62.79 m

T(50): 697.5 s

Ir: 100

Ch: 1.0 cm<sup>2</sup>/min





*Golder Associates*

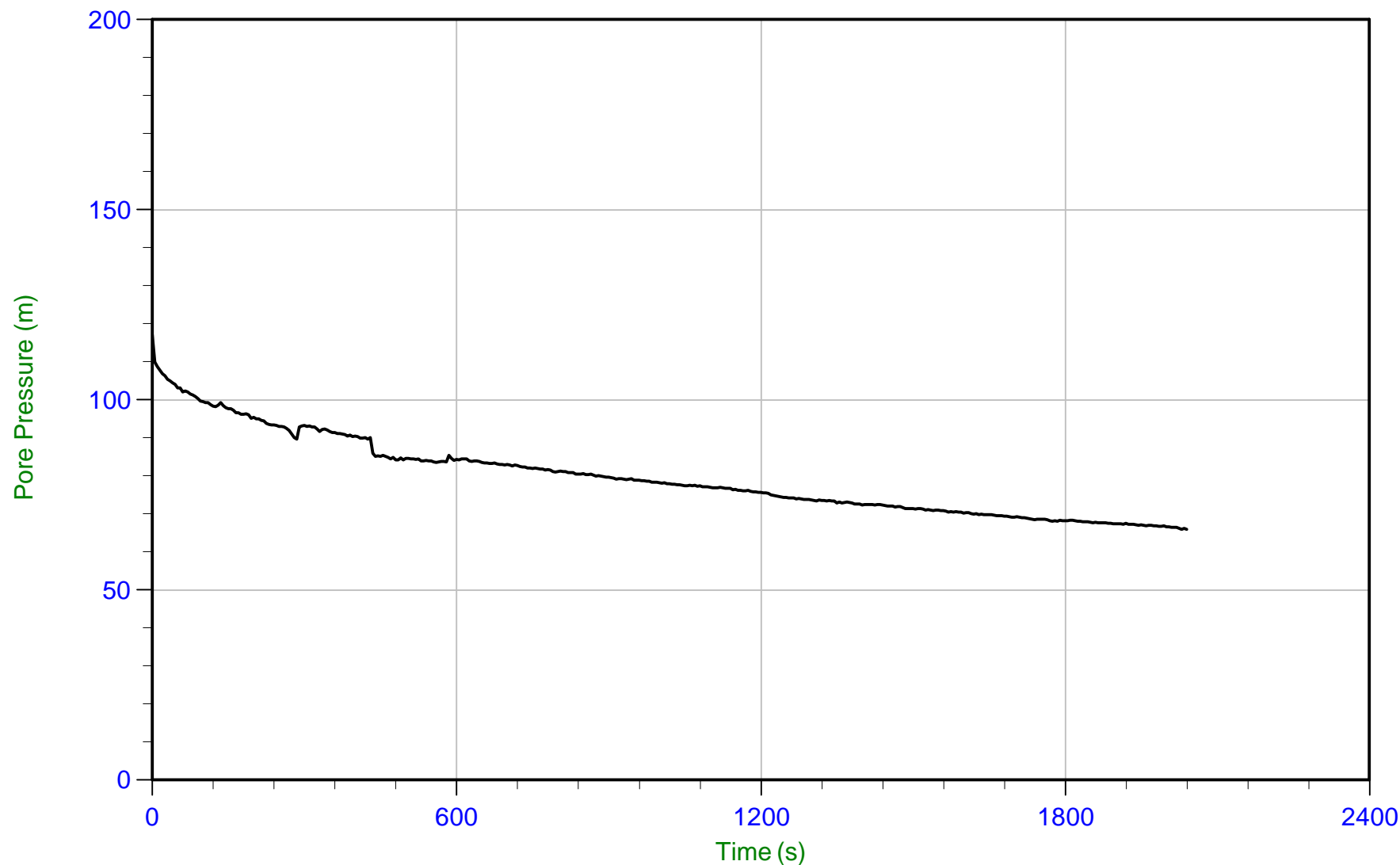
Job No: 19-05015

Date: 03/08/2019 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 19-05015\_SP02.PPF

Depth: 26.000 m / 85.301 ft

Duration: 2040.0 s

u Min: 65.9 m

u Max: 117.1 m

u Final: 65.9 m

WT: 0.303 m / 0.994 ft

Ueq: 25.7 m

U(50): 71.42 m

T(50): 1483.6 s

Ir: 100

Ch: 0.5 cm<sup>2</sup>/min





*Golder Associates*

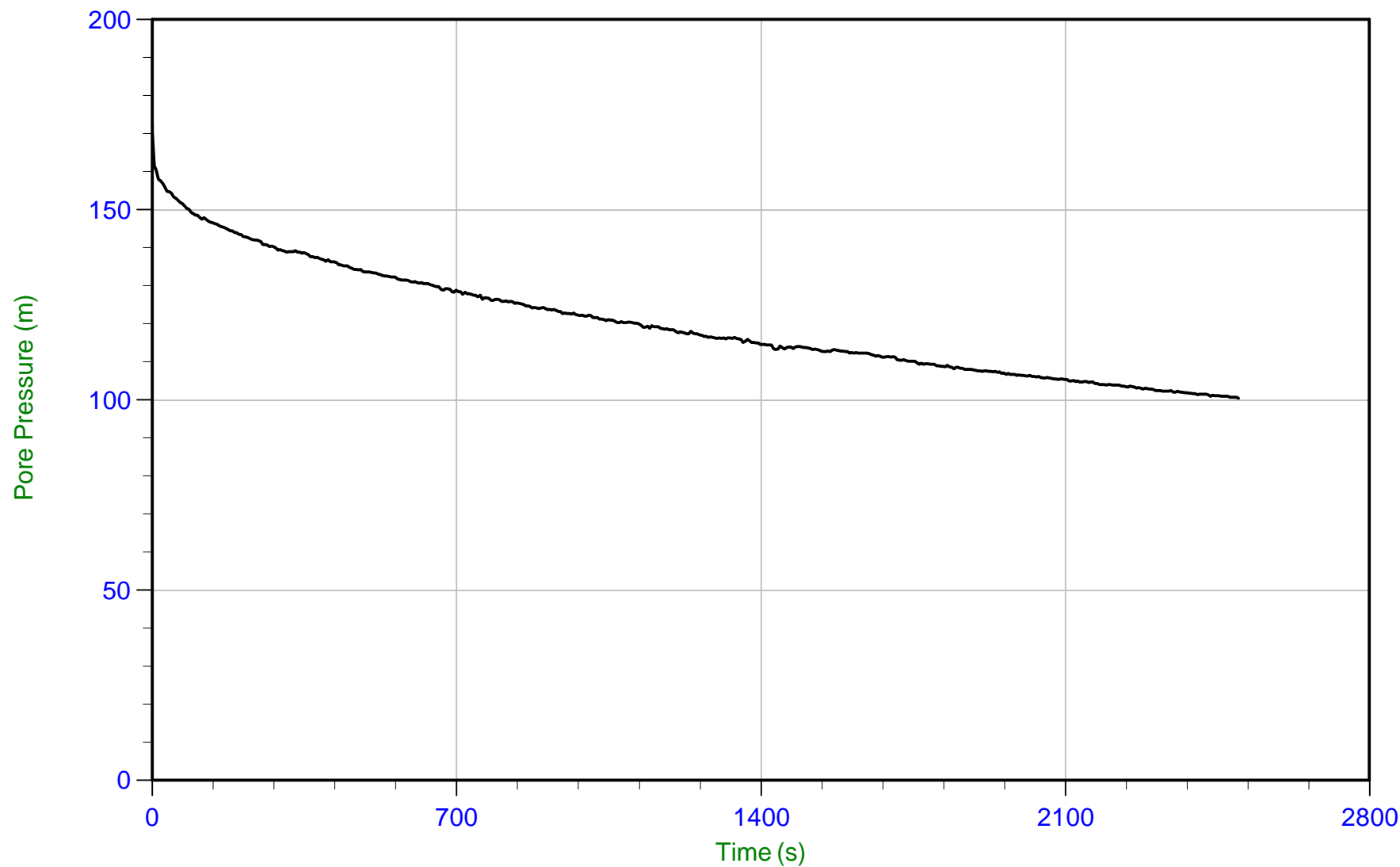
Job No: 19-05015

Date: 03/08/2019 10:37

Site: Hwy 9 and Hwy 8 Schomberg ON

Sounding: SCPT19-02

Cone: 545:T1500F15U500 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 19-05015\_SP02.PPF

Depth: 37.000 m / 121.390 ft

Duration: 2500.0 s

u Min: 100.5 m

u Max: 172.7 m

u Final: 100.5 m

WT: 0.303 m / 0.994 ft

Ueq: 36.7 m

U(50): 104.70 m

T(50): 2134.7 s

Ir: 100

Ch: 0.3 cm<sup>2</sup>/min



**APPENDIX C**

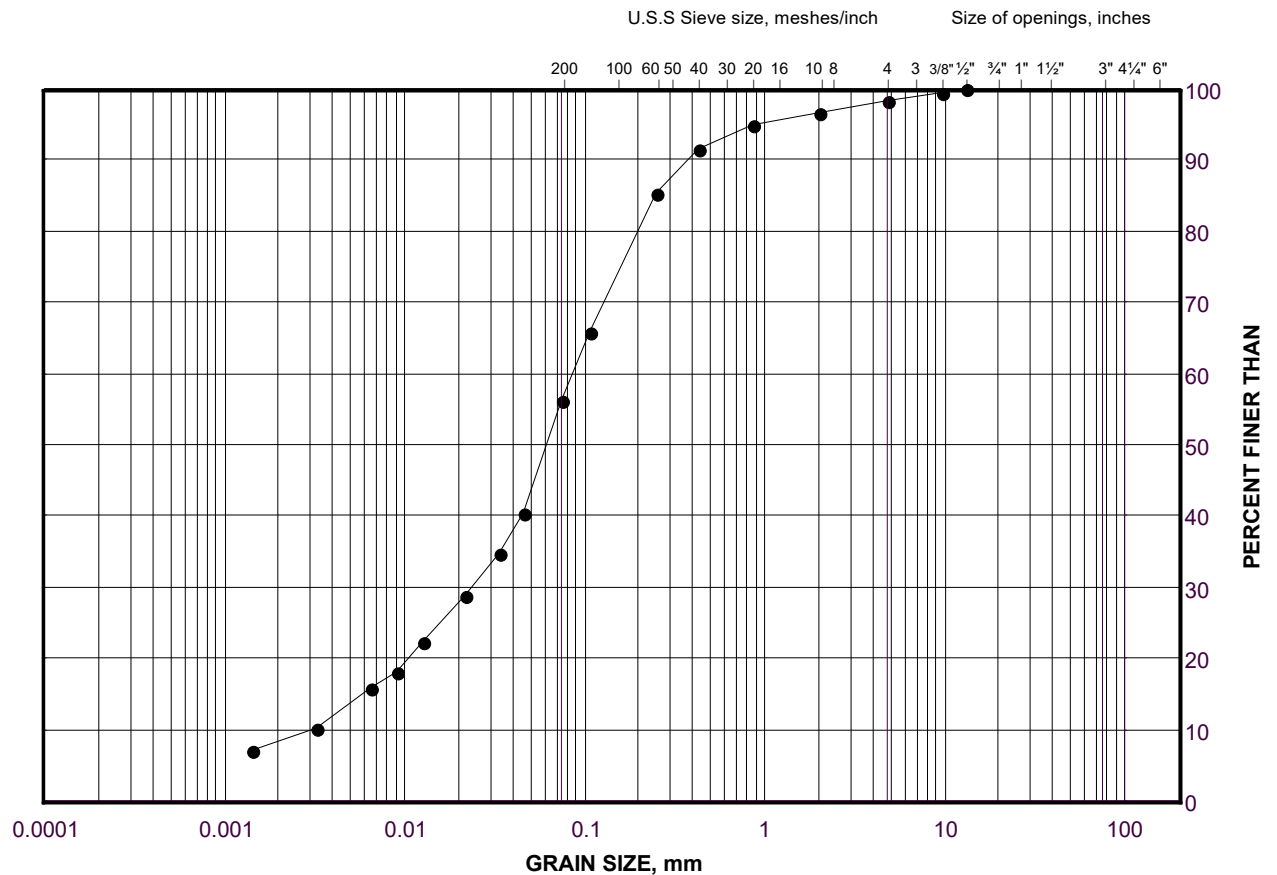
# Geotechnical Laboratory Test Results



# GRAIN SIZE DISTRIBUTION

Silt and Sand (Fill)

FIGURE C1



SILT AND CLAY SIZES			FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED			SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	A2-1	4	218.0

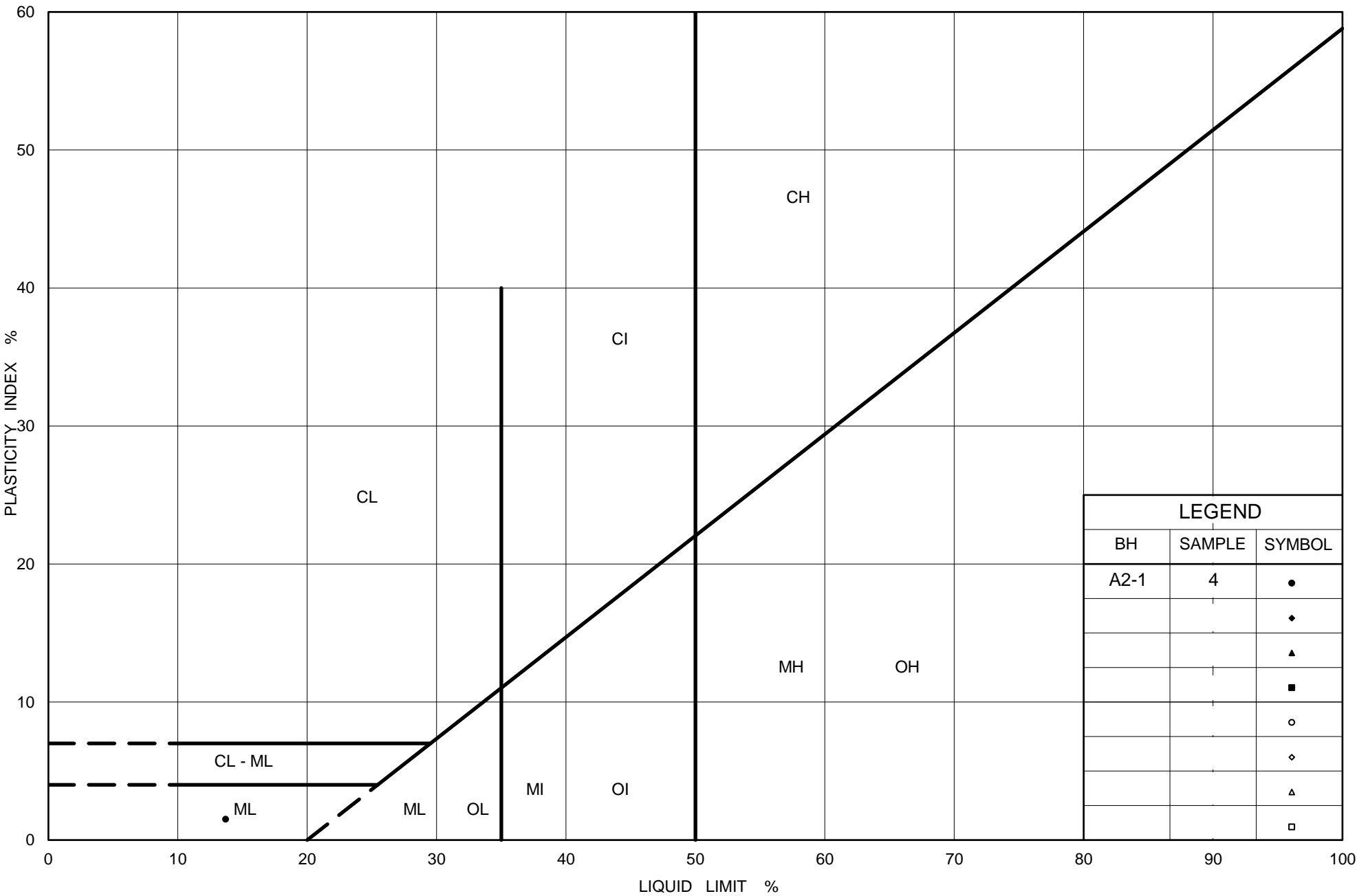
Project Number: 1671430

Checked By: \_\_\_\_\_

**Golder Associates**

Date: 04-Jul-19



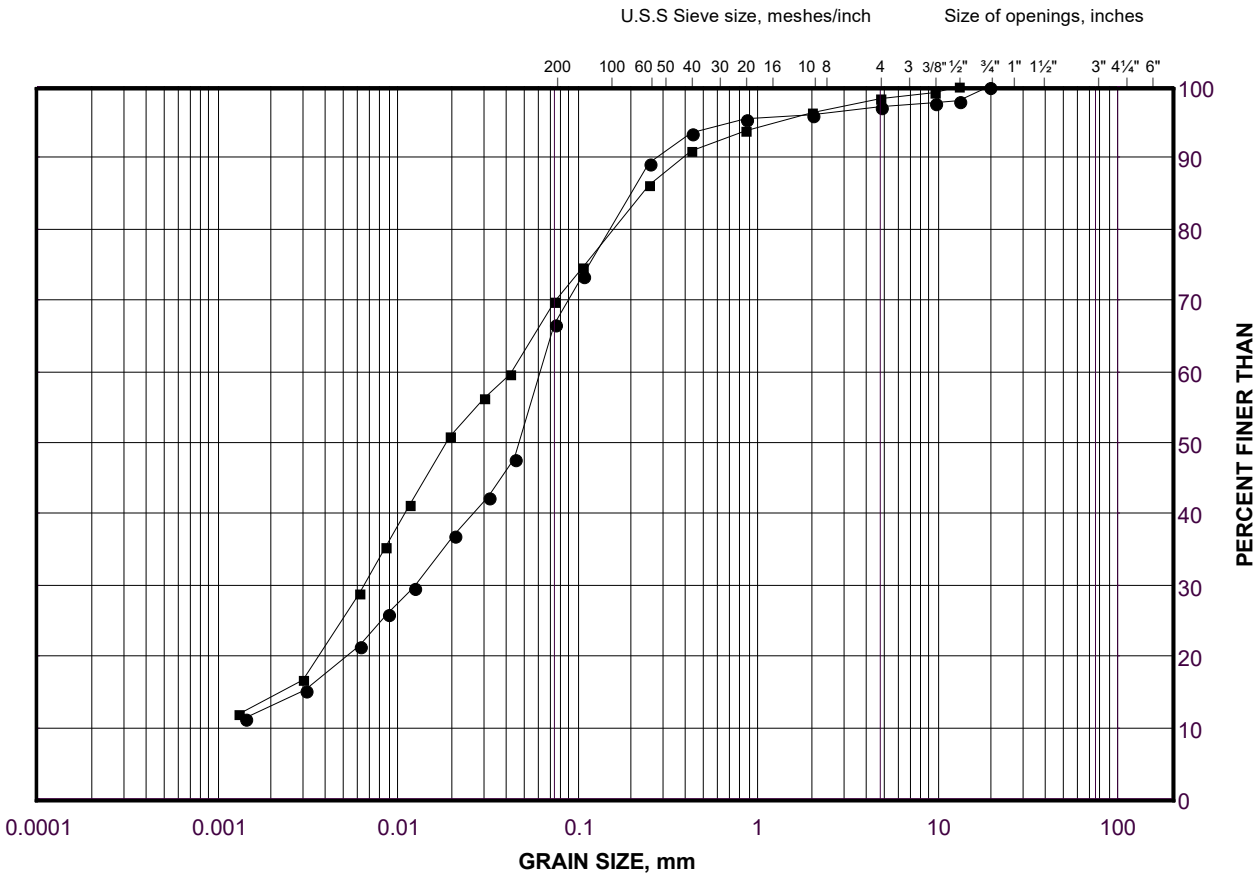




# GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Fill)

FIGURE C3

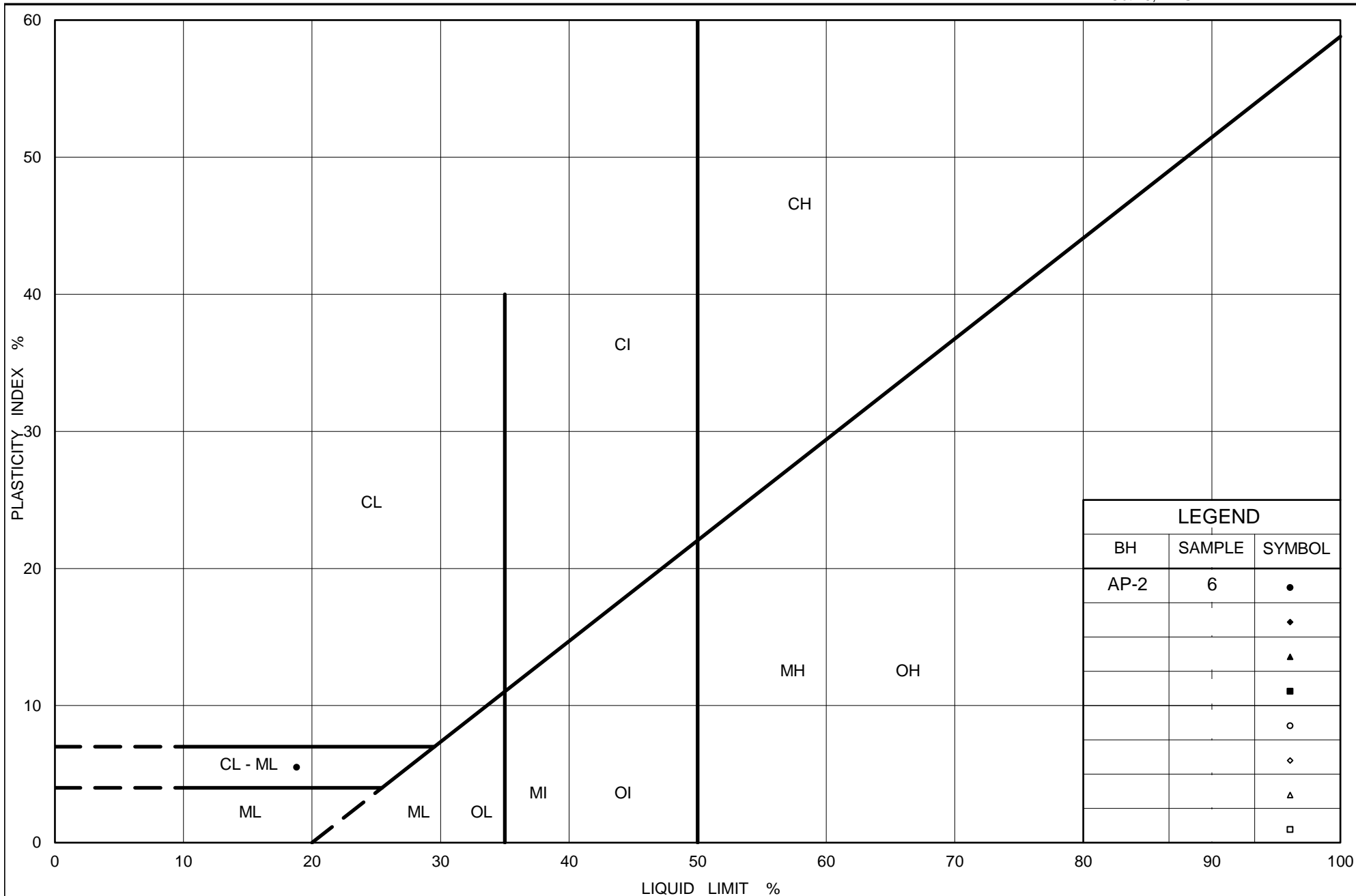


SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AP-2	3	218.9
■	AP-2	6	216.6





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# **PLASTICITY CHART** **Clayey Silt with Sand (FILL)**

Figure No. C4

Project No. 1671430

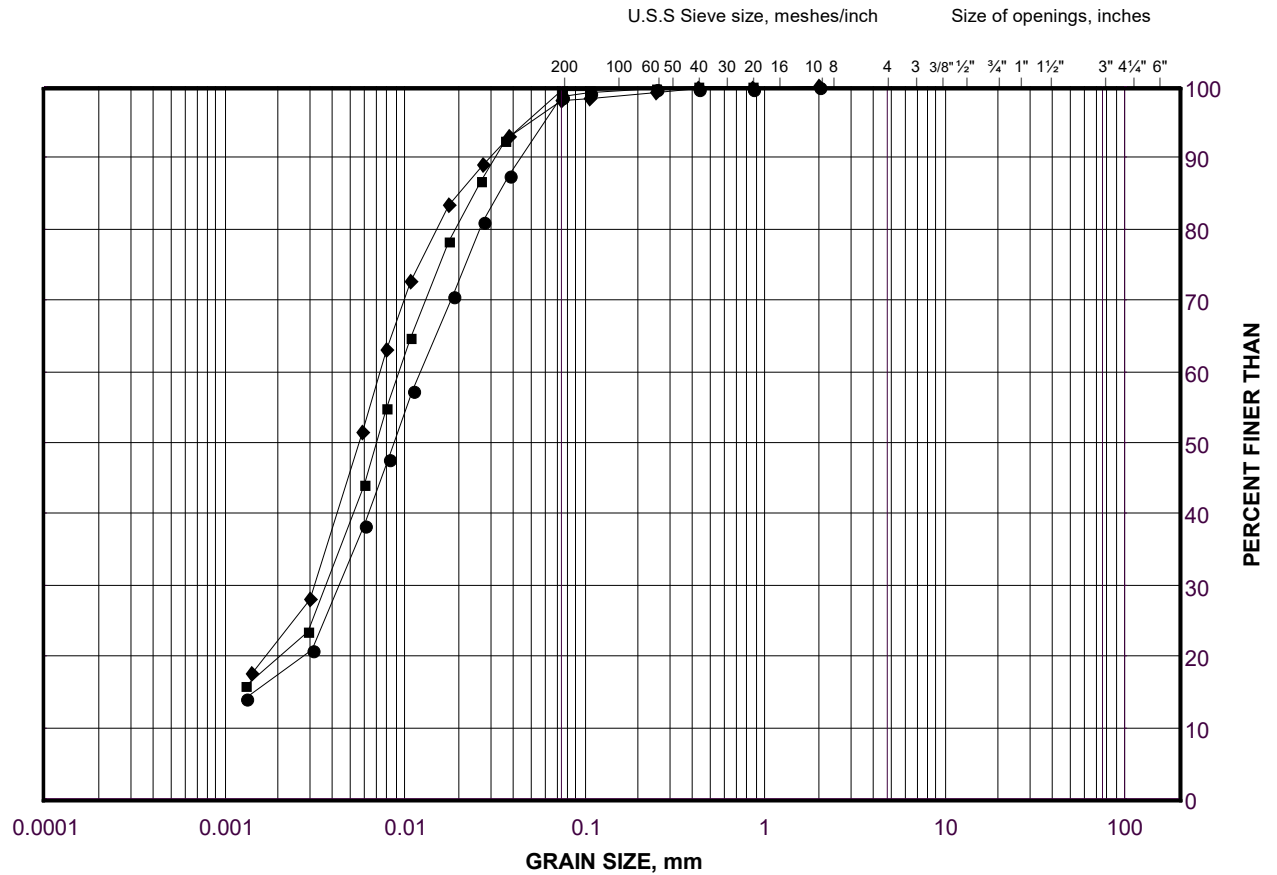
Checked By:



# GRAIN SIZE DISTRIBUTION

Upper Silty Clay to Clayey Silt

FIGURE C5



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	A2-2	4	216.2
■	A1-1	6	215.8
◆	A1-2	6A	215.3

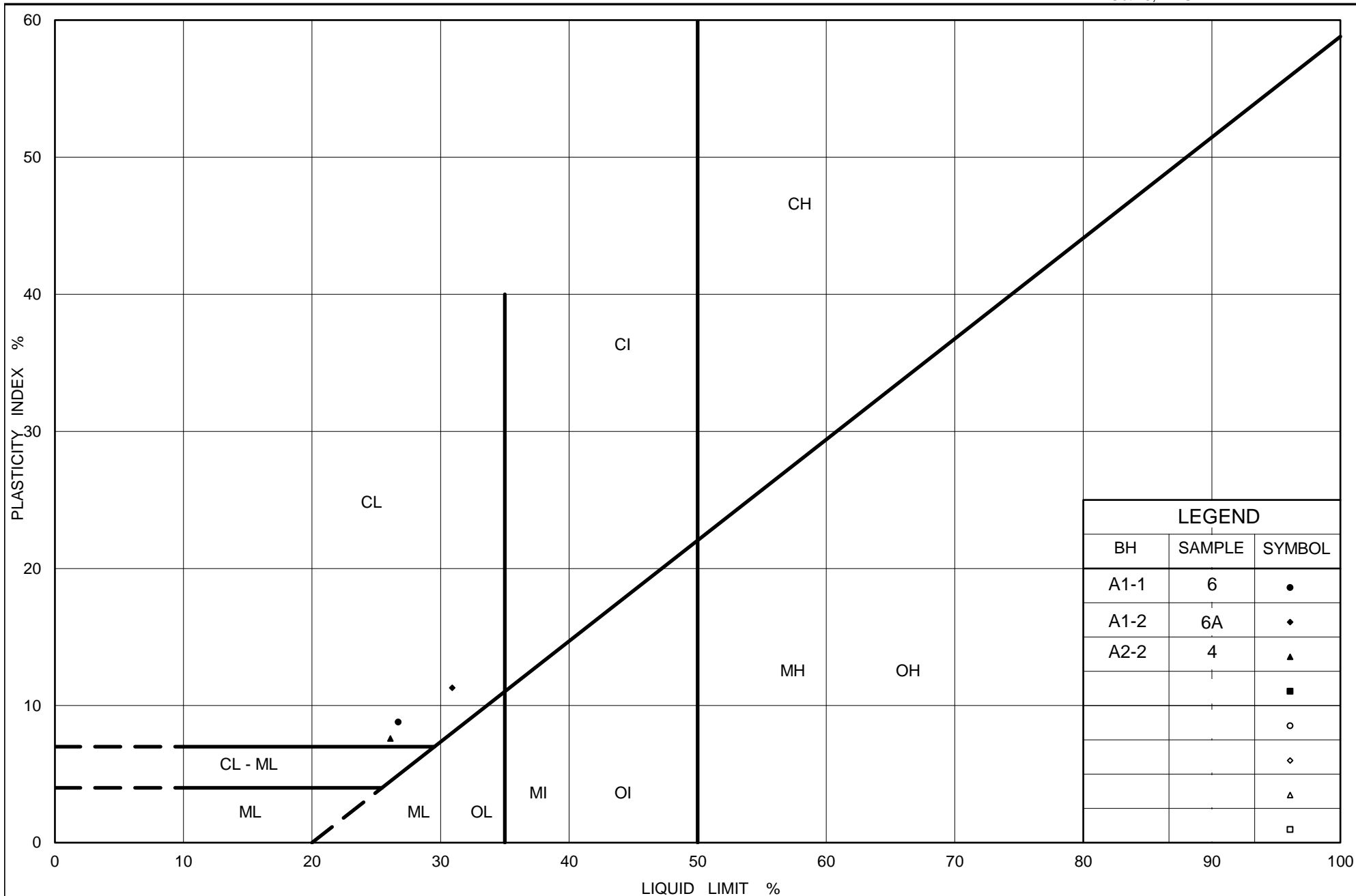
Project Number: 1671430

Checked By: \_\_\_\_\_

**Golder Associates**

Date: 04-Jul-19





Ministry of Transportation

Ontario

## PLASTICITY CHART

### Upper Clayey Silt

Figure No. C6

Project No. 1671430

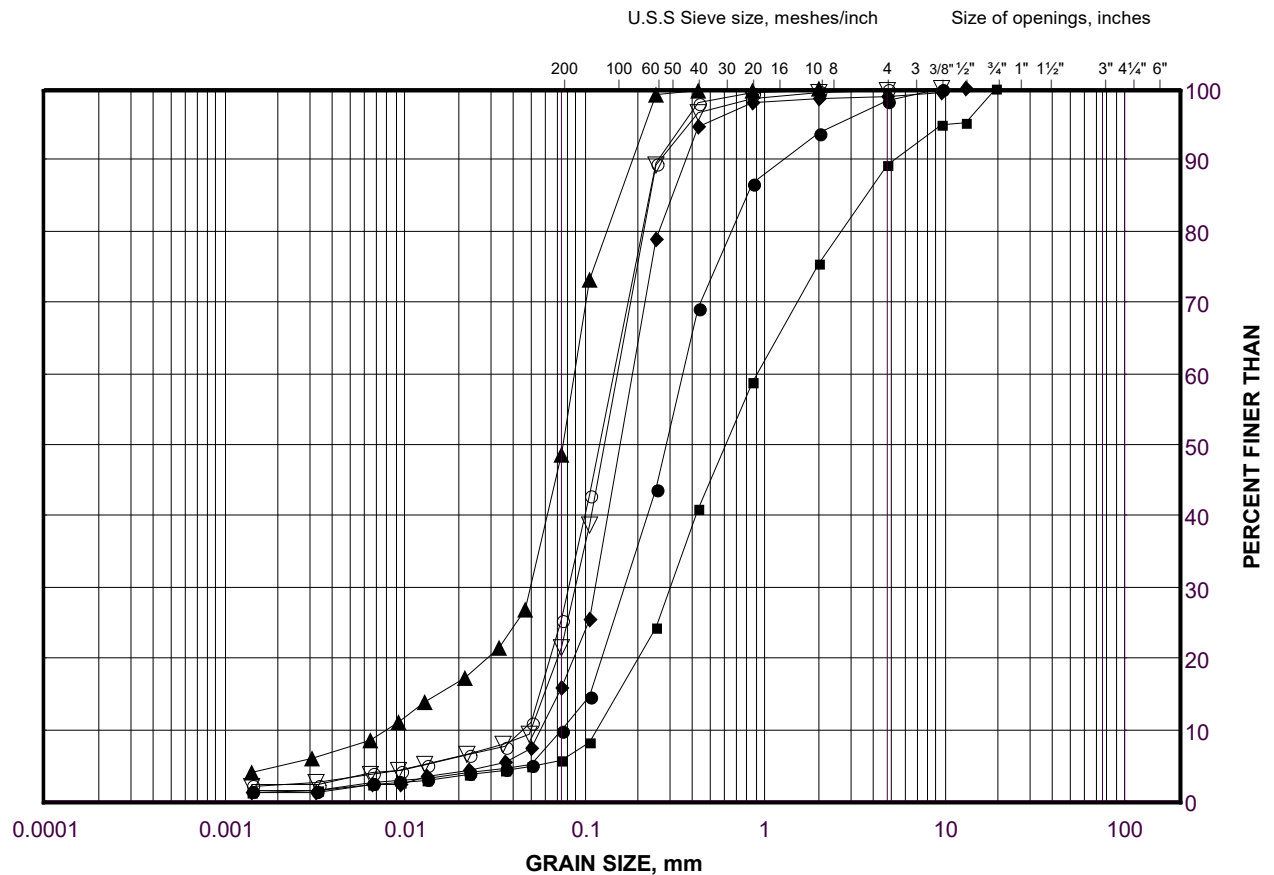
Checked By:



# GRAIN SIZE DISTRIBUTION

Silt and Sand to Sand

FIGURE C7



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AP-2	10	211.2
■	A2-1	10	211.2
◆	A2-2	6	212.4
▲	AP-1	7	213.2
▽	A1-1	8	212.8
○	A1-2	8	212.8

Project Number: 1671430

Checked By: \_\_\_\_\_

**Golder Associates**

Date: 04-Jul-19



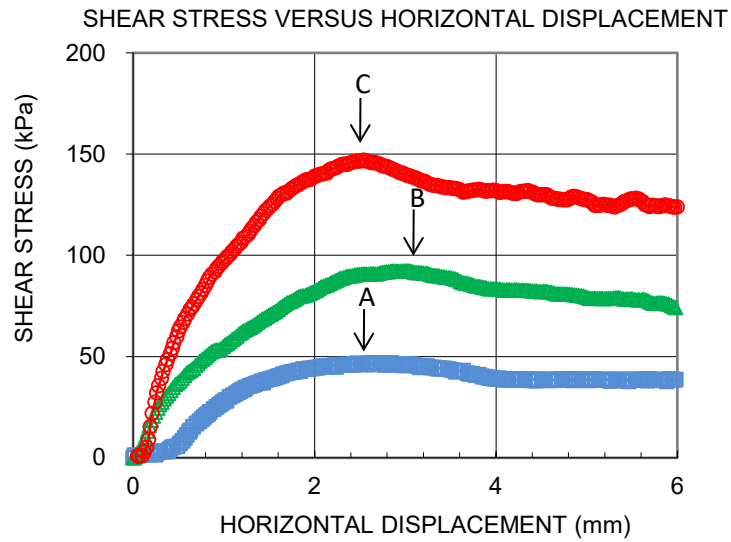
CONSOLIDATED DRAINED DIRECT SHEAR TEST ASTM D3080 SHEET 1 OF 3		FIGURE C8A Silty Sand to Sand	
TEST STAGE	A	B	C
BOREHOLE NUMBER	A1-2		
SAMPLE	7		
SAMPLE DEPTH, (m)	4.57-5.18		
SAMPLE HEIGHT, (mm)	24.94	25.11	24.25
SAMPLE LENGTH, (mm)	60.00	60.00	60.00
WATER CONTENT, BEFORE TEST, (%)	17.1	17.1	17.1
NORMAL (CONSOLIDATION) STRESS, (kPa)	50	100	150
WATER CONTENT, AFTER TEST, (%)	19.4	19.2	19.8
DISPLACEMENT RATE, mm/min	0.0048	0.0048	0.0048
TIME TO FAILURE, hours	9.2	10.0	8.8
PEAK SHEAR STRESS <sup>1</sup> , (kPa)	46.8	92.3	147.1
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	2.6	2.9	2.5
DRY DENSITY, initial, Mg/m <sup>3</sup>	1.686	1.666	1.722
WET DENSITY, initial, Mg/m <sup>3</sup>	1.974	1.951	2.016
TEST NOTES: <div><div>1</div><div>In the absence of a peak, the shear stress reported is at 10 percent relative horizontal displacement (ASTM D3080).</div></div> <div><div>2</div><div>Specimens compacted at target dry density of 1.682Mg/m<sup>3</sup> and 16% water content.</div></div>			
Date: 3/31/2019 Project No. 1671430		Prepared By: LH Checked By:	
Golder Associates			



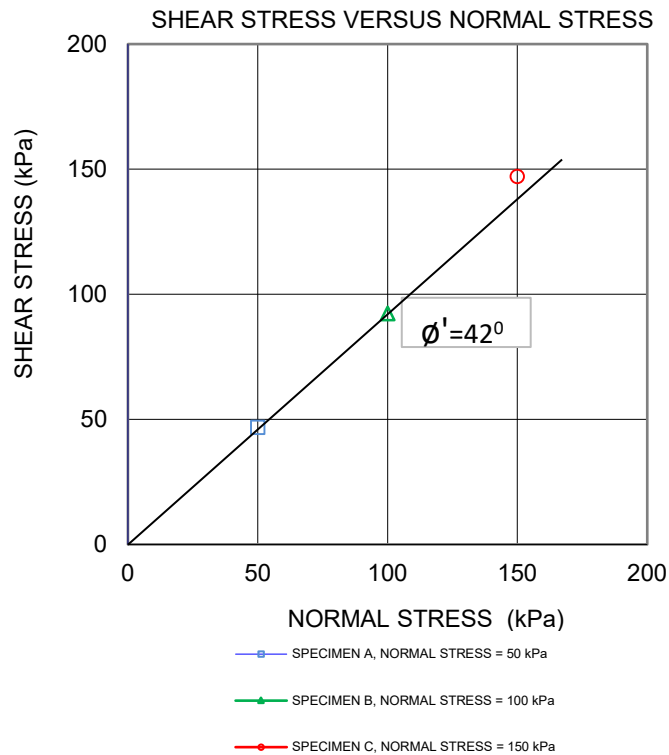
**CONSOLIDATED DRAINED DIRECT SHEAR TEST**  
**ASTM D3080**  
**SHEET 2 OF 3**

**FIGURE**  
**C8B**  
Silty Sand to Sand

BH A1-2 SA 7



BH A1-2 SA 7



Date: 3/31/2019

Project No. 1671430

**Golder Associates**

Prepared By LH

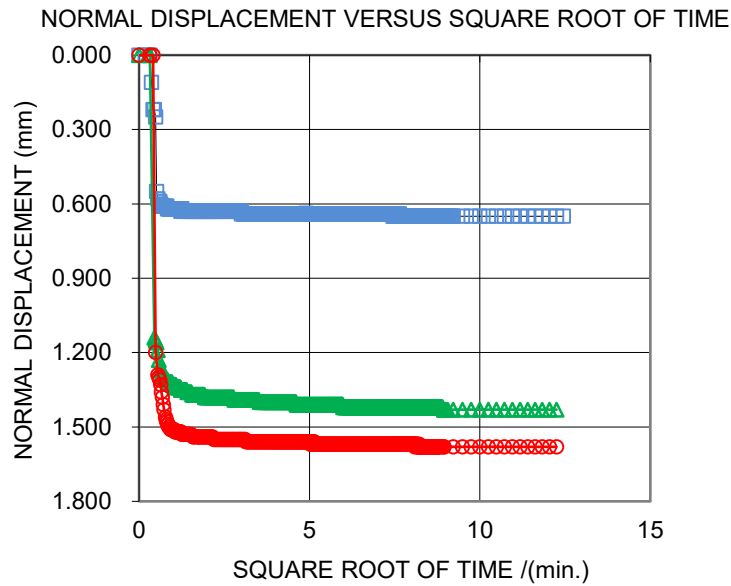
Checked By:



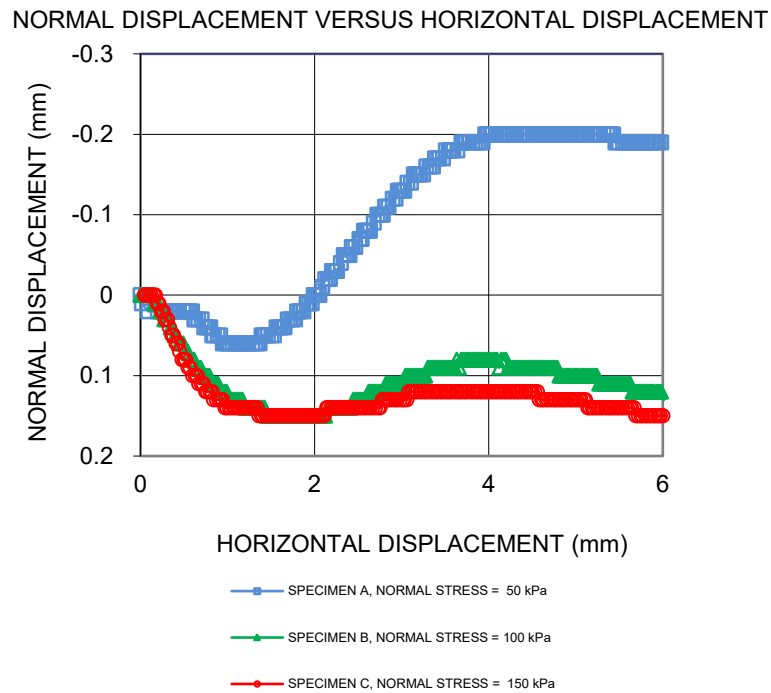
**CONSOLIDATED DRAINED DIRECT SHEAR TEST**  
**ASTM D3080**  
**SHEET 3 OF 3**

**FIGURE**  
**C8C**  
Silty Sand to Sand

BH A1-2 SA 7



BH A1-2 SA 7



Date: 3/31/2019  
Project No. 1671430

**Golder Associates**

Prepared By LH  
Checked By:



<b>CONSOLIDATED DRAINED TRIAXIAL</b> <b>ASTM D7181</b> <b>SHEET 1 OF 4</b>		<b>FIGURE</b> <b>C9A</b> <b>Sand</b>
TEST STAGE	A	
BOREHOLE NUMBER	AP-2	
SAMPLE NUMBER	SS9-SS10	
DEPTH, ft	-	
SPECIMEN DIAMETER, cm	5.04	
SPECIMEN HEIGHT, cm	10.12	
NATURAL WATER CONTENT, %	15.0	
DRY DENSITY, Mg/m <sup>3</sup>	1.74	
WATER CONTENT BEFORE CONSOLIDATION, %	16.2	
CELL PRESSURE, $\sigma_3$ , kPa	600.0	
BACK PRESSURE, kPa	550.0	
PORE PRESSURE PARAMETER "B"	0.94	
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa	50.0	
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	0.0	
WATER CONTENT AFTER CONSOLIDATION, %	16.1	
AVERAGE RATE OF STRAIN, %/hr	0.5	
TIME TO FAILURE, HOURS	11	
WATER CONTENT AFTER TEST, %	16.2	
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	166.0	
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	5.4	
MAX PRINCIPAL STRESS RATIO, $(\sigma'_1 / \sigma'_3)$ maximum	4.2	
FILTER DRAINS USED, y/n	y	
TEST NOTES:  <div style="margin-left: 40px;">Specimen A was combined sample compacted at 15% water content and density of 1.742g/cm<sup>3</sup>.</div>		
FAILURE PLANE NUMBER	-	
ANGLE OF FAILURE, DEGREES	-	
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CONSOLIDATED DRAINED TRIAXIAL

ASTM D7181

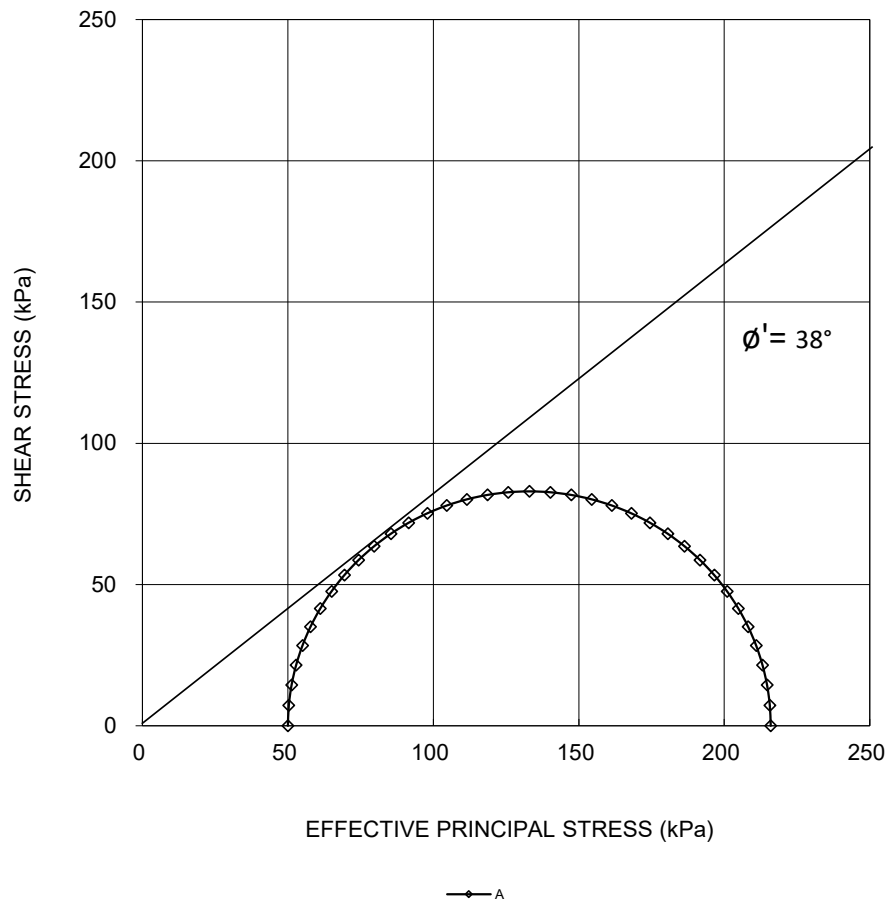
SHEET 2 OF 4

FIGURE

C9B

Sand

A2-2 TO1 & AP-2 SS9-SS10



Date: 4/22/2019

Project No. 1671430

**Golder Associates**

Prepared By LH

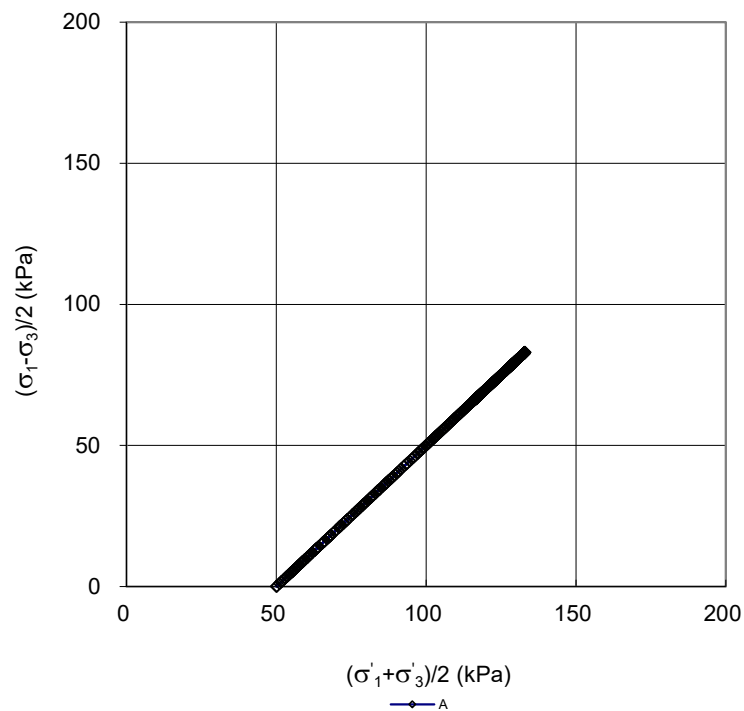
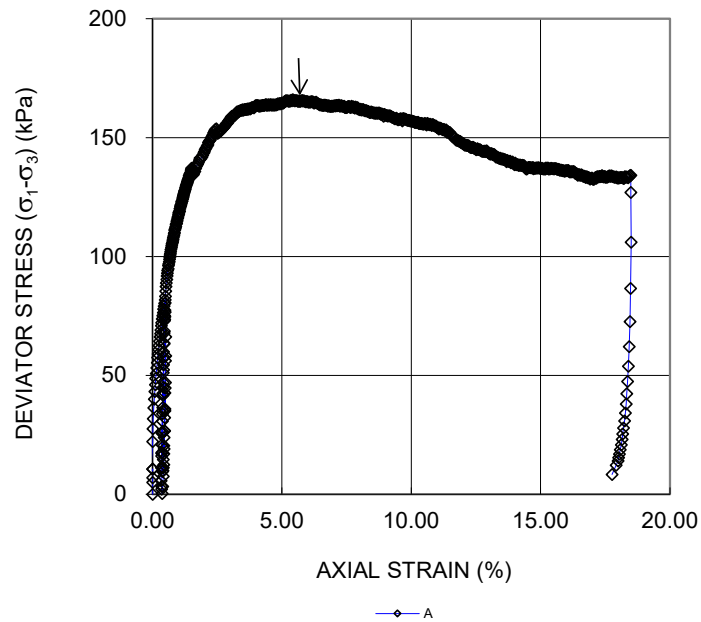
Checked By:



**CONSOLIDATED DRAINED TRIAXIAL  
ASTM D7181  
SHEET 3 OF 4**

**FIGURE  
C9C  
Sand**

A2-2 TO1 & AP-2 SS9-SS10



Date: 4/22/2019  
Project No. 1671430

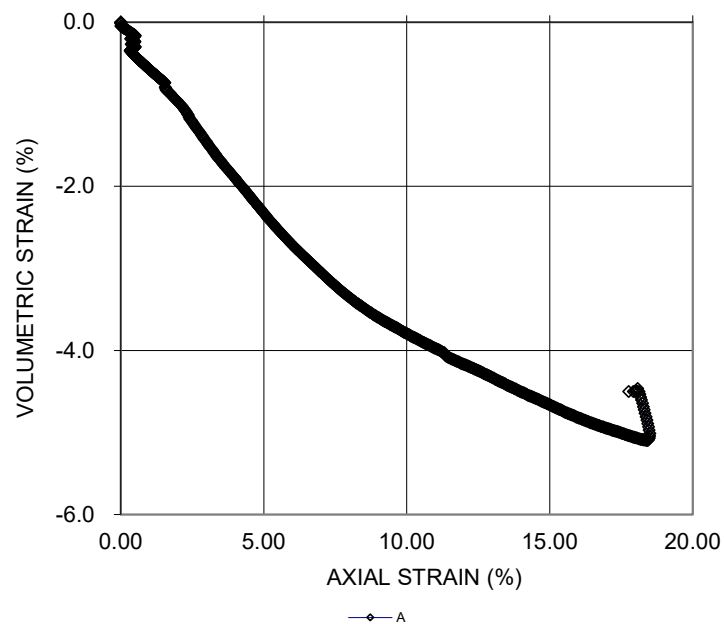
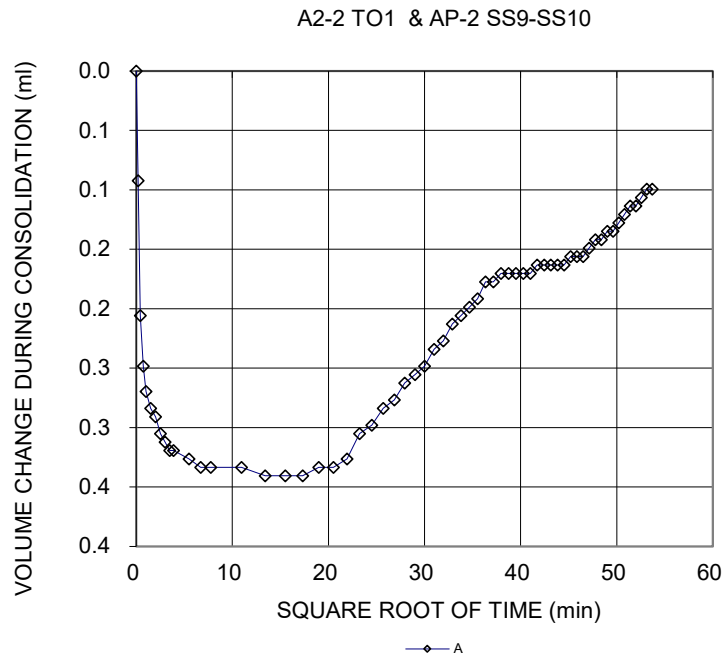
**Golder Associates**

Prepared By LH  
Checked By:



**CONSOLIDATED DRAINED TRIAXIAL  
ASTM D7181  
SHEET 4 OF 4**

**FIGURE  
C9D  
Sand**



NOTES: POSITIVE (+) VOLUMETRIC STRAIN = SAMPLE VOLUME DECREASING  
 NEGATIVE(-) VOLUMETRIC STRAIN = SAMPLE VOLUME INCREASING

Date: 4/22/2019  
 Project No. 1671430

**Golder Associates**

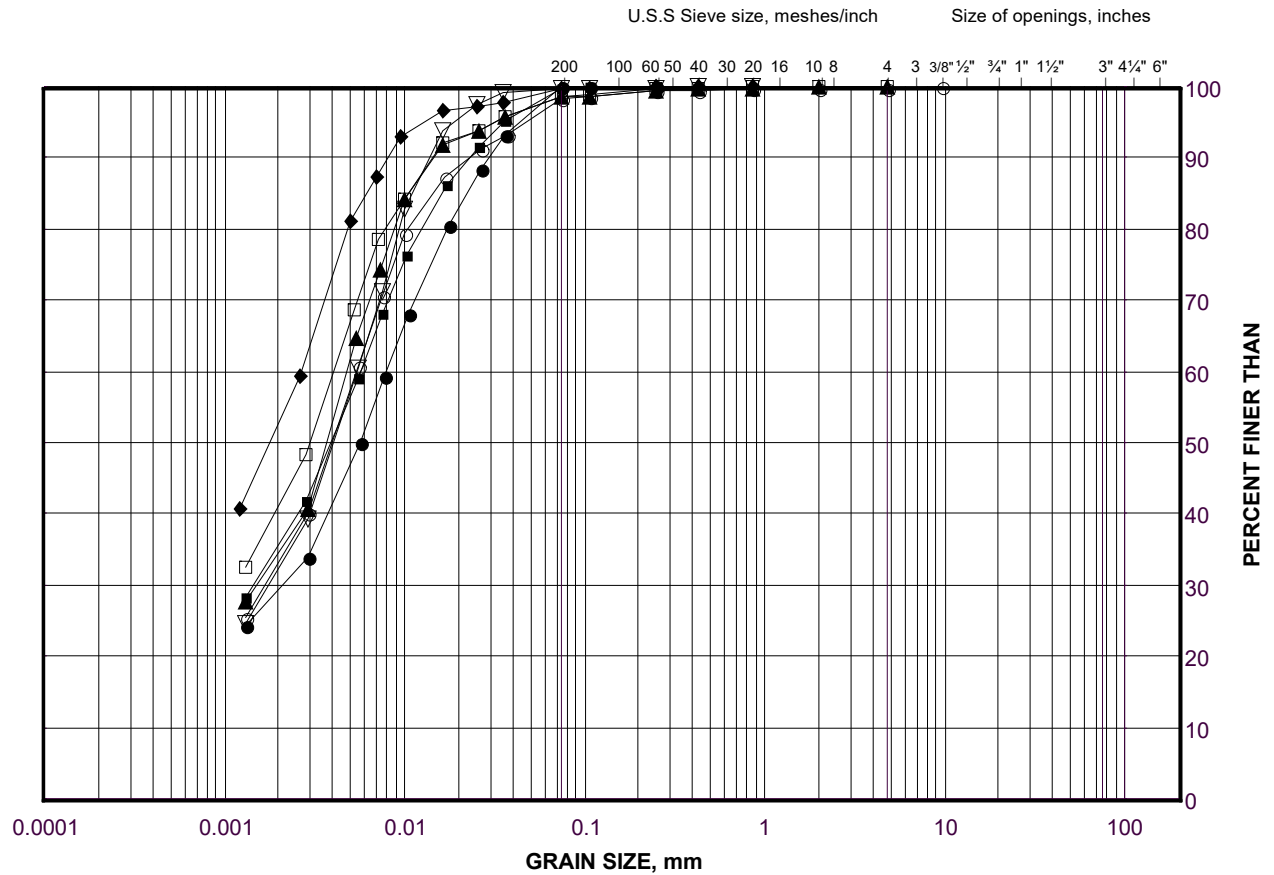
Prepared By LH  
 Checked By:



# GRAIN SIZE DISTRIBUTION

Lower Silty Clay to Clayey Silt

FIGURE C10A



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	A1-1	10	209.7
■	A1-2	11	208.3
◆	A1-2	14	202.2
▲	A1-1	16	189.9
▽	A1-2	16	197.6
○	A1-1	20	180.8
□	A1-2	21	185.4

Project Number: 1671430

Checked By: \_\_\_\_\_

**Golder Associates**

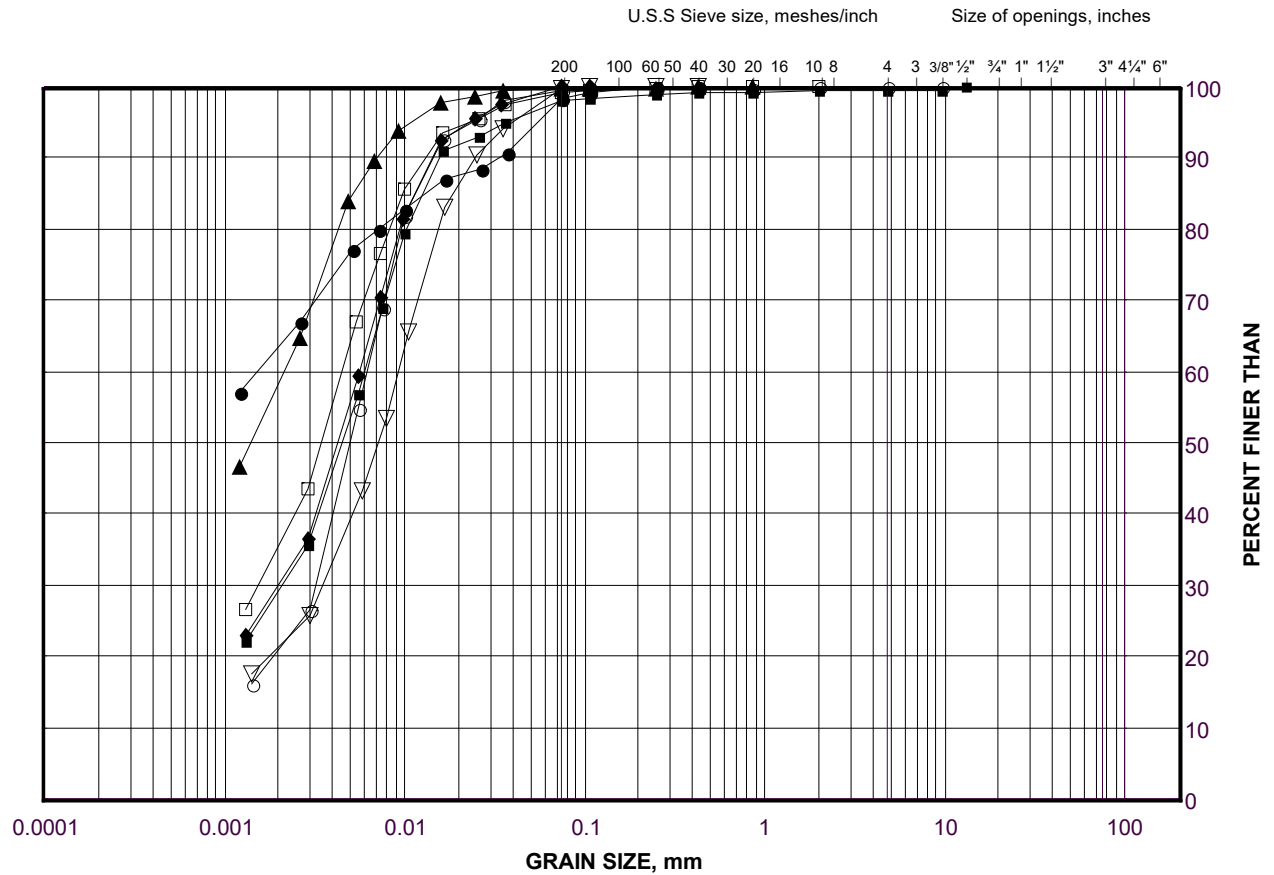
Date: 04-Jul-19



# GRAIN SIZE DISTRIBUTION

Lower Clayey Silt to Silty Clay

FIGURE C10B



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	A2-1	13	206.6
■	A2-1	18	197.4
◆	A2-1	22	191.3
▲	A2-1	24	186.8
▽	A1-2	25	173.2
○	A2-1	27	176.1
□	A2-1	29	168.5

Project Number: 1671430

Checked By: NK/JPD

**Golder Associates**

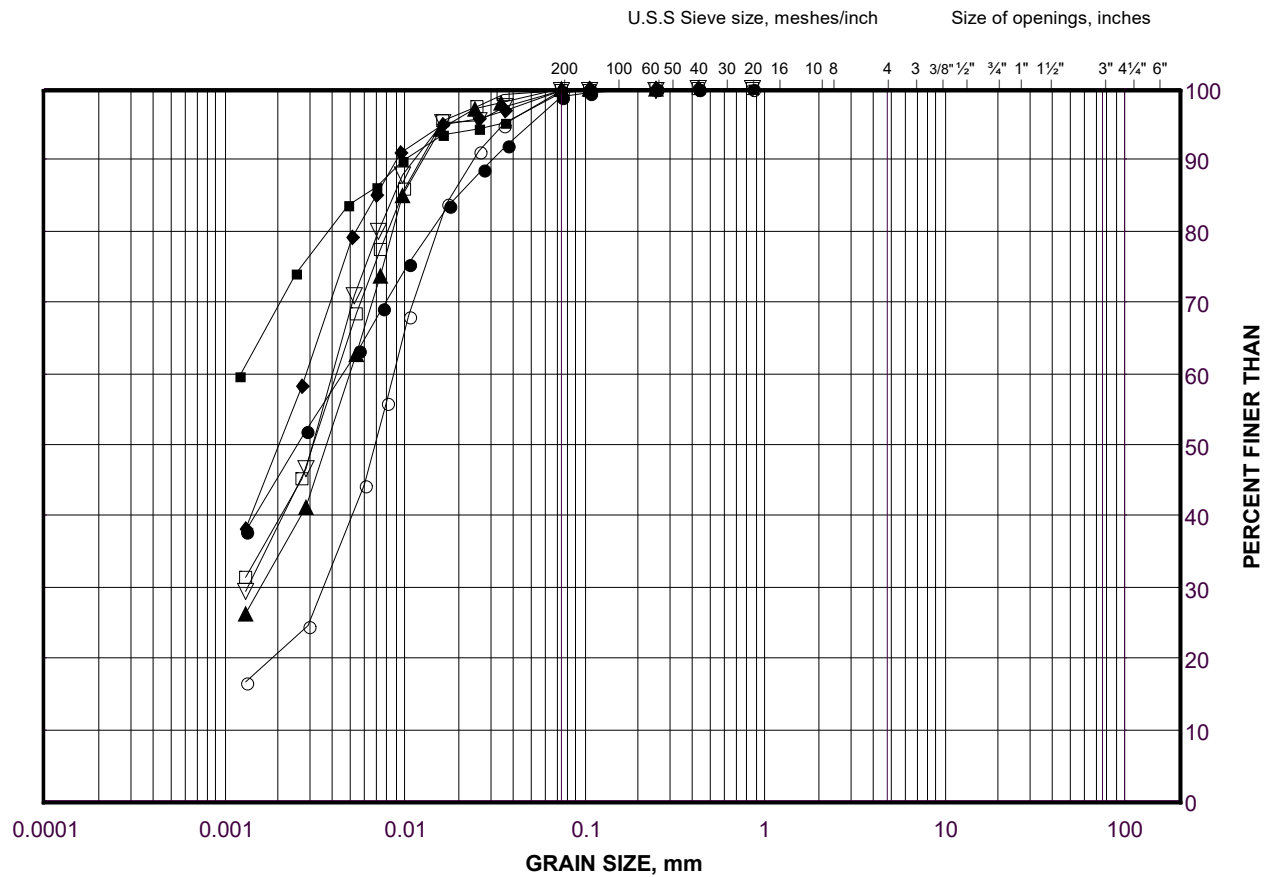
Date: 29-Jul-19



# GRAIN SIZE DISTRIBUTION

Lower Clayey Silt to Silty Clay

FIGURE C10C



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AP-1	11	207.1
■	A2-2	12	203.3
◆	A2-2	13	201.2
▲	AP-1	16	191.3
▽	A2-2	19	184.4
○	A2-2	24	172.8
□	A1-2	29	162.6

Project Number: 1671430

Checked By: NK/JPD

**Golder Associates**

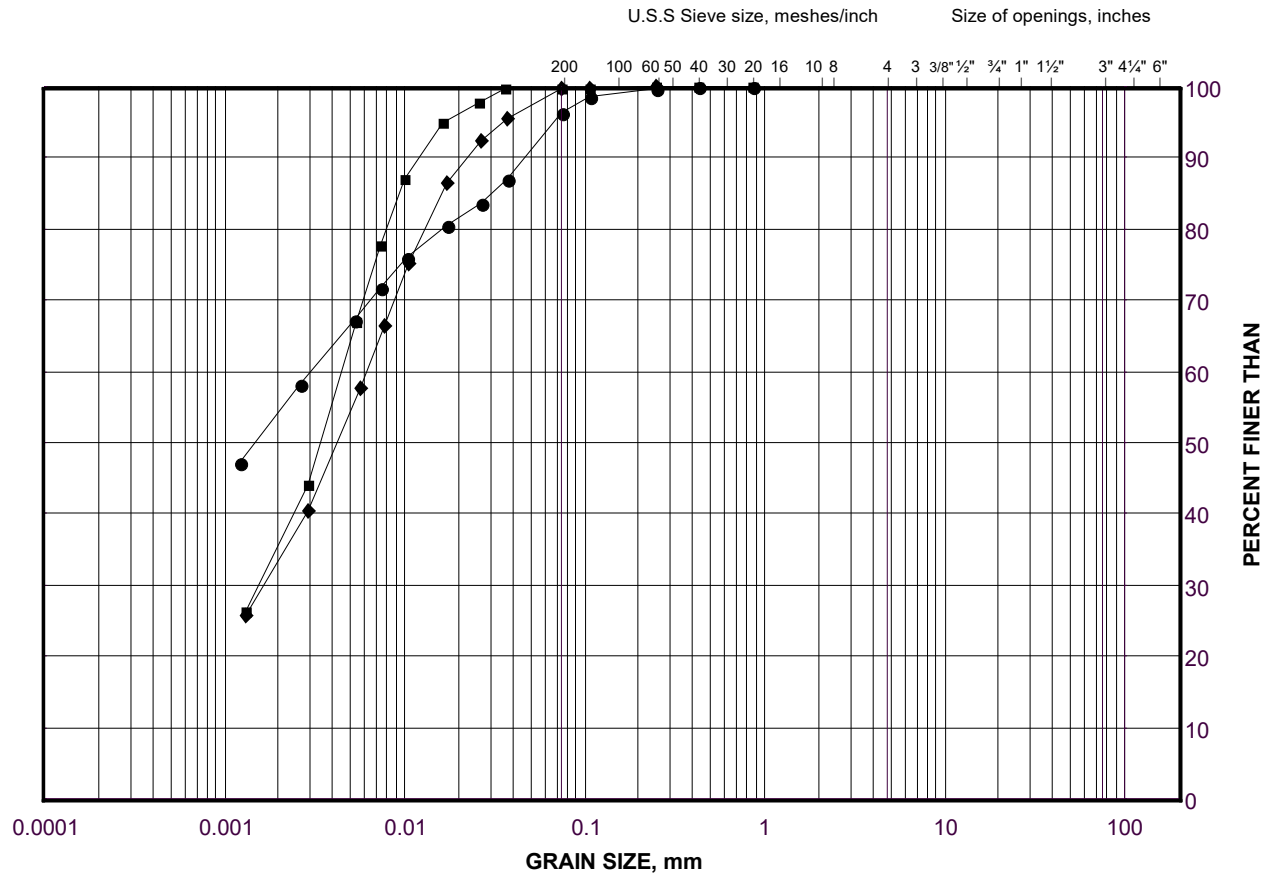
Date: 29-Jul-19



# GRAIN SIZE DISTRIBUTION

Lower Clayey Silt to Silty Clay

FIGURE C10D



## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	AP-2	13	206.7
■	AP-2	17	196.0
◆	AP-1	9	210.2

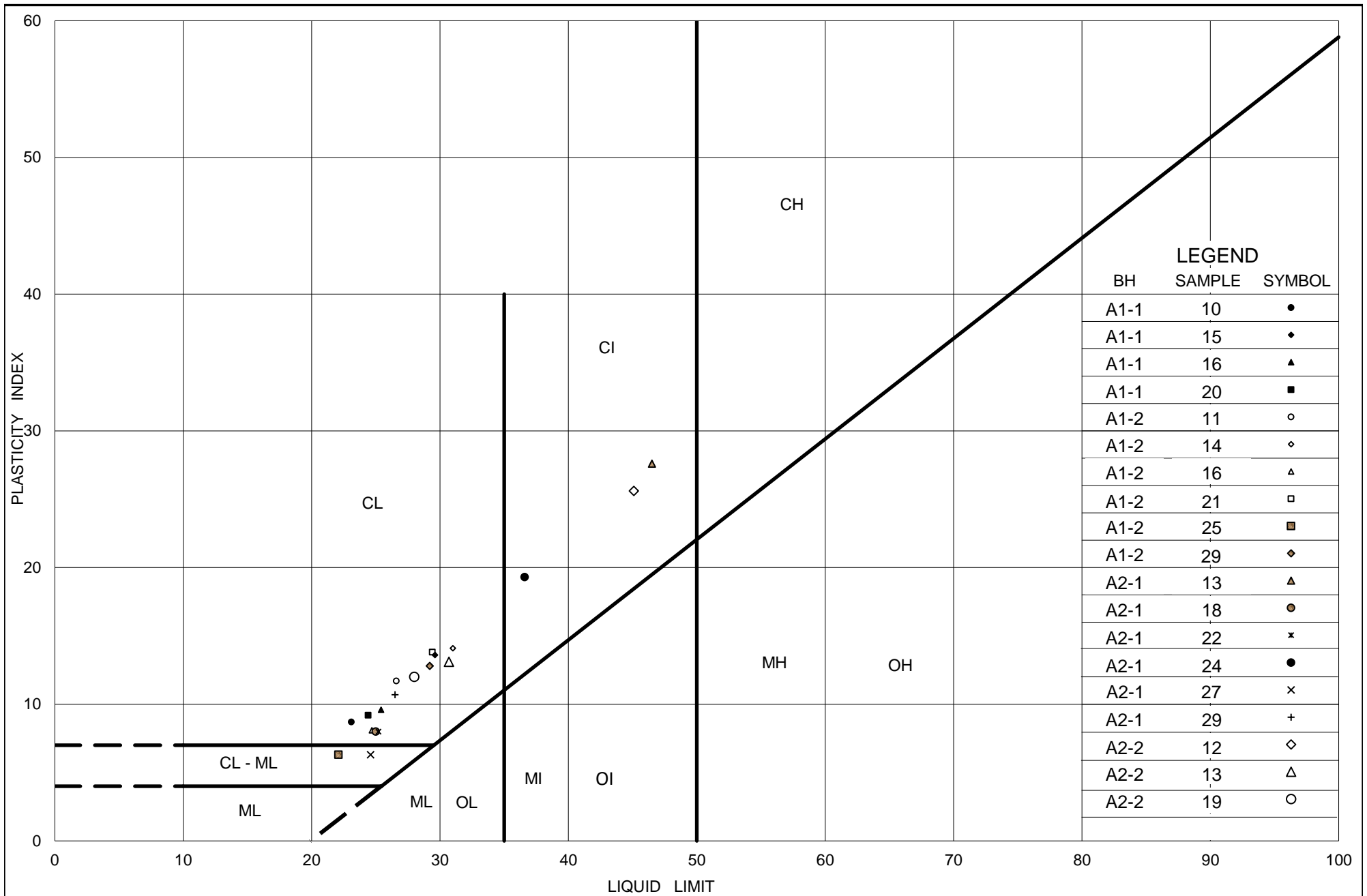
Project Number: 1671430

Checked By: NK/JPD

**Golder Associates**

Date: 29-Jul-19





Ministry of Transportation

Ontario

## PLASTICITY CHART

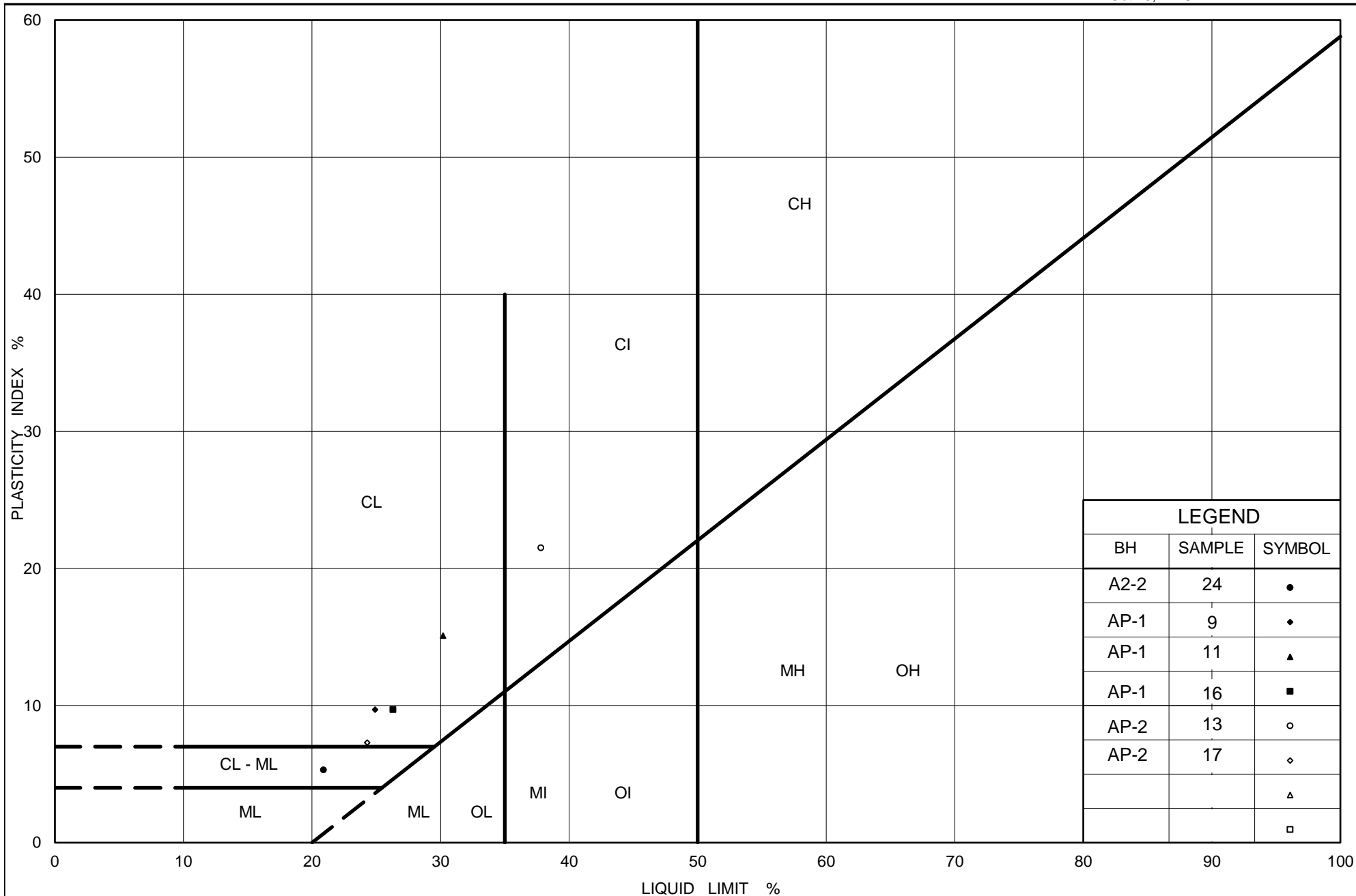
### Lower Clayey Silt to Silty Clay

Figure No. C11A

Project No. 1671430

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## PLASTICITY CHART

### Lower Silty Clay to Clayey Silt

Figure No.C11B

Project No. 1671430

Checked By:



CONSOLIDATED DRAINED TRIAXIAL ASTM D7181 SHEET 1 OF 4		FIGURE C12A Lower Silty Clay to Clayey Silt	
TEST STAGE		B	C
BOREHOLE NUMBER		A2-2	
SAMPLE NUMBER		TO 1	
DEPTH, ft		-	
SPECIMEN DIAMETER, cm		5.04	5.08
SPECIMEN HEIGHT, cm		10.08	10.08
NATURAL WATER CONTENT, %		23.3	22.4
DRY DENSITY, Mg/m <sup>3</sup>		1.68	1.67
WATER CONTENT BEFORE CONSOLIDATION, %		23.8	24.0
CELL PRESSURE, $\sigma_3$ , kPa		205.0	350.0
BACK PRESSURE, kPa		130.0	200.0
PORE PRESSURE PARAMETER "B"		0.97	0.97
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa		75.0	150.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %		1.9	5.6
WATER CONTENT AFTER CONSOLIDATION, %		22.7	20.6
AVERAGE RATE OF STRAIN, %/hr		0.5	0.5
TIME TO FAILURE, HOURS		20	21
WATER CONTENT AFTER TEST, %		22.0	22.1
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa		10.2	356.1
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %		10.2	10.3
MAX PRINCIPAL STRESS RATIO, $(\sigma'_1 / \sigma'_3)$ maximum		3.6	3.4
FILTER DRAINS USED, y/n		y	y
TEST NOTES:			
Specimen B and C are Selbytube sampes.			
FAILURE PLANE NUMBER		1.0	1.0
ANGLE OF FAILURE, DEGREES		50.0	50.0
<div> <div>Date: 4/22/2019</div> <div>Project No. 1671430</div> </div> <div> <div>Golder Associates</div> </div> <div> <div>Prepared By LH</div> <div>Checked By:</div> </div>			



CONSOLIDATED DRAINED TRIAXIAL

ASTM D7181

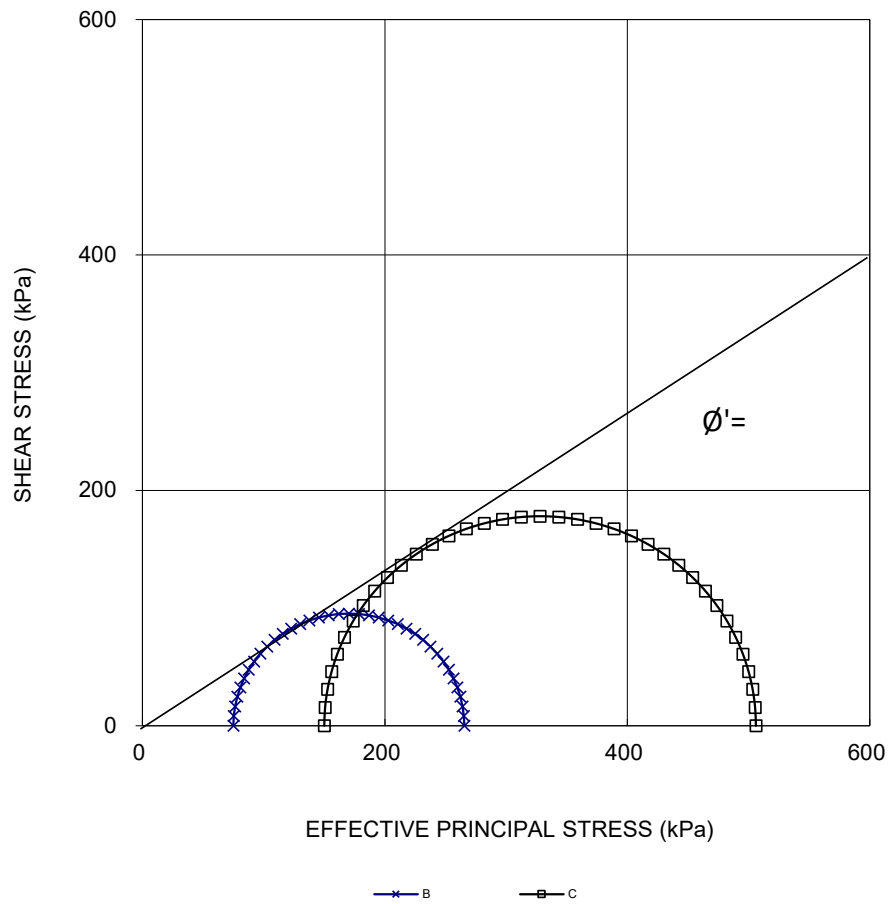
SHEET 2 OF 4

FIGURE

C12B

Lower Silty Clay to Clayey Silt

A2-2 TO1



Date: 4/22/2019

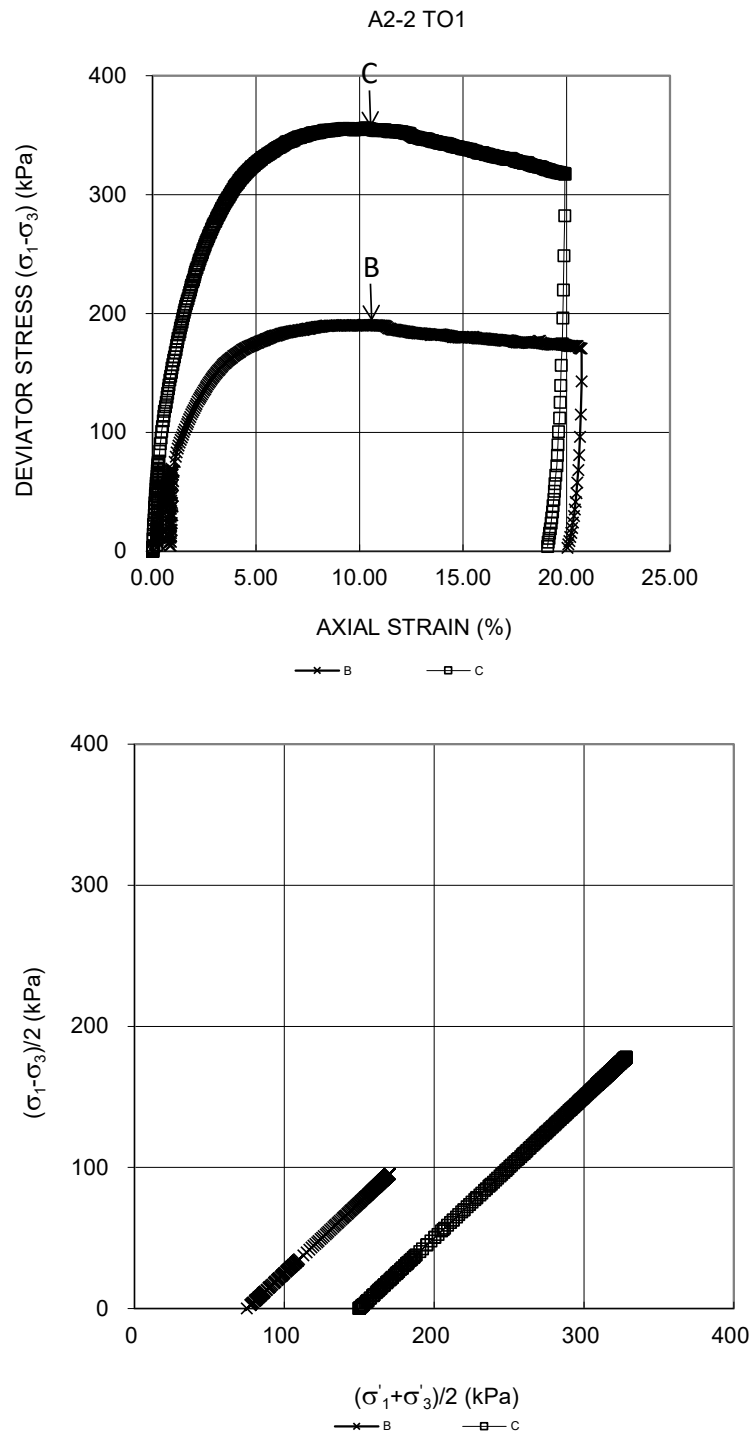
Project No. 1671430

**Golder Associates**

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Date: 4/22/2019  
Project No. 1671430

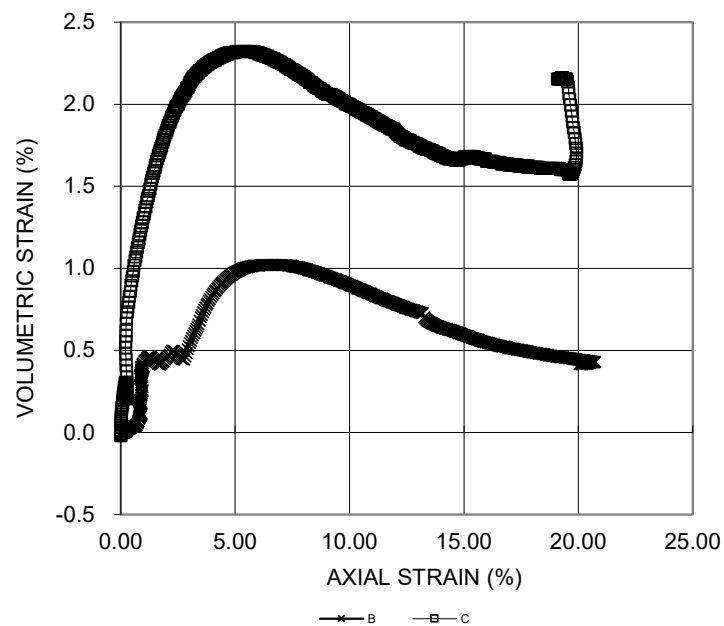
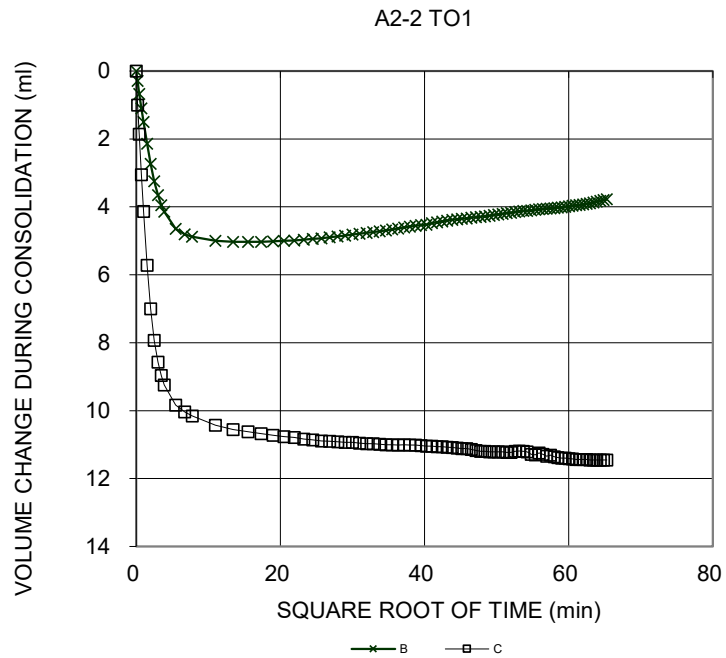
**Golder Associates**

Prepared By LH  
Checked By:



CONSOLIDATED DRAINED TRIAXIAL  
ASTM D7181  
SHEET 4 OF 4

FIGURE  
C12D  
Lower Silty Clay to Clayey Silt



NOTES: POSITIVE (+) VOLUMETRIC STRAIN = SAMPLE VOLUME DECREASING  
NEGATIVE ( - ) VOLUMETRIC STRAIN = SAMPLE VOLUME INCREASING

Date: 4/22/2019  
Project No. 1671430

**Golder Associates**

Prepared By LH  
Checked By:



<b>CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS</b>  <b>ASTM D4767</b>  <b>SHEET 1 OF 4</b>		<b>FIGURE</b>  <b>C13A</b>  <b>Lower Silty Clay to Clayey Silt</b>		
TEST STAGE	A	B	C	
BOREHOLE NUMBER		A1-2		
SAMPLE		TO-1		
DEPTH, m		19.82-20.43		
SPECIMEN DIAMETER, cm	5.09	5.07	5.09	
SPECIMEN HEIGHT, cm	10.20	10.13	9.89	
NATURAL WATER CONTENT, %	31.1	30.0	27.7	
DRY DENSITY, Mg/m <sup>3</sup>	1.48	1.53	1.60	
WATER CONTENT AFTER SATURATION, %	31.5	30.4	32.3	
CELL PRESSURE, $\sigma_3$ , kPa	230.0	400.0	430.0	
BACK PRESSURE, kPa	130.0	200.0	130.0	
PORE PRESSURE PARAMETER "B"	0.96	0.96	0.97	
EFFECTIVE CONSOLIDATION STRESS, $\sigma_c$ , kPa	100.0	200.0	300.0	
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	2.9	5.3	18.5	
WATER CONTENT AFTER CONSOLIDATION, %	29.5	26.9	20.7	
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5	
TIME TO FAILURE, HOURS	2.6	2.2	3.0	
WATER CONTENT AFTER TEST, %	28.4	24.2	21.2	
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	77.1	126.0	184.6	
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ maximum, %	1.3	1.1	1.5	
MAX EFFECTIVE PRINCIPAL STRESS RATIO, $(\sigma'_1 / \sigma'_3)$ maximum	2.4	2.8	2.2	
DEVIATOR STRESS AT $(\sigma'_1 / \sigma'_3)$ maximum, kPa	61.7	100.6	143.2	
AXIAL STRAIN AT $(\sigma'_1 / \sigma'_3)$ maximum, %	5.3	10.8	14.4	
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ maximum	0.50	0.65	0.70	
PORE PRESSURE PARAMETER, Af, AT $(\sigma'_1 / \sigma'_3)$ maximum	0.93	1.42	1.24	
FILTER DRAINS USED, y/n	y	y	y	
TEST NOTES:	<p>Effective consolidation stresses are assigned by the client.</p> <p>Specimen A taken 1-12cm from top of tube.</p> <p>Specimen B taken 12-23 cm from top of tube.</p> <p>Specimen C taken 23-34 cm from top of tube.</p>			
FAILURE PLANE NUMBER	-	-	-	
ANGLE OF FAILURE PLANE, DEGREES	Bulged	Bulged	Bulged	
<div> <div>Date: 5/2/2019</div> <div>Project No. 1671430.0</div> </div> <div> <b>Golder Associates</b> </div> <div> <div>Prepared By LH</div> <div>Checked By:</div> </div>				



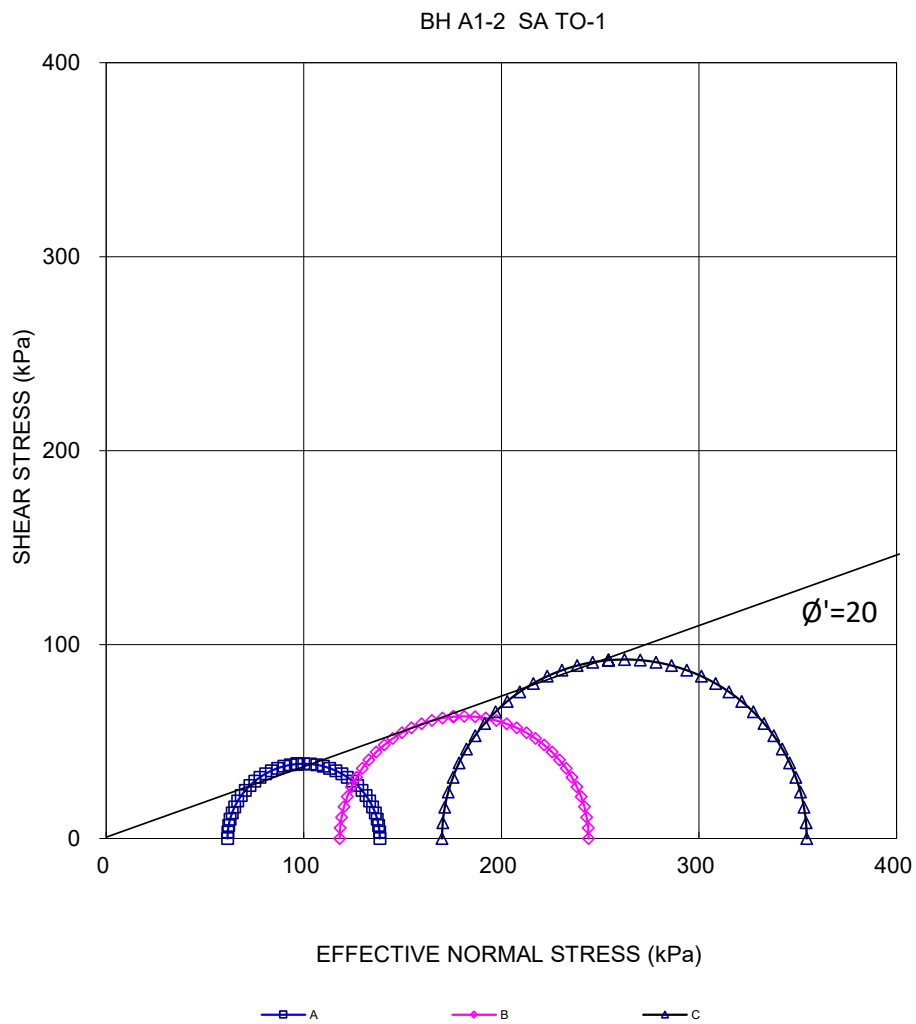
CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS

ASTM D4767  
SHEET 2 OF 4

FIGURE

C13B

Lower Silty Clay to Clayey Silt



Date: 5/2/2019  
Project No. 1671430

Golder Associates

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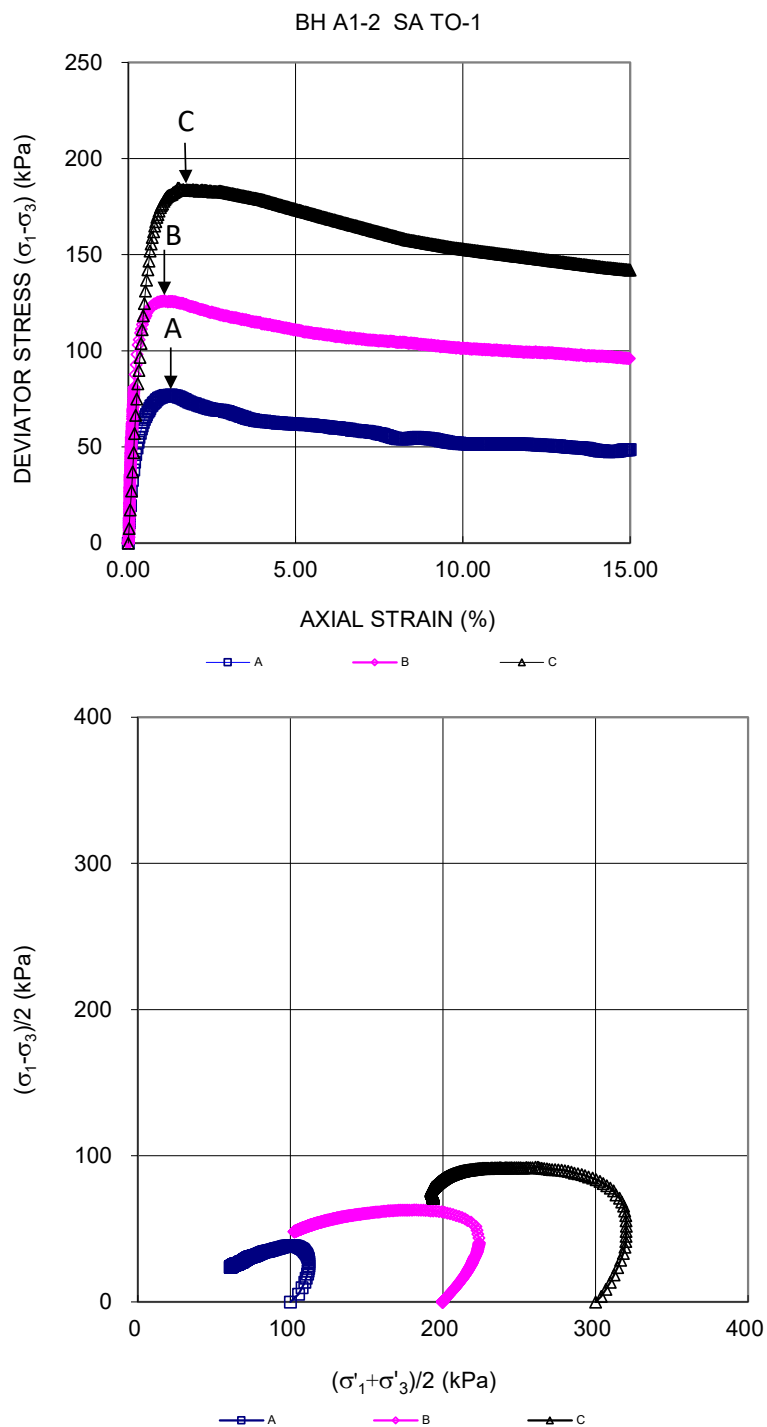
CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS

ASTM D4767  
SHEET 3 OF 4

FIGURE

C13C

Lower Silty Clay to Clayey Silt



Date: 5/2/2019  
Project No. 1671430

**Golder Associates**

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**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS**

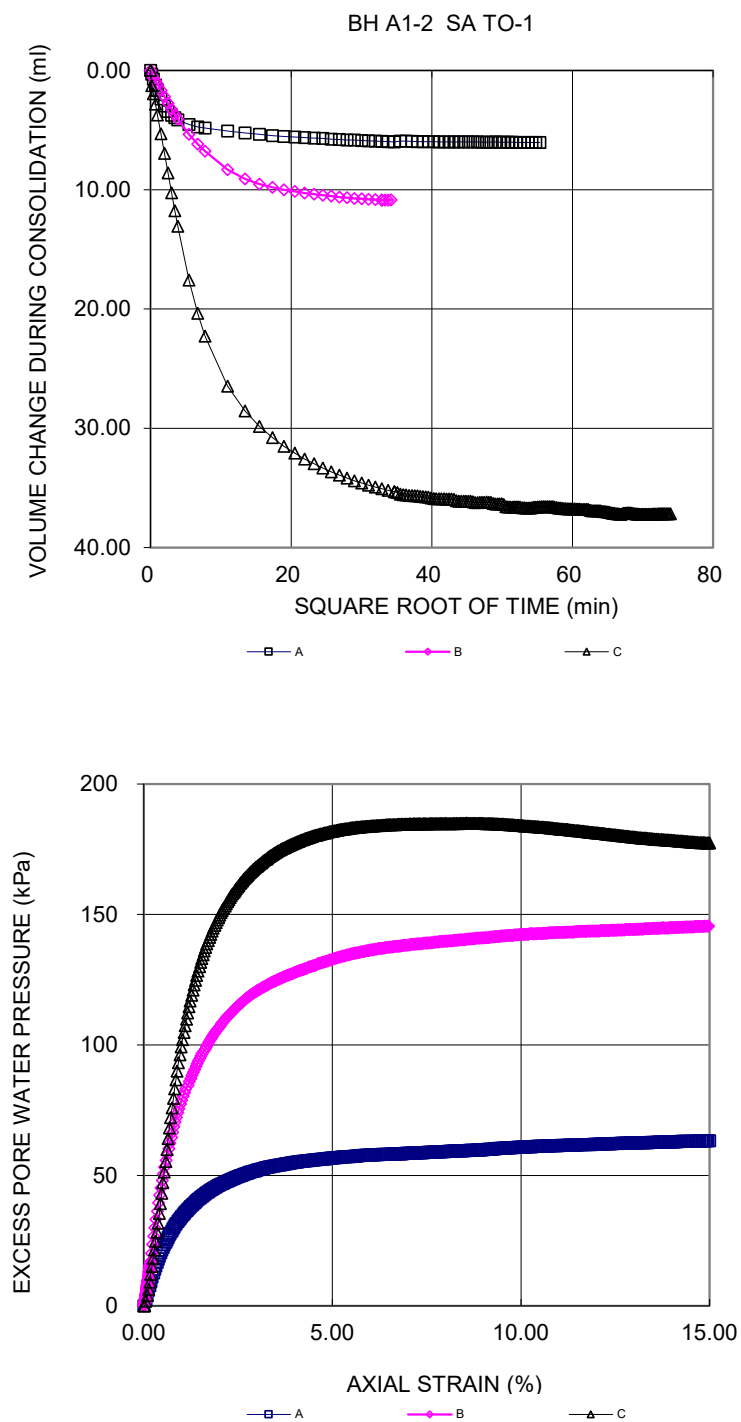
**ASTM D4767**

**SHEET 4 OF 4**

**FIGURE**

**C13D**

**Lower Silty Clay to Clayey Silt**



Date: 5/2/2019  
Project No. 1671430

**Golder Associates**

Prepared By LH  
Checked By:



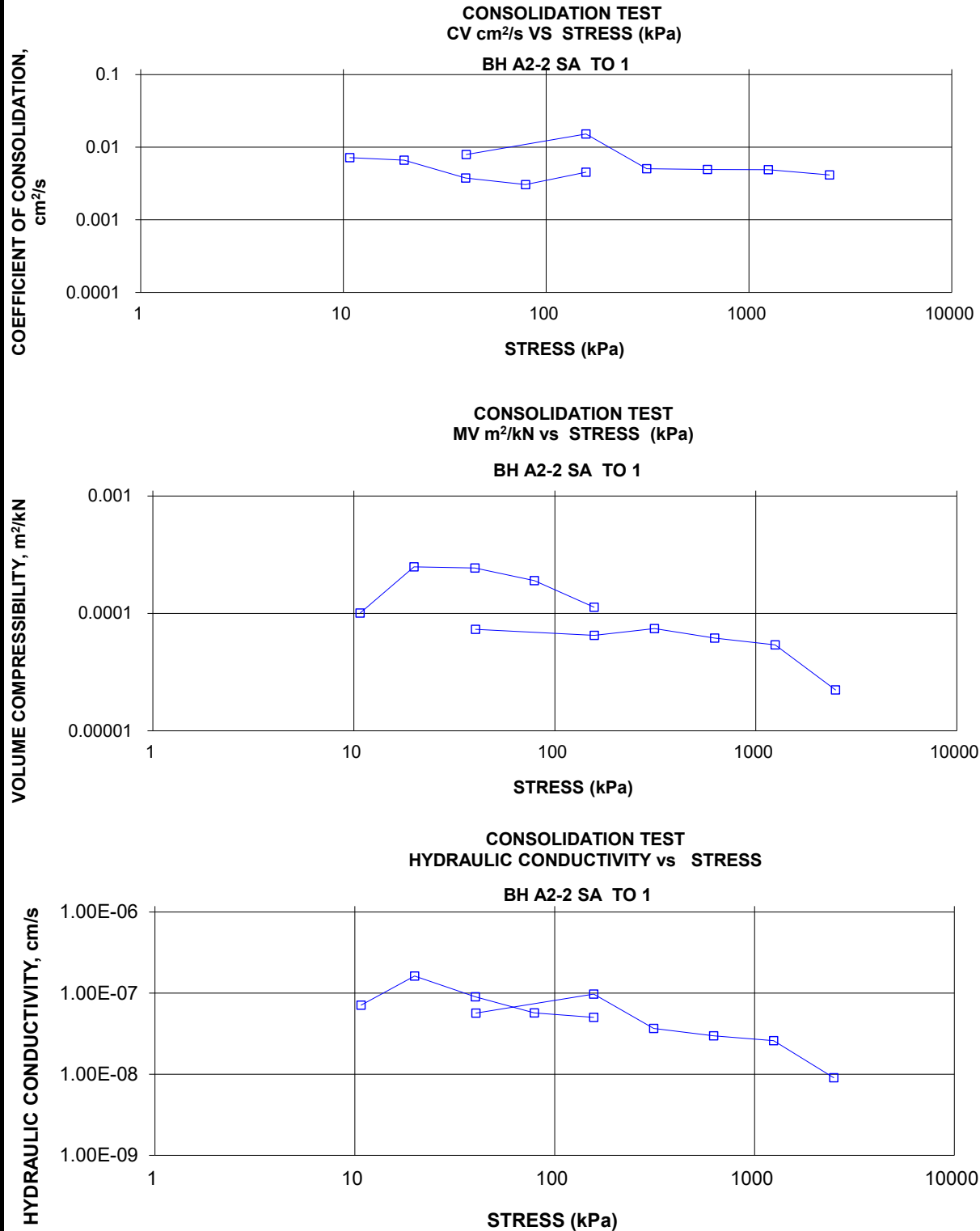
<b>CONSOLIDATION TEST SUMMARY</b> <b>ASTM D2435/D2435M</b>				<b>FIGURE C14A</b> <b>Lower Silty Clay to Clayey Silt</b>			
<b>SAMPLE IDENTIFICATION</b>							
Project Number	1671430	Sample Number	TO 1				
Borehole Number	A2-2	Sample Depth, m	12.20-12.80				
<b>TEST CONDITIONS</b>							
Test Type	Laboratory Standard	Load Duration, hr	24				
Oedometer Number	4						
Date Started	04/12/2019						
Date Completed	04/28/2019						
<b>SAMPLE DIMENSIONS AND PROPERTIES - INITIAL</b>							
Sample Height, cm	2.54	Unit Weight, kN/m <sup>3</sup>	19.92				
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m <sup>3</sup>	16.13				
Area, cm <sup>2</sup>	31.45	Specific Gravity, measured	2.75				
Volume, cm <sup>3</sup>	79.88	Solids Height, cm	1.520				
Water Content, %	23.44	Volume of Solids, cm <sup>3</sup>	47.79				
Wet Mass, g	162.23	Volume of Voids, cm <sup>3</sup>	32.09				
Dry Mass, g	131.42	Degree of Saturation, %	96.0				
<b>TEST COMPUTATIONS</b>							
Stress	Corr. Height	Void Ratio	Average Height	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
kPa	cm		cm				
0.00	2.540	0.672	2.540				
6.07	2.540	0.672	2.540				
10.75	2.539	0.671	2.539	190	7.19E-03	1.01E-04	7.12E-08
19.91	2.533	0.667	2.536	205	6.65E-03	2.49E-04	1.62E-07
40.08	2.520	0.659	2.527	360	3.76E-03	2.44E-04	8.99E-08
79.11	2.502	0.646	2.511	437	3.06E-03	1.91E-04	5.71E-08
156.84	2.479	0.632	2.490	290	4.53E-03	1.13E-04	5.02E-08
40.23	2.488	0.637	2.484				
10.78	2.499	0.645	2.494				
40.33	2.494	0.641	2.496	167	7.91E-03	7.33E-05	5.68E-08
157.01	2.474	0.628	2.484	86	1.52E-02	6.51E-05	9.71E-08
312.75	2.445	0.609	2.460	254	5.05E-03	7.43E-05	3.68E-08
624.06	2.396	0.577	2.421	252	4.93E-03	6.16E-05	2.98E-08
1247.09	2.311	0.521	2.354	240	4.89E-03	5.40E-05	2.59E-08
2492.81	2.240	0.474	2.276	265	4.14E-03	2.23E-05	9.05E-09
622.77	2.254	0.483	2.247				
157.04	2.281	0.501	2.268				
40.14	2.313	0.522	2.297				
10.75	2.342	0.541	2.328				
Note: Consolidation loading and unloading schedule assigned by the client. cv and k are approximate only based on t <sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M) Specimen taken 36-42 cm from top of the tube.							
<b>SAMPLE DIMENSIONS AND PROPERTIES - FINAL</b>							
Sample Height, cm	2.34	Unit Weight, kN/m <sup>3</sup>	21.14				
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m <sup>3</sup>	17.50				
Area, cm <sup>2</sup>	31.45	Specific Gravity, measured	2.75				
Volume, cm <sup>3</sup>	73.66	Solids Height, cm	1.520				
Water Content, %	20.83	Volume of Solids, cm <sup>3</sup>	47.79				
Wet Mass, g	158.80	Volume of Voids, cm <sup>3</sup>	25.87				
Dry Mass, g	131.42						
<div style="display: flex; justify-content: space-between;"> <span>Prepared By: LH</span> <span><b>Golder Associates</b></span> <span>Checked By:</span> </div>							



**CONSOLIDATION TEST SUMMARY**  
ASTM D2435/D2435M

**FIGURE C14B**

**Lower Silty Clay to Clayey Silt**



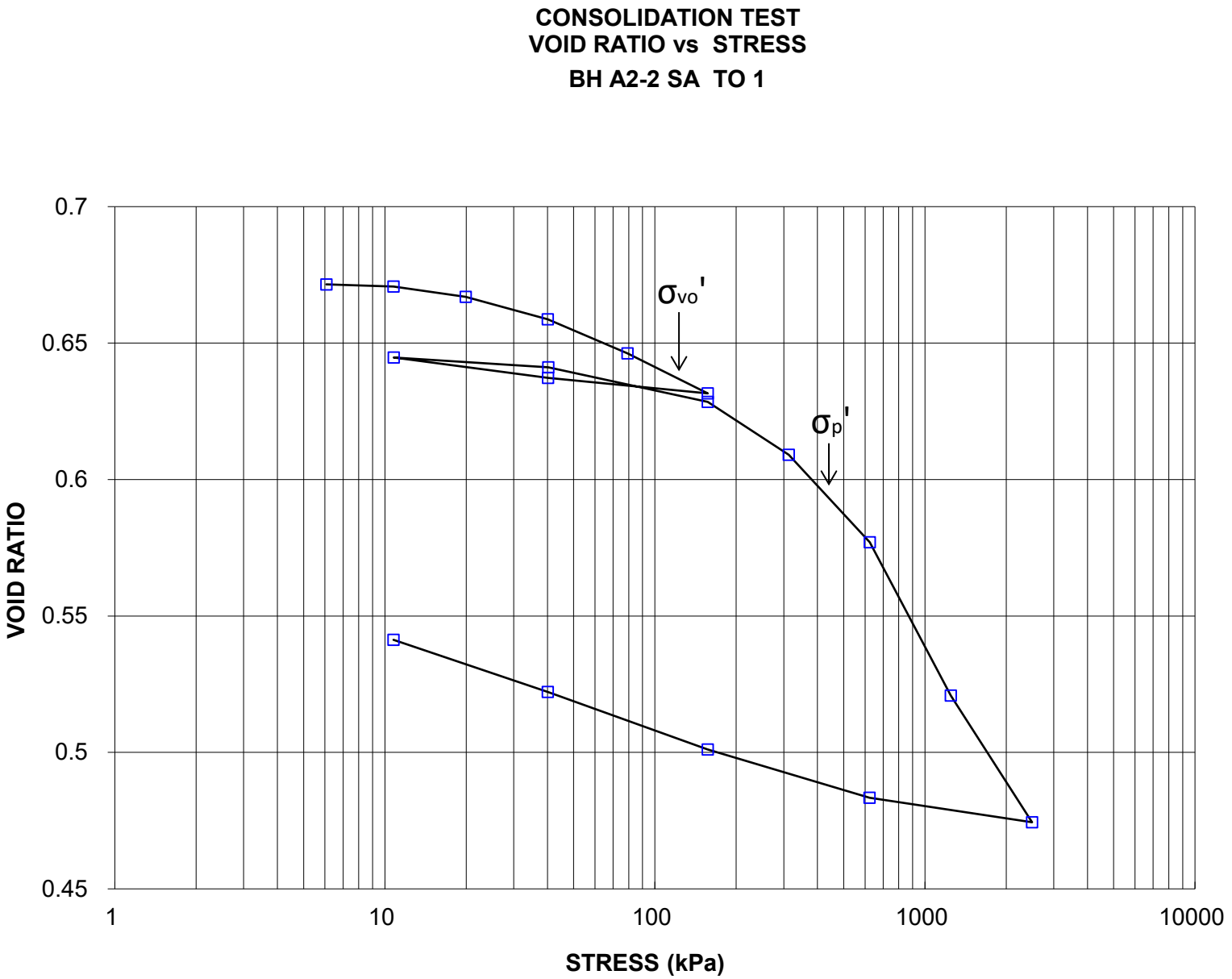
Project No.1671430

Prepared By: LH

**Golder Associates**

Checked By:

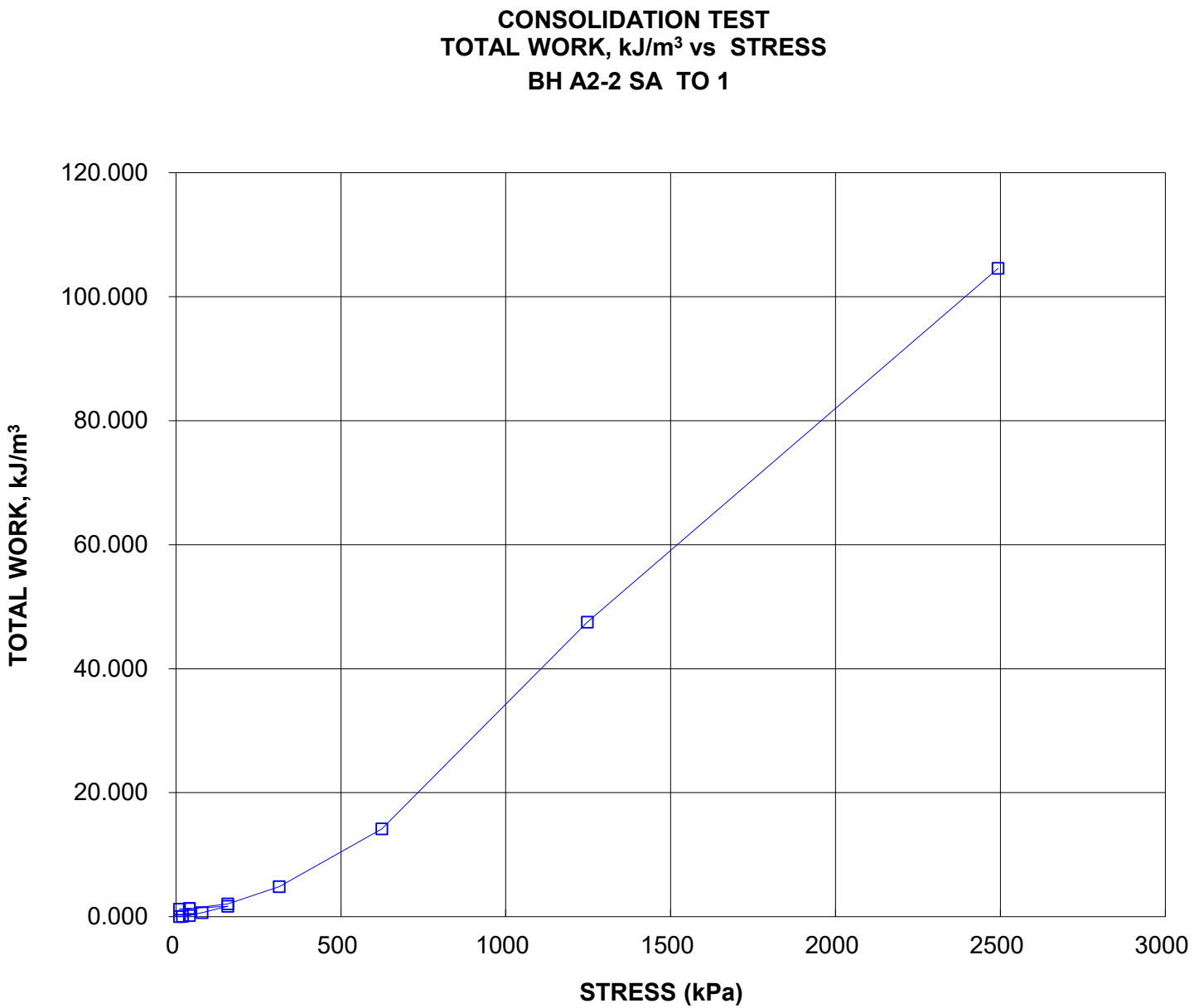






CONSOLIDATION TEST  
TOTAL WORK VS STRESS

FIGURE 14D  
Lower Silty Clay to Clayey Silt



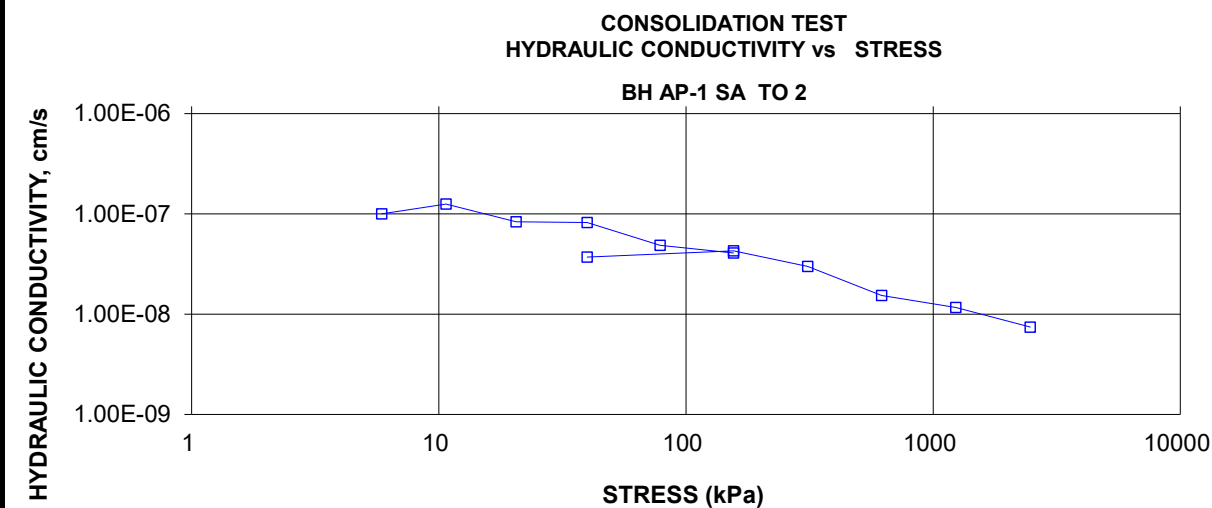
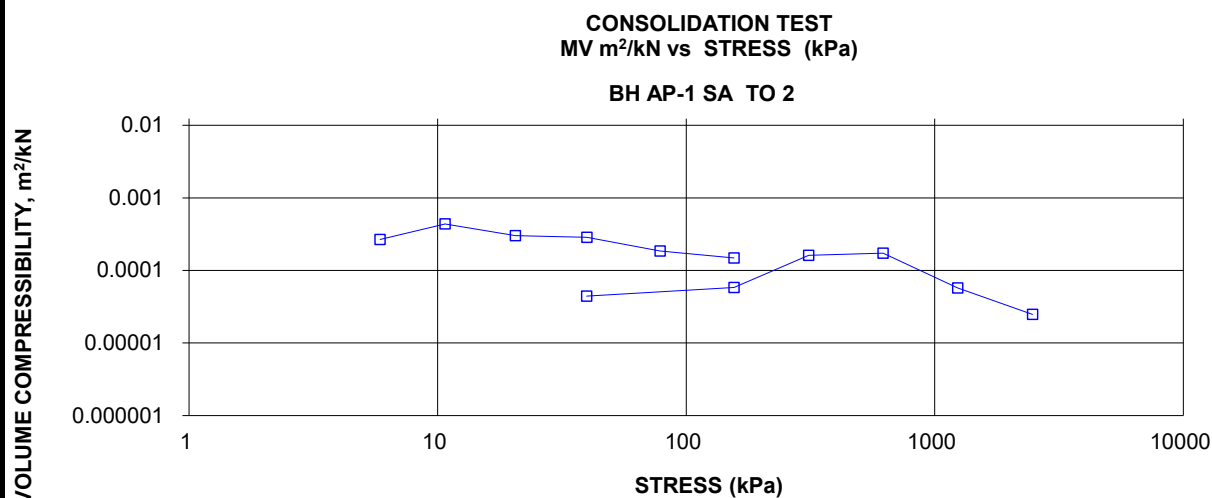
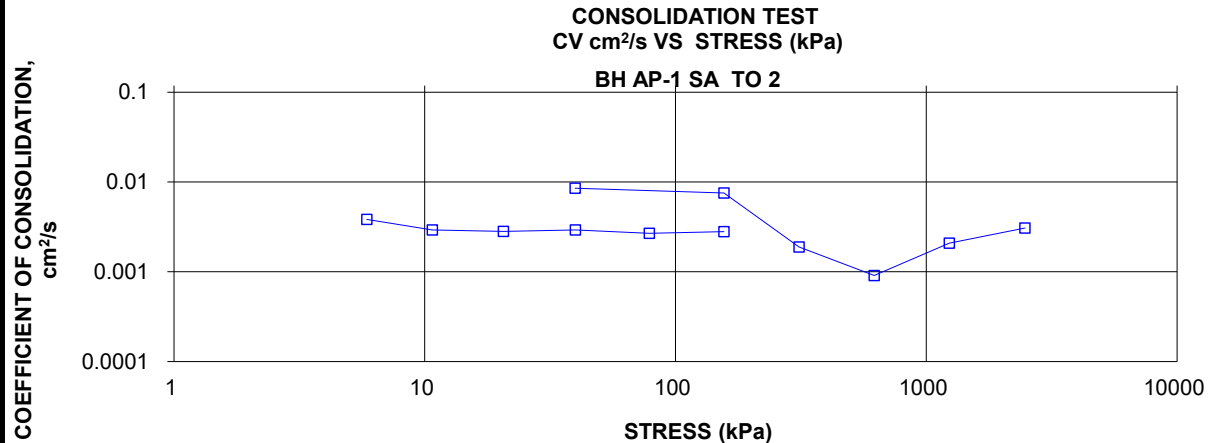


<b>CONSOLIDATION TEST SUMMARY</b> <b>ASTM D2435/D2435M</b>					<b>FIGURE C15A</b> <b>Lower Silty Clay to Clayey Silt</b>		
<b>SAMPLE IDENTIFICATION</b>							
Project Number	1671430	Sample Number	TO 2				
Borehole Number	AP-1	Sample Depth, m	18.29-18.90				
<b>TEST CONDITIONS</b>							
Test Type	Laboratory Standard	Load Duration, hr	24				
Oedometer Number	1						
Date Started	04/03/2019						
Date Completed	04/21/2019						
<b>SAMPLE DIMENSIONS AND PROPERTIES - INITIAL</b>							
Sample Height, cm	2.56	Unit Weight, kN/m <sup>3</sup>	18.76				
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	14.34				
Area, cm <sup>2</sup>	31.67	Specific Gravity, measured	2.74				
Volume, cm <sup>3</sup>	80.91	Solids Height, cm	1.363				
Water Content, %	30.88	Volume of Solids, cm <sup>3</sup>	43.18				
Wet Mass, g	154.83	Volume of Voids, cm <sup>3</sup>	37.74				
Dry Mass, g	118.3	Degree of Saturation, %	96.8				
<b>TEST COMPUTATIONS</b>							
Stress	Corr. Height	Void	Average Height	t <sub>90</sub>	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	2.555	0.874	2.555				
5.87	2.551	0.871	2.553	360	3.84E-03	2.67E-04	1.00E-07
10.71	2.546	0.867	2.548	470	2.93E-03	4.37E-04	1.25E-07
20.57	2.538	0.862	2.542	485	2.82E-03	3.02E-04	8.35E-08
39.86	2.524	0.851	2.531	463	2.93E-03	2.86E-04	8.22E-08
78.56	2.506	0.838	2.515	501	2.68E-03	1.85E-04	4.85E-08
155.92	2.476	0.816	2.491	470	2.80E-03	1.48E-04	4.07E-08
39.80	2.483	0.821	2.480				
10.71	2.491	0.827	2.487				
39.80	2.488	0.825	2.489	154	8.53E-03	4.44E-05	3.71E-08
155.80	2.470	0.812	2.479	173	7.53E-03	5.80E-05	4.28E-08
311.03	2.406	0.765	2.438	667	1.89E-03	1.62E-04	3.00E-08
619.14	2.270	0.665	2.338	1274	9.10E-04	1.73E-04	1.54E-08
1236.26	2.180	0.599	2.225	505	2.08E-03	5.74E-05	1.17E-08
2473.01	2.102	0.541	2.141	317	3.06E-03	2.48E-05	7.44E-09
619.14	2.117	0.553	2.109				
155.84	2.142	0.571	2.130				
39.80	2.175	0.596	2.159				
10.63	2.206	0.618	2.190				
Note: Consolidation loading and unloading schedule assigned by the client. cv and k are approximate only based on t <sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M) Specimen taken 7-14 cm from top of the tube.							
<b>SAMPLE DIMENSIONS AND PROPERTIES - FINAL</b>							
Sample Height, cm	2.21	Unit Weight, kN/m <sup>3</sup>	20.32				
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	16.61				
Area, cm <sup>2</sup>	31.67	Specific Gravity, measured	2.74				
Volume, cm <sup>3</sup>	69.85	Solids Height, cm	1.363				
Water Content, %	22.37	Volume of Solids, cm <sup>3</sup>	43.18				
Wet Mass, g	144.76	Volume of Voids, cm <sup>3</sup>	26.67				
Dry Mass, g	118.3						
Prepared By: LH				<b>Golder Associates</b>		Checked By:	



**CONSOLIDATION TEST SUMMARY  
ASTM D2435/D2435M**

**FIGURE C15B**  
**Lower Silty Clay to Clayey Silt**



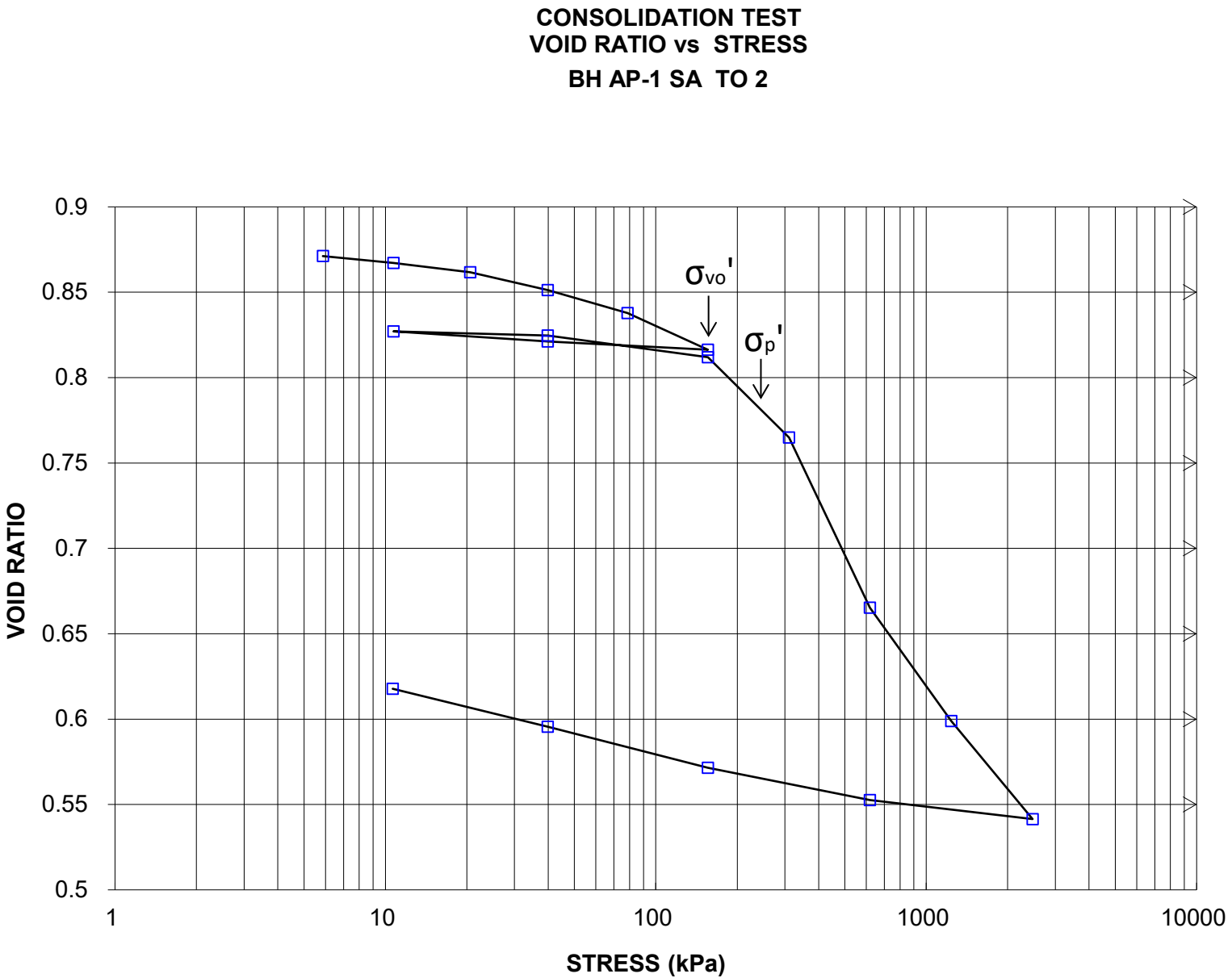
Project No.1671430

Prepared By: LH

**Golder Associates**

Checked By:

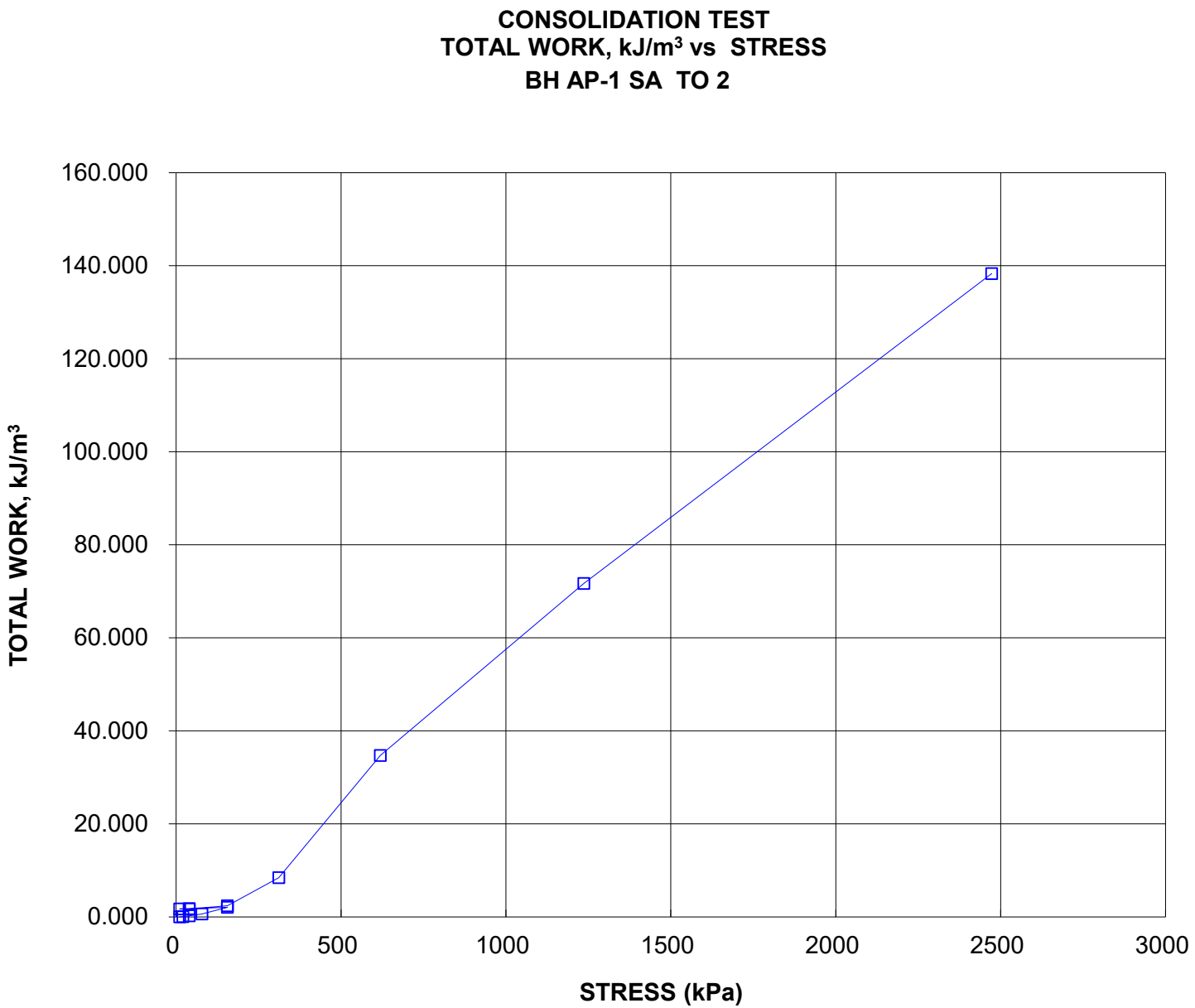






CONSOLIDATION TEST  
TOTAL WORK VS STRESS

FIGURE C15D  
Lower Silty Clay to Clayey Silt





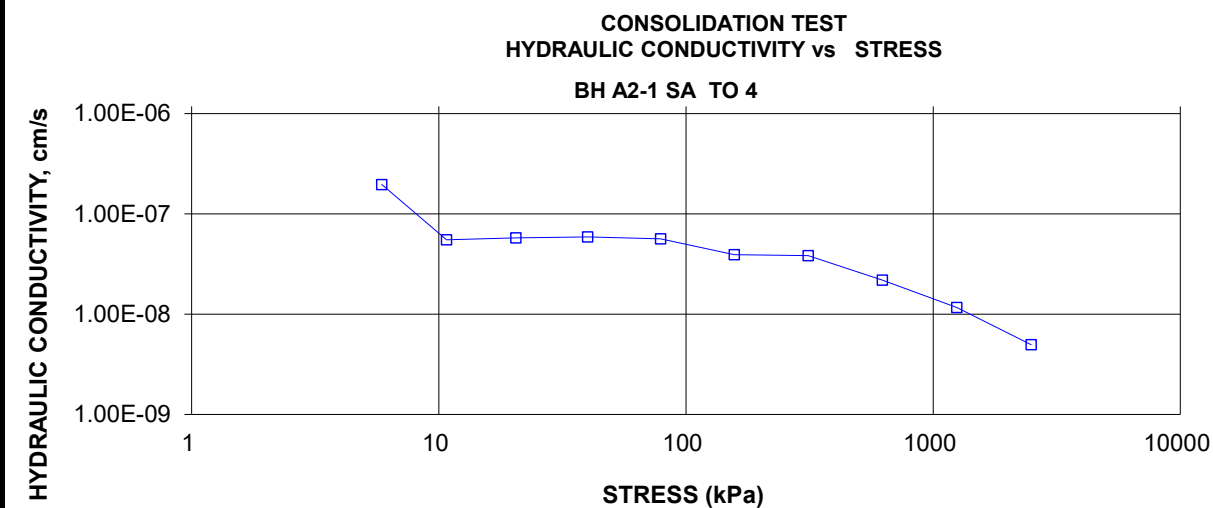
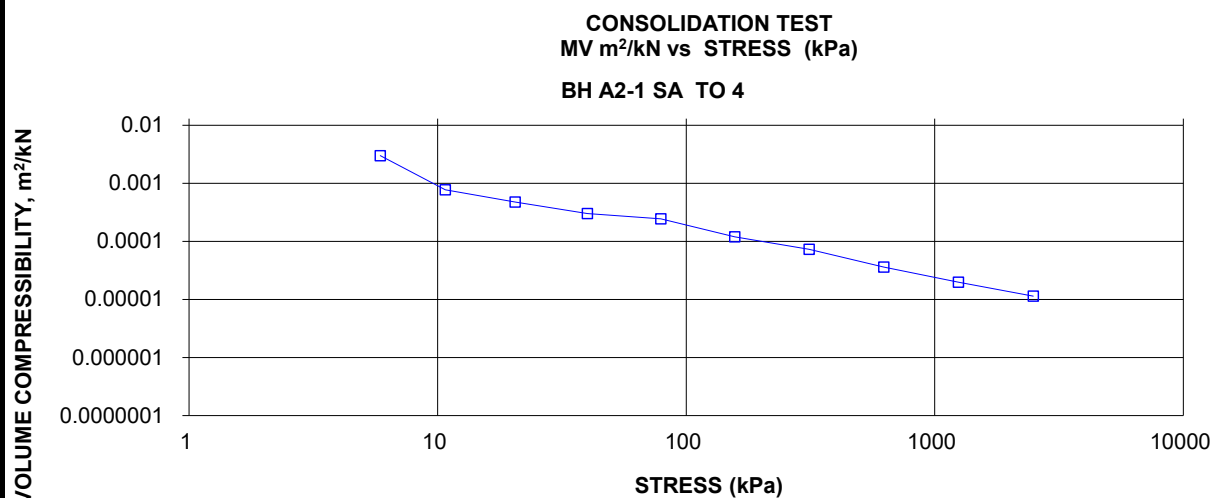
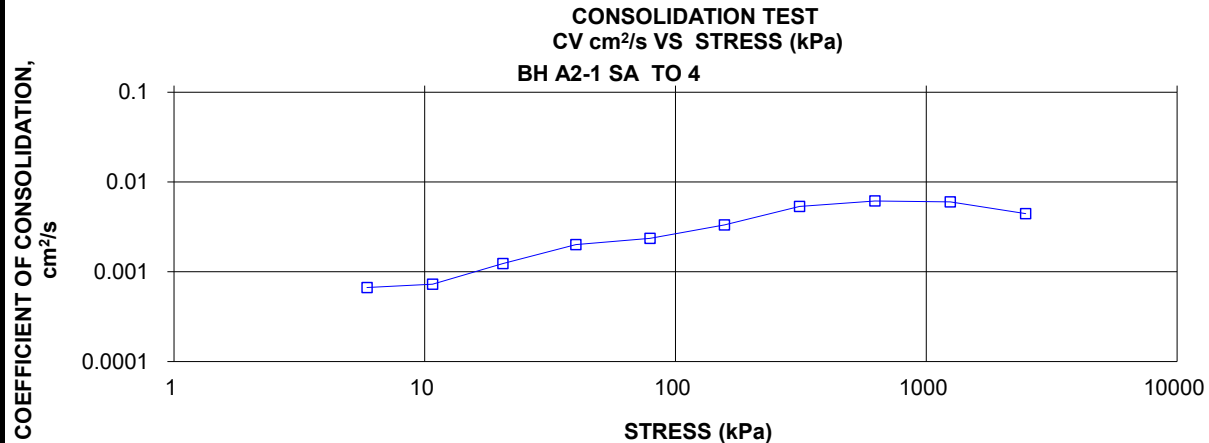
<b>CONSOLIDATION TEST SUMMARY</b>					<b>FIGURE C16A</b>		
<b>ASTM D2435/D2435M</b>					<b>Lower Silty Clay to Clayey Silt</b>		
<b>SAMPLE IDENTIFICATION</b>							
Project Number	1671430			Sample Number	TO 4		
Borehole Number	A2-1			Sample Depth, m	24.39-24.82		
<b>TEST CONDITIONS</b>							
Test Type	Laboratory Standard			Load Duration, hr	24		
Oedometer Number	11						
Date Started	04/03/2019						
Date Completed	04/17/2019						
<b>SAMPLE DIMENSIONS AND PROPERTIES - INITIAL</b>							
Sample Height, cm	2.54			Unit Weight, kN/m <sup>3</sup>	20.33		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m <sup>3</sup>	16.57		
Area, cm <sup>2</sup>	31.47			Specific Gravity, measured	2.74		
Volume, cm <sup>3</sup>	80.06			Solids Height, cm	1.569		
Water Content, %	22.67			Volume of Solids, cm <sup>3</sup>	49.37		
Wet Mass, g	165.95			Volume of Voids, cm <sup>3</sup>	30.69		
Dry Mass, g	135.28			Degree of Saturation, %	99.9		
<b>TEST COMPUTATIONS</b>							
Stress	Corr. Height	Void	Average Height	t <sub>90</sub>	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	2.544	0.622	2.544				
5.88	2.499	0.593	2.522	2018	6.68E-04	3.00E-03	1.96E-07
10.74	2.490	0.587	2.494	1815	7.27E-04	7.76E-04	5.53E-08
20.51	2.478	0.579	2.484	1058	1.24E-03	4.75E-04	5.75E-08
40.01	2.463	0.570	2.470	645	2.01E-03	3.00E-04	5.90E-08
79.04	2.439	0.554	2.451	540	2.36E-03	2.44E-04	5.63E-08
156.74	2.415	0.539	2.427	375	3.33E-03	1.20E-04	3.93E-08
312.07	2.386	0.521	2.400	228	5.36E-03	7.29E-05	3.83E-08
623.75	2.357	0.503	2.372	194	6.15E-03	3.62E-05	2.18E-08
1245.94	2.326	0.483	2.342	194	5.99E-03	1.98E-05	1.16E-08
2489.84	2.290	0.460	2.308	254	4.45E-03	1.14E-05	4.97E-09
623.75	2.292	0.461	2.291				
156.74	2.297	0.464	2.294				
40.25	2.304	0.468	2.300				
10.69	2.310	0.472	2.307				
<p>Note:</p> <p>Consolidation loading and unloading schedule assigned by the client.</p> <p>cv and k are approximate only based on t<sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M)</p> <p>Specimen taken 20-28 cm from top of the tube.</p>							
<b>SAMPLE DIMENSIONS AND PROPERTIES - FINAL</b>							
Sample Height, cm	2.31			Unit Weight, kN/m <sup>3</sup>	21.48		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m <sup>3</sup>	18.25		
Area, cm <sup>2</sup>	31.47			Specific Gravity, measured	2.74		
Volume, cm <sup>3</sup>	72.69			Solids Height, cm	1.569		
Water Content, %	17.70			Volume of Solids, cm <sup>3</sup>	49.37		
Wet Mass, g	159.22			Volume of Voids, cm <sup>3</sup>	23.32		
Dry Mass, g	135.28						
Prepared By: LH				<b>Golder Associates</b>		Checked By:	



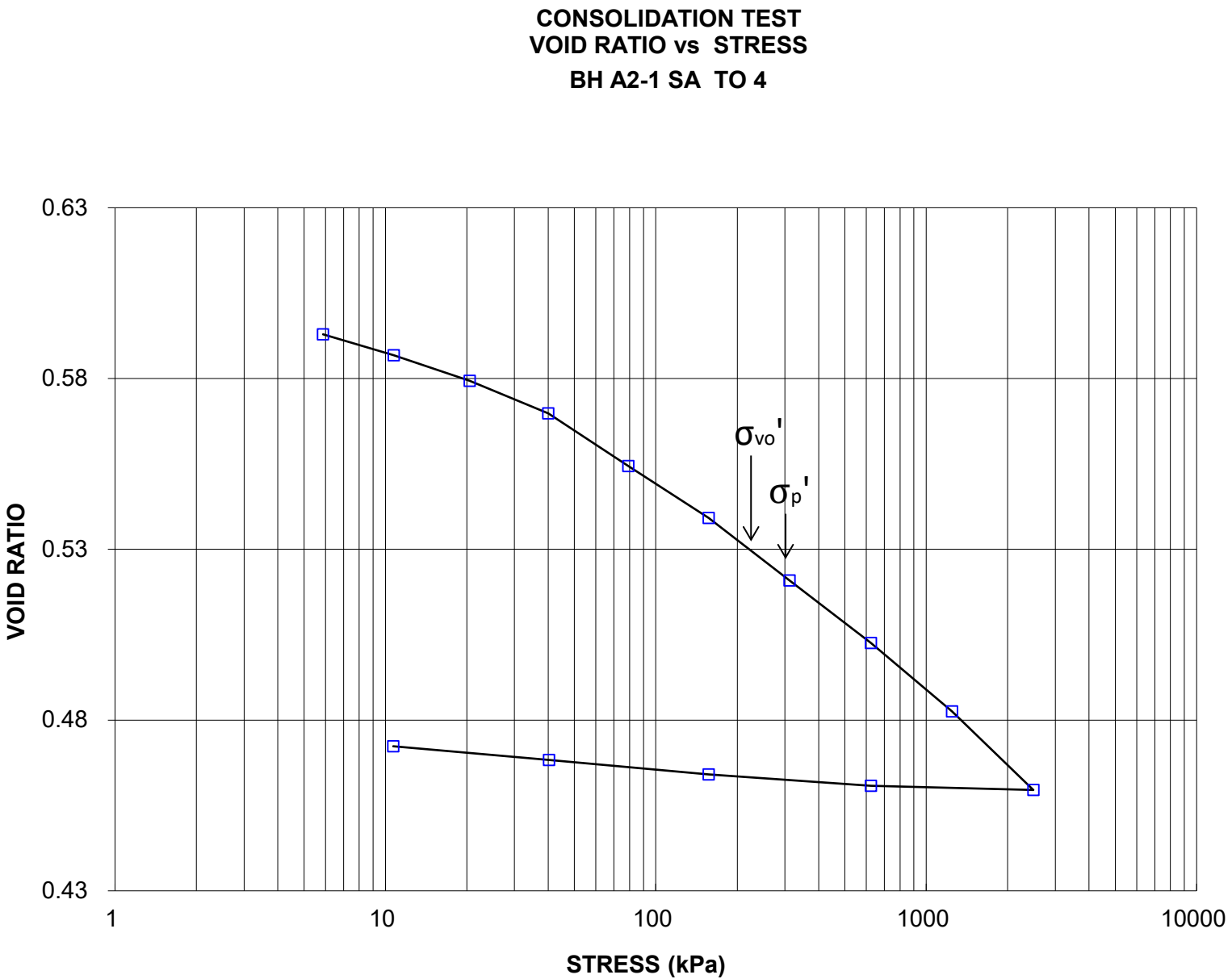
**CONSOLIDATION TEST SUMMARY  
ASTM D2435/D2435M**

**FIGURE C16B**

**Lower Silty Clay to Clayey Silt**



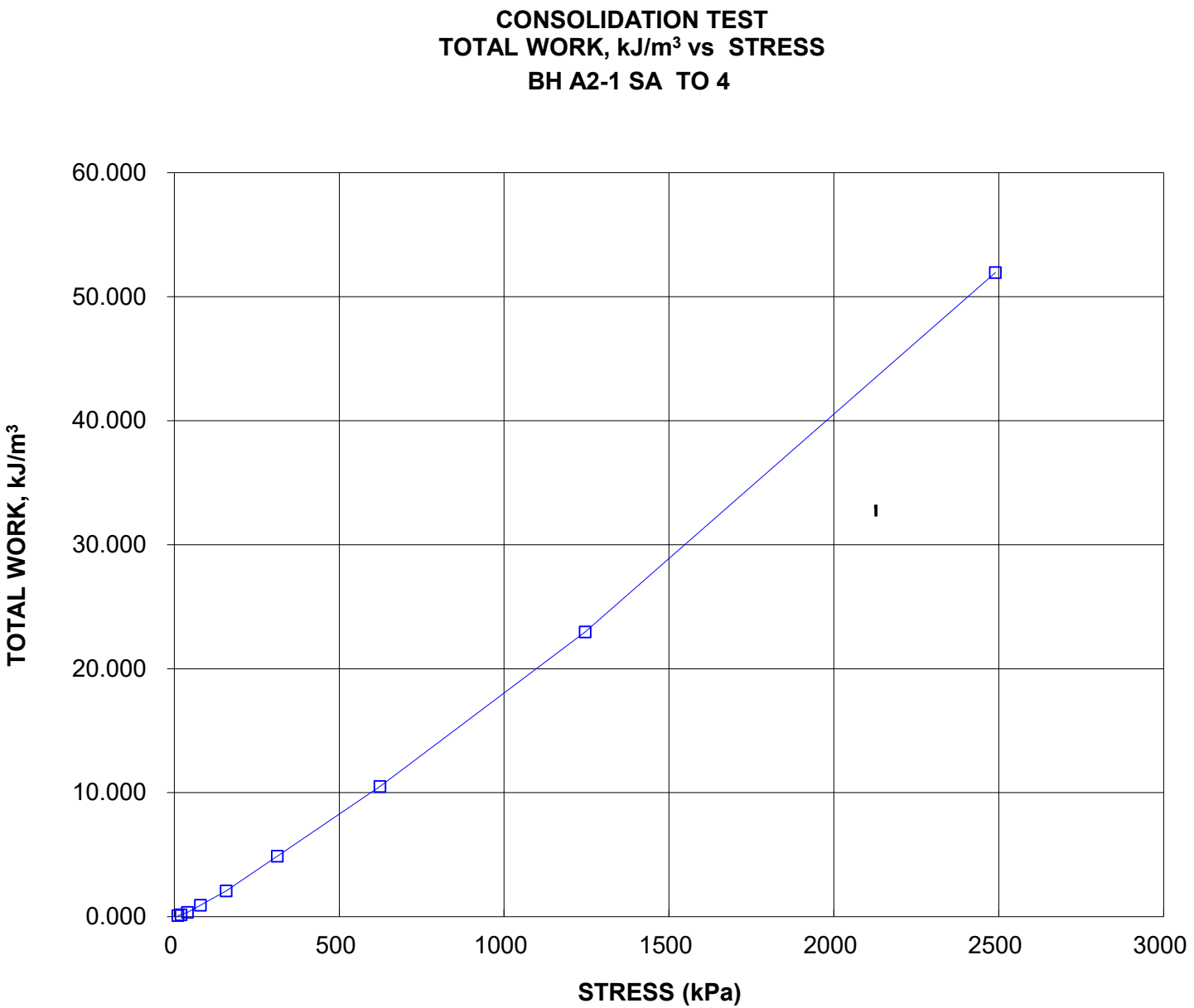






**CONSOLIDATION TEST  
TOTAL WORK VS STRESS**

**FIGURE C16D  
Lower Silty Clay to Clayey Silt**

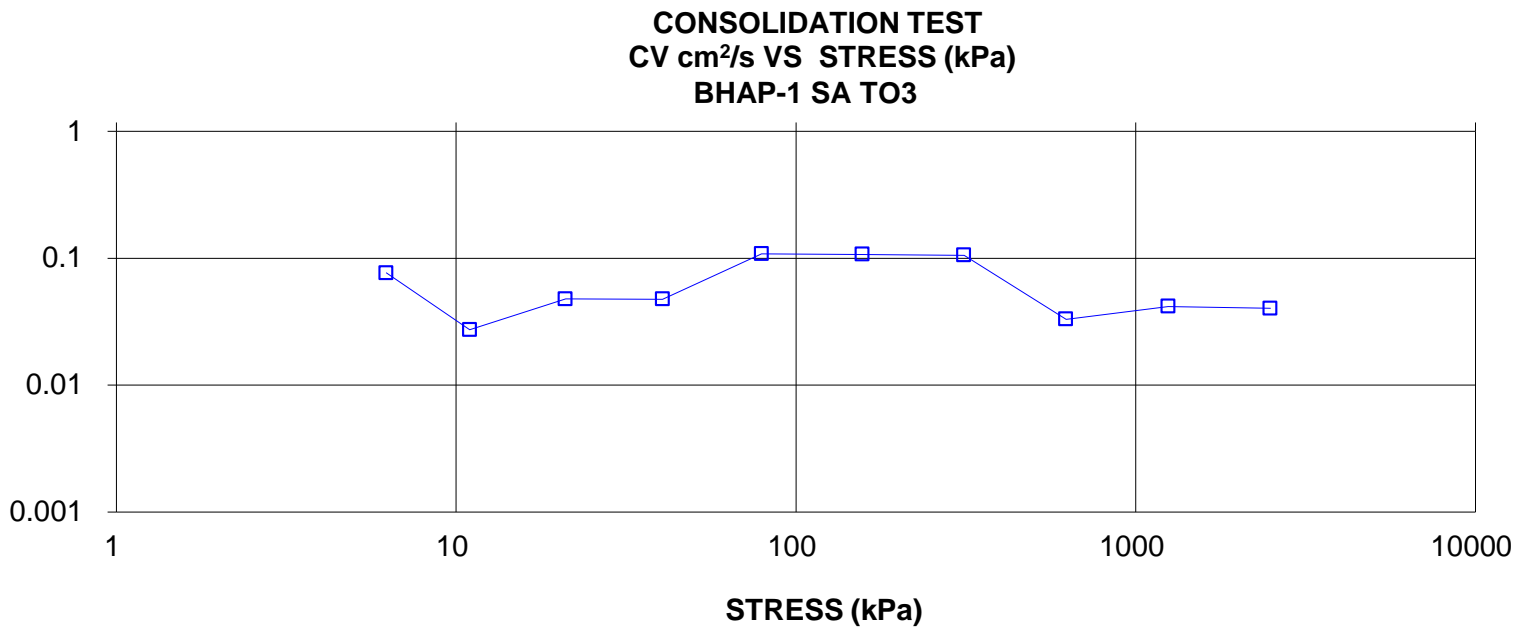




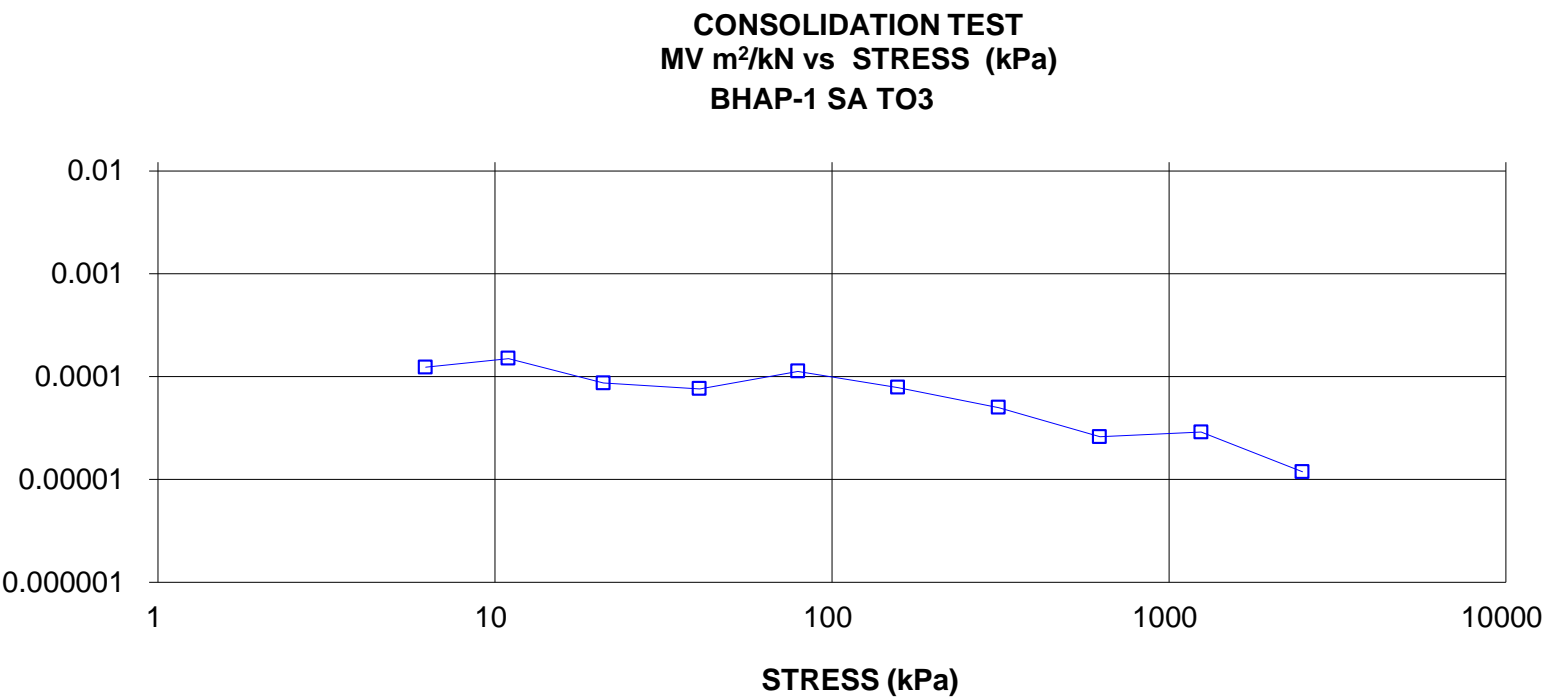
CONSOLIDATION TEST SUMMARY					FIGURE C17A		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number		1671430(WO007)			Sample Number		TO3
Borehole Number		AP-1			Sample Depth, m		27.4-28.0
TEST CONDITIONS							
Test Type		QUICK			Load Duration, hr		-
Oedometer Number		5					
Date Started		14/08/2019					
Date Completed		16/08/2019					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		1.90			Unit Weight, kN/m <sup>3</sup>		20.88
Sample Diameter, cm		6.33			Dry Unit Weight, kN/m <sup>3</sup>		17.52
Area, cm <sup>2</sup>		31.46			Specific Gravity, measured		2.72
Volume, cm <sup>3</sup>		59.84			Solids Height, cm		1.250
Water Content, %		19.13			Volume of Solids, cm <sup>3</sup>		39.31
Wet Mass, g		127.39			Volume of Voids, cm <sup>3</sup>		20.52
Dry Mass, g		106.93			Degree of Saturation, %		99.7
TEST COMPUTATIONS							
	Corr.	Average					
Stress	Height	Void	Height	t <sub>90</sub>	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	1.902	0.522	1.902				
6.23	1.901	0.521	1.901	10	7.66E-02	1.23E-04	9.25E-07
10.97	1.899	0.520	1.900	28	2.73E-02	1.50E-04	4.01E-07
20.99	1.898	0.519	1.898	16	4.78E-02	8.65E-05	4.05E-07
40.51	1.895	0.516	1.896	16	4.76E-02	7.60E-05	3.55E-07
79.27	1.886	0.510	1.891	7	1.08E-01	1.13E-04	1.20E-06
157.09	1.875	0.500	1.881	7	1.07E-01	7.85E-05	8.24E-07
312.63	1.860	0.488	1.867	7	1.06E-01	5.00E-05	5.18E-07
623.86	1.845	0.476	1.852	22	3.31E-02	2.59E-05	8.41E-08
1246.24	1.811	0.449	1.828	17	4.17E-02	2.88E-05	1.18E-07
2491.30	1.782	0.426	1.796	17	4.02E-02	1.19E-05	4.69E-08
623.76	1.788	0.431	1.785				
157.09	1.795	0.437	1.791				
40.32	1.805	0.445	1.800				
6.23	1.817	0.454	1.811				
Note: Consolidation loading and unloading schedule assigned by the client. k calculated using cv based on t <sub>90</sub> values. Specimen taken 6-20cm from top of the tube.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		1.82			Unit Weight, kN/m <sup>3</sup>		21.55
Sample Diameter, cm		6.33			Dry Unit Weight, kN/m <sup>3</sup>		18.34
Area, cm <sup>2</sup>		31.46			Specific Gravity, measured		2.72
Volume, cm <sup>3</sup>		57.18			Solids Height, cm		1.250
Water Content, %		17.48			Volume of Solids, cm <sup>3</sup>		39.31
Wet Mass, g		125.62			Volume of Voids, cm <sup>3</sup>		17.86
Dry Mass, g		106.93					
Prepared By: LH		Golder Associates				Checked By:	



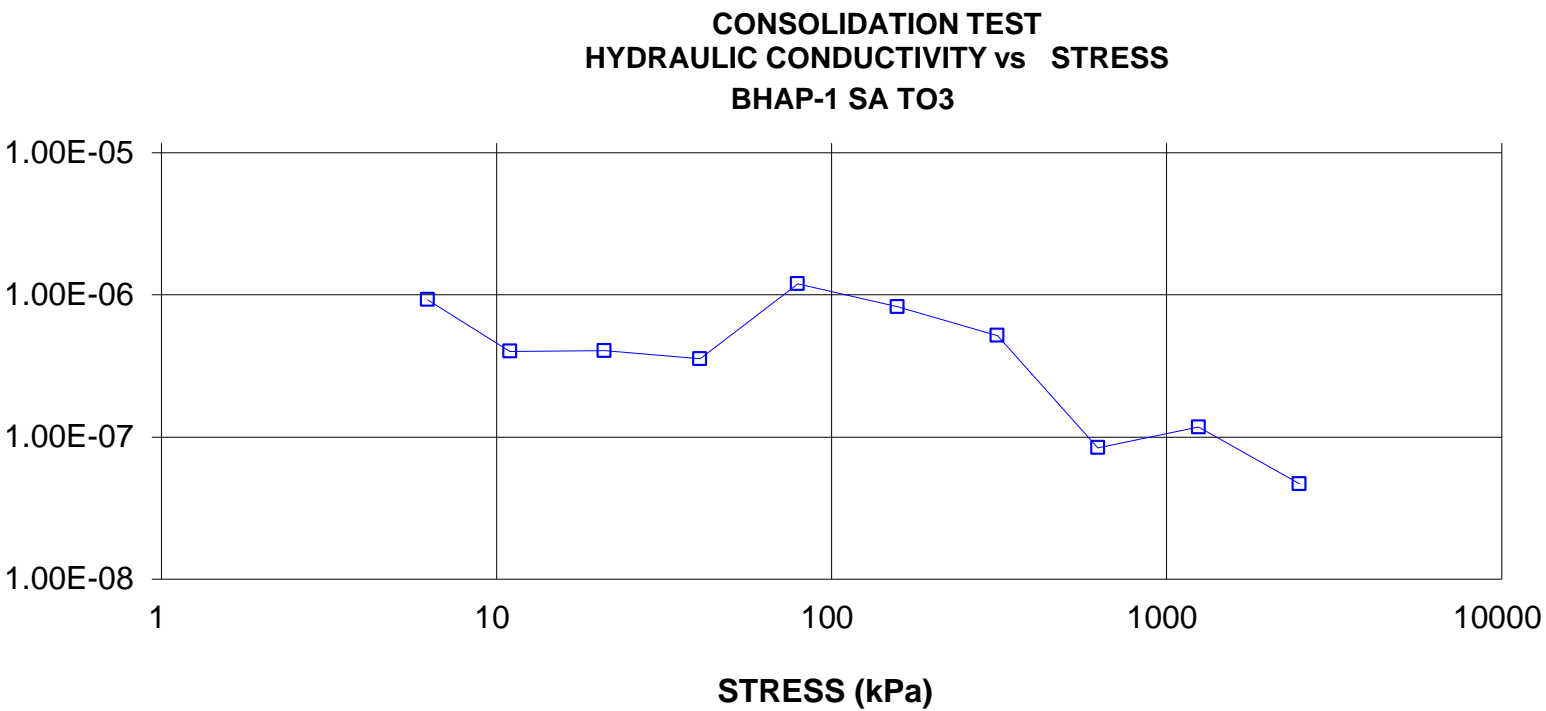
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



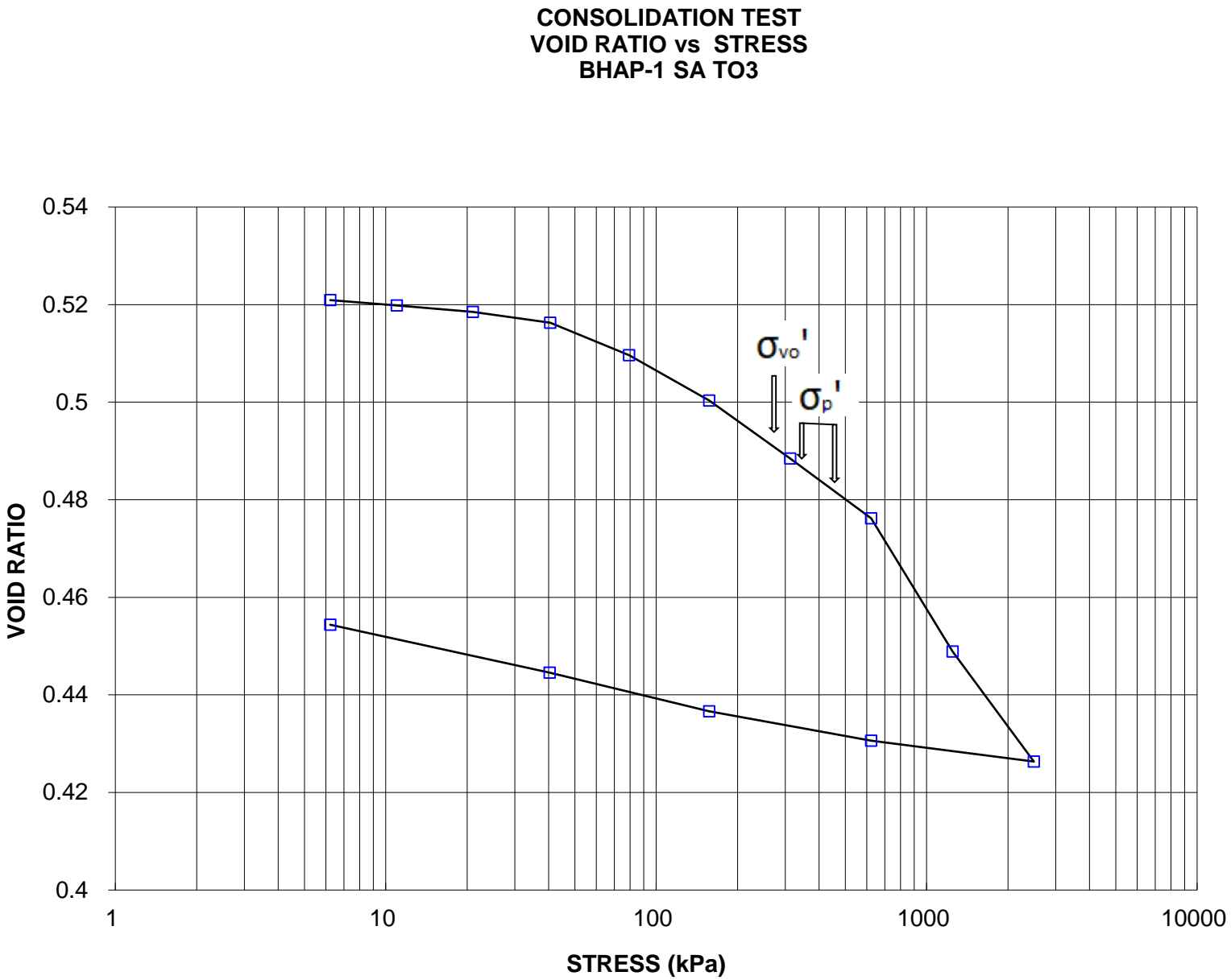
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



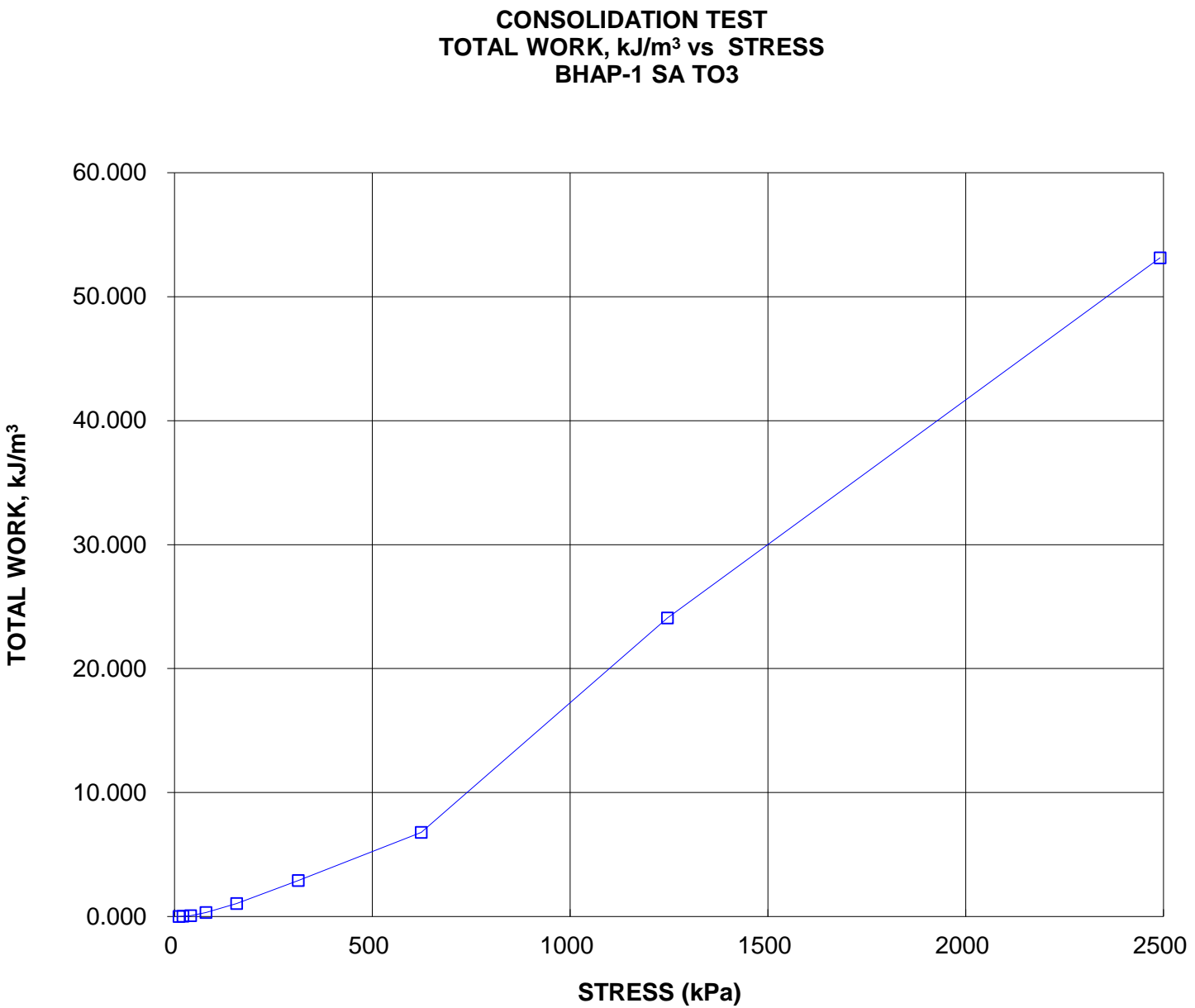
HYDRAULIC CONDUCTIVITY, cm/s









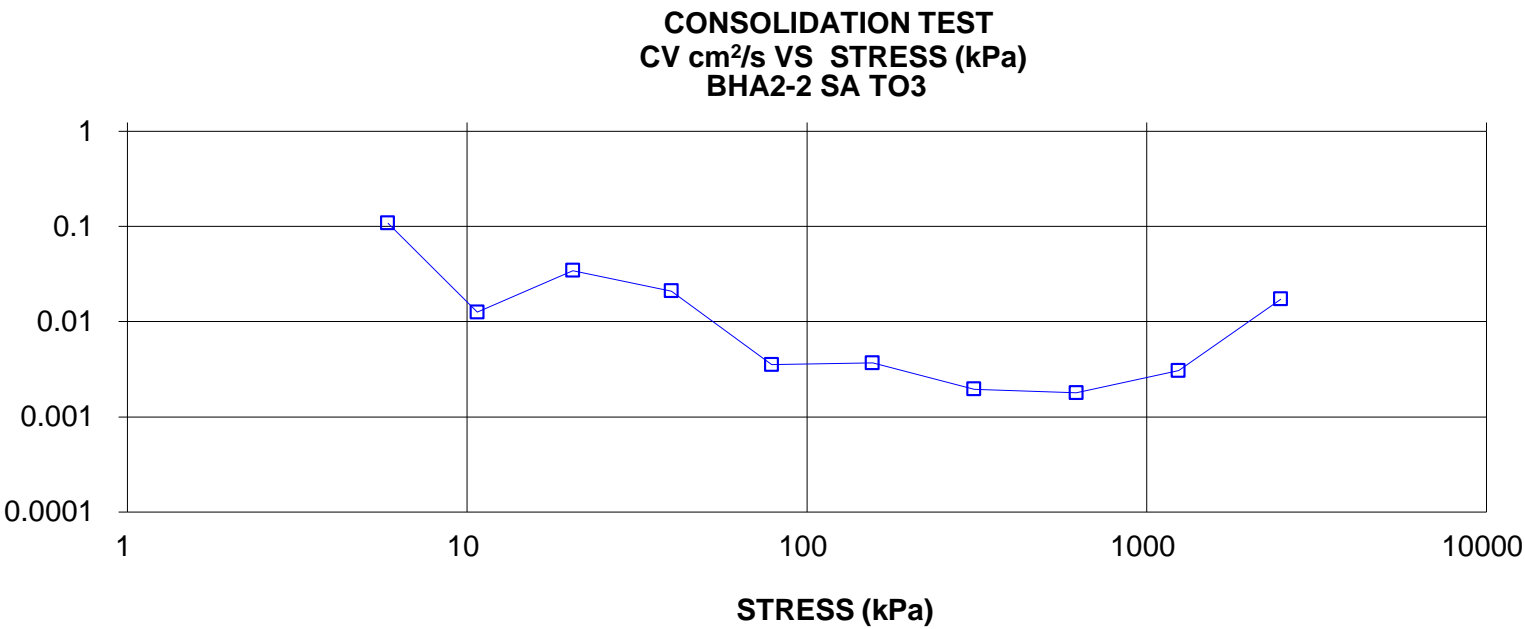




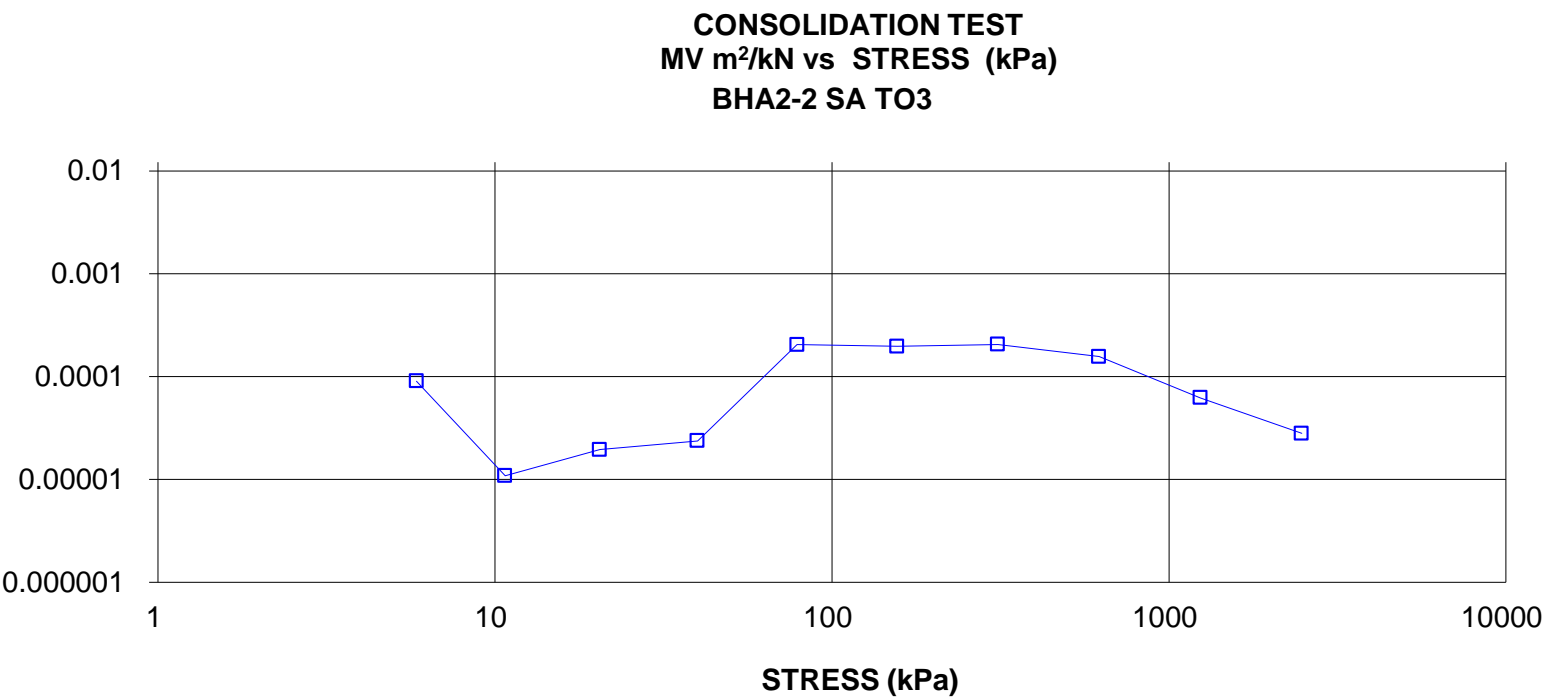
CONSOLIDATION TEST SUMMARY					FIGURE C18A		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number	1671430(WO007)			Sample Number	TO3		
Borehole Number	A2-2			Sample Depth, m	24.6-25.0		
TEST CONDITIONS							
Test Type	QUICK			Load Duration, hr	-		
Oedometer Number	6						
Date Started	14/08/2019						
Date Completed	16/08/2019						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.89			Unit Weight, kN/m <sup>3</sup>	18.78		
Sample Diameter, cm	6.34			Dry Unit Weight, kN/m <sup>3</sup>	14.17		
Area, cm <sup>2</sup>	31.60			Specific Gravity, measured	2.72		
Volume, cm <sup>3</sup>	59.69			Solids Height, cm	1.003		
Water Content, %	32.57			Volume of Solids, cm <sup>3</sup>	31.71		
Wet Mass, g	114.33			Volume of Voids, cm <sup>3</sup>	27.99		
Dry Mass, g	86.24			Degree of Saturation, %	100.4		
TEST COMPUTATIONS							
	Corr.		Average				
Stress	Height	Void	Height	t <sub>90</sub>	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	1.889	0.883	1.889				
5.85	1.888	0.882	1.889	7	1.08E-01	9.05E-05	9.58E-07
10.72	1.888	0.882	1.888	60	1.26E-02	1.09E-05	1.34E-08
20.47	1.888	0.881	1.888	22	3.43E-02	1.95E-05	6.58E-08
39.91	1.887	0.880	1.887	36	2.10E-02	2.37E-05	4.87E-08
78.82	1.872	0.865	1.879	212	3.53E-03	2.04E-04	7.08E-08
156.24	1.843	0.837	1.857	199	3.67E-03	1.97E-04	7.10E-08
310.86	1.783	0.777	1.813	357	1.95E-03	2.05E-04	3.93E-08
620.71	1.691	0.685	1.737	357	1.79E-03	1.57E-04	2.76E-08
1240.48	1.618	0.613	1.655	190	3.05E-03	6.22E-05	1.86E-08
2479.35	1.552	0.547	1.585	31	1.72E-02	2.81E-05	4.73E-08
620.71	1.566	0.561	1.559				
156.24	1.587	0.581	1.576				
40.11	1.610	0.604	1.598				
5.85	1.657	0.651	1.633				
Note: Consolidation loading and unloading schedule assigned by the client. k calculated using cv based on t <sub>90</sub> values. Specimen taken 12-18cm from top of the tube.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.66			Unit Weight, kN/m <sup>3</sup>	20.07		
Sample Diameter, cm	6.34			Dry Unit Weight, kN/m <sup>3</sup>	16.15		
Area, cm <sup>2</sup>	31.60			Specific Gravity, measured	2.72		
Volume, cm <sup>3</sup>	52.35			Solids Height, cm	1.003		
Water Content, %	24.21			Volume of Solids, cm <sup>3</sup>	31.71		
Wet Mass, g	107.12			Volume of Voids, cm <sup>3</sup>	20.65		
Dry Mass, g	86.24						
Prepared By: LH		Golder Associates			Checked By:		



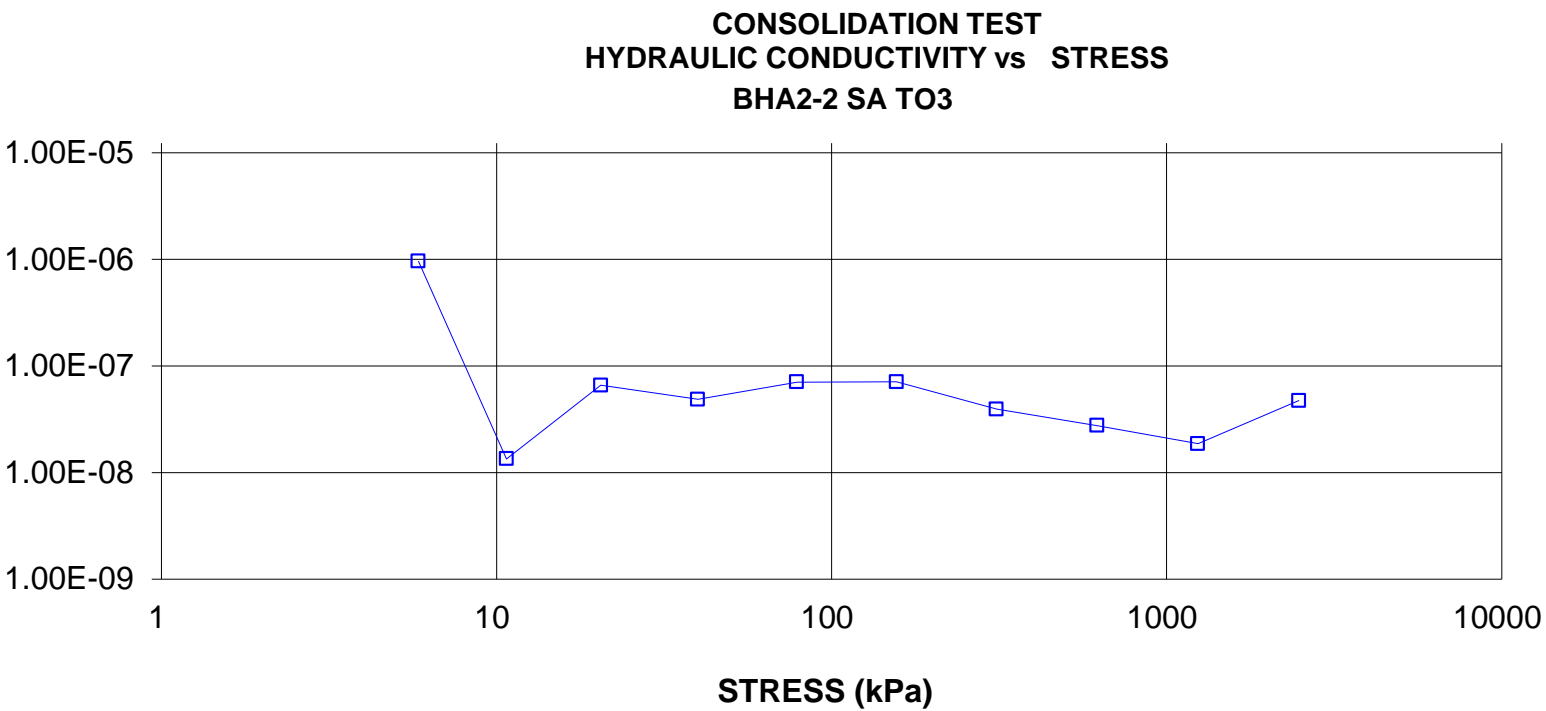
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



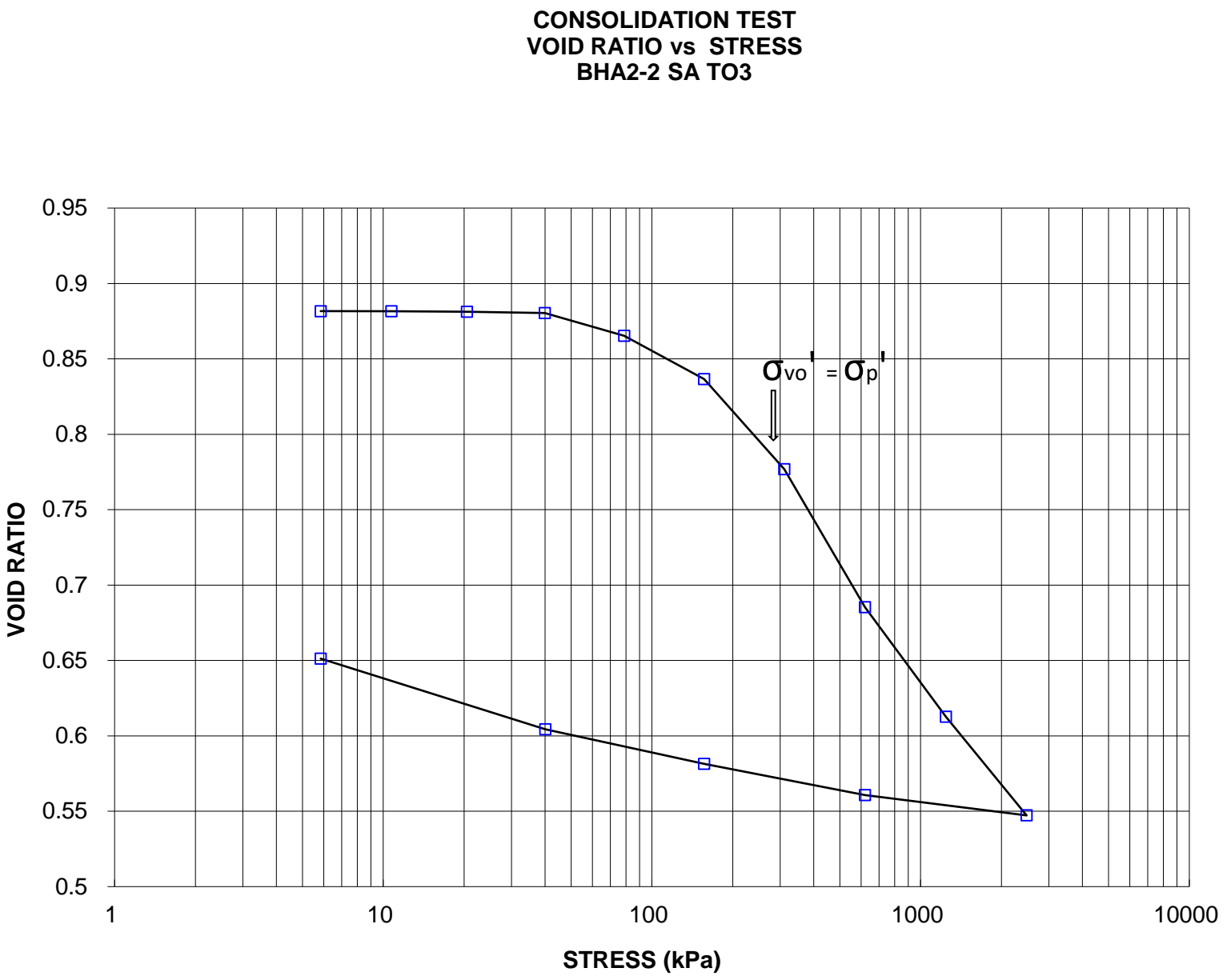
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



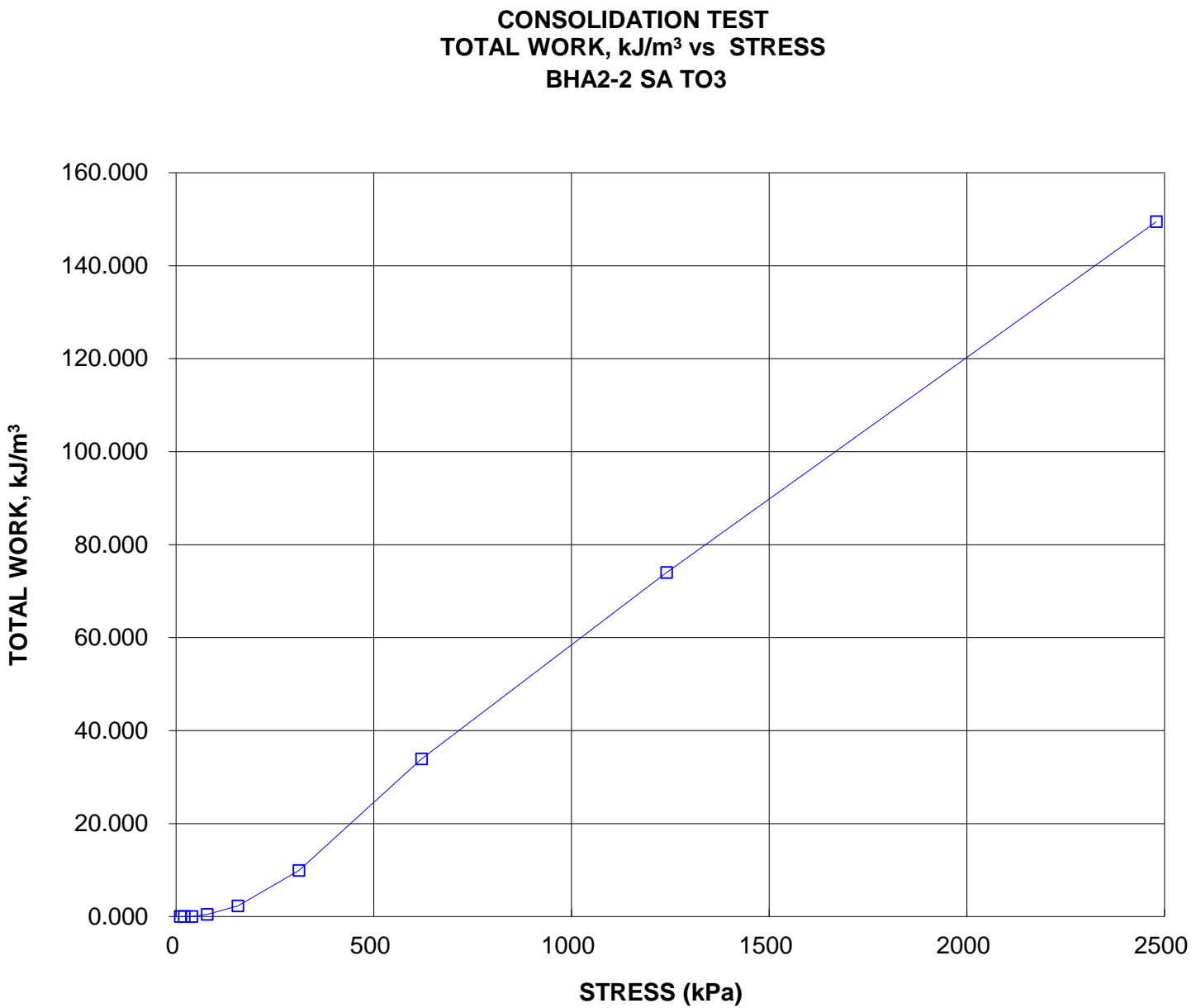
HYDRAULIC CONDUCTIVITY, cm/s









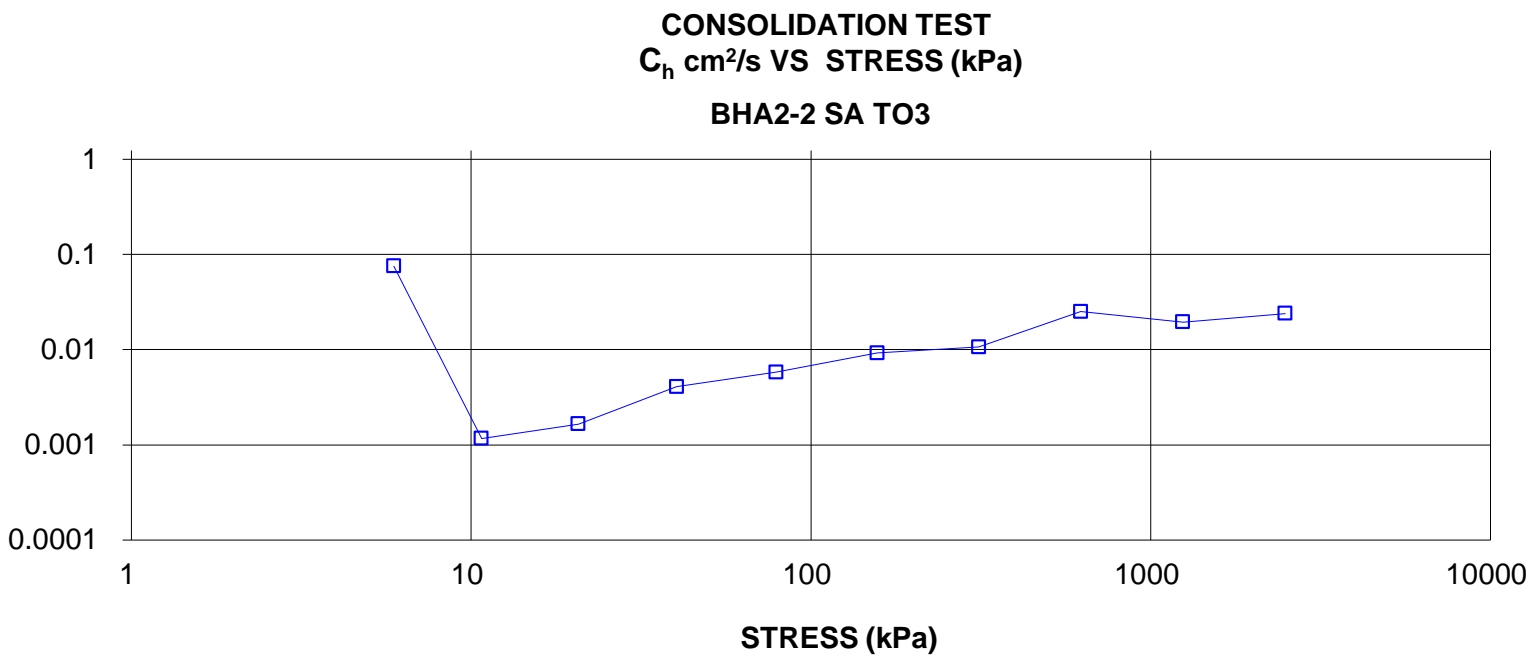




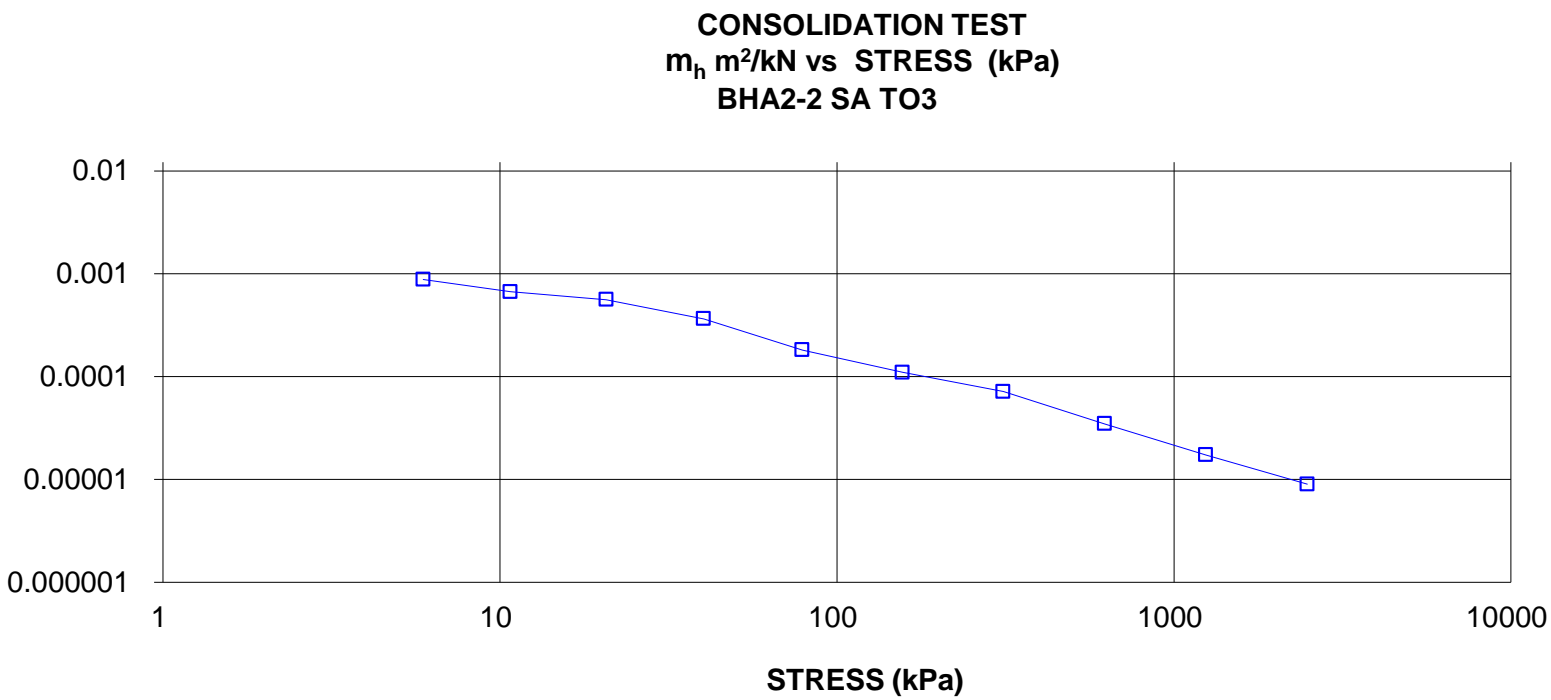
CONSOLIDATION TEST SUMMARY					FIGURE C19A		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number		1671430(WO007)		Sample Number		TO3	
Borehole Number		A2-2		Sample Depth, m		24.62-25.0	
TEST CONDITIONS							
Test Type		QUICK / VTO		Load Duration, hr		-	
Oedometer Number		7					
Date Started		10/10/2019					
Date Completed		10/11/2019					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		1.89		Unit Weight, kN/m <sup>3</sup>		20.40	
Sample Diameter, cm		6.33		Dry Unit Weight, kN/m <sup>3</sup>		16.76	
Area, cm <sup>2</sup>		31.50		Specific Gravity, measured		2.73	
Volume, cm <sup>3</sup>		59.53		Solids Height, cm		1.183	
Water Content, %		21.71		Volume of Solids, cm <sup>3</sup>		37.27	
Wet Mass, g		123.85		Volume of Voids, cm <sup>3</sup>		22.26	
Dry Mass, g		101.76		Degree of Saturation, %		99.2	
TEST COMPUTATIONS							
	Corr.		Average				
Stress	Height	Void	Height	t <sub>90</sub>	c <sub>h</sub>	m <sub>h</sub>	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	1.890	0.597	1.890				
5.92	1.880	0.589	1.885	10	7.53E-02	8.81E-04	6.51E-06
10.73	1.874	0.584	1.877	641	1.17E-03	6.70E-04	7.65E-08
20.68	1.864	0.575	1.869	448	1.65E-03	5.60E-04	9.07E-08
40.26	1.850	0.563	1.857	179	4.08E-03	3.66E-04	1.46E-07
79.01	1.837	0.552	1.843	124	5.81E-03	1.81E-04	1.03E-07
156.71	1.821	0.538	1.829	77	9.21E-03	1.10E-04	9.93E-08
311.87	1.800	0.521	1.810	65	1.07E-02	7.14E-05	7.48E-08
622.65	1.779	0.504	1.789	27	2.51E-02	3.47E-05	8.55E-08
1244.44	1.759	0.486	1.769	34	1.95E-02	1.73E-05	3.31E-08
2487.89	1.738	0.468	1.748	27	2.40E-02	9.01E-06	2.12E-08
622.65	1.742	0.472	1.740				
156.76	1.752	0.481	1.747				
40.07	1.761	0.488	1.757				
10.76	1.772	0.497	1.766				
Note: Consolidation loading and unloading schedule assigned by the client. k calculated using c <sub>h</sub> based on t <sub>90</sub> values. Testing carried out on VTO (Vertically Trimmed Orientation) specimens in order to evaluate the horizontal properties.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		1.77		Unit Weight, kN/m <sup>3</sup>		21.05	
Sample Diameter, cm		6.33		Dry Unit Weight, kN/m <sup>3</sup>		17.88	
Area, cm <sup>2</sup>		31.50		Specific Gravity, measured		2.73	
Volume, cm <sup>3</sup>		55.81		Solids Height, cm		1.183	
Water Content, %		17.71		Volume of Solids, cm <sup>3</sup>		37.27	
Wet Mass, g		119.78		Volume of Voids, cm <sup>3</sup>		18.53	
Dry Mass, g		101.76					
Prepared By: LH		Golder Associates				Checked By:	



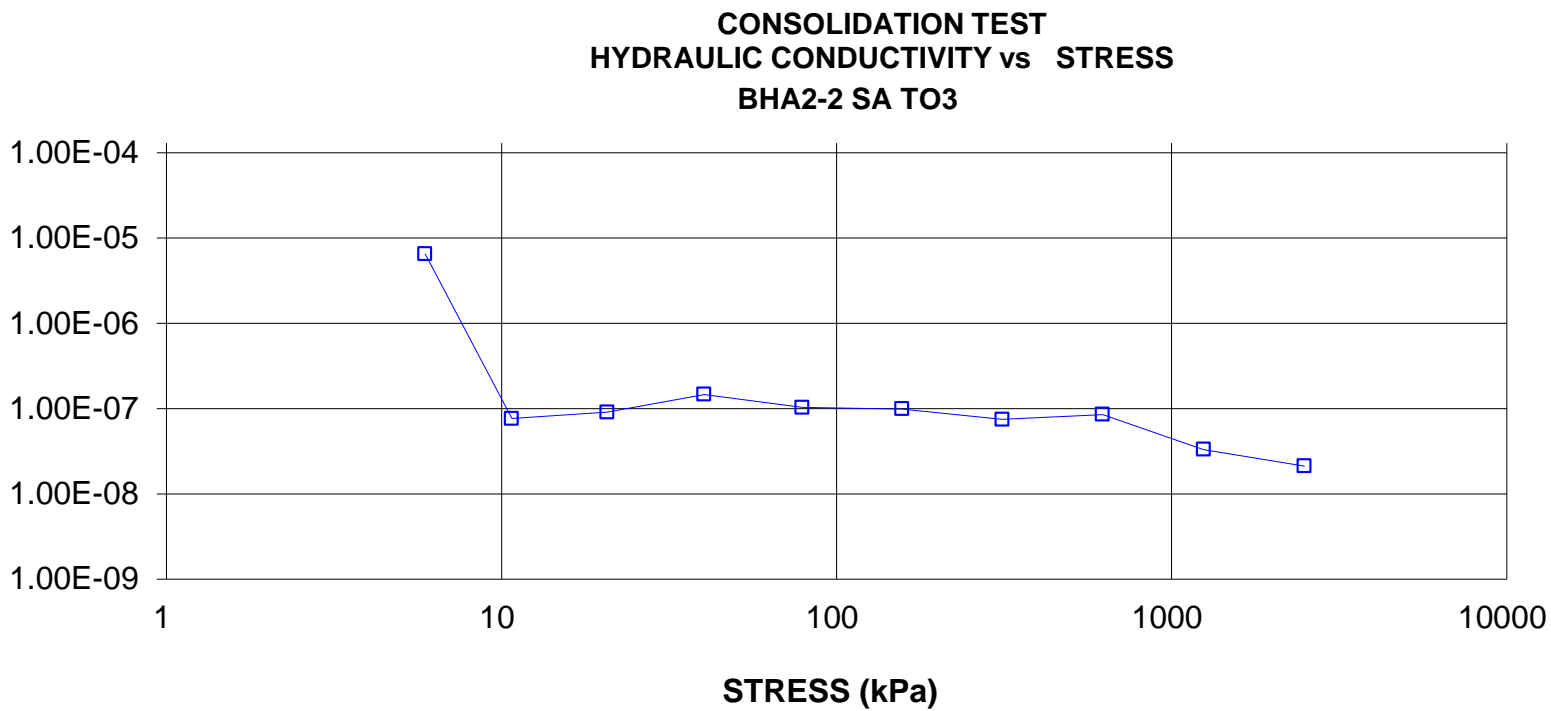
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



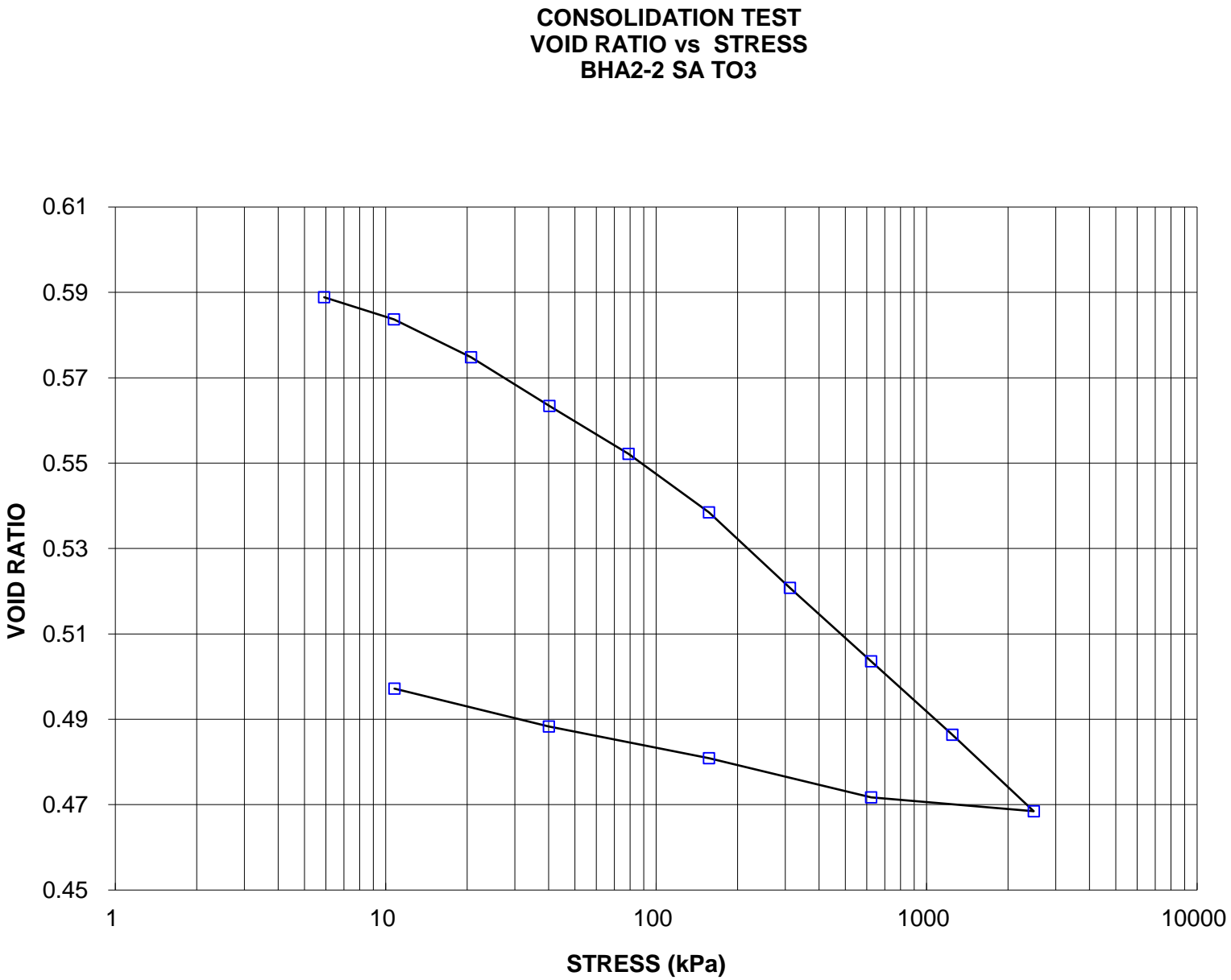
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



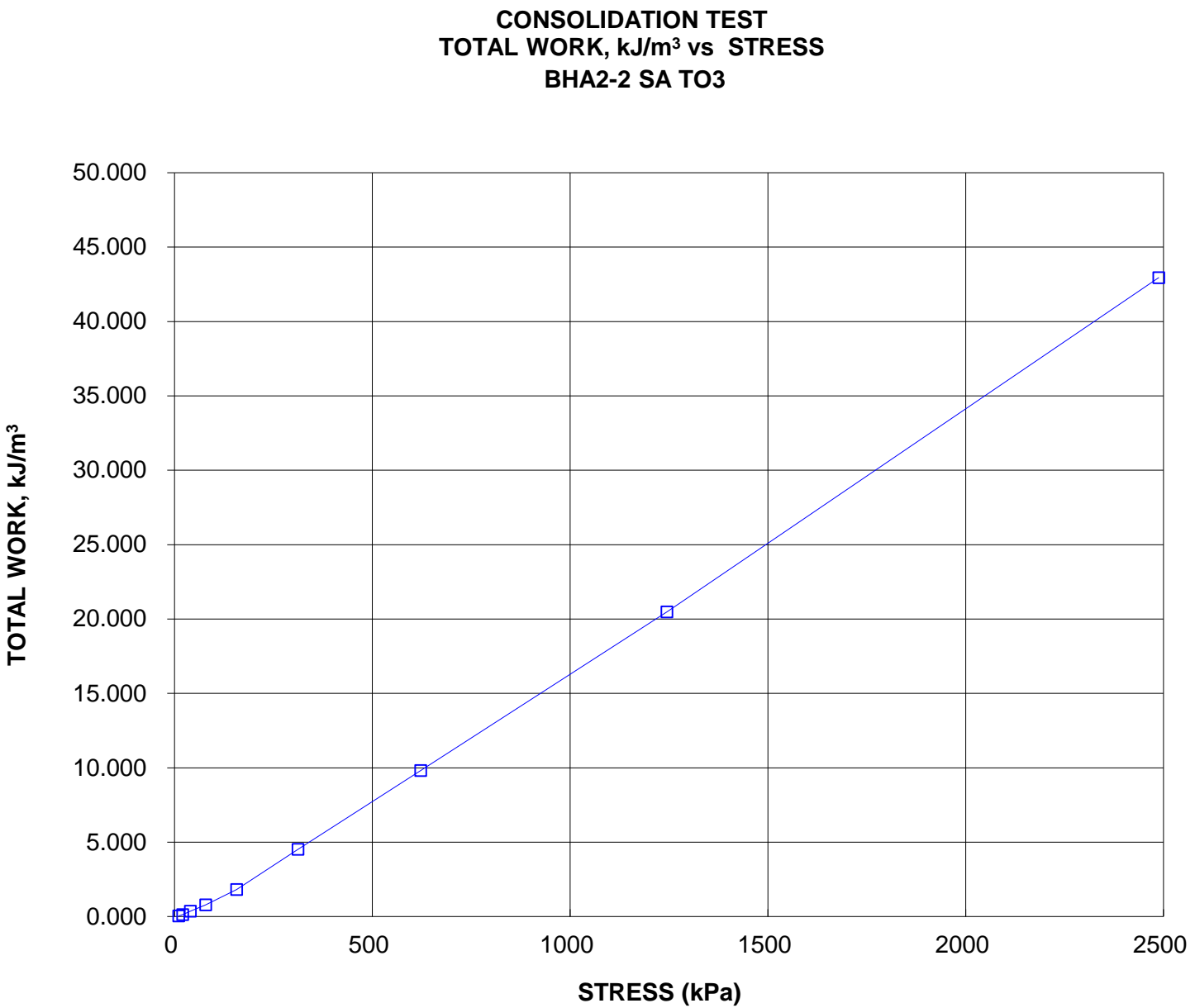
HYDRAULIC CONDUCTIVITY, cm/s









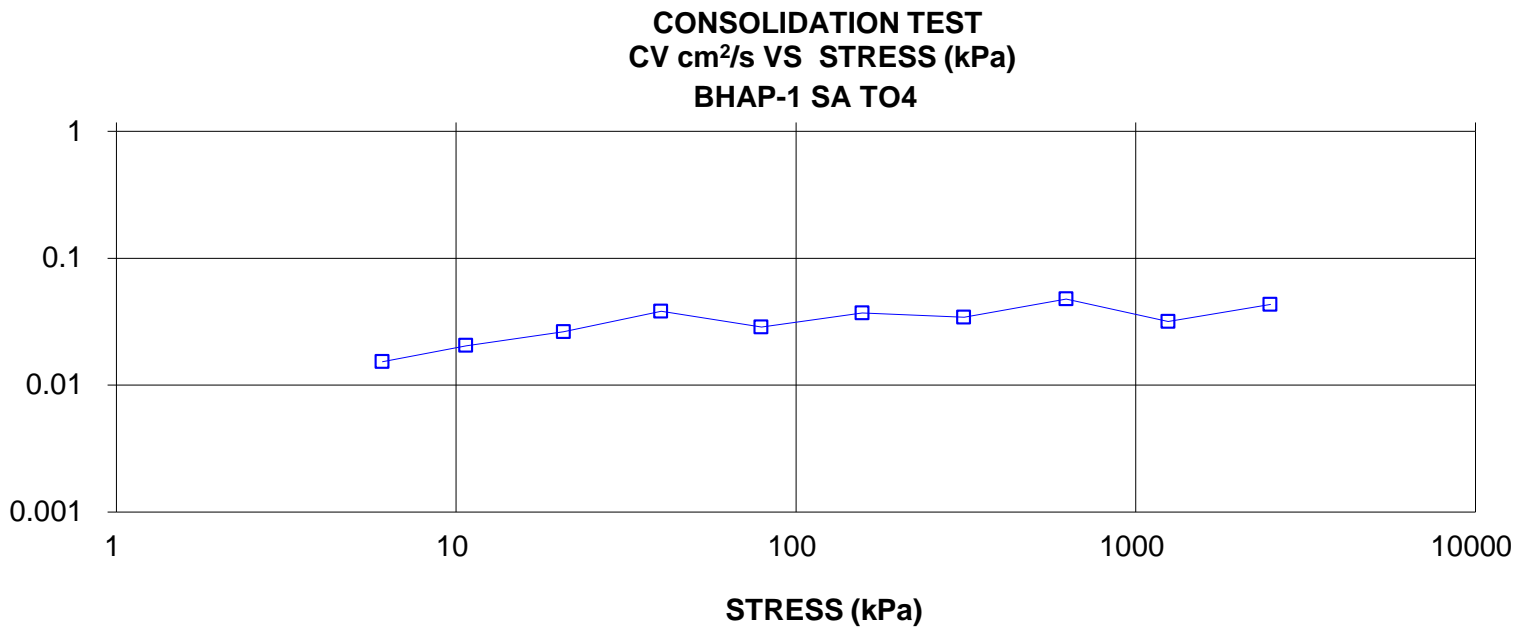




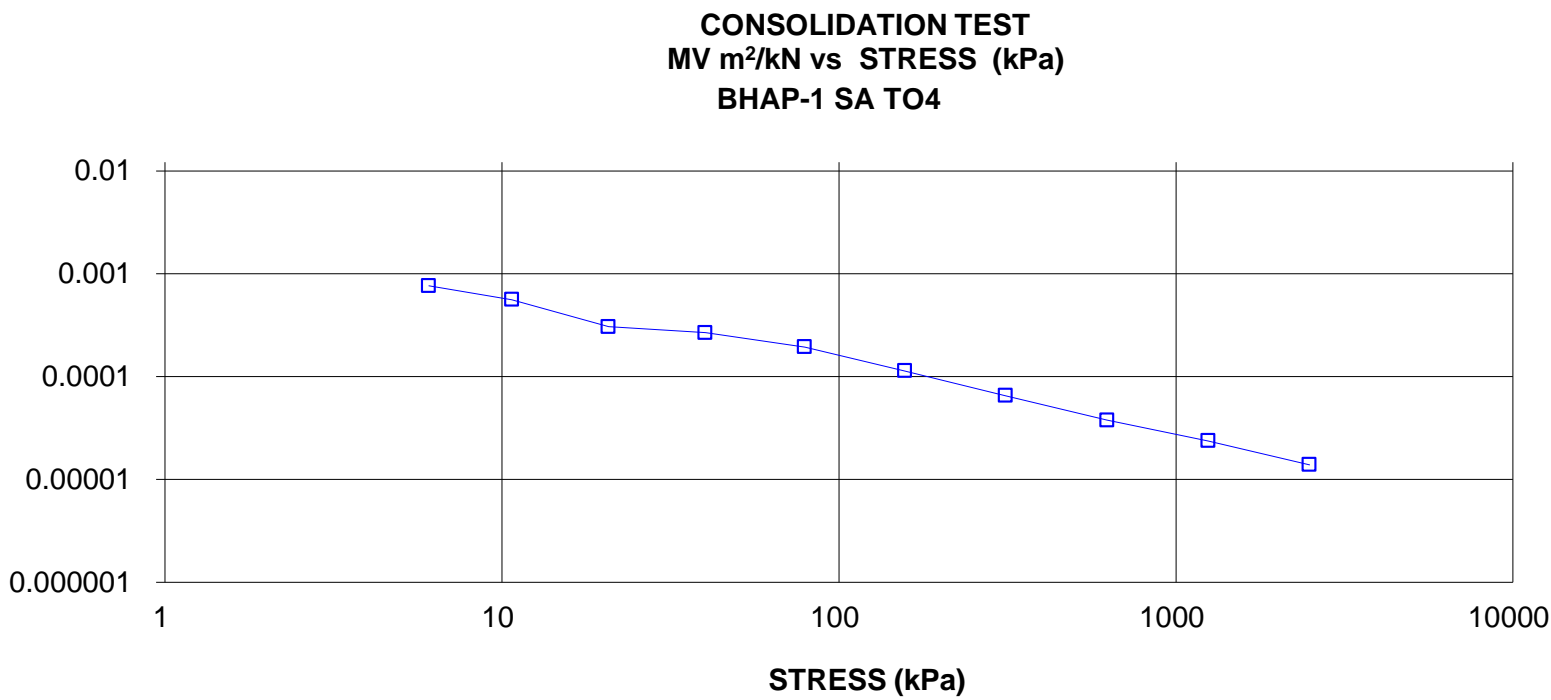
CONSOLIDATION TEST SUMMARY					FIGURE C20A		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number	1671430(WO007)			Sample Number	TO4		
Borehole Number	AP-1			Sample Depth, m	24.4-25.01		
TEST CONDITIONS							
Test Type	QUICK			Load Duration, hr	-		
Oedometer Number	4						
Date Started	14/08/2019						
Date Completed	16/08/2019						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	2.54			Unit Weight, kN/m <sup>3</sup>	20.23		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m <sup>3</sup>	16.49		
Area, cm <sup>2</sup>	31.45			Specific Gravity, measured	2.72		
Volume, cm <sup>3</sup>	79.88			Solids Height, cm	1.570		
Water Content, %	22.68			Volume of Solids, cm <sup>3</sup>	49.39		
Wet Mass, g	164.79			Volume of Voids, cm <sup>3</sup>	30.50		
Dry Mass, g	134.33			Degree of Saturation, %	99.9		
TEST COMPUTATIONS							
	Corr.	Average					
Stress	Height	Void	Height	t <sub>90</sub>	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	2.540	0.618	2.540				
6.07	2.528	0.610	2.534	89	1.53E-02	7.65E-04	1.15E-06
10.70	2.522	0.606	2.525	66	2.05E-02	5.61E-04	1.13E-06
20.72	2.514	0.601	2.518	51	2.63E-02	3.06E-04	7.91E-07
40.14	2.501	0.592	2.507	35	3.81E-02	2.68E-04	9.99E-07
79.14	2.481	0.580	2.491	46	2.86E-02	1.94E-04	5.43E-07
156.86	2.459	0.566	2.470	35	3.70E-02	1.14E-04	4.13E-07
312.32	2.433	0.549	2.446	37	3.43E-02	6.53E-05	2.20E-07
624.44	2.403	0.530	2.418	26	4.77E-02	3.77E-05	1.76E-07
1247.22	2.366	0.507	2.384	38	3.17E-02	2.37E-05	7.37E-08
2492.88	2.322	0.479	2.344	27	4.31E-02	1.39E-05	5.86E-08
623.96	2.331	0.484	2.326				
156.89	2.343	0.492	2.337				
40.08	2.359	0.502	2.351				
6.07	2.380	0.516	2.370				
Note:							
Consolidation loading and unloading schedule assigned by the client.							
k calculated using cv based on t <sub>90</sub> values.							
Specimen taken 20-27.5cm from top of the tube.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	2.38			Unit Weight, kN/m <sup>3</sup>	21.03		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m <sup>3</sup>	17.60		
Area, cm <sup>2</sup>	31.45			Specific Gravity, measured	2.72		
Volume, cm <sup>3</sup>	74.86			Solids Height, cm	1.570		
Water Content, %	19.48			Volume of Solids, cm <sup>3</sup>	49.39		
Wet Mass, g	160.50			Volume of Voids, cm <sup>3</sup>	25.47		
Dry Mass, g	134.33						
Prepared By: LH		Golder Associates				Checked By:	



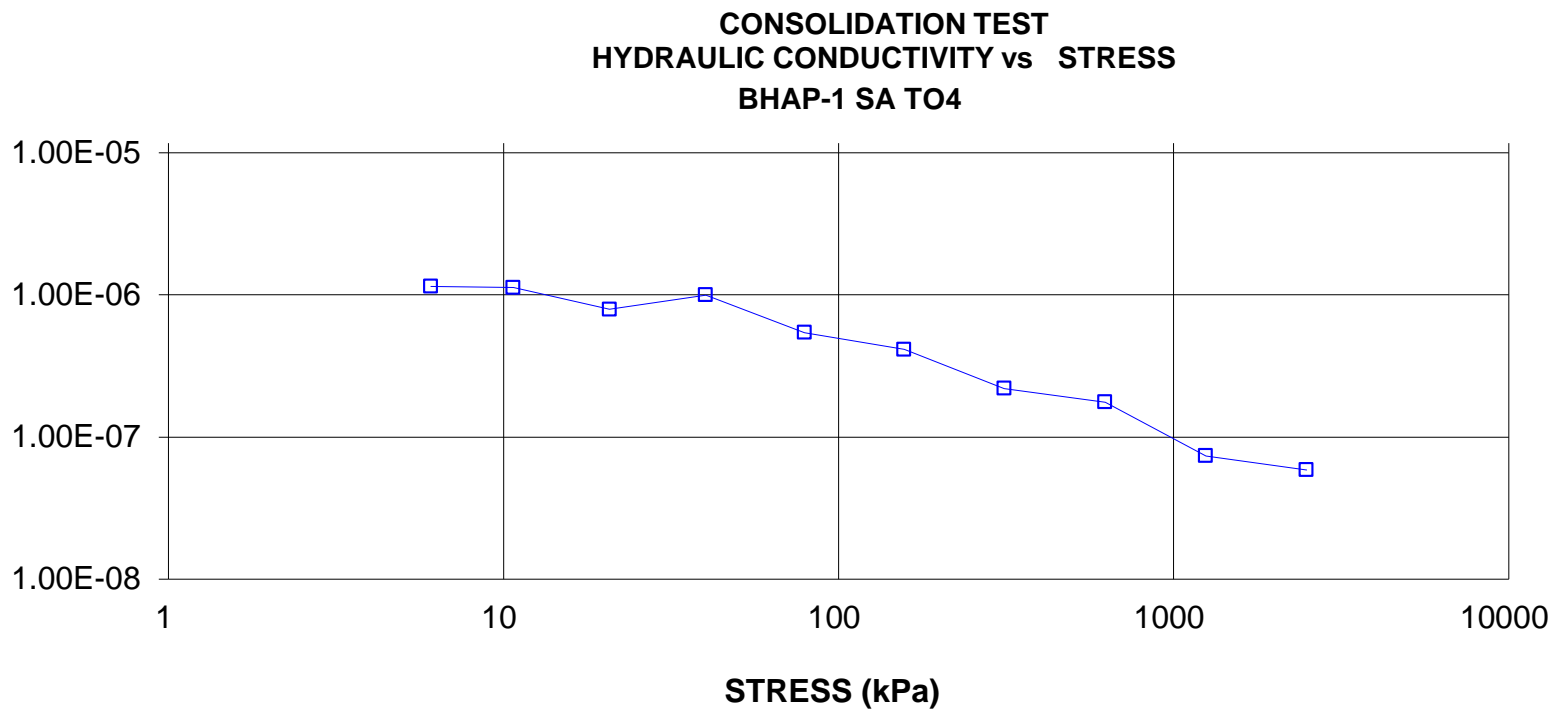
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



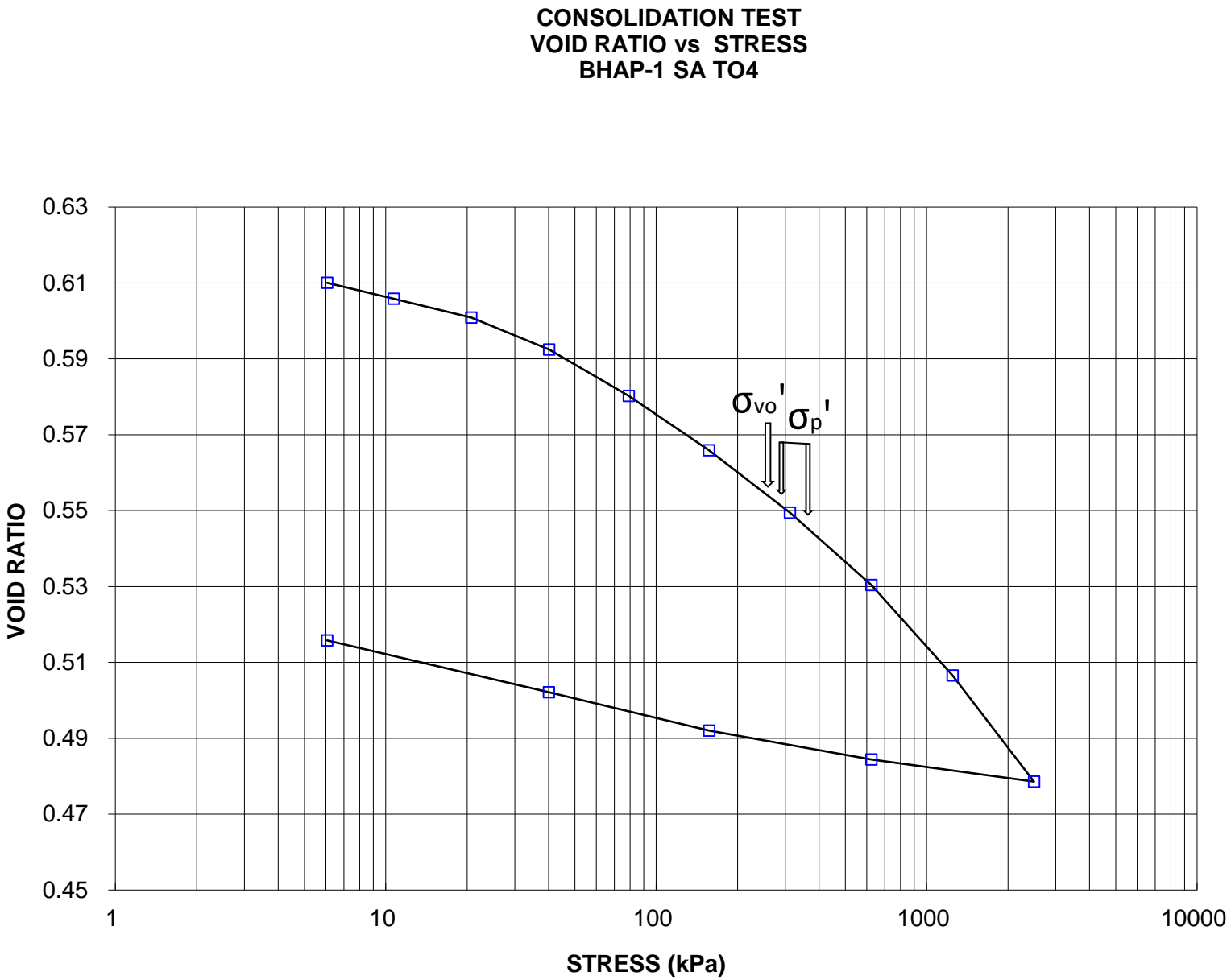
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



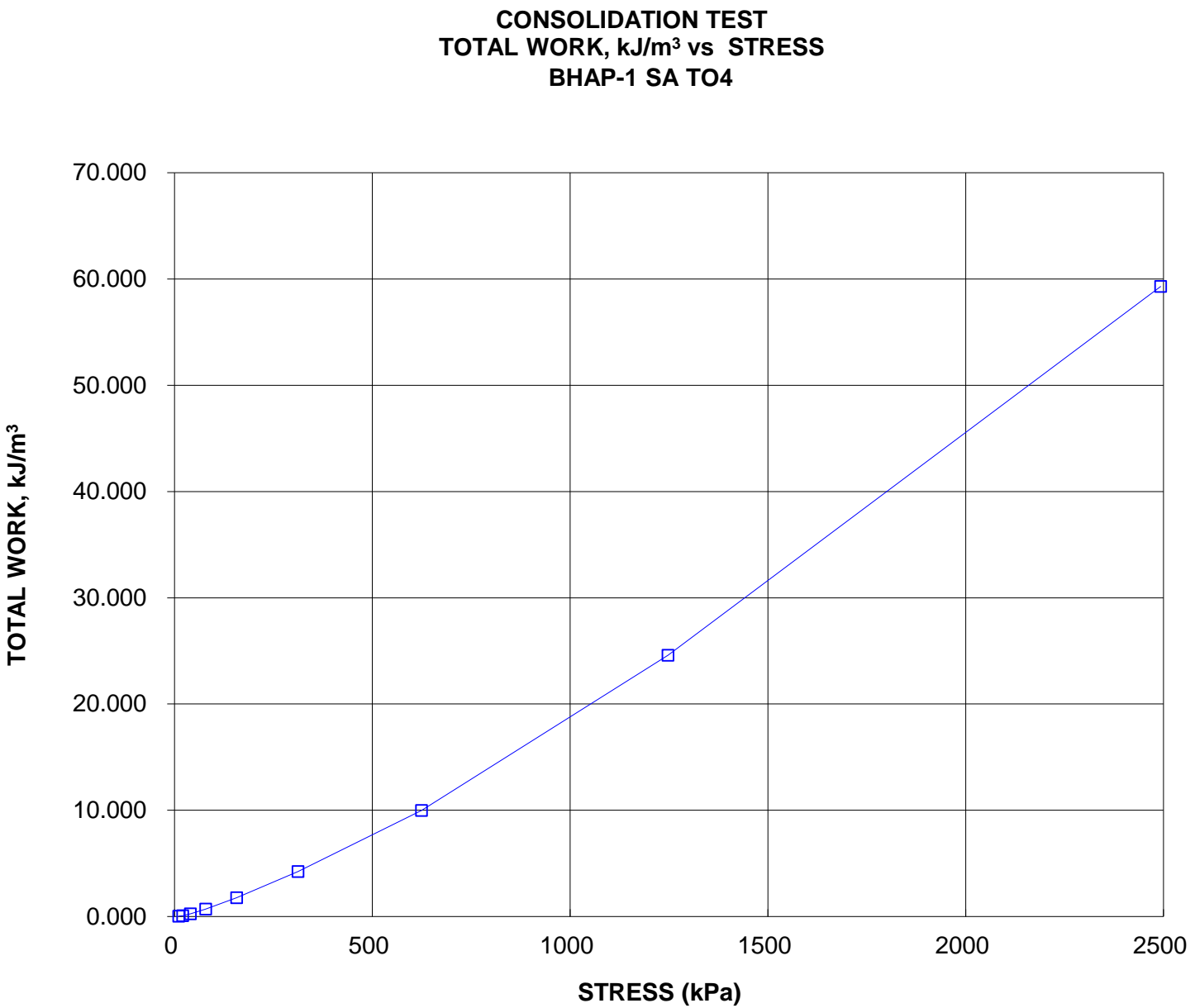
HYDRAULIC CONDUCTIVITY, cm/s







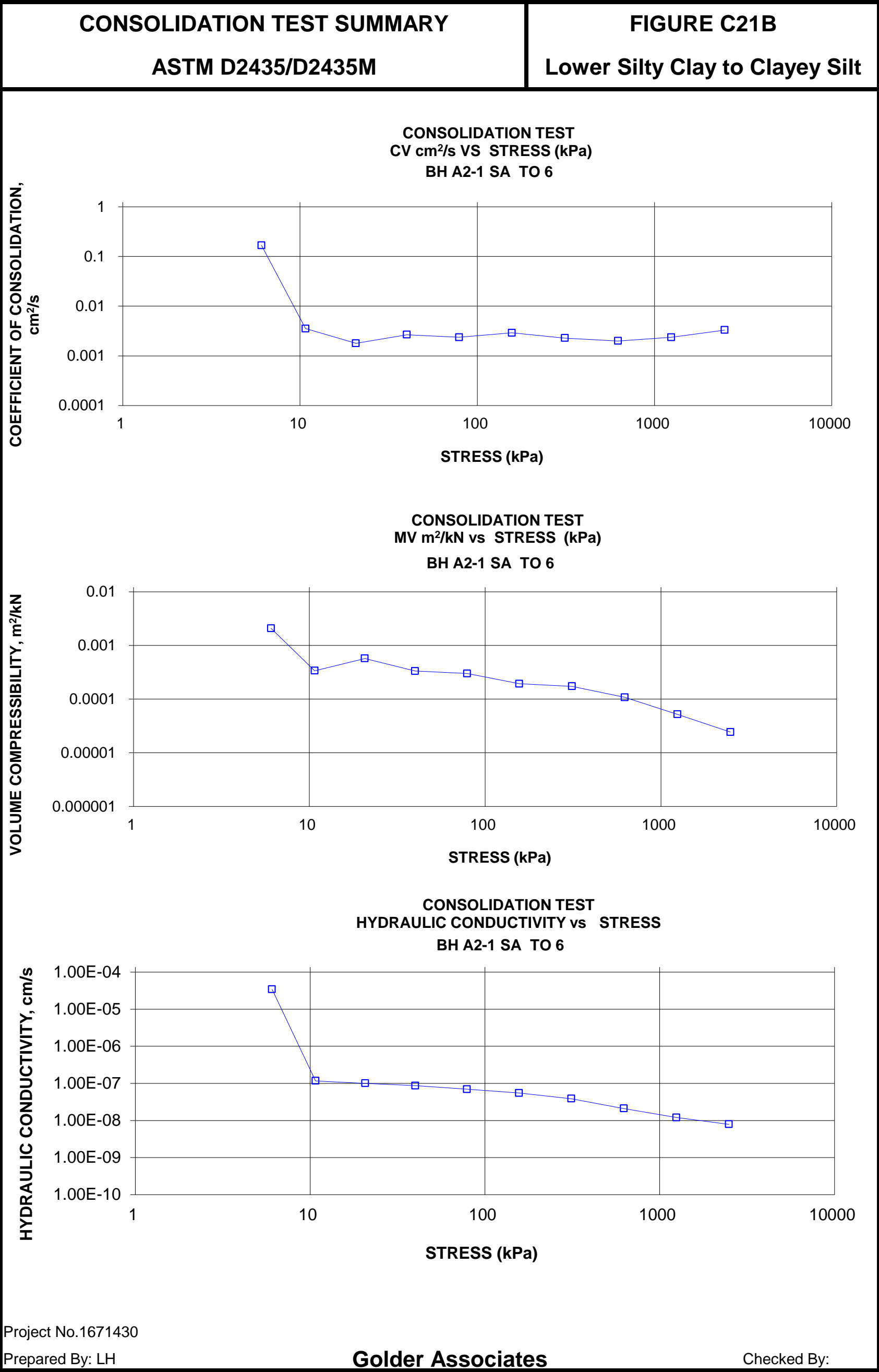




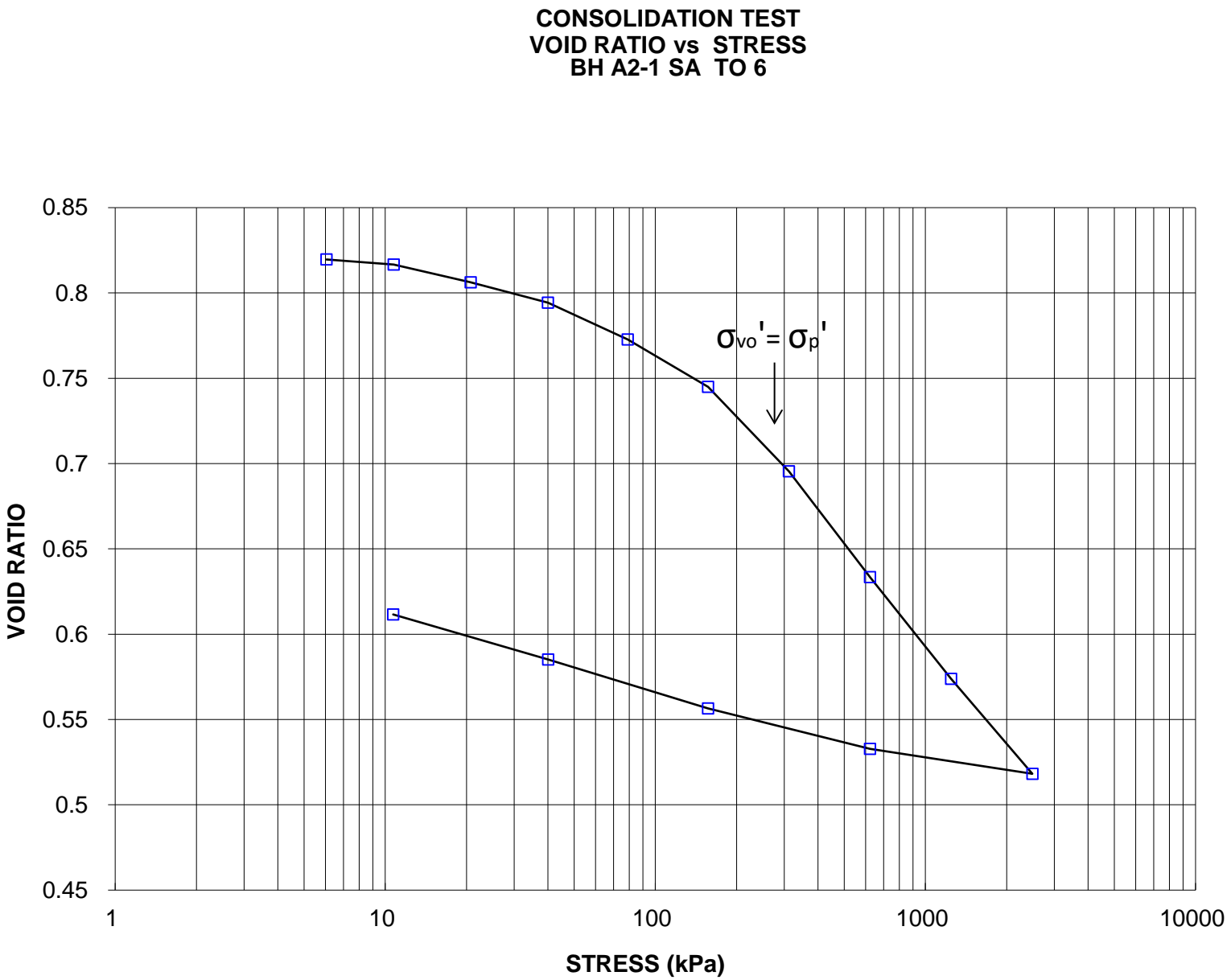


CONSOLIDATION TEST SUMMARY					FIGURE C21A			
ASTM D2435/D2435M					Lower Silty Clay to Clayey Silt			
SAMPLE IDENTIFICATION								
Project Number		1671430			Sample Number		TO 6	
Borehole Number		A2-1			Sample Depth, m		30.49-30.92	
TEST CONDITIONS								
Test Type		Laboratory Standard			Load Duration, hr		24	
Oedometer Number		3						
Date Started		04/02/2019						
Date Completed		04/15/2019						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		2.53			Unit Weight, kN/m <sup>3</sup>		18.92	
Sample Diameter, cm		6.33			Dry Unit Weight, kN/m <sup>3</sup>		14.69	
Area, cm <sup>2</sup>		31.48			Specific Gravity, measured		2.76	
Volume, cm <sup>3</sup>		79.74			Solids Height, cm		1.374	
Water Content, %		28.85			Volume of Solids, cm <sup>3</sup>		43.27	
Wet Mass, g		153.87			Volume of Voids, cm <sup>3</sup>		36.47	
Dry Mass, g		119.42			Degree of Saturation, %		94.5	
TEST COMPUTATIONS								
		Corr.	Average					
Stress	Height				t <sub>90</sub>	cv.	mv	k
kPa	cm	Void Ratio	Height cm		sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	2.533	0.843	2.533					
6.06	2.501	0.820	2.517	8		1.68E-01	2.08E-03	3.43E-05
10.74	2.497	0.817	2.499	375		3.53E-03	3.37E-04	1.17E-07
20.70	2.483	0.806	2.490	735		1.79E-03	5.71E-04	1.00E-07
40.05	2.466	0.794	2.474	487		2.67E-03	3.33E-04	8.69E-08
79.07	2.437	0.773	2.451	540		2.36E-03	3.00E-04	6.95E-08
156.74	2.399	0.745	2.418	427		2.90E-03	1.94E-04	5.51E-08
312.17	2.330	0.695	2.364	520		2.28E-03	1.73E-04	3.87E-08
623.23	2.245	0.633	2.288	558		1.99E-03	1.08E-04	2.11E-08
1245.36	2.163	0.574	2.204	437		2.36E-03	5.20E-05	1.20E-08
2487.21	2.087	0.518	2.125	290		3.30E-03	2.43E-05	7.87E-09
623.23	2.107	0.533	2.097					
156.70	2.139	0.557	2.123					
40.10	2.179	0.585	2.159					
10.70	2.215	0.612	2.197					
Note: Consolidation loading and unloading schedule assigned by the client. cv and k are approximate only based on t <sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M) Specimen taken 7-11 cm from top of the tube.								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		2.22			Unit Weight, kN/m <sup>3</sup>		20.80	
Sample Diameter, cm		6.33			Dry Unit Weight, kN/m <sup>3</sup>		16.80	
Area, cm <sup>2</sup>		31.48			Specific Gravity, measured		2.76	
Volume, cm <sup>3</sup>		69.73			Solids Height, cm		1.374	
Water Content, %		23.83			Volume of Solids, cm <sup>3</sup>		43.27	
Wet Mass, g		147.88			Volume of Voids, cm <sup>3</sup>		26.46	
Dry Mass, g		119.42						
Prepared By: LH				Golder Associates			Checked By:	





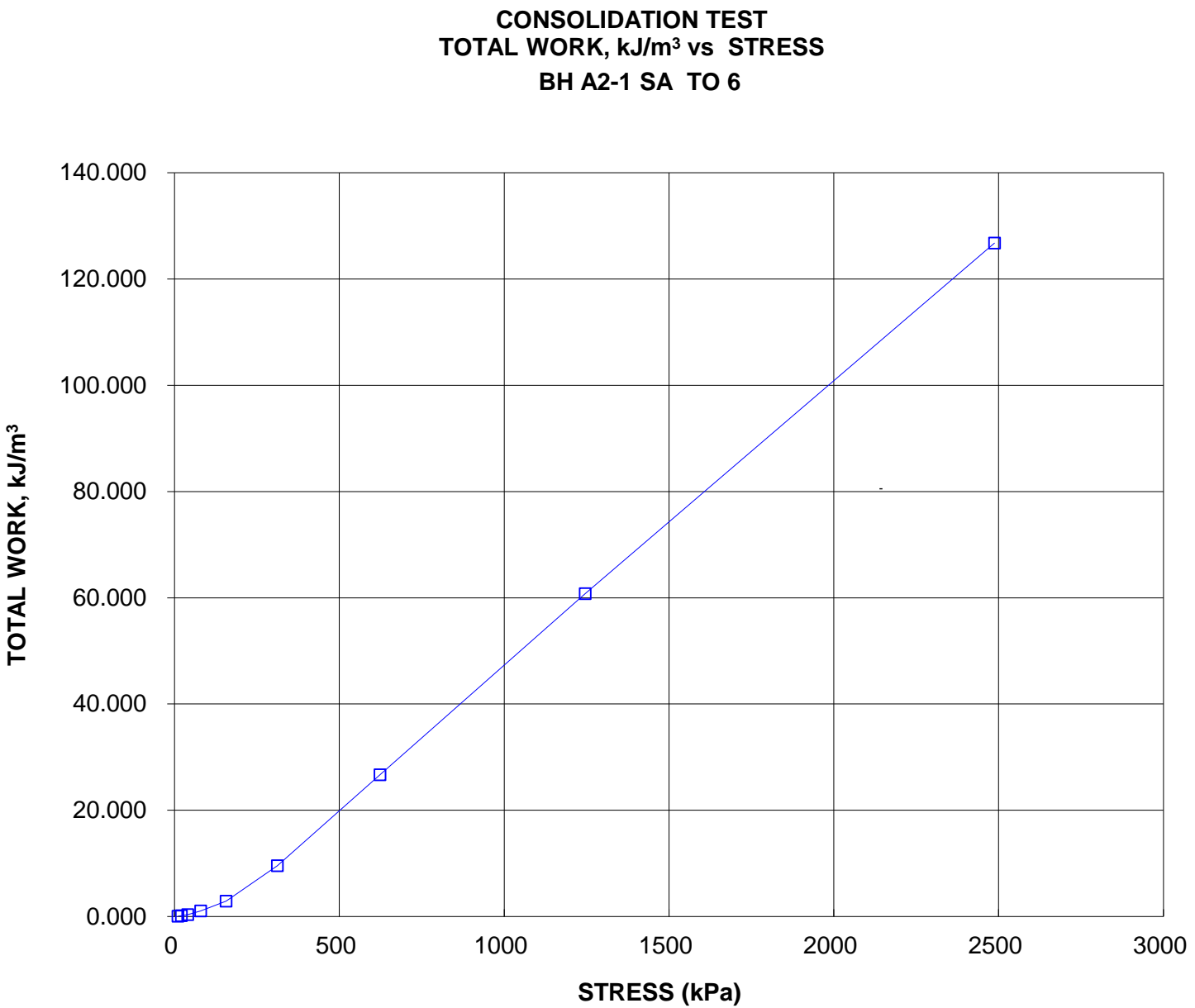






**CONSOLIDATION TEST  
TOTAL WORK VS STRESS**

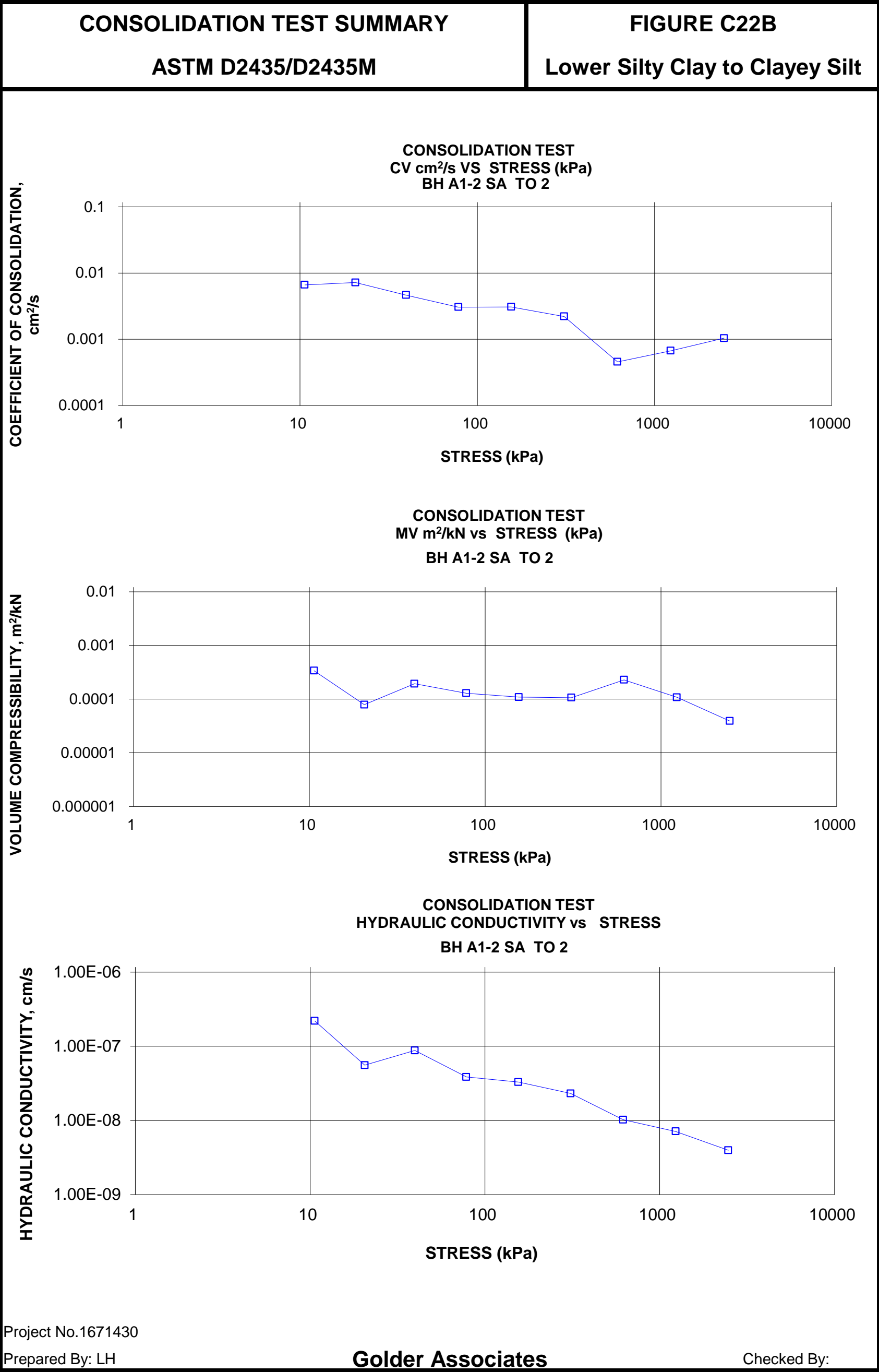
**FIGURE C21D  
Lower Silty Clay to Clayey Silt**



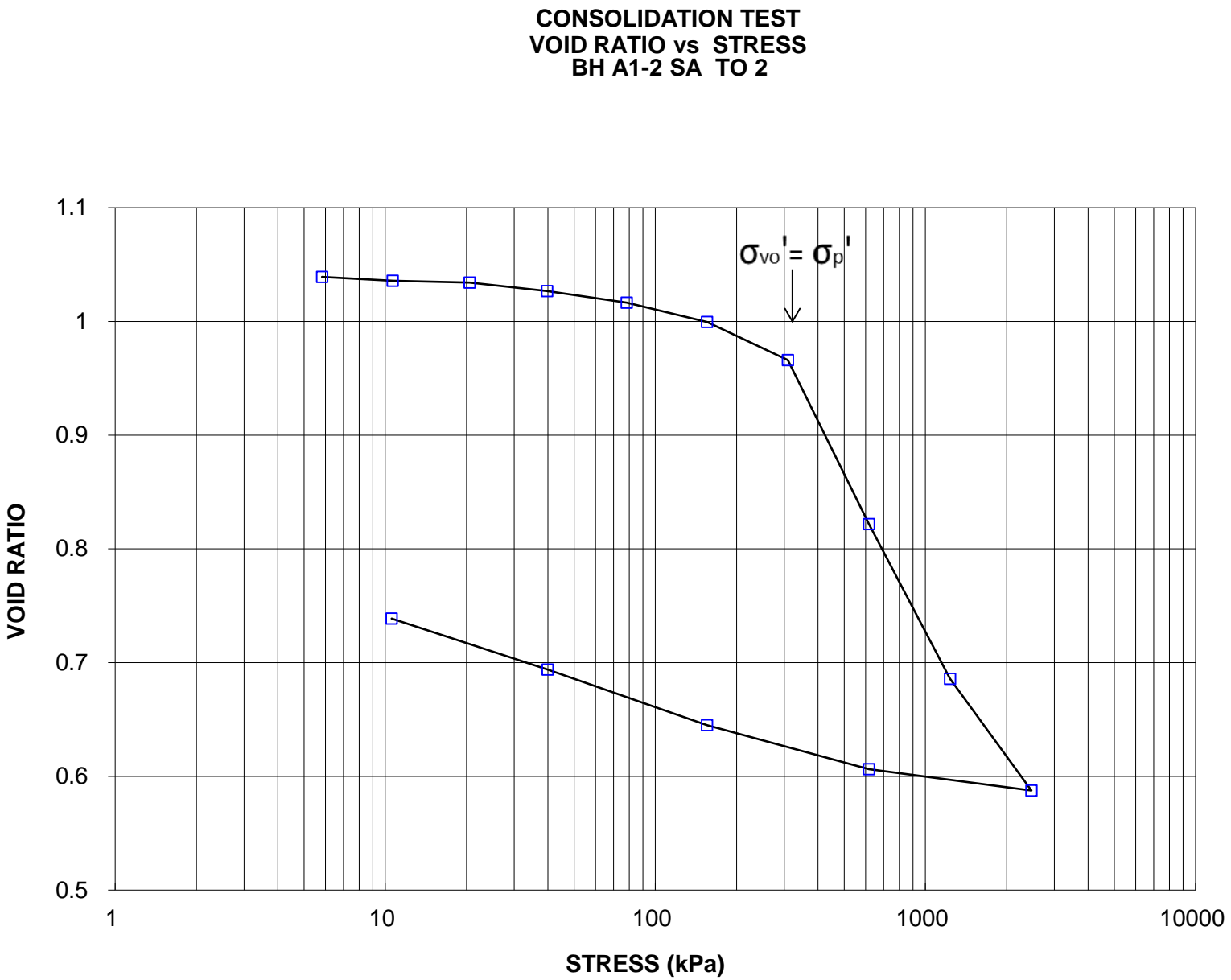


CONSOLIDATION TEST SUMMARY					FIGURE C22A			
ASTM D2435/D2435M					Lower Silty Clay to Clayey Silt			
SAMPLE IDENTIFICATION								
Project Number		1671430			Sample Number		TO 2	
Borehole Number		A1-2			Sample Depth, m		32.01-32.62	
TEST CONDITIONS								
Test Type		Laboratory Standard			Load Duration, hr		24	
Oedometer Number		10						
Date Started		04/02/2019						
Date Completed		04/15/2019						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		2.53			Unit Weight, kN/m <sup>3</sup>		18.17	
Sample Diameter, cm		6.36			Dry Unit Weight, kN/m <sup>3</sup>		13.17	
Area, cm <sup>2</sup>		31.75			Specific Gravity, measured		2.74	
Volume, cm <sup>3</sup>		80.42			Solids Height, cm		1.242	
Water Content, %		37.88			Volume of Solids, cm <sup>3</sup>		39.43	
Wet Mass, g		148.97			Volume of Voids, cm <sup>3</sup>		40.99	
Dry Mass, g		108.04			Degree of Saturation, %		99.9	
TEST COMPUTATIONS								
		Corr.		Average				
Stress		Height	Void	Height	t <sub>90</sub>	cv.	mv	k
kPa		cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00		2.533	1.040	2.533				
5.83		2.532	1.039	2.533				
10.64		2.528	1.036	2.530	205	6.62E-03	3.38E-04	2.19E-07
20.55		2.526	1.034	2.527	188	7.20E-03	7.89E-05	5.57E-08
39.71		2.517	1.027	2.522	290	4.65E-03	1.93E-04	8.80E-08
78.31		2.504	1.017	2.511	437	3.06E-03	1.29E-04	3.86E-08
155.48		2.483	0.999	2.494	427	3.09E-03	1.09E-04	3.29E-08
309.51		2.442	0.966	2.462	581	2.21E-03	1.06E-04	2.31E-08
618.02		2.263	0.822	2.352	2574	4.56E-04	2.29E-04	1.02E-08
1234.74		2.094	0.686	2.178	1500	6.70E-04	1.08E-04	7.11E-09
2467.70		1.972	0.588	2.033	844	1.04E-03	3.90E-05	3.97E-09
618.02		1.995	0.606	1.983				
155.39		2.043	0.645	2.019				
39.90		2.104	0.694	2.074				
10.55		2.159	0.739	2.132				
Note:								
Consolidation loading and unloading schedule assigned by the client.								
cv and k are approximate only based on t <sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M)								
Specimen taken 25-32 cm from top of the tube.								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		2.16			Unit Weight, kN/m <sup>3</sup>		19.93	
Sample Diameter, cm		6.36			Dry Unit Weight, kN/m <sup>3</sup>		15.45	
Area, cm <sup>2</sup>		31.75			Specific Gravity, measured		2.74	
Volume, cm <sup>3</sup>		68.56			Solids Height, cm		1.242	
Water Content, %		28.93			Volume of Solids, cm <sup>3</sup>		39.43	
Wet Mass, g		139.30			Volume of Voids, cm <sup>3</sup>		29.13	
Dry Mass, g		108.04						
Prepared By: LH					Golder Associates		Checked By:	





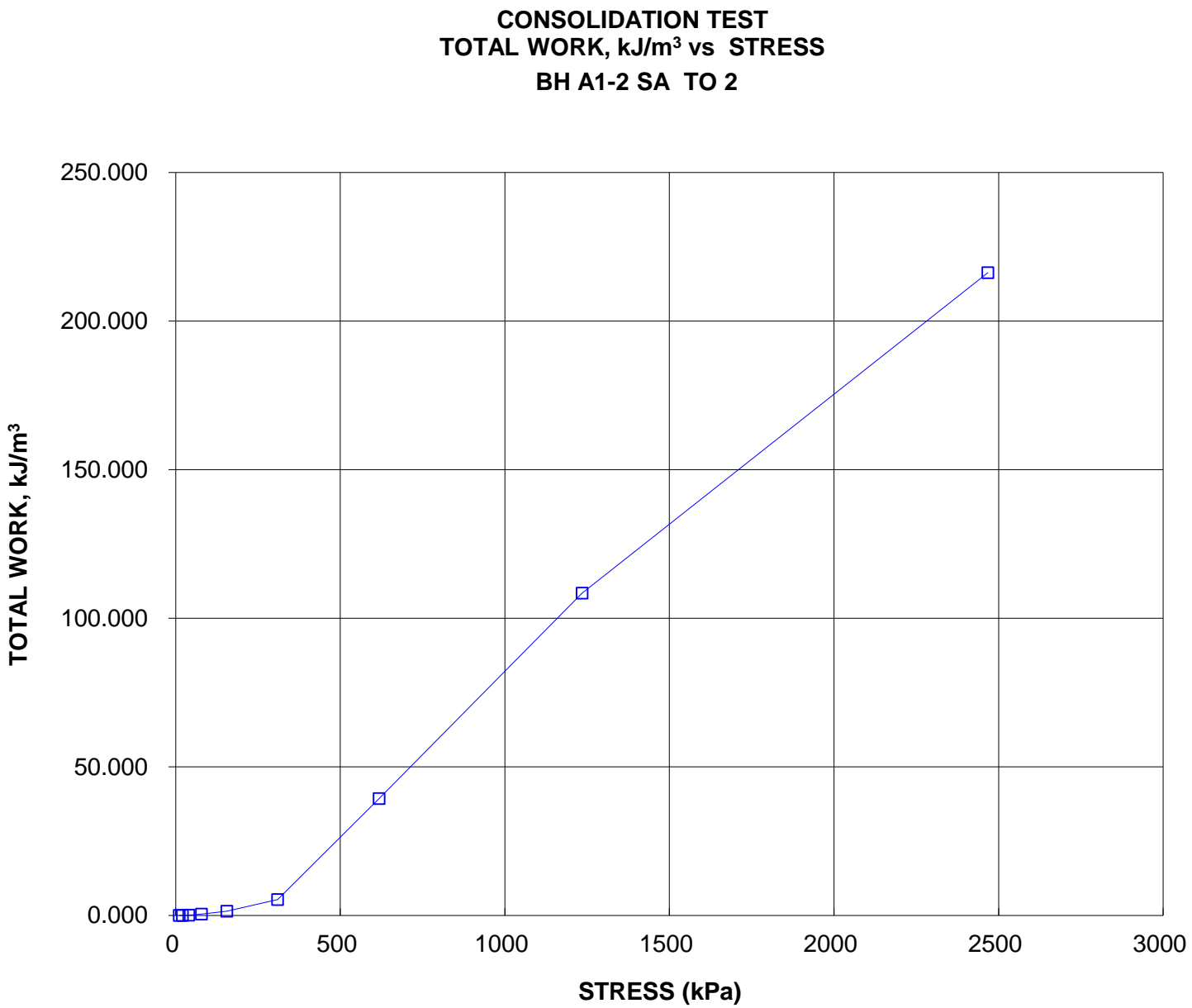






**CONSOLIDATION TEST  
TOTAL WORK VS STRESS**

**FIGURE C22D  
Lower Silty Clay to Clayey Silt**

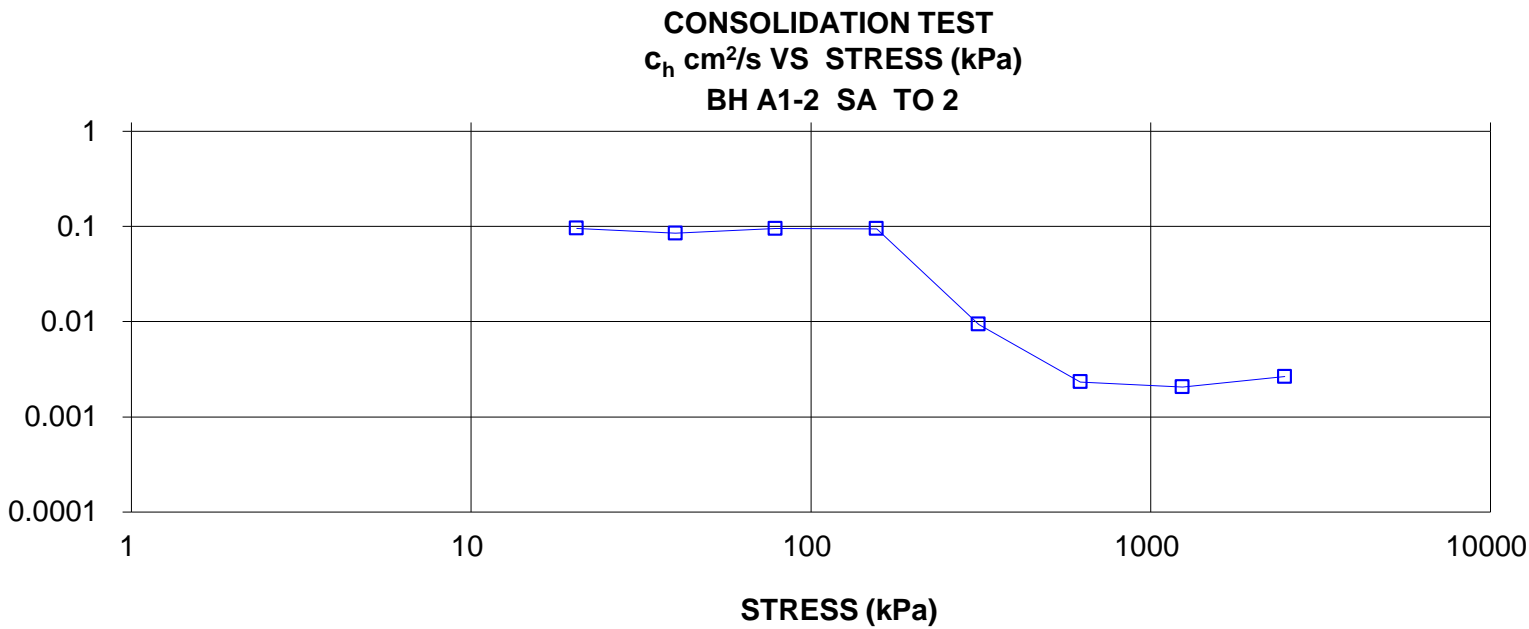




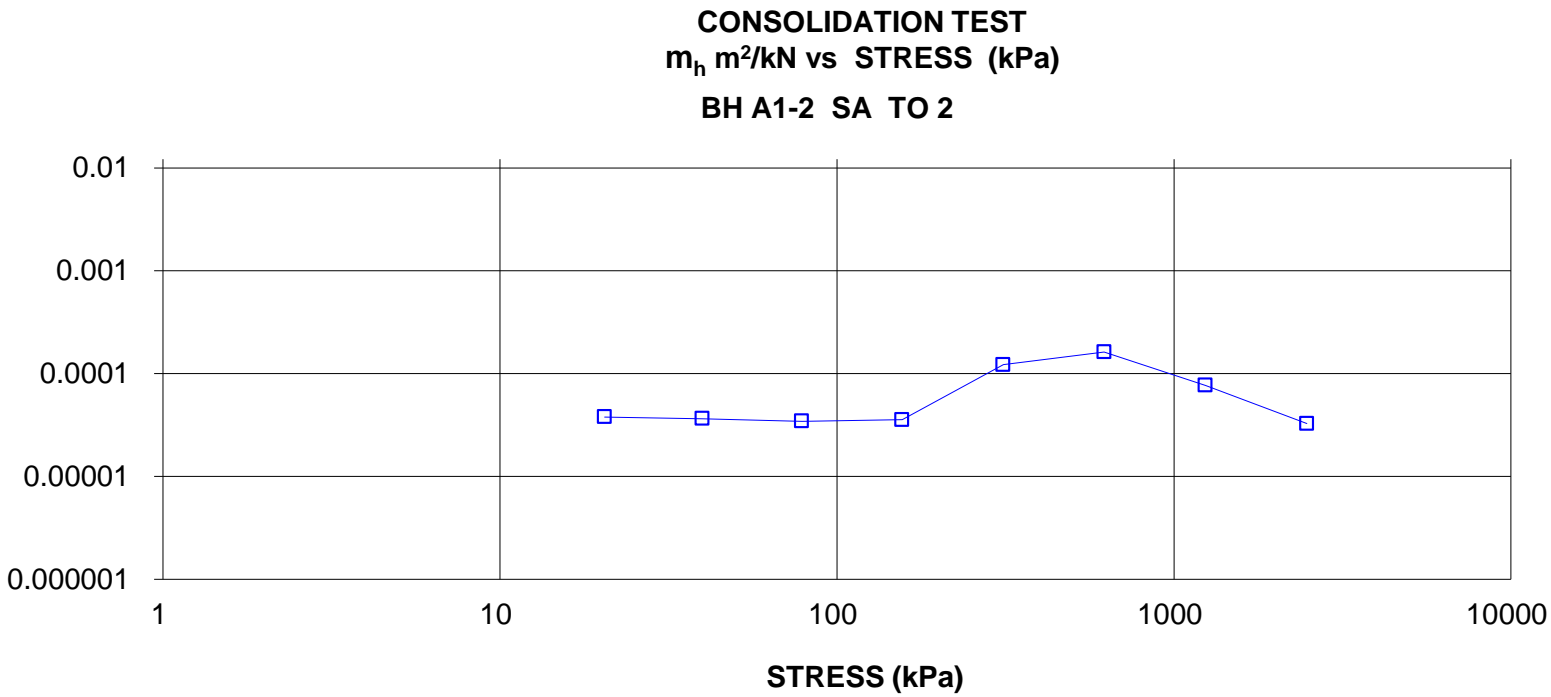
CONSOLIDATION TEST SUMMARY					FIGURE C23A		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number	1671430 WO 007			Sample Number	TO 2		
Borehole Number	A1-2			Sample Depth, m	32.01-32.62		
TEST CONDITIONS							
Test Type	QUICK / VTO			Load Duration, hr	-		
Oedometer Number	6						
Date Started	10-10-2019						
Date Completed	10-11-2019						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.89			Unit Weight, kN/m <sup>3</sup>	19.10		
Sample Diameter, cm	6.34			Dry Unit Weight, kN/m <sup>3</sup>	14.69		
Area, cm <sup>2</sup>	31.60			Specific Gravity, measured	2.73		
Volume, cm <sup>3</sup>	59.69			Solids Height, cm	1.037		
Water Content, %	29.96			Volume of Solids, cm <sup>3</sup>	32.76		
Wet Mass, g	116.24			Volume of Voids, cm <sup>3</sup>	26.93		
Dry Mass, g	89.44			Degree of Saturation, %	99.5		
TEST COMPUTATIONS							
	Corr.	Average					
Stress	Height	Void	Height	t <sub>90</sub>	C <sub>h</sub>	m <sub>h</sub>	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	1.889	0.822	1.889				
6.01	1.893	0.826	1.891				
10.67	1.898	0.830	1.895				
20.45	1.897	0.830	1.897	8	9.54E-02	3.79E-05	3.54E-07
39.92	1.896	0.828	1.896	9	8.47E-02	3.65E-05	3.03E-07
78.70	1.893	0.826	1.894	8	9.51E-02	3.45E-05	3.21E-07
156.17	1.888	0.821	1.890	8	9.47E-02	3.57E-05	3.31E-07
311.78	1.852	0.786	1.870	79	9.38E-03	1.22E-04	1.13E-07
621.16	1.757	0.695	1.804	296	2.33E-03	1.62E-04	3.71E-08
1240.70	1.667	0.608	1.712	302	2.06E-03	7.67E-05	1.55E-08
2480.15	1.590	0.534	1.629	212	2.65E-03	3.27E-05	8.51E-09
621.16	1.600	0.543	1.595				
156.17	1.621	0.563	1.610				
39.86	1.648	0.589	1.634				
10.72	1.680	0.620	1.664				
Note:							
Consolidation loading and unloading schedule assigned by the client.							
k calculated using c <sub>h</sub> based on t <sub>90</sub> values.							
Testing carried out on VTO (Vertically Trimmed Orientation) specimens in order to evaluate the horizontal properties.							
Specimen swelled under 10.67kPa.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.68			Unit Weight, kN/m <sup>3</sup>	22.87		
Sample Diameter, cm	6.34			Dry Unit Weight, kN/m <sup>3</sup>	16.53		
Area, cm <sup>2</sup>	31.60			Specific Gravity, measured	2.73		
Volume, cm <sup>3</sup>	53.07			Solids Height, cm	1.037		
Water Content, %	38.39			Volume of Solids, cm <sup>3</sup>	32.76		
Wet Mass, g	123.78			Volume of Voids, cm <sup>3</sup>	20.31		
Dry Mass, g	89.44						
Prepared By: LH		Golder Associates			Checked By:		



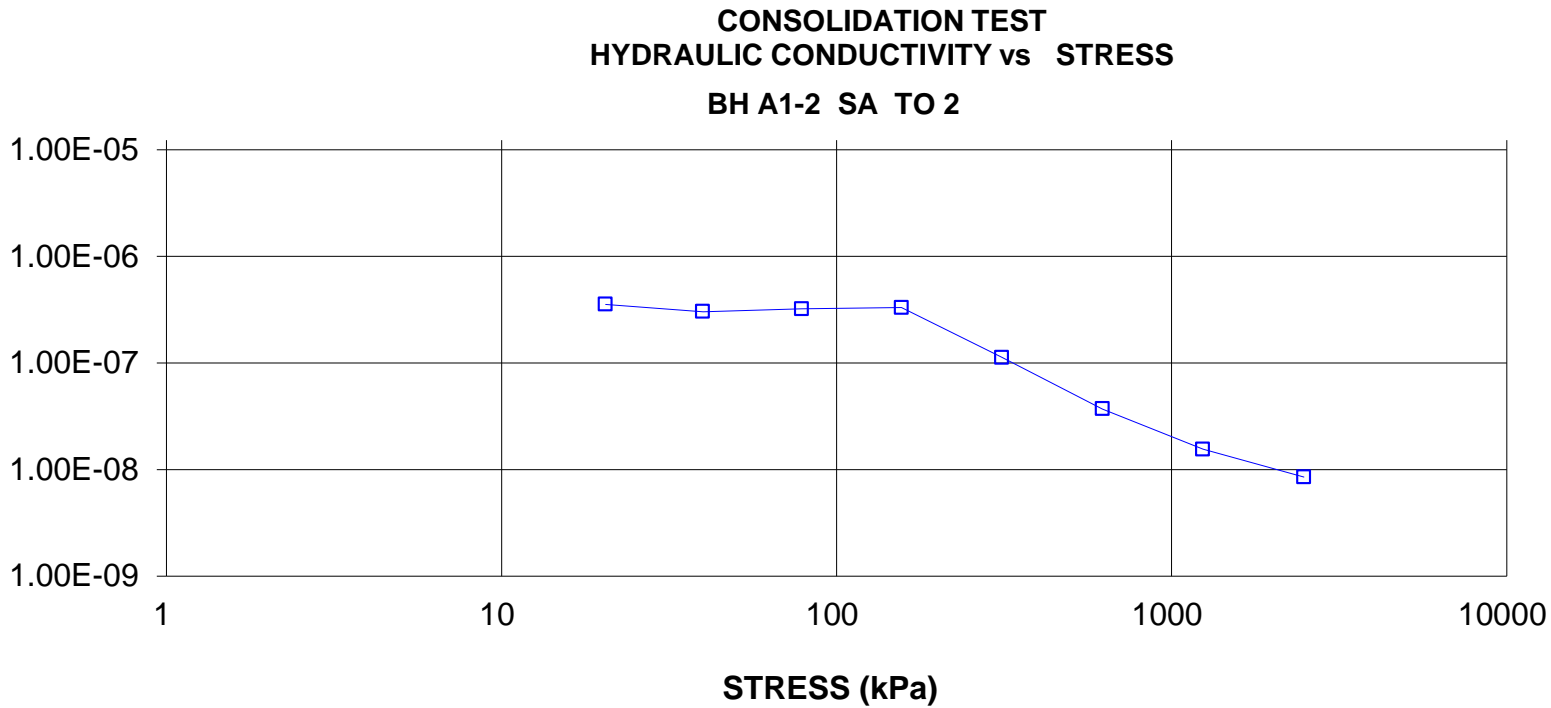
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



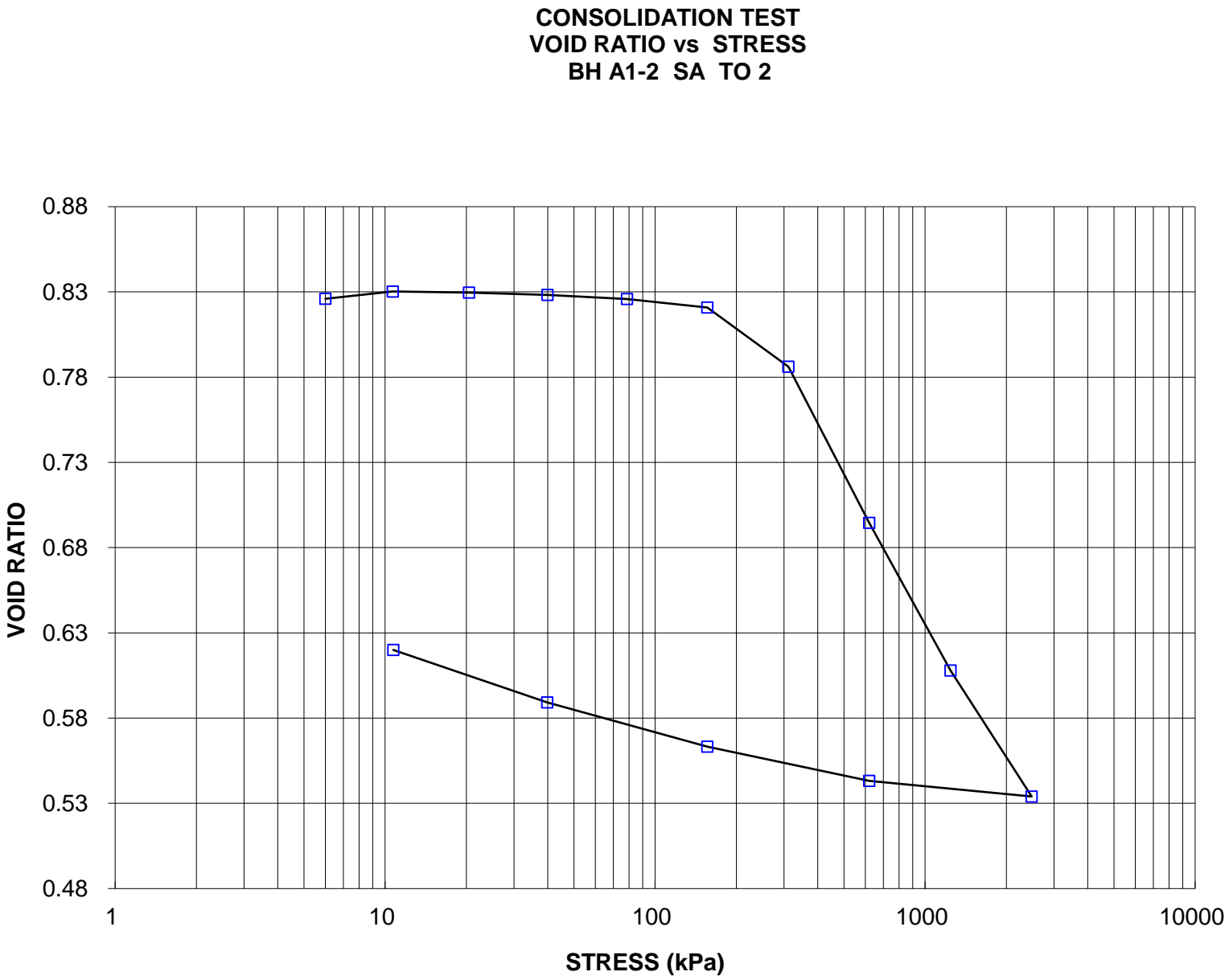
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



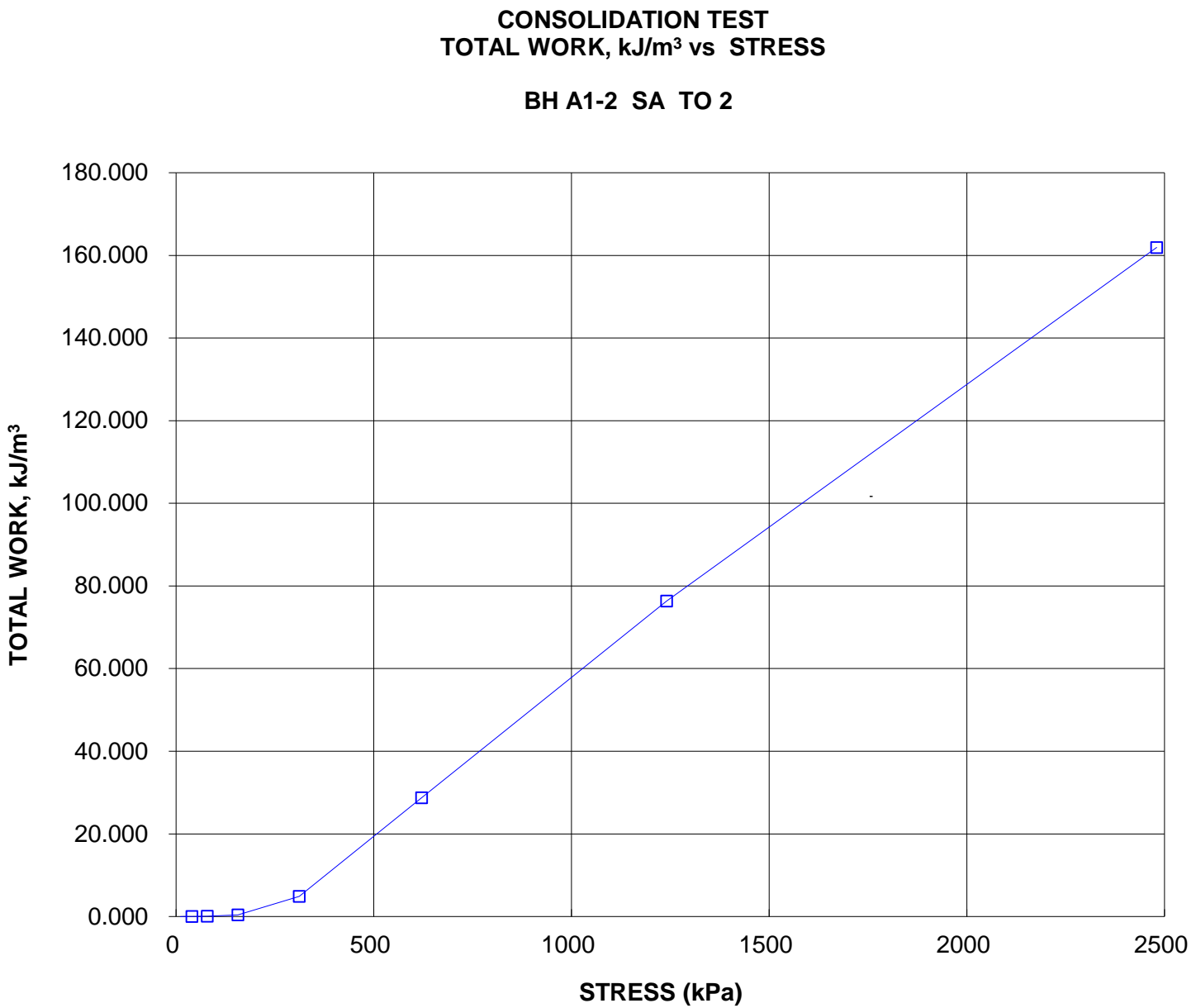
HYDRAULIC CONDUCTIVITY, cm/s









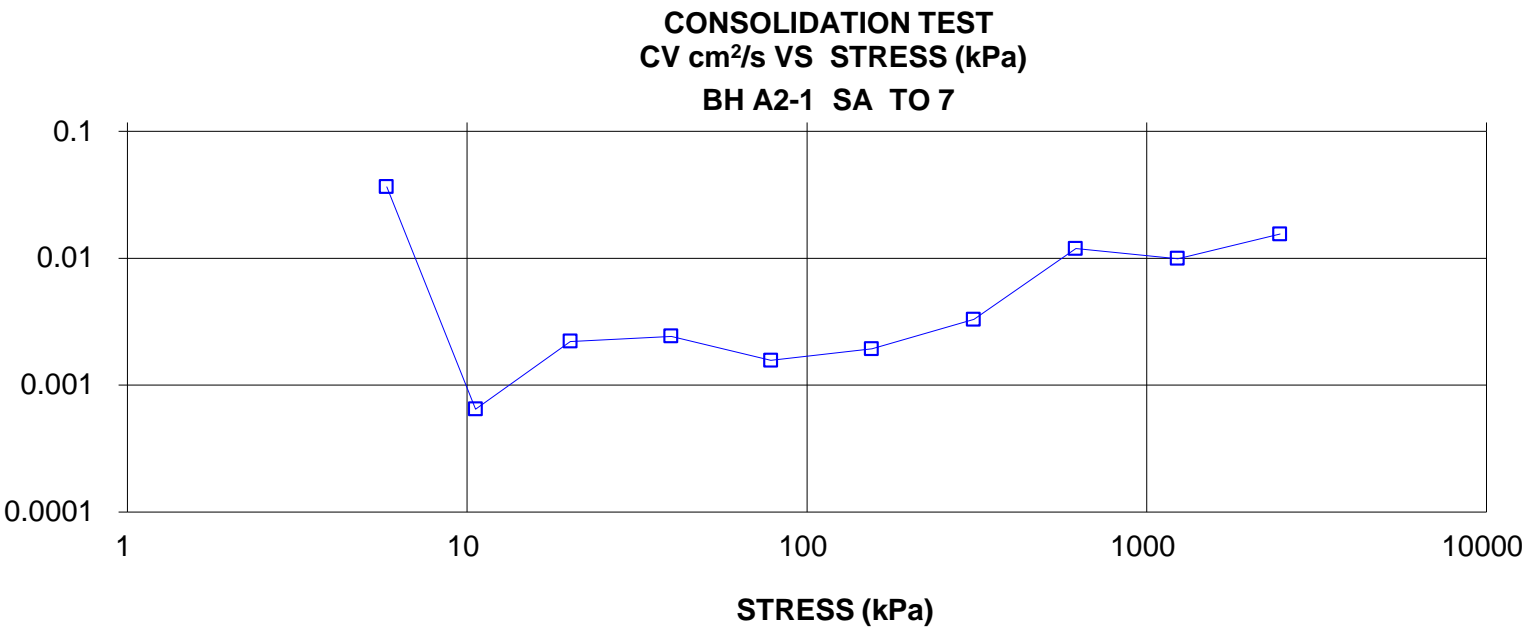




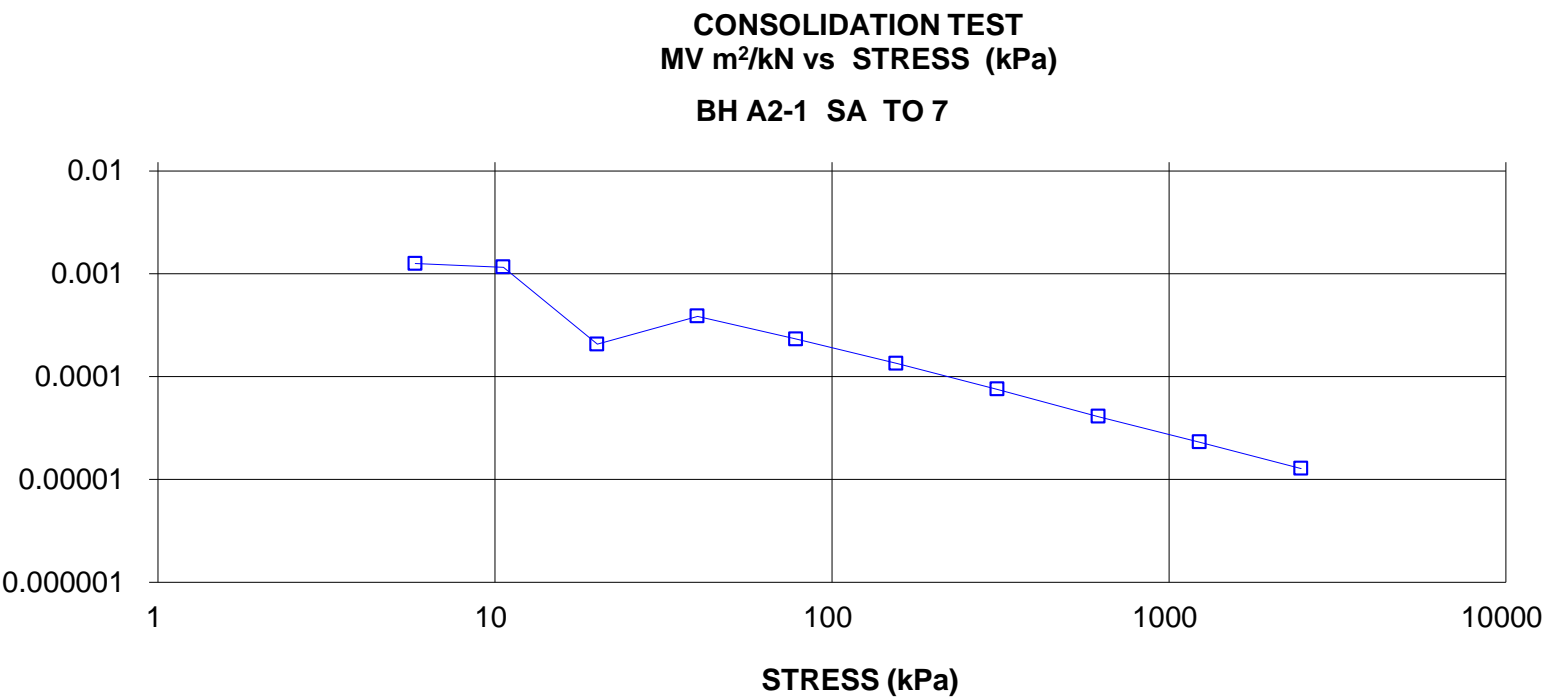
CONSOLIDATION TEST SUMMARY					FIGURE C24A		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number	1671430 WO 007				Sample Number	TO 7	
Borehole Number	A2-1				Sample Depth, m	41.15-41.78	
TEST CONDITIONS							
Test Type	QUICK				Load Duration, hr	-	
Oedometer Number	10						
Date Started	8-14-2019						
Date Completed	8-16-2019						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	2.53				Unit Weight, kN/m <sup>3</sup>	20.68	
Sample Diameter, cm	6.36				Dry Unit Weight, kN/m <sup>3</sup>	17.20	
Area, cm <sup>2</sup>	31.75				Specific Gravity, measured	2.72	
Volume, cm <sup>3</sup>	80.42				Solids Height, cm	1.634	
Water Content, %	20.20				Volume of Solids, cm <sup>3</sup>	51.87	
Wet Mass, g	169.58				Volume of Voids, cm <sup>3</sup>	28.55	
Dry Mass, g	141.08				Degree of Saturation, %	99.8	
TEST COMPUTATIONS							
	Corr.	Average					
Stress	Height	Void	Height	t <sub>90</sub>	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	2.533	0.550	2.533				
5.80	2.514	0.539	2.524	37	3.65E-02	1.26E-03	4.51E-06
10.60	2.500	0.531	2.507	2062	6.46E-04	1.16E-03	7.36E-08
20.12	2.495	0.527	2.498	599	2.21E-03	2.07E-04	4.47E-08
39.90	2.476	0.516	2.486	540	2.43E-03	3.87E-04	9.19E-08
78.31	2.453	0.502	2.465	821	1.57E-03	2.32E-04	3.57E-08
155.31	2.427	0.486	2.440	653	1.93E-03	1.35E-04	2.55E-08
309.59	2.398	0.468	2.412	375	3.29E-03	7.56E-05	2.44E-08
617.99	2.366	0.448	2.382	101	1.19E-02	4.10E-05	4.78E-08
1234.33	2.330	0.426	2.348	118	9.90E-03	2.31E-05	2.24E-08
2468.34	2.290	0.402	2.310	73	1.55E-02	1.28E-05	1.94E-08
618.30	2.293	0.404	2.291				
155.31	2.300	0.408	2.297				
39.70	2.312	0.415	2.306				
5.96	2.334	0.428	2.323				
Note:							
Consolidation loading and unloading schedule assigned by the client.							
cv and k are approximate only based on t <sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M)							
Specimen taken 12-20cm from top of the tube.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	2.33				Unit Weight, kN/m <sup>3</sup>	21.84	
Sample Diameter, cm	6.36				Dry Unit Weight, kN/m <sup>3</sup>	18.67	
Area, cm <sup>2</sup>	31.75				Specific Gravity, measured	2.72	
Volume, cm <sup>3</sup>	74.09				Solids Height, cm	1.634	
Water Content, %	16.95				Volume of Solids, cm <sup>3</sup>	51.87	
Wet Mass, g	164.99				Volume of Voids, cm <sup>3</sup>	22.22	
Dry Mass, g	141.08						
Prepared By: SJ		Golder Associates				Checked By:	



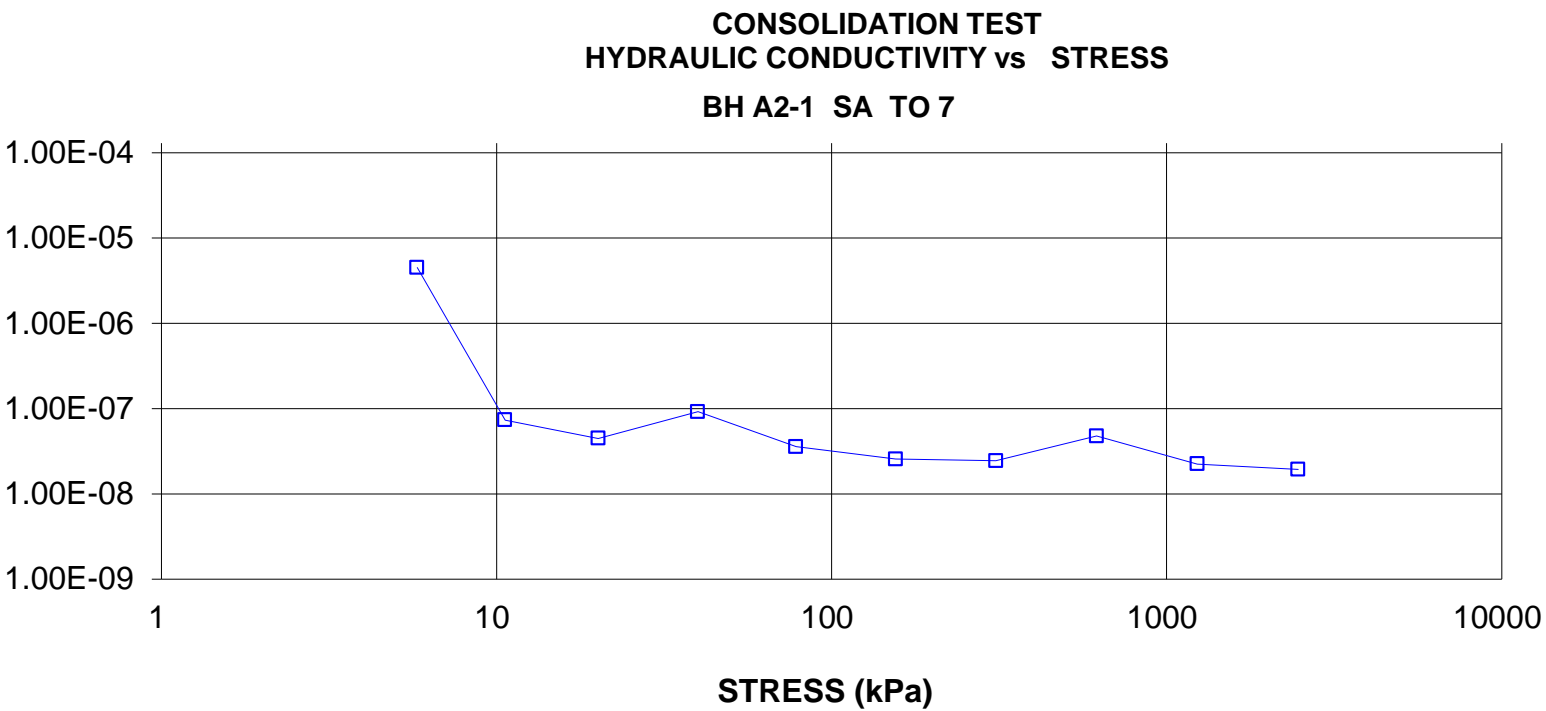
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



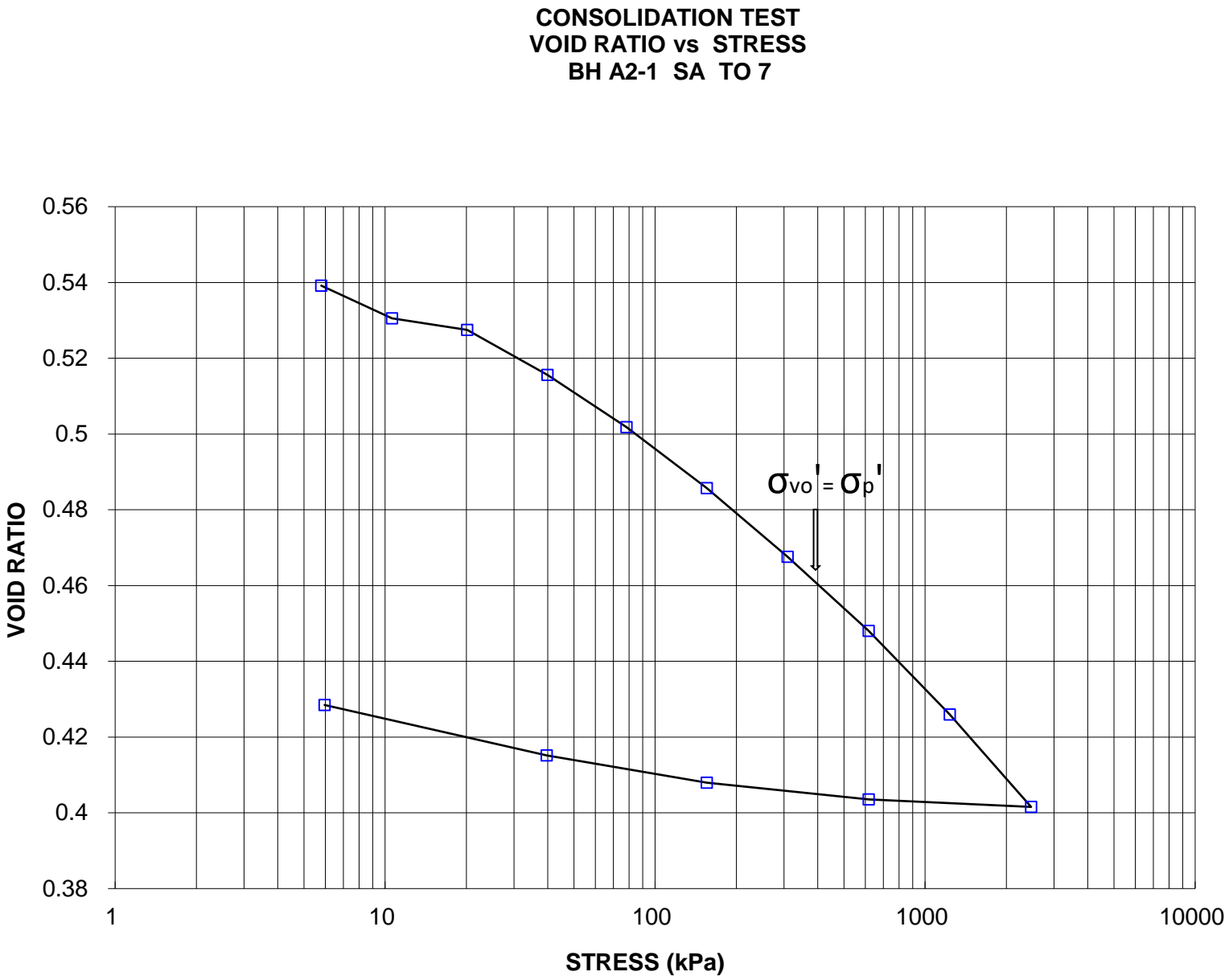
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



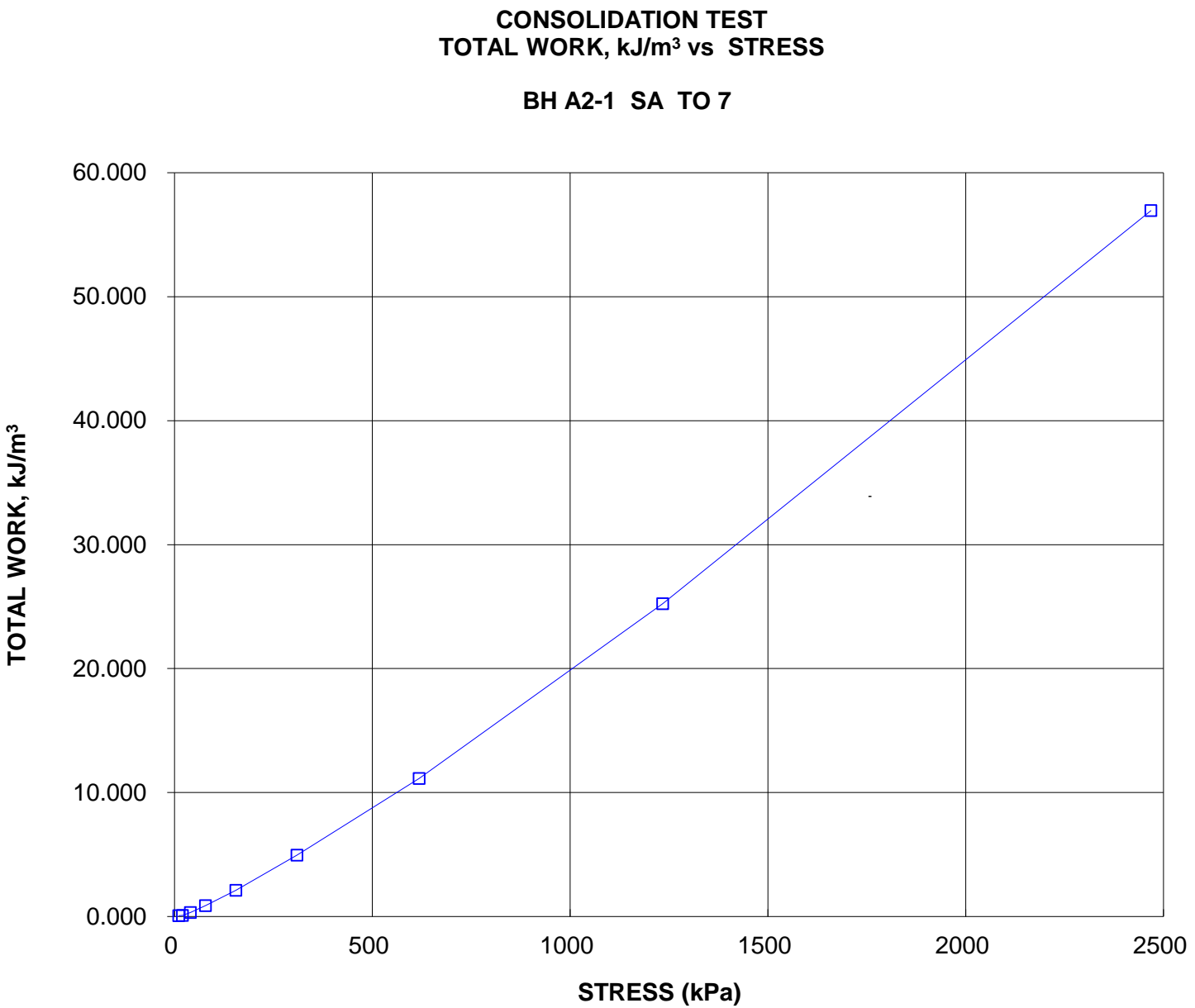
HYDRAULIC CONDUCTIVITY, cm/s









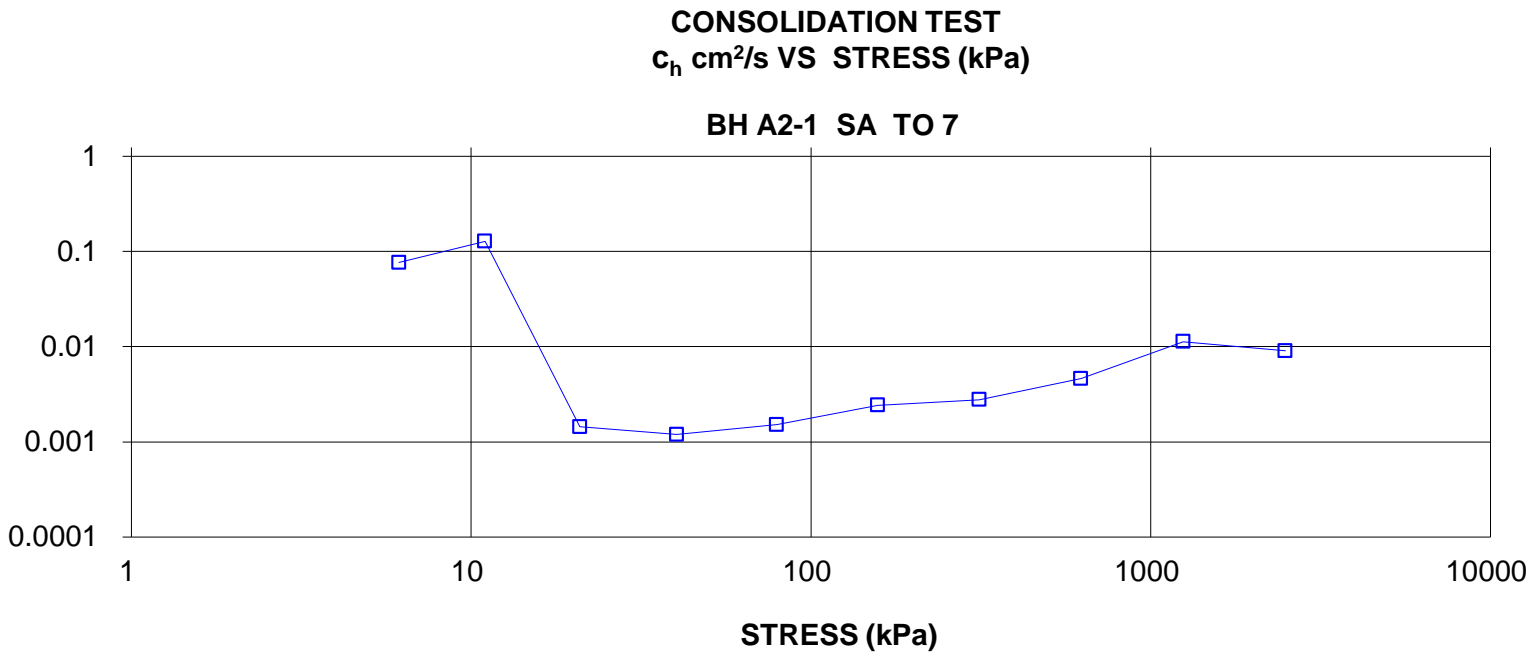




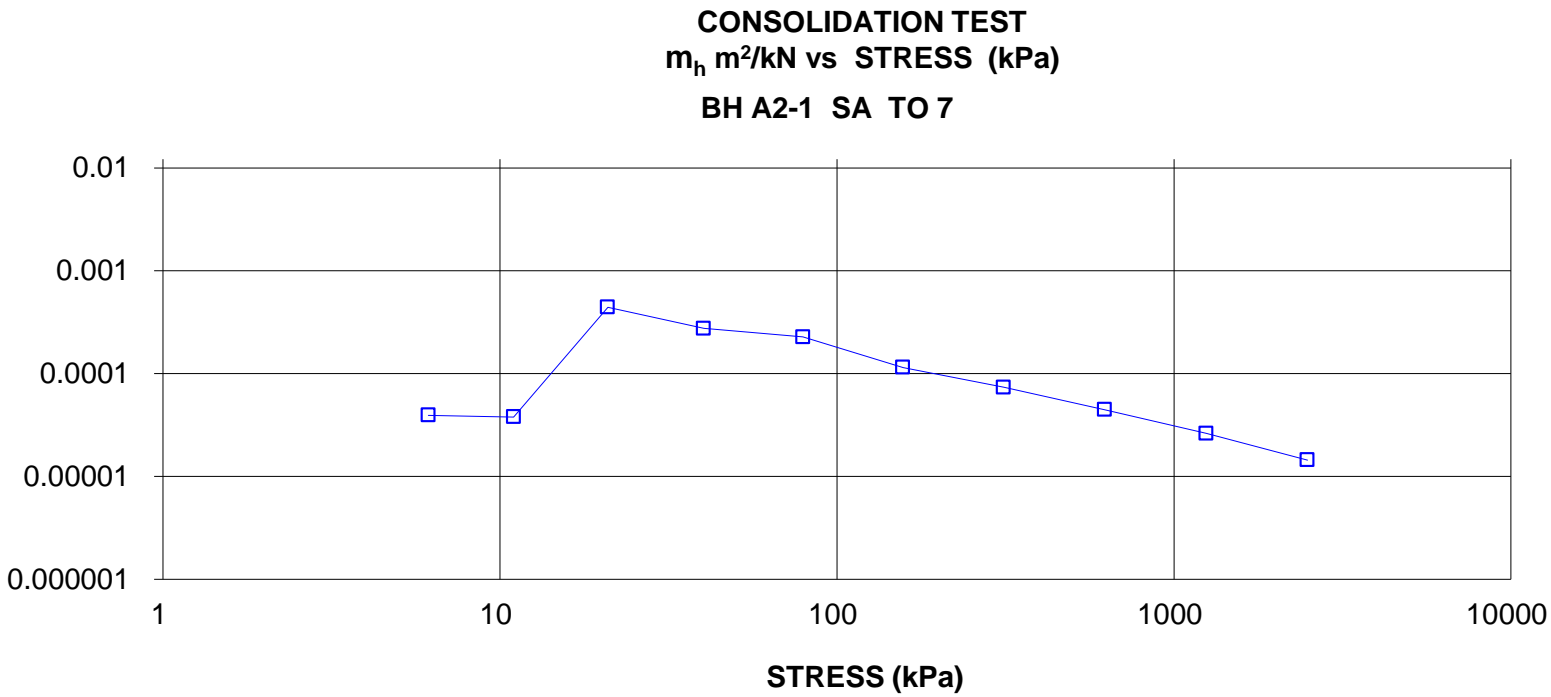
CONSOLIDATION TEST SUMMARY					FIGURE C25A		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number	1671430 WO 007			Sample Number	TO 7		
Borehole Number	A2-1			Sample Depth, m	41.15-41.78		
TEST CONDITIONS							
Test Type	QUICK / VTO			Load Duration, hr	-		
Oedometer Number	5						
Date Started	10-10-2019						
Date Completed	10-11-2019						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.90			Unit Weight, kN/m <sup>3</sup>	20.87		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m <sup>3</sup>	17.44		
Area, cm <sup>2</sup>	31.46			Specific Gravity, measured	2.74		
Volume, cm <sup>3</sup>	59.84			Solids Height, cm	1.234		
Water Content, %	19.70			Volume of Solids, cm <sup>3</sup>	38.83		
Wet Mass, g	127.35			Volume of Voids, cm <sup>3</sup>	21.01		
Dry Mass, g	106.39			Degree of Saturation, %	99.8		
TEST COMPUTATIONS							
	Corr.	Average					
Stress	Height	Void	Height	t <sub>90</sub>	C <sub>h</sub>	m <sub>h</sub>	k
kPa	cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	1.902	0.541	1.902				
6.13	1.902	0.541	1.902	10	7.67E-02	3.95E-05	2.96E-07
10.99	1.901	0.540	1.901	6	1.28E-01	3.79E-05	4.74E-07
20.91	1.893	0.534	1.897	531	1.44E-03	4.42E-04	6.22E-08
40.26	1.883	0.525	1.888	631	1.20E-03	2.75E-04	3.23E-08
79.31	1.866	0.512	1.874	492	1.51E-03	2.28E-04	3.38E-08
157.15	1.849	0.498	1.857	302	2.42E-03	1.15E-04	2.73E-08
312.86	1.827	0.480	1.838	258	2.78E-03	7.36E-05	2.00E-08
624.10	1.801	0.459	1.814	151	4.62E-03	4.45E-05	2.02E-08
1246.67	1.770	0.434	1.785	60	1.13E-02	2.63E-05	2.90E-08
2491.79	1.735	0.406	1.752	72	9.04E-03	1.44E-05	1.28E-08
624.10	1.740	0.410	1.738				
157.15	1.750	0.418	1.745				
40.51	1.763	0.429	1.757				
10.94	1.773	0.437	1.768				
Note: Consolidation loading and unloading schedule assigned by the client. k calculated using c <sub>h</sub> based on t <sub>90</sub> values. Testing carried out on VTO (Vertically Trimmed Orientation) specimens in order to evaluate the horizontal properties.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.77			Unit Weight, kN/m <sup>3</sup>	21.76		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m <sup>3</sup>	18.70		
Area, cm <sup>2</sup>	31.46			Specific Gravity, measured	2.74		
Volume, cm <sup>3</sup>	55.79			Solids Height, cm	1.234		
Water Content, %	16.35			Volume of Solids, cm <sup>3</sup>	38.83		
Wet Mass, g	123.78			Volume of Voids, cm <sup>3</sup>	16.97		
Dry Mass, g	106.39						
Prepared By: LH		Golder Associates			Checked By:		



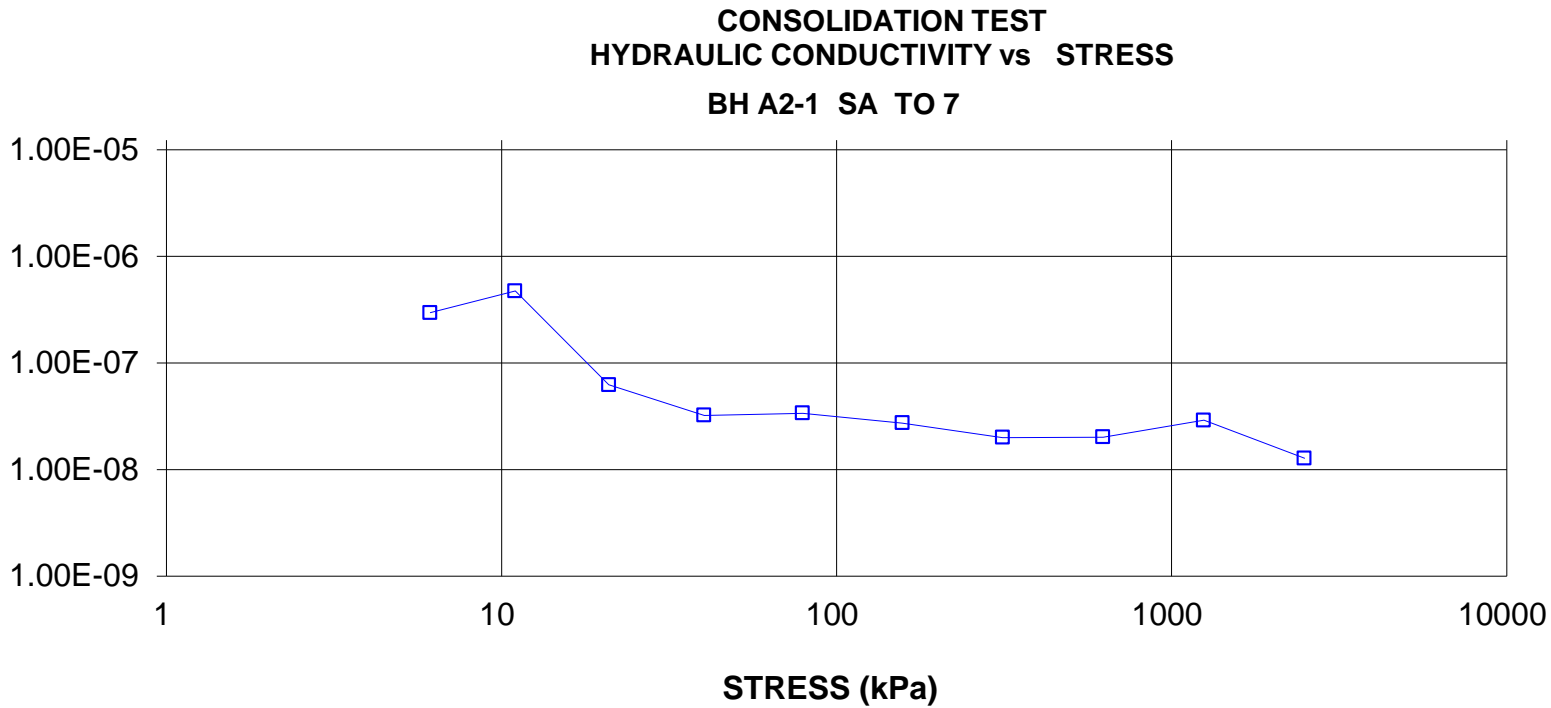
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



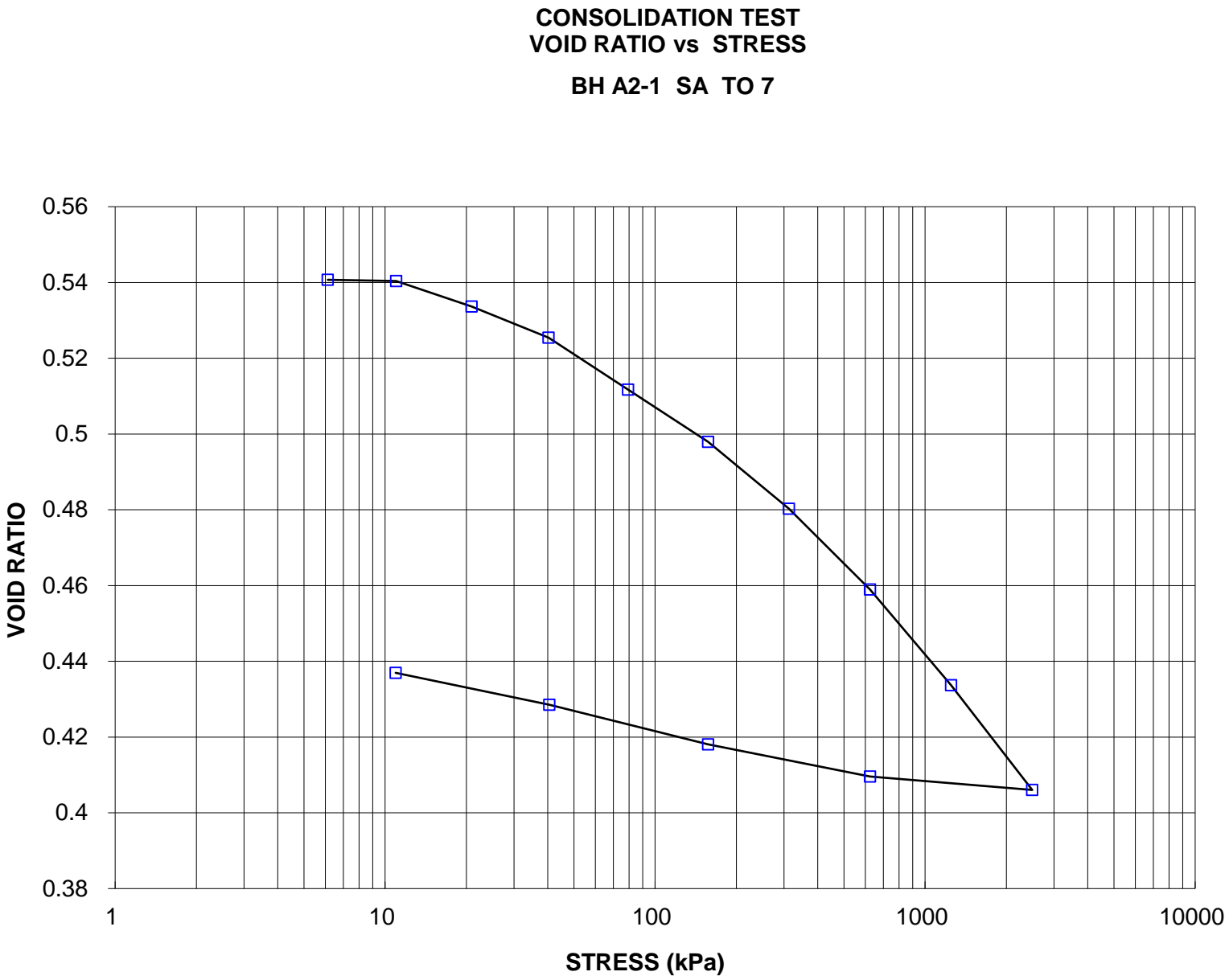
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



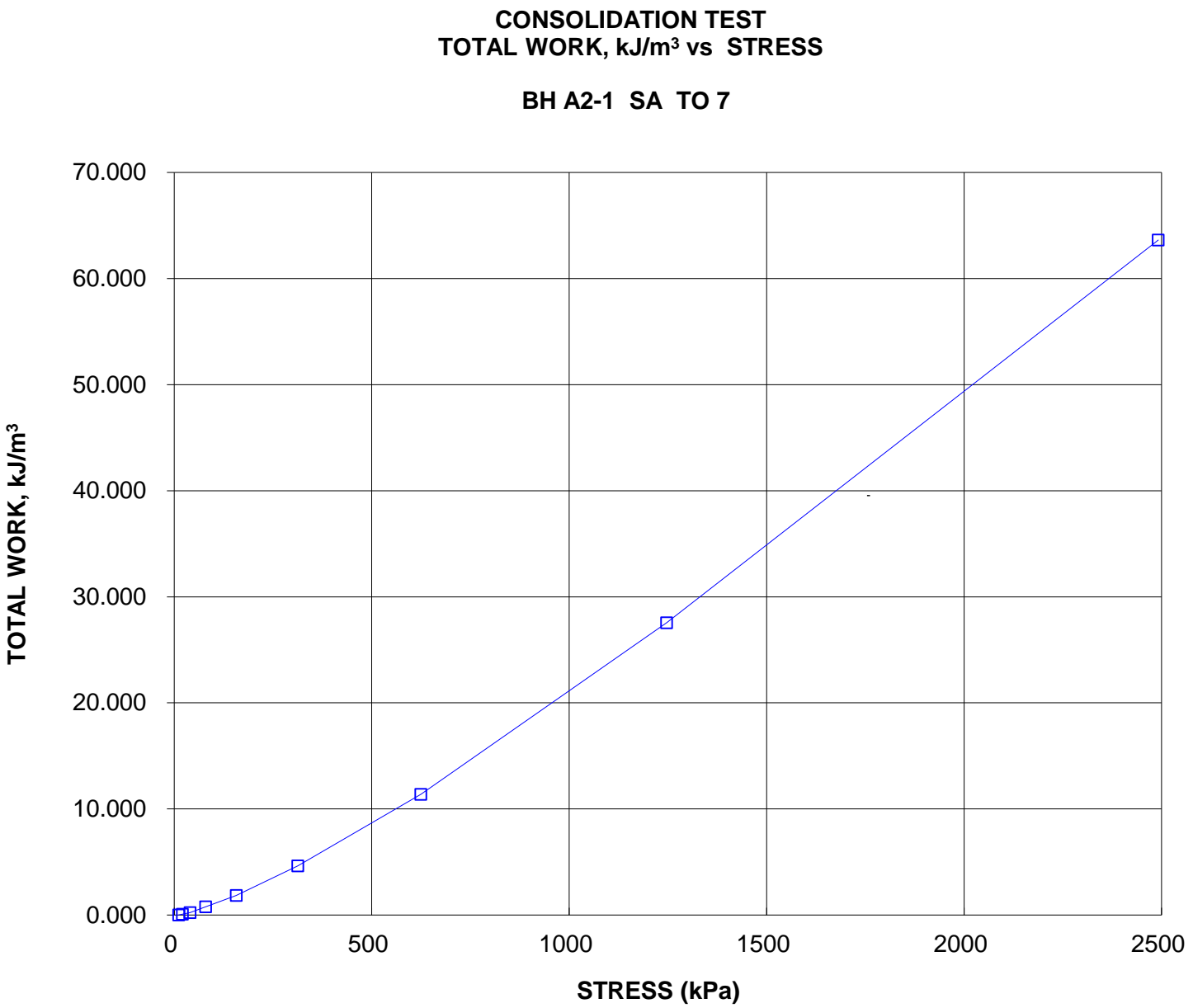
HYDRAULIC CONDUCTIVITY, cm/s







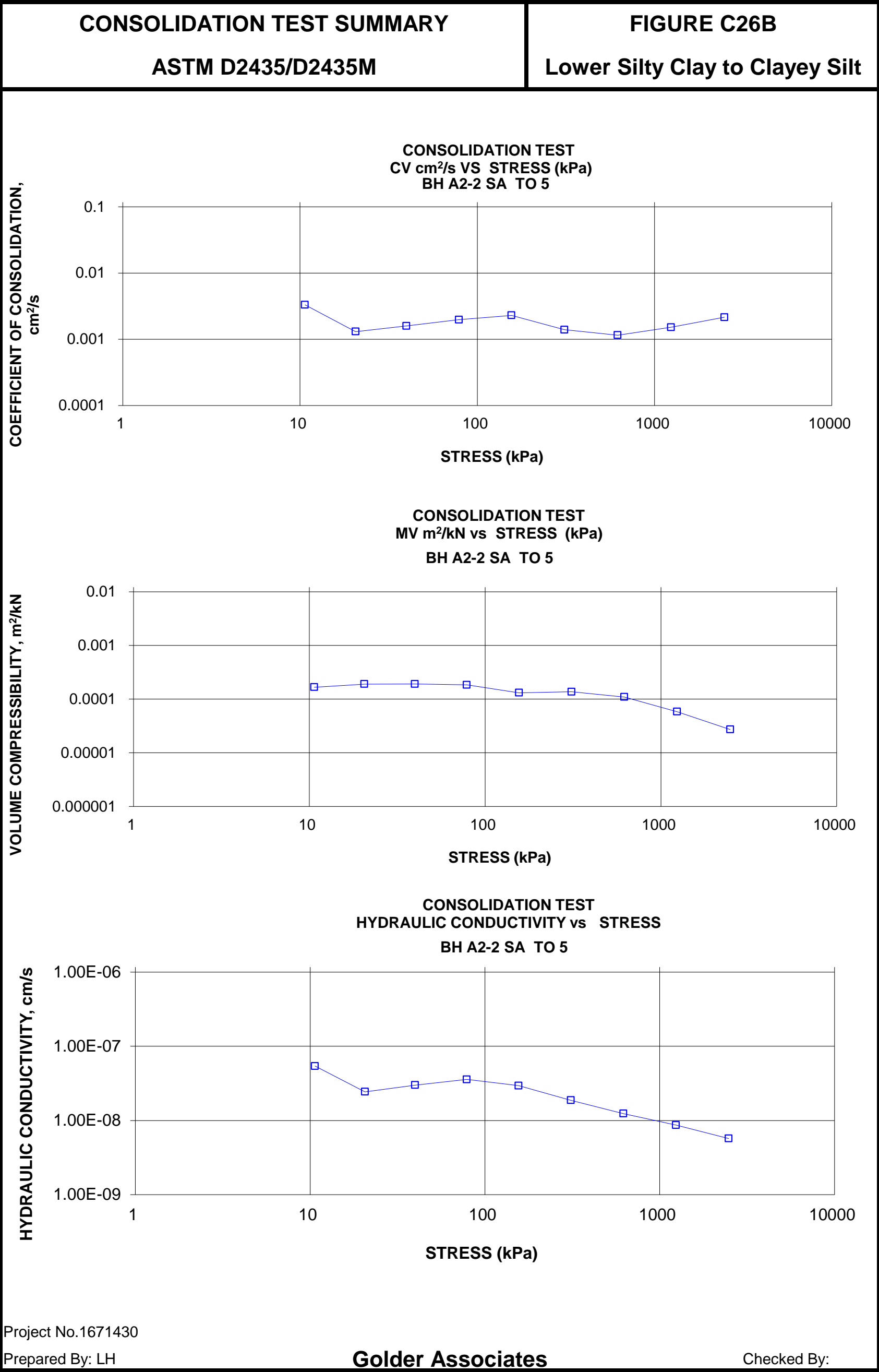




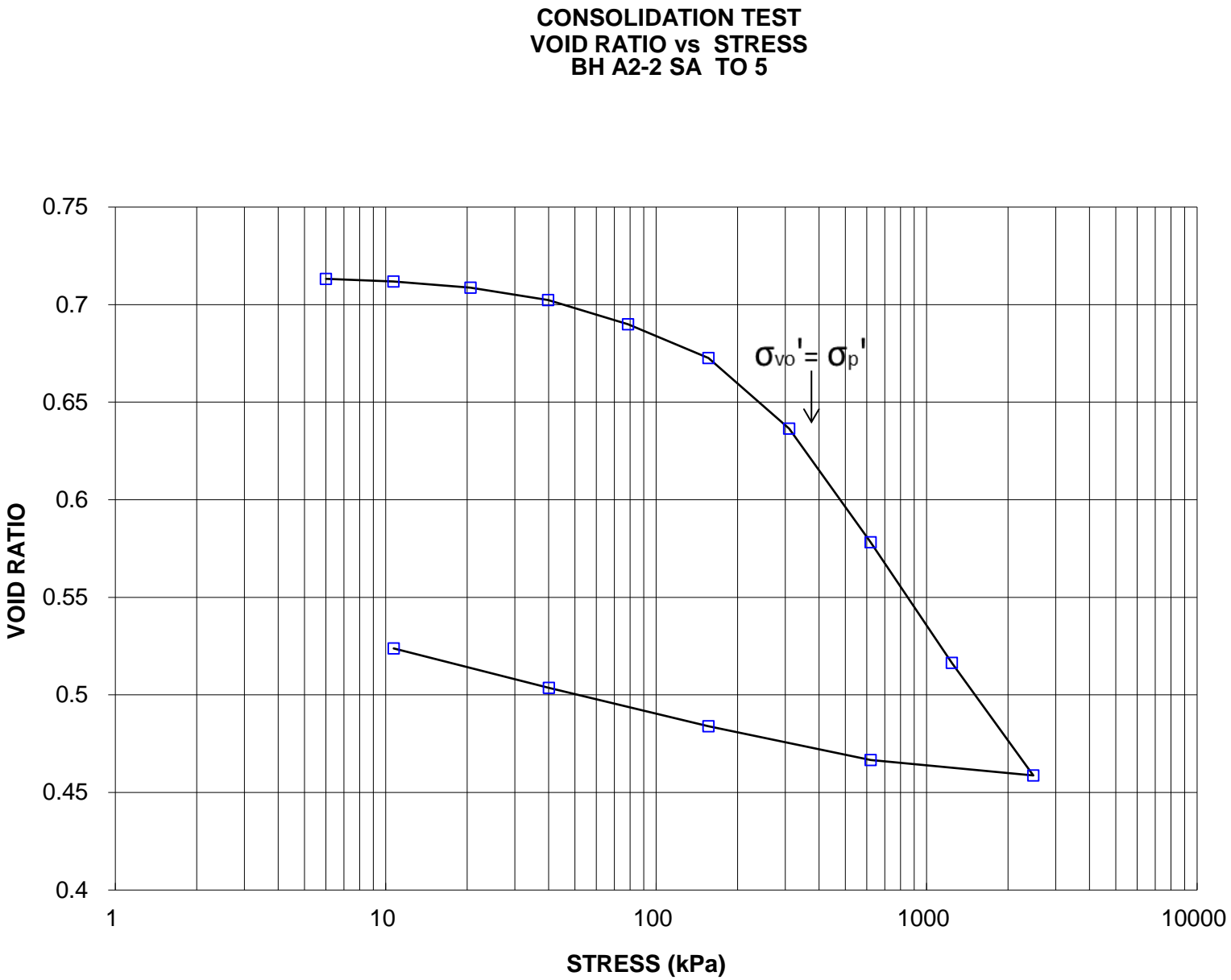


CONSOLIDATION TEST SUMMARY					FIGURE C26A			
ASTM D2435/D2435M					Lower Silty Clay to Clayey Silt			
SAMPLE IDENTIFICATION								
Project Number		1671430			Sample Number		TO 5	
Borehole Number		A2-2			Sample Depth, m		39.76-40.24	
TEST CONDITIONS								
Test Type		Laboratory Standard			Load Duration, hr		24	
Oedometer Number		6						
Date Started		04/09/2019						
Date Completed		04/23/2019						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		1.89			Unit Weight, kN/m <sup>3</sup>		19.64	
Sample Diameter, cm		6.34			Dry Unit Weight, kN/m <sup>3</sup>		15.56	
Area, cm <sup>2</sup>		31.60			Specific Gravity, measured		2.72	
Volume, cm <sup>3</sup>		59.69			Solids Height, cm		1.102	
Water Content, %		26.23			Volume of Solids, cm <sup>3</sup>		34.82	
Wet Mass, g		119.55			Volume of Voids, cm <sup>3</sup>		24.87	
Dry Mass, g		94.71			Degree of Saturation, %		99.9	
TEST COMPUTATIONS								
	Stress	Corr. Height	Void Ratio	Average Height	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
	kPa	cm		cm				
	0.00	1.889	0.714	1.889				
	6.01	1.888	0.713	1.888				
	10.67	1.886	0.712	1.887	228	3.31E-03	1.67E-04	5.42E-08
	20.59	1.883	0.709	1.885	577	1.30E-03	1.91E-04	2.44E-08
	39.86	1.876	0.702	1.879	470	1.59E-03	1.92E-04	2.99E-08
	78.74	1.862	0.690	1.869	375	1.97E-03	1.85E-04	3.58E-08
	156.07	1.843	0.673	1.853	317	2.30E-03	1.31E-04	2.95E-08
	310.38	1.803	0.636	1.823	505	1.40E-03	1.37E-04	1.87E-08
	620.24	1.739	0.578	1.771	577	1.15E-03	1.09E-04	1.24E-08
	1238.72	1.671	0.516	1.705	406	1.52E-03	5.83E-05	8.68E-09
	2479.70	1.607	0.459	1.639	265	2.15E-03	2.71E-05	5.71E-09
	620.17	1.616	0.467	1.612				
	156.07	1.635	0.484	1.626				
	40.11	1.657	0.504	1.646				
	10.70	1.679	0.524	1.668				
Note:								
Consolidation loading and unloading schedule assigned by the client.								
cv and k are approximate only based on t <sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M)								
Specimen taken 10-18 cm from top of the tube.								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		1.68			Unit Weight, kN/m <sup>3</sup>		21.20	
Sample Diameter, cm		6.34			Dry Unit Weight, kN/m <sup>3</sup>		17.50	
Area, cm <sup>2</sup>		31.60			Specific Gravity, measured		2.72	
Volume, cm <sup>3</sup>		53.06			Solids Height, cm		1.102	
Water Content, %		21.14			Volume of Solids, cm <sup>3</sup>		34.82	
Wet Mass, g		114.73			Volume of Voids, cm <sup>3</sup>		18.24	
Dry Mass, g		94.71						
Prepared By: LH					Golder Associates		Checked By:	





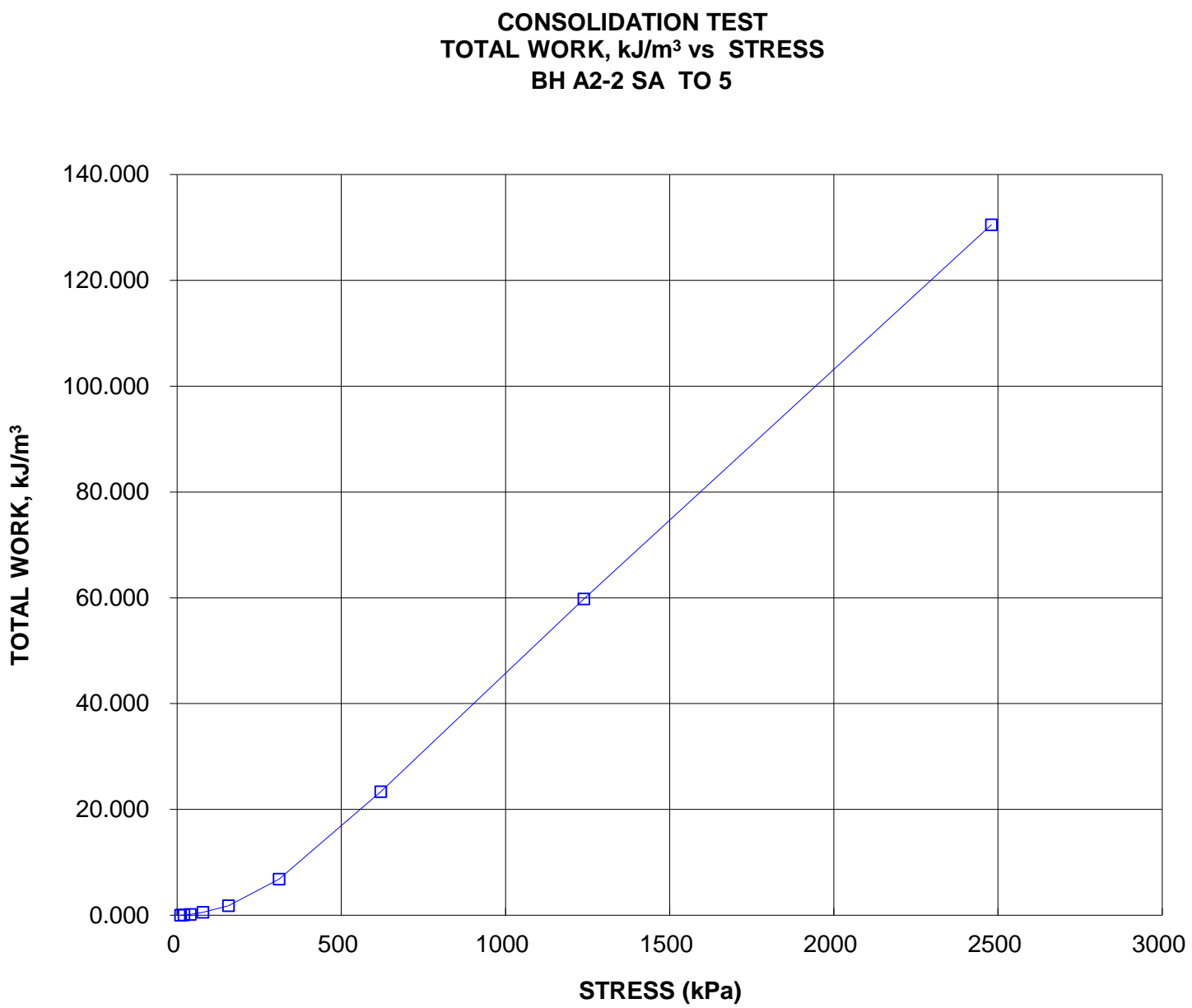






CONSOLIDATION TEST  
TOTAL WORK VS STRESS

FIGURE C26D  
Lower Silty Clay to Clayey Silt



Project No. 1671430  
Prepared By: LH

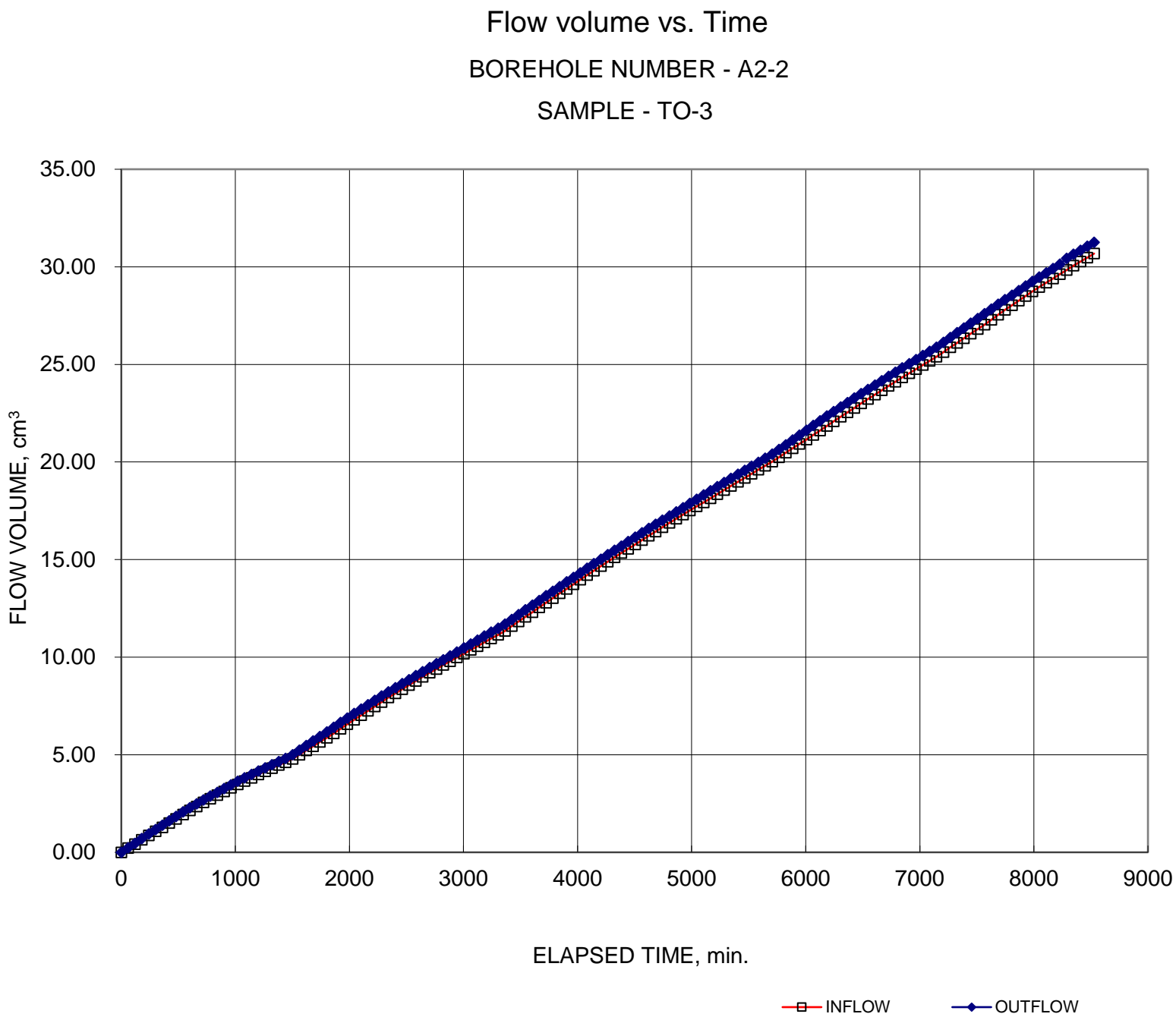
Golder Associates

Checked By:



# HYDRAULIC CONDUCTIVITY TEST

FIGURE C27



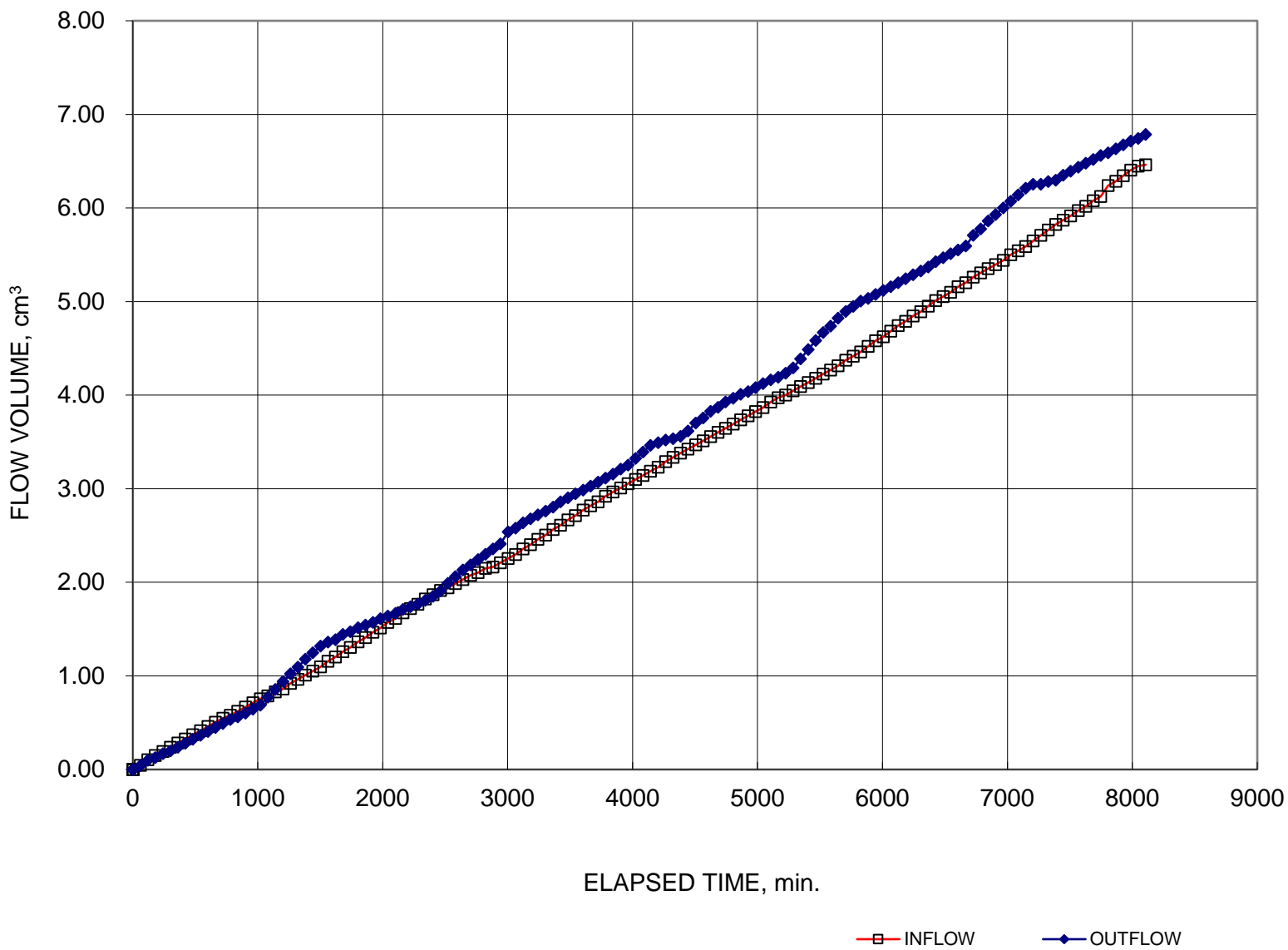
Project number : 1671430-WO 007  
Prepared by : SJ

**Golder Associates**

Checked by : MM



Flow volume vs. Time  
BOREHOLE NUMBER - A2-1  
SAMPLE - TO-7





**APPENDIX D**

# Analytical Chemical Test Results





Your Project #: 1671430 WO 0017  
Site Location: HIGHWAY 9  
Your C.O.C. #: 132046

**Attention: Carter Comish**

Golder Associates Ltd  
6925 Century Ave  
Suite 100  
Mississauga, ON  
CANADA L5N 7K2

**Report Date: 2019/06/28**  
Report #: R5776748  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BV LABS JOB #: B9H3646**

**Received: 2019/06/25, 10:54**

Sample Matrix: Soil  
# Samples Received: 2

Analyses	Quantity	Date	Date	Laboratory Method	Reference
		Extracted	Analyzed		
Chloride (20:1 extract)	2	2019/06/27	2019/06/28	CAM SOP-00463	SM 4500-Cl E m
Conductivity	2	2019/06/28	2019/06/28	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	1	2019/06/27	2019/06/27	CAM SOP-00413	EPA 9045 D m
pH CaCl2 EXTRACT	1	2019/06/28	2019/06/28	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2019/06/25	2019/06/28	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2019/06/27	2019/06/28	CAM SOP-00464	EPA 375.4 m

**Remarks:**

Bureau Veritas Laboratories are accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by BV Labs are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in BV Labs profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and BV Labs in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

BV Labs liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. BV Labs has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by BV Labs, unless otherwise agreed in writing. BV Labs is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by BV Labs, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

\* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.





Your Project #: 1671430 WO 0017  
Site Location: HIGHWAY 9  
Your C.O.C. #: 132046

**Attention: Carter Comish**

Golder Associates Ltd  
6925 Century Ave  
Suite 100  
Mississauga, ON  
CANADA L5N 7K2

**Report Date: 2019/06/28**  
Report #: R5776748  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BV LABS JOB #: B9H3646**  
**Received: 2019/06/25, 10:54**

**Encryption Key**

Please direct all questions regarding this Certificate of Analysis to your Project Manager.  
Ema Gitej, Senior Project Manager  
Email: Ema.Gitej@bvlabs.com  
Phone# (905)817-5829

=====

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.





BUREAU  
VERITAS

BV Labs Job #: B9H3646  
Report Date: 2019/06/28

Golder Associates Ltd  
Client Project #: 1671430 WO 0017  
Site Location: HIGHWAY 9  
Sampler Initials: CC

### SOIL CORROSIVITY PACKAGE (SOIL)

<b>BV Labs ID</b>		KCF970			KCF971		KCF971		
<b>Sampling Date</b>		2019/01/21 15:00			2018/11/20 15:00		2018/11/20 15:00		
<b>COC Number</b>		132046			132046		132046		
	<b>UNITS</b>	<b>A1-1 SS-3</b>	<b>RDL</b>	<b>QC Batch</b>	<b>A2-1 SS-3</b>	<b>QC Batch</b>	<b>A2-1 SS-3 Lab-Dup</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>									
Resistivity	ohm-cm	230		6195406	670	6195406			
<b>Inorganics</b>									
Soluble (20:1) Chloride (Cl-)	ug/g	2400	100	6200038	660	6200038	660	20	6200038
Conductivity	umho/cm	4330	2	6202502	1490	6202502	1480	2	6202502
Available (CaCl2) pH	pH	8.03		6202543	8.19	6200145			
Soluble (20:1) Sulphate (SO4)	ug/g	99	20	6200039	120	6200039	100	20	6200039
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate									





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VERITAS

BV Labs Job #: B9H3646

Report Date: 2019/06/28

Golder Associates Ltd

Client Project #: 1671430 WO 0017

Site Location: HIGHWAY 9

Sampler Initials: CC

## TEST SUMMARY

**BV Labs ID:** KCF970  
**Sample ID:** A1-1 SS-3  
**Matrix:** Soil

**Collected:** 2019/01/21  
**Shipped:**  
**Received:** 2019/06/25

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6200038	2019/06/27	2019/06/28	Deonarine Ramnarine
Conductivity	AT	6202502	2019/06/28	2019/06/28	Surinder Rai
pH CaCl2 EXTRACT	AT	6202543	2019/06/28	2019/06/28	Surinder Rai
Resistivity of Soil		6195406	2019/06/28	2019/06/28	Anastassia Hamanov
Sulphate (20:1 Extract)	KONE/EC	6200039	2019/06/27	2019/06/28	Deonarine Ramnarine

**BV Labs ID:** KCF971  
**Sample ID:** A2-1 SS-3  
**Matrix:** Soil

**Collected:** 2018/11/20  
**Shipped:**  
**Received:** 2019/06/25

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6200038	2019/06/27	2019/06/28	Deonarine Ramnarine
Conductivity	AT	6202502	2019/06/28	2019/06/28	Surinder Rai
pH CaCl2 EXTRACT	AT	6200145	2019/06/27	2019/06/27	Surinder Rai
Resistivity of Soil		6195406	2019/06/28	2019/06/28	Anastassia Hamanov
Sulphate (20:1 Extract)	KONE/EC	6200039	2019/06/27	2019/06/28	Deonarine Ramnarine

**BV Labs ID:** KCF971 Dup  
**Sample ID:** A2-1 SS-3  
**Matrix:** Soil

**Collected:** 2018/11/20  
**Shipped:**  
**Received:** 2019/06/25

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6200038	2019/06/27	2019/06/28	Deonarine Ramnarine
Conductivity	AT	6202502	2019/06/28	2019/06/28	Surinder Rai
Sulphate (20:1 Extract)	KONE/EC	6200039	2019/06/27	2019/06/28	Deonarine Ramnarine





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VERITAS

BV Labs Job #: B9H3646

Report Date: 2019/06/28

Golder Associates Ltd

Client Project #: 1671430 WO 0017

Site Location: HIGHWAY 9

Sampler Initials: CC

### GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	13.0°C
-----------	--------

Samples have been kept cold prior to sample submission as per client information.

Samples have been received and analyzed past the recommended hold time.

**Results relate only to the items tested.**





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VERITAS

BV Labs Job #: B9H3646

Report Date: 2019/06/28

## QUALITY ASSURANCE REPORT

Golder Associates Ltd

Client Project #: 1671430 WO 0017

Site Location: HIGHWAY 9

Sampler Initials: CC

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
6200038	Soluble (20:1) Chloride (Cl <sup>-</sup> )	2019/06/28	NC	70 - 130	103	70 - 130	<20	ug/g	0.048	35
6200039	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2019/06/28	NC	70 - 130	100	70 - 130	<20	ug/g	17	35
6200145	Available (CaCl <sub>2</sub> ) pH	2019/06/27			100	97 - 103			0.81	N/A
6202502	Conductivity	2019/06/28			104	90 - 110	<2	umho/cm	0.55	10
6202543	Available (CaCl <sub>2</sub> ) pH	2019/06/28			100	97 - 103			0.25	N/A

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)





BUREAU  
VERITAS

BV Labs Job #: B9H3646

Report Date: 2019/06/28

Golder Associates Ltd

Client Project #: 1671430 WO 0017

Site Location: HIGHWAY 9

Sampler Initials: CC

### VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Anastassia Hamanov, Scientific Specialist

---

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.



Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name: <u>Golder Associates Ltd.</u>		Company Name: <u>    </u>		Quotation #: <u>    </u>		<input type="checkbox"/> Regular TAT (5-7 days) Most analyses <b>PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS</b> - Rush TAT (Surcharges will be applied) <input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input checked="" type="checkbox"/> 3-4 Days	
Contact Name: <u>Carter Cornish</u>		Contact Name: <u>    </u>		P.O. # / AFE#: <u>1671430 WO 0017</u>			
Address: <u>8725 Century Ave. Unit 100</u>		Address: <u>    </u>		Project #: <u>1671430 WO 0017</u>			
<u>Mississauga, ON L5N 7K2</u>				Site Location: <u>Highway 9</u>			
Phone: <u>416-571-0342</u> Fax: <u>    </u>		Phone: <u>    </u> Fax: <u>    </u>		Site #:			
Email: <u>ccornish@golder.com</u>		Email: <u>    </u>		Site Location Province: <u>ON</u>		Date Required: <u>Fri Jun 28 2019</u>	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY				Sampled By: <u>    </u>		Rush Confirmation #:	
Regulation 153		Other Regulations		Analysis Requested			
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/ Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table <u>    </u> FOR RSC (PLEASE CIRCLE) Y / N		<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQO Region <u>    </u> <input type="checkbox"/> Other (Specify) <u>    </u> <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		REG 153 METALS & INORGANICS REG 153 METALS (Pb, Cr, V, Cu, Ni, Ag, Hg, Mn, Fe, Zn, Al, Cd, Se, B, Mo, As, Sb, Sn, Bi, Ti, Si, F, Cl, Br, I, S, P, N, O, C, H)			
Include Criteria on Certificate of Analysis: Y / N		SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM		LABORATORY USE ONLY			
CUSTODY SEAL <u>0/ N</u>				COOLER TEMPERATURES			
Present		Intact		<u>12/12/15</u>			
COOLING MEDIA PRESENT: Y / N		<u>0</u>		COMMENTS			
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX			
1	<u>A1-1 SS-3</u>	<u>2019/01/21</u>	<u>3:00pm</u>	<u>1</u>	<u>Set #1 PH, CHLORIDE, SULPHATE, EC</u>		
2	<u>A2-1 SS-3</u>	<u>2018/11/20</u>	<u>3:00pm</u>	<u>1</u>	<u>Set #1 PH, Chloride, Sulphate, EC, Resistivity</u>		
3							
4							
5							
6							
7							
8							
9							
10							
RELINQUISHED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)
<u>Carter Cornish</u>		<u>2019/06/25</u>	<u>10:45</u>	<u>[Signature]</u>		<u>2019/06/25</u>	<u>10:54</u>

25-Jun-19 10:54

Ema Gitej



B9H3646

MAF

ENV-803

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**APPENDIX E**

# Foundations Risk Register




RISK REGISTER WORKSHEET

Option #1


\$3,913,000	= Pessimistic Cost
\$3,255,880	= Reasoned Cost

\*NOTE: MUST FILL IN ESTIMATED COST BELOW

Estimated Low Project Cost	\$3,131,000	= Optimistic Cost
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Risk Identification					No Action against Risk						Action against Risk						Risk Quantification						Strategy		Status			
Risk #	Date (Month-Day- Year)	Improvement Type	Risk Breakdown Structure	Detailed Description of Risk Event (Specific, Measurable, Attributable, Relevant, Time bound) [SMART]	Risk Cost (without response) (\$)	Probability	Cost Impact	Schedule Impact	Cost Criticality	Schedule Criticality	Risk Response Strategy	Risk Cost (if Response Strategy Implemented) (\$)	Probability	Cost Impact	Schedule Impact	Cost Criticality	Schedule Criticality	Weighted Risk Cost (\$) without Response	Weighted Risk Cost (\$) with Response	Cost of Risk Response Strategy (\$)		Risk Allowance (\$)	Risk Allowance Rationale	Strategy	Adopted Risk Strategy & Risk History	Status	Risk Owner	Risk Review Date (MM-DD-YY)
			 Risk Breakdown Structure																	Capital	Other							
Option 1				23 m span (centred)														\$0	\$0									
1		New Structures - Precast Box	Structures and Foundations STF	- Based on the design information, the existing timber piles are battered inward and spaced at 1270 mm c/c. '- New steel piles would have to also be spaced at 1270 mm c/c and fit between battered timber piles. '- Theoretically 326 mm clear between old and new piles. -Theoretically only 182 mm clear between old piles and new CSPs ** Potential conflict	\$300,000	4	4	4	H	H	- Excavate existing abutment pile cap and expose tops of existing piles to accurately locate so reduce chance of interference '- Contractor to allow 2 weeks for Owner to review prior to pile driving operations.	\$0	1	1	1	L	L	\$192,000	\$0		\$2,500	\$2,500	- No cost to excavation as this is work the Contractor will have to do anyway. '- Further cost impacts for scenarios that develop after excavation are shown in the following entries.	Avoid		Active	Structural	
2				- Upon exposing existing piles, discover instances where existing spacing is too tight to fit new piles or CSPs where originally designed, resulting in redesign of abutments and piles	\$60,000	3	2	3	M	M	- Design abutment to be stiff (built-in beam) to accommodate any odd spacing requirements.	\$2,000	1	1	1	L	L	\$14,400	\$80	\$18,000	\$2,400	\$14,400	- Appears that upfront cost to strengthen abutment is slightly more expensive than weighted risk cost.	Avoid	- Abutment will be designed to accommodate a large range of pile spacing(s) so no redesign will be needed to meet field conditions. While slightly more expensive upfront it does reduce risk of schedule slip and gives cost certainty	Active	Structural	
				- Actual spacing may impact pile capacity due to group effect. '- Redesign during constuction, longer piles.	\$42,000	3	2	2	M	M	- Will have to accept	\$32,000	3	2	2	M	M	\$10,080	\$7,680			\$7,680	No mitigation possible.	Accept	- Will have to be dealt with when spacing is known.	Active	Geotechnical	
				- Actual spacing may impact pile capacity due to group effect. '- Redesign during design, longer piles.	\$10,000						-Modify current pile group design to include conservatism in capacity to account for potential closer pile spacing; cannot anticipate all scenarios, but will reduce time for redseign during construction.																	
				- May discover that existing piles are spaced such that 3 or more piles are too close to allow new piles to fit. '- Would be the case if 3 adjacent piles are about 900 mm apart each. ** Result is that Contractor would have to pull an existing pile. '- Schedule impact, potential reduction in pile capacity.	\$90,000	2	3	4	L	M	- Geotech to provide instructions for how Contractor to address pulled piles before construction to prevent much schedule slippage. '- Pulling and filling void will still have to be done but with cost certainty.	\$75,000	2	3	2	L	L	\$21,600	\$18,000		\$10,000	\$28,000	- This is mitigated cost plus upfront design work by Geotech	Mitigate	Geotech to investigate and provide recommendations. To be incorporated into contract.	Active	Geotechnical	



				Estimated Low Project Cost				\$3,131,000				= Optimistic Cost																			
Risk Identification					No Action against Risk						Action against Risk						Risk Quantification						Strategy		Status						
Risk #	Time (Month-Day- Year)	Improvement Type	<div><div>Risk Breakdown Structure</div><div>Risk Breakdown Structure</div></div>	Detailed Description of Risk Event (Specific, Measurable, Attributable, Relevant, Time bound) [SMART]	Risk Cost (without response) (\$)	Probability	Cost Impact	Schedule Impact	Asset Criticality	Reputation Criticality	Risk Response Strategy	Risk Cost (if Response Strategy Implemented) (\$)	Probability	Cost Impact	Schedule Impact	Asset Criticality	Reputation Criticality	Weighted Risk Cost (\$) without Response	Weighted Risk Cost (\$) with Response	Cost of Risk Response Strategy (\$)	Risk Allowance (\$)	Risk Allowance Rationale	Strategy	Adopted Risk Strategy & Risk History	Status	Risk Owner	Next Review Date (MM-DD-YY)				
				- Despite exposing existing piles, when driving new piles it is still possible that new piles impact existing. **1st Case: pile impacts cleanly and stops. Contractor must pull pile and reinstall adjacent to planned location.	\$45,000	1	2	2	L	L	None	\$45,000	1	2	2	L	L	\$3,600	\$3,600		\$3,600		Accept		Active	Foundations					
				- Despite exposing existing piles, when driving new piles it is still possible that new piles impact existing. **2nd Case: pile deflects slightly, resulting in out of plumb piles that may not align under abutment. *- Redesign of wider abutment. Must check if integral abutment action is still valid. *- Geotech must examine reduction in capacity.	\$145,000	3	3	4	M	M	- Structural to set abutment width more than normal to accept out-of-alignment piles (still must check integral action). *- Geotech must still assess reduction in capacity for out-of-plumb and group effect.	\$3,000	3	1	1	L	L	\$52,200	\$360	\$138,000	\$0	\$52,200	Use Unmitigated cost	Accept	- Not sure if pre-specifying wider abutment makes sense. Perhaps just have design ready if needed to avoid schedule impact?						
				- 2nd Case part 2: If pile deflects too much, integral action is lost and pile will have to be pulled and redone. *- Geotech may need to check capacity of new pile next to new hole. May have to fill hole. -Geotech may need to provide new (lower) spring constants for AECOM to evaluate response of pile group	\$90,000	1	3	4	L	L	- Geotech to provide instructions for how Contractor to address pulled piles before construction to prevent much schedule slippage. *- Pulling and filling void will still have to be done but with cost certainty.	\$75,000	1	3	3	L	L	\$10,800	\$9,000		\$7,500	\$16,500	Mitigate	Geotech to investigate and provide recommendations. To be incorporated into contract.	Active	Geotechnical					
																		\$0	\$0												



\$3,681,000	= Pessimistic Cost
\$3,681,000	= Reasoned Cost

<b>Estimated Low Project Cost</b>	<b>\$3,681,000</b>	= Optimistic Cost
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[illegible]



\$4,131,000	= Pessimistic Cost
\$4,105,000	= Reasoned Cost

<b>Estimated Low Project Cost</b>	<b>\$3,831,000</b>	= Optimistic Cost
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[illegible]



**APPENDIX F**

**Non-Standard Special Provisions,  
Special Provisions and Notices to  
Contractor**



## **Restrictions on Construction Operations – Excavation of Organic Materials**

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### **Notice to Contractor**

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Sub-excavation of existing organic materials at Highway 9 Holland Drainage Canal Bridge during Stage 1 (as indicated in the Contract Drawings) shall be undertaken in such a way that the maximum length of open excavation in any area in any direction at any given time shall be 3 m. The Contractor shall place Granular B Type II (OPSS 1010) immediately behind the peat excavation operation so that the 3 m maximum open excavation requirement is maintained. At the end of each day's work, the Contractor shall ensure that all excavations are backfilled with Granular B Type II.



## **PRELOAD PERIOD – East and West of Abutments for Highway 9 Holland Drainage Canal Overpass**

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### Notice to Contractor

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The Contractor shall schedule his operation to include preloading of the full preload and surcharge embankment height for the Highway 9 Holland Drainage Canal Overpass. The area to be preloaded area must extend 100 m from the abutment(s) and be constructed according to the contract drawings. The full height embankment and surcharge shall remain in place for a minimum preload period of 8 months, prior to driving the piles at the abutments.

The Contractor shall not proceed with removal of the preloaded full surcharge embankment height and driving of the piles until approval has been given by the Contract Administrator.



## **DEEP FOUNDATIONS – Item No.**

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### **Special Provision**

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#### **Amendment to OPSS.PROV 903, April 2016**

##### **903.01 SCOPE**

Section 903.01 of OPSS.PROV 903 is amended by the addition of the following:

Under the above tender items, the Contractor shall:

- a) Supply and install H-Piles
- b) Coordinate with the Contractor Administrator or an independent testing company retained by the Contract Administrator for Pile Dynamic Analyzer (PDA) testing.

All as shown on the Contract Drawings.

##### **903.02 REFERENCES**

Section 903.02 of OPSS.PROV 903 is amended by the addition of the following:

The subsurface conditions at the site are described in the following Foundation Investigation Report:

Highway 9 - Holland Drainage Canal Bridge (Site No. 37-31) Replacement, Schomberg, Ontario,  
G.W.P. 2266-18-00

##### **903.07 CONSTRUCTION**

###### **903.07.02.07 Monitoring Driven Piles**

###### **903.07.02.07.03 Driving to a Specified Ultimate Resistance**

###### **903.07.02.07.03.01 General**

Clause 903.07.02.07.03.01 of OPSS.PROV 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be validated using high-strain dynamic testing at end of drive (EOD) as performed by Contract Administrator or an independent testing company retained by the Contract Administrator. If the specified ultimate resistance is not achieved, retap / restrike should be conducted after sufficient time has passed to allow soil setup. The requirements for soil setup are as specified in the Contract Documents.



#### **903.07.02.07.04.02 High-Strain Dynamic Testing**

High-strain dynamic testing shall be performed by the Contract Administrator or an independent testing company retained by the Contract Administrator using the Pile Driving Analyzer, or approved equivalent, for the determination of pile ultimate resistance, establishment of pile installation criteria, assessment of pile integrity, monitoring of hammer/drive system performance and driving stresses, as specified in the Contract Documents. The method and equipment for testing and its reporting shall be according to ASTM D 4945.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing, and shall be submitted to the Contract Administrator for information purposes. The final piles to be tested will be decided by the Contract Administrator.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 10% of piles at each foundation element, rounded up, but no fewer than 2 piles for each stage of construction or as specified in the Contract Documents. Therefore, a minimum of 8 piles shall be tested using high-strain dynamic methods at the bridge location at the end of initial driving.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out during the retapping of piles, as specified in the Retapping Tests on Piles clause. Restrike testing shall be performed on a minimum of 10% of piles at each foundation element, rounded up, but no fewer than 2 piles for each stage of construction or as specified in the Contract Documents. Therefore, retapping a minimum of 8 piles using high strain dynamic testing shall be performed at the bridge location.

Restrike / retap testing shall be carried out no sooner than fourteen (14) days after installation of the individual pile and as specified in the Contract Documents. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

#### **903.07.02.07.06 Retapping Tests on Piles**

Section 903.07.02.06 is deleted in its entirety and replaced by the following:

In each pile group, 10% of the piles rounded up to the next whole number, but no fewer than two piles per stage of construction, shall be retapped no sooner than fourteen (14) days after installation of the individual pile to confirm that the ultimate axial geotechnical resistance has been achieved and/or sustained.



## **HIGH-STRAIN DYNAMIC TESTING, DEEP FOUNDATIONS – Item No.**

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Special Provision

October 2017

### **Amendment to OPSS 903, April 2016**

#### **903.02 REFERENCES**

Section 903.02 of OPSS 903 is amended by the addition of the following under **ASTM International**:

D 4945-12                      Standard Test Method for High-Strain Dynamic Testing of Deep Foundations

#### **903.03 DEFINITIONS**

Section 903.03 of OPSS 903 is amended by the addition of the following:

**High Strain Dynamic Testing** means a method of evaluating the quality of deep foundations and/or performance of the drive system. It is a form of load testing and involves the instrumenting and application of dynamic loads to a tested pile.

#### **903.04 DESIGN AND SUBMISSION REQUIREMENTS**

##### **903.04.02 Submission Requirements**

Subsection 903.04.02 of OPSS 903 is amended by the addition of the following clause:

##### **903.04.02.07 High-Strain Dynamic Testing**

Prior to commencing high-strain dynamic testing, calibration certificates of all equipment used shall be submitted to the Contract Administrator. All equipment used shall be in good working condition, and shall have been calibrated within the last 2 years according to ASTM D 4945. Equipment set-up may be completed by trained Contractor personnel, however, testing shall be performed under the direction of an Engineer with at least 5 years of experience in high-strain dynamic testing and holding a proficiency rating at the Intermediate level or better for Dynamic Measurement and Analysis Proficiency Test as administered by the Pile Driving Contractors Association (PDCA). After December 31, 2020, the Engineer shall be required to hold a proficiency rating level of Advanced or better.

A preliminary report on the test results and its analysis shall be submitted to the Contract Administrator on the same day of the testing. The analysis shall be based on a closed-form solution (Case Method or approved equivalent) or signal-matching analyses (Case Pile Wave Analysis Program - CAPWAP or approved equivalent). As a minimum, the preliminary report shall include:

- a) Pile ultimate resistance and integrity.
- b) Calculated driving stresses.
- c) Transferred energy and hammer efficiency at the time of the test.

A final report shall be submitted to the Contract Administrator within 10 Days of the field testing. The final report shall include the following:



- a) Results of pile ultimate resistance and pile integrity based on signal-matching analyses (CAPWAP or approved equivalent), hammer performance and comparisons with any applicable static load test.
- b) Discussion and recommendations for soil setup/relaxation, and/or revised pile installation criteria.
- c) An appendix shall be included containing the following documents:
  - i. Pile installation record
  - ii. Reference subsurface information (borehole record)
  - iii. Pile location drawing
  - iv. Initial calibration check by the test computer unit
  - v. Test set up geometry

The report shall be signed and sealed by two Engineers of the testing company, one of whom shall be identified as MTO's designated contact and one of whom shall have the required experience in high-strain dynamic testing and hold the required certificate of PDCA Proficiency Test.

## **903.07 CONSTRUCTION**

### **903.07.02.07 Monitoring Driven Piles**

#### **903.07.02.07.03 Driving to a Specified Ultimate Resistance**

##### **903.07.02.07.03.01 General**

Clause 903.07.02.07.03.01 of OPSS 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be validated using high-strain dynamic testing at end of drive (EOD). If the specified ultimate resistance is not achieved, retap/restrike should be conducted after sufficient time has passed to allow soil setup. The requirements for soil setup are as specified in the Contract Documents.

The results of the high-strain dynamic tests shall be submitted to the Contract Administrator who shall, in collaboration with the independent testing company, verify that the specified ultimate resistance has been achieved.

##### **903.07.02.07.04 Wave Equation Analysis**

Clause 903.07.02.07.04 is deleted in its entirety and replaced with the following:

#### **903.07.02.07.04 Wave Equation Analysis and High-Strain Dynamic Testing**

##### **903.07.02.07.04.01 Wave Equation Analysis**

Prior to mobilizing piling equipment to the site, a Wave Equation Analysis of Piles (WEAP) analysis shall be performed by the Contractor to demonstrate the potential for the proposed piling equipment to activate the specified ultimate resistance specified in the Contract Documents.

When requested by the Contract Administrator, all equipment, material, and personnel shall be supplied to conduct the wave equation analysis procedure.



#### **903.07.02.07.04.02      High-Strain Dynamic Testing**

An independent testing company with no corporate affiliation with the Contractor shall be employed to perform the high-strain dynamic testing. The independent testing company shall be RAQs qualified (Specialty: Geotechnical (Structures and Embankments – Medium or High Complexity)).

High-strain dynamic tests shall be performed by an Engineer employed by the independent testing company. The Engineer shall have documented evidence of training and experience in foundation engineering and wave equation analyses, and a certificate of proficiency (intermediate level or better) in the PDCA Dynamic Measurement and Analysis Proficiency Test.

High-strain dynamic testing shall be performed using the Pile Driving Analyzer, or approved equivalent, for the determination of pile ultimate resistance, establishment of pile installation criteria, assessment of pile integrity, monitoring of hammer/drive system performance and driving stresses, as specified in the Contract Documents. The method and equipment for testing and its reporting shall be according to ASTM D 4945.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing, and shall be submitted to the Contract Administrator for information purposes.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 10% of piles at each foundation element, rounded up, but no fewer than 2 piles for each stage of construction or as specified in the Contract Documents. Therefore, a minimum of 8 piles shall be tested using high-strain dynamic methods at the bridge location at the end of initial driving.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out during the retapping of piles, as specified in the Retapping Tests on Piles clause. Restrike testing shall be performed on a minimum of 10% of piles at each foundation element, rounded up, but no fewer than 2 piles for each stage of construction or as specified in the Contract Documents. Therefore, retapping a minimum of 8 piles using high strain dynamic testing shall be performed at the bridge location.

Restrike testing shall be carried out no sooner than fourteen (14) days after installation of the individual pile and at a time specified in the Contract Documents. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

#### **903.10                      BASIS OF PAYMENT**

Section 903.10 of OPSS 903 is amended by the addition of the following subsection:

##### **903.10.04                      High-Strain Dynamic Testing, Deep Foundations - Item**

Payment for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

WARRANT: Always with this item.



**TEMPORARY PROTECTION SYSTEMS - Item No.**

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Special Provision

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**Amendment to OPSS 539, November 2014**

**593.07.02 Removal of Protection Systems**

Subsection 539.07.02 of OPSS 539 is deleted in its entirety and replaced with the following:

Protection systems shall be removed from the right-of-way unless it is specified in the Contract Documents that the protection system may be left in place.

Where piles are left in place, the top shall be removed to at least 1.5 m below the finished grade or ground level.

The method and sequence of construction of the approach embankments and abutment foundations shall be such that there shall be no damage to the new work, existing work and facility being protected.

All disturbed areas shall be restored to an equivalent or better condition than existing prior to the commencement of construction.



## **DEWATERING STRUCTURE EXCAVATIONS - Item No.**

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Special Provision No. FOUN 0003

March 8, 2018

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### **Amendment to OPSS 902, November 2010**

#### **902.02 REFERENCES**

Section 902.02 of OPSS 902 is amended by the addition of the following:

#### **Ontario Provincial Standard Specifications, Construction**

OPSS 517      Dewatering  
OPSS 805      Temporary Erosion and Sediment Control Measures

#### **902.03 DEFINITIONS**

Section 903.03 of OPSS 902 is amended by the addition of the following:

**Automatic Transfer Switch** means as defined in OPSS 517.

**Cofferdam** means as defined in OPSS 539.

**Cut-Off Wall** means as defined in OPSS 517.

**Design Storm Return Period** means as defined in OPSS 517.

**Dewatering System** means as defined in OPSS 517.

**Groundwater Control System** means as defined in OPSS 517.

**Plug** means as defined in OPSS 517.

**Sediment** means as defined in OPSS 517.

**Sediment Control Measure** means as defined in OPSS 517.

**Temporary Flow Passage System** means as defined in OPSS 517.

**Unwatering** means as defined in OPSS 517.

**Vegetated Discharge Area** means as defined in OPSS 517.

**Waterbody** means as defined in OPSS 517.

**Watercourse** means as defined in OPSS 517.



## **902.04 DESIGN AND SUBMISSION REQUIREMENTS**

### **902.04.01 Design Requirements**

#### **902.04.01.01 Dewatering**

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a [\* Designer Fill-In, See Notes to Designer] year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

### **902.04.02 Submission Requirements**

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

#### **902.04.02.01 Working Drawings**

Working Drawings for the dewatering system shall be according to OPSS 517.

#### **902.04.02.02 Preconstruction Survey**

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of 300 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

#### **902.04.02.03 Milestone Inspections**

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

## **902.07 CONSTRUCTION**

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:



## **902.07.04                      Dewatering Structure Excavation**

### **902.07.04.01                      General**

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

### **902.07.04.02                      Discharge of Water**

The discharge of water shall be according to OPSS 517.

### **902.07.04.03                      Monitoring**

Monitoring shall be according to OPSS 517.

### **902.07.04.04                      System Amendments**

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

### **902.07.04.05                      Removal**

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.



NOTES TO DESIGNER:

\* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

\*\* Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this standard tender item only on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.



**INTERFERENCE WITH EXISTING PILES – PRE-AUGERING AND PILE EXTRACTION – Item No.**

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**Special Provision**

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Following the removal of the pile cap at the existing east and west abutments, at a minimum, the contractor is to provide the following information to the Contract Administrator:

- Quantity of piles visible at time of pile cap removal;
- Dimensions of visible piles (including diameter, layout position and spacing);
- Material type of existing piles; and,
- Clearance distance between the existing piles and the proposed locations of the new H piles.

The Contractor shall not proceed with installation of the production piles until the above information has been given to the Contract Administrator along with a Request to Proceed and approval and a Notice to Proceed has been provided by the Contract Administrator.

The existing piles are expected to consist of twenty-two (22) 300 mm diameter timber piles at each abutment. The piles are expected to be present in two rows per abutment with eleven (11) piles in each row. The pile row closest to the Holland Drainage Canal at each abutment is expected to be battered at a slope of 1H:6V towards the Holland Drainage Canal. The spacing between the piles in a given row is anticipated to be approximately 1.27 m centre-to-centre. The two rows of piles are anticipated to be spaced at 1.22 m from the pile centers. The length of the existing piles is estimated to be approximately 14 m below the underside of the existing pile cap.

Prior to pile driving, the Contractor shall check for the potential for a conflict between the existing and new pile by pre-augering with a 457 mm (18 inch) diameter auger to a depth of 5 m below the bottom of new pile cap (to Elevation 213.0 m). If a conflict is detected during the pre-augering, the Contractor shall inform the Contract Administrator. The Contractor shall not continue or attempt to drive the pile at the location where contact has been made unless approval has been given by the Contract Administrator.

If a production pile contacts an existing pile during driving of the production piles, the Contractor shall stop driving the pile in which contact has been made and shall inform the Contract Administrator. The Contractor shall not continue or attempt to advance the pile in which contact has been made unless approval has been given by the Contract Administrator. The Contractor shall propose, and following approval, attempt mitigation strategies (for example, removing and adjusting the location of the production pile) if the Contractor receives direction to do so from the Contract Administrator.

As a last resort, and only if no other options are available, prior to the Contractor attempting to extract an existing timber pile, the Contractor must submit a Request to Proceed to the Contract Administrator and receive a Notice to Proceed from Contract Administrator. The Contract Administrator may review the information presented by the Contractor for up to fourteen (14) days prior to issuing the Notice to Proceed. The Contractor shall understand that pile extraction is viewed as the least favorable option to mitigate production pile contact with the existing timber piles.

In the event that the Contract Administrator issues the Contractor written approval (Notice to Proceed) to extract an existing timber pile, the Contractor will determine the most suitable extraction method and proceed with the extraction. During a pile extraction, at minimum, the following details will be provided to the Contract Administrator:



- Length of extracted pile;
- A description of the method used to extract the pile; and,
- Detailed notes and photographs outlining the events of the extraction procedure.

For the purposes of bidding, the Contractor shall assume that a total of two (2) pile extractions will be required during the installation of the production piles.



## **CELLULAR CONCRETE - Item No.**

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Special Provision – Draft Date: February 3, 2020

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### **1.0 SCOPE**

This specification covers the requirements for the supply and placement of cellular concrete for use as lightweight fill at the locations and in accordance with the details shown in the Contract Documents.

### **2.0 REFERENCES**

This specification refers to the following standards, specifications, or publications:

#### **Ontario Provincial Standard Specifications, Construction:**

OPSS 517	Dewatering
OPSS 539	Temporary Protection System
OPSS 904	Concrete Structures

#### **Ontario Provincial Standard Specifications, Material:**

OPSS 1301	Cementing Materials
OPSS 1302	Water
OPSS 1303	Admixtures for Concrete

#### **CSA Standards**

A23.2-17C	Temperature of Freshly Mixed Hydraulic Cement Concrete
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#### **ASTM Standards**

ASTM C869	Standard Specification for Foaming Agents Used in Making Preformed Foam for Cellular Concrete
ASTM C796	Standard Test Method for Foaming Agents for Use in Producing Cellular Concrete Using Preformed Foam
ASTM C495	Standard Test Method for Compressive Strength of Lightweight Insulating Concrete

#### **Ontario Ministry of Transportation Publications**

Designated Sources for Materials (DSM)

MTO Forms:  
PH-CC-433A Concrete Mix Design Submission Form A

### **3.0 CONSTRUCTION**

For the purpose of this specification the following definitions apply:



**Cellular Concrete** means a lightweight cement-based material containing cement, water and foaming agent. The foaming agent is used to create stable and uniformly distributed air cells (voids) in the cellular concrete.

**Plastic Density** means the density of fresh cellular concrete at the point of placement measured according to ASTM C796.

**Oven-Dry Density** means the density of hardened cellular concrete at 28 days measured according to ASTM C495.

## **4.0 DESIGN AND SUBMISSION REQUIREMENTS**

### **4.1 Submission of Working Drawings and Placement Procedures**

The working drawings and the proposed placement procedures shall be submitted to the Contract Administrator.

The working drawings shall be signed and sealed by the design Engineer and the design-check Engineer.

The submission of the working drawing and proposed method of placement shall include a description of the proposed method of installation including, as a minimum, the following:

- a) A work plan outlining the schedule, procedures and work site details;
- b) Proposed dewatering procedure (in accordance with OPSS 517);
- c) Environmental protection strategy;
- d) Method for sealing cracks (if any) to prevent leakage of cellular concrete;
- e) Method for bulkhead construction;
- f) List of equipment to be used
- g) Calibration records for the equipment (not more than 12 months old at the time of submission);
- h) List of all materials to be used in the cellular concrete, using MTO Form PH-CC-433A – Concrete Mix Design Submission Form A;
- i) Target plastic density ( $\text{kg/m}^3$ );
- j) Identification of how the placement procedure will be monitored.
- k) A currently valid Certificate of Ready Mixed Concrete Production Facilities as issued by the Ready Mixed Concrete Association of Ontario (RMCAO) for any plant to be used on the Contract.

A Request to Proceed shall be submitted to the Contract Administrator, with the submission of the working drawings and proposed placement procedures, at least fifteen (15) Business Days prior to commencement of the work.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

### **4.2 Submission of Environmental Protection Strategy**

At least fifteen (15) Business Days before the commencement of work, six copies of an environmental protection strategy shall be submitted to the Contract Administrator as specified under the Environmental Protection Strategy Subsection.

### **4.3 As-Built Drawings**



As-built drawings shall be submitted to the Contract Administrator in a reproducible format

The as-built drawings shall be dated and bear the seal and signature of the design Engineer and design-check Engineer.

## **5.0 MATERIAL**

### **5.1 Cementing Materials**

Cementing materials shall be according to OPSS 1301. Supplementary cementing materials shall not be used.

### **5.2 Water**

Water used for production shall be according to OPSS 1302.

### **5.3 Admixtures**

Admixtures shall be according to OPSS 1303.

### **5.4 Foaming Agents**

Foaming agents shall conform to the requirements of ASTM C869 when tested according to ASTM C796.

### **5.5 Cellular Concrete Properties**

The cellular concrete product used in the work shall be on the DSM list #2.35.30 for Lightweight Fill Material.

Cellular concrete shall have the following properties:

- a) Minimum unconfined compressive strength at 28 days shall be 1.0 MPa.
- b) The design density shall be 510 kg/m<sup>3</sup>.
- c) Temperature of the plastic cellular concrete at the time of discharge shall be between 10 and 28 °C.
- d) Plastic density shall be within  $\pm 5\%$  of the target plastic density submitted by the Contractor.
- e) Oven-dry density shall be within  $\pm 5\%$  of the design density.

## **6.0 EQUIPMENT**

Cellular concrete shall be produced utilizing automated proportioning, mixing, and foam producing equipment, which is capable of producing cellular concrete meeting the specified properties.

Dry-mix equipment must be able to receive bulk cement and process it continuously from one piece of equipment, and pump through hoses or pipes up to a flat lineal distance of 1000 m.

Wet-mix equipment must be able to receive slurry on-site into the equipment and process it continuously during ready-mix supply, and pump through hoses or pipes up to a flat lineal distance of 200 m. When wet-mix equipment is used the batching plant and equipment used to batch the slurry shall be certified by the RMCAO prior to producing slurry for the work and shall meet the requirements for certification



throughout the production of concrete. All truck mixers used to deliver the slurry shall be certified by RMCAO and shall display valid certification stickers.

Cellular concrete must be pumped by a positive displacement pump. A foam generator shall be used to continuously produce pre-formed foam, which shall be injected and mixed with the cementitious slurry downstream of the positive displacement slurry pump. The equipment shall be calibrated to produce a precise, consistent and predictable volumetric rate of foam with stable uniform microbubbles.

## **7.0 CONSTRUCTION**

### **7.1 Excavation and Subgrade Preparation**

Foundation excavation shall be carried out to the design elevations and the horizontal and vertical limits shown in the Contract Drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be sub excavated.

The prepared subgrade shall be good competent level ground and snow and ice must be removed from the area prior to placement.

### **7.2 Dewatering**

The prepared subgrade shall be free of standing water during placement of cellular concrete and until backfill is placed on top of the cellular concrete. If necessary, dewatering shall be continuous during placement of materials.

Dewatering shall be according to OPSS 517.

### **7.3 Roadway Protection System**

The construction of all protection schemes shall be according to OPSS 539 and paid for under the appropriate tender item. Where the stability, safety or function of an existing roadway, railway, other works, or proposed works may be impaired due to the method of operation, such protection as may be required shall be provided.

### **7.4 Placement**

The placement of cellular concrete shall be under the direct supervision of the Contractor's Engineer.

The Contractor's Engineer shall be on site to oversee the placement of the cellular concrete and to verify that the cellular concrete is being supplied and placed in accordance with the Contract Documents.

A Certificate of Conformance shall be submitted to the Contract Administrator upon completion of the placement of the cellular concrete.

The maximum lift thickness shall be determined based on density and any other considerations that may impact placement. The depth of the cellular concrete lifts shall be designed to prevent any thermal damage to the cellular concrete caused by heat of hydration. Cellular concrete placement, within a formed area, shall not exceed 2 hours.

Any items to be fully or partially encased in the cellular concrete shall be properly set and stable prior to



the installation of the cellular concrete.

Where required, formwork should be designed and installed to withhold cellular concrete. When working near surface water, formworks shall be lined with an impermeable liner to prevent any leakage.

Cellular concrete shall not be placed on frozen ground. Cellular concrete may be placed during freezing conditions, provided measures are taken to prevent damage to the cellular concrete until sufficient strength has been attained. Cellular concrete shall not freeze before initial set. Cold weather protection shall be provided in accordance with OPSS 904.

Fresh cellular concrete shall be protected from contact with rain or snow. The cellular concrete shall be placed in the dry condition and above any groundwater table. All surfaces against which cellular concrete is placed shall be free of standing water. Cellular concrete shall be placed above any groundwater table.

Once mixed, the cellular concrete shall be conveyed promptly to the location of placement without excessive handling.

Finished surface elevation shall be within  $\pm 25$  mm of the design grades shown in the Contract Drawings. Cellular concrete can be placed with a maximum slope of 1%. Slopes greater than 1% will require profiling by creating steps for the cellular concrete with formwork.

Vehicles, equipment, backfills, successive lifts of cellular concrete or other loadings on the cellular concrete shall be prevented until the material has attained sufficient strength to withstand the loads with no damage. Backfill can commence on the cellular concrete when the cellular concrete has attained sufficient strength such that foot traffic can be supported without leaving an indentation.

## **7.5 Environmental Protection Strategy**

Materials and conduct of the work shall be handled in a manner that will ensure protection of the natural environment and prohibit cellular concrete from entering surface or ground water. Measures shall be taken as necessary to prevent the material from entering the natural environment and/or leaking outside of the intended placement location and shall have established methods for stopping flow of the product as required, and for prompt remediation of any leaks or spills. These measures and any other contingency planning requirements shall be documented in an environmental protection strategy.

## **7.6 Material Sampling and Testing**

### **7.6.1 Testing of Plastic Cellular Concrete for Temperature and Density**

The plastic cellular concrete shall be sampled and tested for temperature and density once for each placement, or once for every 50 m<sup>3</sup>, or once every 30 minutes, whichever is more frequent. Samples of the cellular concrete shall be taken at the point of discharge into the work. Temperature shall be measured according to CSA A23.2-17C. Plastic density shall be measured according to ASTM C796.

Cellular concrete which does not meet the specified requirements for temperature and plastic density shall be rejected and not used in the work.

### **7.6.2 Sampling of Cellular Concrete for 28-Day Compressive Strength Testing and Oven-Dry Density**



Cylinders shall be cast for testing by the Owner of 28-day compressive strength and oven-dry density. Cylinders shall be cast and cured according to ASTM C495, except that the cylinder size and number of samples for determination of oven-dry density shall be according to the following paragraph. Cylinders shall be provided to the Contract Administrator along with a transmittal form including the date of casting of the cylinders, and the lot number.

One set of four 75 mm diameter by 150 mm long cylinders shall be cast for each lot of cellular concrete for determination of 28-day compressive strength. One set of three 150 mm diameter by 300 mm long cylinders shall be cast for each lot of cellular concrete for determination of oven-dry density. The lot size shall be according to the Quality Assurance Section.

## **7.7 Submission of Daily Summary for Plastic Cellular Concrete**

After each Day's work, a daily summary shall be submitted to the Contract Administrator. The daily summary shall include the following:

- a) Date and time of placement;
- b) Air temperature;
- c) Temperature of constituent materials;
- d) Batch quantities;
- e) For packaged products, manufacturer's batch number and date of manufacture;
- f) Water content;
- g) Location of the backfilling application;
- h) Total volume placed.
- i) Plastic temperature and density test results.

## **8.0 QUALITY ASSURANCE**

### **8.1 Testing of 28-Day Compressive Strength and Oven-Dry Density**

#### **8.1.1 Lot Size**

The lot shall consist of all the cellular concrete for one Day's continuous placement, up to a maximum of 100 m<sup>3</sup>. If more than 100 m<sup>3</sup> is placed in one Day, the cellular concrete lightweight fill shall be divided into the smallest number of equal sized lots not exceeding 100 m<sup>3</sup>.

#### **8.1.2 Acceptance of 28-Day Compressive Strength**

The set of four cylinders representing the lot shall be tested for compressive strength according to ASTM C495.

Compressive strength shall be considered acceptable when the average compressive strength of the set of four cylinders is equal to or greater than the specified strength.

Unacceptable cellular concrete shall be subject to removal and replacement.

#### **8.1.3 Acceptance of Oven-Dry Density**

The set of three cylinders representing the lot shall be tested for over-dry density according to ASTM C495.



Oven-dry density shall be considered acceptable when the average oven-dry density of the set of three cylinders is within  $\pm 5\%$  of the design density.

Unacceptable cellular concrete shall be subject to removal and replacement.

## **9.0 MEASUREMENT FOR PAYMENT**

Measurement will be Plan Quantity as may be revised by adjusted Plan Quantity of the cellular concrete in cubic metres.

## **10.0 BASIS OF PAYMENT**

Payment at the contract price for the cellular concrete shall be full compensation for all labour, equipment and material to do the work.



## **VIBRATION MONITORING - Item No.**

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Special Provision

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### **Scope**

This special provision describes requirements for vibration monitoring during foundation piling works, installation of temporary protection system(s), and operation of vibratory compaction equipment for the construction of the abutment foundations and embankment widenings / grade raise for the Highway 9 – Holland Drainage Canal Bridge structure replacement.

### **References**

The subsurface conditions at the site are described in the following Foundation Investigation Report:

Highway 9 - Holland Drainage Canal Bridge (Site No. 37-31) Replacement  
Schomberg, Ontario  
Assignment No. 2016-E-0029-07 and -17, G.W.P. 2266-18-00  
GEOCREC No.: 31D-734

### **Definitions**

**Contractor's Engineer (CE)** means an Engineer with a minimum of five (5) years of experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Contractor's Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificate(s) of conformance.

**Peak Particle Velocity (PPV)** means the maximum component velocity in millimetres per second that ground particles move as a result of energy released from vibratory construction operations.

**Pre-Construction Condition Survey** means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, prior to the commencement of vibratory construction operations.

**Post-Construction Condition Survey** means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, after completion of vibratory construction operations.

### **Submission Requirements**

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring plan to the Contract Administrator for information purposes. The submittals shall satisfy the specifications and at a minimum contain the following specific information:



- a) Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
- b) Qualifications of vibration monitoring specialist.
- c) Details regarding proposed instrumentation.
- d) Proposed location of instruments adjacent to the residences / buildings, utilities, wells, or other potentially vibration-sensitive structures within a 250 m radius from the new bridge foundations, protection system(s), and/or location(s) of vibratory compaction equipment as applicable.
- e) Action plan to be taken to adjust installation methods / construction techniques if readings show vibrations exceeding tolerable levels.

## **Equipment**

All vibration monitoring equipment shall be capable of measuring and recording ground vibration PPV up to 200 mm/s in the vertical, transverse, and radial directions. The equipment shall have been calibrated within the last 12 months either by the manufacturer or other qualified agent. Proof of calibration shall be submitted to the Contract Administrator prior to commencement of any monitoring operations.

## **Construction**

### ***Pre- and Post-Construction Condition Surveys***

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, and facilities within 250 m of the bridge foundations, protection system(s), and location(s) of proposed vibratory compaction equipment.

### ***Pre-Construction Condition Surveys***

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

The Pre-Construction Condition Survey, at each structure/well within a 250 m radius of the bridge, protection system(s), and location(s) of proposed vibratory compaction equipment, shall be completed a minimum of two (2) weeks prior to commencement of construction activities. Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of such construction activities, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

The Pre-Construction Condition Survey shall include, as a minimum, the following information:

- a) Type of structure, including type of construction and if possible, the date when built.
- b) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
- c) Digital photographs or digital video or both, as necessary, to record areas of significant concern.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.



A copy of the Pre-Construction Construction Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request.

### ***Post-Construction Condition Surveys***

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

A Post-Construction Condition Survey at each structure within a 250 m radius of the bridge, protection system(s), and location(s) of proposed vibratory compaction equipment, is required within two (2) months of completion of such activities on this contract.

The Post-Construction Condition Survey shall include, as a minimum, the following information:

- a) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
- b) Digital photographs or digital video or both, as necessary, to record areas of significant concern.
- c) Comparison between pre-condition survey documented concerns and post-condition concerns.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Post-Construction Condition Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request. The report shall confirm that there have been no changes to the property between the Pre-Construction Condition Survey and the Post-Construction Condition Survey as a result of the installation of deep foundations, protection systems, and operation of vibratory compaction equipment.

### **Monitoring**

The vibration monitoring equipment shall be placed on the ground surface as close as possible to the vibratory works and on the ground surface at radial distances of 25 m, 50 m, and 100 m from the foundation element, protection system or vibratory equipment locations at the project site. The Contractor/ Contractor's Engineer shall take readings on the existing bridge foundations and on existing residential/commercial/agricultural structures and utilities located within 250 m of the works during installation of any deep foundation elements (including piles for temporary protection systems) or operation of vibratory equipment, starting with the pile or operation of vibratory equipment furthest away from such structures / utilities. The Contractor / Contractor's Engineer shall take readings continuously during installation of deep foundations, temporary protection systems, and during operation of vibratory compaction equipment and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.



The vibrations measured on the existing and new bridge structures shall not exceed 50 mm/s. The vibrations measured on private structures, wells, etc. shall not exceed 25 mm/s. Those measured on utilities, if applicable, shall not exceed 10 mm/s.

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations at the various locations are within acceptable levels.

## **Records**

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring to the Contract Administrator as follows:

- a) The time/duration of each reading.
- b) Construction operations (e.g. installation of sheet piling) and timing of such relative to the readings.
- c) Details of exceedances and modifications to operations.
- d) Final report containing all relevant data including vibration monitoring and Pre- and Post-Construction Condition Surveys.

The results shall be submitted to the Contract Administrator after each pile / protection system element installation and operation of vibratory compaction equipment, prior to continuing with the subsequent piles, protection systems or compaction. As a minimum, the pile number (and set criteria / driving record) / temporary protection element and/or compaction location must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next pile(s), protection system element or area of vibratory compactive effort with subsequent readings taken as appropriate. The results of subsequent operations should be submitted to the Contract Administrator after each new pile / temporary protection system element / area of compaction has been completed.

If the readings are not within the limits stated above, the Contractor must alter the installation / construction procedures until the vibrations at the existing structures are within acceptable levels. The above process must be repeated for each pile, temporary protection element, or area/strip associated with embankment widening / grade raise requiring vibratory compactive effort.

## **Certificate of Conformance (CoC)**

Upon completion of the work in each area of pile driving, temporary protection system or use of vibratory equipment, the Contractor shall submit to the Contract Administrator a CoC sealed and signed by the Contractor's Engineer. The certificate shall state that the vibrations on the existing structures / utilities were below the limits stated above, and where the levels were exceeded, what procedures were used to reduce the vibrations to below the limits stated above.

## **Basis of Payment**

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material required to do the work.

## **END OF SECTION**



## **SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT – Item No.**

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### **Special Provision**

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## **1.0 GENERAL**

### **1.1 Scope**

This special provision describes requirements to supply and install monitoring instrumentation to verify the response of the foundation soils to the new embankment construction required as part of the replacement of the Highway 9 Holland Drainage Canal Overpass Bridge structure. This includes soil and embankment instrumentation to investigate settlements and pore pressures using the following geotechnical monitoring instrumentation:

- Settlement Plates (SP);
- Settlement Points (S); and,
- Multi-Level Vibrating Wire Piezometers (VWP).

This special provision also contains the requirements for the supply and installation of temporary survey Benchmarks (TBMs) related to the geotechnical monitoring instrumentation described herein and also for the Shape Accel Array (SAA) instrumentation described elsewhere in the Contract Documents.

### **1.2 Purpose**

The purpose of these instruments and equipment is to monitor the progress of settlement and pore pressure development and dissipation in the foundation soils under and adjacent to the widened fill embankments along Highway 9 at the Holland Drainage Canal Overpass. The purpose of the temporary survey benchmarks is to provide non-settling reference points for the surveying of the monitoring instruments.

The duration of the preloading/surcharging period prior to driving the steel H-piles at the abutments will be controlled by the instrumentation readings, as specified elsewhere in the Contract Documents. The completed, preloaded/surcharged embankments at the abutments shall remain undisturbed until such time as the monitoring indicates that a sufficient degree of consolidation of the foundation soil has been achieved.

### **1.3 Personnel**

The Contractor shall carry out the supply and installation of the geotechnical monitoring instrumentation (temporary benchmarks, settlement plates, settlement points, and multi-level vibrating wire piezometers) in accordance with this Special Provision and the Contract Drawings. The installation of the instruments and set-up for monitoring shall be carried out by a Foundation Engineering consultant (Contractor's Engineer) retained by the Contractor and registered in MTO's Consultant Registry, Appraisal and Qualifications System (RAQS) for "Geotechnical (Structures and Embankments) – Medium Complexity" or higher. All surveying shall be carried out by a qualified and registered surveyor retained by the Contractor.

## **2.0 REFERENCES**

### **2.1 General**

When the Contract Documents indicate that provincial oriented specifications are to be used and there is a provincial oriented specification of the same number as those listed below, references within this specification



to an OPSS shall be deemed to mean OPSS.PROV, unless use of a municipal oriented specification is specified in the Contract Documents. When there is not a corresponding provincial-oriented specification, the references below shall be considered to be the OPSS listed, unless use of a municipal oriented specification is specified in the Contract Documents.

This Special Provision refers to the following standards, specifications or publications:

#### **Ontario Provincial Standard Specifications, Construction**

OPSS 905      Steel Reinforcement for Concrete

#### **Ontario Provincial Standards Specifications, Material**

OPSS1010      Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS.PROV 1350 Concrete – Materials and Production

OPSS 1250      Clay Seal

OPSS 1301      Cementing Materials

OPSS 1801      Corrugated Steel Pile (CSP) Products

#### **Ontario Water Resources Act RRO 1990:**

Regulation 903 Wells

### **2.2      Subsurface Conditions**

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

Foundation Investigation Report - Highway 9 - Holland Drainage Canal Bridge (Site No. 37-31)  
Replacement, Schomberg, Ontario, G.W.P. 2266-18-00

### **3.0      DEFINITIONS**

**Automated Data Acquisition Monitoring System (ADAMS):** means a compatible system of multiplexers, data loggers, modem, solar panel and software that allows secure and remote (web based) monitoring of the instrumentation.

**Contractor** means the Contractor and his Foundation Engineering consultant (Contractor's Engineer).

**Contractor's Engineer (CE):** An engineering firm that is registered with MTO RAQS "Geotechnical (Structures and Embankments)" with a minimum of Medium Complexity. It is expected that the selected firm will have carried out at least four (4) similar instrumentation installation projects for the MTO within the last ten (10) years. The CE shall be retained by the Contractor to coordinate / support the supply and installation of the instrumentation, complete initial monitoring, and all associated reporting as outlined herein.



**Equal** shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

**Foundation Engineering Specialist (FES):** An engineering firm that is registered with MTO RAQs for high complexity. The FES shall be retained by the CA to ensure general conformance with the contract documents, to carry out the monitoring and interpretation of the instrumentation data, and shall issue certificate(s) of conformance.

**Monitoring Program** means the monitoring readings conducted by others as part of the Contract Administration Assignment.

**Settlement Plate** means a plate installed at the defined level with a series of rods attached to a plate for the purposes of settlement monitoring.

**Surface Settlement Points** means a reinforcing bar (or equivalent) embedded in concrete and installed at a defined location for the purpose of settlement monitoring.

**Temporary Survey Benchmark** means a non-yielding, deep-seated survey reference point.

**Multi-Level Vibrating Wire Piezometer** means a sensor attached to a cable installed in a borehole for the purposes of measuring pore pressure response.

## **4.0 SUBMISSION AND DESIGN REQUIREMENTS**

### **4.1 Submission Requirements**

#### **4.1.1 Notification**

The Contract Administrator shall be notified a minimum of fifteen (15) working days in advance of commencing the installation of instruments.

#### **4.1.2 Installation Methods**

The Contractor shall submit details of the proposed installation methods including locations and types of the data acquisition system(s), monitoring enclosure(s), temporary survey benchmarks and installation schedule, to the Contract Administrator, a minimum of fifteen (15) working days before the start of instrument installation.

## **5.0 MATERIALS**

### **5.1 Materials for Temporary Benchmarks (TBM)**

The Contractor shall supply all materials and equipment required for the installation of the benchmarks.

#### **5.1.1 Rod**

The Contractor shall supply a steel pipe, Schedule 40, with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 7.2.2.

The top end of each length of TBM rod shall be threaded to receive a cap or to allow for connection of successive lengths of rods. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.



### **5.1.2 Sand**

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

### **5.1.3 Grout**

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

### **5.1.4 Rod Anchor Grout**

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 14 kg of bentonite (OPSS 1205), 49 litres of water and 40 kg of cement (Type GU – OPSS 1301).

### **5.1.5 Friction-Reducing Sleeve**

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

## **5.2 Materials for Settlement Plate (SP)**

The Contractor shall supply all materials and equipment required for the installation of the Settlement Plates.

### **5.2.1 Plate**

The Contractor shall supply a steel plate with thickness of at least 6.35 mm. The plate shall be at least 0.5 m by 0.5 m in plan dimensions.

### **5.2.2 Rod**

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4 mm (1"), supplied in lengths as required to complete the installation.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

### **5.2.3 Friction Reducing Sleeve**

The Contractor shall supply a friction reducing sleeve consisting of Schedule 40 - 50.8 mm (2") O.D. PVC pipe cut perpendicular to the axis of the pipe.

### **5.2.4 Protective Surround**

The Contractor shall supply a protective surround for the portion of the rod within the embankment.

The surround shall consist of 300 mm diameter corrugated steel pipe (CSP - OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the Friction Reduction Sleeve (PVC pipe) shall be filled with medium to coarse sand.



### **5.3 Materials for Surface Settlement Points (S)**

The Contractor shall supply all materials and equipment required for the installation of the Surface Settlement Points, as follows:

#### **5.3.1 Reinforcing Bar**

The Contractor shall supply 25 mm diameter reinforcing bar with a minimum length of 400 mm. The top of reinforcing bar shall be such that a single survey point can be clearly identified and returned to including the use of a reflective device or paint, as may be required depending on the survey equipment used.

#### **5.3.2 Concrete**

The Contractor shall supply a suitable quickset concrete in which the reinforcing bar will be embedded. The concrete will be mixed according to the suppliers instructions and protecting (as appropriate) during curing.

### **5.4 Materials for Vibrating Wire Piezometer**

The Contractor shall supply all materials and equipment required for the installation of the Vibrating Wire Piezometer.

#### **5.4.1 Vibrating Wire Piezometer**

Vibrating wire piezometers shall be GEOKON Model 4500MLP, or approved equivalent. The VWP is to work within the temperature range of -20°C to 80°C.

All VWPs shall be of the same make/supplier. All VWPs shall be calibrated prior to installation and the calibration data for each piezometer shall be provided to the Contract Administrator.

#### **5.4.2 Signal Cable**

The Contractor shall supply GEOKON Model 02-250P4 cable, or approved equivalent. The length of cable for each piezometer shall be carefully estimated from the construction drawings to ensure that there is enough signal cable for each piezometer to provide enough slack in the borehole and along the trenches (or routes over the existing bridge) until each cable is out of the construction area and embankment footprint where they shall be protected from earthmoving equipment and extended to the monitoring station.

#### **5.4.3 Grout**

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consist of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU - OPSS 1301).

#### **5.4.4 Trench Burial and Conduit**

The signal cable for each VWP shall be buried in a shallow trench at the locations indicated in Table 1C and taken out of the embankment footprint area and to an area that will not be impacted by construction operations. The Contractor shall supply suitable conduits to protect the signal cables in the trenches and above ground surface (e.g. Schedule 40 – 75 mm - 3" - steel pipe or Schedule 80 – 75 mm – 3" – rigid PVC pipe). A minimum 300 mm protective surround consisting of OPSS.PROV 1010 Granular 'A' in accordance OPSS.PROV 1010



shall be placed around the conduit. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench.

The signal cables and conduits should be routed such that future grading works do not interfere with the cables or conduits.

#### **5.4.5 Data Acquisition System**

##### ***General***

The signal cables from the vibrating wire piezometers shall be connected to a GEOKON Model 8600-1 Datalogger, or approved equivalent. Each Datalogger shall be encased in a water and humidity proof enclosure and shall include protection against lightning damage and shall include control module, battery, battery charger, external power cable, USB-to-serial converter, VW interface and two (2) integral multiplexers for 20 VW sensors and 20 thermistors.

The multiplexer shall be GEOKON model 8032 or approved equivalent to allow for 16 VW sensors and 16 thermistors per multiplexer.

A minimum of two (2) dataloggers shall be installed. The contractor shall submit a detailed proposal on the setup of the datalogging system (i.e. number and location of the datalogging unit(s)) to the CA for review and approval, prior to ordering the data-logger(s).

##### ***Software***

LoggerNet and Vista Data Vision software, or approved equivalent, shall be provided to the CA at the completion of the installation.

##### ***Modem***

A cellular modem for TELUS, BELL, or ROGERS Mobility networks, DC or AC with Whip antenna. Activation Fee, SIM Card and 2-year Service, Support and Defective Replacement shall be included. A monthly cellular subscription shall be set up with MTO Foundations as the recipient.

##### ***Monitoring Shed***

The data logger(s) shall be installed in a walk-in monitoring shed to prevent vandalism and minimize exposure of the data-logger(s) to extreme weather conditions. The Monitoring Shed shall be lockable, tamper resistant, and weather resistant. The Monitoring Shed shall be seated on a gravel pad and securely tied down to ground. The location of the Monitoring Shed shall not be susceptible to ground settlement.

The location of the monitoring shed shall be chosen by the Contractor but must be approved by the CA. The location shall be chosen to allow for access from Highway 9. The Contractor shall always ensure access to the monitoring shed, including but not limited to snow clearing in the winter.

##### ***Solar Panel***

A 30-watt solar panel with side-of-pole mounts, a 5 m interconnect cable and a charge controller for an external battery shall be included with the monitoring shed. It is expected that the instrumentation shall be monitored for up to 2 years.

The data-loggers shall be programmed according to the following:

- Recording Software: VWP data shall be recorded six (6) times a day (i.e. one (1) reading every 4 hours); and,



- Test Software: once this program is transferred to the data-logger, the system shall be able to be tested to confirm readings can be gathered manually at the site and remotely by use of the cellular network.

The real-time data shall be retrieved remotely by cellular network. The Contractor shall be responsible for obtaining the cellular plan to allow for retrieval of the data by the cellular network for the duration of the construction.

## **6.0 EQUIPMENT**

### **6.1 Monitoring Equipment Operation and Weather Conditions**

All monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. The instruments shall be capable of operating within the manufacturer's stated accuracy throughout the temperature range. Monitoring will be conducted potentially year-round by the Contract Administrator.

## **7.0 CONSTRUCTION**

### **7.1 Monitoring Instrument Installations**

#### **7.1.1 Drawings**

Reference shall be made to the following drawings that are contained elsewhere in the Contract Documents:

- Monitoring Instrumentation Plans;
- Typical Monitoring Sections; and
- Typical Instrument Installation Details.

#### **7.1.2 Quantities and Locations of Instruments**

The quantities and approximate locations of instruments are presented in Tables 1A, 1B, 2, 3 and 4 and are shown on the Contract Drawings. The final locations shall be "field fit" by the Contractor to take account of any utilities that may be present, construction operations, and safe access conditions.

**Table 1A – Instrument Quantities and Locations**

<b>Monitoring Section Station</b>	<b>Monitoring Section Type</b>	<b>Quantities</b>		
		<b>SP</b>	<b>S</b>	<b>VWP<sup>1</sup></b>
STA 21+749	Type C	2	--	1
STA 21+771.5	Type B	2	2	1
STA 21+789	Type A	3	2	1
STA 21+816.5		3	2	1
STA 21+819	Type D	1	2	1
STA 21+862		1	2	1
STA 21+864.5	Type A	3	2	1
STA 21+892		3	2	1
STA 21+909.5	Type B	2	2	1
STA 21+932	Type C	2	--	1
<b>TOTAL:</b>		<b>22</b>	<b>16</b>	<b>10</b>



Note(s): 1. VWPs are multi-level VWPs with 4 VWP tips per installation location.

### **7.1.3 Materials and Equipment**

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless otherwise noted.

### **7.1.4 Instrument Location**

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground elevation at each instrument location.

### **7.1.5 Underground Utilities**

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to underground utilities caused by the Contractor's work shall be repaired by the Contractor at no cost to the Owner or Contract Administrator.

### **7.1.6 Marking and Labelling**

The location of any above-ground monitoring fixture shall be made clearly visible to nearby traffic before, during and after embankment construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls, if and where applicable.

Instruments shall be clearly labelled in the field, with each instrument having a unique identifier. The labelling shall remain legible for the entire duration of monitoring.

### **7.1.7 Protection of Instruments**

The Contractor shall adequately protect all instruments such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced by the Contractor at no cost to the Owner or Contract Administrator.

### **7.1.8 Survey Personnel**

Surveying to establish the benchmarks and other elevations shall be carried out by a registered surveyor with appropriate equipment. The surveyor shall be retained by the Contractor.

### **7.1.9 Accuracy of Surveying for Elevations**

Elevations shall be surveyed to an accuracy of  $\pm 2$  mm or better.

### **7.1.10 Boreholes**

The Contractor shall make a basic stratigraphic log of boreholes as they are being drilled for the installation of monitoring instruments. In-situ or laboratory geotechnical testing is not required.

Boreholes shall be advanced using conventional drilling methods, where applicable, and shall be as straight and vertical as practicable.



### 7.1.11 Installation Program

The instruments shall be installed prior to the commencement of the embankment construction. Table 1B gives a summary of the installation schedule requirements.

**Table 1B – Instrument Installation Program**

<b>Instrument Type</b>	<b>Instrument Location</b>	<b>Start Installation</b>	<b>Finish Installation</b>
SP	At top of drainage blanket or top of backfill following organics removal, prior to embankment and/or surcharge construction. Rods extended to above fill surface during embankment and/or surcharge construction.	After placement of fill for drainage blanket construction and/or placement of fill following removal of organics. Prior to placement of fill for embankment and surcharge construction.	At completion of surcharge embankment construction.
S	At ground surface / top of embankment or surcharge fill.	At completion of embankment and/or surcharge fill construction.	At completion of embankment and/or surcharge construction.
VWP	Within embankment widening sections.	After placement of fill for drainage blanket construction and/or placement of fill following removal of organics.	Prior to placement of fill for embankment and surcharge construction.

## 7.2 Temporary Benchmark Installation

### 7.2.1 General Procedure

The benchmarks shall be installed in boreholes and anchored into the compact to dense silt and sand stratum located approximately 10 m to 15 m below ground surface.

### 7.2.2 Number and Locations

The minimum number and approximate locations of the benchmarks are to be determined by the Contractor and the Contractor's Engineer in conjunction with the Contract Administrator, the Foundation Monitoring Consultant, and Surveyors. For bidding purposes assume that 4 benchmarks are required: all benchmarks are to be anchored at approximately 10 m to 15 m depth and within the compact to dense silt and sand stratum above about Elevation 210 m. The number and locations of benchmarks shall be determined in the field to satisfy the following conditions:

- Direct sighting is possible from all instruments to at least one benchmark.
- Each benchmark is located in an area that will not experience a change in loading (due to grade raise or excavation) that could induce settlement or heave in the ground in which the Benchmark is installed (i.e. non-settling benchmark).
- Each benchmark is located in such a way to minimize interference with and damage by construction activities.



- The rod anchor elevation shall be adjusted in the field to extend approximately 1 m into soils having Standard Penetration Test 'N' values of greater than 25 blows per 0.3 m of penetration. Reference shall be made to the Foundation Investigation Reports for information in order to determine the anchor elevation for each Benchmark location selected.

Intermediate tie-in points may be required as deemed necessary by the surveyor, and shall be tied into the temporary benchmarks during each reading.

### **7.2.3 Installation**

The Contractor shall install benchmarks in accordance with the following:

### **7.2.4 Borehole**

The borehole shall be advanced to rod anchor elevations controlled by the Standard Penetration Test "N" values given above, using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction-reducing sleeve and rod anchor. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

### **7.2.5 Rod**

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

### **7.2.6 Rod Anchor**

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole to form a concrete/soil anchor.

Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 0.5 m length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

### **7.2.7 Friction-Reducing Sleeve**

The friction-reducing sleeve shall be installed over the entire length of the rod above the rod anchor and sand, extending up to ground surface.

### **7.2.8 Installation Details**

The elevation, easting and northing of the top of the Benchmark rod shall be surveyed.

The total distance from the rod anchor to the top of the rod shall be measured and recorded by the Contractor to an accuracy of  $\pm 2$  mm or better.



The Contractor is responsible for preventing damage to the benchmarks during the embankment construction. If a benchmark is damaged during fill placement, the rods, rod anchor, and friction-reducing sleeve shall be replaced before resuming the fill placement.

### 7.3 Settlement Plate (SP) Installation

#### 7.3.1 General Procedure

The base plate of the settlement plate shall be installed on the top of the granular backfill / drainage blanket following removal and replacement of the organic deposit. As embankment construction proceeds, the rods shall be extended above the new top of embankment fill and surcharge fill. Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate.

#### 7.3.2 Number and Location

The Contractor shall install SPs at the locations shown on the Contract Drawings and given in Table 2. The instrument locations should be field fit to avoid the Contractor's operations, but to be as close to the intended locations as practicable.

**Table 2 – Settlement Plate Locations**

Monitoring Location	Offset from New Highway 9 Centreline (m)		
	Left	Right	Right of Right
STA 21+816.5 STA 21+864.5	3.5	5.75	8
STA 21+789 STA 21+892	3.5	5.25	7
STA 21+771.5 STA 21+909.5	5	N/A	7.5
STA 21+749 STA 21+932	6	N/A	9
STA 21+819 STA 21+862	N/A	-17	N/A
<b>TOTAL SPs: 11</b>	4	3	4

The Contractor shall install SPs as shown on the Contract Drawings and the Typical Installation Detail, in addition to what is stated below.

#### 7.3.3 Plate

The settlement plate shall be installed horizontally on the top of the granular backfill / drainage blanket following removal and replacement of the organic deposit. If a SP is located in an area where no organic soils are present and therefore not removed and replaced with granular backfill, the settlement plate shall be installed horizontally on the undisturbed native (non-organic) soil just below the existing ground surface.

#### 7.3.4 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.



### 7.3.5 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod that is below ground and within the embankment fill and surcharge fill except that the cap on top of the SP rod shall extend 25 mm above the top of the friction sleeve at all times.

### 7.3.6 Extension of Rod

The SP rods shall be extended upwards as the embankment widening and surcharge is constructed so that the top of the rod is always at least 0.3 m, but not more than 2 m above the surrounding fill.

### 7.3.7 Protective Surround

The CSP, friction-reducing sleeve and sand surround shall be extended concurrent with the rods, where applicable. The SP rod shall be in the centre of the CSP and friction-reducing sleeve. The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the sleeve.

## 7.4 Surface Settlement Point (S) Installation

### 7.4.1 General Procedure

The surface settlement points shall be installed on the top of the embankment / surcharge fill after completion of filling.

### 7.4.2 Number and Locations

The Contractor shall install Surface Settlement Points (S) at the locations shown on the Contract Drawings and given in Table 3. The instrument locations should be field fit to avoid the Contractor's operations, but to be as close to the intended locations as practicable.

**Table 3 – Settlement Point Locations**

<b>Monitoring Location</b>	<b>Offset from Highway 9 Centreline (m)</b>	
STA 21+816.5 STA 21+864.5	2.75	8.75
STA 21+789 STA 21+892	2.75	7.75
STA 21+771.5 STA 21+909.5	4.25	8
STA 21+819 STA 21+862	-16.25	-17.75
<b>TOTAL Ss: 8</b>	4	4

### 7.4.3 Reinforcing Bar:

The reinforcing bar, embedded in concrete, should generally be installed on the top of the embankment fill near the crests following the placement of the preload or surcharge fill. The reinforcing bar shall be embedded



in quickset concrete that has been placed in an excavation up to about 500 mm deep within the preload or surcharge fill and setback about 0.5 m from the fill crest as shown in the Contract Drawings.

## 7.5 Multi-Level Vibrating Wire Piezometer (VWP) Installation

### 7.5.1 General Procedure

The VWPs shall be installed in boreholes after removal and replacement of the organic deposit(s) and following placement of the drainage blanket, and prior to construction of the preload or surcharge embankment.

### 7.5.2 Number and Locations

The locations of the Multi-Level VWPs are shown on the Contract Documents and in Table 4. The Multi-Level VWPs shall be installed at the tip elevations shown on the Contract Drawings and in Table 4. Installation of the VWPs shall be as per the manufacturer's recommendations in addition to what is stated or emphasised below.

The VWP signal cables shall be extended to the data-logger enclosure area (monitoring shed) through a steel or rigid PVC conduit buried in trenches with protective surround, as specified in Section 5.4.4. The final location of the enclosure (monitoring shed) should be determined on-site prior to ordering instruments to ensure there is sufficient cable length(s).

**Table 4 – Multi-Level Vibrating Wire Piezometer Locations and Elevations**

Monitoring Locations	Offset from Highway 9 Centreline (m)	Tip Elevation (m)
STA 21+814.5 STA 21+866.5	5.75	206.5
		199
		194
		182.5
STA 21+787 STA 21+894	5.25	206.5
		199
		194
		182.5
STA 21+769.5 STA 21+911.5 STA 21+751 STA 21+934	7.25	206.5
		199
		194
		185
STA 21+817 STA 21+860	-17	206.5
		199
		194
		182.5
Total Multi-Level VWPs:4; Total VWP Tips: 16		

Note(s): 1. VWPs are multi-level VWPs with 4 VWP tips per installation location.



### **7.5.3 Borehole Installation**

The borehole at each VWP location shall be advanced to 300 mm below the lowest tip elevation using suitable drilling techniques. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris. A split-spoon sample shall be taken at the proposed installation depth(s) to confirm the soil stratum at the VWP tip elevation(s). The borehole shall be filled with water prior to installation of the VWP tip(s).

### **7.5.4 Protective Enclosures for Data Loggers**

The data-logger(s) shall be installed in a protective enclosure (monitoring shed) to prevent vandalism and prolonged wear-out of the data-loggers against extreme weather. The protective enclosure (monitoring shed) shall be lockable and weather proofed. The Contractor shall submit a detailed proposal of the protective enclosure (i.e. materials and location(s) etc.) to the Contract Administrator for review, prior to construction/installation.

The Contractor shall ensure access to the protective enclosure (monitoring shed) at all times, including but not limited to snow clearing in the winter.

### **7.5.5 Completion of Installation**

It is known that the process of installing VWPs can temporarily alter the pore water pressure acting on the piezometer tip. The installation of a VWP shall not be considered to be complete until the pore pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. The Contractor shall take daily reading of the pore pressures, for the period noted below until the value has stabilized as determined by the Contract Administrator. Stabilization shall be deemed to have occurred:

- When no change in the measured value has occurred over a period of five (5) consecutive days and the measured value is within 10 per cent of the anticipated hydrostatic value; and,
- When the daily rate of change is less than four (4) kPa per day for three (3) consecutive days and the measured value is within 5 per cent of the anticipated hydrostatic value.

The Contractor should be prepared to wait for a period of 10 days to 15 days after completion of installation of the instruments for the baseline readings to stabilize.

## **7.6 Monitoring Program**

### **7.6.1 Notification**

The Contractor shall notify the Contract Administrator no later than three (3) working days after the completion of installation of Benchmarks, Settlement Plates, Settlement Points, and Vibrating Wire Piezometers.

### **7.6.2 Reporting**

The Contractor shall supply the information outlined in the following sections to the Contract Administrator within three (3) days of completion of installation of each instrument.

#### **7.6.2.1 Temporary Survey Benchmarks**

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:



- TBM Northing and Easting in MTM NAD 83 coordinates;
- Elevation of the rod anchor bottom, rod anchor length, and top of rod in Geodetic datum;
- Date of installation;
- Stratigraphic log of subsurface conditions at the TBMs, including notes on drilling method obstructions it encountered;
- Installation notes/sketches; and,
- Description of TBM (rod), sleeves and rod anchors.

#### **7.6.2.2 Settlement Plates and Settlement Points**

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- SP and S Northing and Easting in MTM NAD 83 coordinates;
- Elevation of base of plate and top of rod and top of reinforcing bar in Geodetic datum;
- Date of installation;
- Installation notes/sketches; and,
- Description of SP rods, sleeves and plates.

Adjustments in the length of any SP rod shall be coordinated with the Contract Administrator to allow surveying by others of the elevation of the top of the rod immediately before and immediately after adjustment. This surveying is necessary to accurately track the settlement data.

#### **7.6.2.3 Vibrating Wire Piezometers**

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- VWP Northings and Eastings in MTM NAD 83 coordinates;
- Elevations of VW sensors (tips) in Geodetic datum;
- Dates of installation;
- Stratigraphic log of subsurface conditions, including drilling method notes;
- Installation notes / sketches;
- Model, make and serial numbers of VW sensors, readout unit and signal cable; and,
- Calibration details of VW sensors.

### **7.6.3 Monitoring**

#### **7.6.3.1 Automated Data Acquisition System Commissioning**

This section contains the requirements for the supply and installation of a data management and web-based interface for monitoring the vibrating wire piezometers.

The CE shall provide the project management and labour necessary to commission the ADAMS. The data management and web-based interface shall be installed prior to construction.

The data management and web-based system shall include files of reduced data in real-time and shall store all monitoring data. It shall provide graphs and tables in a format acceptable to the FES.



The CE shall install the material as described above and in accordance with the manufacturer's recommendations and maintain for the duration of the monitoring period of 10 years.

The web-based interface shall be secure, and password protected and ensure the data is automatically backed-up daily.

The CA and FES shall have uninterrupted and continuous access to the web-based interface during construction. The CE shall provide the programming and setting up data transfer to client FTP site

#### **7.6.3.2 Hand Over to CA and FES**

The Contractor shall meet with the Contract Administrator and FES (responsible for the ongoing monitoring) immediately after installation of the instruments and before the start of embankment construction. At this meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments, and all equipment to be supplied by the Contractor, as identified in the special provision.

Monitoring by the Contract Administrator's representative (or FES) for the baseline readings shall commence within seven working days after the hand-over meeting. The monitoring shall continue on a schedule to be determined by the Contract Administrator throughout the construction of the embankments, and for up to approximately 8 months following the completion of construction to the preload/surcharge grade.

#### **7.6.4 Decommissioning of Instruments**

At the end of the monitoring period, the Contractor shall decommission all the temporary survey benchmarks and tie-in points by removing the rod and friction-reducing sleeve to at least 1.5 m below grade by excavating and backfilling with compacted granular fill in accordance with the specifications for fill placement.

At the end of the monitoring period, the Contractor shall decommission all Settlement Plates and Settlement Points, unless otherwise advised by the Contract Administrator. The Vibrating Wire Piezometers shall remain in place and in operation. Decommissioning of instrumentation shall be carried out according to the Ontario Water Resources Act, Regulation 903 (as amended).

### **8.0 QUALITY ASSURANCE – Not Used**

### **9.0 MEASUREMENT FOR PAYMENT – Not Used**

## **10.0 BASIS OF PAYMENT**

### **10.1 Supply and Installation of Embankment Monitoring Equipment - Item**

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work, including the supply, installation and decommissioning of survey benchmarks, Settlement Plates, Settlement Points and Multi-Level Vibrating Wire Piezometers, as well as performing all required monitoring and reporting.



**SUPPLY AND INSTALLATION OF DOWNDRAG MONITORING EQUIPMENT – Item No .**

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Special Provision

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**1.0 GENERAL**

**1.1 Scope**

This special provision describes requirements to supply and install monitoring instrumentation for a verification of downdrag loading on the piles at the Highway 9 Holland Drainage Canal Overpass Bridge site. This includes soil and pile instrumentation to investigate the magnitude, the distribution, and time dependency of downdrag forces on piles installed at the site as discussed below.

**1.2 Purpose**

The downdrag test program includes the supply, installation, and monitoring of one (1) instrumented production pile (HP 360x132), one (1) multi-level vibrating wire piezometer, one (1) multi-point pile extensometer, one (1) multi-point borehole extensometer, one (1) thermistor, and installation of a remote monitoring station. Details for the instrumentation, including location, installation and monitoring frequency, shall be as specified below.

Dynamic analysis using a Pile Driving Analyzer (PDA) is required on the one (1) instrumented production pile, within the EBL east embankment/abutment area. The PDA testing procedure shall be as specified below.

**1.3 Personnel**

The Contractor shall carry out the supply and installation of the instrumented pile and installation of the associated monitoring instruments in accordance with this Special Provision and the Contract Drawings. The installation of the instruments and set-up for monitoring shall be carried out by a Foundation Engineering consultant (Contractor's Engineer) retained by the Contractor and registered in MTO's Consultant Registry, Appraisal and Qualifications System (RAQS) for "Geotechnical (Structures and Embankments) – Medium Complexity" or higher. All surveying shall be carried out by a qualified and registered surveyor retained by the Contractor.

**2.0 REFERENCES**

This special provision refers to the following standards, specifications and/or publications. Where specifications and reference documents conflict, the MTO Designer shall make the final determination of the applicable document.

**ASTM Standards**

D4945 Standard Test Method for High-Strain Dynamic Testing for Deep Foundations



## **Ontario Provincial Standard Specifications, Construction**

OPSS 206	Grading
OPSS 501	Compaction
OPSS 903	Construction Specification for Deep Foundations
OPSS 1010	Material Specification for Regular Granular
OPSS 1205	Material Specification for Clay Seal
OPSS 1301	Material Specification for Cementing Materials
OPSS 1801	Material Specification for Corrugated Steel Pipe

## **Others**

Canadian Foundation Engineering Manual, 2006

### **3.0 DEFINITIONS**

**Automated Data Acquisition Monitoring System (ADAMS):** means a compatible system of multiplexers, data loggers, modem, solar panel and software that allows secure and remote (web based) monitoring of the instrumentation.

**Contract Administrator (CA):** means the contractor administrator for this project.

**Contractor:** means the General Contractor for the construction of the project.

**Contractor's Engineer (CE):** An engineering firm that is registered with MTO RAQS "Geotechnical (Structures and Embankments)" with a minimum of Medium Complexity. It is expected that the selected firm will have carried out at least four (4) similar instrumentation installation projects for the MTO within the last ten (10) years. The CE shall be retained by the Contractor to coordinate / support the supply and installation of the instrumentation, complete initial monitoring, and all associated reporting as outlined herein.

**Foundation Engineering Specialist (FES):** An engineering firm that is registered with MTO RAQS for high complexity. The FES shall be retained by the CA to ensure general conformance with the contract documents, to carry out the monitoring and interpretation of the instrumentation data, to conduct the PDA testing, and shall issue certificate(s) of conformance.

**Instrumented Production Pile:** means a production pile (i.e., a pile installed at a structure as part of that structure's foundation) fitted with instrumentation to assess down drag.

**Multi-Point Borehole Extensometer:** means a series of displacement transducers installed in a sampled borehole for the purposes of measuring settlement corresponding to depth.

**Multi-Point Pile Extensometer:** means a series of displacement transducers installed along the pile for the purposes of measuring compression and/or elongation along the pile.

**Or approved equivalent:** The term "or approved equivalent" shall be understood to indicate that the equivalent product is the same or better than the specified product in function, performance, reliability, quality and configuration.

**Thermistor:** means a sensor attached to a cable to assess temperature.

**Multi-Level Vibrating Wire Piezometer (VWP):** means a sensor attached to a cable installed in



a borehole for the purposes of measuring pore pressure responses.

**Vibrating Wire Strain Gauges (VWGS):** means a sensor attached to a cable welded to the pile for the purpose of measuring strain along the pile.

#### **4.0 DESIGN AND SUBMISSION REQUIREMENTS**

##### **4.1 General**

The Contractor shall submit the following shop drawings and documents to the CA within two (2) weeks of the approval of the critical path schedule for review and approval by the design team:

- a) Access and Site Preparation Plan, including proposed locations of conduit, and monitoring shed location;
- b) H-pile installation procedures, and type and capacity of hammer for pile driving and PDA testing;
- c) Proposed installation details for pile instrumentation including protection of the gauges, cables, and splicing details;
- d) Setup details for PDA Testing, hammer warming pile, and type and capacity of striking hammer or alternative method for warming up hammer prior to PDA testing.
- e) Information on the instrumentation proposed for the site including make and model of the instrument and detailed installation instructions.

At the end of each month, the Contractor shall provide the CA details on the activities on the site including but not limited to:

- a) The amount, and location of fill placed on site each day;
- b) The number, type, and location of the instrumentation installed each day;
- c) The number, and length of piles installed each day, and;
- d) Any other construction activities including any heavy machinery on site.

All submissions related to the downdrag study test shall bear the seal and signature of the CE.

Installation reports for all instruments installed on site shall be submitted within two (2) weeks of installation. The reports must include, but are not limited to the following:

- Date of installation;
- Installation notes and sketches;
- Northing, easting, and elevation of the instrument, if appropriate;
- Calibration details, if appropriate;
- Stratigraphic log of subsurface condition



- Passwords and any applicable software and licenses necessary to read all files and formats and to view all graphs and tables of the web-based interface;
- Model, make, and serial numbers of all cables, sensors, and readout units, if applicable;

#### **4.2 PDA Testing**

The FES shall submit a PDA test report to the CA as per the ASTM D4945 for each PDA test within forty-eight (48) hours of the completion of testing.

#### **4.3 Instrumentation**

The Contractor shall retain the CE to develop the instrumentation installation plan, oversee the installation of the instrumentation, and take initial baseline readings.

The Contractor and CE shall prepare an instrumentation installation plan submittal and provide it to the CA, FES, and MTO Designer for approval 10 Working Days prior to installation.

The instrumentation installation submittal shall include details of the proposed installation methods for the instrumentation described herein, including but not limited to, how to protect instrumentation during welding, installation, and construction, details of instrumentation if a substitution is proposed, details on cable splicing and pile welding, details of the loading frame, and proposed layout of the cable conduits and monitoring shed.

The CA shall be notified a minimum of ten (10) Working Days in advance of commencing the installation of any instrumentation.

The Contractor shall record and submit relevant installation details to the CA. These include, but are not limited to:

- a) Instrument location, easting, northing;
- b) Elevation of top of surface point or top of instrument;
- c) Distance between bottom of hole/pile and top of instrument;
- d) As-built layout of cable trenches and conduit;
- e) Dates of installation and datum/baseline readings;
- f) Installation notes/sketches;
- g) Grout mix for instrumentation installation, as required;
- h) Description and photographs of instrument;
- i) Calibration records;
- j) Manufacturers' documentation, including instrument specifications and software manuals; and,
- k) Web log-in credentials.



The Contractor shall record and submit the instrumentation readings during installation and the baseline readings to the CA within twenty-four (24) hours of completion of each survey/reading. Each report shall include all survey data collected in tabular and graphical format as plots of time versus settlement in comparison to survey data collected prior to commencement of the work.

## **5.0 INSTRUMENTATION MATERIALS**

### **5.1 General**

The Contractor shall supply all the material and equipment required to install the instrumentation described herein.

The term “or approved equivalent” shall be understood to indicate the equivalent product is the same or better than the specified product in function, performance, reliability, quality, and configuration. Any equivalents are required to be approved by the FES and CA prior to installation.

All vibrating wire instrumentation including strain gauges, extensometers, and piezometers, shall be provided by the same manufacturer and shall all be compatible with a single data acquisition system/software.

All instrumentation is to be calibrated prior to installation. The calibration records and instrumentation documents, including make, model, and serial numbers are to be provided to the CA five (5) Working Days prior to installation.

### **5.2 Vibrating Wire Strain Gauges (VWSG)**

#### *General*

Vibrating wire strain gauges shall be GEOKON Model 4150 arc weldable strain gauge, or approved equivalent. The gauge is to include plucking coil, hose clamp, thermistor, and mounting blocks.

The strain gauges shall have a range of 3000 $\mu\epsilon$ , resolution of 1 $\mu\epsilon$ , and shall work in a temperature range of -20°C to 80°C.

#### *Signal Cable*

The Contractor shall supply GEOKON Model 02-250T cable, or approved equivalent. The length of cable for each strain gauge shall be carefully estimated from the Contract Drawings to ensure that there is enough signal cable for each strain gauge to provide enough slack along the pile and along the trenches until each cable is out of the construction area where they shall be protected from earthmoving equipment.

#### *Trench Burial and Conduit*

The signal cable from each instrumented pile shall be buried in a shallow trench as shown in the Contract Drawings and taken out of the construction area. The Contractor shall supply suitable conduits (e.g. Schedule 40 - 75mm - 3" - steel pipe or Schedule 80 - 75mm - 3" - rigid PVC pipe) to protect the signal cables in the trenches and above ground surface. Several signal cables may be housed in a single conduit and laid in a common trench. A locator strip shall be installed 0.3 m above the conduit to allow for easier location of the cables in the future.

The signal cables and conduits should be routed such that future grading works do not interfere with the cables or conduits.



### **5.3 Vibrating Wire Piezometer (VWP)**

#### *General*

Vibrating wire piezometers shall be GEOKON Model 4500MLP, or approved equivalent. The VWP is to work within the temperature range of -20°C to 80°C.

VWPs shall be of the same make/supplier. All VWPs shall be calibrated prior to installation and the calibration data for each piezometer shall be provided to the Contract Administrator.

#### *Signal Cable*

The Contractor shall supply GEOKON Model 02-250P4 cable, or approved equivalent. The length of cable for each piezometer shall be carefully estimated from the Contract Drawings to ensure that there is enough signal cable for each piezometer to provide enough slack in the borehole and along the trenches (or routes over the existing bridge) until each cable is out of the construction area where they shall be protected from earthmoving equipment.

#### *Grout*

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 23 kg of bentonite (OPSS 1205), 143 liters of water and 40 kg of cement (Type 10 - OPSS 1301).

#### *Trench Burial and Conduit*

The signal cable for each piezometer shall be buried in a shallow trench as shown in the Contract Drawings and taken out of the construction area. The Contractor shall supply suitable conduits (e.g. Schedule 40 - 75mm - 3" - steel pipe or Schedule 80 - 75mm - 3" - rigid PVC pipe) to protect the signal cables in the trenches and above ground surface. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench. A locator strip shall be installed 0.3 m above the conduit to allow for easier location of the cables in the future.

The signal cables and conduits should be routed such that future grading works do not interfere with the cables or conduits.

### **5.4 Multi-Point Borehole Extensometer (EXT)**

#### *General*

Multi-point borehole extensometers (EXT) for measuring the compression of the soil shall be GEOKON Model A-6 with groutable anchors or approved equivalents. The EXT shall have a range of 100mm.

The EXT shall be fitted with a vibrating wire displacement transducer (GEOKON model 4450 or approved equivalent) to allow for reading with the selected data acquisition system.

#### *Measurement Rod*

The Contractor shall supply a steel rod encased in PVC with a diameter of 6.35mm (1/4"), supplied in lengths as required to complete the installation. The rods are to be flush coupled to form a continuous string.

#### *Signal Cable*

The Contractor shall supply GEOKON Model 02-250P4 cable or approved equivalent. The length



of cable for each piezometer shall be carefully estimated from the Contract Drawings to ensure that there is enough signal cable for each instrument to provide **enough slack in the borehole and along the trenches** (or routes over the existing bridge) until each cable is out of the construction area where they shall be protected from earthmoving equipment.

#### *Trench Burial and Conduit*

The signal cable for each extensometer shall be buried in a shallow trench as shown in the Contract Drawings and taken out of the construction area. The Contractor shall supply suitable conduits (e.g. Schedule 40 - 75mm - 3" - steel pipe or Schedule 80 - 75mm - 3" - rigid PVC pipe) to protect the signal cables in the trenches and above ground surface. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench. A locator strip shall be installed 0.3 m above the conduit to allow for easier location of the cables in the future.

The signal cables and conduits should be routed such that future grading works do not interfere with the cables or conduits.

### **5.5 Multi-Point Pile Extensometer (MPPE)**

#### *Pile Extensometer*

Multiple point extensometers for measuring the compression of the pile shall be GEOKON Model A-5 with hydraulic anchors, or approved equivalents. The MPPE shall have a range of 25 mm.

The MPPE shall be fitted with a vibrating wire displacement transducer (GEOKON model 4450 or approved equivalent) to allow for reading with the selected data acquisition system.

#### *Measurement Rod*

The Contractor shall supply a steel rod encased in PVC with a diameter of 6.35mm (1/4"), supplied in lengths as required to complete the installation. The rods are to be flush coupled to form a continuous string.

#### *Signal Cable*

The Contractor shall supply GEOKON Model 02-250P4 cable, or approved equivalent. The length of cable for each piezometer shall be carefully estimated from the Contract Drawings to ensure that there is enough signal cable for each instrument to provide enough slack in the along the trenches (or routes over the existing bridge) until each cable is out of the construction area where they shall be protected from earthmoving equipment.

#### *Trench Burial and Conduit*

The signal cable for each extensometer shall be buried in a shallow trench as shown in the Contract Drawings and taken out of the construction area. The Contractor shall supply suitable conduits (e.g. Schedule 40 - 75mm - 3" - steel pipe or Schedule 80 - 75mm - 3" - rigid PVC pipe) to protect the signal cables in the trenches and above ground surface. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench. A locator strip shall be installed 0.3 m above the conduit to allow for easier location of the cables in the future.

The signal cables and conduits should be routed such that future grading works do not interfere with the cables or conduits.



## **5.6 Thermistor**

### *General*

Vibrating wire thermistor shall be GEOKON Model 4700MLP, or approved equivalent.

### *Signal Cable*

The Contractor shall supply GEOKON Model 02-250P4 cable, or approved equivalent. The length of cable for the thermistor shall be carefully estimated from the Contract Drawings to ensure that there is enough signal cable.

## **5.7 Handheld Readout**

The Contractor shall supply a new handheld readout for the vibrating wire (VW) instrumentation. The handheld readout shall be GEOKON Model GK-404, or approved equivalent. Following the successful demonstration of operation, the handheld readout, along with any related manuals, shall be provided to the CA at the completion of the instrumentation and data acquisition system installation.

## **5.8 Data Acquisition System**

### *General*

The signal cables from the vibrating wire instrumentation including the strain gauges, piezometers, and extensometers shall be connected to a GEOKON Model 8600-1 Datalogger, or approved equivalent. Each Datalogger shall be encased in a water and humidity proof enclosure and shall include protection against lightning damage and shall include control module, battery, battery charger, external power cable, USB-to-serial converter, VW interface and two (2) integral multiplexers for 19 VW sensors and 19 thermistors.

The multiplexer shall be GEOKON model 8032 or approved equivalent to allow for 16 VW sensors and 16 thermistors per multiplexer.

A minimum of one (1) datalogger shall be installed. The contractor shall submit a detailed proposal on the setup of the datalogging system (i.e. number and location of the datalogging unit(s)) to the CA for review and approval, prior to ordering the data-logger(s).

### *Software*

LoggerNet and Vista Data Vision software, or approved equivalent, shall be provided to the CA at the completion of the installation.

### *Modem*

A cellular modem for TELUS, BELL, or ROGERS Mobility networks, DC or AC with Whip antenna. Activation Fee, SIM Card and 5-year Service, Support and Defective Replacement shall be included. A monthly cellular subscription shall be set up with MTO Foundations as the recipient.

### *Monitoring Shed*

The data logger shall be installed in a walk-in monitoring shed to prevent vandalism and minimize exposure of the data-logger(s) to extreme weather conditions. The Monitoring Shed shall be lockable, tamper resistant, and weather resistant. The Monitoring Shed shall be seated on a gravel pad and securely tied down to ground. The location of the Monitoring Shed shall not be susceptible to ground settlement.



The location of the monitoring shed shall be chosen by the Contractor but must be approved by the CA. The location shall be chosen to allow for access from Highway 9. The Contractor shall always ensure access to the monitoring shed, including but not limited to snow clearing in the winter.

#### *Solar Panel*

A 30-watt solar panel with side-of-pole mounts, a 5 m interconnect cable and a charge controller for an external battery shall be included with the monitoring shed. It is expected that the instrumentation shall be monitored for up to 10 years.

## **6.0 INSTRUMENTATION INSTALLATION**

### **6.1 Instrumentation Number and Location**

#### *Vibrating Wire Strain Gauges*

Vibrating wire strain gauges shall be installed along one (1) instrumented production pile on the east abutment of the HWY 9 Holland Drainage Canal bridge. The vibrating wire strain gauges shall be installed on flanges as shown on the Contract Drawings.

A total of thirty-eight (38) strain gauges shall be installed on the instrumented production pile with the spacing based on elevation and as shown on the Contract Drawings:

- Strain gauges every 2 m from Elevation 217.6 m to 191 m;
- Strain gauges every 5 m from Elevation 190 m to 175 m.

The location of the instrumented production pile is shown on the Contract Drawings.

#### *Multi-Level Vibrating Wire Piezometer*

One (1) multi-level vibrating wire piezometer shall be installed on site as shown on the Contract Drawings. The vibrating wire tip elevations shall be as follows:

**Table 1 - Vibrating Wire Piezometer Location and Elevations**

VWP	Chainage (m)	Offset South from Centerline (m)	VWP Tip Elevation (m)
1	21+854	2.3	208
			203
			198
			189
			181.5
			172

#### *Multi-point Pile Extensometer*

One (1) extensometer shall be attached to the instrumented production pile located two piles south of the new Highway 9 centerline to measure the compression of the pile itself. An anchor shall be placed at the following depths: pile tip, 2/3 pile tip, 1/3 pile tip, and surface.



#### *Multi-Point Borehole Extensometer*

One (1) multi-point borehole extensometer shall be installed in conjunction with the instrumented pile as shown in the table below and on the Contract Drawings.

**Table 2 - Extensometer Anchor Location and Elevations**

<b>EXT</b>	<b>Chainage (m)</b>	<b>Offset South from Centerline (m)</b>	<b>Anchor Elevation (m)</b>
1	21+854	3.3	215
			210
			205
			201
			194
			189
			184
			179
			171

#### *Thermistor*

The thermistor shall be mounted to the solar panel pole of the ADAMS. It shall be installed in such a way to be protected from vandalism and extreme weather conditions.

### **6.2 Vibrating Wire Strain Gauges**

Installation on the instrumented production pile located two piles south of the new Highway 9 centerline shall be as per manufacturer's specification in addition to what is stated in this special provision.

Vibrating wire strain gauges shall be installed at the elevations stated in herein and attached to the flanges of the pile complete with the vibrating wire cables. The vibrating wire stain gauges shall be protected with angle or channel welded to the pile as indicated on the Contract Drawings.

The vibrating wire strain gauges shall be attached to mounting blocks or equivalent to avoid damage that can be caused by arc welding the gauges directly to the steel H pile. The mounting blocks shall be correctly spaced. The strain gauges shall be appropriately positioned and attached to the mounting blocks.

The vibrating wire stain gauges shall be connected to an automated data acquisition system. The automated data acquisition system shall consist of a datalogger, multiplexer, a cellular modem as outlined herein. A solar panel shall be provided to provide power to the system.

The appropriate sequence of arc welding shall be undertaken to avoid damage to the gauges. The cabling shall be spliced as per manufacture specifications. A reading from each instrument must be taken before and directly after splicing to confirm the splice is adequate. Readings shall also be taken as the pile is lifted into position for driving.

Gauges shall be protected from corrosion, direct sunlight and the heat of welding. They shall also be protected from mechanical damage.

### **6.3 Vibrating Wire Piezometer**



Prior to the installation of any piles, the VWP's shall be installed in accordance with the manufacturer's recommendations in addition to what is stated or emphasized below.

The borehole shall be advanced to 300 mm below the tip elevation using suitable drilling techniques. The sides of the borehole shall be stable, and the borehole shall be free of drilling mud and debris.

The piezometer sensor shall be saturated, per the manufacturer's recommendations. In addition, the borehole shall be filled with water upon installation of the sensor into the base of the hole to maintain saturation of the sensor throughout the installation process.

It is known that the process of installing VWPs can temporarily alter the porewater pressure acting on the piezometer tip. The installation of a VWP shall not be complete until the porewater pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. The CE shall take daily readings of the porewater pressure at each VWP until the value has stabilized. Stabilization shall be deemed to have occurred as follows:

- When no change in the measured value has occurred over a period of five (5) consecutive days and the measured value is within 10 percent of the anticipated hydrostatic value; and,
- When the daily rate of change is less than four (4) kPa per day for three (3) consecutive days and the measured value is within 5 percent of the anticipated hydrostatic value.

#### **6.4 Multi- Point Borehole Extensometers**

Installation shall be as per manufacturer's specification. The extensometer should be installed prior to the placement of embankment.

#### **6.5 Multi- Point Pile Extensometers**

Installation shall be as per manufacturer's specification and shall be installed within a metal pipe which is to be welded to the instrumented production pile prior to pile installation. The extensometer shall be held in place with inflatable anchors located at the tip and cut off elevation of the pile.

#### **6.6 Pile Installation**

The Contractor shall use an appropriate low-energy pile driving hammer for installing the pile. Swinging leads will not be permitted as per Clause 903.06.03.

The HP 360x132 instrumented production pile shall be installed at the same time as the rest of the production piles and shall be driven to achieve the capacities indicated on the Contract Drawings.

**If less than 80 percent of the strain gauges survive the pile driving, the Contractor shall remove and reinstall the instrumented piles at no additional cost.**

#### **6.7 PDA Testing**

PDA tests will be carried out on the instrumented pile by the FES and as shown on the Contract Drawings. The PDA tests will be conducted as per ASTM D4945. The Contractor shall coordinate with the FES to conduct the PDA testing.

The Contractor shall notify the CA and FES a minimum of ten (10) working days before each proposed round of testing.



## **7.0 MONITORING PROGRAM**

A portion of the monitoring readings are required to be taken during the construction and at key stages during the installation of the piling. In this regard, the Contractor shall notify the CA and FES a minimum of ten (10) working days before each stage of construction as it relates to the monitoring described below. The CE shall work with the FES to ensure a smooth transition from installation and commissioning of the instrumentation to recording of the data during the key stages of construction and pile installation.

### **7.1 Automated Data Acquisition Monitoring System Commissioning**

This section contains the requirements for the supply and installation of a data management and web-based interface for monitoring the vibrating wire strain gauges, vibrating wire piezometers, multi-point pile extensometers, multi-point borehole soil extensometers, and thermistors.

The CE shall provide the project management and labour necessary to commission the ADAMS. The data management and web-based interface shall be installed prior to construction.

The data management and web-based system shall include files of reduced data in real-time and shall store all monitoring data. It shall provide graphs and tables in a format acceptable to the FES.

CE shall install the material as described above and in accordance with the manufacturer's recommendations and maintain for the duration of the monitoring period of 10 years.

The web-based interface shall be secure, and password protected and ensure the data is automatically backed-up daily.

The CA and FES shall have uninterrupted and continuous access to the web-based interface during construction. The CE shall provide the programming and setting up data transfer to client FTP site.

### **7.2 Vibrating Wire Strain Gauges**

The vibrating wire strain gauges shall be attached to the datalogger and set up for permanent web-based and temporary on-site monitoring and shall be monitored with the following frequency:

Automated readings shall be taken every minute during piling. The automated reading frequency shall be set-up by the CE; the readings shall be monitored by the FES.

A series of baseline readings shall be taken within 12 hours of the completion of piling and of the completion of re-striking by the FES.

Automated readings shall be taken daily for the first 12 months after piling and shall be monitored by the FES.

Automated readings shall be taken monthly from 12 months to 10 years and shall be monitored by MTO or FES.

### **7.3 Vibrating Wire Piezometers**

The VWP's shall be attached to the datalogger and set up for permanent web-based and temporary on-site monitoring and shall be monitored with the following frequency:



Daily automated readings of the porewater pressure and temperature at each VWP until the value has stabilized, with readings to be taken by the CE and provided to the CA and FES prior to start of embankment construction/backfill in the abutment area.

Automated daily readings shall be taken for the first six months following the beginning of embankment construction/backfill in the abutment area and shall be monitored by the FES.

Automated weekly readings shall be taken from 6 months to one year and shall be monitored by the FES.

Automated monthly readings shall be taken from one year to 10 years and shall be monitored by MTO or FES.

#### **7.4 Multi-Point Pile Extensometers**

The pile extensometers shall be attached to the datalogger and set up for permanent web-based and temporary on-site monitoring and shall be monitored with the following frequency:

A series of baseline readings shall be taken within 12 hours of the completion or piling and of the completion of re-striking by the FES.

Automated daily readings shall be taken for the first six months and shall be monitored by the FES.

Automated weekly readings shall be taken from 6 months to one year and shall be monitored by the FES.

Automated monthly readings shall be taken from one year to 10 years and shall be monitored by MTO or FES.

#### **7.5 Multi-Point Borehole Soil Extensometers**

The multi-point Borehole extensometers shall be attached to the datalogger and set up for permanent web-based and temporary on-site monitoring and shall be monitored with the following frequency:

Automated daily readings shall be taken for the first six months and shall be monitored by the FES.

Automated weekly readings shall be taken from 6 months to one year and shall be monitored by the FES.

Automated monthly readings shall be taken from one year to 10 years and shall be monitored by MTO or FES.

#### **7.6 Thermistor**

The thermistor shall be attached to the datalogger and set up for permanent web-based and temporary on-site monitoring. The thermistor shall be monitored with the following frequency:

Automated daily readings shall be taken for the first six months following the beginning of embankment construction and shall be monitored by the FES.



Automated weekly readings shall be taken from 6 months to one year and shall be monitored by the FES.

Automated monthly readings shall be taken from one year to 10 years and shall be monitored by MTO or FES.

#### **7.7 Hand Over to CA and FES**

The Contractor shall meet with the Contract Administrator and FES (responsible for the ongoing monitoring) immediately after installation of the instruments. At this meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments, and all equipment to be supplied by the Contractor, as identified in the special provision.

Monitoring by the Contract Administrator's representative (or FES) shall commence immediately after the hand-over meeting. The monitoring shall continue on a schedule to be determined by the Contract Administrator throughout the remainder of the construction, and for up to approximately 10 years following the completion of construction.

#### **8.0 BASIS OF PAYMENT**

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

**END OF SECTION**



**Special Provision**

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**1.0 GENERAL**

**1.1 Scope**

This Special Provision describes the requirements for the supply and installation of a Shape Accelerometer Array (SAA) to monitor the performance of the new embankment construction required as part of the replacement of the Highway 9 Holland Drainage Canal Overpass Bridge structure.

This special provision includes the requirement for the installation of a temporary benchmark (TBM) at one end of the SAA to act as a fixed reference point for the purposes of monitoring. The details of the requirements for the materials and installation of the TBM are as described elsewhere in the Contract Documents.

**1.2 Purpose**

The purpose of the SAA is to monitor the long-term, post-construction embankment settlement across a cross-section of the new Highway 9 embankment. The data from the SAA will provide a detailed profile of the magnitude and shape of the differential settlements across the embankment.

**1.3 Personnel**

The Contractor shall carry out the supply and installation of the SAA in accordance with this Special Provision and the Contract Drawings. The installation of the instrument and set-up for monitoring shall be carried out by a Foundations Engineering Consultant (Contractor's Engineer) retained by the Contractor and registered in MTO's Consultancy Registry, Appraisal and Qualifications System (RAQS) for "Geotechnical (Structures and Embankments) – Medium Complexity", or higher. All surveying shall be carried out by a qualified and registered surveyor retained by the Contractor.

**2.0 REFERENCES**

**2.1 General**

When the Contract Documents indicate that provincial oriented specifications are to be used and there is a provincial oriented specification of the same number as those listed below, references within this specification to an OPSS shall be deemed to mean OPSS.PROV, unless use of a municipal oriented specification is specified in the Contract Documents. When there is not a corresponding provincial-oriented specification, the references below shall be considered to be the OPSS listed, unless use of a municipal oriented specification is specified in the Contract Documents.

This Special Provision refers to the following standards, specifications or publications:

**Ontario Provincial Standard Specifications, Construction**

OPSS 905 Steel Reinforcement for Concrete

**Ontario Provincial Standards Specifications, Material**

OPSS1010 Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS.PROV 1350 Concrete – Materials and Production



OPSS 1250 Clay Seal

OPSS 1301 Cementing Materials

OPSS 1801 Corrugated Steel Pile (CSP) Products

## 2.2 Subsurface Conditions

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

Foundation Investigation Report - Highway 9 - Holland Drainage Canal Bridge (Site No. 37-31) Replacement, Schomberg, Ontario, G.W.P. 2266-18-00

## 2.3 Drawings

Reference shall be made to the Contract Drawings with specific reference to the following:

Contract No. 2020-2015, GWP No. 266-18-00, Holland Drainage Canal Site 37-31 – Differential Settlement Monitoring

## 3.0 DEFINITIONS

**Automated Data Acquisition Monitoring System (ADAMS):** means a compatible system of multiplexers, data loggers, modem, solar panel and software that allows secure and remote (web based) monitoring of the instrumentation.

**Contractor** means the Contractor and his Foundation Engineering consultant (Contractor's Engineer).

**Contractor's Engineer (CE):** An engineering firm that is registered with MTO RAQS "Geotechnical (Structures and Embankments)" with a minimum of Medium Complexity. It is expected that the selected firm will have carried out at least four (4) similar instrumentation installation projects for the MTO within the last ten (10) years. The CE shall be retained by the Contractor to coordinate / support the supply and installation of the instrumentation, complete initial monitoring, and all associated reporting as outlined herein.

**Foundation Engineering Specialist (FES):** An engineering firm that is registered with MTO RAQs for high complexity. The FES shall be retained by the CA to ensure general conformance with the contract documents, to carry out the monitoring and interpretation of the instrumentation data, and shall issue certificate(s) of conformance.

**Monitoring Program** means the monitoring readings conducted by others as part of the Contract Administration Assignment.

**Or approved equivalent:** The term "or approved equivalent" shall be understood to indicate that the equivalent product is the same or better than the specified product in function, performance, reliability, quality and configuration.

**Shape Accel Array (SAA)** means an automated shape-measuring, inclinometer-style instrument which is comprised of an array of rigid segments, separated by joints that move in any direction and containing MEMS accelerometers (gravity sensors) to measure tilt in three-dimensions when installed vertically and in two-dimensions when installed horizontally.

**Temporary Survey Benchmark** means a non-yielding, deep-seated survey reference point.



## **4.0 SUBMISSION AND DESIGN REQUIREMENTS**

### **4.1 Submission Requirements**

#### **4.1.1 Notification**

The Contract Administrator shall be notified a minimum of fifteen (15) working days in advance of commencing the installation of the instrument(s).

#### **4.1.2 Installation Methods**

The Contractor shall submit details of the proposed installation methods including locations and types of the data acquisition system(s), monitoring enclosure(s), temporary survey benchmark and installation schedule, to the Contract Administrator, a minimum of fifteen (15) working days before the start of instrument installation.

## **5.0 MATERIALS**

### **5.1 General**

The Contractor shall supply all materials and equipment required for the installation of the SAA.

### **5.2 Shape Accelerometer Array Equipment**

The Contractor shall supply Shape Accelerometer Arrays of Field Array type (i.e. SAAV) with all its components as manufactured by Measurand Inc., Fredericton, New Brunswick, Canada or approved equivalent. The SAA shall have segment lengths of 500 mm and shall be constructed to the lengths shown on the Contract Drawings. The total length of each SAA shall be measured and recorded to a minimum accuracy of 5 mm. Adjustments in the length of any SAA shall be coordinated with the Contract Administrator.

### **5.3 Conduit**

The Contractor shall supply PVC conduit with an inner diameter of 27 mm and outer diameter of 32 mm, into which the SAA and the communication cable will be inserted. The Contractor shall supply PVC conduit for the entire length of the SAAs and communication cables as shown on the Contract Drawings.

### **5.4 Protective Surround for Connections**

The Contractor shall supply protective surround consisting of 300 mm diameter Corrugated Steel Pipe (CSP) in accordance with OPSS 1801 with the ends cut perpendicular to the axis of the SAA and free of burrs and sharp edges. The CSP is to provide access point to maintain the connections at both ends of the SAA.

### **5.5 Data Acquisition System**

All requirements for the data acquisition system for the SAA including, but not limited to a data logger, cabling, software, modem, monitoring shed, solar panel, and cellular plan shall be as per the ADAMS system(s) as described elsewhere in the Contract Documents.

The Contractor shall supply a data logger and interface unit (as required) that are fully compatible with the SAA instrument. The data-logger shall be programmed according to the following:

- Recording Software: SAA data shall be recorded twice (2) times a day (i.e. one (1) reading every 12 hours); and,
- Test Software: once this program is transferred to the data-logger, the system shall be able to be tested to confirm readings can be gathered manually at the site and remotely by use of the cellular network.



The real-time data shall be retrieved remotely by cellular network. The Contractor shall be responsible for obtaining the cellular plan to allow for retrieval of the data by the cellular network for the duration of the monitoring. It is expected that the instrumentation shall be monitored for up to 10 years.

## **6.0 INSTALLATION**

### **6.1 General Procedure**

The SAA shall be installed according to the manufacturer's installation instructions. The SAA shall be attached to a stable survey monument (Temporary Benchmark) at one end and shall be attached to communication cables connecting to the Automated Data Acquisition Monitoring System (ADAMS) being installed as part of the instrumentation as described elsewhere in the Contract Documents.

### **6.2 Number and Location**

A total of one (1) SAA shall be installed. One SAA shall be installed in each specified area as shown on the Contract Drawings and presented in Table 1.

**Table 1 – Number and Location of SAA**

<b>Location</b>	<b>SAA No.</b>	<b>Station</b>	<b>Quantity</b>	<b>Elevation of SAA (m)</b>	<b>Approximate Length of SAA (m)</b>
Highway 9, 6 m east of east abutment (oriented N/S) from an offset of -22 m from CL to 17 m from CL	SAA-1	21+858	1	219.4	39

### **6.3 Instrument Operation and Weather Conditions**

The SAA and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. The SAAs and associated materials shall be capable of operating within the manufacturer's stated accuracy throughout the temperature range. Monitoring shall be conducted by the FES or MTO year-round.

### **6.4 Lightning Protection for Shape Accelerometer Arrays**

The SAA shall be separately protected from lightning strikes by the use of a copper grounding rod and cable.

### **6.5 Access to Instruments**

The Contractor shall provide access to the SAA to the Contract Administrator and the FES for use and quality assurance purposes.

### **6.6 Installation Details**

The Contractor shall install the SAA in accordance with the manufacturer's recommended procedure in addition to what is stated or emphasized in this special provision.



- a. The PVC conduit shall be assembled in a flat area using PVC cement suitable for the temperature and weather conditions of the installation location;
- b. The SAA reel shall be placed on a reel stand with a minimum height of 0.62 m such that the SAA can be pulled from the bottom of the reel;
- c. The SAA shall be pulled into the PVC conduit using a rope (or a cable) with a swivel attachment;
- d. The X-marks shown on the SAA shall be marked onto both ends of the PVC conduit. The X marks on the PVC conduit will be verified to ensure that the PVC conduit is not twisted;
- e. The end cap shall be glued onto the bottom end of the PVC conduit, at the eyebolt end of the SAA;
- f. The Cross-linked Polyethylene (PEX) at the communication cable end of the SAA shall be secured to the PVC conduit using the set screw assembly provided in the SAA installation kit;
- g. A geotextile lined trench shall be provided such that PVC conduit is bedded in sand with a minimum thickness of 150 mm;
- h. The PVC conduit shall be surrounded on the exposed side and covered with sand of a minimum thickness of 150 mm;
- i. The geotextile shall be wrapped over the top of the bedding;
- j. The CSP protective surround shall be placed at both ends of the PVC conduit to provide access points to maintain the connections at both ends of the SAA;
- k. The communication cables protected by the PVC conduit and coming from the SAA location shall be extended to the ADAMS monitoring shed;
- l. The communication cables shall be protected by a PVC conduit extending from the SAA to the monitoring shed. The conduit shall be protected against damage by burying into a trench at depth of at least 0.5 m and backfilled with a Granular A material;
- m. The reference end of the SAA shall be attached to a temporary benchmark (TBM). The temporary benchmark shall be installed to Elevation 171.0 m at an offset of -22.0 m from the centerline (north of centerline) in accordance with the requirements described elsewhere in the Contract Documents. The Contractor shall protect the temporary benchmark from any disturbance due to backfilling behind the monument. The temporary benchmark shall be protected by a layer of Granular A material similar to the SAA conduit protection (an arrangement shown on the Contract Drawings) and may be adjusted in field in cooperation with the FES, MTO Staff and the Contract Administrator);
- n. The PVC conduit shall be installed in two separate stages. The portion of the conduit to be placed within the widened portions of the embankment shall be installed during Stage 1 of Construction. The portion of the conduit to be placed beneath the grade raise portion of embankment on the existing roadway (i.e. below the cellular concrete) shall be installed during Stage 2 of Construction. The Contractor is responsible for maintaining the integrity of the PVC conduit through the entirety of the embankment construction. The Contractor shall ensure that the different sections of the conduit (i.e. from the Stage 1 and Stage 2 installation) are properly aligned, connected and secured so at the completion of the new embankment construction, a single, continuous PVC conduit, free from obstruction, is installed below the width of the final embankment.
- o. The SAA will be installed within the PVC conduit and connected to the ADAMS as soon as possible after completion of the new embankment section at and adjacent to the location of the SAA at Station 21+858.

## **6.7 Instrument Location**

Prior to the installation of the SAA, the Contractor shall accurately survey and stake/paint the location of the completed PVC conduit and obtain an elevation at each end of the conduit and at the top of the temporary benchmark (TBM) at the reference end of the SAA. Elevations shall be surveyed to accuracy of 1 mm.

## **6.8 Instrument Program**

The SAA installation within the PVC conduit shall be carried out as soon as possible after the embankment construction at Station 21+858.



## **6.9 Marking and Labelling**

The location of any above ground monitoring fixture shall be made clearly visible before, during and after embankment construction. The SAA and its cable(s) shall be clearly labelled in the field. The labelling shall remain legible for a minimum period of 5 years.

## **6.10 Protection of Instruments**

The Contractor shall adequately protect the SAA and its associated materials/cabling from damage during the installation and during the remainder of the construction. Any damaged instrument shall be immediately replaced at the Contractor's cost.

## **7.0 MONITORING PROGRAM**

Monitoring of the SAA is to be carried out by the FES or MTO and shall be done remotely through the ADAMS. Monitoring shall be conducted after completion of the embankment construction at Station 21+858 and during the remaining construction (i.e. adjacent embankments, pavement structure). The Contractor shall provide installation information as specified in Section 7.1 and provide access to the ends of SAA for equipment maintenance if required and snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed for checking the instrument.

### **7.1 Notification**

The Contractor shall notify the Contract Administrator no later than three (3) working days after the completion of installation of TMB and SAA.

### **7.2 Reporting**

The Contractor shall record and report relevant installation details to the Contract Administrator within three (3) days of completion of the SAA. These include, but are not limited to:

- Northings and eastings of both ends of SAA and survey monument in MTM NAD 83 coordinates;
- Total length of each SAA to a minimum accuracy of 5 mm;
- Elevation of SAA and survey monument referenced to Geodetic datum;
- Calibration and instrument license;
- Dates of installation; and,
- Installation notes and sketches.

### **7.3 Monitoring**

#### **7.3.1 Automated Data Acquisition System Commissioning**

This section contains the requirements for the supply and installation of a data management and web-based interface for monitoring the SAA.

The CE shall provide the project management and labour necessary to commission the ADAMS. The data management and web-based interface shall be installed prior to construction.

The data management and web-based system shall include files of reduced data in real-time and shall store all monitoring data. It shall provide graphs and tables in a format acceptable to the FES.

The CE shall install the material as described above and in accordance with the manufacturer's recommendations and maintain for the duration of the monitoring period of 10 years.

The web-based interface shall be secure, and password protected and ensure the data is automatically backed-up daily.



The CA and FES shall have uninterrupted and continuous access to the web-based interface during construction. The CE shall provide the programming and setting up data transfer to client FTP site

### **7.3.2 Hand Over to CA and FES**

The Contractor shall meet with the Contract Administrator and FES (responsible for the ongoing monitoring) immediately after installation of the instruments. At this meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments, and all equipment to be supplied by the Contractor, as identified in the special provision.

Monitoring by the Contract Administrator's representative (or FES) shall commence immediately after the hand-over meeting. The monitoring shall continue on a schedule to be determined by the Contract Administrator throughout the remainder of the construction, and for up to approximately 10 years following the completion of construction.

## **7.0 PAYMENT**

### **7.1 Measurement for Payment**

Measurement shall be for each SAA.

### **7.2 Basis of Payment**

Payment at the Lump Sum price for this tender shall be full compensation for all labour, equipment and material to do the work.



## **CSP FOR INTEGRAL ABUTMENT - Item No.**

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### **Special Provision**

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#### **1.0 SCOPE**

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

All as shown on the Contract drawings and/or as directed by the Contract Administrator on site.

#### **2.0 REFERENCES – Not Used**

#### **3.0 DEFINITIONS – Not Used**

#### **4.0 DESIGN AND SUBMISSION REQUIREMENTS**

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

#### **5.0 MATERIALS**

##### **5.0.1 Corrugated steel pipe**

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.



Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

### 5.0.2 Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

**Table 1 – Sand Fill Gradation Requirements**

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 µm	#30	80% - 100%
425 µm	#40	40% - 80%
250 µm	#60	5% - 25%
150 µm	#100	0% - 6%

### 6.0 EQUIPMENT – Not Used

### 7.0 CONSTRUCTION

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Construct levelling pad and place CSPs and spacers.
2. Install piles by driving to design criteria.
3. Place loose sand into 600 diameter CSP.
4. Remove temporary spacers.
5. Place 19mm plywood on top of filled CSP pipes to separate top of CSPs from the abutment wet concrete prior to concrete pouring. Plywood shall be cut to fit steel piles.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.



The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

**8.0            QUALITY ASSURANCE – Not Used**

**9.0            MEASUREMENT FOR PAYMENT – Not Used**

**10.0          BASIS OF PAYMENT**

Payment at the Contract price of the above tender item shall include all necessary labour, material and equipment required to do the work.



## APPENDIX G

# P-Y Curves



SUMMARY OF P-y CURVES FOR A H-Pile 310x132 at Abutments																																				
Description Depth (z) * Elevation P-y Curves	Very Loose to Compact Silt and Sandy to Silty Sand Fill						Soft to Firm Clayey Silt						Compact to Dense Silt and Sand to Sand						Stiff Clayey Silt						Firm Clayey Silt to Silty Clay											
	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 4.0 m		z= 5.0 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m	
	Elev. 217.5 m		Elev. 217. m		Elev. 216.5 m		Elev. 216. m		Elev. 215.5 m		Elev. 215. m		Elev. 214. m		Elev. 213. m		Elev. 212. m		Elev. 211. m		Elev. 210. m		Elev. 209. m		Elev. 208. m		Elev. 207. m		Elev. 206. m		Elev. 205. m		Elev. 204. m		Elev. 203. m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.002	2.645	0.002	4.188	0.002	8.417	0.000	4.420	0.000	5.178	0.000	5.936	0.001	73.731	0.001	117.309	0.001	170.942	0.001	200.061	0.000	12.019	0.000	11.268	0.000	10.517	0.000	9.766	0.000	9.014	0.000	8.075	0.000	8.075	0.000	8.075
	0.004	5.099	0.003	8.073	0.004	16.225	0.000	8.840	0.000	10.356	0.000	11.872	0.002	142.120	0.002	226.119	0.003	329.500	0.003	385.628	0.000	24.038	0.000	22.536	0.000	21.034	0.000	19.531	0.000	18.029	0.000	16.151	0.000	16.151	0.000	16.151
	0.006	7.221	0.005	11.433	0.007	22.978	0.001	13.260	0.001	15.534	0.001	17.808	0.003	201.270	0.004	320.230	0.004	466.636	0.004	546.125	0.000	36.058	0.000	33.804	0.000	31.550	0.000	29.297	0.000	27.043	0.001	24.226	0.001	24.226	0.001	24.226
	0.008	8.948	0.007	14.168	0.009	28.474	0.001	17.680	0.001	20.712	0.001	23.744	0.004	249.412	0.005	396.826	0.006	578.252	0.006	676.754	0.001	48.077	0.001	45.072	0.001	42.067	0.001	39.062	0.001	36.058	0.001	32.302	0.001	32.302	0.001	32.302
	0.010	10.286	0.008	16.286	0.011	32.730	0.002	22.100	0.002	25.890	0.002	29.680	0.005	286.694	0.006	456.143	0.007	664.689	0.007	777.915	0.002	60.096	0.002	56.340	0.002	52.584	0.002	48.828	0.002	45.072	0.002	40.377	0.002	40.377	0.002	40.377
0.012	11.282	0.010	17.864	0.013	35.901	0.004	26.520	0.004	31.068	0.004	35.616	0.006	314.468	0.007	500.334	0.009	729.083	0.009	853.278	0.003	72.115	0.003	67.608	0.003	63.101	0.003	58.594	0.003	54.086	0.004	48.452	0.004	48.452	0.004	48.452	
0.014	12.004	0.011	19.005	0.015	38.196	0.006	30.940	0.006	36.246	0.006	41.552	0.007	334.568	0.008	532.314	0.010	775.685	0.010	907.818	0.006	84.134	0.006	78.876	0.006	73.618	0.006	68.359	0.006	63.101	0.006	56.528	0.006	56.528	0.006	56.528	
0.016	12.515	0.013	19.815	0.017	39.822	0.009	35.360	0.009	41.424	0.009	47.488	0.007	348.812	0.010	554.976	0.012	808.707	0.012	946.466	0.010	96.154	0.010	90.144	0.010	84.134	0.010	78.125	0.010	72.115	0.009	64.603	0.009	64.603	0.009	64.603	
0.019	12.871	0.015	20.379	0.020	40.957	0.014	39.780	0.014	46.602	0.014	53.424	0.008	358.755	0.011	570.795	0.013	831.759	0.013	973.445	0.016	108.173	0.016	101.412	0.016	94.651	0.016	87.890	0.016	81.130	0.014	72.679	0.014	72.679	0.014	72.679	
0.021	13.118	0.016	20.769	0.022	41.741	0.019	44.200	0.019	51.780	0.019	59.360	0.009	365.623	0.012	581.724	0.014	847.684	0.014	992.082	0.025	120.192	0.025	112.680	0.025	105.168	0.025	97.656	0.025	90.144	0.019	80.754	0.019	80.754	0.019	80.754	
0.023	13.287	0.018	21.037	0.024	42.279	0.025	48.620	0.025	56.958	0.025	65.296	0.010	370.334	0.013	589.218	0.016	858.605	0.016	1004.863	0.036	132.211	0.036	123.948	0.036	115.685	0.036	107.422	0.036	99.158	0.025	88.829	0.025	88.829	0.025	88.829	
0.025	13.402	0.020	21.220	0.026	42.646	0.032	53.040	0.032	62.136	0.032	71.232	0.011	373.548	0.014	594.333	0.017	866.058	0.017	1013.586	0.051	144.230	0.051	135.216	0.051	126.202	0.051	117.187	0.051	108.173	0.032	96.905	0.032	96.905	0.032	96.905	
0.027	13.481	0.021	21.344	0.028	42.895	0.041	57.459	0.041	67.314	0.041	77.168	0.012	375.734	0.015	597.811	0.019	871.126	0.019	1019.517	0.071	156.250	0.071	146.484	0.071	136.718	0.071	126.953	0.071	117.187	0.041	104.980	0.041	104.980	0.041	104.980	
0.029	13.534	0.023	21.428	0.031	43.065	0.051	61.879	0.051	72.491	0.051	83.104	0.013	377.218	0.017	600.171	0.020	874.565	0.020	1023.542	0.095	168.269	0.095	157.752	0.095	147.235	0.095	136.718	0.095	126.202	0.051	113.056	0.051	113.056	0.051	113.056	
0.031	13.570	0.024	21.485	0.033	43.179	0.063	66.299	0.063	77.669	0.063	89.040	0.014	378.223	0.018	601.770	0.022	876.895	0.022	1026.269	0.125	180.288	0.125	169.020	0.125	157.752	0.125	146.484	0.125	135.216	0.063	121.131	0.063	121.131	0.063	121.131	
0.033	13.594	0.026	21.524	0.035	43.257	0.067	66.299	0.067	77.669	0.067	89.040	0.015	378.903	0.019	602.852	0.023	878.472	0.023	1028.114	0.157	180.288	0.157	169.020	0.157	157.752	0.157	146.484	0.157	135.216	0.067	121.131	0.067	121.131	0.067	121.131	
Description Depth (z) * Elevation P-y Curves	Firm Silty Clay to Clayey Silt						Stiff Clayey Silt to Silty Clay																													
	z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m		z= 22.0 m																									
	Elev. 202.0 m		Elev. 201.0 m		Elev. 200.0 m		Elev. 199.0 m		Elev. 198.0 m		Elev. 196.0 m																									
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)																								
	0.000	0.000	0.000	0.00	0.000	0.00	0.000	0.000	0.000	0.000	0.000	0.000																								
	0.000	8.075	0.000	8.08	0.000	8.08	0.000	8.075	0.000	12.144	0.000	11.143																								
	0.000	16.151	0.000	16.15	0.000	16.15	0.000	16.151	0.000	24.289	0.000	22.286																								
	0.001	24.226	0.001	24.23	0.001	24.23	0.001	24.226	0.000	36.433	0.000	33.428																								
	0.001	32.302	0.001	32.30	0.001	32.30	0.001	32.302	0.001	48.578	0.001	44.571																								
	0.002	40.377	0.002	40.38	0.002	40.38	0.002	40.377	0.002	60.722	0.002	55.714																								
0.004	48.452	0.004	48.45	0.004	48.45	0.004	48.452	0.003	72.866	0.003	66.857																									
0.006	56.528	0.006	56.53	0.006	56.53	0.006	56.528	0.006	85.011	0.006	78.000																									
0.009	64.603	0.009	64.60	0.009	64.60	0.009	64.603	0.010	97.155	0.010	89.142																									
0.014	72.679	0.014	72.68	0.014	72.68	0.014	72.679	0.016	109.300	0.016	100.285																									
0.019	80.754	0.019	80.75	0.019	80.75	0.019	80.754	0.025	121.444	0.025	111.428																									
0.025	88.829	0.025	88.83	0.025	88.83	0.025	88.829	0.036	133.588	0.036	122.571																									
0.032	96.905	0.032	96.90	0.032	96.90	0.032	96.905	0.051	145.733	0.051	133.714																									
0.041	104.980	0.041	104.98	0.041	104.98	0.041	104.980	0.071	157.877	0.071	144.856																									
0.051	113.056	0.051	113.06	0.051	113.06	0.051	113.056	0.095	170.022	0.095	155.999																									
0.063	121.131	0.063	121.13	0.063	121.13	0.063	121.131	0.125	182.166	0.125	167.142																									
0.067	121.131	0.067	121.13	0.067	121.13	0.067	121.131	0.157	182.166	0.157	167.142																									



P-y CURVES

1671430 WO 0017 Holland Drainage Canal Bridge Site 37-31 for 310x132 Pile

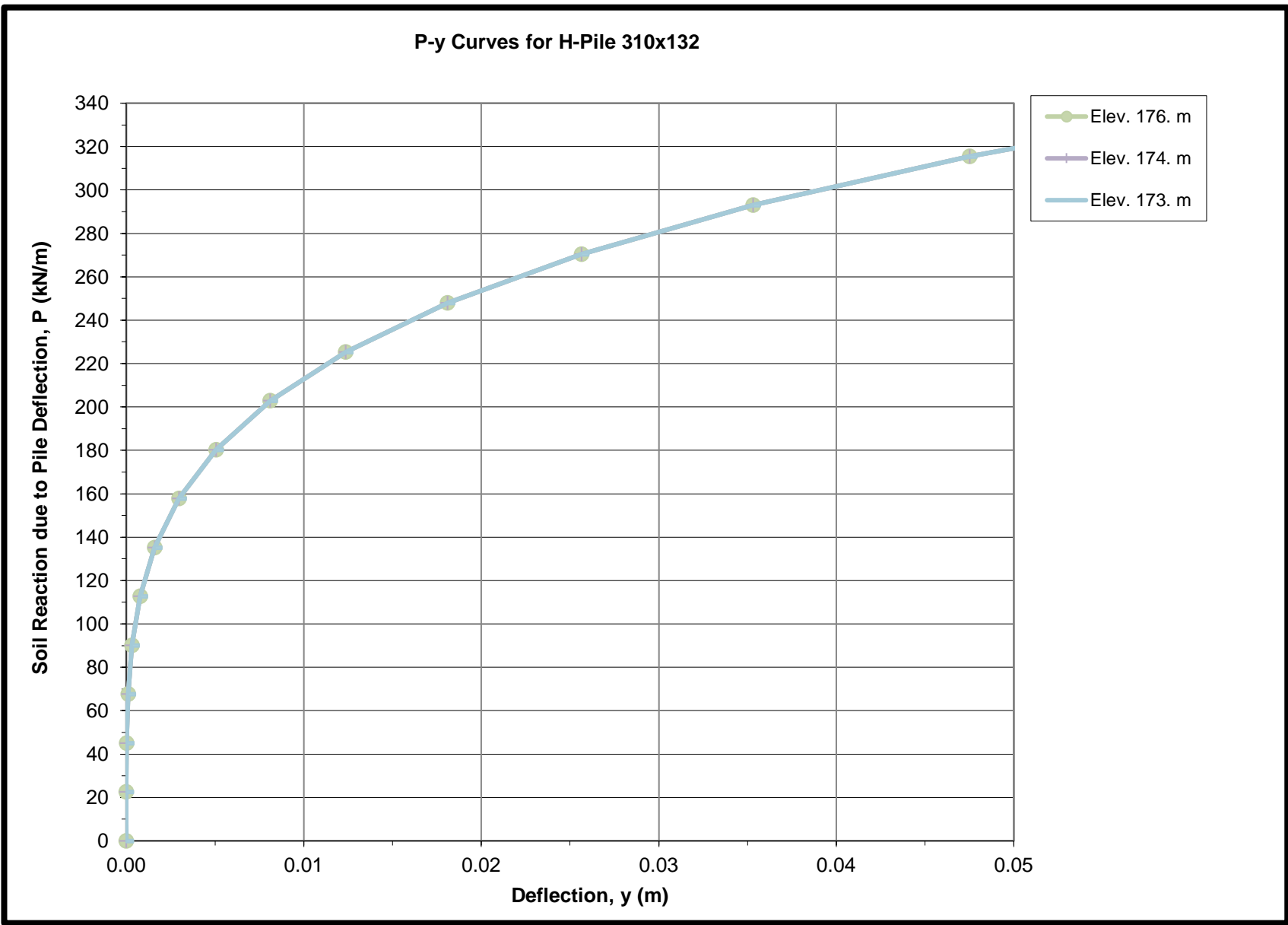
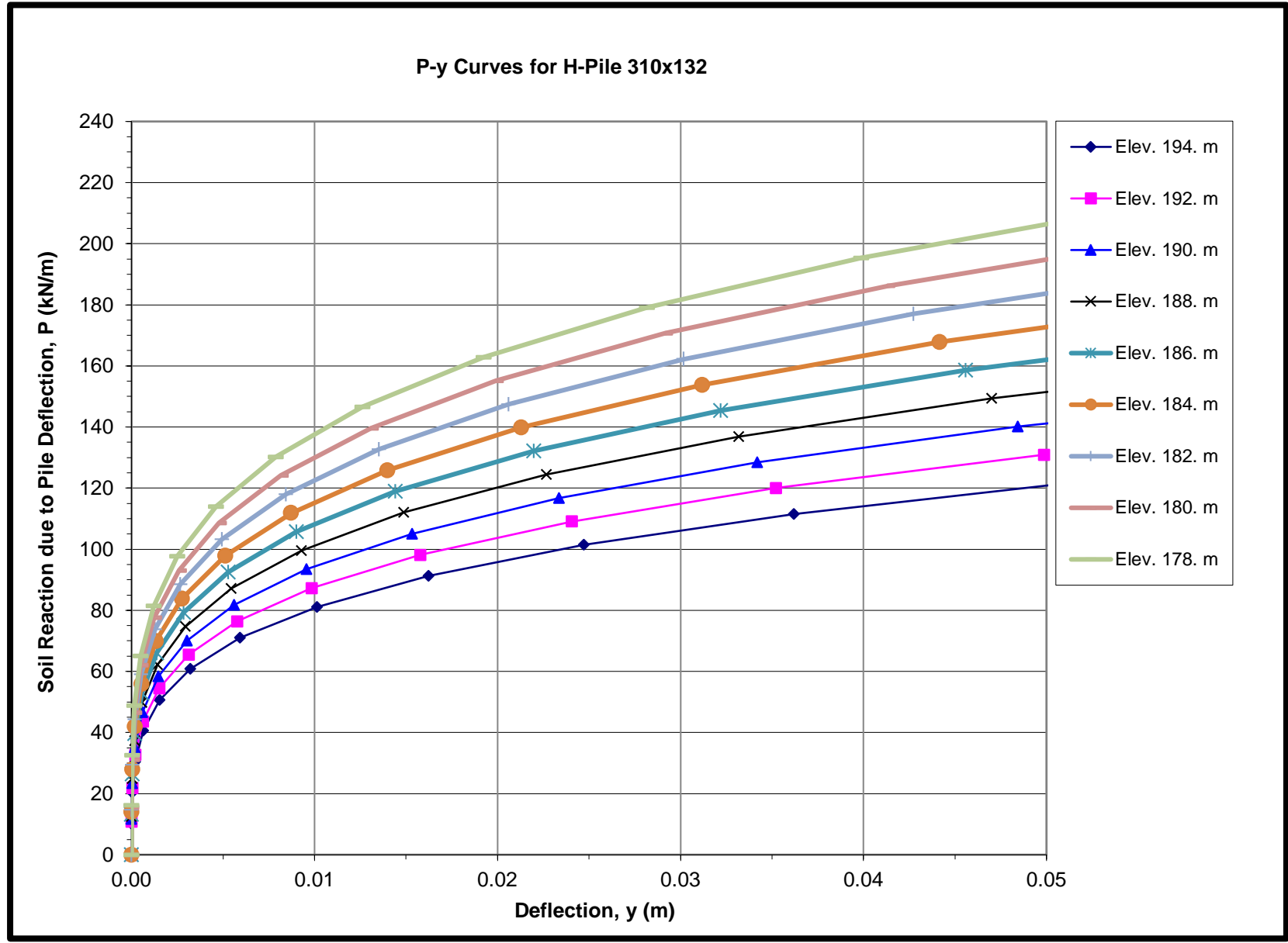
SUMMARY OF P-y CURVES FOR A H-Pile 310x132 at Abutments

Description Depth (z) * Elevation P-y Curves	Stiff Clayey Silt to Silty Clay																							
	z= 24.0 m		z= 26.0 m		z= 28.0 m		z= 30.0 m		z= 32.0 m		z= 34.0 m		z= 36.0 m		z= 38.0 m		z= 40.0 m		z= 42.0 m		z= 44.0 m		z= 45.0 m	
	Elev. 194. m		Elev. 192. m		Elev. 190. m		Elev. 188. m		Elev. 186. m		Elev. 184. m		Elev. 182. m		Elev. 180. m		Elev. 178. m		Elev. 176. m		Elev. 174. m		Elev. 173. m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	10.142	0.000	10.909	0.000	11.677	0.000	12.445	0.000	13.213	0.000	13.981	0.000	14.749	0.000	15.516	0.000	16.284	0.000	22.536	0.000	22.536	0.000	22.536
	0.000	20.283	0.000	21.818	0.000	23.354	0.000	24.890	0.000	26.426	0.000	27.961	0.000	29.497	0.000	31.033	0.000	32.569	0.000	45.072	0.000	45.072	0.000	45.072
	0.000	30.425	0.000	32.727	0.000	35.031	0.000	37.335	0.000	39.638	0.000	41.942	0.000	44.246	0.000	46.549	0.000	48.853	0.000	67.608	0.000	67.608	0.000	67.608
	0.001	40.567	0.001	43.636	0.001	46.708	0.001	49.780	0.001	52.851	0.001	55.923	0.001	58.994	0.001	62.066	0.000	65.137	0.000	90.144	0.000	90.144	0.000	90.144
	0.002	50.709	0.002	54.545	0.001	58.385	0.001	62.224	0.001	66.064	0.001	69.903	0.001	73.743	0.001	77.582	0.001	81.422	0.001	112.680	0.001	112.680	0.001	112.680
	0.003	60.850	0.003	65.455	0.003	70.062	0.003	74.669	0.003	79.277	0.003	83.884	0.003	88.491	0.003	93.099	0.002	97.706	0.002	135.216	0.002	135.216	0.002	135.216
	0.006	70.992	0.006	76.364	0.006	81.739	0.005	87.114	0.005	92.489	0.005	97.865	0.005	103.240	0.005	108.615	0.005	113.990	0.003	157.752	0.003	157.752	0.003	157.752
	0.010	81.134	0.010	87.273	0.010	93.416	0.009	99.559	0.009	105.702	0.009	111.845	0.008	117.988	0.008	124.132	0.008	130.275	0.005	180.288	0.005	180.288	0.005	180.288
	0.016	91.275	0.016	98.182	0.015	105.093	0.015	112.004	0.014	118.915	0.014	125.826	0.014	132.737	0.013	139.648	0.013	146.559	0.008	202.824	0.008	202.824	0.008	202.824
	0.025	101.417	0.024	109.091	0.023	116.770	0.023	124.449	0.022	132.128	0.021	139.807	0.021	147.486	0.020	155.165	0.019	162.843	0.012	225.360	0.012	225.360	0.012	225.360
	0.036	111.559	0.035	120.000	0.034	128.447	0.033	136.894	0.032	145.341	0.031	153.787	0.030	162.234	0.029	170.681	0.028	179.128	0.018	247.896	0.018	247.896	0.018	247.896
	0.051	121.700	0.050	130.909	0.048	140.124	0.047	149.339	0.046	158.553	0.044	167.768	0.043	176.983	0.041	186.197	0.040	195.412	0.026	270.432	0.026	270.432	0.026	270.432
	0.071	131.842	0.069	141.818	0.067	151.801	0.065	161.783	0.063	171.766	0.061	181.749	0.059	191.731	0.057	201.714	0.055	211.697	0.035	292.968	0.035	292.968	0.035	292.968
	0.095	141.984	0.092	152.727	0.090	163.478	0.087	174.228	0.084	184.979	0.082	195.729	0.079	206.480	0.077	217.230	0.074	227.981	0.048	315.504	0.048	315.504	0.048	315.504
	0.125	152.126	0.122	163.636	0.118	175.155	0.115	186.673	0.111	198.192	0.108	209.710	0.104	221.228	0.101	232.747	0.097	244.265	0.063	338.040	0.063	338.040	0.063	338.040
	0.157	152.126	0.152	163.636	0.148	175.155	0.143	186.673	0.139	198.192	0.135	209.710	0.130	221.228	0.126	232.747	0.122	244.265	0.078	338.040	0.078	338.040	0.078	338.040

NOTES: \* Depth (z) is measured to be positive below the underside of the pile cap (Elevation 218.0 m).

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

1. P-y curves are generated for vertical piles (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).



Date: December 2019  
Project No: 1671430 WO 0017

Prepared By: CC  
Checked By: ARV





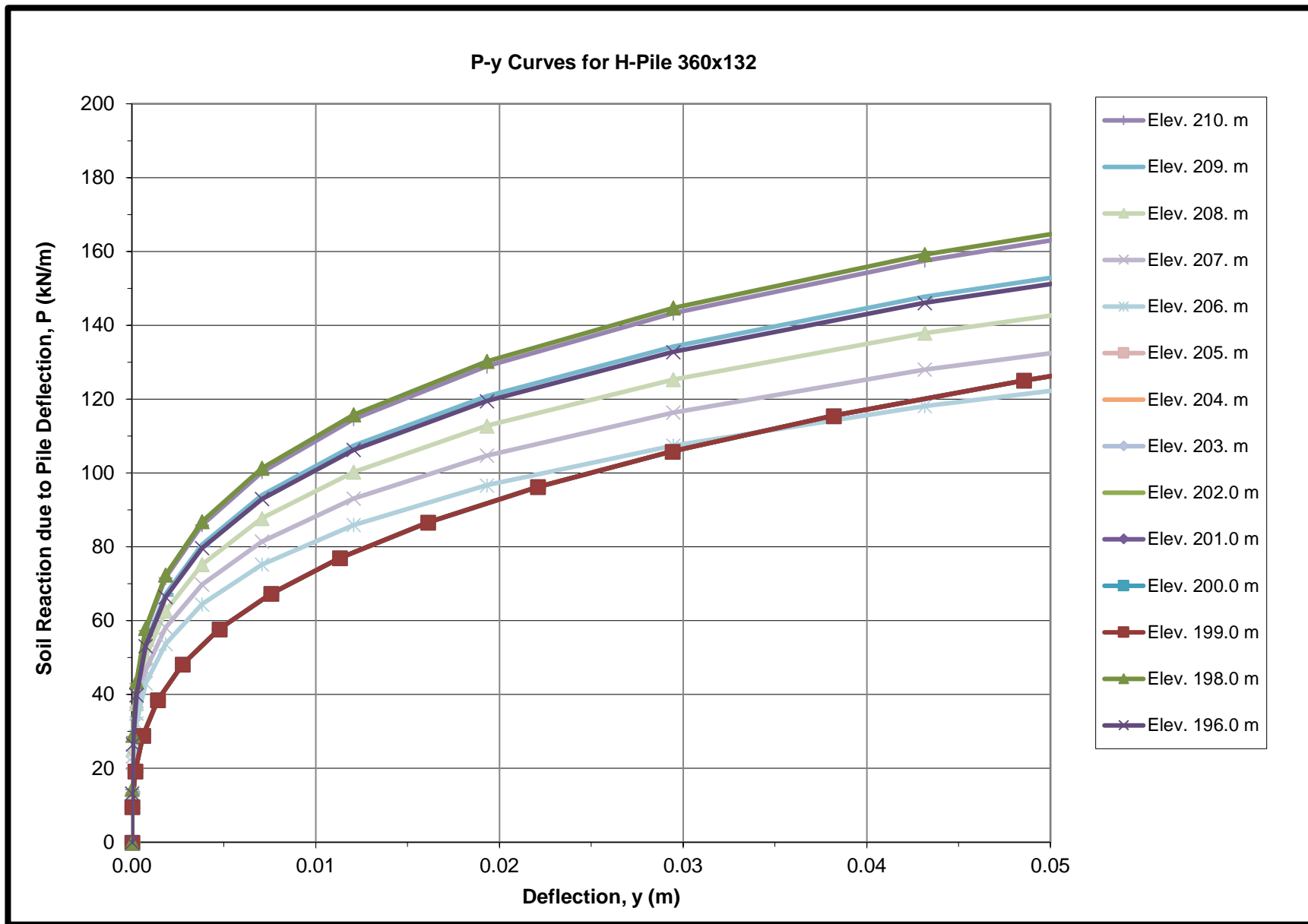
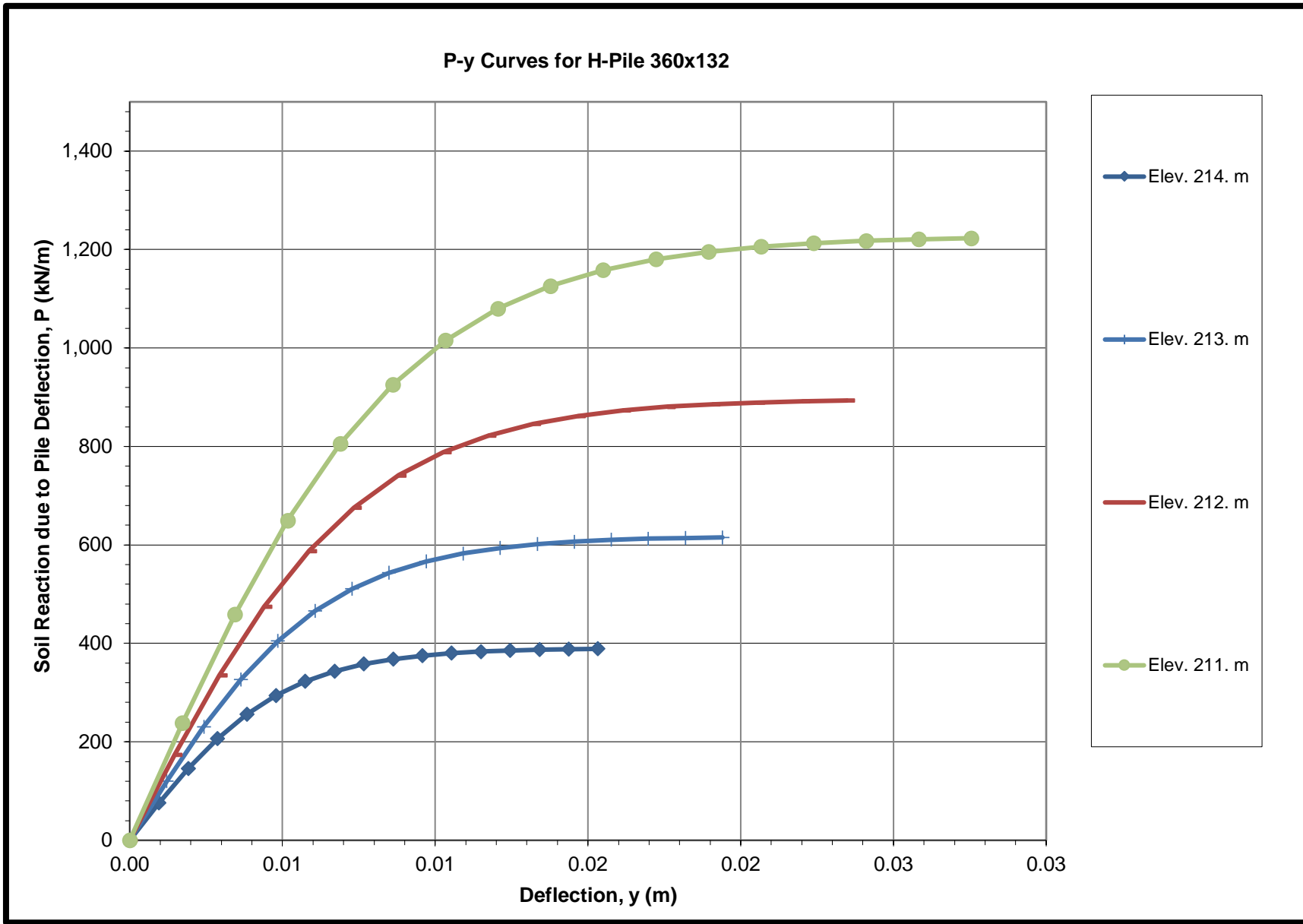
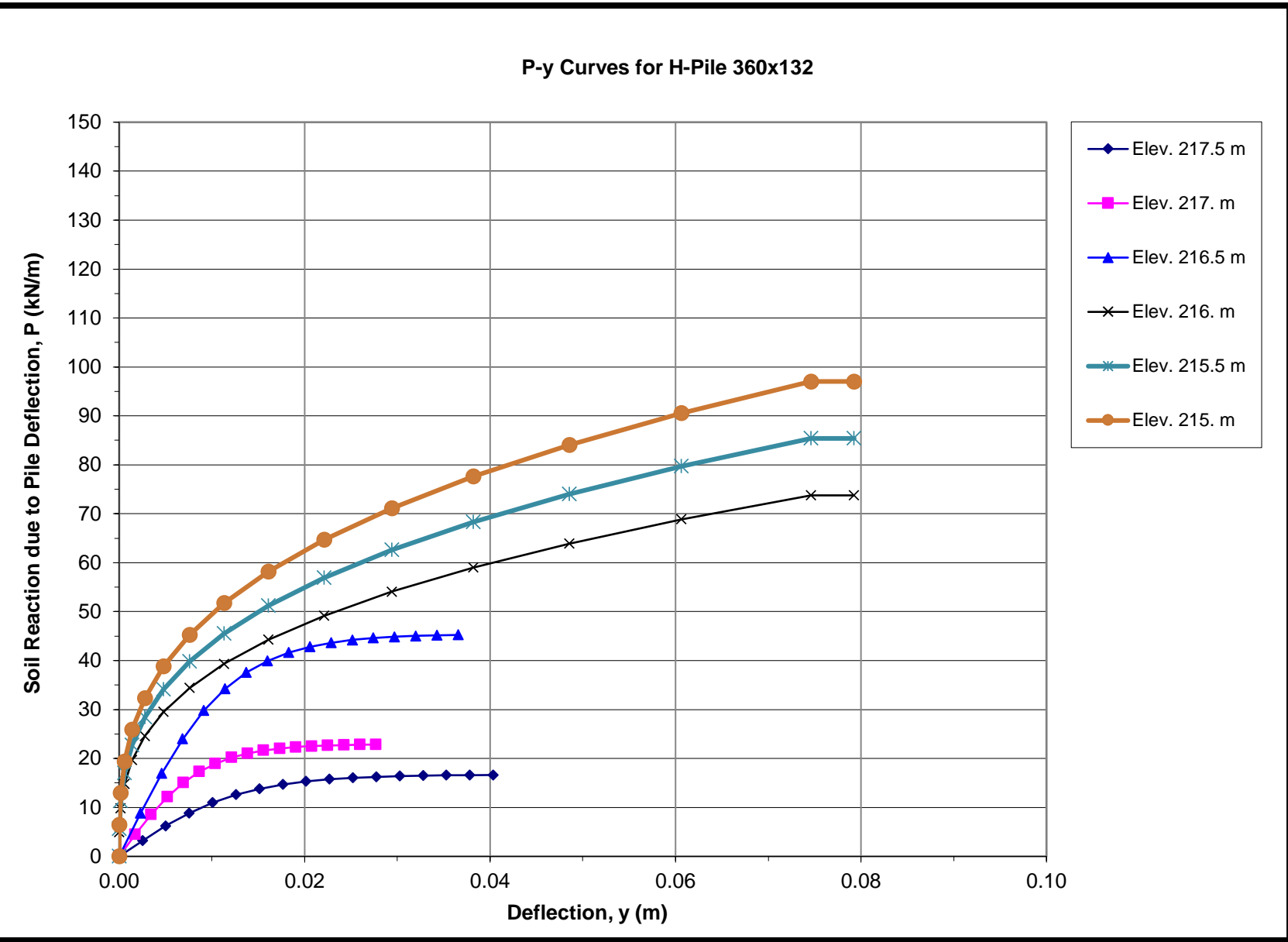
P-y CURVES  
1671430 WO 0017 Holland Drainage Canal Bridge Site 37-31 for 360x132 Pile

SUMMARY OF P-y CURVES FOR A H-Pile 360x132 at Abutments

Description Depth (z) * Elevation P-y Curves	Very Loose to Compact Silt and Sandy to Silty Sand Fill						Soft to Firm Clayey Silt						Compact to Dense Silt and Sand to Sand						Stiff Clayey Silt										Firm Silty Clay to Clayey Silt							
	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 4.0 m		z= 5.0 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m	
	Elev. 217.5 m		Elev. 217. m		Elev. 216.5 m		Elev. 216. m		Elev. 215.5 m		Elev. 215. m		Elev. 214. m		Elev. 213. m		Elev. 212. m		Elev. 211. m		Elev. 210. m		Elev. 209. m		Elev. 208. m		Elev. 207. m		Elev. 206. m		Elev. 205. m		Elev. 204. m		Elev. 203. m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.003	3.237	0.002	4.446	0.002	8.803	0.000	4.917	0.000	5.693	0.000	6.469	0.001	75.663	0.001	119.723	0.001	173.836	0.002	238.004	0.000	14.323	0.000	13.428	0.000	12.533	0.000	11.638	0.000	10.742	0.000	9.623	0.000	9.623	0.000	9.623	
0.005	6.239	0.003	8.569	0.005	16.969	0.000	9.835	0.000	11.386	0.000	12.938	0.002	145.846	0.002	230.772	0.003	335.079	0.003	458.767	0.000	28.646	0.000	26.856	0.000	25.066	0.000	23.275	0.000	21.485	0.000	19.247	0.000	19.247	0.000	19.247	
0.008	8.835	0.005	12.135	0.007	24.031	0.001	14.752	0.001	17.079	0.001	19.406	0.003	206.546	0.004	326.818	0.004	474.537	0.005	649.704	0.000	42.970	0.000	40.284	0.000	37.598	0.000	34.913	0.000	32.227	0.001	28.870	0.001	28.870	0.001	28.870	
0.010	10.948	0.007	15.038	0.009	29.779	0.001	19.669	0.001	22.772	0.001	25.875	0.004	255.950	0.005	404.991	0.006	588.043	0.007	805.108	0.001	57.293	0.001	53.712	0.001	50.131	0.001	46.550	0.001	42.970	0.001	38.494	0.001	38.494	0.001	38.494	
0.013	12.585	0.009	17.286	0.011	34.230	0.003	24.586	0.003	28.465	0.003	32.344	0.005	294.209	0.006	465.528	0.007	675.943	0.009	925.454	0.002	71.616	0.002	67.140	0.002	62.664	0.002	58.188	0.002	53.712	0.003	48.117	0.003	48.117	0.003	48.117	
0.015	13.804	0.010	18.961	0.014	37.546	0.005	29.504	0.005	34.158	0.005	38.813	0.006	322.712	0.007	510.628	0.009	741.427	0.010	1015.111	0.004	85.939	0.004	80.568	0.004	75.197	0.004	69.826	0.004	64.454	0.005	57.740	0.005	57.740	0.005	57.740	
0.018	14.686	0.012	20.173	0.016	39.946	0.008	34.421	0.008	39.851	0.008	45.282	0.007	343.339	0.008	543.266	0.010	788.818	0.012	1079.995	0.007	100.262	0.007	93.996	0.007	87.730	0.007	81.463	0.007	75.197	0.008	67.364	0.008	67.364	0.008	67.364	
0.020	15.312	0.014	21.031	0.018	41.647	0.011	39.338	0.011	45.544	0.011	51.751	0.008	357.956	0.010	566.394	0.012	822.400	0.014	1125.973	0.012	114.586	0.012	107.424	0.012	100.262	0.012	93.101	0.012	85.939	0.011	76.987	0.011	76.987	0.011	76.987	
0.023	15.748	0.016	21.631	0.021	42.834	0.016	44.255	0.016	51.237	0.016	58.219	0.009	368.159	0.011	582.539	0.013	845.842	0.015	1158.069	0.019	128.909	0.019	120.852	0.019	112.795	0.019	104.738	0.019	96.682	0.016	86.611	0.016	86.611	0.016	86.611	
0.025	16.050	0.017	22.045	0.023	43.654	0.022	49.173	0.022	56.930	0.022	64.688	0.010	375.208	0.012	593.692	0.015	862.036	0.017	1180.240	0.029	143.232	0.029	134.280	0.029	125.328	0.029	116.376	0.029	107.424	0.022	96.234	0.022	96.234	0.022	96.234	
0.028	16.256	0.019	22.329	0.025	44.217	0.029	54.090	0.029	62.623	0.029	71.157	0.011	380.042	0.013	601.341	0.016	873.143	0.019	1195.446	0.043	157.555	0.043	147.708	0.043	137.861	0.043	128.014	0.043	118.166	0.029	105.857	0.029	105.857	0.029	105.857	
0.030	16.398	0.021	22.523	0.027	44.600	0.038	59.007	0.038	68.317	0.038	77.626	0.011	383.341	0.015	606.561	0.018	880.721	0.021	1205.823	0.061	171.878	0.061	161.136	0.061	150.394	0.061	139.651	0.061	128.909	0.038	115.481	0.038	115.481	0.038	115.481	
0.033	16.494	0.023	22.655	0.030	44.861	0.049	63.925	0.049	74.010	0.049	84.095	0.012	385.584	0.016	610.111	0.019	885.876	0.022	1212.879	0.084	186.202	0.084	174.564	0.084	162.926	0.084	151.289	0.084	139.651	0.049	125.104	0.049	125.104	0.049	125.104	
0.035	16.559	0.024	22.744	0.032	45.039	0.061	68.842	0.061	79.703	0.061	90.563	0.013	387.106	0.017	612.519	0.021	889.373	0.024	1217.668	0.113	200.525	0.113	187.992	0.113	175.459	0.113	162.926	0.113	150.394	0.061	134.728	0.061	134.728	0.061	134.728	
0.038	16.603	0.026	22.805	0.034	45.159	0.075	73.759	0.075	85.396	0.075	97.032	0.014	388.138	0.018	614.151	0.022	891.742	0.026	1220.912	0.149	214.848	0.149	201.420	0.149	187.992	0.149	174.564	0.149	161.136	0.075	144.351	0.075	144.351	0.075	144.351	
0.040	16.633	0.028	22.846	0.037	45.240	0.079	73.759	0.079	85.396	0.079	97.032	0.015	388.835	0.019	615.255	0.023	893.346	0.028	1223.107	0.187	214.848	0.187	201.420	0.187	187.992	0.187	174.564	0.187	161.136	0.079	144.351	0.079	144.351	0.079	144.351	

Description Depth (z) * Elevation P-y Curves	Firm Silty Clay to Clayey Silt						Stiff Silty Clay to Clayey Silt					
	z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m		z= 22.0 m	
	Elev. 202.0 m		Elev. 201.0 m		Elev. 200.0 m		Elev. 199.0 m		Elev. 198.0 m		Elev. 196.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.00	0.000	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.000	9.623	0.000	9.62	0.000	9.62	0.000	9.623	0.000	14.472	0.000	13.279	0.000
0.000	19.247	0.000	19.25	0.000	19.25	0.000	19.247	0.000	28.945	0.000	26.558	0.000
0.001	28.870	0.001	28.87	0.001	28.87	0.001	28.870	0.000	43.417	0.000	39.836	0.000
0.001	38.494	0.001	38.49	0.001	38.49	0.001	38.494	0.001	57.890	0.001	53.115	0.001
0.003	48.117	0.003	48.12	0.003	48.12	0.003	48.117	0.002	72.362	0.002	66.394	0.002
0.005	57.740	0.005	57.74	0.005	57.74	0.005	57.740	0.004	86.834	0.004	79.673	0.004
0.008	67.364	0.008	67.36	0.008	67.36	0.008	67.364	0.007	101.307	0.007	92.952	0.007
0.011	76.987	0.011	76.99	0.011	76.99	0.011	76.987	0.012	115.779	0.012	106.230	0.012
0.016	86.611	0.016	86.61	0.016	86.61	0.016	86.611	0.019	130.252	0.019	119.509	0.019
0.022	96.234	0.022	96.23	0.022	96.23	0.022	96.234	0.029	144.724	0.029	132.788	0.029
0.029	105.857	0.029	105.86	0.029	105.86	0.029	105.857	0.043	159.196	0.043	146.067	0.043
0.038	115.481	0.038	115.48	0.038	115.48	0.038	115.481	0.061	173.669	0.061	159.346	0.061
0.049	125.104	0.049	125.10	0.049	125.10	0.049	125.104	0.084	188.141	0.084	172.624	0.084
0.061	134.728	0.061	134.73	0.061	134.73	0.061	134.728	0.113	202.614	0.113	185.903	0.113
0.075	144.351	0.075	144.35	0.075	144.35	0.075	144.351	0.149	217.086	0.149	199.182	0.149
0.079	144.351	0.079	144.35	0.079	144.35	0.079	144.351	0.187	217.086	0.187	199.182	0.187

NOTES: \* Depth (z) is measured to be positive below the underside of the pile cap (Elevation 218.0 m).  
The P-y curves have been generated based on the following assumptions:  
1. P-y curves are generated for vertical piles (i.e. no inclination)  
2. Static loading condition is considered.  
3. There are no pile group effects (i.e. analysis is based on a single pile).

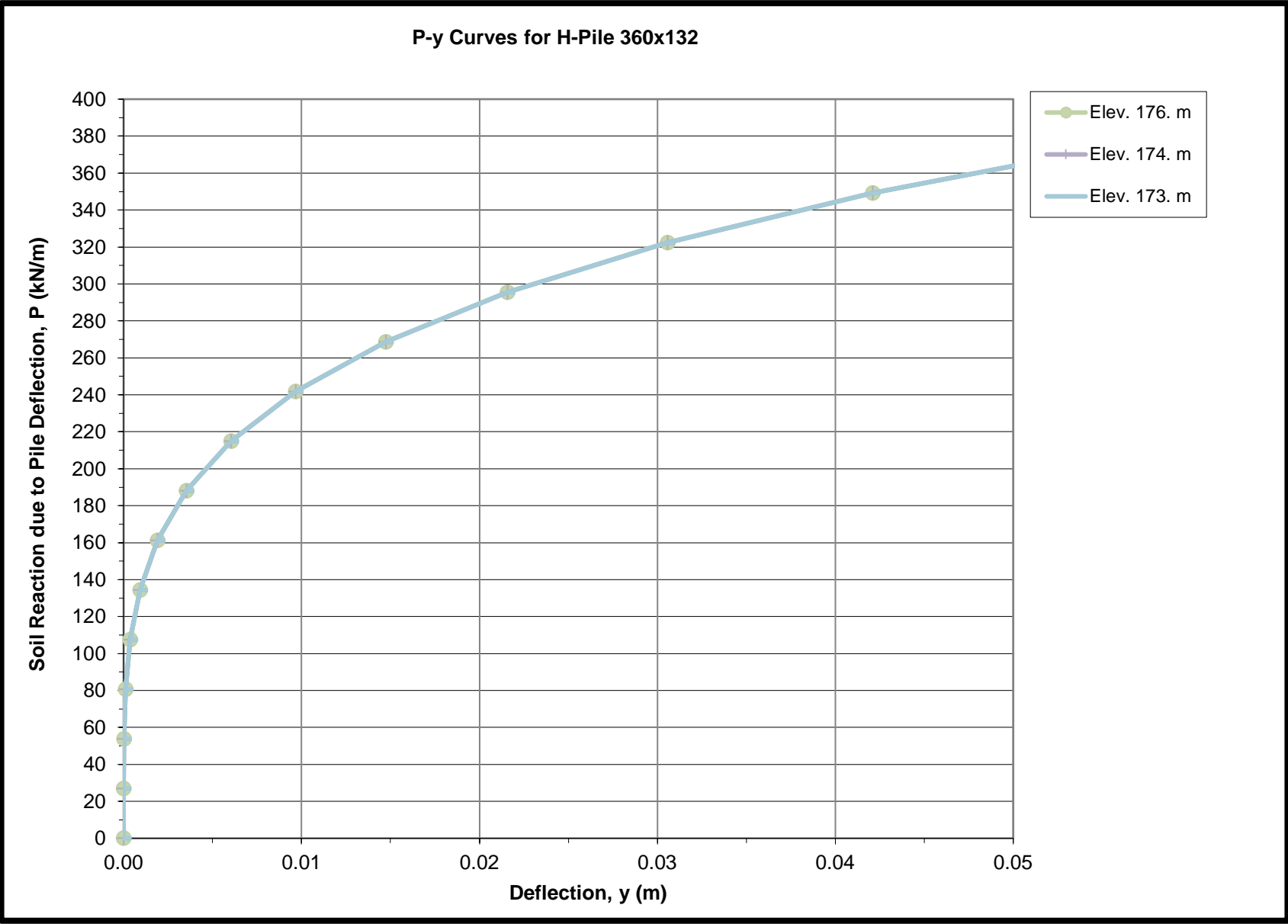
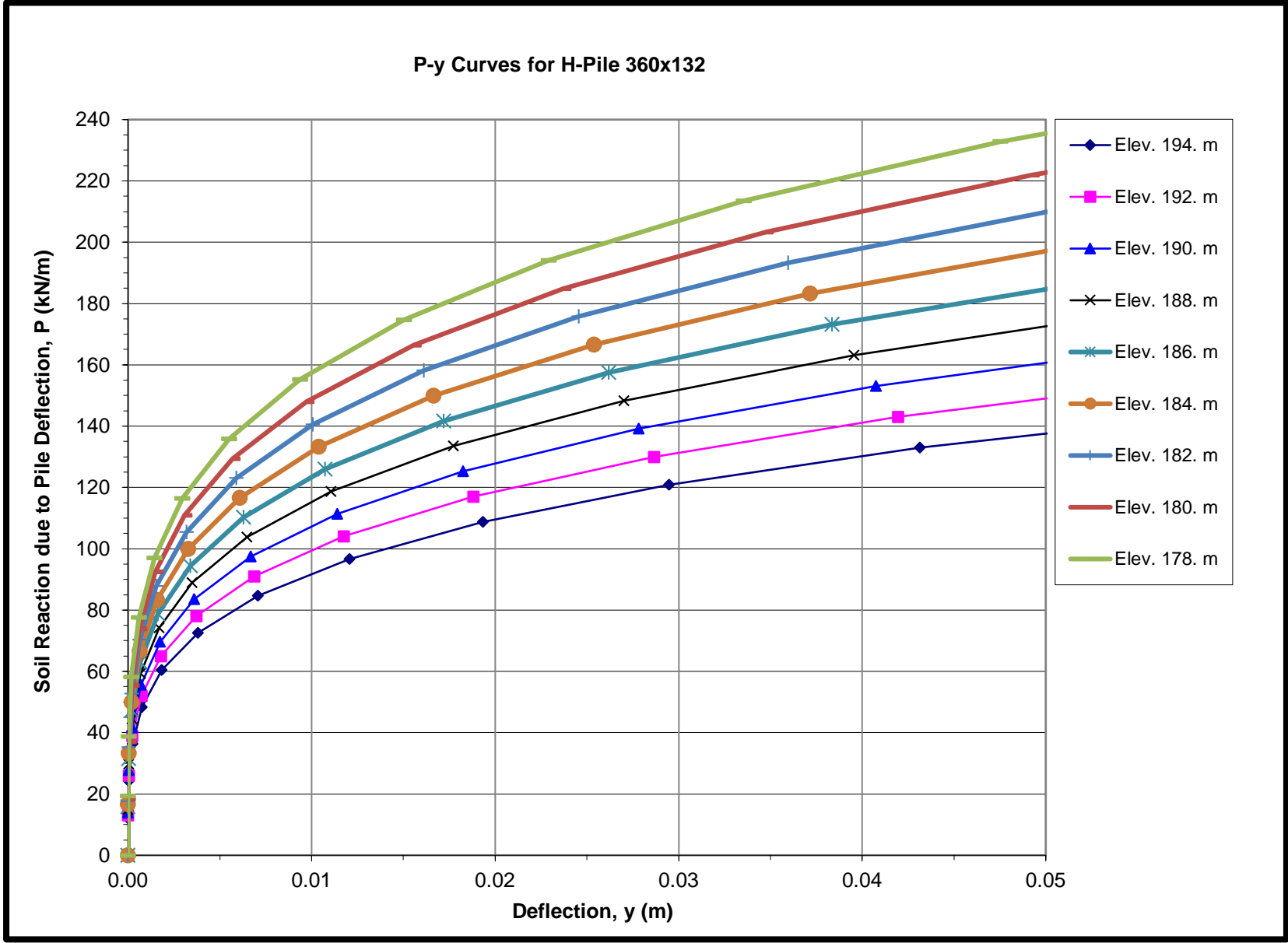




SUMMARY OF P-y CURVES FOR A H-Pile 360x132 at Abutments

Description Depth (z) * Elevation P-y Curves	Stiff Clayey Silt to Silty Clay																					
	z= 24.0 m		z= 26.0 m		z= 28.0 m		z= 30.0 m		z= 32.0 m		z= 34.0 m		z= 36.0 m		z= 38.0 m		z= 40.0 m		z= 42.0 m		z= 44.0 m	
	Elev. 194. m		Elev. 192. m		Elev. 190. m		Elev. 188. m		Elev. 186. m		Elev. 184. m		Elev. 182. m		Elev. 180. m		Elev. 178. m		Elev. 176. m		Elev. 174. m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	12.086	0.000	13.000	0.000	13.915	0.000	14.830	0.000	15.746	0.000	16.661	0.000	17.576	0.000	18.491	0.000	19.406	0.000	26.856	0.000	26.856
	0.000	24.172	0.000	26.001	0.000	27.831	0.000	29.661	0.000	31.491	0.000	33.321	0.000	35.152	0.000	36.982	0.000	38.812	0.000	53.712	0.000	53.712
	0.000	36.257	0.000	39.001	0.000	41.746	0.000	44.491	0.000	47.237	0.000	49.982	0.000	52.727	0.000	55.473	0.000	58.218	0.000	80.568	0.000	80.568
	0.001	48.343	0.001	52.001	0.001	55.662	0.001	59.322	0.001	62.982	0.001	66.643	0.001	70.303	0.001	73.963	0.001	77.624	0.000	107.424	0.000	107.424
	0.002	60.429	0.002	65.001	0.002	69.577	0.002	74.152	0.002	78.728	0.002	83.303	0.002	87.879	0.001	92.454	0.001	97.030	0.001	134.280	0.001	134.280
	0.004	72.515	0.004	78.002	0.004	83.492	0.004	88.983	0.003	94.473	0.003	99.964	0.003	105.455	0.003	110.945	0.003	116.436	0.002	161.136	0.002	161.136
	0.007	84.601	0.007	91.002	0.007	97.408	0.006	103.813	0.006	110.219	0.006	116.625	0.006	123.030	0.006	129.436	0.006	135.842	0.004	187.992	0.004	187.992
	0.012	96.686	0.012	104.002	0.011	111.323	0.011	118.644	0.011	125.965	0.010	133.285	0.010	140.606	0.010	147.927	0.009	155.248	0.006	214.848	0.006	214.848
	0.019	108.772	0.019	117.003	0.018	125.238	0.018	133.474	0.017	141.710	0.017	149.946	0.016	158.182	0.016	166.418	0.015	174.654	0.010	241.704	0.010	241.704
	0.029	120.858	0.029	130.003	0.028	139.154	0.027	148.305	0.026	157.456	0.025	166.607	0.025	175.758	0.024	184.909	0.023	194.059	0.015	268.560	0.015	268.560
	0.043	132.944	0.042	143.003	0.041	153.069	0.040	163.135	0.038	173.201	0.037	183.267	0.036	193.333	0.035	203.399	0.034	213.465	0.022	295.416	0.022	295.416
	0.061	145.030	0.059	156.004	0.058	166.985	0.056	177.966	0.054	188.947	0.053	199.928	0.051	210.909	0.049	221.890	0.048	232.871	0.031	322.272	0.031	322.272
	0.084	157.115	0.082	169.004	0.079	180.900	0.077	192.796	0.075	204.692	0.072	216.589	0.070	228.485	0.068	240.381	0.065	252.277	0.042	349.128	0.042	349.128
	0.113	169.201	0.110	182.004	0.107	194.815	0.104	207.627	0.101	220.438	0.097	233.249	0.094	246.061	0.091	258.872	0.088	271.683	0.057	375.984	0.057	375.984
	0.149	181.287	0.145	195.004	0.141	208.731	0.137	222.457	0.133	236.184	0.128	249.910	0.124	263.636	0.120	277.363	0.116	291.089	0.075	402.840	0.075	402.840
	0.187	181.287	0.181	195.004	0.176	208.731	0.171	222.457	0.166	236.184	0.161	249.910	0.155	263.636	0.150	277.363	0.145	291.089	0.093	402.840	0.093	402.840

NOTES: \* Depth (z) is measured to be positive below the underside of the pile cap (Elevation 218.0 m).  
The P-y curves have been generated based on the following assumptions:  
1. P-y curves are generated for vertical piles (i.e. no inclination)  
2. Static loading condition is considered.  
3. There are no pile group effects (i.e. analysis is based on a single pile).



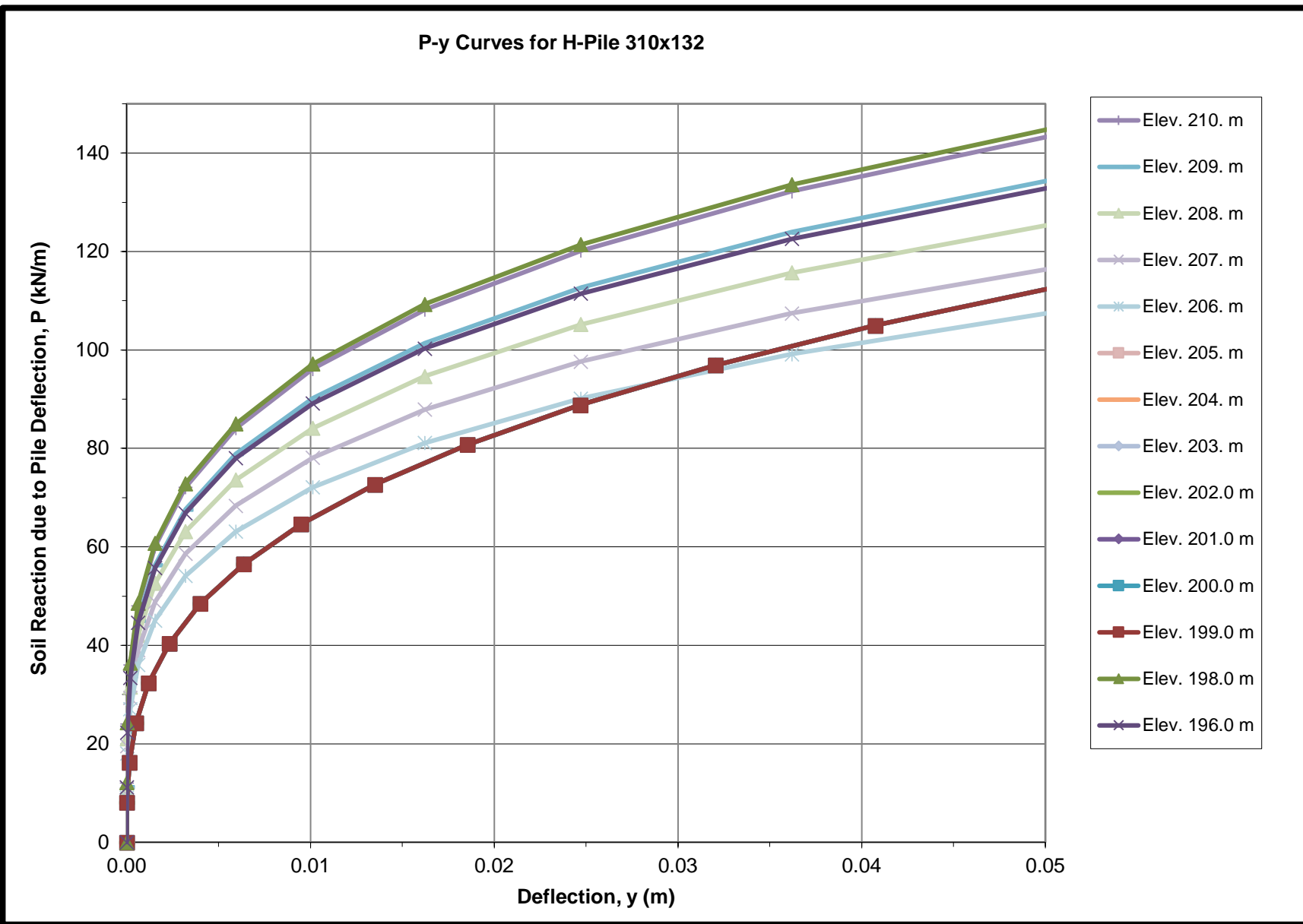
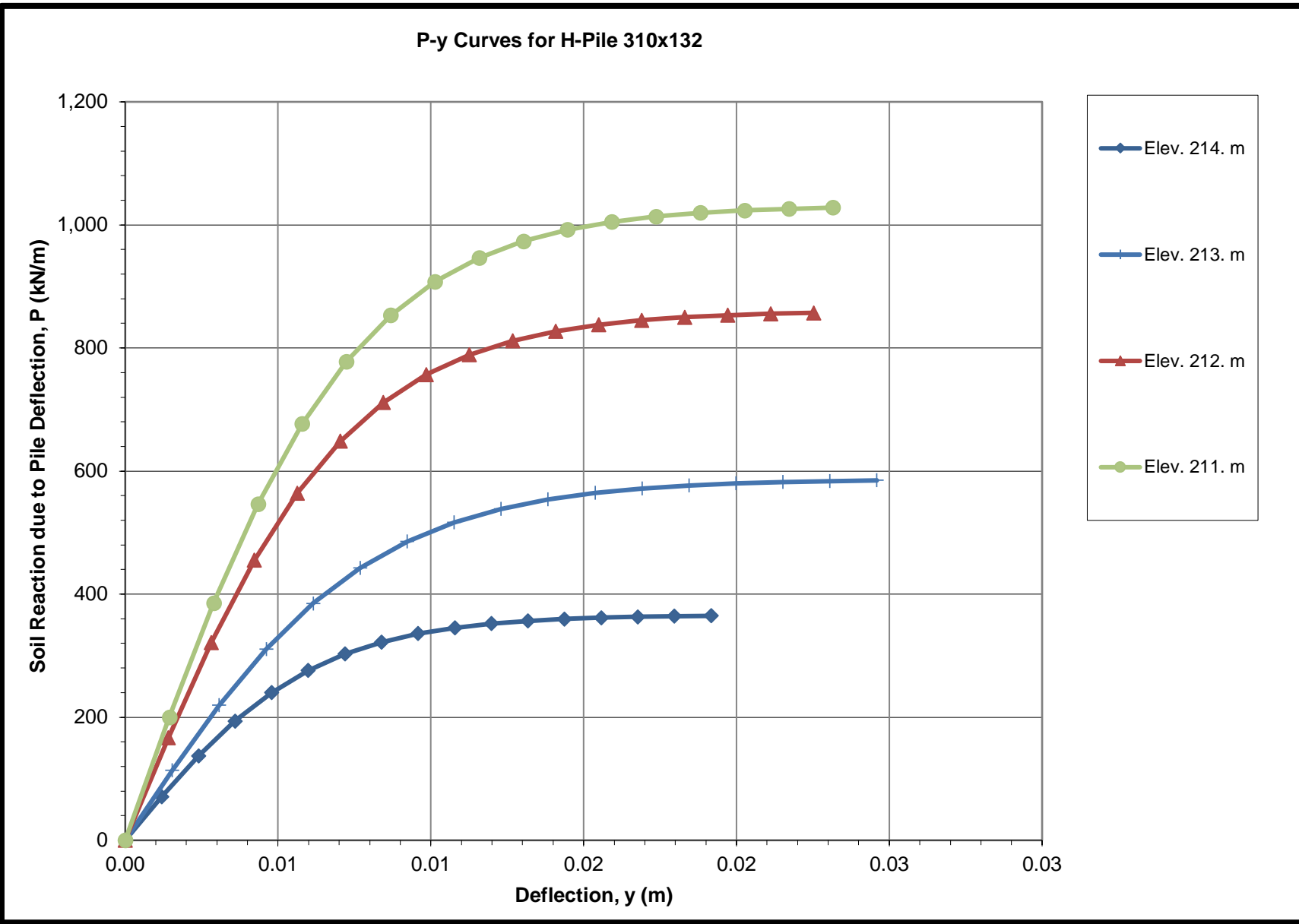
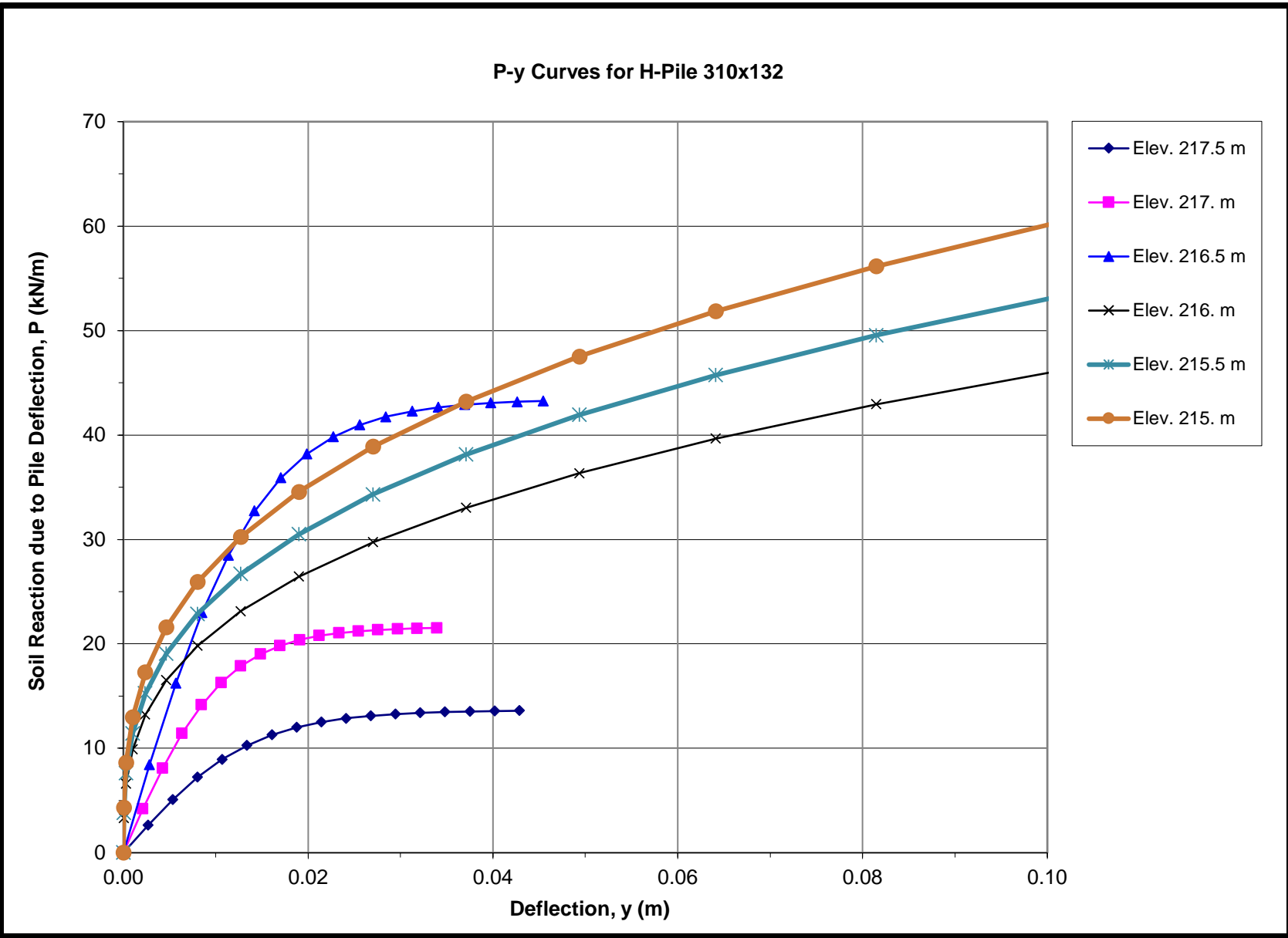


SUMMARY OF P-y CURVES FOR A H-Pile 310x132 at Abutments Assuming Pre-Auger of Upper 5 m

Description Depth (z) * Elevation P-y Curves	Pre-Augered Silt and Sandy to Silty Sand Fill						Pre-Augered Clayey Silt						Pre-Augered Silt and Sand to Sand				Compact to Dense Silt and Sand to Sand				Stiff Clayey Silt										Firm Silty Clay to Clayey Silt					
	z= .5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 4.0 m		z= 5.0 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0 m		z= 11.0 m		z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m	
	Elev. 217.5 m		Elev. 217. m		Elev. 216.5 m		Elev. 216. m		Elev. 215.5 m		Elev. 215. m		Elev. 214. m		Elev. 213. m		Elev. 212. m		Elev. 211. m		Elev. 210. m		Elev. 209. m		Elev. 208. m		Elev. 207. m		Elev. 206. m		Elev. 205. m		Elev. 204. m		Elev. 203. m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.003	2.645	0.002	4.188	0.003	8.417	0.000	3.303	0.000	3.812	0.000	4.320	0.001	70.960	0.002	113.801	0.001	166.793	0.001	200.061	0.000	12.019	0.000	11.268	0.000	10.517	0.000	9.766	0.000	9.014	0.000	8.075	0.000	8.075	0.000	8.075	
0.005	5.099	0.004	8.073	0.006	16.225	0.000	6.607	0.000	7.624	0.000	8.641	0.002	136.779	0.003	219.358	0.003	321.502	0.003	385.628	0.000	24.038	0.000	22.536	0.000	21.034	0.000	19.531	0.000	18.029	0.000	16.151	0.000	16.151	0.000	16.151	
0.008	7.221	0.006	11.433	0.009	22.978	0.001	9.910	0.001	11.436	0.001	12.961	0.004	193.706	0.005	310.654	0.004	455.311	0.004	546.125	0.000	36.058	0.000	33.804	0.000	31.550	0.000	29.297	0.000	27.043	0.001	24.226	0.001	24.226	0.001	24.226	
0.011	8.948	0.008	14.168	0.011	28.474	0.002	13.214	0.002	15.248	0.002	17.282	0.005	240.039	0.006	384.960	0.006	564.217	0.006	676.754	0.001	48.077	0.001	45.072	0.001	42.067	0.001	39.062	0.001	36.058	0.001	32.302	0.001	32.302	0.001	32.302	
0.013	10.286	0.011	16.286	0.014	32.730	0.005	16.517	0.005	19.060	0.005	21.602	0.006	275.920	0.008	442.504	0.007	648.556	0.007	777.915	0.002	60.096	0.002	56.340	0.002	52.584	0.002	48.828	0.002	45.072	0.002	40.377	0.002	40.377	0.002	40.377	
0.016	11.282	0.013	17.864	0.017	35.901	0.008	19.821	0.008	22.872	0.008	25.923	0.007	302.651	0.009	485.373	0.008	711.387	0.009	853.278	0.003	72.115	0.003	67.608	0.003	63.101	0.003	58.594	0.003	54.086	0.004	48.452	0.004	48.452	0.004	48.452	
0.019	12.004	0.015	19.005	0.020	38.196	0.013	23.124	0.013	26.684	0.013	30.243	0.008	321.996	0.011	516.397	0.010	756.858	0.010	907.818	0.006	84.134	0.006	78.876	0.006	73.618	0.006	68.359	0.006	63.101	0.006	56.528	0.006	56.528	0.006	56.528	
0.021	12.515	0.017	19.815	0.023	39.822	0.019	26.428	0.019	30.496	0.019	34.564	0.010	335.704	0.012	538.381	0.011	789.079	0.012	946.466	0.010	96.154	0.010	90.144	0.010	84.134	0.010	78.125	0.010	72.115	0.009	64.603	0.009	64.603	0.009	64.603	
0.024	12.871	0.019	20.379	0.026	40.957	0.027	29.731	0.027	34.308	0.027	38.884	0.011	345.273	0.014	553.728	0.013	811.572	0.013	973.445	0.016	108.173	0.016	101.412	0.016	94.651	0.016	87.890	0.016	81.130	0.014	72.679	0.014	72.679	0.014	72.679	
0.027	13.118	0.021	20.769	0.028	41.741	0.037	33.035	0.037	38.120	0.037	43.205	0.012	351.883	0.015	564.329	0.014	827.110	0.014	992.082	0.025	120.192	0.025	112.680	0.025	105.168	0.025	97.656	0.025	90.144	0.019	80.754	0.019	80.754	0.019	80.754	
0.029	13.287	0.023	21.037	0.031	42.279	0.049	36.338	0.049	41.932	0.049	47.525	0.013	356.417	0.017	571.600	0.015	837.766	0.016	1004.863	0.036	132.211	0.036	123.948	0.036	115.685	0.036	107.422	0.036	99.158	0.025	88.829	0.025	88.829	0.025	88.829	
0.032	13.402	0.025	21.220	0.034	42.646	0.064	39.642	0.064	45.744	0.064	51.846	0.014	359.511	0.018	576.561	0.017	845.037	0.017	1013.586	0.051	144.230	0.051	135.216	0.051	126.202	0.051	117.187	0.051	108.173	0.032	96.905	0.032	96.905	0.032	96.905	
0.035	13.481	0.028	21.344	0.037	42.895	0.082	42.945	0.082	49.556	0.082	56.166	0.016	361.615	0.020	579.935	0.018	849.983	0.019	1019.517	0.071	156.250	0.071	146.484	0.071	136.718	0.071	126.953	0.071	117.187	0.041	104.980	0.041	104.980	0.041	104.980	
0.038	13.534	0.030	21.428	0.040	43.065	0.102	46.249	0.102	53.368	0.102	60.487	0.017	363.042	0.022	582.225	0.020	853.339	0.020	1023.542	0.095	168.269	0.095	157.752	0.095	147.235	0.095	136.718	0.095	126.202	0.051	113.056	0.051	113.056	0.051	113.056	
0.040	13.570	0.032	21.485	0.043	43.179	0.125	49.552	0.125	57.180	0.125	64.807	0.018	364.009	0.023	583.776	0.021	855.612	0.022	1026.269	0.125	180.288	0.125	169.020	0.125	157.752	0.125	146.484	0.125	135.216	0.063	121.131	0.063	121.131	0.063	121.131	
0.043	13.594	0.034	21.524	0.045	43.257	0.133	49.552	0.133	57.180	0.133	64.807	0.019	364.664	0.025	584.825	0.023	857.150	0.023	1028.114	0.157	180.288	0.157	169.020	0.157	157.752	0.157	146.484	0.157	135.216	0.067	121.131	0.067	121.131	0.067	121.131	

Description Depth (z) * Elevation P-y Curves	Firm Clayey Silt to Silty Clay						Stiff Clayey Silt to Silty Clay					
	z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m		z= 22.0 m	
	Elev. 202.0 m		Elev. 201.0 m		Elev. 200.0 m		Elev. 199.0 m		Elev. 198.0 m		Elev. 196.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.00	0.000	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.000	8.075	0.000	8.08	0.000	8.08	0.000	8.075	0.000	12.144	0.000	11.143	0.000
0.000	16.151	0.000	16.15	0.000	16.15	0.000	16.151	0.000	24.289	0.000	22.286	0.000
0.001	24.226	0.001	24.23	0.001	24.23	0.001	24.226	0.000	36.433	0.000	33.428	0.000
0.001	32.302	0.001	32.30	0.001	32.30	0.001	32.302	0.001	48.578	0.001	44.571	0.001
0.002	40.377	0.002	40.38	0.002	40.38	0.002	40.377	0.002	60.722	0.002	55.714	0.002
0.004	48.452	0.004	48.45	0.004	48.45	0.004	48.452	0.003	72.866	0.003	66.857	0.003
0.006	56.528	0.006	56.53	0.006	56.53	0.006	56.528	0.006	85.011	0.006	78.000	0.006
0.009	64.603	0.009	64.60	0.009	64.60	0.009	64.603	0.010	97.155	0.010	89.142	0.010
0.014	72.679	0.014	72.68	0.014	72.68	0.014	72.679	0.016	109.300	0.016	100.285	0.016
0.019	80.754	0.019	80.75	0.019	80.75	0.019	80.754	0.025	121.444	0.025	111.428	0.025
0.025	88.829	0.025	88.83	0.025	88.83	0.025	88.829	0.036	133.588	0.036	122.571	0.036
0.032	96.905	0.032	96.90	0.032	96.90	0.032	96.905	0.051	145.733	0.051	133.714	0.051
0.041	104.980	0.041	104.98	0.041	104.98	0.041	104.980	0.071	157.877	0.071	144.856	0.071
0.051	113.056	0.051	113.06	0.051	113.06	0.051	113.056	0.095	170.022	0.095	155.999	0.095
0.063	121.131	0.063	121.13	0.063	121.13	0.063	121.131	0.125	182.166	0.125	167.142	0.125
0.067	121.131	0.067	121.13	0.067	121.13	0.067	121.131	0.157	182.166	0.157	167.142	0.157

NOTES: \* Depth (z) is measured to be positive below the underside of the pile cap (Elevation 218.0 m).  
The P-y curves have been generated based on the following assumptions:  
1. P-y curves are generated for vertical piles (i.e. no inclination)  
2. Static loading condition is considered.  
3. There are no pile group effects (i.e. analysis is based on a single pile).





P-y CURVES

1671430 WO 0017 Holland Drainage Canal Bridge Site 37-31 for 310x132 Pile

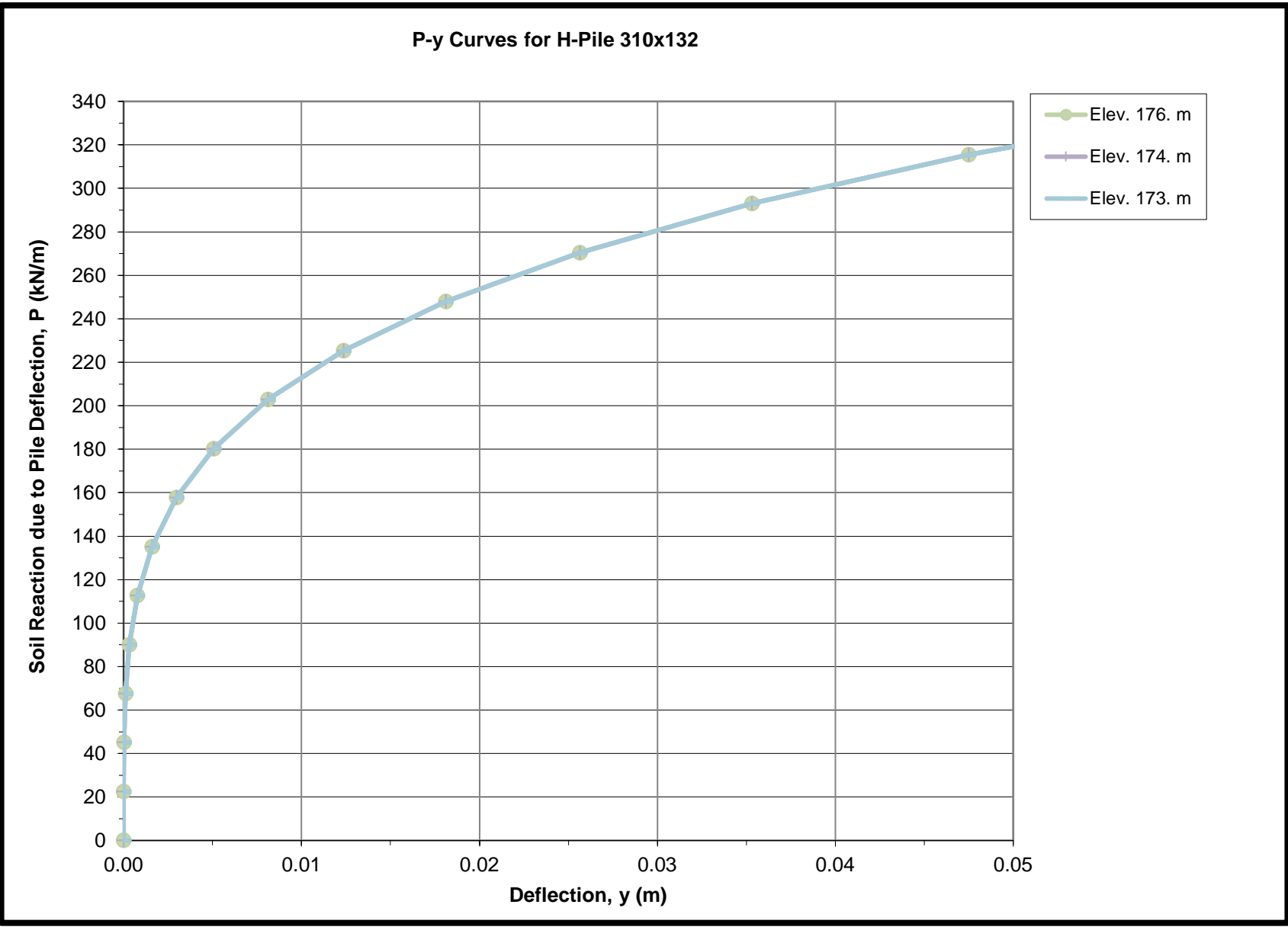
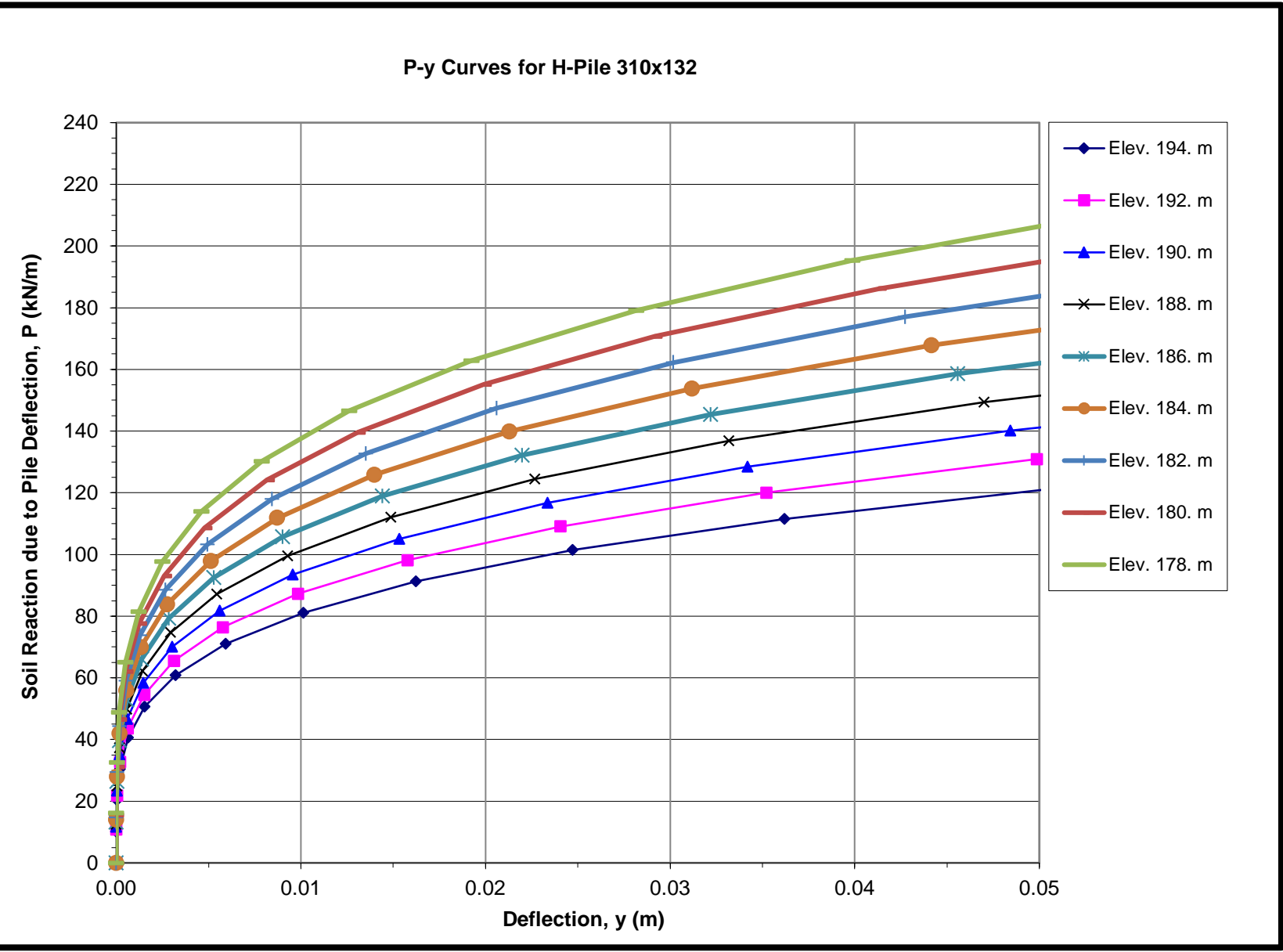
SUMMARY OF P-y CURVES FOR A H-Pile 310x132 at Abutments Assuming Pre-Auger of Upper 5 m

Description Depth (z) * Elevation P-y Curves	Stiff Clayey Silt to Silty Clay																							
	z= 24.0 m		z= 26.0 m		z= 28.0 m		z= 30.0 m		z= 32.0 m		z= 34.0 m		z= 36.0 m		z= 38.0 m		z= 40.0 m		z= 42.0 m		z= 44.0 m		z= 45.0 m	
	Elev. 194. m		Elev. 192. m		Elev. 190. m		Elev. 188. m		Elev. 186. m		Elev. 184. m		Elev. 182. m		Elev. 180. m		Elev. 178. m		Elev. 176. m		Elev. 174. m		Elev. 173. m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	10.142	0.000	10.909	0.000	11.677	0.000	12.445	0.000	13.213	0.000	13.981	0.000	14.749	0.000	15.516	0.000	16.284	0.000	22.536	0.000	22.536	0.000	22.536
	0.000	20.283	0.000	21.818	0.000	23.354	0.000	24.890	0.000	26.426	0.000	27.961	0.000	29.497	0.000	31.033	0.000	32.569	0.000	45.072	0.000	45.072	0.000	45.072
	0.000	30.425	0.000	32.727	0.000	35.031	0.000	37.335	0.000	39.638	0.000	41.942	0.000	44.246	0.000	46.549	0.000	48.853	0.000	67.608	0.000	67.608	0.000	67.608
	0.001	40.567	0.001	43.636	0.001	46.708	0.001	49.780	0.001	52.851	0.001	55.923	0.001	58.994	0.001	62.066	0.000	65.137	0.000	90.144	0.000	90.144	0.000	90.144
	0.002	50.709	0.002	54.545	0.001	58.385	0.001	62.224	0.001	66.064	0.001	69.903	0.001	73.743	0.001	77.582	0.001	81.422	0.001	112.680	0.001	112.680	0.001	112.680
	0.003	60.850	0.003	65.455	0.003	70.062	0.003	74.669	0.003	79.277	0.003	83.884	0.003	88.491	0.003	93.099	0.002	97.706	0.002	135.216	0.002	135.216	0.002	135.216
	0.006	70.992	0.006	76.364	0.006	81.739	0.005	87.114	0.005	92.489	0.005	97.865	0.005	103.240	0.005	108.615	0.005	113.990	0.003	157.752	0.003	157.752	0.003	157.752
	0.010	81.134	0.010	87.273	0.010	93.416	0.009	99.559	0.009	105.702	0.009	111.845	0.008	117.988	0.008	124.132	0.008	130.275	0.005	180.288	0.005	180.288	0.005	180.288
	0.016	91.275	0.016	98.182	0.015	105.093	0.015	112.004	0.014	118.915	0.014	125.826	0.014	132.737	0.013	139.648	0.013	146.559	0.008	202.824	0.008	202.824	0.008	202.824
	0.025	101.417	0.024	109.091	0.023	116.770	0.023	124.449	0.022	132.128	0.021	139.807	0.021	147.486	0.020	155.165	0.019	162.843	0.012	225.360	0.012	225.360	0.012	225.360
	0.036	111.559	0.035	120.000	0.034	128.447	0.033	136.894	0.032	145.341	0.031	153.787	0.030	162.234	0.029	170.681	0.028	179.128	0.018	247.896	0.018	247.896	0.018	247.896
	0.051	121.700	0.050	130.909	0.048	140.124	0.047	149.339	0.046	158.553	0.044	167.768	0.043	176.983	0.041	186.197	0.040	195.412	0.026	270.432	0.026	270.432	0.026	270.432
	0.071	131.842	0.069	141.818	0.067	151.801	0.065	161.783	0.063	171.766	0.061	181.749	0.059	191.731	0.057	201.714	0.055	211.697	0.035	292.968	0.035	292.968	0.035	292.968
	0.095	141.984	0.092	152.727	0.090	163.478	0.087	174.228	0.084	184.979	0.082	195.729	0.079	206.480	0.077	217.230	0.074	227.981	0.048	315.504	0.048	315.504	0.048	315.504
	0.125	152.126	0.122	163.636	0.118	175.155	0.115	186.673	0.111	198.192	0.108	209.710	0.104	221.228	0.101	232.747	0.097	244.265	0.063	338.040	0.063	338.040	0.063	338.040
	0.157	152.126	0.152	163.636	0.148	175.155	0.143	186.673	0.139	198.192	0.135	209.710	0.130	221.228	0.126	232.747	0.122	244.265	0.078	338.040	0.078	338.040	0.078	338.040

NOTES: \* Depth (z) is measured to be positive below the underside of the pile cap (Elevation 218.0 m).

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

1. P-y curves are generated for vertical piles (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).



Date: December 2019  
Project No: 1671430 WO 0017

Prepared By: CC  
Checked By: ARV





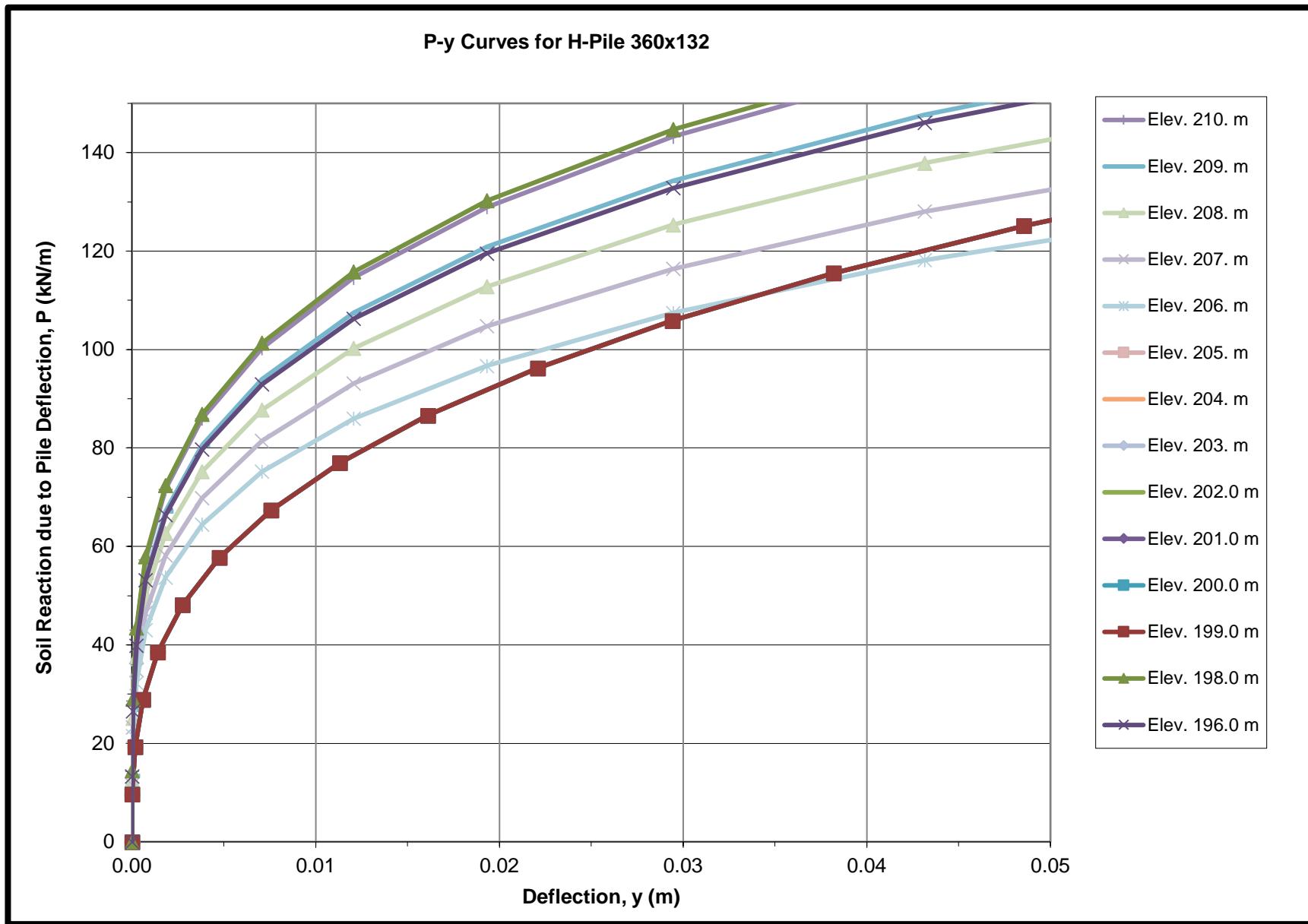
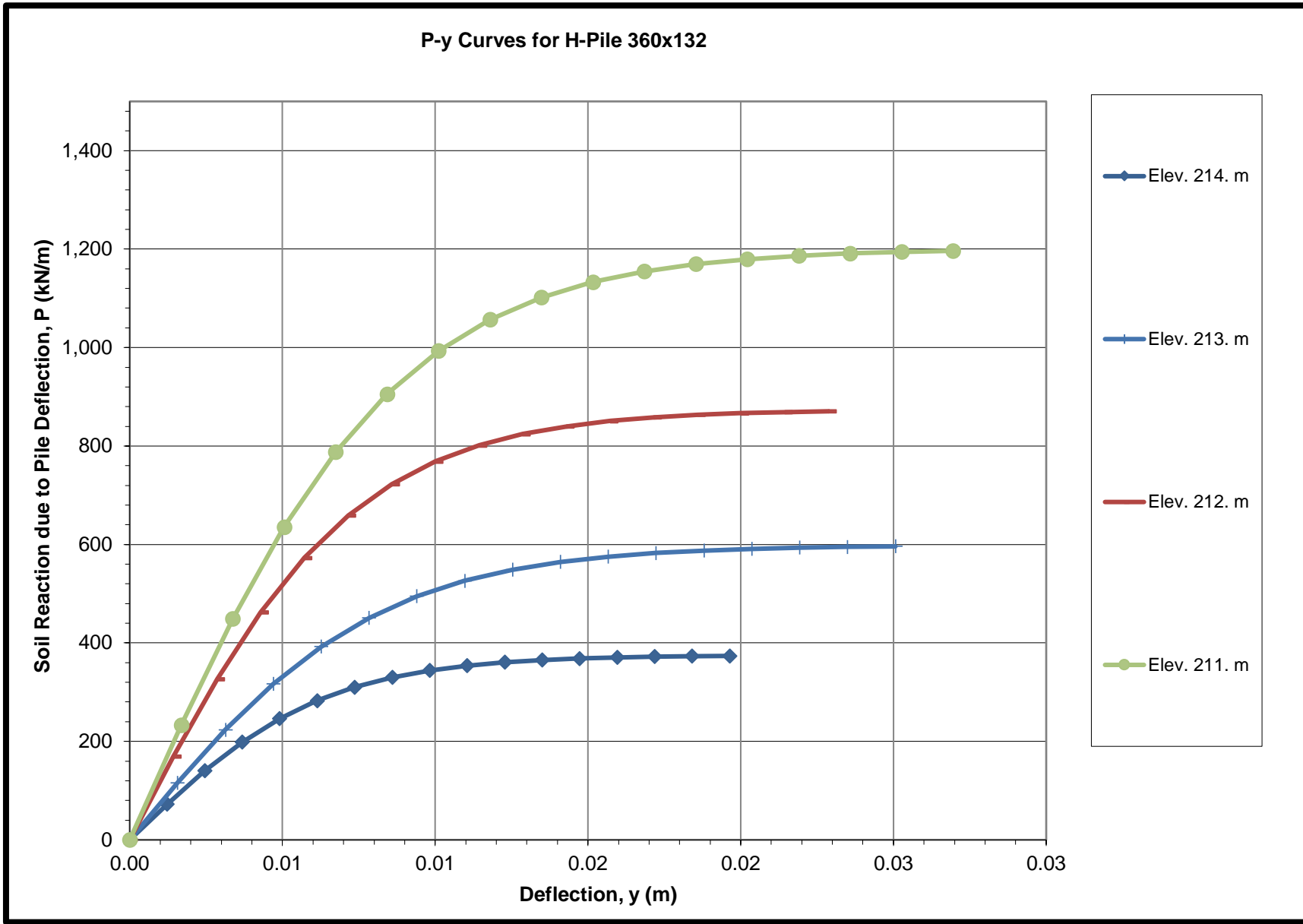
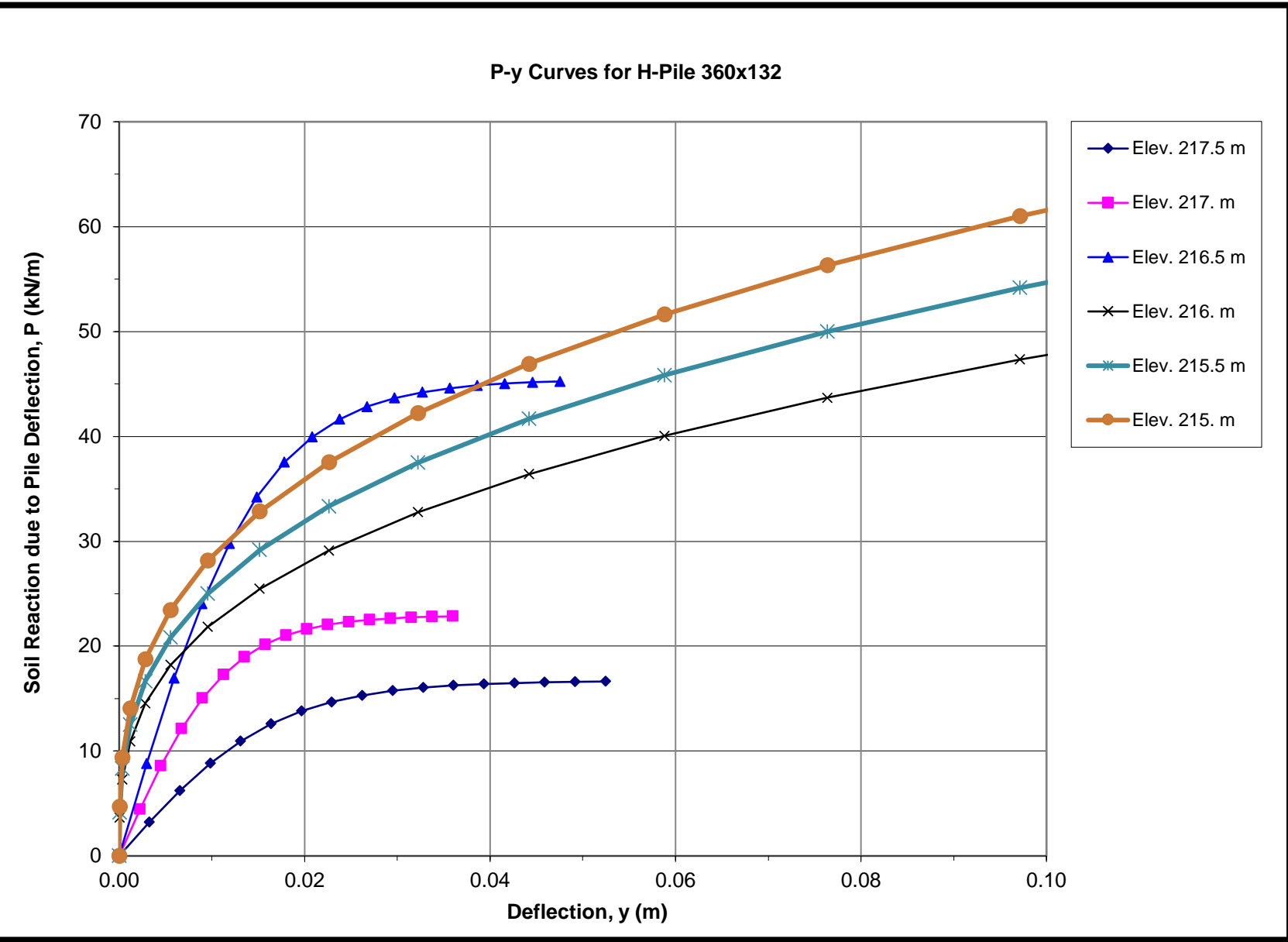
P-y CURVES  
1671430 WO 0017 Holland Drainage Canal Bridge Site 37-31 for 360x132 Pile

SUMMARY OF P-y CURVES FOR A H-Pile 360x132 at Abutments Assuming Pre-Auger of Upper 5 m

Description Depth (z) * Elevation P-y Curves	Pre-Augered Silt and Sandy to Silty Sand Fill						Pre-Augered Clayey Silt						Pre-Augered Silt and Sand to Sand				Compact to Dense Silt and Sand to Sand				Stiff Clayey Silt										Firm Clayey Silt to Silty Clay						
	z≈ 5.5 m		z≈ 1.0 m		z≈ 1.5 m		z≈ 2.0 m		z≈ 2.5 m		z≈ 3.0 m		z≈ 4.0 m		z≈ 5.0 m		z≈ 6.0 m		z≈ 7.0 m		z≈ 8.0 m		z≈ 9.0 m		z≈ 10.0 m		z≈ 11.0 m		z≈ 12.0 m		z≈ 13.0 m		z≈ 14.0 m		z≈ 15.0 m		
	Elev. 217.5 m		Elev. 217. m		Elev. 216.5 m		Elev. 216. m		Elev. 215.5 m		Elev. 215. m		Elev. 214. m		Elev. 213. m		Elev. 212. m		Elev. 211. m		Elev. 210. m		Elev. 209. m		Elev. 208. m		Elev. 207. m		Elev. 206. m		Elev. 205. m		Elev. 204. m		Elev. 203. m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.003	3.237	0.002	4.446	0.003	8.803	0.000	3.641	0.000	4.167	0.000	4.693	0.001	72.703	0.002	115.977	0.001	169.402	0.002	232.834	0.000	14.323	0.000	13.428	0.000	12.533	0.000	11.638	0.000	10.742	0.000	9.623	0.000	9.623	0.000	9.623	0.000	9.623
0.007	6.239	0.005	8.569	0.006	16.969	0.000	7.281	0.000	8.334	0.000	9.387	0.002	140.138	0.003	223.552	0.003	326.533	0.003	448.801	0.000	28.646	0.000	26.856	0.000	25.066	0.000	23.275	0.000	21.485	0.000	19.247	0.000	19.247	0.000	19.247	0.000	19.247
0.010	8.835	0.007	12.135	0.009	24.031	0.001	10.922	0.001	12.501	0.001	14.080	0.004	198.463	0.005	316.593	0.004	462.434	0.005	635.590	0.000	42.970	0.000	40.284	0.000	37.598	0.000	34.913	0.000	32.227	0.001	28.870	0.001	28.870	0.001	28.870	0.001	28.870
0.013	10.948	0.009	15.038	0.012	29.779	0.003	14.563	0.003	16.668	0.003	18.773	0.005	245.934	0.006	392.320	0.006	573.045	0.007	787.618	0.001	57.293	0.001	53.712	0.001	50.131	0.001	46.550	0.001	42.970	0.001	38.494	0.001	38.494	0.001	38.494	0.001	38.494
0.016	12.585	0.011	17.286	0.015	34.230	0.006	18.204	0.006	20.835	0.006	23.466	0.006	282.696	0.008	450.964	0.007	658.703	0.008	905.351	0.002	71.616	0.002	67.140	0.002	62.664	0.002	58.188	0.002	53.712	0.003	48.117	0.003	48.117	0.003	48.117	0.003	48.117
0.020	13.804	0.014	18.961	0.018	37.546	0.010	21.844	0.010	25.002	0.010	28.160	0.007	310.084	0.009	494.652	0.009	722.517	0.010	993.060	0.004	85.939	0.004	80.568	0.004	75.197	0.004	69.826	0.004	64.454	0.005	57.740	0.005	57.740	0.005	57.740	0.005	57.740
0.023	14.686	0.016	20.173	0.021	39.946	0.015	25.485	0.015	29.169	0.015	32.853	0.009	329.904	0.011	526.270	0.010	768.699	0.012	1056.535	0.007	100.262	0.007	93.996	0.007	87.730	0.007	81.463	0.007	75.197	0.008	67.364	0.008	67.364	0.008	67.364	0.008	67.364
0.026	15.312	0.018	21.031	0.024	41.647	0.023	29.126	0.023	33.336	0.023	37.546	0.010	343.948	0.013	548.674	0.011	801.425	0.013	1101.513	0.012	114.586	0.012	107.424	0.012	100.262	0.012	93.101	0.012	85.939	0.011	76.987	0.011	76.987	0.011	76.987	0.011	76.987
0.030	15.748	0.020	21.631	0.027	42.834	0.032	32.767	0.032	37.503	0.032	42.239	0.011	353.753	0.014	564.314	0.013	824.269	0.015	1132.912	0.019	128.909	0.019	120.852	0.019	112.795	0.019	104.738	0.019	96.682	0.016	86.611	0.016	86.611	0.016	86.611	0.016	86.611
0.033	16.050	0.023	22.045	0.030	43.654	0.044	36.407	0.044	41.670	0.044	46.933	0.012	360.525	0.016	575.118	0.014	840.050	0.017	1154.602	0.029	143.232	0.029	134.280	0.029	125.328	0.029	116.376	0.029	107.424	0.022	96.234	0.022	96.234	0.022	96.234	0.022	96.234
0.036	16.256	0.025	22.329	0.033	44.217	0.059	40.048	0.059	45.837	0.059	51.626	0.013	365.170	0.017	582.528	0.016	850.873	0.019	1169.478	0.043	157.555	0.043	147.708	0.043	137.861	0.043	128.014	0.043	118.166	0.029	105.857	0.029	105.857	0.029	105.857	0.029	105.857
0.039	16.398	0.027	22.523	0.036	44.600	0.076	43.689	0.076	50.004	0.076	56.319	0.015	368.340	0.019	587.584	0.017	858.259	0.020	1179.629	0.061	171.878	0.061	161.136	0.061	150.394	0.061	139.651	0.061	128.909	0.038	115.481	0.038	115.481	0.038	115.481	0.038	115.481
0.043	16.494	0.029	22.655	0.039	44.861	0.097	47.329	0.097	54.171	0.097	61.012	0.016	370.496	0.020	591.023	0.019	863.281	0.022	1186.532	0.084	186.202	0.084	174.564	0.084	162.926	0.084	151.289	0.084	139.651	0.049	125.104	0.049	125.104	0.049	125.104	0.049	125.104
0.046	16.559	0.032	22.744	0.042	45.039	0.121	50.970	0.121	58.338	0.121	65.706	0.017	371.958	0.022	593.356	0.020	866.690	0.024	1191.217	0.113	200.525	0.113	187.992	0.113	175.459	0.113	162.926	0.113	150.394	0.061	134.728	0.061	134.728	0.061	134.728	0.061	134.728
0.049	16.603	0.034	22.805	0.045	45.159	0.149	54.611	0.149	62.505	0.149	70.399	0.018	372.949	0.023	594.937	0.021	868.998	0.025	1194.390	0.149	214.848	0.149	201.420	0.149	187.992	0.149	174.564	0.149	161.136	0.075	144.351	0.075	144.351	0.075	144.351	0.075	144.351
0.052	16.633	0.036	22.846	0.048	45.240	0.159	54.611	0.159	62.505	0.159	70.399	0.020	373.620	0.025	596.006	0.023	870.561	0.027	1196.537	0.187	214.848	0.187	201.420	0.187	187.992	0.187	174.564	0.187	161.136	0.079	144.351	0.079	144.351	0.079	144.351	0.079	144.351

Description Depth (z) * Elevation P-y Curves	Firm Clayey Silt to Silty Clay						Stiff Clayey Silt to Silty Clay					
	z≈ 16.0 m		z≈ 17.0 m		z≈ 18.0 m		z≈ 19.0 m		z≈ 20.0 m		z≈ 22.0 m	
	Elev. 202.0 m		Elev. 201.0 m		Elev. 200.0 m		Elev. 199.0 m		Elev. 198.0 m		Elev. 196.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.00	0.000	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.000	9.623	0.000	9.62	0.000	9.62	0.000	9.623	0.000	14.472	0.000	13.279	0.000
0.000	19.247	0.000	19.25	0.000	19.25	0.000	19.247	0.000	28.945	0.000	26.558	0.000
0.001	28.870	0.001	28.87	0.001	28.87	0.001	28.870	0.000	43.417	0.000	39.836	0.000
0.001	38.494	0.001	38.49	0.001	38.49	0.001	38.494	0.001	57.890	0.001	53.115	0.001
0.003	48.117	0.003	48.12	0.003	48.12	0.003	48.117	0.002	72.362	0.002	66.394	0.002
0.005	57.740	0.005	57.74	0.005	57.74	0.005	57.740	0.004	86.834	0.004	79.673	0.004
0.008	67.364	0.008	67.36	0.008	67.36	0.008	67.364	0.007	101.307	0.007	92.952	0.007
0.011	76.987	0.011	76.99	0.011	76.99	0.011	76.987	0.012	115.779	0.012	106.230	0.012
0.016	86.611	0.016	86.61	0.016	86.61	0.016	86.611	0.019	130.252	0.019	119.509	0.019
0.022	96.234	0.022	96.23	0.022	96.23	0.022	96.234	0.029	144.724	0.029	132.788	0.029
0.029	105.857	0.029	105.86	0.029	105.86	0.029	105.857	0.043	159.196	0.043	146.067	0.043
0.038	115.481	0.038	115.48	0.038	115.48	0.038	115.481	0.061	173.669	0.061	159.346	0.061
0.049	125.104	0.049	125.10	0.049	125.10	0.049	125.104	0.084	188.141	0.084	172.624	0.084
0.061	134.728	0.061	134.73	0.061	134.73	0.061	134.728	0.113	202.614	0.113	185.903	0.113
0.075	144.351	0.075	144.35	0.075	144.35	0.075	144.351	0.149	217.086	0.149	199.182	0.149
0.079	144.351	0.079	144.35	0.079	144.35	0.079	144.351	0.187	217.086	0.187	199.182	0.187

NOTES: \* Depth (z) is measured to be positive below the underside of the pile cap (Elevation 218.0 m).  
The P-y curves have been generated based on the following assumptions:  
1. P-y curves are generated for vertical piles (i.e. no inclination)  
2. Static loading condition is considered.  
3. There are no pile group effects (i.e. analysis is based on a single pile).

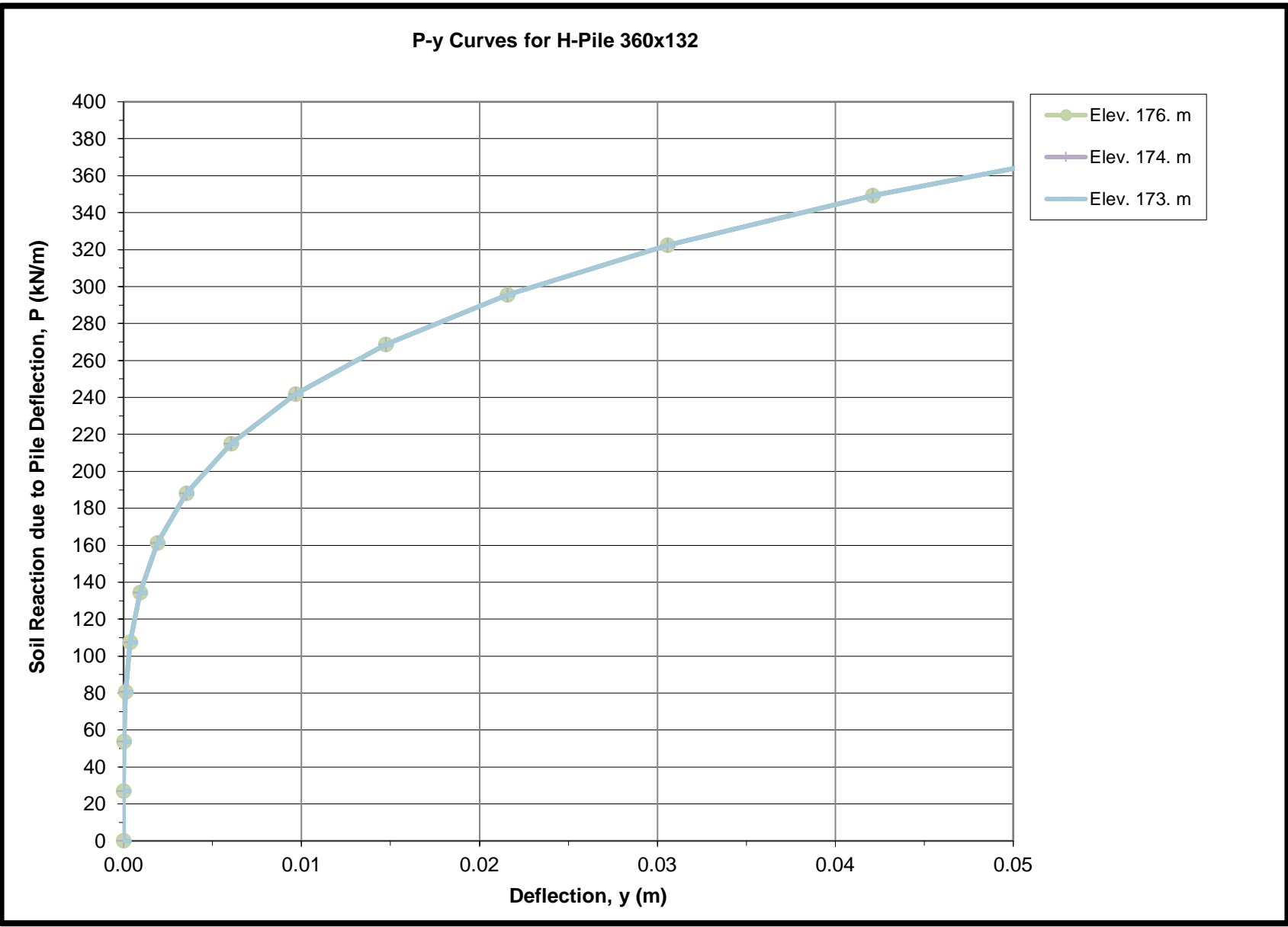
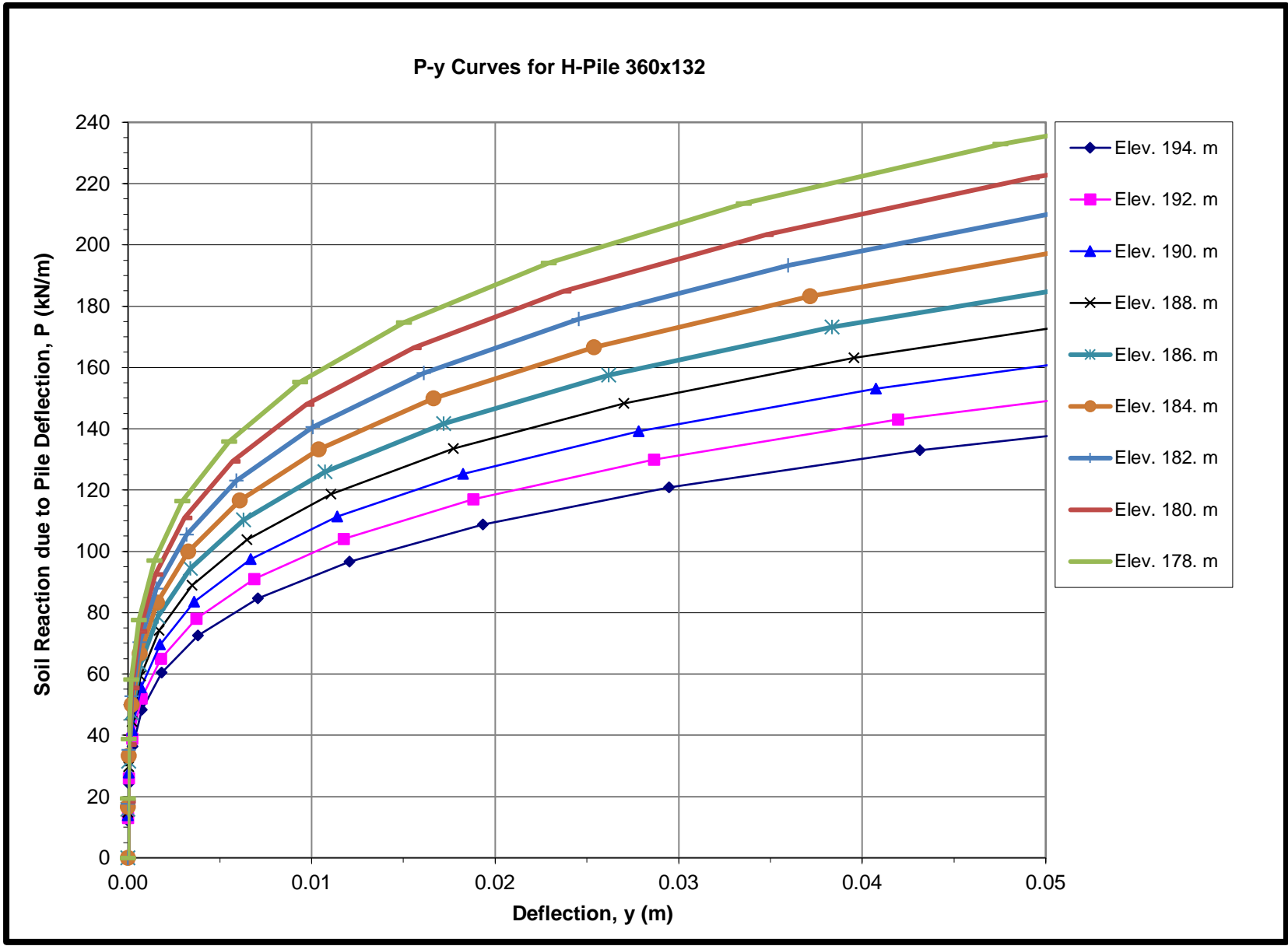




SUMMARY OF P-y CURVES FOR A H-Pile 360x132 at Abutments Assuming Pre-Auger of Upper 5 m

Description Depth (z) * Elevation P-y Curves	Stiff Clayey Silt to Silty Clay																								
	z= 24.0 m		z= 26.0 m		z= 28.0 m		z= 30.0 m		z= 32.0 m		z= 34.0 m		z= 36.0 m		z= 38.0 m		z= 40.0 m		z= 42.0 m		z= 44.0 m		z= 45.0 m		
	Elev. 194. m		Elev. 192. m		Elev. 190. m		Elev. 188. m		Elev. 186. m		Elev. 184. m		Elev. 182. m		Elev. 180. m		Elev. 178. m		Elev. 176. m		Elev. 174. m		Elev. 173. m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.000	12.086	0.000	13.000	0.000	13.915	0.000	14.830	0.000	15.746	0.000	16.661	0.000	17.576	0.000	18.491	0.000	19.406	0.000	26.856	0.000	26.856	0.000	26.856	0.000	26.856
0.000	24.172	0.000	26.001	0.000	27.831	0.000	29.661	0.000	31.491	0.000	33.321	0.000	35.152	0.000	36.982	0.000	38.812	0.000	53.712	0.000	53.712	0.000	53.712	0.000	53.712
0.000	36.257	0.000	39.001	0.000	41.746	0.000	44.491	0.000	47.237	0.000	49.982	0.000	52.727	0.000	55.473	0.000	58.218	0.000	80.568	0.000	80.568	0.000	80.568	0.000	80.568
0.001	48.343	0.001	52.001	0.001	55.662	0.001	59.322	0.001	62.982	0.001	66.643	0.001	70.303	0.001	73.963	0.001	77.624	0.000	107.424	0.000	107.424	0.000	107.424	0.000	107.424
0.002	60.429	0.002	65.001	0.002	69.577	0.002	74.152	0.002	78.728	0.002	83.303	0.002	87.879	0.001	92.454	0.001	97.030	0.001	134.280	0.001	134.280	0.001	134.280	0.001	134.280
0.004	72.515	0.004	78.002	0.004	83.492	0.004	88.983	0.003	94.473	0.003	99.964	0.003	105.455	0.003	110.945	0.003	116.436	0.002	161.136	0.002	161.136	0.002	161.136	0.002	161.136
0.007	84.601	0.007	91.002	0.007	97.408	0.006	103.813	0.006	110.219	0.006	116.625	0.006	123.030	0.006	129.436	0.006	135.842	0.004	187.992	0.004	187.992	0.004	187.992	0.004	187.992
0.012	96.686	0.012	104.002	0.011	111.323	0.011	118.644	0.011	125.965	0.010	133.285	0.010	140.606	0.010	147.927	0.009	155.248	0.006	214.848	0.006	214.848	0.006	214.848	0.006	214.848
0.019	108.772	0.019	117.003	0.018	125.238	0.018	133.474	0.017	141.710	0.017	149.946	0.016	158.182	0.016	166.418	0.015	174.654	0.010	241.704	0.010	241.704	0.010	241.704	0.010	241.704
0.029	120.858	0.029	130.003	0.028	139.154	0.027	148.305	0.026	157.456	0.025	166.607	0.025	175.758	0.024	184.909	0.023	194.059	0.015	268.560	0.015	268.560	0.015	268.560	0.015	268.560
0.043	132.944	0.042	143.003	0.041	153.069	0.040	163.135	0.038	173.201	0.037	183.267	0.036	193.333	0.035	203.399	0.034	213.465	0.022	295.416	0.022	295.416	0.022	295.416	0.022	295.416
0.061	145.030	0.059	156.004	0.058	166.985	0.056	177.966	0.054	188.947	0.053	199.928	0.051	210.909	0.049	221.890	0.048	232.871	0.031	322.272	0.031	322.272	0.031	322.272	0.031	322.272
0.084	157.115	0.082	169.004	0.079	180.900	0.077	192.796	0.075	204.692	0.072	216.589	0.070	228.485	0.068	240.381	0.065	252.277	0.042	349.128	0.042	349.128	0.042	349.128	0.042	349.128
0.113	169.201	0.110	182.004	0.107	194.815	0.104	207.627	0.101	220.438	0.097	233.249	0.094	246.061	0.091	258.872	0.088	271.683	0.057	375.984	0.057	375.984	0.057	375.984	0.057	375.984
0.149	181.287	0.145	195.004	0.141	208.731	0.137	222.457	0.133	236.184	0.128	249.910	0.124	263.636	0.120	277.363	0.116	291.089	0.075	402.840	0.075	402.840	0.075	402.840	0.075	402.840
0.187	181.287	0.181	195.004	0.176	208.731	0.171	222.457	0.166	236.184	0.161	249.910	0.155	263.636	0.150	277.363	0.145	291.089	0.093	402.840	0.093	402.840	0.093	402.840	0.093	402.840

NOTES: \* Depth (z) is measured to be positive below the underside of the pile cap (Elevation 218.0 m).  
The P-y curves have been generated based on the following assumptions:  
1. P-y curves are generated for vertical piles (i.e. no inclination)  
2. Static loading condition is considered.  
3. There are no pile group effects (i.e. analysis is based on a single pile).







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