



## FINAL REPORT

# Foundation Investigation and Design Report

*Replacement of the Highway 401 Underpass at Fraser Road,  
Site No. 31-230*

*United Counties of Stormont, Dundas and Glengarry, Ontario  
MTO WP 4290-15-01, Agreement No. 4021-E-0021*

Submitted to:

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**PART A**

# **Foundation Investigation Report**

Replacement of the Highway 401 Underpass

at Fraser Road, Site No. 31-230

United Counties of Stormont, Dundas and Glengarry, Ontario

MTO WP 4290-15-01, Agreement No. 4021-E-0021

## 1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd., (Golder) now a member of WSP Canada Inc.) has been retained by the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with the replacement of the existing Highway 401 Underpass structure at Fraser Road (Site No. 31-230).

The scope of work for the foundation investigation services associated with this project was carried out as part of MTO Agreement No. 4021-E-0021, Assignment No. 1, delivered in association with MTO WP 4290-15-01.

Golder carried out geotechnical investigation and foundation design services for the Department of Highways (now MTO) for the existing structure in 1965, as well as a more recent preliminary foundation investigation and design for the proposed replacement in 2018. The results of both of these previous studies have been reviewed and used to augment the results of the current detailed foundation design investigation. The results of the previous investigations are contained in the following reports:

- Report on “*Foundation Investigation and Design, Preliminary Design of replacement of Fraser Road Underpass, Highway 401, Site No. 31-230, United Counties of Stormont, Dundas and Glengarry, Ontario, GWP 4248-15-00, WP 4290-15-01*”, by Golder Associates Ltd., dated May 2019. GEOCRES 31G5-273.
- Report on “*Soil Conditions and Foundations, Proposed Fraser Road Underpass, Highway 401, Glengarry County, Ontario, WP 107-59*”, by H.Q. Golder & Associates, dated January 1966. GEOCRES 31G00 142.

## 2.0 SITE DESCRIPTION

The Fraser Road site (Site No. 31-230) is located at about Station 23+050 on Highway 401 approximately 4.4 km west of Lancaster in the United Counties of Stormont, Dundas and Glengarry (SDG), Ontario. The site location is shown in the key plan in Drawing 1.

At this location, Highway 401 has a four-lane cross-section with two eastbound and two westbound through lanes with paved shoulders separated by a wide, vegetated median. A gravel median is present approximately 50 m east and west of the centre pier of the overpass structure. Steel beam guiderails are also present along both sides of the highway in the vicinity of the underpass structure and surrounding the centre pier. There are no interchange ramps at this location.

Fraser Road is an undivided secondary road with a rural cross-section and a single travel lane in each direction that carries traffic over Highway 401. Concrete parapet walls with steel railings are present along the bridge and steel beam guiderails are present along both sides of Fraser Road beyond the bridge. The land surrounding the structure site is agricultural, with a generally flat topography.

The existing bridge was constructed in late 1968 under MTO Contract 67-18. The existing construction drawings for the bridge indicate that the structure consists of a four-span, prestressed, precast concrete girder, and a cast-in-place deck slab structure, with the abutments founded on concrete-filled pipe (tube) piles (0.6 m outside diameter with about 9 mm wall thickness), and the piers founded on 12 BP 53 steel piles. The existing structure is aligned on an approximate 5° skew northwest to southeast and is about 89.6 m long and 10.4 m wide.

The existing bridge embankments are approximately 8 m in height above the natural ground level with the top of the abutments at about Elevation 56 m. The existing approach embankment side slopes are variable with slope profiles ranging from about 2.1 horizontal to 1 vertical (2.1H:1V) to 2.7H:1V. For stability reasons, the embankment fill was provided with large toe berms on the front and side slopes of the approaches; the existing toe berm(s) is about 4 m in height with the crest of the berms at about Elevation 53 m, and about 16 m to 18 m in

length. The front slope toe berms have forward slopes at 1.5H:1V, immediately adjacent to the existing piers, and the side slope toe berms have sides sloped at approximately 2H:1V.

Based on visual observations at the time of the current site investigation, the existing embankment side slopes appear to be performing satisfactorily.

The existing embankment loading over the native foundation strata which contains a sensitive and compressible clay deposit resulted in post-construction settlements of the approach embankments following the completion of the original bridge. Based on the available documentation from MTO (GEOCRE 31G00-192), settlement readings on the approach embankments taken a few years following construction measured up to about 0.3 m of settlement at that time, which necessitated restoration of the approach pavement structure. The bridge itself has not settled as the structure is founded on deep foundations supported on bedrock.

Selected site photographs taken by WSP Golder personnel showing the existing structure and surrounding area at the time of the current investigations are included in Appendix E.

## **3.0 INVESTIGATION PROCEDURES**

### **3.1.1 2023 Detailed Design Investigation Summary**

The detailed design subsurface investigation was carried out between January 30 and February 12, 2023, and included advancing eight boreholes (numbered 22-01A/B, 22-02A/B, 22-03A/B, 22-04A/B) and four seismic cone penetration tests (SCPT) (numbered SCPT 22-01C to SCPT 22-04C). The investigation locations are shown in Drawings 1 and 2.

Boreholes 22-01A/B to 22-04A/B were advanced using 200 mm outside diameter hollow-stem augers with a CME55 track-mounted drilling rig, supplied and operated by George Downing Estate Drilling Ltd. of Hawkesbury, Ontario. A water truck was on site to supply the drilling rig with water for coring the bedrock. Traffic control required to close the driving lanes and shoulders of Fraser Road was provided by Beacon Lite Limited of Ottawa, Ontario in accordance with the Ontario Traffic Manual, Book 7, Temporary Conditions.

Soil samples were obtained in Boreholes 22-01A to 22-04A using a 50 mm outer diameter split-spoon sampler in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). Soil samples were obtained at vertical sampling intervals of about 0.76 m and 1.5 m. In-situ vane testing was carried out within the cohesive deposits, using an MTO N-size vane, with the reaction (torque) measured by a pair of calibrated scales, to measure the undrained shear strength of the cohesive soils. After measuring the undrained shear strength, remoulded shear strengths were also measured at selected intervals. Bedrock coring was carried out in Boreholes 22-01A and 22-03A using a rotary diamond drilling technique and a triple-tube core-barrel to retrieve NQ3-sized rock core samples.

Boreholes 22-01B to 22-04B were advanced unsampled through the existing non-cohesive fill and upper native granular overburden soils to the top of the cohesive clay stratum (at approximate Elevations 47.5 m to 48.7 m). These boreholes were then further advanced to the underside of the clay strata (to approximate Elevations 41.1 m to 44.8 m) to retrieve relatively undisturbed 73 mm diameter thin-walled Shelby tube samples of the clay using a fixed piston sampler. Shelby tube samples were obtained at vertical sampling intervals of about 0.76 m, throughout the clay strata. To minimize disturbance in the boreholes advanced for Shelby tube sampling, the hollow-stem augers were filled with water to maintain pressure at the base. Although the use of drilling mud would have increased the density of the fluid in the boreholes and applied additional pressure at the base, no evidence of base heave was noted during the drilling or sampling which would have indicated excessive disturbance to the



soil below the augers within the sampling zone. The use of a fixed piston sampler mitigated the risk of sample loss and specimen 'stretching' and ensured maximum recovery of the soil specimens within the Shelby tubes. In our opinion, the drilling and sampling methods utilized at this site resulted in predominantly high-quality specimens with no justification to attempt the use of specialty soil samplers, such as the Sherbrooke sampler, which is primarily used for research (if at all) and generally not commercially available. Further justification on the quality of the Shelby tube specimens retrieved from the site is provided in the sample 'Rating' discussion included in Sections 4.3.5 and 4.3.6.

A monitoring well was installed in Boreholes 22-01B, 22-03A, and 22-03B to measure the groundwater level at the site. The monitoring wells consist of 52 mm outside diameter PVC tubing with a 1.5 m long slotted screen with the well screened sealed within the silty clay to clay, the silty sand till, and the silty sand fill and native silt strata in Boreholes 22-01B, 22-03A and 22-03B respectively. Well, installation details are shown on the borehole records for Boreholes 22-01B, 22-03A, and 22-03B provided in Appendix A. The water levels in the monitoring wells were measured on March 7, 2023.

The detailed design field investigation program also included four SCPTs. SCPT 22-01C to SCPT 22-04C were advanced with an M5T CPT track-mounted rig drilling rig supplied and operated by ConeTec Investigations Ltd. (ConeTec) of Richmond Hill, Ontario. The details of the seismic cone penetration testing are described below. Full details on the testing procedures and the test results can be found in ConeTec's report provided in Appendix F.

The SCPT testing was carried out with a 15 cm<sup>2</sup> tip base area probe, with an equal end area friction sleeve, and tip and sleeve capacities of 1,000 bar and 10 bar, respectively. At each SCPT test location, a borehole was first advanced by drilling through the existing embankment fill to the top of the clay layer based on the depths encountered at the adjacent boreholes. The tip resistance, shaft friction, and pore water pressures were measured at approximately 0.025 m depth intervals within the cohesive layers. The SCPT was advanced until encountering practical refusal to cone tip advancement at depths ranging from about 8.5 m to 16.5 m below the existing ground surface. Shear wave velocity (Vs) measurements were carried out as part of the seismic cone penetration test to collect interval velocities within the cohesive layer. A total of 27 pore water pressure dissipation tests were carried out in the SCPTs at approximately 1.0 m intervals within the cohesive strata. The groundwater levels at the SCPT locations were inferred based on the pore water pressure measurements taken during advancement.

All boreholes without monitoring well, including the upper portion of the CPT test locations drilled through the embankment fill, were backfilled with bentonite mixed with soil cuttings in general accordance with the intent of Ontario Regulation (O.Reg.) 903, as amended. The site conditions were restored following the completion of the field work.

The field work was supervised on a full-time basis by members of WSP Golder's technical staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers, and transported to WSP Golder's laboratories in Mississauga and Ottawa for further examination and laboratory testing. Index and classification tests consisting of grain size distribution, Atterberg limits, and water content testing were carried out on selected soil samples and uniaxial compressive strength (UCS) testing was carried out on selected samples of the bedrock. High-complexity geotechnical laboratory testing for this project included: ten incremental loading and three long-term loading consolidation (OED) tests, three consolidated isotropic drained (CID) triaxial tests, and three consolidated isotropic undrained (CIU) triaxial tests with pore pressure measurements, performed on

selected samples of the sensitive clay. All laboratory testing was carried out to MTO and/or ASTM standards as appropriate.

Eight soil samples were sent to Eurofins Environmental Testing Canada Inc. (Eurofins) for basic chemical analysis related to the potential corrosion of buried steel elements and sulfate attack on buried concrete elements (corrosion and sulphate attack).

The borehole locations and elevations were surveyed by WSP Golder using a Trimble R10 GPS unit referenced to the NAD83 CSRS CBNv6-2010.0 MTM Zone 9 geodetic datum. The borehole and SCPT test locations, including northing and easting coordinates as well as geographic coordinates, ground surface elevations, and drilled depths are summarized in Table 1.

**Table 1: Summary of Borehole and SCPT Locations**

Borehole	NAD83 CSRS CBNv6-2010.0 MTM Zone 9		Ground Surface Elevation (m)	Drilled Depths (m)	Comments
	Northing (m) (Latitude)	Easting (m) (Latitude)			
22-01A	4997614.3 (45.113720°)	222585.9 (-74.544880°)	52.9	18.9	Includes 3.6 m of Bedrock Coring
22-01B	4997616.0 (45.113740°)	222584.5 (-74.544900°)	52.9	8.1	Terminated in Till layer
22-01C	4997615.5 (45.113744°)	222586.2 (-77.544879°)	52.9	8.5	CPT Refusal
22-02A	4997592.7 (45.113530°)	222607.2 (-74.544610°)	56.0	21.1	Auger Refusal
22-02B	4997591.9 (45.113520°)	222603.3 (-74.544660°)	56.0	12.3	Terminated in Till layer
22-02C	4997590.7 (45.113523°)	222609.1 (-77.544584°)	55.8	15.2	CPT Refusal
22-03A	4997676.2 (45.114280°)	222569.8 (-74.545090°)	52.7	18.2	Includes 5.4 m of Bedrock Coring
22-03B	4997676.6 (45.114280°)	222571.1 (-74.545080°)	52.7	11.6	Terminated in Till layer
22-03C	4997677.9 (45.114303°)	222569.2 (-77.545105°)	52.7	11.1	CPT Refusal
22-04A	4997698.0 (45.114470°)	222548.7 (-74.545370°)	55.9	17.7	Terminated in Till layer
22-04B	4997699.3 (45.114480°)	222547.9 (-74.545380°)	55.9	14.6	Terminated in Till layer
22-04C	4997696.2 (45.114465°)	222549.0 (-77.545365°)	56.0	16.5	CPT Refusal
18-1101	4997670.5 (45.114230°)	222557.1 (-77.545260°)	52.8	20.9	Includes 7.2 m of Bedrock Coring
18-1102	4997639.0 (45.113940°)	222573.6 (-77.545040°)	50.6	17.8	Includes 4.7 m of Bedrock Coring
18-1103	4997614.4 (45.113730°)	222587.4 (-77.544860°)	53.0	21.0	Includes 5.7 m of Bedrock Coring
18-1103A	4997612.2 (45.113710°)	222588.7 (-77.544840°)	53.1	11.3	Terminated in Till layer
18-1103B	4997611.2 (45.113700°)	222587.4 (-77.544860°)	52.9	7.8	Auger Refusal
CPT18-1101	4997668.9 (45.114220°)	222554.5 (-77.545290°)	52.8	10.6	CPT Refusal
CPT18-1103	4997610.3 (45.113697°)	222586.8 (-77.544870°)	52.9	8.9	CPT Refusal

### 3.1.2 2018 Preliminary Design Investigation Summary

Golder previously carried out a preliminary foundation investigation at the site in 2018 as part of the assessment of the proposed replacement alternatives for the Highway 401/Fraser Road underpass. A total of five boreholes that included bedrock coring and two CPTs were advanced at the site as part of the preliminary investigation. In particular, Boreholes 18-1101 to 18-1103 and 18-1103A/B were advanced near the existing foundation unit locations for the north abutment, central pier, and south abutment, respectively. One monitoring well was installed in Borehole 18-1103A to measure the groundwater level at that location on the site. At the time of the 2023 investigation, the monitoring well in Borehole 18-1103A was still viable and the water level was recorded along with the measurements in the new wells installed during the detailed investigation.

The preliminary field investigation also included in-situ hydrogeological testing of the overburden soils at Borehole 18-1103A and packer testing of the bedrock at Borehole 18-1103. In addition to the borehole investigation, Vertical Seismic Profiling (VSP) was also completed at the site at Boreholes 18-1101 and 18-1103 for seismic site characterization.

A copy of the Borehole Location and Soil Strata Drawings and Record of Boreholes relevant to the current investigation are provided for reference in Appendix C. The borehole locations from the preliminary investigation, including northing and easting coordinates as well as geographic coordinates, ground surface elevations, and drilled depths are also summarized in Table 1. The approximate borehole locations are shown in Drawings 1 and 2 provided in Appendix C. In general, the stratigraphy at the boreholes encountered during the 2018 investigation is characterized by an asphalt pavement structure and granular embankment fill, overlying successive layers of native silt and sand, sensitive clay, and silty sand to sand and gravel glacial till, underlain by limestone bedrock.

Full details of the preliminary foundation investigation results are available in GEOCRE Report 31G5-273.

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geologic Conditions

As delineated in *The Physiography of Southern Ontario*<sup>1</sup>, this section of Highway 401 lies within the major physiographic region known as the Lancaster Flats.

The Lancaster Flats region is characterized by relatively thick deposits of sensitive marine clay, silt, and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock. This region is underlain by a series of sedimentary rocks, consisting of limestones and shales that are, in turn, underlain at depth by igneous and metamorphic bedrock of the Precambrian Shield.

The soft and compressible Leda clay deposit that exists at this site, which is known to underlie a large portion of Highway 401 from about Cornwall and extending eastwards beyond the Québec border, will be a key consideration in the foundation design.

### 4.2 General

The subsurface soil, bedrock, and groundwater conditions encountered in the boreholes during the current investigation along with the results of in-situ testing are shown on the Records of Boreholes and Drillholes

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<sup>1</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. Ontario Ministry of Natural Resources.

presented in Appendix A. Photographs of the core recovered from the underlying bedrock at Boreholes 22-01A and 22-03A are shown in Figures A1 to A4, provided in Appendix A. The results of the geotechnical laboratory testing carried out during the investigation are shown on the borehole records and the details are included in Figures B1 to B11 in Appendix B. The results of the basic chemical testing/analysis completed on a selected soil sample are provided in Appendix D.

The borehole and SCPT locations and the interpreted stratigraphic profiles are shown in Drawings 1 and 4.

The SCPT results including profiles of the corrected tip resistance ( $q_t$ ), sleeve friction ( $f_s$ ), porewater pressure ( $u$ ), shear wave velocity ( $V_s$ ), and compression wave velocity ( $V_p$ ) measured during pushing and the corresponding calculated friction ratio ( $R_f$ ), and Soil Behaviour Type (SBT) are presented in the SCPT sounding records included in the ConeTec Investigation Ltd. Report presented in Appendix F.

The stratigraphic boundaries shown on the borehole and drillhole records and the interpreted stratigraphic section in Drawings 1 and 4, are inferred from observations of the drilling progress and noncontinuous sampling and therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

### 4.3 Site Stratigraphy Overview

In general, the subsurface conditions at the site consist of asphaltic concrete and pavement structure, or topsoil surface cover, over granular embankment fill, underlain by a native soil stratum comprised of silt and sand, over a weathered clay crust, over grey silty clay to clay, over granular glacial till all underlain by limestone bedrock.

A more detailed description of the overburdened soil deposits and bedrock geology conditions encountered during the field investigation is provided in the following sections. In the discussion, emphasis is placed on the subsurface conditions from the boreholes advanced during the current investigation and previous 2018 investigation which are in general agreement with the 1966 Geocres information. The 1966 Geocres information is referenced herein only in regard to the clay parameters in Section 4.3.5 and the bedrock surface elevation in Section 4.3.8.

#### 4.3.1 Surface Cover/ Surficial Materials

Surficial topsoil was encountered at the ground surface in Boreholes 22-01A, 22-03A, 18-1101, and 18-1103, with thickness ranging from about 100 mm to 200 mm.

Asphaltic concrete with thicknesses of 300 mm and 200 mm was encountered at the surface at Boreholes 22-02A and 22-04A, respectively.

#### 4.3.2 Pavement Structure and Embankment Fill

A silty sand and gravel pavement structure fill was encountered below the asphaltic concrete in Boreholes 22-02A and 22-04A. The top of this layer was encountered at Elevations 57.0 m and 55.7 m with thickness of 0.7 m and 0.2 m in Boreholes 22-02A and 22-04A, respectively. A single SPT N-value recorded in the pavement structure fill was 71 blows per 0.3 m of penetration indicating the very dense state of compactness. The measured water content of one sample of the pavement structure fill was 3%. The results of grain size analysis testing carried out on a single sample of the pavement structure fill material are provided in Figure B1 in Appendix B.

Embankment fill consisting of silty sand and gravel, gravelly sand to sandy gravel, gravelly silty sand and gravelly sandy silt was encountered below the topsoil and pavement structure in Boreholes 22-01A to 22-04A, 18-1101, 18-1103 and at ground surface at Borehole 18-1102. The top of this layer was encountered at elevations ranging from 50.6 m (Borehole 18-1102 located at existing pier) to 55.5 m (Borehole 22-04A located in Fraser Road). The total thickness of the fill layer as encountered in the boreholes ranges from about 0.6 m to 7.4 m. The SPT N-values recorded in the fill range from 1 blow to greater than 100 blows per 0.3 m of penetration, but more typically 21 to 40 blows were measured in this material indicating a compact to dense state of compactness. The higher blow counts (e.g., 56 blows per 80 mm) recorded in the fill may have been influenced by the presence of cobbles or boulders within the fill, rather than the compactness of the soil matrix.

The measured water contents of 19 samples of the granular fill tested ranged from 5% to 18%. The results of grain size analysis testing carried out on eight samples of the gravelly silty sand fill material are provided in Figure B2 in Appendix B. The results of Atterberg limits testing completed on the fines portion from two samples of the granular fill material indicate liquid limits of 18 and 20, plastic limits of 14 and 15, and plasticity indices of 4 and 5. These Atterberg limits testing results indicate the fill contains fines of slight plasticity. The Atterberg limits analysis results are shown in Figure B3 in Appendix B.

#### **4.3.3 Buried Topsoil**

A buried layer of topsoil was encountered beneath the granular embankment fill in Boreholes 22-01A, 22-03A, 22-04A, 18-1101, and 18-1102 at elevations ranging from 48.3 m to 48.5 m. The thickness of the buried topsoil ranges from about 0.2 m to 0.8 m at the borehole locations. The measured water content of one sample of buried topsoil was 40%.

#### **4.3.4 Silt to Silty Sand to Sandy Silt**

A layer of silt to silty sand to sandy silt was encountered below the buried topsoil and granular embankment fill in all boreholes, except Borehole 18-1103. The top of this layer was encountered at elevations ranging from 48.0 m to 49.2 m. The thickness of this layer ranges from about 0.3 m to 0.9 m. A single SPT N-value recorded in this layer was 9 blows per 0.3 m of penetration indicating a loose state of compactness.

The measured water contents of five samples tested range from 26% to 29%. The results of grain size analysis testing carried out on three samples of the silt to silt and sand layer are provided in Figure B4 in Appendix B.

#### **4.3.5 Silty Clay to Clay (Weathered Crust)**

A thick deposit of cohesive soil was encountered beneath the granular embankment fill and/or the silt to silty sand to sandy silt layer at all boreholes locations.

The upper portion of the cohesive deposit is a grey-brown, weathered silty clay to clay crust. The top of this layer was encountered at elevations ranging from 47.5 m to 48.4 m. The thickness of this layer ranges from about 0.6 m to 1.8 m. The SPT N-values recorded in this layer range from weight of hammer (WH) to 6 blows per 0.3 m of penetration. In-situ field vane testing carried out within the silty clay to clay crust measured undrained shear strengths ranging from about 50 kPa to >100 kPa. Based on the results of the SCPT testing, the undrained shear strength in the silty clay to clay crust ranges from about 50 kPa to >100 kPa. The results of the field vane and SCPT testing indicate the silty clay to clay crust has a stiff consistency.

The results of grain size analysis testing carried out on two samples of weathered clay crust are shown in Figure B5 in Appendix B. The results of Atterberg limits testing completed on ten samples of this material indicate liquid limits ranging from 37 to 91, plastic limits ranging from 17 to 32 and plasticity indices ranging from 19 to 64. The



Atterberg limits test results are shown in Figure B6 in Appendix B and indicate the weathered crust is comprised of a silty clay of intermediate plasticity (CI) to clay of high plasticity (CH). The water content of 14 samples of the clay crust tested range from 28% to 78% which is above the plastic limit, and in a few cases above the liquid limit, of this material.

One laboratory consolidated, undrained triaxial compression test (CIU) with pore pressure measurement and one drained triaxial compression test (CID) were carried out on samples of the weathered clay. In total, one set of three specimens (CIU) and one specimen (CID) were tested. The results of the triaxial testing are shown in Figures B7 and B8 in Appendix B and are summarized in Table 2.

**Table 2: Summary of Triaxial Testing – Silty Clay to Clay (Weathered Crust)**

Borehole	Sample	Ground Surface Elevation (m)	Test Elevation (m)	Test	Peak		Post Peak/ Softened	
					Effective Cohesion Intercept, $c'$ (kPa)	Effective Angle of Internal Friction, $f'$ (°)	Effective Cohesion Intercept, $c'$ (kPa)	Effective Angle of Internal Friction, $f'$ (°)
22-01B	1	52.9	48.3	CIU	0 <sup>1</sup>	34 <sup>1</sup>	0 <sup>2</sup>	26 <sup>2</sup>
22-03B	1	52.7	47.9	CID	0 <sup>3</sup>	37 <sup>3</sup>	-	-

**Notes:**

Assessed shear strength parameters are only valid over a range of stress conditions in the test

<sup>1</sup>Specimen A, B and C

<sup>2</sup>Specimen B and C

<sup>3</sup>Specimen A

Laboratory oedometer consolidation testing with incremental loading was carried out on four samples of the weathered silty clay to clay deposit in accordance with ASTM D2435-04. A slight modification to the Load Increment Ratio (LIR) was utilized for the loading stages around the estimated preconsolidation pressure to provide additional resolution to the change in slope of the  $e$ - $\log \sigma'$  curve and aid in a better definition of the value of  $\sigma'_p$ . This procedure is routinely followed by WSP (and formerly by Golder) when conducting consolidation tests on the structured, sensitive clays in the Eastern Region.

An assessment of sample disturbance was undertaken for the four specimens selected for testing in accordance with the procedure outlined in Lunne et. al. (1997). Based on this procedure, the Sample Quality Category (or Rating) is indicated to be very good to excellent for three of the four samples and good to fair for one sample. The result of the assessment is summarized in Table 3.

**Table 3: Assessment of Sample Disturbance – Silty Clay to Clay (Weathered Crust)**

Borehole	Sample	Ground Surface Elevation (m)	Test Elevation (m)	$e_0$	$\Delta e/e_0$	OCR	Sample Quality Category
22-01B	2	52.9	47.4	1.92	0.01	3.4	Very good to excellent
22-01B	3	52.9	46.9	1.72	0.004	1.9	Very good to excellent
22-02B	2	56.0	47.8	0.83	0.036	2.7	Good to fair
22-04B	1	55.9	47.2	1.81	0.01	1.7	Very good to excellent

The preconsolidation pressures were estimated from the void ratio versus logarithmic stress plot ( $e$ - $\log \sigma'$ ) using the Casagrande method as well as using the Work method (after Becker et al., 1987). The results of the consolidation testing are shown in Figures B9 to B12 in Appendix B and are summarized in the Table 4.

**Table 4: Summary of Incremental Loading Consolidation Testing – Silty Clay to Clay (Weathered Crust)**

Borehole	Sample	Ground Surface Elevation (m)	Test Elevation (m)	Unit Weight, $\gamma'$ ( $\text{kN/m}^3$ )	$e_o$	$\sigma'_p$ (kPa)	$\sigma'_{vo}$ (kPa)	$\sigma'_p - \sigma'_{vo}$ (kPa)	$C_c$	$C_r$	$C_v$ ( $\text{cm}^2/\text{s}$ )	$C_{vr}$ ( $\text{cm}^2/\text{s}$ )	OCR
22-01B	2	52.9	47.4	15.5	1.92	285	85	200	1.06	0.024	-	1.2E-02	3.4
22-01B	3	52.9	46.9	16.1	1.72	175	90	85	1.26	0.018	1.0E-04	4.0E-03	1.9
22-02B	2	56.0	47.8	19.0	0.83	405	150	255	0.23	0.024	-	1.5E-02	2.7
22-04B	1	55.9	47.2	15.7	1.81	265	160	105	0.87	0.030	8.0E-04	7.0E-03	1.7

**Notes:**

$\sigma'_p$  Estimated preconsolidation pressure

$\sigma'_{vo}$  Estimated in-situ vertical effective stress

$C_c$  Compression index

$C_r$  Recompression index

$C_v$  Co-efficient of Consolidation (normally consolidated stress range from 230 kPa to 310 kPa)

$C_{vr}$  Co-efficient of Consolidation (over-consolidated stress range from 20 kPa to 310 kPa)

$e_o$  Initial void ratio

OCR Overconsolidation ratio

Based on the results of the in-situ and laboratory testing, the weathered silty clay to clay crust is considered lightly over-consolidated to over-consolidated.

### 4.3.6 Clay (Unweathered)

The lower portion of the cohesive deposit below the weathered crust is an unweathered grey clay. The top of the unweathered clay was encountered at elevations ranging from 46.1 m to 47.5 m. The thickness of this layer where fully penetrated ranges from about 2.1 m to 5.9 m. Borehole 18-1103B was terminated in this layer. In the original 1966 investigation boreholes, the unweathered grey clay was encountered at elevations ranging from 46.6 m to 47.5 m, with thickness ranging from 2.0 m to 5.9 m.

The SPT N-values recorded in this layer range from weight of hammer (WH) and/or pressure manual (PM) to 2 blows per 0.3 m of penetration. In-situ field vane testing carried out within the unweathered clay measured undrained shear strengths ranging from about 20 kPa to 80 kPa but typically 30 kPa to 50 kPa. Based on the results of the SCPT testing, the undrained shear strength in the unweathered clay ranges from about 30 kPa to 50 kPa. The results of the field vane and SCPT testing indicate the unweathered clay generally has a firm consistency. The ratio of the measured in-situ undisturbed undrained shear strength to the remolded undrained shear strength typically ranges from 4 to 8 based on the field vane testing, indicating a sensitive clay in accordance with Section 3.1.3.4 of CFEM (2006).

The results of grain size analysis testing carried out on three samples of the unweathered clay are shown in Figure B13 in Appendix B. The results of Atterberg limits testing completed on 17 samples of this material indicate liquid limits ranging from 40 to 85, plastic limits ranging from 18 to 28 and plasticity indices ranging from 22 to 56. The Atterberg limits test results are shown in Figures B14 and B15 and indicate this stratum is generally clay of high plasticity (CH). The water contents of 25 samples of the grey clay ranged from 34% to 94% which is above the plastic limit and generally close to or above the liquid limit of this material.

One laboratory consolidated, undrained triaxial compression test (CIU) with pore pressure measurement and two drained triaxial compression test (CID) were carried out samples of the unweathered grey clay from the current investigation. In total, one set of three specimens (CIU), one set of two specimens (CID) and one set of three specimens (CID) were tested. During the original (1966) investigation CIU triaxial testing was carried out on three specimens of the unweathered grey clay. The results of the triaxial testing are shown in Figures 16 to 19 in Appendix B and are summarized in Table 5.

**Table 5: Summary of Triaxial Testing – Clay (Unweathered)**

Borehole	Sample	Ground Surface Elevation (m)	Test Elevation (m)	Type of Test	Peak		Post Peak/ Softened	
					Effective Cohesion Intercept, $c'$ (kPa)	Effective Angle of Internal Friction, $\phi'$ (°)	Effective Cohesion Intercept, $c'$ (kPa)	Effective Angle of Internal Friction, $\phi'$ (°)
22-01B	4	52.9	46.3	CID	5 <sup>1</sup>	15 <sup>1</sup>	-	-
22-03B	2	52.7	47.2	CID	20 <sup>2</sup>	15 <sup>2</sup>	-	-
22-03B	4	52.7	46.0	CIU	7 <sup>3</sup>	18 <sup>3</sup>	0 <sup>4</sup>	21 <sup>4</sup>
1	5	49.0	44.1	CIU	23	15	0	18
2	4	50.6	46.3					
4	7	50.6	44.6					

**Notes:** Assessed shear strength parameters are only valid over a range of stress conditions in the test

<sup>1</sup>Specimen A, B and C

<sup>2</sup>Specimen B and C

<sup>3</sup>Specimen A, B, and C

<sup>4</sup>Specimen A, B and C

Laboratory oedometer consolidation testing with incremental loading was carried out on nine samples of the unweathered grey clay deposit in general accordance with ASTM D2435-04. As noted above a slight modification to the Load Increment Ratio (LIR) was utilized for the loading stages around the estimated preconsolidation pressure in order to provide additional resolution to the change in slope of the  $e$ - $\log \sigma'$  curve and aid in a better definition of the value of  $\sigma'_p$ . This procedure is routinely followed by WSP (and formerly by Golder) when conducting consolidation tests on the structured, sensitive clays in the Eastern Region.

An assessment of sample disturbance was undertaken for the nine specimens selected for testing in accordance with the procedure outlined in Lunne et. al. (1997). Based on this procedure, the Sample Quality Category (or Rating) is indicated to be very good to excellent for all samples of the unweathered clay. The result of the assessment is summarized in Table 6.

**Table 6: Assessment of Sample Disturbance – Clay (Unweathered)**

Borehole	Sample	Ground Surface Elevation (m)	Test Elevation (m)	$e_0$	$\Delta e/e_0$	OCR	Sample Quality Category
22-01B	5	52.9	45.6	1.89	0.0001	1.4	Very good to excellent
22-02B	6	56.0	45.0	1.78	0.01	1.4	Very good to excellent
22-03B	3	52.7	46.4	2.71	0.01	1.1	Very good to excellent
22-03B	5	52.7	45.2	2.36	0.002	1.0	Very good to excellent
22-03B	8	52.7	43.4	2.11	0.037	1.4	Very good to excellent
22-04B	6	55.9	44.0	1.77	0.016	0.9	Very good to excellent
18-1101	9	52.8	46.2	2.42	0.009	1.0	Very good to excellent
18-1101	10	52.8	44.9	2.19	0.018	1.2	Very good to excellent
18-1103B	1	52.9	45.4	2.02	0.009	1.4	Very good to excellent

The preconsolidation pressures were estimated from the void ratio versus logarithmic stress plot ( $e$ - $\log \sigma'$ ) using the Casagrande method as well as using the Work method (after Becker et al., 1987). The results of the consolidation testing are shown in Figures B20 to B28 in Appendix B and are summarized in Table 7.

**Table 7: Summary of Incremental Loading Consolidation Testing – Clay (Unweathered)**

Borehole	Sample	Ground Surface Elevation (m)	Test Elevation (m)	Unit Weight, $\gamma'$ (kN/m <sup>3</sup> )	$e_o$	$\sigma'_p$ (kPa)	$\sigma'_{vo}$ (kPa)	$\sigma'_p - \sigma'_{vo}$ (kPa)	$C_c$	$C_r$	$c_v$ (cm <sup>2</sup> /s)	$c_{vr}$ (cm <sup>2</sup> /s)	OCR
22-01B	5	52.9	45.6	15.7	1.89	145	100	41	1.24	0.030	4.0E-05	6.0E-03	1.4
22-02B	6	56.0	45.0	16.1	1.78	240	165	74	1.09	0.024	1.1E-04	6.0E-03	1.4
22-03B	3	52.7	46.4	15.0	2.34	105	95	7	1.88	0.041	1.0E-04	9.0E-03	1.1
22-03B	5	52.7	45.2	15.0	2.36	110	105	4	2.07	0.044	4.0E-05	4.0E-03	1.0
22-03B	8	52.7	43.4	15.2	2.11	125	90	36	1.37	0.046	7.0E-05	4.0E-03	1.4
22-04B	6	55.9	44.0	16.0	1.77	165	185	-	0.97	0.025	7.0E-05	5.0E-03	0.9
18-1101	9	52.8	46.2	14.9	2.42	110	105	4	2.00	0.024	2.0E-05	7.0E-03	1.0
18-1101	10	52.8	44.9	15.1	2.19	140	115	24	1.86	0.015	5.0E-05	7.0E-03	1.2
18-1103 B	1	52.9	45.4	15.6	2.03	175	125	-	1.57	0.019	5.0E-05	5.0E-03	1.4

**Notes:** $\sigma'_p$  Estimated preconsolidation pressure $\sigma'_{vo}$  Estimated in-situ vertical effective stress $C_c$  Compression index $C_r$  Recompression index $c_v$  Co-efficient of Consolidation (normally consolidated stress range from 115 kPa to 398 kPa) $c_{vr}$  Co-efficient of Consolidation (over-consolidated stress range from 5 kPa to 229 kPa) $e_o$  Initial void ratio

OCR Overconsolidation Ratio

Based on the results of the in-situ and laboratory testing, the unweathered clay is considered to be normally consolidated to lightly over-consolidated.

Long-term laboratory consolidation (creep) testing with the load held constant for about 4 to 5 weeks was carried out on two samples of the unweathered grey clay from the current investigation. The load (or stress level) held in the creep test was based on the estimated average final effective stress within the clay stratum under the new approach embankment height. During the preliminary design (2018) investigation, long-term laboratory consolidation (creep) testing was carried out on one specimen of the unweathered grey clay. The results of the long-term creep testing are shown in Figures B29 to B31 in Appendix B and are summarized in the Table 8.

**Table 8: Summary of Long-Term Loading Consolidation Testing – Clay (Unweathered)**

Borehole	Sample	Ground Surface Elevation (m)	Test Elevation (m)	Unit Weight (kN/m <sup>3</sup> )	C <sub>αε</sub>
22-01B	5	52.9	45.9	15.3	0.008
22-03B	7	52.7	44.2	15.1	0.006
18-1101	10	52.8	44.9	15.1	0.007

**Notes:**C<sub>αε</sub>: Modified Secondary Compression Index

A summary of the engineering properties of the clay deposit are shown on the plots in Figures B32 and B33 provided in Appendix B.

### 4.3.7 Glacial Till

A granular glacial till was encountered below the unweathered clay at all borehole locations, except Borehole 18-1103B, where the clay layer was not fully penetrated. The glacial till is generally described as consisting of sand and gravel, gravelly silty sand, sandy gravel with a trace to some clay, and containing boulders and cobbles. The top of this layer was encountered at elevations ranging from 41.4 m to 45.0 m, and the layer is about 1.4 m to 4.3 m thick, where glacial till was fully penetrated. Boreholes 22-01B, 22-02A/B, 22-03B, 22-04A/B, and 18-1103A were terminated in this layer. The recorded SPT N-values within the glacial till range from 3 blows to 100 blows per 0.3 m penetration, suggesting very loose to very dense compactness condition, however, the blow counts have likely been influenced by the presence of cobbles, boulders, or the proximity to the bedrock surface rather than the actual compactness of the soil matrix.

The measured water contents of 19 samples tested ranged from 7% to 17%. The results of grain size analysis testing carried out on seven samples of the gravelly silty sand to silty sand and gravel till are shown in Figure B34 in Appendix B. The results of Atterberg limits testing completed on two samples of the glacial till material indicate liquid limits of 13 and 15, plastic limits of 12 and 13, and plasticity indices of 2 and 1. These Atterberg limits test results indicate the glacial till contains fines of slight plasticity. The Atterberg limits analysis results are shown in Figure B35 in Appendix B.

Interbedded deposits of sand and sand and gravel were encountered within the glacial till in Boreholes 22-01A, 22-04A, 18-1102, and 18-1103. The top of this layer was encountered at elevations ranging from 39.4 m to 42.3 m. The thickness of this layer ranges from about 0.3 m to 0.6 m. The recorded SPT N-values within the sand and sand and gravel range from 13 to 47 blows per 0.3 m of penetration, suggesting compact to dense compactness.

The measured water contents of two samples tested of the interbedded deposits of sand were about 10% and 11%.

### 4.3.8 Bedrock

The overburden materials are underlain by limestone bedrock.

Bedrock core samples were obtained in Boreholes 22-01A, 22-03A, 18-1103, 18-1102 and 18-1103 using a triple-tube HQ3- or NQ3-sized core barrel.

Table 9 summarizes the depths and the elevations of the bedrock surface as encountered at the borehole locations during the current, preliminary 2018 and original 1966 investigations at the site.



**Table 9: Summary of Bedrock Surface Depths and Elevations**

Borehole	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation <sup>1</sup> (m)
22-01A	52.9	15.3	37.6
22-03A	52.7	12.8	39.9
18-1101	52.8	13.7	39.1
18-1102	50.6	13.1	37.5
18-1103	53.0	15.3	37.7
1	49.0	11.4	37.6
2	50.6	12.4	38.2
3	49.6	13.1	36.5
4	50.6	10.8	39.8
5	49.1	11.3	37.9

**Note(s):** 1. Bedrock surface elevation confirmed by rock coring.

The bedrock encountered in the boreholes consists of fresh to slightly weathered, thinly to medium bedded, medium grey to dark grey to black, fine grained limestone with occasional shale interbeds. Rock Quality Designation (RQD) values measured on the recovered limestone bedrock core samples range from about 10% to 100%, but are typically 64% to 99%, indicating a fair to excellent rock quality.

The results of uniaxial compressive strength (UCS) testing carried out on three bedrock core samples from the current investigation and three from the preliminary investigation are summarized in Figure B36 in Appendix B. The UCS testing carried out for the current investigation indicates compressive strength values of 53 MPa and 59 MPa, indicating a strong bedrock. UCS testing carried out for the preliminary investigation indicated compressive strength values of 22 MPa and 37 MPa, indicating a weak to medium strong bedrock.

## 4.4 Groundwater Condition

A monitoring well was installed in Boreholes 22-01B, 22-03A/B, and 18-1103A to measure the groundwater level at these locations at the site. The groundwater levels measured in the monitoring well are presented in Table 10.

It is expected that the groundwater levels will be subject to fluctuations both seasonally and as a result of precipitation events.

**Table 10: Summary of Groundwater Conditions**

Borehole	Screened Interval	Ground Surface Elevation (m)	Ground Water Depth (m)	Ground Water Elevation (m)	Date
22-01B	Weathered Crust/ Clay	52.9	3.6	49.3	March 7, 2023
22-03A	Glacial Till / Bedrock	52.7	3.7	49.0	March 7, 2023
22-03B	Inferred Topsoil/ Inferred Silt/ Weathered Crust	52.7	3.3	49.4	March 7, 2023
18-1103A	Glacial Till	53.1	4.0	49.1	March 7, 2023
			6.1	47.0	September 18, 2018

## 4.5 Steel Corrosion and Sulphate Attack, Chemical Analysis

Eight soil samples from the current investigation were submitted to Eurofins for chemical testing/analysis related to the potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The test results are provided in Appendix D and are summarized in Table 11.



**Table 11: Steel Corrosion and Sulphate Attack, Chemical Analysis**

Borehole	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
22-01A	1.52-2.13	<0.002	0.04	0.46	7.46	2174
22-01A	7.62-8.23	0.011	0.02	0.38	7.86	2632
22-02A	3.05-3.66	0.007	0.01	0.33	7.73	3030
22-02A	16.76-17.37	0.003	0.03	0.43	7.71	2326
22-03A	2.29-2.89	0.003	0.03	0.47	7.67	2128
22-03A	9.14-9.75	<0.002	0.02	0.30	8.00	3333
22-04A	3.05-3.66	0.010	0.05	0.62	7.58	1613
22-04A	14.63-15.24	0.004	0.02	0.32	7.89	3125



## 5.0 CLOSURE

This Foundation Investigation Report was prepared by Kinjal Gajjar and reviewed by Kenton Power, P.Eng., a senior geotechnical engineer with WSP Golder. Paul Dittrich, P.Eng., a Fellow and Senior Geotechnical Principal of WSP and an MTO Foundations Designated Contact for WSP Golder, conducted an independent technical and quality review of this report.

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[https://wsponline.sharepoint.com/sites/gld-158306/project files/6 deliverables/22513877a fraser rd/2-final/4021-e-0021 rev0 final fidr fraser rd up \(22513877a\) 2024-03-04.docx](https://wsponline.sharepoint.com/sites/gld-158306/project%20files/6%20deliverables/22513877a%20fraser%20rd/2-final/4021-e-0021%20rev0%20final%20fidr%20fraser%20rd%20up%20(22513877a)%202024-03-04.docx)

**PART B**

# Foundation Design Report

Replacement of the Highway 401 Underpass

at Fraser Road, Site No. 31-230

United Counties of Stormont, Dundas and Glengarry, Ontario

MTO WP 4290-15-01, Agreement No. 4021-E-0021

## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides geotechnical and foundation design input associated with the detailed design of the proposed replacement of the Highway 401 underpass at Fraser Road near Cornwall, Ontario as part of MTO WP 4290-15-01. The input provided herein is based on interpretation of the factual data obtained from the boreholes advanced during the current and previous subsurface investigations at the site, and in accordance with the current Canadian Highway Bridge Design Code CSA S6:19 (CHBDC). In accordance with Section 4.4.2 of the CHBDC, we understand that the proposed bridge structure has an importance category of *Other Bridge*.

The Foundation Design Report (Part B of this report) including the discussion and recommendations are intended for the use of MTO and their designers for the design and shall not be relied upon for any other purpose or by any other parties, including the future construction contractor or design-build proponents. Contractors undertaking the work must make their own interpretation based on the factual data presented in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the concept and design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of detail design and construction must make their own interpretation of the factual information provided and supplement as necessary, as such interpretation may affect detail design, equipment selection, proposed construction methods, scheduling and the like.

### 6.1 Project Understanding

Based on the preliminary General Arrangement (GA) drawing dated October 2019, provided for reference after text of this report, the proposed structure replacement will be maintained along the existing Fraser Road alignment. The existing four-span structure is proposed to be replaced with a two-span structure with a total length of approximately 68 m, with a centre pier located in the Highway 401 median. The new north and south abutments are proposed to be located approximately 11 m front of the existing abutments; it is anticipated that the existing abutments will need to be removed and that the new abutments will be founded on driven steel piles, while the new pier will be supported on caissons (drilled shafts).

It is our understanding that Highway 401 will be widened from four lanes to six lanes at the location of the Fraser Road structure. We further understand that the existing Highway 401 grade will be maintained at approximate Elevation 50.6 m (elevation at the existing pier Borehole 18-1102), while the Fraser Road grade will be raised by approximately 0.65 m to about Elevation 57.5 m at the north abutment and Elevation 57.4 m at the south abutment as noted on GA drawing, such that the new approach embankments are up to approximately 7 m in height at the abutments above the Highway 401 surface.

Based on the new configuration shown in the preliminary GA, the area under the current north and south spans on top of the existing front slope toe berm will be backfilled to accommodate the construction of the new approach embankments. The additional embankment loading including the proposed grade raise over the thick sensitive and compressible clay deposit will result in settlements requiring mitigation measures to be adopted as part of the design as well as potentially monitoring and maintenance post-construction.

## 6.2 General Foundation Design Context

### 6.2.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of CHBDC 2019 and its Commentary, the bridge structure and its foundation system may be classified as having low 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.

Based on the level of foundation investigation completed to date at this location (see Part A of this report) in comparison to the degree of site understanding, the level of confidence for design of the bridge foundation elements and approach embankments has generally been assessed as a “high” degree of site and prediction model understanding. Accordingly, the ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor,  $\Psi$  and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$ , for a high degree of site understanding, from Tables 6.1 and 6.2 of CHBDC 2019 have been used for the design.

For seismic design, the consequence factor  $\Psi$  and resistance factor,  $\phi_{gu}$  should be taken as unity, and the geotechnical resistance factor shall be as specified in Table 6.3. as per Section 6.14.4 of CHBDC (2019).

### 6.2.2 Seismic Design

The seismic hazard values associated with the design earthquakes are those established for the National Building Code of Canada (NBC 2020) by the Geological Survey of Canada (GSC). The current seismic hazard maps (referred to as the 6<sup>th</sup> generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2020.

#### 6.2.2.1 Seismic Site Classification

During 2018 investigation, VSP geophysical testing was carried out at two locations, immediately west of the existing north and south embankments respectively at Boreholes BH18-1101 and BH18-1103, to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site. The results of the VSP geophysical testing are provided in Golder’s Technical Memorandum dated November 2018 in Appendix I. The shear wave velocities measured indicate that the average shear wave velocity in the upper 30 m of the subsurface stratigraphy is 404 m/s and 459 m/s adjacent to the north and south embankments, respectively. Based on these results and using Table 4.1 of the CHBDC, a Site Class of C may be used the design of the structure.

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

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the proposed structure, the Class C peak seismic hazard values based on data obtained from Earthquakes Canada ([www.earthquakescanada.nrcan.gc.ca](http://www.earthquakescanada.nrcan.gc.ca)) are provided in Table 12.



**Table 12: Site Class C Spectral Values for Subject Site**

Parameter	2% Probability of Exceedance in 50 years (2,475-year return period) (g)
PGA	0.460
Sa(0.2)	0.843
Sa(0.5)	0.489
Sa(1.0)	0.256
Sa(2.0)	0.114
Sa(5.0)	0.0294
Sa(10.0)	0.00945
PGV [m/s]	0.335

From correspondence with MTO it is understood that the design fundamental period the structure has not been determined and will depend on the final design of the superstructure; however, MTO has recommended that it be assumed to be between 0.5 s and 0.75 s for the purposes of the analyses in this assignment. Based on this, and in consideration of the structure's "Other" importance category and the site-specific seismic hazard values given in Table 12, in accordance Table 4.10 of the CHBDC the bridge would fall in a Seismic Performance Category (SPC) 2.

### 6.2.2.3 Liquefaction and Cyclic Mobility Assessment

An assessment of the potential for the near surface silty sand to liquefy and the underlying clayey soils to undergo cyclic mobility has been carried out as part of the seismic response analysis of the approach embankments as discussed in Section 6.7.

### 6.2.3 Frost Protection

Strip footings and/or pile caps should be found at a minimum depth of 1.7 m below the lowest surrounding final grade, including any distance measured perpendicular to a sloping ground surface if applicable, to provide adequate protection against frost penetration (as interpreted from OPSD 3090.101).

## 6.3 Foundation Options

Based on the subsurface conditions at the site, only deep foundation options have been considered for the replacement of the existing structure, as shallow foundations would not provide sufficient bearing resistances or acceptable settlement performance for the new structure.

A summary of the advantages and disadvantages associated with each deep foundation option is provided below.

- **Driven steel H-piles:** Steel H-piles driven to refusal on the limestone bedrock are feasible for support of the replacement bridge structure. This option would provide high geotechnical resistances and minimal post-construction settlements; in addition, this option would permit the use of integral abutments. The use of driving shoes is recommended to minimize damage while penetrating the glacial till deposit (which is

expected to contain cobbles and boulders) and seating onto the limestone bedrock. A single row of vertical piles is also likely to minimize conflicts with the existing abutment piles; however, according to the Department of Highways, Fraser Road Underpass, General Plan, Drawing No. D5888-1, dated Feb. 1966, it appears that the ends of the existing wing walls are supported on piles, and the potential for conflicts between the new and existing piles in this area will need to be checked as part of the detail structural design. Driven steel piles are also feasible at the centre pier, although construction of battered piles for the pier would likely require more working space and present more constraints to traffic staging during construction as compared with the use of drilled shafts (caissons). Also driven piles at the pier would require a below-grade pile cap for support of the structural columns which would likely conflict with existing foundation elements.

- *Driven steel pipe (tube) piles:* Closed-ended steel tube (pipe) piles driven to refusal on the limestone bedrock could also be considered as a deep foundation option for support of the abutments and central pier. This foundation option would have similar advantages/disadvantages to steel H-piles in terms of high geotechnical resistances and minimal settlements as well as potential conflicts with existing foundation elements. This option may also permit the use of integral abutments. However, pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered/orientation if cobbles and/or boulders are encountered within the till deposits during driving.
- *Drilled shafts (caissons):* Rock socketed, cast-in-place concrete caissons deriving their support from bearing within the limestone bedrock are also feasible for this site. This foundation option is considered feasible at the pier, however, conventional caissons are likely not preferred at the abutments as they would preclude the use of an integral abutment design. However, it is understood that MTO Foundations has considered the use of a hybrid H-pile/caisson configuration feasible for integral abutments. In such a design the pile cap is supported with foundation elements consisting of an H-pile encased in a corrugated steel pipe (CSP) backfilled with loose sand consistent with the MTO Structural Office Report SO-96-01 to allow for structural movement. The H-pile is then embedded within a concrete caisson that extends through the overburden and is founded in the underlying bedrock. This hybrid system would provide a higher axial geotechnical resistance associated with the larger diameter caisson while also allowing the movement required for an integral abutment design (i.e., via the H-pile). Caisson construction will require the use of temporary liners and/or other measures such as polymer slurry to stabilize the drill hole(s) and mitigate the potential risk of ground loss from squeezing or flowing clay or heave of water-bearing cohesionless soils during drilling. In addition, concrete placement by tremie methods is expected to be required based on the groundwater level at the site and the need to advance the casing through the non-cohesive gravelly silty sand (till) stratum. The rock socket of the caisson could be advanced by rock coring and/or chisel drilling and the base of the socket will require cleaning and proper inspection (i.e., using a SQUID) to check that minimal debris is left in the base prior to concrete placement. Finally, with the use of larger diameter drilled shafts (caissons), there would be a higher risk of conflict with the existing piles during installation (depending on layout) and this would need to be checked by the structural designer as part of the overall assessment of risk associated with this foundation type.

### 6.3.1 Feasibility of Integral Abutments

As outlined in MTO Structural Office Report SO-96-1, dated July 1996, integral abutment bridges are single span or multiple span continuous deck type bridges with a movement system composed primarily of abutments on flexible integral foundations and approach slabs, in lieu of movable deck expansion joints and expansion bearings at abutments.

The feasibility of integral abutments is influenced by a number of factors, including structure geometry and subsurface conditions. The primary criterion is the need to support the abutments on relatively flexible piles. Integral abutments are not recommended for sites where the soil is susceptible to liquefaction, slip failure, sloughing or boiling. Where the load bearing stratum is near the surface or where the use of short piles or caissons (less than 5 m in length) is planned, the site would similarly not be considered suitable for integral abutment bridges. Geometric constraints on the use of integral abutments are also applicable and include: overall bridge length less than 150 m; skew angle less than 35°; and abutment wall heights less than 6 m without a retained soil system.

For integral abutment design, based on the subsurface conditions in the vicinity of the abutments, corrugated steel pipes (CSPs) backfilled with loose sand are recommended to be installed consistent with the MTO Structural Office Report SO-96-01.

From a foundation perspective, integral abutments are considered feasible at this location.

### **6.3.2 Downdrag and Drag Loads**

The construction of the new approach fills between the original abutment areas and the new abutments will raise the effective stress level in the clay deposit, leading to consolidation and settlement of the deposit. As discussed previously, settlement and downward movement of the cohesive deposit relative to the stiff, end-bearing abutment piles will result in downdrag (negative skin friction) and drag loads on the piles. Although the use of lightweight EPS fill within the backfill zone behind the abutments will reduce the post-construction settlements (see Section 6.6.2.3 of this report) the magnitudes of settlement will still be large enough to result in drag loads on the piles. Since the construction constraints at this site will not allow the settlements to be completed prior to pile installation, the drag loads must be considered in the structural design of the piles.

Since there is no grade raise proposed at the central pier, no drag loads are anticipated on deep foundations at the pier location.

### **6.3.3 Recommended Foundation Types**

Based on the evaluation of foundation alternatives discussed above, the preferred approach from a foundations perspective is to support new abutments on driven steel piles and the central pier on drilled shafts socketed into the limestone bedrock respectively. The structural designers will need to determine the location and installation batter/orientation of the existing deep foundation elements at the abutments and center pier to access the risk of conflict/interference posed during installation of the proposed foundation elements. A Notice to Contractor highlighting the risk of interference with existing piles is included in Appendix K.

## **6.4 Deep Foundations**

### **6.4.1 Driven Steel H-Pile Foundations**

The following two standard steel H-pile sizes have been considered in the deep foundation assessment. The heavier section is provided due to its higher structural resistance which may be necessary to accommodate the estimated drag loads:

- 310 x 110 steel H-pile
- 310 x 132 steel H-pile

The new bridge abutments may be founded on either pile size noted above driven to refusal in/on sound bedrock. The estimated pile lengths required to support the abutments, assuming approximate underside of pile cap at Elevation 53 m as indicated on the preliminary GA drawing are summarized in Table 13.

Acknowledging the potential constraints (i.e., working space, traffic staging, below-grade pile cap inference with existing foundation elements, requirements for shoring and dewatering, etc.) noted in Section 6.3, it is also considered technically feasible from a foundation's perspective to support the centre pier on driven steel H-piles. The estimated pile lengths required to support the centre pier, assuming the approximate underside of pile cap of 1.7 m below existing grade (frost penetration depth) at Elevation 48.8 m have been summarized in Table 13.

**Table 13: Estimated Pile Tip Elevations – Abutments and Centre Pier**

Location	Approximate Underside of Pile Cap Elevation (m)	Estimated Pile Tip Elevations <sup>1</sup> (m)	Estimated Pile Lengths (m)
North abutment	51.0	38.2 to 39.9	12.8 to 11.1
South abutment		34.9 to 37.7	16.1 to 13.4
Centre Pier	48.8	36.5 to 37.9	12.3 to 10.9

**Note:**

<sup>1</sup> Bedrock surface elevation at the reference borehole location

Piles must be installed in accordance with OPSS.PROV 903 Section 903.07.02.07.03.03 Piles to Bedrock. The following note (Note 5 from the MTO Structural Manual, Section 3.3.3 (MTO, 2016)), should be shown on the Contract Drawings:

**“PILES TO BE DRIVEN TO BEDROCK”**

For the installation of the steel H-piles, consideration must be given to the presence of cobbles and boulders within the lower portions of the fill and within the granular till stratum above the bedrock. In addition, sloping bedrock, and a varying depth to the bedrock surface should be anticipated. It is recommended that piles be reinforced at the tip with driving shoes and/or flange plates in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe), to reduce the potential for damage to the piles during driving. As a result, piles should be fitted with appropriate driving shoes and rock point as per OPSS.PROV 903 Section 903.07.02.02 (Driving Shoes and Rock Points); the APF Hard-Bite points (Model: HP-77750-B) or equivalent are recommended for this site. Suggested wording for a Nonstandard Special Provisions (NSSPs) is provided in Appendix K to alert the Contractor of the potential for the presence of obstructions (cobbles and boulders) in the overburden as well as the sloping bedrock.

#### **6.4.1.1 Factored Geotechnical Axial Resistance – Steel Piles**

Based on the estimated uniaxial compressive strength of the bedrock at this site and assuming good to excellent rock quality, the following factored axial geotechnical resistance can be used in the design:

- 310x110 steel H-pile – factored geotechnical axial resistance at ULS of 3,375 kN per pile.
- 310x132 steel H-pile – factored geotechnical axial resistance at ULS of 3,900 kN per pile.

The factored ULS axial geotechnical resistance may be greater than the structural capacity of the pile, which could govern design and should be checked by the structural design engineer.

SLS does not apply to piles driven to and founded on or in the type of bedrock at this site, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

#### **6.4.1.2 Drag Loads – Abutment Piles**

The structural design of the piles should be based on full drag loads acting on the piles. The following estimated drag loads acting on a single pile can be used in the design, assuming the upper 3 m portion of the pile below the pile cap is surrounded by a CSP filled with loose sand, and that the CSP is not connected to the pile cap:

- 310x110 steel H-pile – unfactored downdrag load 350 kN per pile.
- 310x132 steel H-pile – unfactored downdrag load 360 kN per pile.

The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.14.6.3 of the CHBDC. Since the steel piles are to be driven to bedrock, the effect of the drag loads on the settlement (or SLS geotechnical axial resistance) of the piles will be negligible.

#### **6.4.2 Drilled Shaft (Caisson) Foundations**

##### **6.4.2.1 Caisson Installation**

The preliminary GA drawing indicates that 1.5 m diameter caissons (drilled shafts) socketed into the limestone bedrock will be used to support the pier. It is understood that the structural designers may consider the use of a hybrid H-pile/caisson configuration at the abutments. The following sections include geotechnical discussion and design recommendations for caissons for both the centre pier and abutments. As the structural design has not been finalized it is anticipated that 1.5 m diameter caissons will be used at the abutments similar to those shown on the existing Preliminary GA Drawing for the centre pier.

The native marine (Champlain Sea) clay at this site is a sensitive soil and as such the disturbed clay could “flow” into the auger hole during drilled shaft installation if left unsupported. Furthermore, water-bearing non-cohesive soils within the glacial till stratum under hydrostatic head conditions are present above the bedrock. Where caisson foundations are adopted, temporary liners will be required for construction (to a depth to be determined by the contractor) and these should be advanced while filled with a head of water or polymer slurry to minimize the potential for non-cohesive materials (“flowing sands”) to migrate into the drillhole, to control base disturbance / basal heave due to groundwater pressures/seepage, and to minimize ground loss.

It is expected that the temporary liners/casings would be installed using rotation methods, although a vibratory hammer may be feasible for some portions of the installation. If a vibratory hammer is used, vibration monitoring of the existing utilities is recommended. Casing installation through the bouldery glacial till deposits is expected to require rotary drilling methods, and churn drilling or down-hole hammer techniques may also be required to advance the caisson to the required depth if and where boulders are encountered and to form the rock socket in the bedrock. Based on the subsurface conditions at the site, segmental casing could be used for support of the sidewalls through the overburden and could be seated into the upper bedrock prior to drilling the rock socket, if deemed necessary by the Contractor. The segmental casing would be extracted during concrete placement by tremie methods, with appropriate access tubes or TIP instrumentation installed within the caisson reinforcing steel cage to evaluate the integrity of the concrete.

Based on the groundwater conditions encountered at the time of the investigation tremie concreting techniques are expected to be required for concrete placement.

Caissons must be installed in accordance with OPSS.PROV 903 and given the subsurface conditions at this site it is recommended that MTO's Special Provision for drilled shafts also be incorporated into the contract documents. This NSSP has been provided in Appendix K and should be included in the Contract Documents as it addresses among other items the requirements for the use of temporary casings/liners and slurry for the installation of drilled shafts, the placement of concrete by tremie methods, and cleaning and inspection of the base of the drilled shafts. Suggested wording has also been provided in Appendix K for a Notice to Contractor (NOC) to advise the Contractor of the soil and groundwater pressures / seepage conditions present at this site.

Given that the drilled shaft capacities provided below in Section 6.4.2.3 have a significant end-bearing component, the performance of the drilled shafts in compression relies on the quality of the rock and upon the final cleaning and verification of the condition of the drilled rock socket at the base of the caisson. As such, it is imperative that the rock surface be adequately cleaned of all loose cuttings, rock, and debris so that the concrete is in intimate contact with the base of the socket within the limestone bedrock. It is recommended that provision be made for the caisson bases to be inspected using a Shaft Quantitative Inspection Device (SQUID), as weighted tape soundings or the use of a downhole camera may not be adequate in at this depth of caisson in wet or slurried conditions. Should the inspection indicate that loosened material is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected. A Notice to Contractor has been added to Appendix K to advise the Contractor that SQUID techniques are to be used for inspection of the caisson bases.

With the use of temporary liners over this depth of caisson in water-bearing soil conditions, it is important to verify the integrity of the concrete. It is recommended that either or both of Thermal Integrity Profiling (TIP) instrumentation and/or Crosshole Sonic Logging (CSL) tubes be installed in each caisson or in a selected proportion of the caissons to allow for such verification. Specifications related to CSL testing are included in MTO's Non-Standard Special Provision for drilled shafts (caissons) as included in Appendix K; further development of specifications for TIP testing would be required if MTO elects to incorporate TIP in the production drilled shafts.

#### 6.4.2.2 *Founding Elevations*

The estimated shaft lengths required to found the caissons in the limestone bedrock, are summarized in Table 14.

**Table 14: Estimated Caisson Tip Elevations – Centre Pier and Abutments**

Location	Approximate Underside of Pile Cap Elevation <sup>2</sup> (m)	Existing Ground Surface (m)	Bedrock Surface Elevation (m)	Estimated Shaft Length <sup>1</sup> (m)
Central Pier	N/A <sup>3</sup>	50.6	36.4 to 37.5	15.2 to 14.1
North Abutment	≈50 <sup>4</sup>	N/A	39.9 to 37.8	11.1 to 13.2
South Abutment			37.7 to 34.9	13.3 to 16.1

**Note:**

<sup>1</sup> From ground surface or underside of pile cap. Includes a 1.0 m bedrock socket length.

<sup>2</sup> Based on the preliminary General Arrangement (GA) drawing dated October 2019

<sup>3</sup> N/A as there is no pile cap for a caisson installation, caisson lengths estimates are based on the existing ground surface

<sup>4</sup> Top of caisson is assumed to be the underside of the sand filled CSP used in the hybrid H-pile/caisson foundation configuration

### 6.4.2.3 Factored Geotechnical Axial Resistance – Caissons

Based on the estimated uniaxial compressive strength of the bedrock at this site, and a minimum 1.0 m long bedrock socket, the following factored axial geotechnical resistance can be used in the design:

- 1.5 m diameter caisson – factored geotechnical resistance at ULS of 16,500 kN per caisson.
- SLS does not apply to caissons end bearing on/in bedrock at this site, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

The majority of the geotechnical resistance provided above is derived from end-bearing in the rock socket; as such, the base of each caisson shall be thoroughly cleaned of any cuttings or other material and inspected and tested as noted in Section 6.4.2.1.

Since there is no grade raise proposed at the pier, no downdrag forces are anticipated on the deep foundations at the pier location. If caisson caps are to be included as part of the design at or below the existing ground surface, they should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101.

It is noted that from a foundations perspective it is considered feasible to found the perched abutments on hybrid H-pile/caisson foundation configuration and the geotechnical resistance provided above would also apply in such a case for 1.5 m diameter caissons. However, the magnitude of drag loads generated on caissons installed at the abutments (due to the construction associated with the new approach embankments) will be larger than those indicated in Section 6.4.1.2 due to the larger diameter and circumference of the caisson relative to the steel H-pile.

### 6.4.2.4 Drag Loads – Abutment Caissons

The structural design of the hybrid H-pile caissons at the abutments (if utilized) should be based on full drag loads acting on the caissons. The following estimated drag load acting on a single caisson can be used in the design, assuming the upper 3 m portion of the pile below the pile cap is surrounded by a CSP filled with loose sand, and that the CSP is not connected to the pile cap:

- 1.5 m dia. caisson – unfactored downdrag load 1,150 kN per caisson.

The structural capacity of the hybrid H-pile caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.14.6.3 of the CHBDC. Since the steel piles are to be driven to bedrock, the effect of the drag loads on the settlement (or SLS geotechnical axial resistance) of the piles will be negligible.

## 6.4.3 Resistance to Lateral Loads

For the proposed deep foundations, the SLS lateral geotechnical reaction of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction,  $k_h$ , is based on the equations given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (CFEM 1992) and API (2003). The equations provided, and the associated resistances are based on vertical piles; a modification factor would need to be applied for inclined piles.

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 behaviour (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1 percent 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 behaviour of the soil should be considered using P-y curves.

Based on discussions with MTO Foundations, it is understood that P-y curves are being requested for this project by the structural designer; however, it is understood that they are to be provided in a separate technical memorandum once the structural design has progressed, and the design of the structural elements (i.e., type, size/diameter, and spacing) has been finalized.

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where:  $n_h$  is the constant of horizontal subgrade reaction (kPa/m) (Table below);  
 $z$  is the depth (m); and,  
 $B$  is the pile diameter or width (m).

For cohesive soils:

$$k_h = \frac{67s_u}{B}$$

Where:  $s_u$  is the undrained shear strength of the soil (kPa); and,  
 $B$  is the pile diameter/width (m).

Table 15 outlines the ranges for the values of  $n_h$  and  $s_u$  that may be used in the lateral analysis of the piles at this site. 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.

**Table 15:  $n_h$  and  $s_u$  values for Lateral Load Resistance Calculations**

Soil Type	$f_{horz}$ (kPa)	$n_h$ (kPa/m)	$s_u$ (kPa)
Embankment Fill	-	12,000 to 14,000	-
Silt to Sandy Silt	-	8,000 to 10,000	-
Weathered Silty Clay to Clay (Crust)	-		Decreasing with depth 60 to 30
Unweathered Clay	-		30
Granular Glacial Till	-	20,000 to 22,000	-
Limestone Bedrock	2,000	-	-

For piles arranged in closely spaced groups, the pile-soil-pile interaction causes the individual piles in a group to be less effective than a single pile. These “group effects” can be incorporated into the design by reducing the calculated coefficient of horizontal subgrade reaction values either in the direction of loading or perpendicular to the direction of loading using a method that modifies the single pile lateral resistance by some factor (i.e., a p-reduction factor). Generalized p-multipliers (i.e., p-reduction factors) based on pile spacing and diameter are provided in Section C6.11.3.4 of CHBDC.



The existing foundation elements may remain in place from a geotechnical prospective, provided that the new piles can be driven without interference with the existing pile group (i.e., without contacting the existing piles). However, if the existing piles interfere with the new foundations as might be the case due to the piles supporting the ends of the existing wing walls, they may require removal prior to construction of the new foundations.

## 6.5 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. Given the Seismic Performance Category for the proposed bridge structure, seismic (earthquake) loading will 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 Section 6.12.3 and Figure 6.8 of CHBDC (2019). 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 (2019). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line flatter than 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 (2019).

### 6.5.1 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 (2019) Clause C6.12.1, Figures C6.28 (active earth pressure) and C6.29 (passive earth pressure), and Clause C6.12.2.2 (at-rest earth pressure).

If the wall does not allow lateral yielding (i.e., a 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 allows lateral yielding (i.e., unrestrained structure), active earth pressures should 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 C6.12 of the Commentary to CHBDC (2019).

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

**Table 16: Static Lateral Earth Pressure Coefficients, Existing Embankment Fill**

Material	Existing Granular Embankment Fill
Soil Unit Weight:	21.5 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure: Active, $K_a$ At rest, $K_o$	0.27 0.43

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

**Table 17: Static Lateral Earth Pressure Coefficients, Embankment Fill**

Material	Granular 'A'	Granular 'B' Type II	EPS
Soil Unit Weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>	0.5 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	- -

## 6.5.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading will have to be considered in the design of abutment / wingwalls in accordance with Section 6.14 of the CHBDC (2019). 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 Section C6.14.7.2 (*Displacement-Based Methods*) of the Commentary to CHBDC (2019), for structures that 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 provided that the wall is able to move a distance equal to  $250 k_h$  (mm).

- For structures that are restrained against lateral movement (non-yielding),  $k_h$  is taken as equal to the site-specific PGA in accordance with Section C6.14.7.2 (*M-0 Method*) of the Commentary.
- For both cases the value of the vertical seismic coefficient  $k_v$  is taken as zero in accordance with Section C6.14.7.2 (*M-0 Method*) of the Commentary.
- 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 6.13.3.2 of the *Commentary to CHBDC* (2019).

The following seismic active pressure coefficients ( $K_{AE}$ ) in Table 18 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.

**Table 18: Seismic Active Pressure Coefficients,  $K_{AE}$  for Various Materials**

Condition	Design Earthquake	Site PGA (g)	Seismic Active Pressure Coefficients, $K_{AE}$			
			Granular A $\phi' = 35^\circ$	Granular B Type II $\phi' = 35^\circ$	Existing Granular Fill $\phi' = 35^\circ$	EPS $\phi' = \text{N/A}$
Non-Yielding Wall (Restrained)	2,475 Yr	0.46	0.66	0.66	0.66	—
Yielding Wall (Unrestrained)		0.23	0.42	0.42	0.42	—

## 6.6 Approach Embankments

Based on the preliminary GA drawing, the construction of the replacement of the Highway 401 Underpass structure at Fraser Road will require a nominal grade raise (about 0.65 m) on the existing approach embankments. In addition, because the abutments for the new structure are proposed to be located approximately 10 m in front of the existing abutments, new fill (up to about 4 m thick) will be required to be placed on top of the existing embankment front slope toe berm to achieve the new approach embankment design grade.

The sections below discuss the assessment of the affects of the required fill placement on the stability and settlement of the approach embankments. The analyses assume the following:

- existing approach abutments will be removed (i.e., no resistance is provided by the abutment wall(s) or existing pile foundations);
- all vegetation and topsoil within the plan limits of the new embankment filling footprint will be removed;
- all new fill will be keyed into the side slopes and front slope of the existing embankment(s) in accordance with the requirements of OPSD 208.010.

All stability and settlement analyses have been carried out using the embankment geometry and subsurface conditions at the north approach due to the thicker sensitive and compressible clay stratum at this location which is more critical to the design.

### 6.6.1 Global Stability

The following subsections describe the method used to evaluate the static and pseudo-static global stability of the existing and proposed approach embankments. The geotechnical soil parameters used in the analyses are presented in Section 6.6.1.2 of this report and the results of the stability analyses are summarized in Section 6.6.1.3. An assessment of the stability of the existing approach embankment geometry is also included for comparison. The results of the stability and settlement analyses along with recommendations regarding feasible and the preferred design and construction alternative(s) to mitigate post-construction settlement and achieve the required FoS for stability are discussed in Section 6.6.2.4.

#### 6.6.1.1 Method of Analysis

Two-dimensional limit equilibrium slope stability analyses were performed using the commercially available program Slope/W which is part of the GeoStudio 2021 package, 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 (both circular and non-circular) 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 1.25 and 1.43 has been used for the design of the embankments for consideration of the global stability under short-term/temporary and long-term/permanent static conditions, respectively as per Table 6.2 of CHBDC (2019). For the pseudo-static loading condition, the minimum factor of safety value that is typically required for design is a FoS of 1.1.

The stability analyses have been carried out for both the front slope and the side slope of the existing and new approach embankment(s). The geometry of the existing embankment and adjacent ground/highway surface was taken from a digital topographic drawing (CAD file) for the site that was provided to us by Dillon Consulting (Dillon) in 2018 as part of the Mega 10 Project No. 4017-E-0019; there is no reference in the CAD file to the source of the topographic data. The geometry of the new embankment was based on the preliminary GA drawing for the Fraser Road Underpass prepared by Dillon (dated October 2019), the proposed Fraser Road typical cross-section included in the Design Criteria of the Fraser Road Underpass, Structural Design Report (dated November 2020), and the information in the Work Item No. 1 for Agreement #: 4021-E-0021 that notes the preferred replacement alternative is to be on the existing alignment with an approximate 650 mm grade raise (i.e., above top of existing embankments). It is noted that no new survey information, DTM or DEM for the site, or new General Arrangement drawing(s) were available at the time of preparation of this report.

As noted in Section 6.6.2, the front slope and side slope sections of the north approach were considered to be critical for analyses based on the thickness of the sensitive clay foundation stratum at this location. The following additional details were used in the stability analysis:

- A seismic horizontal loading of 0.23 g, equal to  $\frac{1}{2}$  of the site adjusted PGA value (0.46 g Site Class C) was used for the pseudo-static analysis (see Section 6.2.2.2). Half the site adjusted PGA value is used in accordance with the commentary for the CHBDC Section C6.14.9.1, considering the potential amplification of the seismic ground motions that could occur through the clay deposit.
- For the north approach embankment front slope, the soil stratigraphy used in the stability model is based on Section A-A' and B-B' as shown in Drawings 1 and 2; and, for the east-west side slope, the soil stratigraphy was based on Section C-C' as shown in Drawing 3.
- For the side slope stability analyses, since the existing side slope profiles vary from about 2.1H:1V to 2.7H:1V, the side slopes of the new approach fills were modelled at 2H:1V and 2.75H:1V to check the affect on the calculated FoS.
- Groundwater level at Elevation 49.5 m.

### 6.6.1.2 Soil Strength Design Parameters

For the non-cohesive soils present at the site, the effective stress parameters employed in the analyses were estimated 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 estimated 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 (in accordance with ASTM D2573 based on the measured plasticity indices and using Bjerrum's correction method), from the laboratory oedometer tests (following the correlation proposed by Mesri, (1975)), and from the SCPT results (based on Mesri (1975) and by Demers and Leroueil (2002)). The design undrained shear strength parameters for the weathered crust, used in the analysis were estimated in accordance with Tavenas and Leroueil (1980). A plot of the undrained shear strengths versus elevation for the cohesive soils encountered at the site below the embankment, below the toe berm and outside the toe berm based on the in-situ and laboratory testing obtained during current and previous geotechnical investigations are shown on Figure G1, along with design lines employed in the stability analysis. 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 and liquid limit proposed by Terzaghi et al. (1996) and Gamez and Stark (2014).

For the existing granular 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 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.

The unit weights of the weathered clay crust and the unweathered clay were based on the results of the consolidation and triaxial testing as well as inferred from the measured water content data for these deposits.

The geotechnical design parameters used for the short-term/total stress and pseudo-static stability analyses (i.e., undrained shear strengths,  $s_u$ ) and for the long-term/effective stress stability analysis (i.e., drained shear strengths,  $c'$  and  $\phi'$ ) are summarized in Table 19.

**Table 19: Geotechnical Design Parameters for Stability Analysis**

Material	Bulk Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	Adjusted Undrained Shear Strength, $s_{u(mob)}$ , (kPa)	Drained Analysis			
			Peak		Post-Peak/ Softened	
			Effective Angle of Friction, $\phi'$ , (°)	Effective Cohesion, $c'$ , (kPa)	Effective Angle of Friction, $\phi'$ , (°)	Effective Cohesion, $c'$ , (kPa)
Existing Granular Embankment Fill	21.5	-	35	0	35	0
Topsoil	15.0	-	25	0	25	0
Silt to Sandy Silt	19.0	-	32	0	32	0
Weathered Crust	15.8	Decreasing with depth 40 to 30 (Outside and below the berm) <sup>1</sup>	Design Fully Specified Shear Strength Envelope derived from Triaxial Test Results <sup>2</sup>		26 <sup>3</sup>	0 <sup>3</sup>
		Decreasing with depth 60 to 40 (Below the embankment) <sup>1</sup>				
Grey Clay	15.3	30 (Outside and below the berm) <sup>1</sup>	Design Fully Specified Shear Strength Envelope derived from Triaxial Test Results <sup>4</sup>		20 <sup>5</sup>	0 <sup>5</sup>
		40 (Below the embankment) <sup>1</sup>				
Glacial Till	22.0	-	35	0	35	0

**Notes:**

<sup>1</sup>Refer Figure G1 in Appendix G

<sup>2</sup>Refer Figure G2 in Appendix G

<sup>3</sup>Refer Figure G3 in Appendix G

<sup>4</sup>Refer Figure G4 in Appendix G

<sup>5</sup>Refer Figure G5 in Appendix G

### 6.6.1.3 Results

The results of the approach embankment slope stability analysis are shown in Figures G6 to G17 in Appendix G and the calculated FoS values are summarized in the tables below as follows:

- Table 20–Front Slope – Existing Embankment, New Embankment (Granular Fill), New Embankment (EPS Fill); and,
- Table 21 – Side Slope – Existing Embankment, New Embankment (Granular Fill), New Embankment (EPS Fill) with Slope Profiles at 2H:1V and 2.75H:1V.

The stability analyses include assessment of the affect of constructing the new fill between the old (existing) abutments and the new abutments with both conventional granular fill as well as with lightweight EPS fill. This assessment has been carried out because EPS fill is required as part of the new approach embankment construction to mitigate the settlements that would otherwise occur following construction of the new approaches as discussed in Section 6.6.2.4. In addition, placement of EPS fill between the old (existing) and new abutments is also required to reduce the magnitude of the deformations that would occur under seismic loading for a new approach embankment constructed entirely of conventional granular fill as discussed in Section 6.6.2.4.

**Table 20: Summary of Approach Embankment Slope Stability Analysis – Front Slope (North-South)**

Case	Slope	Type of Analysis		Target Minimum Factor of Safety	Calculated Factor of Safety	Acceptable	Figure in Appendix G
1	Existing Approach Embankment	Undrained	Static	1.25	1.70	Yes	G6
			Pseudo Static	1.1	0.76	Maybe <sup>1</sup>	G7
		Drained	Static with Peak Parameters	1.43	2.30	Yes	G8
			Static with Post peak/ Softened Parameters	1.43	1.94	Yes	G9
2	Approach Embankment with <b>Granular Fill</b> for 0.65 m Grade Raise and for New Fill between old and new abutments.	Undrained	Static	1.25	1.21	No	G10
			Pseudo Static	1.1	0.69	Maybe <sup>1</sup>	G11
		Drained	Static with Peak Parameters	1.43	1.55	Yes	G12
			Static with Post peak/ Softened Parameters	1.43	1.45	Yes	G13
3	Approach Embankment with Granular Fill for 0.65 m Grade Raise, and <b>EPS Fill</b> (approx. 4 m thick) for New Fill between old and new abutments.	Undrained	Static	1.25	1.56	Yes	G14
			Pseudo Static	1.1	0.71	Maybe <sup>1</sup>	G15
		Drained	Static with Peak Parameters	1.43	2.15	Yes	G16
			Static with Post peak/ Softened Parameters	1.43	1.82	Yes	G17

**Notes:**

<sup>1</sup>Factor of safety value is less than typically required for pseudo-static stability analysis. However, considering the conservatism inherent in the pseudo-static stability analysis, a dynamic 2D finite element analysis has been carried out to evaluate the deformation behaviour of the embankment under seismic load (see Section 6.7).



**Table 21: Summary of Approach Embankment Slope Stability Analysis – Side Slope (East-West)**

Case	Slope	Type of Analysis		Target Minimum Factor of Safety	Calculated Factor of Safety	Acceptable	Figure
1	Existing Side Slope	Undrained	Static	1.25	1.94	Yes	G18
			Pseudo Static	1.1	0.82	Maybe <sup>1</sup>	G19
		Drained	Static with Peak Parameters	1.43	2.54	Yes	G20
			Static with Post peak/ Soften Parameters	1.43	2.07	Yes	G21
2	New <b>2.75H:1V Side Slope</b> with <b>Granular Fill</b> for 0.65 m Grade Raise and for New Fill between old and new abutments	Undrained	Static	1.25	1.61	Yes	G22
			Pseudo Static	1.1	0.73	Maybe <sup>1</sup>	G23
		Drained	Static with Peak Parameters	1.43	2.35	Yes	G24
			Static with Post peak/ Soften Parameters	1.43	1.91	Yes	G25
3	New <b>2H:1V Side Slope</b> with Granular Fill for 0.65 m Grade Raise, and <b>EPS Fill</b> (approx. 4 m thick) for New Fill between old and new abutments	Undrained	Static	1.25	1.78	Yes	G26
			Pseudo Static	1.1	0.75	Maybe <sup>1</sup>	G27
		Drained	Static with Peak Parameters	1.43	2.34	Yes	G28
			Static with Post peak/ Soften Parameters	1.43	1.93	Yes	G29

**Notes:**

<sup>1</sup>Factor of safety value is less than typically required for pseudo-static stability analysis. However, considering the conservatism inherent in the pseudo-static stability analysis, a dynamic 2D finite element analysis has been carried out to evaluate the deformation behaviour of the embankment under seismic load (see Section 6.7).

The results of the stability analyses can be summarized as follows:

- **Case 1 - Existing approach embankment:** the front and side slopes of the existing embankment(s) have a FoS that satisfies the CHBDC requirements for the short-term (total stress/undrained) and long-term (effective stress/drained) limit states considering both peak and post-peak/softened shear strengths. However, the pseudo-static limit equilibrium analysis indicates a  $FoS < 1.1$  (i.e., less than the CHBDC requirements) under the loading associated with the site PGA.
- **Case 2 – New approach embankment with 0.65 m of grade raise and fill between old and new abutments constructed with conventional granular fill:** the front slopes of the new embankments have a FoS that does not satisfy the CHBDC requirements for the short-term (total stress/undrained) limit state, nor for the long-term (effective stress/drained) limit state considering post-peak/softened shear strengths. Further, the pseudo-static limit equilibrium analysis indicates a  $FoS < 1.1$  (i.e., less than the CHBDC requirements) under the loading associated with the site PGA.



- **Case 3 – New approach embankment with 0.65 m of conventional granular fill grade raise and fill between old and new abutments constructed with EPS fill:** the front and side slopes of the new embankment(s) have a FoS that satisfies the CHBDC requirements for the short-term (total stress/undrained) and long-term (effective stress/drained) limit states considering both peak and post-peak/softened shear strengths. However, the pseudo-static limit equilibrium analysis indicates a  $FoS < 1.1$  (i.e., less than the CHBDC requirements) under the loading associated with the site PGA.

Based on the above, conventional granular fill cannot be used for construction of the portion of the new approach embankments between the existing and new abutments. In order to satisfy the minimum FoS requirements in the CHBDC for embankment stability, the portion of the new fill within this area between the top of the existing toe berm up to about 1 m below the design top of pavement will need to be constructed with lightweight EPS fill.

Further, although the pseudo-static limit equilibrium analysis suggests that the requirements of the CHBDC are not satisfied under the seismic loading limit state (i.e., calculated  $FoS < 1.1$  even with the use of EPS fill), a two-dimension dynamic finite element analysis has been carried out (as discussed in Section 6.7) to evaluate the calculated deformations of the approach embankment(s) under the design seismic event.

### 6.6.2 Settlement Analysis

The preliminary GA drawing dated October 2019 shows the proposed grade raise and changes in geometry of the existing approach embankments required for the approaches to the new bridge as summarized in Section 6.1. To estimate the magnitudes of expected settlement due to construction of the new approach embankment(s), analysis was carried out for the north approach embankment which is considered the more critical location due to the slightly thicker clay deposit in the foundation stratum.

It is noted that analysis carried out as part of the 1966 foundation investigation and design (GEOCRETS Report 31G00-142) indicates the construction of the original (existing) approach embankments constructed entirely of granular fill would result in long-term settlements on the order of 0.45 m to 0.60 m. It is understood that the existing bridge was completed in 1968, and a memorandum prepared in 1975 by the MTC's Kingston Materials & Testing Office (included in GEOCRETS Report 31G00-192) reports that approximately 0.23 m to 0.3 m of settlement of the approach embankments occurred in the first few years after construction necessitating pavement, curb and gutter restoration in 1971. No further settlement/maintenance records have been provided.

Based on available topographic information (i.e., the digital topographic drawing (CAD file) for the site described in Section 6.6.1.1), the footprint of the existing Fraser Road embankment(s) leading up to the bridge, including the front slope toe berm are approximately 100 m long (north-south) with an irregular, tear-drop shape. The width of the existing toe berm(s) vary along the length of the embankment(s) and have a maximum width (east-west) of approximately 72 m at the location where the height of the approach fills is at a maximum. The side slopes of the toe berm have a profile ranging from approximately 2.5H:1V to 3.5H:1V, while the side slopes of the approach embankment above the berm have a profile ranging from approximately 2.1H:1V to 2.6H:1V. The plan view provided in Drawing 1 shows the footprint of the approach embankment(s) including the toe berms.

The results of the boreholes advanced at the site indicate that the clay layer thickness increases from south to north along the Fraser Road alignment as well as from west to east parallel to Highway 401. The profile drawings shown in Drawings 1 to 4 illustrate the varying clay layer thickness encountered at the site. As noted previously, the settlement analysis was carried out for the north approach embankment where the clay is thickest.

To accommodate the vertical alignment of the new Fraser Road leading up to the new bridge, an approximately 0.65 m grade raise will be required above the existing approaches. However, to construct the approach embankments up to the new abutments, approximately 4.8 m to 5.3 m of new fill will be required to be placed in the current front slope area on top of the existing front slope toe berm. The additional embankment loading including the proposed grade raise over the thick sensitive and compressible clay deposit will result in new settlements, the mitigation of which needs to be considered as part of the project design. The following subsections provide a description of the settlement analysis methodology including a summary of the simplified stratigraphy and geotechnical parameters used in the model, the results of the analysis including an assessment of the settlement of the existing embankment for model calibration, and a discussion on the recommended mitigation measure(s) to deal with the settlements.

#### **6.6.2.1 Method of Analysis**

The settlement analyses were carried out using the commercially available program Settle3 version 5.012. One of the challenges of the analysis was modeling the irregular shape of the existing approach embankment and toe berms as well as the new fill placement on top of the existing toe berm. This was accomplished in the Settle3 model using the following approach:

- The irregular shape of the embankment over the entire approach embankment footprint including the toe berm (100 m in length) was divided into 21 slices/segments with a varying width between 1.5 m and 6 m;
- The toe berms were modelled as individual segments of 3.7 m high embankment sections;
- The approach fills (i.e., on top of the toe berm fills) were model, as 10.5 m wide embankment sections with 2H:1V side slopes. A total of 14 slices/segments were considered to capture the footprint of 78 m long approach embankment. The height of these 14 slices was gradually decreased from 4.1 m to 0.7 m to approximate the existing profile of Fraser Road;
- The infilling between the new and existing abutments were modelled with an additional two segments of embankment load with 2H:1V side slopes in front of the existing approach fills on top of previously constructed toe berm, while the 0.65 m grade raise was modelled as an additional fill load on top of the previously constructed embankment fill.

The geometry of the north approach embankment used in the Settle3 analysis is shown in Figure H3.

Settlement of the new approach embankments will occur as a result of the grade raise and new fill placement on top of the existing front slope toe berm. The total settlement of the new approach embankments will be comprised of:

- Compression of the existing and new embankment fill(s);
- Short-term (immediate) compression of the non-cohesive/granular foundation soils; and,
- Long-term consolidation and creep settlement of the firm to stiff clay stratum.

Given that the existing embankment fills are comprised of granular material and the new embankment fill will be comprised of granular fill and/or lightweight fill, the compression of the embankment fills will be negligible and are expected to occur during construction.

The geotechnical design parameters used in the settlement analysis are provided in Figures H1 and H2 in Appendix H based on assessment of all of the subsurface information and in-situ and laboratory testing results from the current and previous investigations as detailed in Section 4.3.

The immediate compression of the non-cohesive granular silty sand deposit and sand and gravel till 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 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 and SCPTs to estimate the stress history and primary consolidation 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 Azzouz et al. (1976), Koppula (1981), and Leroueil (1983).

The coefficient of consolidation,  $c_v$  ( $\text{cm}^2/\text{s}$ ), required in the time-rate settlement analysis was established using the results of the laboratory consolidation tests for the appropriate stress range applicable to the field loading condition and the results from the in-situ pore pressure dissipation testing carried out during the SCPT 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 to the conditions at this site. As such the  $c_v$  values provided in Table 22 and used in the settlement analysis were selected over the appropriate field stress range generally in the recompression/over-consolidated range.

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. The creep parameters used in the analysis were estimated from the laboratory conventional 24-hour incrementally loaded consolidation tests, the long-term consolidation tests. This data was compared with the empirical correlations for modified secondary compression index by Mesri (1973), Anagnostopoulos and Grammatikopoulos (2011), and Leroueil (1999).

The design lines shown in pink on the individual plots in Figures H1 and H2 were selected based on consideration of all of the data plotted as well as information available for similar soils in literature and WSP’s past experience. In addition, the design lines were checked and/or calibrated by using the settlement model to compare the predicted settlements of the existing embankment construction with the measured settlements a few years after original embankment construction. The parameters assigned to the Settle3 model are summarized in Table 22.

**Table 22: Foundation Strata Geotechnical Design Parameters for Settlement Analysis**

Soil Layer	Layer Thickness (m)		Unit Weight, $\gamma'$ (kN/m <sup>3</sup> )	Modulus of Elasticity (MPa)	Consolidation Parameters
	North Approach	South Approach			
Granular Fill / Upper Silty Sand	0.8	0.8	20	20	NA
Weathered Silty Clay to Clay Crust	1.6	1.2	17	NA	$C_c = 0.7$ to $2.0$ ; $C_r = 0.018$ ; $e_o = 1.8$ $\sigma'_p = 250$ kPa to $135$ kPa; $C_v = 0.00875$ cm <sup>2</sup> /s; $C_{vr} = 0.012$ cm <sup>2</sup> /s to $0.0055$ cm <sup>2</sup> /s
Upper Unweathered Clay	1.5	1.5	15.5	NA	$C_c = 2.0$ ; $C_r = 0.025$ ; $e_o = 2.2$ $\sigma'_p = 130$ kPa to $120$ kPa; $C_\alpha = 0.0384$ ; $C_{\alpha r} = 0.00384$ $C_v = 5 \times 10^{-5}$ cm <sup>2</sup> /s; $C_{vr} = 0.0055$ cm <sup>2</sup> /s;
Lower Unweathered Clay	5.2	2.8	15.5	NA	$C_c = 2.0$ to $0.323$ ; $C_r = 0.025$ ; $e_o = 2.2$ to $1.25$ ; $\sigma'_p = 120$ kPa to $75$ kPa; $C_\alpha = 0.0384$ ; $C_{\alpha r} = 0.00384$ ; $C_v = 5 \times 10^{-5}$ cm <sup>2</sup> /s; $C_{vr} = 0.0055$ cm <sup>2</sup> /s
Sand and Gravel Till	3.1	6.7	22	60	NA

$\sigma'_p$  Estimated preconsolidation pressure  
 $C_c$  Compression index  
 $C_r$  Recompression index  
 $C_v$  Co-efficient of Consolidation (normally consolidated stress range)  
 $C_{vr}$  Co-efficient of Consolidation (over-consolidated stress range)  
 $e_o$  Initial void ratio  
 $C_\alpha$  Modified Secondary Compression index (normally consolidated stress range)  
 $C_{\alpha r}$  Modified Secondary Compression index (over-consolidated stress range)

Other assumptions used in the settlement analysis are as follows:

- Groundwater table at Elevation 47.0 m for the original ~1968 construction, and at Elevation 49.0 m for the new proposed construction; and,
- Although there is variability in the thickness of sensitive clay layer encountered in different boreholes below the north approach, the model used a maximum 6.7 m thick clay layer under the footprint of embankment which is the worst-case scenario as encountered in Borehole 22-03A.

### 6.6.2.2 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. Fraser Road is considered to a 'surface treated and gravel' roadway. The allowable total post-construction settlement limits of embankments in this category of roadway vary as a function of distance from the bridge abutment. The settlement limits applicable to Fraser Road over a 15-year period following construction, are as outlined Table 23.

**Table 23: MTO Settlement Criteria for Surface Treated and Gravel Roadway**

Distance from Abutment (m)	Tolerable Settlement (mm)
0 to 20	25
20 to 50	75
50 to 75	150
>75	300

### 6.6.2.3 Results

The results of the estimated settlements for the different cases considered in the Settle3 analysis are presented in the tables below as follows:

- Table 24 – Settlement of existing approach embankment(s) due to original construction (model verification).
- Table 25– **(Case #1 and #2)** Settlement of new approach embankment(s) due to grade raise and in-fill between new abutment(s) and original abutment areas (considering granular fill and EPS fill).
- Table 26 – **(Case #3)** Settle Settlement of new approach embankment(s) due to grade raise and in-fill between new abutment(s) and beyond original abutment areas (considering EPS fill).

The results presented in Table 24 demonstrate that the Settle3 model predicts settlements about 295 mm within the first few years after construction which is generally consistent with the reported settlements in GEOCREST Report 31G00-192. The contours of the calculated settlements at 50 years after construction are shown in Figure H4 which indicate that the maximum settlement would have occurred along the centreline of the embankment approximately 4 m behind the original abutment(s). Although the model indicates there would have been about 280 mm of additional settlement between the reported pavement rehabilitation in 1971 and current day, it is likely that additional grading/pavement structure rehabilitation has occurred in the approach areas of the existing bridge during the last 45 years.

**Table 24: Estimated settlements during and after construction of existing embankment**

Stage/Time	Total Settlement (mm)
3 years after original construction (~1971)	55
5 years after original construction (~1973)	295
50 years after original construction (~2018)	575

The results presented in Table 25 show the model predicted total settlements due to the new approach embankment construction. Two scenarios have been considered: (i) construction of the in-fill area between the new abutment and the existing abutment area with conventional granular fill; and (ii) construction of the in-fill area between the new abutment and the existing abutment area with lightweight EPS fill ( $\gamma_{\text{EPS}} = 0.5 \text{ kN/m}^3$ ). Both analyses assume that the 0.65 m thick grade raise required on top of the existing approaches is constructed with conventional granular fill, while the analysis with the EPS assumes a 1.0 m thickness of conventional granular fill overlying the EPS.

Query points were selected at 5 m, 10 m, and 25 m behind the new abutment location in the model as indicated as Points A, B and C in Figures 5 and 6. These points are considered most relevant to the settlement assessment as they lie within and immediately beyond the approach area.

**Table 25: Estimated Settlement for Construction of New Embankment (Case #1 and Case #2)**

Stage/Time	Estimated Post-Construction Settlement (Primary Consolidation + Secondary Compression) (mm)					
	Granular Fill (Case #1)			EPS Geofoam 10 m length (Case #2)		
	5 m	10 m	25 m	5 m	10 m	25 m
<b>North Embankment (Unweathered Clay Thickness = 6.7 m)</b>						
2.5 years after construction	200	165	25	25	35	25
15 years after construction	325	260	55	55	75	50
<b>South Embankment (Unweathered Clay Thickness = 4.3 m)</b>						
2.5 years after construction	180	160	25	18	35	20
15 years after construction	280	240	40	35	55	35

The results indicate that unacceptably large settlements that exceed the MTO Settlement Criteria as presented in Table 23 will occur within the in-fill area (between the old and new abutments) if conventional granular fill is used for construction. If lightweight EPS fill is used for construction, the settlements will be much smaller, however, exceedance of the MTO Settlement Criteria will still occur approximately 2.5 years after construction which would require maintenance of the pavement structure at that time. Additional settlements, on the order of about 35 mm to 50 mm will continue after that time, however, these would be corrected during the course of future pavement rehabilitation(s).

The settlements summarized in Table 25 indicate similar magnitudes at both the north approach and south approach areas. This may seem counterintuitive given that the clay stratum is relatively thinner below the south approach and thicker below the north approach; however, the results are correct for the time frames considered (i.e., at 2.5 years and 15 years after construction). Although the total consolidation settlement at the south approach (i.e., at the end of primary consolidation which requires much longer than 15 years to complete) is

smaller than at the north approach, the thinner clay stratum below the south approach results in a relatively faster rate of consolidation in this area. Given this, a larger percentage of the total primary consolidation is completed below the south approach at the time frames considered. The faster consolidation below the south approach and the slower consolidation below the north approach coincidentally results in similar magnitudes of settlement at 2.5 years and 15 years after construction.

As the settlement estimates for the case of the in-fill between abutments constructed with EPS do not meet MTO's settlement criteria an additional analysis was carried out to see the effect on settlement if a portion of the existing fill embankment behind the existing abutment was removed and the EPS fill zone extended further into the existing embankment. For this analysis the EPS was extended to 16 m behind the new abutment meaning approximately 6 m of the existing embankment would be removed to approximate Elevation 53.0 m and brought back up to grade using EPS light weight fill.

Query points were selected at 5 m, 15 m and 25 m behind the new abutment location as indicated as Points A, B and C in Figure H7.

Table 26 summarizes the total settlements estimated following removal of 6 m of the existing embankment fill (behind the existing abutment area, down to Elevation 53 m) and re-construction with EPS fill, extending to behind the new abutment and up to 1 m below the final roadway surface.

**Table 26: Estimated Settlement for Construction of New Embankment EPS Geofoam 16 m Length (Case #3)**

Stage/Time	Total Maximum Settlement (mm)		
	EPS Geofoam 16 m Length (Case #3)		
	Point A 5 m	Point B 15 m	Point C 25 m
2.5 years after construction	10	-3	15
15 years after construction	30	-1	35

The results of the settlement analyses can be summarized as follows:

- **Case 1 – New approach embankment(s) with 0.65 m of grade raise and fill between old and new abutments constructed with conventional granular fill:** settlements of up to 200 mm at 2.5 years after construction, and up to 325 mm at 15 years after construction, exceed the MTO Settlement Criteria of 25 mm within 20 m of the abutment. As such, this option is not considered feasible, unless the MTO is prepared to undertake numerous pavement maintenance and rehabilitation periods following construction.
- **Case 2 – New approach embankment(s) with 0.65 m of conventional granular fill grade raise and fill between old and new abutments constructed with EPS fill (approx. 10 m long):** settlement estimates of about 25 mm to 35 mm at 2.5 years after construction, and up to 75 mm at 15 years after construction, exceed the MTO Settlement Criteria of 25 mm within 20 m of the abutment. However, these settlements could be managed with one or two pavement maintenance or rehabilitation periods following construction. If



adopted, a basic settlement monitoring program is recommended the details of which are described in Section 6.6.2.4.

- **Case 3 – New approach embankment(s) with 0.65 m of conventional granular fill grade raise and fill between old and new abutments and behind existing abutment constructed with EPS fill (approx. 20 m long):** settlement estimates of about 10 mm to 15 mm at 2.5 years after construction, and up to 35 mm at 15 years after construction. These settlements satisfy the MTO Settlement Criteria of 25 mm within 20 m of the abutment.

#### 6.6.2.4 Mitigation Options

Considering the results of the settlement analyses described above, the proposed new bridge structure and associated approach embankment geometry, and the subsurface conditions at the site, the following settlement mitigation options are presented and the feasibility, advantages and disadvantages of each discussed.

**Preloading:** If the new approach embankment is constructed with conventional fill, it is estimated that the time required to complete the majority of the primary consolidation of the clay stratum is about 50 years. If a preload period of about 2.5 years could be accommodated in the construction schedule, the settlements that would occur between the end of preload and the 15-year design service life of the roadway would be about 100 mm which would still require maintenance of the pavement structure during this time. However, as discussed in Section 6.6.1.3, the front slope of the approach will have a  $FoS < 1.3$  if constructed with granular fill, which may not be acceptable given the proximity to the existing Highway 401. Given the stability concerns and the fact that post-construction settlements would still occur after a 2.5-year preload period, preloading is not recommended to mitigate the settlements at this site.

**Surcharging:** The addition of a surcharge fill on top of the preload fill would accelerate the rate of consolidation in the clay stratum and result in more settlement during the surcharge period than would be achieved under preloading alone, thereby reducing the magnitude of post-construction settlement and associated amount of maintenance of the pavement structure during the design service life. However, given that the front slope of the approach embankments has a  $FoS < 1.3$  for the design fill height, the  $FoS$  will be even less if a surcharge fill is added. Given the stability concerns at this site, and the associated risks to the adjacent Highway 401, surcharging is not recommended to mitigate settlements at this site.

**Wick Drains (in conjunction with Preloading):** Installation of wick drains on an approximately 1.5 m triangular spacing extending through the full depth of clay stratum, in combination with a preload comprised of conventional granular fill would shorten the time to complete the majority of the primary consolidation to about 2 years. However, the low  $FoS$  associated with the front slope of the approach embankment would still be an issue unless the fill placement was staged and carefully monitored which would increase the construction time required for this option. In addition, the stiff to very stiff nature of the weathered silty clay to clay crust, as well as the potential for cobbles within the existing embankment fill, may make installation of the wick drains difficult and could require pre-drilling. Considering these disadvantages and risks, the use of wick drains to reduce the preload period is not recommended at this site.

**Lightweight (EPS) Fill:** The use of lightweight expanded polystyrene (EPS) fill for the construction of the approach embankment(s) between the new and existing abutment areas is a viable and preferred option to reduce the magnitude of the post-construction settlements within the approach areas. Installing EPS in this area also mitigates the approach front slope stability issue as discussed in Section 6.6.1.3. However, since the EPS requires a minimum 1 m thickness of conventional soil cover to mitigate differential icing effects, the loading from



the cover fill will still result in some post-construction settlements in the approach area, on the order of about 25 mm within the first 2.5 years following construction, with an additional 25 mm to 50 mm at the 15 year design service life. This post-construction settlement would likely require at least one roadway pavement maintenance within the first few years after construction. Alternatively, these post-construction settlements could be further reduced by either extending the EPS further back into the existing embankment fill, or by extending the base of the EPS deeper into/below the top of the front slope toe berm, or both. Although the EPS option has many advantages to mitigate the settlement and stability issues within the approach areas, it also comes with a high cost.

**Ground Improvement (GI):** An intrusive form of ground improvement could be considered to mitigate the settlements from the new approach fills. Considering that the subsurface soils primarily contributing to the long-term settlements are comprised of firm (borderline soft), sensitive clay soils, GI methods such as stone columns and aggregate piers that involve dynamic and repetitive force for construction may not be recommended due to the risk of causing disturbance and remoulding of the clay. A system that uses Rigid Inclusions (i.e., cementitious columns created by placing concrete through a hollow steel mandrel pushed into the ground) would likely be preferable to minimize the risk of disturbance to the sensitive soils. The rigid inclusions would need to fully penetrate the clay stratum and toe into the underlying compact to dense granular till. A Load Transfer Platform (LTP) comprised of either granular material (potentially with geogrid) or concrete would have to be constructed on top of the RIs and beneath the new embankment fill to distribute the fill loads onto the RIs. Such a system is a viable alternative at this site and has the advantage of minimizing post-construction settlements to likely less than 25 mm as well as allow placement of the fills following a minimum curing period; however, there will be a risk of differential settlement between the area with the RIs/LTP and the adjacent area without (likely on the order of about 25 mm to 50 mm), and the RI/LTP ground improvement will have a high cost compared with some of the other mitigation options.

**Lengthening Bridge Span(s):** As an alternative to constructing the new approach embankments with lightweight EPS fill, consideration could be given to lengthening the bridge spans such that the new abutments would be located at or behind the existing abutments. It is noted that the Structural Design Report (2020) included an option for longer span lengths (2-span configuration, each span 44.8 m long) that would result in the new abutments being located at about the same location as the existing abutments. With this option, only minor filling (i.e., 0.65 m grade raise) would be required as part of the new approach embankment construction which would limit the post-construction settlements to about 50 mm at 15 years, with about 25 mm occurring within the first 2.5 years after construction. However, this option would require significantly higher costs and the full removal of the existing abutments, including the existing pile foundations, would likely be required with this option to avoid conflicts during pile installation.

**Do Nothing (Long-term Maintenance Required):** The allowable total post-construction settlement for 'Surface Treated and Gravel Roadway' approach embankments is 25 mm over a 15 year period following completion of construction (per MTO's embankment settlement criteria and Section 6.6.2.2). However, if the approach embankments and approach slabs to the new bridge structure could tolerate up to about 360 mm of total long-term, post-construction settlement, the constructing the new approach fills with no settlement mitigation measures could be considered. MTO would have to accept and plan for the additional maintenance requirements associated

with these long-term settlements and it is likely that several maintenance and/or pavement rehabilitations would be required.

Based on the above, conventional granular fill is not recommended for construction of the portion of the new approach embankments between the existing and new abutments. In order to reduce the post-construction settlement within the approach areas to manageable amounts, it is recommended that the portion of the new fill within this area (approx. 10 m length) between the top of the existing toe berm and up to about 1 m below the design top of pavement be constructed with lightweight EPS fill. Adoption of this settlement mitigation strategy will still require at least one pavement maintenance/rehabilitation within about 2.5 years following completion of construction.

## 6.7 Seismic Response Analysis of Approach Embankments

As discussed in Section 6.6.1, the pseudo-static limit equilibrium analysis completed to evaluate the embankment stability under seismic loading resulted in calculated FoS values less than 1.0. Considering the potential conservatism in the pseudo-static method of analysis and the fact that a limit equilibrium analysis can only calculate a factor of safety (with no context on the associated movement of the embankment), seismic site response analyses have been carried out to better assess the associated deformations of the approach embankments under the design seismic loading conditions.

The seismic site response analyses were carried out using both one-dimensional (1D) and two-dimensional (2D) models. The nonlinear 1D seismic site response analysis was carried out using the computer software DEEPSOIL 7.0 to assess the propagation of seismic waves generated from the bedrock, and to estimate the maximum cyclic shear strains developed in the clay foundation stratum for comparison with the strain-softening behaviour of the sensitive clay. The nonlinear 2D seismic site response analysis was carried out using the computer software PLAXIS2D (Version 21.01) to assess the free field seismic deformations of the proposed approach embankments under dynamic shaking resulting from the input spectrum and spectrum-compatible time histories discussed in Sections 6.7.1, and 6.7.2, respectively. The material model type HS-Small was utilized mainly for the purpose of capturing the cyclic behavior of the clays, while PM4Sand was used to capture the cyclic behavior of cohesionless materials and liquefaction potential. The shear moduli used in the analysis were evaluated based on the shear wave velocities measured from the seismic CPTs and VSP testing carried out at the site. The 2D stress-deformation, finite element analysis was also used to estimate a seismic displacement profile in the abutment areas through the embankment fill and foundation soils.

The details of the seismic response analysis including the soil model types and input parameters are provided in a Technical Memorandum included in Appendix J. Additional information on the input spectrum, the ground motion time histories and the results of the analysis are summarized in the following sections.

### 6.7.1 Target Input Spectrum

Based on the results of the borehole investigation carried out at the site, the bedrock was encountered at depths of about 10 to 15 m below the existing ground surface at the borehole locations. A Vertical Seismic Profile (VSP) test was carried out in the previous 2018 investigation in boreholes 18-1101 and 18-1103 at the site to record shear wave velocity measurements within the boreholes.

Based on the results of VSP testing, shear wave velocity measurements of between 1,500 and 2,000 m/s were measured in the bedrock, which corresponds to a Site Class A in accordance with Section 4.1.8.4 of the CHBDC S-19. Therefore, the firm-ground target spectrum used to scale the input time histories was developed using the interpolated Site Class A, representative of the bedrock conditions. The interpolation was carried out using reference Site Class C seismic hazard values as outlined in Table 12 and were adjusted for Site Class A in accordance with the factors outlined in the CHBDC S-19. The resulting spectrum is outlined below in Table 27.

**Table 27: Target Seismic Hazard Values for Ground Motion Scaling for Site Class A (Bedrock)**

Seismic Hazard Values (g)	2% Probability of Exceedance in 50 years (2,475 return period)
PGA	0.414
Sa (0.2)	0.582
Sa (0.5)	0.279
Sa (1.0)	0.146
Sa (2.0)	0.066
Sa (5.0)	0.018
Sa (10.0)	0.006

**6.7.2 Spectrum-Compatible Time Histories**

There are two commonly accepted approaches to scaling input time histories to match a target spectrum: linear scaling and spectral matching. Linear scaling involves simply scaling the ordinates of the record to achieve the best fit to the target response spectrum over the period range of interest. Linear scaling provides input time histories that are more representative of the original records of ground shaking (i.e., less modification), but can be difficult to match the target spectrum over a large period range. Spectral matching involves changing the frequency and phase contents of the record to match the target spectrum. Spectral matching allows for development of input records that provide a closer match to the target spectrum over a broad range of periods but involves more modification of the original records since no real earthquake spectrum will match the entire target spectrum.

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

Linearly scaled time histories were used to develop the horizontal ground motions. Figure J1 shows that the geometric mean response spectrum of selected and scaled earthquake acceleration time histories (EATHs) is generally within the 90% to 120% of the target response spectrum from about 0.2 to 1.5 seconds, which is about 0.4 to 1.5 times the MTO provided fundamental structure period of about 0.5 seconds. The fundamental period of the site is approximately 0.3 seconds and also falls within this range. The linear scale factors range from 1.1 to 4.0, which are within the generally recommended range from 0.25 to 4.0.

Twelve time-histories were selected for these analyses. Earthquake acceleration time histories were selected from the NGA-East database (<https://ngawest2.berkeley.edu/>) maintained by the Pacific Earthquake Engineering Research Center (PEER). A summary of the earthquake records used is provided in Table 28 below.

**Table 28: Summary of Input Time History Earthquake Events**

Database	Event Name	Event Year	Station / Suite Name	Magnitude	Distance (km)	Scaling Factor
PEER	San Fernando	1971	Lake Hughes #4 (H1)	6.6	19.5	1.9
PEER	San Fernando	1971	Lake Hughes #4 (H2)	6.6	19.5	1.8
PEER	Livermore	1980	APEEL 3E Hayware CSUH (H1)	5.8	29.2	3.5
PEER	Coalinga	1983	Parkfield Stone Corral 3E (H2)	6.4	32.8	2.2
PEER	Coyote Lake	1979	Gilroy Array #1 (H2)	5.8	10.7	2.4
PEER	Big Bear	1992	Silent Valley – Poppet Flat (H1)	6.5	34.4	3.8
PEER	Northridge	1994	LA – Wonderland Ave (H1)	6.7	15.1	2.0
PEER	Sierra Madre	1991	LA – City Terrace (H2)	5.6	23.7	2.6
PEER	L'Aquila Italy	2009	Celano (H2)	6.3	17.8	3.3
PEER	Friuli Italy	1976	Conegliano (H2)	6.5	80.4	3.0
PEER	Trinidad	1980	Rio Dell Overpass_E Ground (H2)	7.2	76.1	1.1
PEER	Borah Peak	1983	PBF (second bsmt) (H1)	6.9	87.7	4.0

Additional details of the spectral acceleration plots of the input ground motions, the acceleration time history plots, and the scaled peak ground acceleration values are included in the Technical Memorandum in Appendix J.

### 6.7.3 Summary of Results

The full details of the results of the seismic response analysis are presented in the Seismic Response Analysis Technical Memorandum in Appendix J. The results of the seismic response analysis are summarized as follows:

- 1) While some excess pore water pressure is predicted to build-up within the near surface silty sand layer below the embankment fill during the seismic shaking, the layer is not predicted to liquefy.
- 2) The maximum cyclic shear strains that develop within the sensitive clay stratum are predicted to be about 1% or less away from the abutments (based on the 1D model) and less than about 3% (based on the 2D model) in the abutment area due to the geometry of the front slope. Given that the results of the CIU and CID

triaxial testing indicate peak strengths at about 3% as shown in the stress-strain curves, the risk of strain softening (and cyclic mobility) of the clay soils under seismic loading is considered low.

- 3) For the case of the approach embankment fill constructed of conventional granular fill, the maximum earthquake-induced total displacements at the crest of the front slope are predicted to be less than about 110 mm (north approach), and less than about 90 mm (south approach), while the permanent total displacements are predicted to be slightly less than these values.
- 4) Though there is no liquefaction and no cyclic mobility/flow failure anticipated due to seismic loading there are displacements that will result in lateral loading on the structure and foundation elements. Based on the displacements provided in Bullets 2 and 3 above, the structural engineer will need to estimate the loading, (kinematic and inertial) and based on the soil structural interaction evaluate the consequence of the predicted displacement(s) on the foundations/piles.
- 5) For the case of the approach embankment fill constructed using lightweight EPS fill between the new and old (existing) abutment areas, the predicted total displacements are about 25% to 35% less than those with the embankment constructed entirely of conventional granular fill.
- 6) For the case of zoning the undrained shear strength in the 2D FEA model (i.e., using higher undrained shear strengths below the embankment, and lower undrained shear strengths below and beyond the existing toe berm in accordance with the design lines presented in Figure G1), the predicted total displacements are about 5% to 10% more than those indicated above.

#### **6.7.4 Embankment Seismic Functionality**

Section 6.14.2 of the CHBDC indicates that the design of the structure shall address the post-seismic functionality of the travelled lanes within and outside the *Approach Embankment Bridge Interface Zone*.

The embankment bridge interface zone is defined in Section 6.14.2.2 of the CHBDC as equal to the greater of:

- a) horizontal distance behind an abutment defined by a 2H:1V plane projected up from the toe of the abutment head slope; or,
- b) a horizontal distance of 20 m.

##### **6.7.4.1 Within Approach Embankment Bridge Interface Zone**

Based on the proposed bridge structure importance category of *Other Bridge* and the results of the seismic response analysis, the proposed geotechnical system will satisfy the following seismic performance criteria for within *Approach Embankment Bridge Interface Zone* listed in Section 6.14.2.1c of the CHBDC:

- i) Not collapsing following ground motions with a return period of at least 2,475 years; and,
- ii) Have 50% of the travelled lanes, but not less than one, available for use following ground motions with a return period of at least 475 years.

##### **6.7.4.2 Outside Approach Embankment Bridge Interface Zone**

In accordance with Section 6.14.2.3 of the CHBDC there are no seismic performance requirements for travelled lanes outside the *Approach Embankment Interface Zone* for structures with an importance category of *Other Bridge*.

## 6.8 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete and/or steel of foundations buried in the soil. The long-term performance and durability of the foundations are directly related to their corrosion resistance. Generally, the corrosivity potential to a structure can be assessed based on the soil resistivity / electrical conductivity, hydrogen ion concentration (pH), and salts (chloride and sulphate) concentrations. The analytical results for the soil samples submitted for testing are summarized in Section 0 and the analytical laboratory test reports are included in Appendix C.

The potential for sulphate attack and corrosion are discussed in the following sub-sections; however, it is ultimately up to the structural designer to determine the appropriate construction materials, including the exposure class, and ensure that all aspects of CSA A23.1:19 Section 4.1.1 “Durability Requirements” are followed when designing concrete elements, as applicable.

### 6.8.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-19 Table 3 for potential sulphate attack on concrete. The sulphate concentrations measured in eight tested sample ranged from 0.01% to 0.05% and is below the exposure class of S-3 (Moderate). Therefore, based on the soil sample tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered. Accordingly, GU cement in accordance with Table 6 of the CSA Standard A23.1-19 could be specified for concrete in below grade applications.

### 6.8.2 Potential for Corrosion

The test results indicate a pH value ranged from 7.5 to 8.0 and a resistivity ranged from 1,613 ohm-cm to 3,333 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not detrimental to concrete durability. The resistivity indicates that the soil corrosiveness is Moderate ( $4500 \text{ ohm-cm} > R > 2000 \text{ ohm-cm}$ ) to Severe ( $2000 \text{ ohm-cm} > R$ ), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014), and appropriate corrosion protection should be applied to the foundation element / materials. Accordingly, GU cement in accordance with Table 6 of the CSA Standard A23.1-19 could be specified for concrete in below grade applications.

These recommendations are provided as guidance only; the designer should take the results of the laboratory testing into consideration for selecting and specifying appropriate materials and corrosion susceptibility for design service of the structure foundations and determine the appropriate exposure class and ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

## 6.9 Construction Considerations

### 6.9.1 Subgrade Preparation and Embankment Construction

Prior to reconstruction of the existing approach embankments and construction of the portion of the new approach embankments in front of the existing abutments, it is recommended that all vegetation and topsoil/organic soil be stripped from the embankment footprint. The requirements for removal or pulverizing of the existing surface treatment on Fraser Road prior to the grade raise is to be addressed by others.

Fill for construction for the grade raise and widenings of the new approach 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. All new fill will be keyed into the side slopes and front slope of the existing embankment(s) in accordance with the requirements of OPSD 208.010. For the in-fill area between the new and existing abutments, it is recommended that the new approach be constructed with lightweight expanded polystyrene (EPS) fill in accordance with the requirements of the Special Provision (Rigid Expanded Polystyrene Embankment Fill) included in Appendix K. The backslope of the EPS fill zone into the existing embankment fill should be sloped at a profile no steeper than 2H:1V to limit the lateral earth pressures from the granular embankment fill onto the EPS fill.

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.

## 6.9.2 Excavation and Control of Groundwater and Surface Water

Removal of the topsoil/organic soils within the footprints of the new approaches are anticipated to require only limited excavations (i.e., less than 0.5 m deep). However, removal of the existing abutment including the pile(s) supporting the ends of the wing walls (as shown on the Department of Highways, Fraser Road Underpass, General Plan, Drawing No. D5888-1, dated Feb. 1966) will likely require excavations extending through the existing approach fills down to at least Elevation 52 m.

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 existing granular embankment fill materials 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 3 soils should be made with side slopes no steeper than 1H:1V.

It is understood that, in accordance with the Structural Design Report (SDR) prepared by Dillon (dated November 2020), full closure of Fraser Road for the duration of construction of the replacement bridge and approaches is the preferred approach given the low traffic volumes and available detour routes. As such, temporary protection systems are not anticipated to be required along Fraser Road for construction of the new approach embankments, grade raise, or new abutments. Further, given that the Preliminary GA Drawing indicates the central pier of the new bridge will be constructed on caissons (drilled shafts) extending from bedrock up to a pile cap located immediately below the slab-on-steel box girders supporting the bridge deck, excavation and an associated temporary protection system will not necessarily be required for pier construction within the median of Highway 401, depending on the design top of caisson elevation (see below).



Based on the Preliminary GA Drawing, the new abutments will be constructed on top of the existing front slope toe berms with the underside of pile cap at about Elevation 53 m and the base of the CSPs surrounding the integral abutment piles extending to between about Elevations 49.5 m to 50.0 m. Given that the groundwater level measured in the monitoring wells was at Elevation 49.4 m or lower, dewatering for construction of the new abutments is not anticipated to be required.

The new pier is to be comprised of a series of caissons (drilled shafts) advanced from within the median of Highway 401. Based on the Preliminary GA Drawing, the ground surface within the median area appears to be at about Elevation 50.5 m, and the top of the caissons appear to be shown at about Elevation 49 m. To avoid the requirements for excavation and possibly a temporary protection system with dewatering within the pier area, it is recommended that the top of caissons be modified to a higher elevation closer to the existing ground surface. It is noted however, that adopting this design will still likely require the use of temporary liners during caisson construction to avoid groundwater inflows and sloughing of the upper granular fills and silt to sandy silt soils into the upper part of the caissons.

The existing standpipe piezometers (installed in Boreholes 22-01B, 22-03A, 22-03B and 18-1103A) 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.

### 6.9.3 Temporary Protection Systems

As discussed in Section 6.9.2, based on the SDR, full closure of Fraser Road for the duration of construction of the replacement bridge and approaches is the preferred approach given the low traffic volumes and available detour routes. As such, temporary protection systems will not be required for construction of the approaches and new bridge abutments.

Based on the current Preliminary GA Drawing, temporary protection systems may be required with the median area of Highway 401 to construct the caissons supporting the new central pier. However, it is believed that the requirements for TPS at the pier area could be avoided by modifying the design top of caissons to be closer to the existing ground surface.

However, temporary protection systems are likely to be required at the centre pier should the current design be changed to incorporate driven steel piles, the construction of a pile cap and/or to facilitate the extent of removals that may be required. Where required, temporary protection systems must be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection System*) and Special Provision 105S09. The lateral movement of the temporary protection systems must meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent utilities can tolerate this magnitude of deformation. However, it is understood that there is no vibration sensitive infrastructure within the vicinity of the pier location and as such vibration monitoring is likely not required.



#### 6.9.4 Existing Utilities

It is noted that based on the service locates and utility clearances carried out as part of the field work for this assignment, a Bell Canada fibre optic cable is located approximately 12 to 15 m south of the south abutment of the existing bridge. The vibration and settlement tolerances of this cable are unknown but will need to be investigated as part of the detail design. Depending on the settlement tolerance, additional mitigation measure(s) that minimize the settlement in this area may be required. All construction within the vicinity of the cable must be in accordance with the “Canadian Guidelines for Safe Excavation in the Vicinity of the Bell Network” (2018) guidelines and/or any other restrictions imposed by Bell Canada.

#### 6.9.5 Instrumentation and Monitoring



As discussed in Section 6.6.2.3, settlement of the new approach embankment fills is expected to occur even if the EPS fill is used for construction of the in-fill area between the new abutment(s) and existing abutments. To assess the time at which maintenance of the surface treatment/pavement structure will be required in the years after completion of construction, it will be necessary to measure the post-construction settlements within the approach areas. Given the predicted settlement magnitudes are relatively modest, it is recommended that the post-construction settlement monitoring program simply consist of PK survey nails embedded in the finished pavement structure surface, or Settlement Rods (SRs) in shallow boreholes (about 1.7 m deep below finished pavement structure surface) installed after completion of construction. A total of six (6) settlement monitoring points (3 on each approach embankment) should be installed at 5 m, 10 m and 20 m behind the new abutments.

Vibration monitoring of any existing sensitive utilities may be warranted depending on the contractor's selected method(s) for installation of protection systems, pile driving (if the preferred foundation type is changed), and compaction of granular materials. In this case, MTO will develop terms of reference for vibration monitoring to be included as part of the Construction Contract Administration services, and a Notice to Contractor should be included in the contract documents to alert the contractor to the locations of vibration monitoring equipment and the requirement to cooperate with the CA for access to equipment locations, protect the monitoring equipment throughout construction, and replace the any damaged or missing monitoring equipment. The Notice to Contractor should also include information regarding vibration thresholds for any nearby utilities, and required actions should the review and alert levels be reached.



## 7.0 CLOSURE

This report was prepared by Kinjal Gajjar, EIT, and Kenton Power, P.Eng. The report was reviewed by Paul Dittrich P.Eng. a Senior Principal Geotechnical Engineer with WSP/Golder and the Designated MTO Foundations Contact for this project.

### WSP Canada Inc.



Kenton Power, P.Eng.  
*Senior Geotechnical Engineer*



Paul Dittrich, Ph.D., P.Eng.  
*MTO Foundations Principal Contact*

KG/KCP/JPD/yj

[https://wsonline.sharepoint.com/sites/gld-158306/project files/6 deliverables/22513877a fraser rd/2-final/4021-e-0021 rev0 final fidr fraser rd up \(22513877a\) 2024-03-04.docx](https://wsonline.sharepoint.com/sites/gld-158306/project%20files/6%20deliverables/22513877a%20fraser%20rd/2-final/4021-e-0021%20rev0%20final%20fidr%20fraser%20rd%20up%20(22513877a)%202024-03-04.docx)

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- Ontario Geological Survey 2011. *1:250 000 scale bedrock geology of Ontario*; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1.
- Occupational Health and Safety Act and Regulation for Construction Projects (as amended)

**ASTM International**

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil
ASTM D2435-04	Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading

**Ontario Provincial Standard Specifications (OPSS)**

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 422	Construction Specification for Installation of Precast Reinforced Concrete Box Culverts with Span 3m or Less in Open Cut
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.MUNI 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 1002	Material Specification for Aggregates – Concrete
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

**OPSS Standard Special Provisions**

SSP 102S05	Amendment to OPSS 206
SSP 105S09	Amendment to OPSS 539
SSP 105S22	Amendment to OPSS 501
SSP 110S16	Amendment to OPSS 1004
SSP 110S17	Amendment to OPSS 1002
SSP FOUN0003	Amendment to OPSS 902
SSP 206F04	Amendment to OPSS 206
SSP 206F06	Amendment to OPSS 206

**Ontario Provincial Standard Drawings (OPSD)**

OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0 m
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario

**Ontario Water Resource Act**

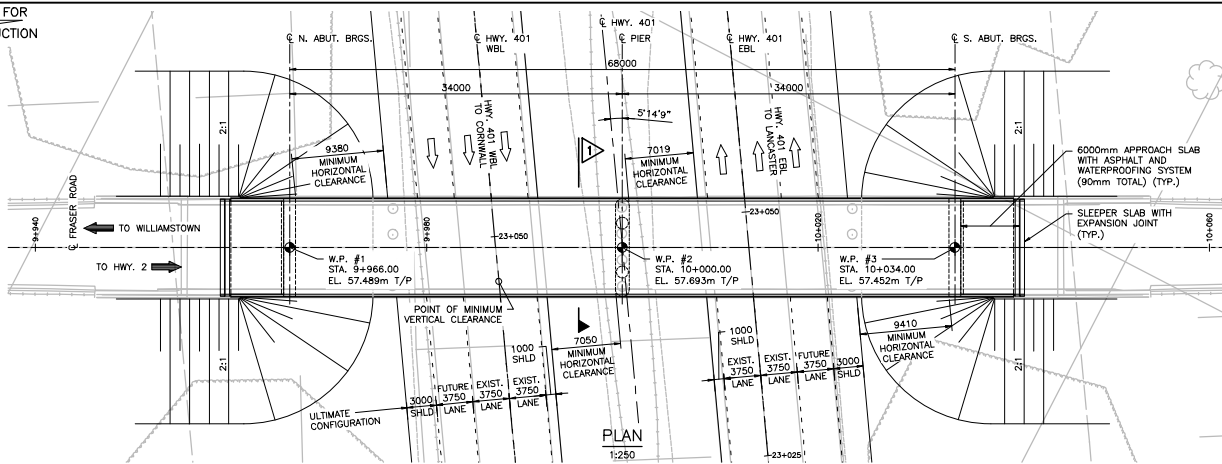
Regulation 903	Wells (as amended)
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**DRAWINGS**

Fraser Road Underpass Preliminary General Arrangement Drawing

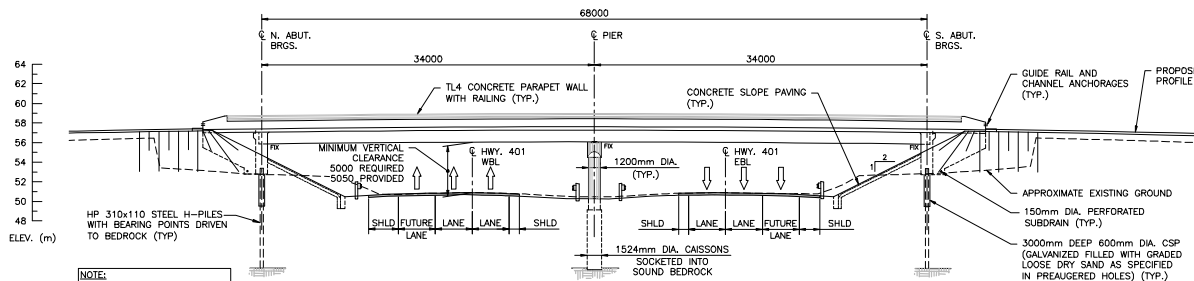
Drawings 1 to 4

NORTH FOR CONSTRUCTION



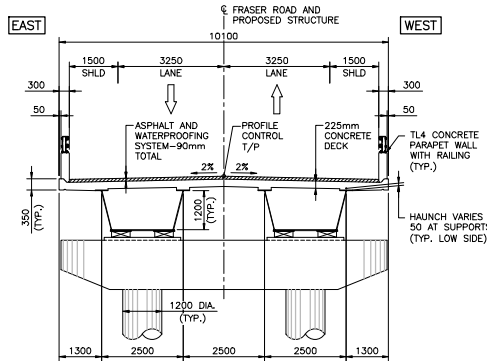
PLAN

1:250



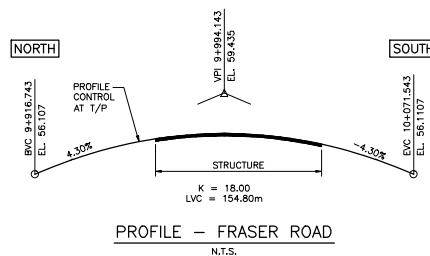
WEST ELEVATION

1:250



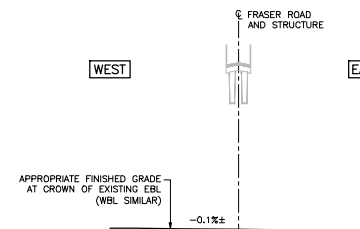
SECTION 1

1:75



PROFILE - FRASER ROAD

N.T.S.



PROFILE - HWY. 401

N.T.S.

### APPLICABLE STANDARD DRAWINGS

OPSD-3101.150 WALLS-ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT  
OPSD-3370.100 DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD  
OPSD-3370.101 DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS  
OPSD-3390.100 DECK DRIP CHANNEL  
OPSD-3419.100 BARRIERS AND RAILINGS, STEEL GUIDE RAIL AND CHANNEL ANCHORAGE  
OPSD-3941.200 FIGURES IN CONCRETE, SITE NUMBER AND DATE LAYOUT

DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

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

### LIST OF ABBREVIATIONS

WBL WEST BOUND LANES  
EBL EAST BOUND LANES  
WB WEST BOUND  
EB EAST BOUND  
SHLD SHOULDER  
W.P. WORKING POINT  
FIX FIXED  
EXP EXPANSION  
T/P TOP OF PAVEMENT  
PS PAVED SHOULDER  
SCL SPEED CHANGE LANE  
STA STATION

HWY. 401

CONT No  
WP No 4290-15-01

FRASER ROAD UNDERPASS

PRELIMINARY  
GENERAL ARRANGEMENT



PREFERRED REPLACEMENT  
ALTERNATIVE

NOT FOR CONSTRUCTION

### GENERAL NOTES

#### 1. CLASS OF CONCRETE

CLASS OF CONCRETE SHALL BE 30 MPa

#### 2. CLEAR COVER TO REINFORCING STEEL

CAISSONS 100 ± 25  
DECK TOP 70 ± 20  
DECK BOTTOM 40 ± 10  
PIER CAPS 70 ± 10  
REMAINDER 70 ± 20 UNLESS OTHERWISE NOTED

#### 3. REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400W.  
UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL BARS SHALL BE CLASS B.  
STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa, UNLESS OTHERWISE SPECIFIED.

BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.

GLASS FIBRE REINFORCED POLYMER REINFORCING BARS SHALL BE GRADE I, GRADE II OR GRADE III AS SPECIFIED IN THE CONTRACT DRAWINGS. THE NOMINAL DIAMETER, TENSILE MODULUS OF ELASTICITY AND GUARANTEED MINIMUM TENSILE STRENGTH SHALL BE AS SPECIFIED IN THE CONTRACT DOCUMENTS.

BAR MARKS WITH THE PREFIX GI DENOTE GRADE I GLASS FIBRE REINFORCED POLYMER BARS.

BAR MARKS WITH THE PREFIX GII DENOTE GRADE II GLASS FIBRE REINFORCED POLYMER BARS.

BAR MARKS WITH THE PREFIX GIII DENOTE GRADE III GLASS FIBRE REINFORCED POLYMER BARS.

BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12-1, UNLESS INDICATED OTHERWISE.

#### 4. CONSTRUCTION NOTES

BACKFILL SHALL NOT BE PLACED BEHIND THE ABUTMENTS UNTIL THE DECK SLAB IS IN PLACE AND HAS REACHED 70% OF ITS DESIGN STRENGTH.

BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500 mm.

CONSTRUCT ABUTMENTS AND WINGWALLS TO THE BEARING SEAT ELEVATIONS. THE CONTRACTOR SHALL SUPPLY TEMPORARY LATERAL BRACING FOR THE ABUTMENTS, FORMWORK AND LATERAL BRACING SHALL NOT BE REMOVED UNTIL CONCRETE HAS REACHED 70% OF ITS SPECIFIED 28-DAY STRENGTH.

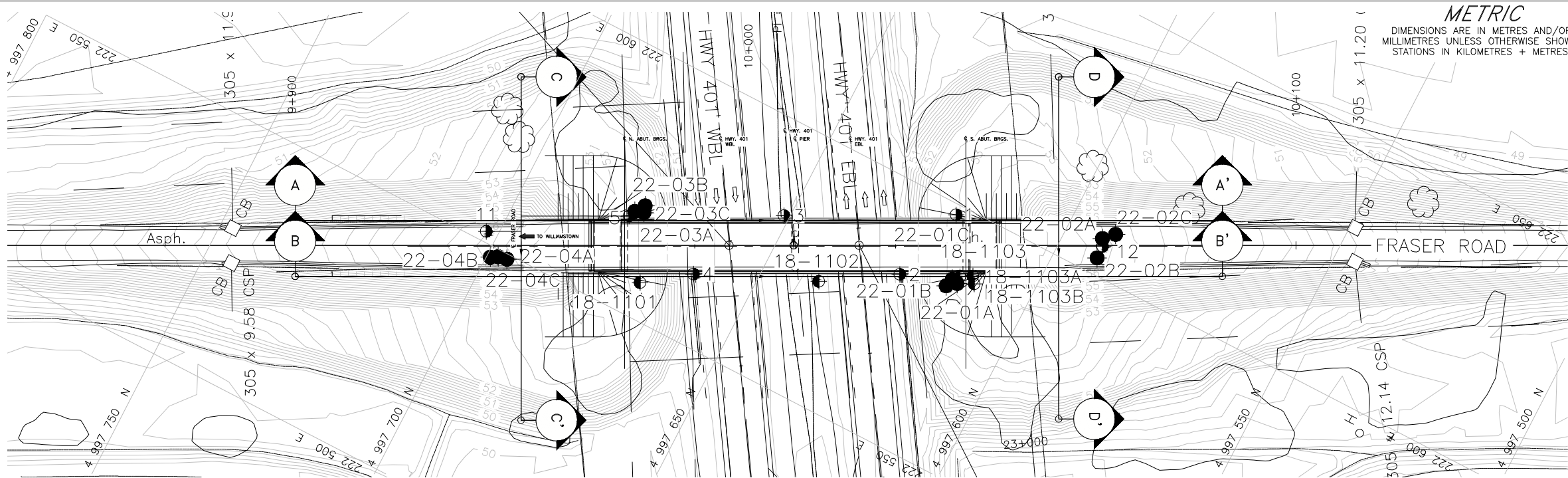
THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT AND INFORM THE CONTRACT ADMINISTRATOR.

THE CONTRACTOR IS RESPONSIBLE FOR THE DESIGN AND INSTALLATION OF ALL TEMPORARY STRUCTURES AND CONSTRUCTION PLATFORMS AND DEBRIS CONTAINMENT SYSTEMS.

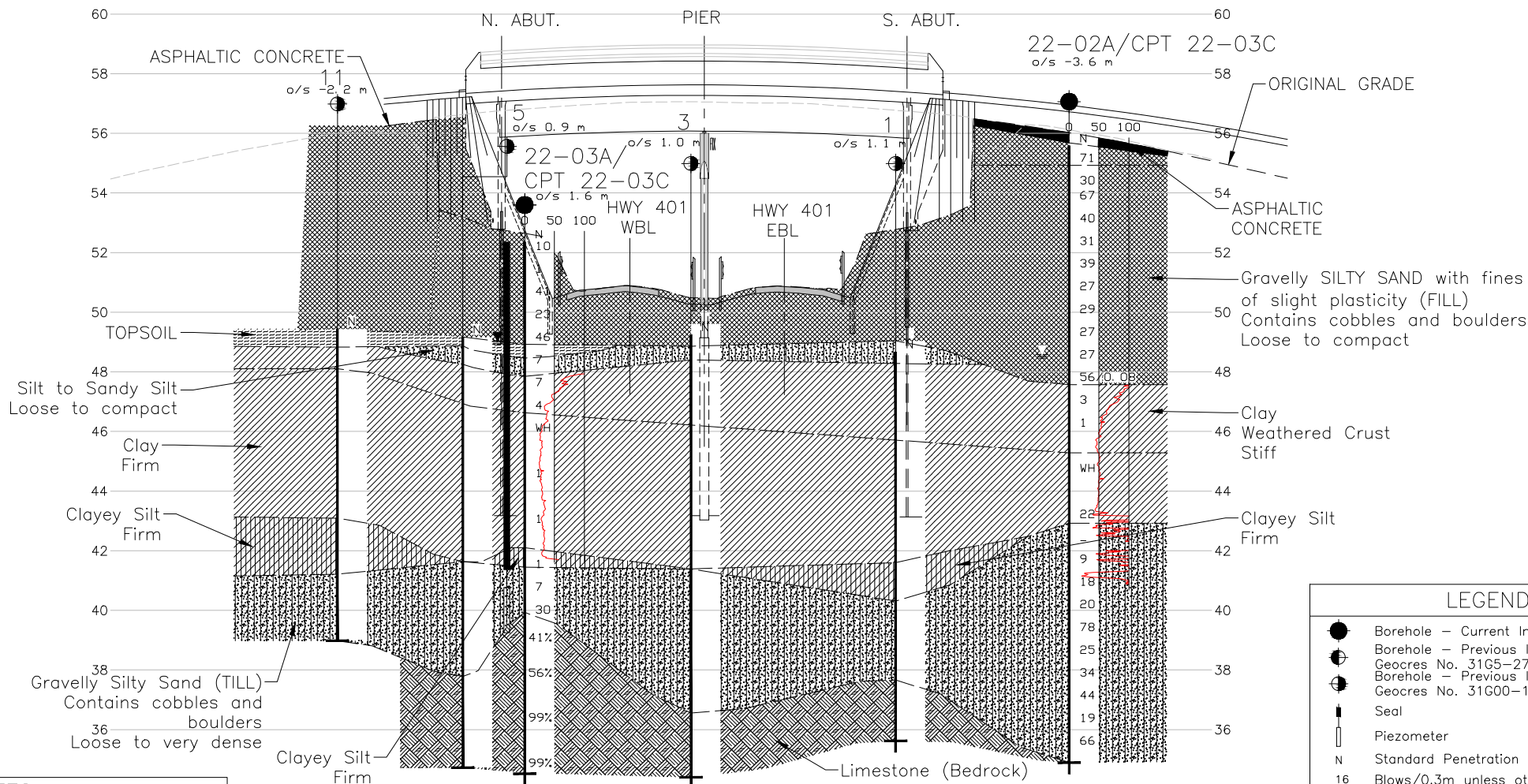
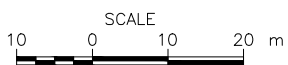
ALL ELEVATIONS ARE TO GEODETIC DATUM.

DATE	BY	DESCRIPTION
DESIGN	ARK	CHK
DRAWN	RJD	CHK
DATE	OCT 2019	DWG

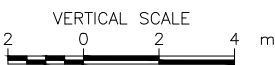
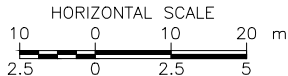




PLAN



CROSS-SECTIONS A-A'



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.

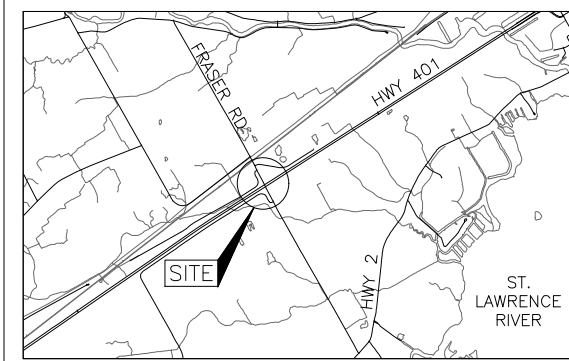
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

METRIC

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

CONT No. WP No. 4290-15-01

REPLACEMENT OF HIGHWAY 401 UNDERPASS AT FRASER RD BOREHOLE LOCATIONS AND SOIL STRATA



KEY PLAN



BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 8)

No.	ELEVATION	NORTHING	EASTING
22-01A	52.9	4997614.3	222585.9
22-01B	52.9	4997616.0	222584.5
22-01C	52.9	4997615.5	222586.2
22-02A	56.0	4997592.7	222607.2
22-02B	56.0	4997591.9	222603.3
22-02C	55.8	4997590.7	222609.1
22-03A	52.7	4997676.2	222569.8
22-03B	52.7	4997676.6	222571.1
22-03C	52.7	4997677.9	222569.2
22-04A	55.9	4997698.0	222548.7
22-04B	55.9	4997699.3	222547.9
22-04C	56.0	4997696.2	222549.0
18-1101	52.8	4997670.5	222557.1
18-1102	50.6	4997639.0	222573.6
18-1103	53.0	4997614.4	222587.4
18-1103A	53.1	4997612.2	222588.7
18-1103B	52.9	4997611.2	222587.4
1	49.0	4997620.7	222598.0
2	50.6	4997625.3	222582.3
3	49.6	4997651.1	222582.1
4	50.6	4997661.8	222563.5
5	49.2	4997678.5	222567.8
11	49.4	4997702.4	222552.0
12	49.1	4997592.0	222605.8

Structural Site Location: Latitude: 44.114025 Longitude: -74.554989

REFERENCE

Base plans provided in digital format by Dillon, drawing file no. WP 4328-11-01 - Hwy 401 Charlottenburgh, received JULY 04, 2018.  
GA provided in digital format by MTO, drawing file no. 188202-09-GA - Fraser Rd, received January 23, 2023.

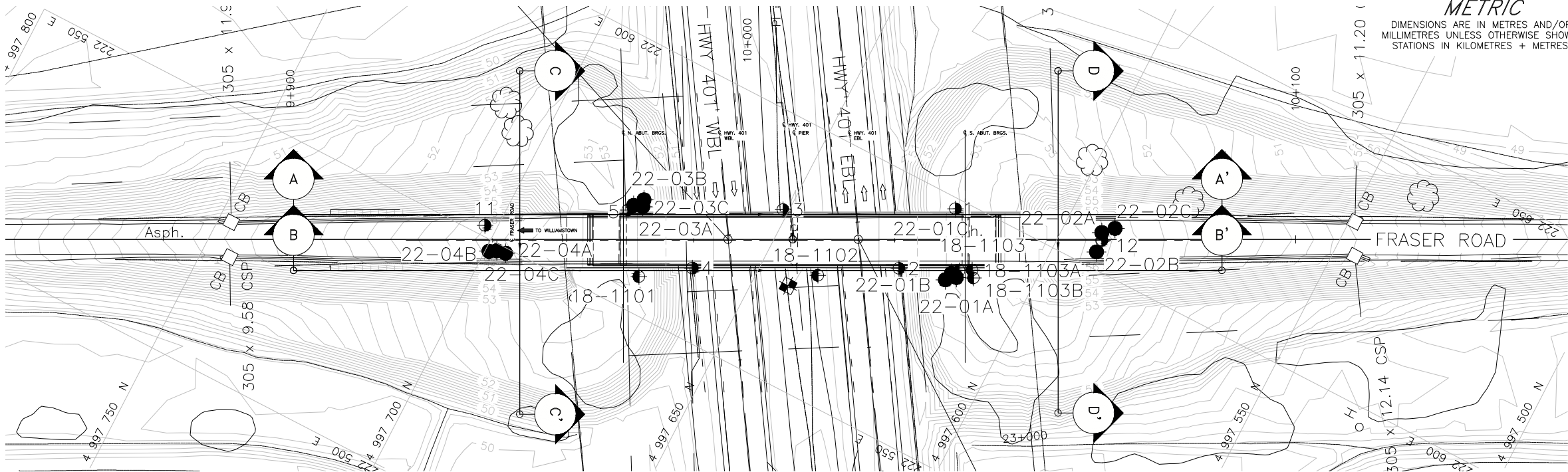
NO.	DATE	BY	REVISION
1	2024	JPD	Initial Issue
2	2024	JPD	Revised for final design
3	2024	JPD	Revised for final design
4	2024	JPD	Revised for final design
5	2024	JPD	Revised for final design
6	2024	JPD	Revised for final design
7	2024	JPD	Revised for final design
8	2024	JPD	Revised for final design
9	2024	JPD	Revised for final design
10	2024	JPD	Revised for final design
11	2024	JPD	Revised for final design
12	2024	JPD	Revised for final design
13	2024	JPD	Revised for final design
14	2024	JPD	Revised for final design
15	2024	JPD	Revised for final design
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68	2024	JPD	Revised for final design
69	2024	JPD	Revised for final design
70	2024	JPD	Revised for final design
71	2024	JPD	Revised for final design
72	2024	JPD	Revised for final design
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83	2024	JPD	Revised for final design
84	2024	JPD	Revised for final design
85	2024	JPD	Revised for final design
86	2024	JPD	Revised for final design
87	2024	JPD	Revised for final design
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91	2024	JPD	Revised for final design
92	2024	JPD	Revised for final design
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94	2024	JPD	Revised for final design
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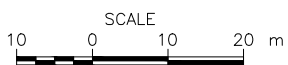
LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- Geocres No. 31G5-273
- Borehole - Previous Investigation
- Geocres No. 31G00-142
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Rock Quality Designation (RQD)
- WL in piezometer, measured on March. 7, 2023
- WL upon completion of drilling
- CPTU Results





PLAN



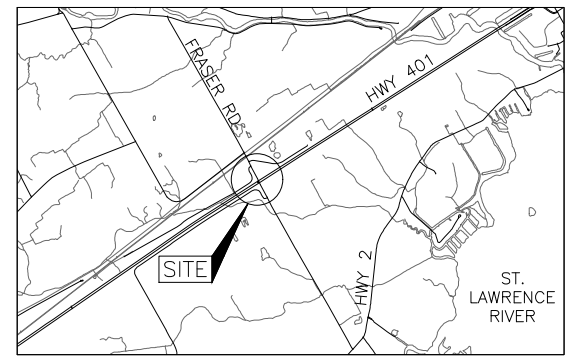
**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No. 4290-15-01

REPLACEMENT OF HIGHWAY 401  
UNDERPASS AT FRASER RD  
BOREHOLE LOCATIONS AND SOIL STRATA

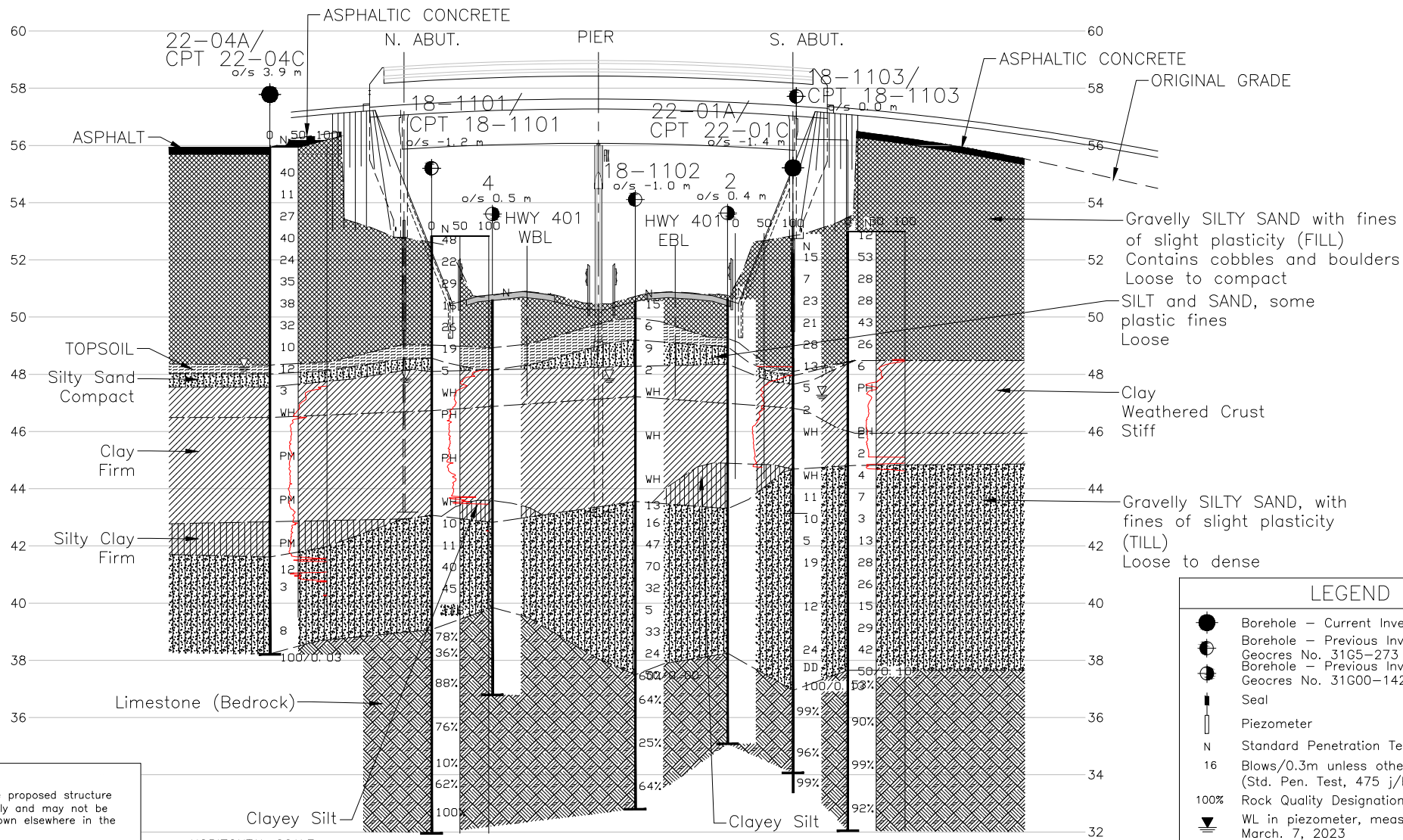


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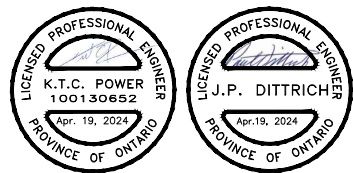
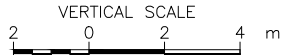
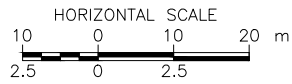


KEY PLAN

SCALE  
1 0 1 2 km



CROSS-SECTIONS B-B'



NOTES

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- Rock Quality Designation (RQD)
- WL in piezometer, measured on March. 7, 2023
- WL upon completion of drilling
- CPTU Results

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 8)

No.	ELEVATION	NORTHING	EASTING
22-01A	52.9	4997614.3	222585.9
22-01B	52.9	4997616.0	222584.5
22-01C	52.9	4997615.5	222586.2
22-02A	56.0	4997592.7	222607.2
22-02B	56.0	4997591.9	222603.3
22-02C	55.8	4997590.7	222609.1
22-03A	52.7	4997676.2	222569.8
22-03B	52.7	4997676.6	222571.1
22-03C	52.7	4997677.9	222569.2
22-04A	55.9	4997698.0	222548.7
22-04B	55.9	4997699.3	222547.9
22-04C	56.0	4997696.2	222549.0
18-1101	52.8	4997670.5	222557.1
18-1102	50.6	4997639.0	222573.6
18-1103	53.0	4997614.4	222587.4
18-1103A	53.1	4997612.2	222588.7
18-1103B	52.9	4997611.2	222587.4
1	49.0	4997620.7	222598.0
2	50.6	4997625.3	222582.3
3	49.6	4997651.1	222582.1
4	50.6	4997661.8	222563.5
5	49.2	4997678.5	222567.8
11	49.4	4997702.4	222552.0
12	49.1	4997592.0	222605.8

Structural Site Location: Latitude: 44.114025 Longitude: -74.554989

REFERENCE

Base plans provided in digital format by Dillon, drawing file no. WP 4328-11-01 - Hwy 401 Charlottenburgh, received JULY 04, 2018.  
GA provided in digital format by MTO, drawing file no. 188202-09-GA - Fraser Rd, received January 23, 2023.

NO.	DATE	BY	REVISION
1	4/18/2024	ZS/SA	PROJECT NO. 22513877A
2	4/18/2024	JPD	DIST. EASTERN
3	4/18/2024	JPD	SITE: 21-230
4	4/18/2024	JPD	DWG. 2

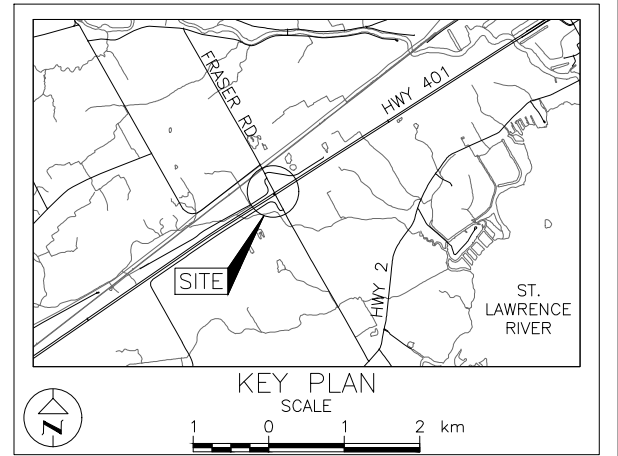


**METRIC**  
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STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No. 4290-15-01

REPLACEMENT OF HIGHWAY 401  
UNDERPASS AT FRASER RD  
BOREHOLE LOCATIONS AND SOIL STRATA

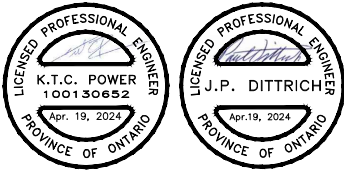
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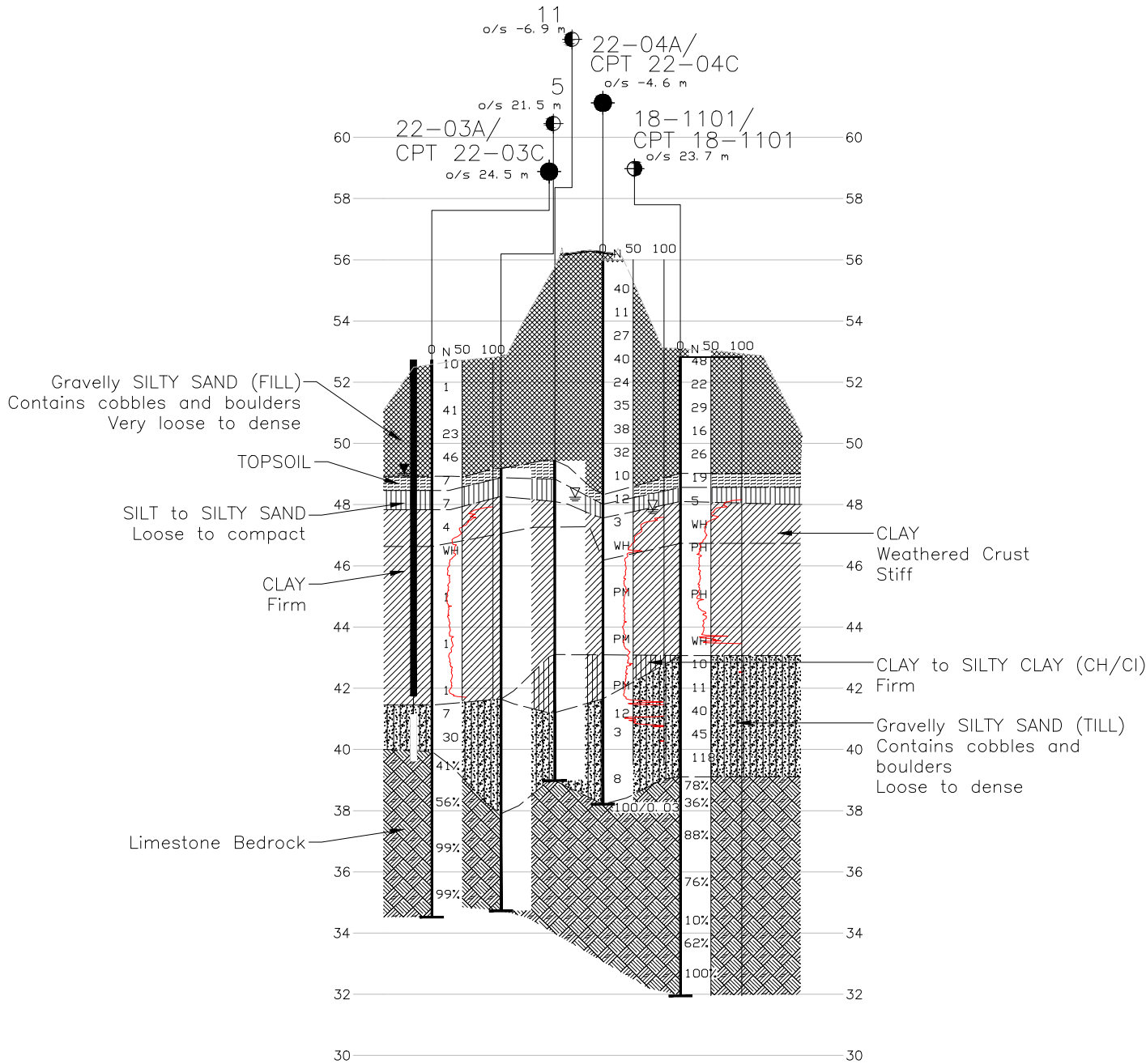
LEGEND

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- Borehole - Previous Investigation
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11	49.4	4997702.4	222552.0



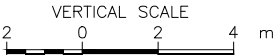
CROSS-SECTIONS C-C'- NORTH EMBANKMENT



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GA provided in digital format by MTO, drawing file no. 188202-09-GA - Fraser Rd, received January 23, 2023.

Structural Site Location: Latitude: 44.114025 Longitude: -74.554989

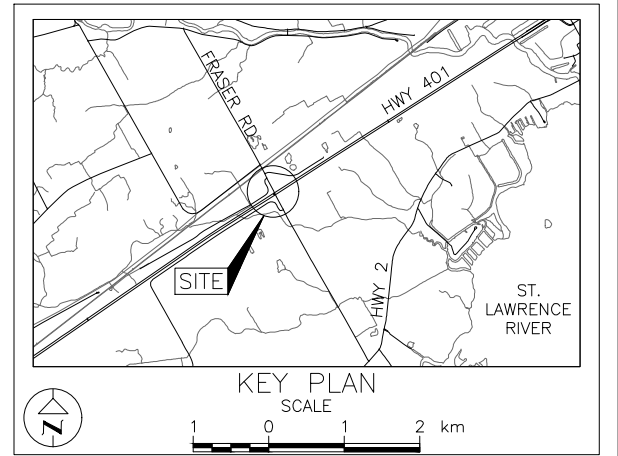
NO.	DATE	BY	REVISION
Geocres No. 31G-291			
HWY. 401	PROJECT NO. 22513877		DIST. EASTERN
SUBM'D. KG	CHKD. KCP	DATE: 4/18/2024	SITE: 31-230
DRAWN: ZS/SA	CHKD. JPD	APPD. JPD	DWG. 3

**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
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CONT No.  
WP No. 4290-15-01

REPLACEMENT OF HIGHWAY 401  
UNDERPASS AT FRASER RD  
SOIL STRATA

SHEET



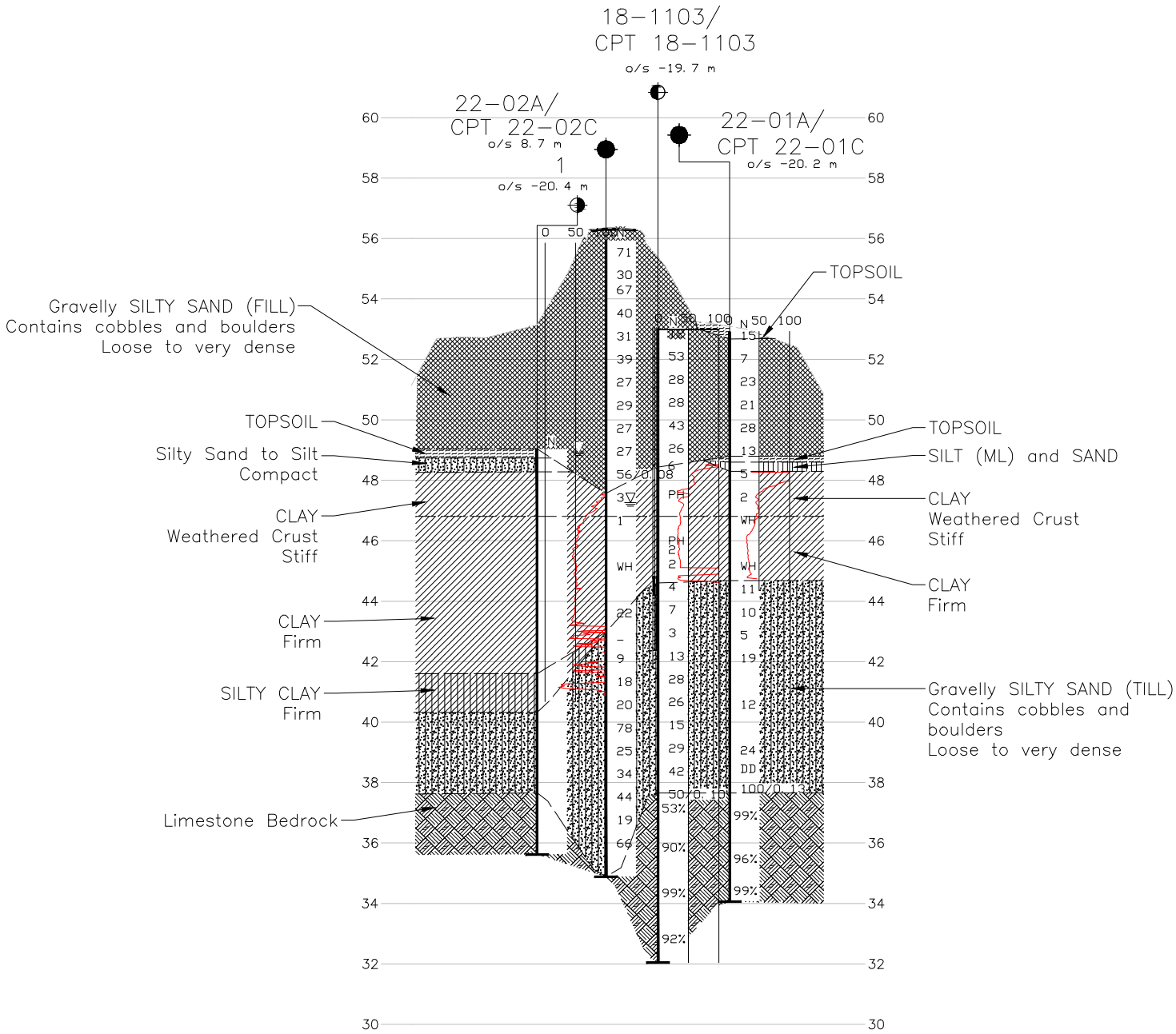
LEGEND

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1	49.0	4997620.7	222598.0



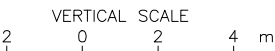
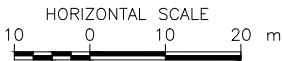
CROSS-SECTIONS D-D'-SOUTH EMBANKMENT



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GA provided in digital format by MTO, drawing file no. 188202-09-GA - Fraser Rd, received January 23, 2023.

Structural Site Location: Latitude: 44.114025 Longitude: -74.554989

NO.	DATE	BY	REVISION
Geocres No. 31G-291			
HWY. 401	PROJECT NO. 22513877		DIST. EASTERN
SUBM'D. KG	CHKD. KCP	DATE: 4/18/2024	SITE: 31-230
DRAWN: ZS/SA	CHKD. JPD	APPD. JPD	DWG. 4

**APPENDIX A**

**Record of Boreholes-Current Investigation  
Bedrock Core Photographs Figures A1 to A4**

# ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

## MINISTRY OF TRANSPORTATION, ONTARIO

### PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
		2.00 to 4.75	(10) to (4)
SAND	Coarse	0.425 to 2.00	(40) to (10)
	Medium	0.075 to 0.425	(200) to (40)
	Fine		
FINES	Classified by plasticity	<0.075	< (200)

### MODIFIERS FOR SECONDARY COMPONENTS<sup>1,2</sup>

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component ( <i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some ( <i>i.e.</i> , some sand)
≤ 10	trace ( <i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

### 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.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

#### Cone Penetration Test (CPT)

An 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<sub>t</sub>*), porewater pressure (*u*) and sleeve friction (*f<sub>s</sub>*) are recorded electronically at 25 mm penetration intervals.

#### Dynamic Cone Penetration Resistance (DCPT); N<sub>d</sub>:

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

### SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

### SOIL TESTS

w	water content
PL, w <sub>p</sub>	plastic limit
LL, w <sub>L</sub>	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 <sub>R</sub>	relative density (specific gravity, G <sub>s</sub> )
DS	direct shear test
GS	specific gravity
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 (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

### COARSE-GRAINED SOILS

#### Compactness<sup>1</sup>

Term	SPT 'N' (blows/0.3m) <sup>2</sup>
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

1. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

2. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

### FINE-GRAINED SOILS

#### Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' <sup>1,2</sup> (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

### Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

# LIST OF SYMBOLS

## MINISTRY OF TRANSPORTATION, ONTARIO

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 $= (w_L - w_P)$
NP	non-plastic
$w_s$	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
$C_{a(e)}$	secondary compression index
$C_a$	rate of secondary compression
$C_{a(e)}$	modified 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
$c'$	effective cohesion
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction $= \tan \delta$
$c_u, s_u$	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 or $q'$	$(\sigma_1 - \sigma_3)/2$ 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  $\gamma = \rho \cdot g$  (i.e., mass density multiplied by  
acceleration due to gravity)

Notes: 1  
2

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

# LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

## WEATHERING CLASSIFICATION

**Fresh (W1):** no visible sign of rock material weathering.

**Slightly Weathered (W2):** discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

**Moderately Weathered (W3): less than 50%** of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

**Highly Weathered (W4): more than 50%** of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

**Completely Weathered (W5): 100%** of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

**Residual Soil (W6): all rock material is converted to soil.** The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

## 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, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

## 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

AXJ Axial Joint	KV Karstic Void
BD Bedding	K Slickensided
BC Broken Core	LC Lost Core
CC Continuous Core	MB Mechanical Break
CL Closed	PL Planar
CO Contact	PO Polished
CU Curved	RO Rough
CT Coated	SA Slightly Altered
FLT Fault	SH Shear
FOL Foliation	SM Smooth
FR Fracture	SR Slightly Rough
GO Gouge	SY Stylolite
IN Infilled	UN Undulating
IR Irregular	VN Vein
JN Joint	VR Very Rough

## ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250



PROJECT		RECORD OF BOREHOLE		No 22-01A		SHEET 1 OF 4		METRIC					
G.W.P.		LOCATION		N 4997614.3; E 222585.9 MTM NAD 83 ZONE 9 (LAT. 45.113720; LONG. -74.544880)		ORIGINATED BY		BW					
DIST		HWY		BOREHOLE TYPE		COMPILED BY		NV					
DATUM		DATE		February 8, 2023		CHECKED BY		KCP					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	GR SA SI CL
52.9	GROUND SURFACE												
0.0	TOPSOIL												
52.7	Dark brown to black												
0.2	Moist		1	SS	15								
	Gravelly SILTY SAND (SM), with fines of slight plasticity, contains cobbles and boulders (FILL)												
	Brown to Grey		2	SS	7		52						
	Loose to compact												
	Moist												
			3	SS	23		51						
			4	SS	21		50						24 35 (41)
			5	SS	28		49						22 39 28 11
48.8	TOPSOIL		6	SS	13		48						
4.3	Dark brown to black												
48.3	Moist												
4.6	SILT (ML) and sand, some plastic fines		7	SS	5		47						0 36 (64)
	Moist to wet												
	SILTY CLAY to CLAY (CI/CH), trace sand, highly fissured, contains thin to thick laminations of silty sand (WEATHERED CRUST)		8	SS	2		46						
	Grey-brown												
	Stiff												
	W>PL												
46.8	CLAY (CH)		9	SS	WH		45						
6.1	Grey												
	Firm												
	W>PL												
44.7	Gravelly SILTY SAND (SM) (TILL)		10	SS	WH		44						
8.2	Loose to compact		11	SS	11		43						21 43 (36)
	Wet		12	SS	10								

Continued Next Page

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



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



PROJECT: 22513877A

**RECORD OF DRILLHOLE: 22-01A**

SHEET 4 OF 4

LOCATION: N 4997614.30 ;E 222585.94

DRILLING DATE: February 8, 2023

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME -CC-55 Trackmount

DRILLING CONTRACTOR: Downing

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY																FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS	DISCONTINUITY DATA			WEATH- ERING INDEX	Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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DEPTH SCALE

1 : 50



LOGGED: BW

CHECKED:



PROJECT		22513877A		RECORD OF BOREHOLE No 22-01B		SHEET 1 OF 1		METRIC					
G.W.P.		4290-15-01		LOCATION		N 4997616.0; E 222584.5 MTM NAD 83 ZONE 9 (LAT. 45.113740; LONG. -74.544900)		ORIGINATED BY		RI			
DIST		Eastern HWY 401		BOREHOLE TYPE		Power Auger, 200 mm Dia. (Hollow Stem)		COMPILED BY		NV			
DATUM		Geodetic		DATE		February 9, 2023		CHECKED BY		KCP			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>			
52.9	GROUND SURFACE												
0.0	For soil stratigraphy from 0 m to 4.6 m refer to Record of Borehole 22-01A												
48.3	SILTY CLAY to CLAY (CI/CH), trace sand, contains silty sand laminations (WEATHERED CRUST) Grey-brown w>PL		1	TO	PH							17.8	CIU
4.6			2	TO	PH							15.5	OED
			3	TP	PH							16.1	OED
46.5	CLAY (CH) Grey w>PL		4	TP	PH							15.2	CID
6.4			5	TP	PH							15.7	OED
			6	TP	PH							15.3	OED-LT
45.0	Gravelly SILTY SAND (SM) (TILL) Grey Wet												
8.1	END OF BOREHOLE												
NOTES: 1. Water level measured in screen at a depth of 3.6 m (Elev. 49.3 m) on March 7, 2023													

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PROJECT		RECORD OF BOREHOLE		No 22-02A		SHEET 2 OF 3		METRIC					
G.W.P. 4290-15-01		LOCATION		N 4997592.7; E 222607.2 MTM NAD 83 ZONE 9 (LAT. 45.113530; LONG. -74.544610)		ORIGINATED BY		BW					
DIST Eastern HWY 401		BOREHOLE TYPE		Power Auger, 200 mm Dia. (Hollow Stem), NQ Core		COMPILED BY		NV					
DATUM Geodetic		DATE		January 30, 2023		CHECKED BY		KCP					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	GR SA SI CL
--- CONTINUED FROM PREVIOUS PAGE ---													
43.0	CLAY (CH) Firm Grey brown w>PL		14	SS	WH		45						0 2 14 84
			15	SS	WH		44						
13.0	Gravelly SILTY SAND (SM), possible cobble and boulders (TILL) Loose to compact Grey Wet		16	SS	-		42					23 44 (33)	
			17	SS	9		41						
			18	SS	18		40						
			19	SS	20		39						
40.0	SILTY SAND (SM), some gravel, with fines of slight plasticity, (TILL) Compact to very dense Grey Wet		20	SS	78		38						18 37 35 10
16.0			21	SS	25		37						
			22	SS	34		36						
	- sand layer from 18.3 m to 18.5 m		23	SS	44								19 67 (14)
			24	SS	19								
36.0													

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

O 3% STRAIN AT FAILURE

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



PROJECT		RECORD OF BOREHOLE		No 22-02B		SHEET 1 OF 2		METRIC				
G.W.P. 4290-15-01		LOCATION		N 4997591.9; E 222603.3 MTM NAD 83 ZONE 9 (LAT. 45.113520; LONG. -74.544660)		ORIGINATED BY		RI				
DIST Eastern HWY 401		BOREHOLE TYPE		Power Auger, 200 mm Dia. (Hollow Stem)		COMPILED BY		NV				
DATUM Geodetic		DATE		February 1, 2023		CHECKED BY		KCP				
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>		
56.0	GROUND SURFACE											
0.0	For soil stratigraphy from 0 m to 7.3 m refer to Record of Borehole 22-02A											
48.7												
7.3	SILTY SAND (SM) Grey-brown Moist		1	TO	PH							
48.1												
7.9	SILTY CLAY (CI), trace to some sand, highly fissured (WEATHERED CRUST) Grey-brown w>PL		2	TO	PH							
			3	TP	PH							
46.9												
9.2	CLAY (CH), contains thin laminations of silty sand Grey w>PL		4	TP	PH							

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+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

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PROJECT 22513877A			RECORD OF BOREHOLE No 22-03A			SHEET 2 OF 4			METRIC				
G.W.P. 4290-15-01			LOCATION N 4997676.2; E 222569.8 MTM NAD 83 ZONE 9 (LAT. 45.114280; LONG. -74.545090)			ORIGINATED BY BW							
DIST Eastern HWY 401			BOREHOLE TYPE Power Auger, 200 mm Dia. (Hollow Stem), NQ Core			COMPILED BY NV							
DATUM Geodetic			DATE February 10, 2023			CHECKED BY KCP							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)		
--- CONTINUED FROM PREVIOUS PAGE ---													
42.0	CLAY (CH), trace sand Firm Grey w>PL						+	6.0					
10.7	SILTY CLAY (CI) Firm Grey w>PL		12	SS	1								
41.4	Gravelly SILTY SAND (SM), contains cobbles and boulders (TILL) Loose to dense Grey Wet		13	SS	7								
11.3			14	SS	30								
39.9													
12.8	LIMESTONE (BEDROCK)  Bedrock cored from 12.8 m to 18.2 m  For rock coring details see Record of Drillhole 22-03A		1	RC	REC 78%								RQD = 41%
			2	RC	REC 77%								RQD = 56%
			3	RC	REC 100%								RQD = 99%
			4	RC	REC 100%								RQD = 99%
34.5													
18.2	END OF BOREHOLE												

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PROJECT: 22513877A

## RECORD OF DRILLHOLE: 22-03A

SHEET 4 OF 4

LOCATION: N 4997676.18 ;E 222569.81

DRILLING DATE: February 10, 2023

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME -CC-55 Trackmount

DRILLING CONTRACTOR: Downing

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS	DISCONTINUITY DATA			WEATH- ERING INDEX	Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
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DEPTH SCALE

1 : 50



LOGGED: BW

CHECKED:

PROJECT 22513877A		RECORD OF BOREHOLE No 22-03B		SHEET 1 OF 2		METRIC	
G.W.P. 4290-15-01		LOCATION N 4997676.6; E 222571.1 MTM NAD 83 ZONE 9 (LAT. 45.114280; LONG. -74.545080)		ORIGINATED BY		RI	
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Dia. (Hollow Stem)		COMPILED BY		NV	
DATUM Geodetic		DATE February 12, 2023		CHECKED BY		KCP	

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



PROJECT		RECORD OF BOREHOLE				No 22-03B		SHEET 2 OF 2		METRIC							
G.W.P. 4290-15-01		LOCATION				N 4997676.6; E 222571.1 MTM NAD 83 ZONE 9 (LAT. 45.114280; LONG. -74.545080)				ORIGINATED BY RI							
DIST Eastern HWY 401		BOREHOLE TYPE				Power Auger, 200 mm Dia. (Hollow Stem)				COMPILED BY NV							
DATUM Geodetic		DATE				February 12, 2023				CHECKED BY KCP							
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 ---																
41.7	CLAY (CH), trace to some sand, trace gravel Grey w>PL		9	TP	PH												
			10	TP	PH												
11.0	Gravelly SILTY SAND (SM) (TILL) Grey Wet		11	TP	PH												
41.1																	
11.6	END OF BOREHOLE																
	NOTES:  1. Water level measured in screen at a depth of 3.3 m (Elev. 49.4 m) on March 7, 2023																

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PROJECT		RECORD OF BOREHOLE		No 22-04A		SHEET 1 OF 4		METRIC					
G.W.P. 4290-15-01		LOCATION		N 4997698.0; E 222548.7 MTM NAD 83 ZONE 9 (LAT. 45.114470; LONG. -74.545370)		ORIGINATED BY		BW					
DIST Eastern HWY 401		BOREHOLE TYPE		Power Auger, 200 mm Dia. (Hollow Stem), NQ Core		COMPILED BY		NV					
DATUM Geodetic		DATE		February 3, 2023		CHECKED BY		KCP					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	GR SA SI CL
55.9	GROUND SURFACE												
0.0	ASPHALTIC CONCRETE												
55.7													
55.5	Gravelly SAND (SP) (Pavement Structure) (FILL)		1	GS									
0.4	Grey Moist												
	Gravelly SILTY SAND (SM), contains cobbles, possible boulders (FILL)		2	SS	40		55						
	Compact to dense												
	Brown to grey brown												
	Moist												
			3	SS	11		54						
			4	SS	27								25 44 (31)
							53						
			5	SS	40								
			6	SS	24		52					NP	21 34 36 9
			7	SS	35		51						
			8	SS	38								21 44 (35)
							50						
			9	SS	32								
							49						
			10	SS	10								
48.3	TOPSOIL												
7.6	Dark brown to black												
48.0	Wet		11	SS	12		48						
7.9	SILTY SAND (SM)												
	Compact												
47.5	Grey brown												
8.4	Wet												
	CLAY (CH), highly weathered fissured (WEATHERED CRUST)		12	SS	3		47						
	Stiff												
	Grey-brown												
	w>PL												
			13	SS	WH								
46.1													
9.8							46						

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+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

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PROJECT		RECORD OF BOREHOLE		No 22-04A		SHEET 2 OF 4		METRIC							
G.W.P. 22513877A		LOCATION		N 4997698.0; E 222548.7 MTM NAD 83 ZONE 9 (LAT. 45.114470; LONG. -74.545370)		ORIGINATED BY		BW							
DIST Eastern HWY 401		BOREHOLE TYPE		Power Auger, 200 mm Dia. (Hollow Stem), NQ Core		COMPILED BY		NV							
DATUM Geodetic		DATE		February 3, 2023		CHECKED BY		KCP							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
	--- CONTINUED FROM PREVIOUS PAGE ---														
	CLAY (CH), contains thin laminations of silty sand Stiff to firm Grey with red brown banding w>PL		14	SS	PM										0 2 19 79
			15	SS	PM										
42.2															
13.7	CLAY to SILTY CLAY (CH/CI), contains thin laminations of silty sand Firm Grey w>PL		16	SS	PM										0 7 31 62
41.6															
14.3	Gravelly SILTY SAND (SM), with fines of slight plasticity (TILL) Compact to very loose Grey Wet		17	SS	12										
			18	SS	3										23 46 22 9
39.4															
16.5	SAND (SP), fine to medium, trace silt Loose Grey Wet														
38.8															
17.1	Sandy GRAVEL (GW), trace silt (TILL) Grey Wet		19	SS	8										
38.2															
17.7	END OF BOREHOLE		20	SS	100/0.03										
	NOTES: 1. Water level measured in open hole at a depth of 7.7 m (Elev. 48.2 m) upon completion of drilling.														

GTA-MTO 001 S:\CLIENTS\MTTO\FRASERROAD\02\_DATA\GINT\FRASERROAD.GPJ GAL-GTA.GDT 4/1/24 ZS gtaib.glb



GTA-MTO 001 S:\CLIENTS\MTO\FRASERROAD\02\_DATA\GINT\FRASERROAD.GPJ GAL-GTA.GDT 4/1/24 ZS gtaib.glb

Continued Next Page

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



PROJECT <u>22513877A</u>			RECORD OF BOREHOLE <b>No 22-04B</b>			SHEET 2 OF 2			METRIC						
G.W.P. <u>4290-15-01</u>			LOCATION <u>N 4997699.3; E 222547.9 MTM NAD 83 ZONE 9 (LAT. 45.114480; LONG. -74.545380)</u>			ORIGINATED BY <u>RI</u>									
DIST <u>Eastern</u> HWY <u>401</u>			BOREHOLE TYPE <u>Power Auger, 200 mm Dia. (Hollow Stem)</u>			COMPILED BY									
DATUM <u>Geodetic</u>			DATE <u>February 7, 2023</u>			CHECKED BY <u>KCP</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 $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
	--- CONTINUED FROM PREVIOUS PAGE ---														
	CLAY (CH) (WEATHERED CRUST)		3	TP	PH										
			4	TP	PH										
			5	TP	PH										
			6	TP	PH										
			7	TP	PH										
	- Coarse sand seams from a depth of 12.8 m to 14.3 m.		8	TP	PH										
			9	TP	PH										
			10	TP	PH										
41.6	SILTY CLAY (CI), some sand, some gravel (TILL)														
14.3															
41.3															
14.6	END OF BOREHOLE														

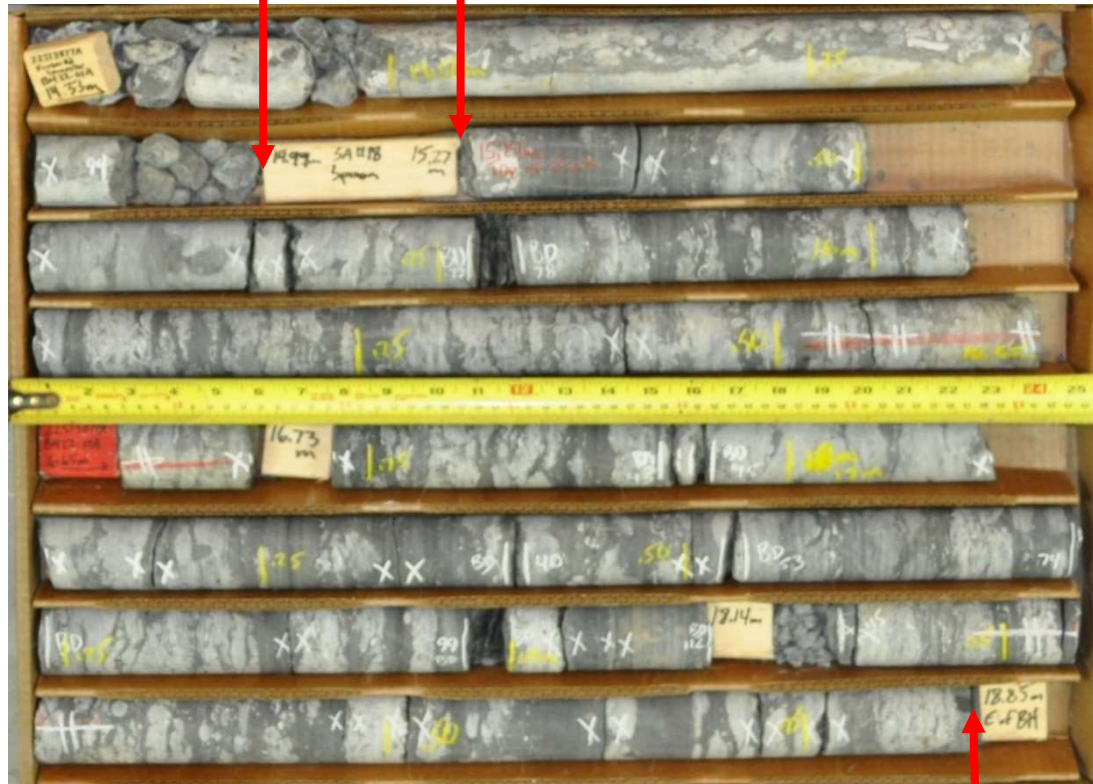
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**BH 22-01A (Dry)**  
**Core Boxes 1 and 2 of 2**

Cobbles and Boulders ends, see Note 1

Elevation 37.6 m Top of Bedrock



Elevation 34.1 m End of Drillhole

- Note:
1. Elevation 38.6 m to 37.6 m - Boulders and Cobbles
  2. Elevation 37.6 m to 34.1 m - Limestone Bedrock



**Site 31-230 - Highway 401 Underpass at Fraser Road,**  
**United Counties of Stormont, Dundas and Glengarry, Ontario**  
**Agreement No 4021-E-0021; Assignment 1**

Project No.	22513877A
Date:	2023-03-01
Drawn:	BW, KG
Review:	KCP
Approved	JPD

**Figure A1**

**BH 22-01A (Wet)**  
**Core Boxes 1 and 2 of 2**

Cobbles and Boulders ends, see note 1

Elevation 37.6 m Top of Bedrock



Elevation 34.1 m End of Drillhole

- Note:
1. Elevation 38.6 m to 37.6 m - Boulders and Cobbles
  2. Elevation 37.6 m to 34.1 m - Limestone Bedrock



**Site 31-230 - Highway 401 Underpass at Fraser Road,**  
**United Counties of Stormont, Dundas and Glengarry, Ontario**  
**Agreement No 4021-E-0021; Assignment 1**

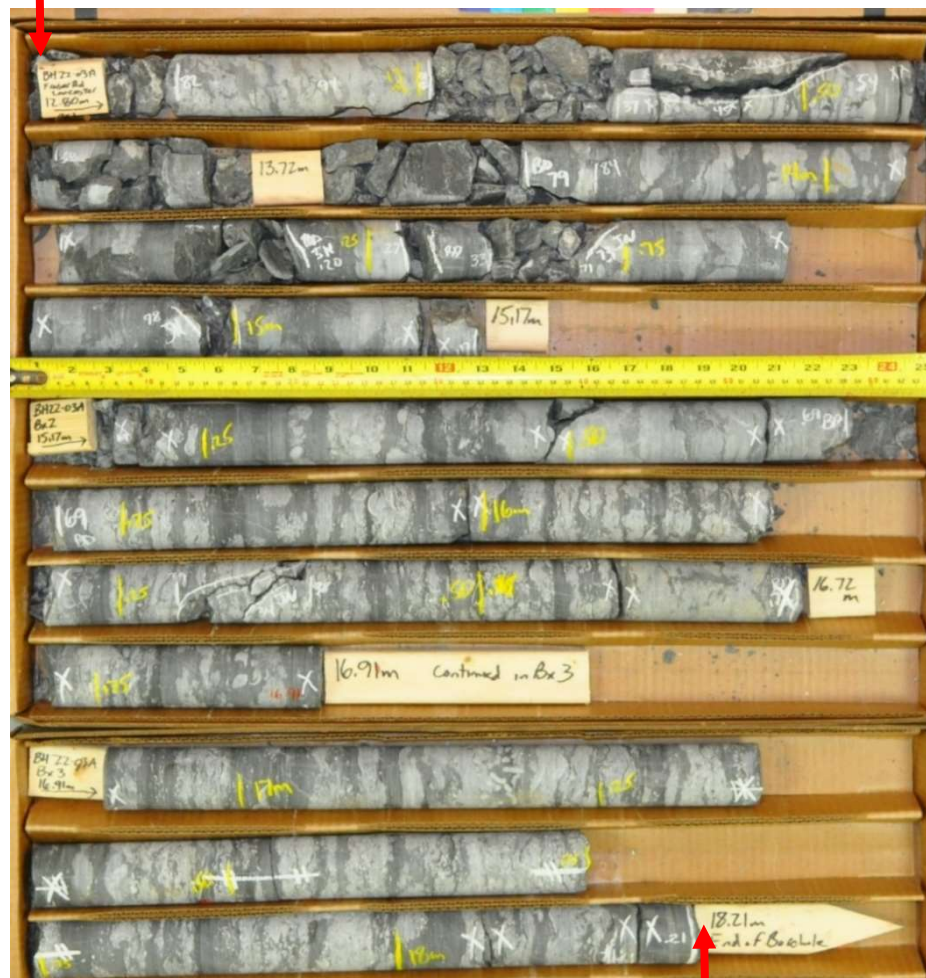
Project No.	22513877A
Date:	2023-03-01
Drawn:	BW, KG
Review:	KCP
Approved	JPD

**Figure A2**

# BH 22-03A (Dry)

## Core Boxes 1 to 3 of 3

Elevation 39.9 m  
Top of Bedrock



Elevation 34.5 m  
End of Drillhole



Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario  
Agreement No 4021-E-0021; Assignment 1

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Review:	KCP
Approved	JPD

**Figure A3**



# 22-03A (Wet)

## Core Boxs 1 to 3 of 3

Elevation 39.9 m  
Top of Bedrock



Elevation 34.5 m  
End of Drillhole



Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario  
Agreement No 4021-E-0021; Assignment 1

Project No.	22513877A
Date:	2023-03-01
Drawn:	BW, KG
Review:	KCP
Approved	JPD

**Figure A4**

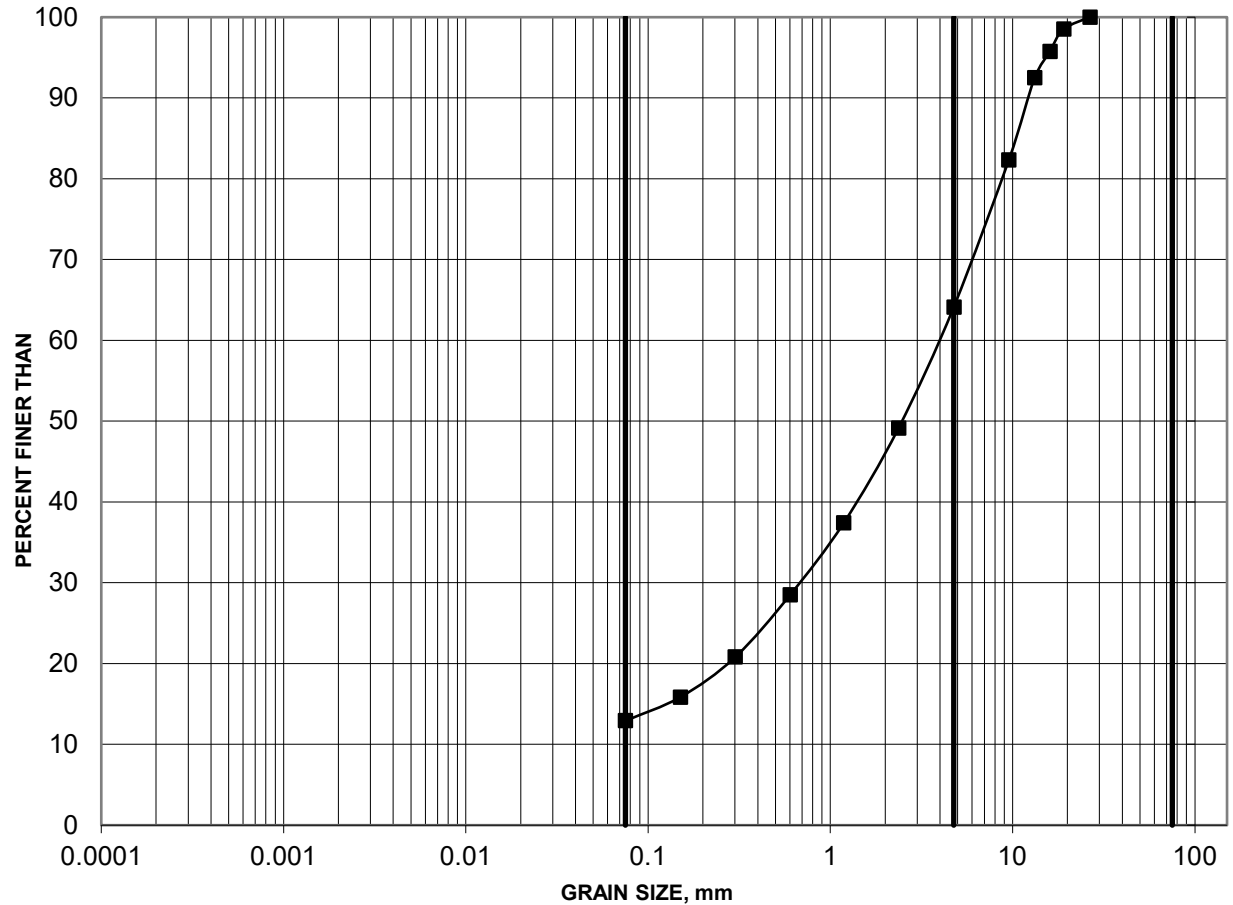
**APPENDIX B**

# Laboratory Test Results- Current Investigation

# GRAIN SIZE DISTRIBUTION

FIGURE B1

## SILTY SAND (SM) and gravel (FILL)



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

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■ 22-02A	1	0.25-0.86	36	51	13	

Project: 22513877A



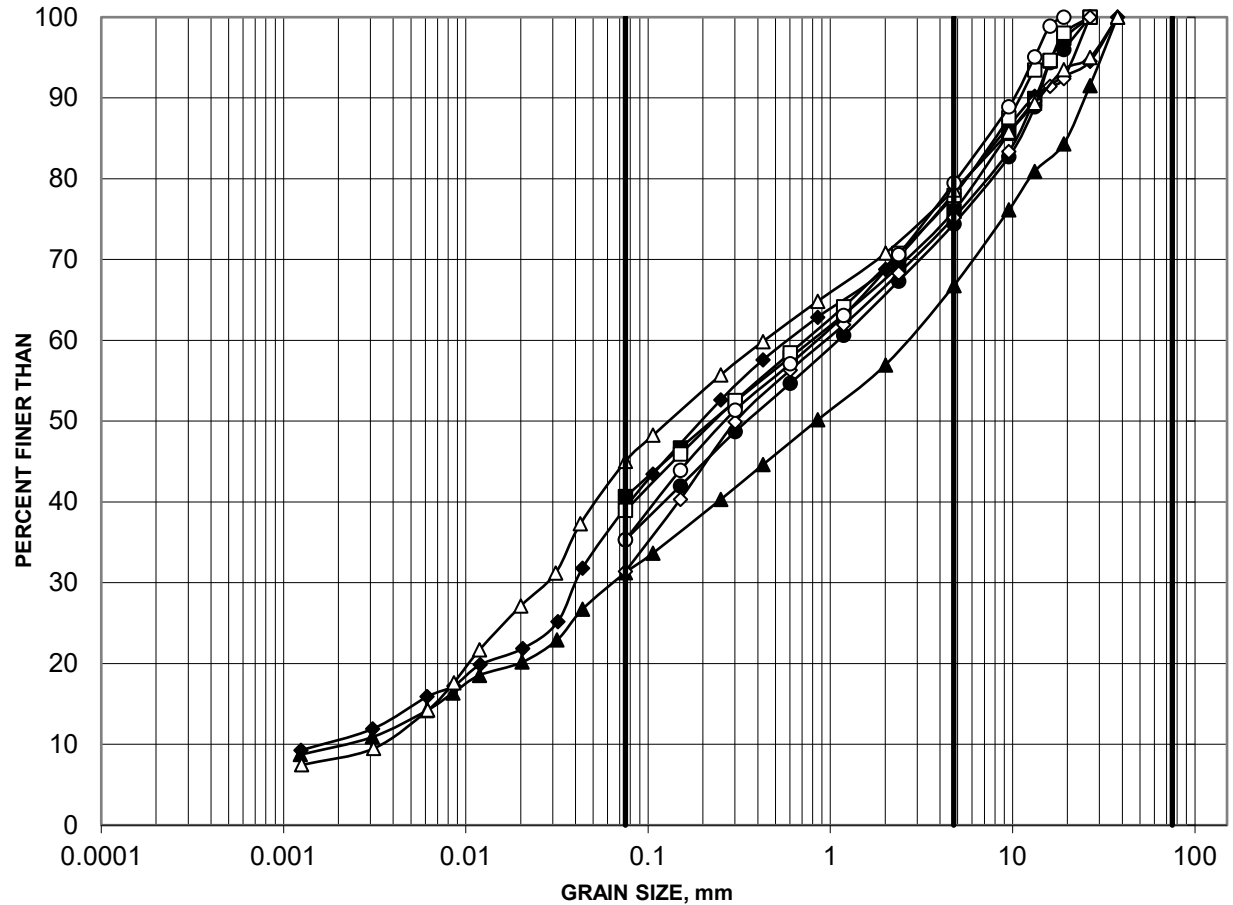
Created by: MI  
Checked by: KCP

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2022/22513877A/Figures/>

# GRAIN SIZE DISTRIBUTION

FIGURE B2

## Gravelly SILTY SAND (SM) (FILL)



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

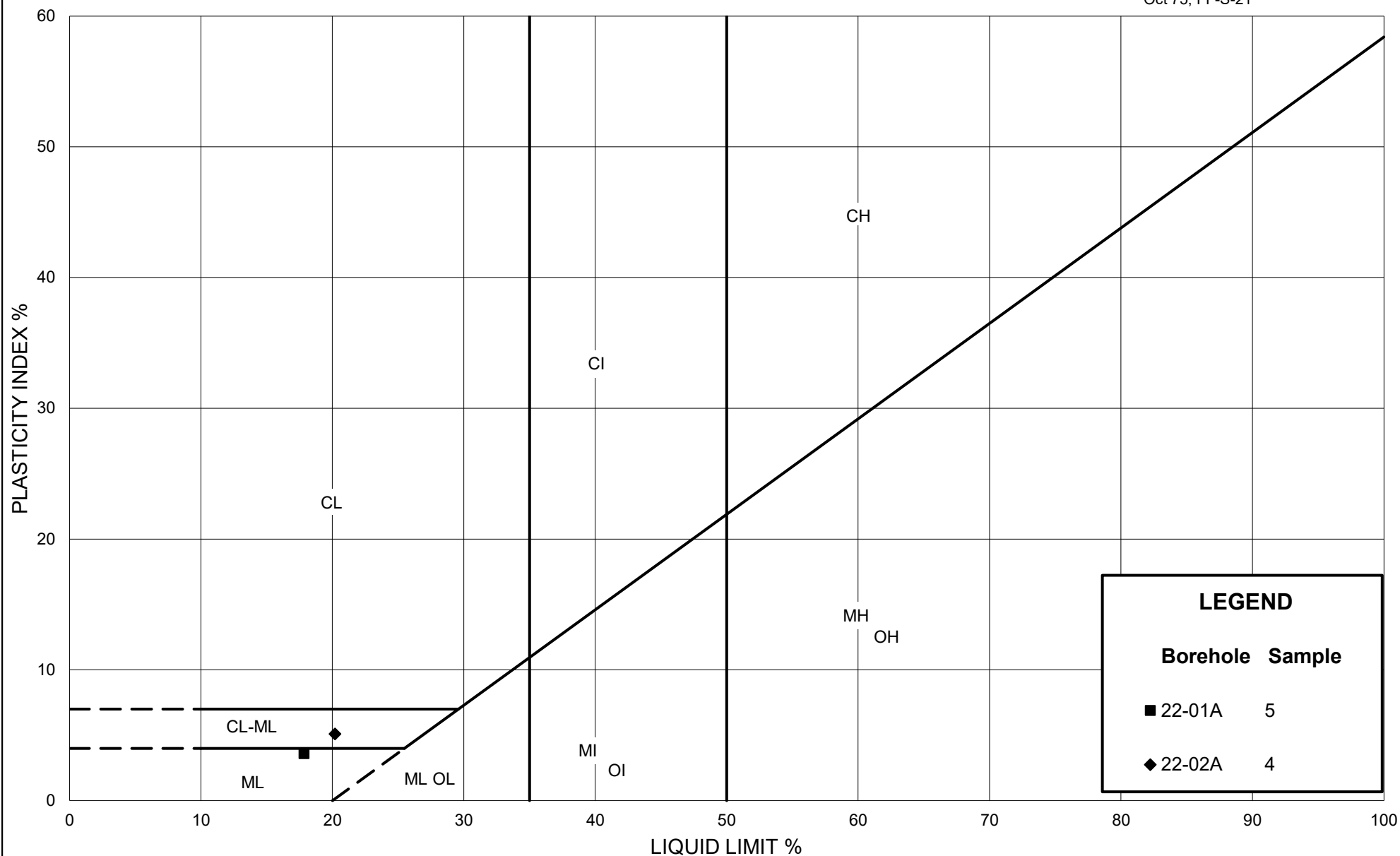
	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	22-01A	4	2.29-2.90	24	35		41
◆	22-01A	5	3.05-3.66	22	39	28	11
▲	22-02A	4	2.29-2.90	33	36	21	10
●	22-02A	9	6.10-6.71	26	39		35
□	22-03A	5	3.05-3.66	22	39		39
◇	22-04A	4	2.29-2.90	25	44		31
△	22-04A	6	3.81-4.42	21	34	36	9
○	22-04A	8	5.33-5.94	21	44		35

Project: 22513877A

**wsp** GOLDER

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# PLASTICITY CHART

Gravelly SILTY SAND (SM) (FILL) with fines of slight plasticity

Figure: B3

Project: 22513877A

Created By: MI

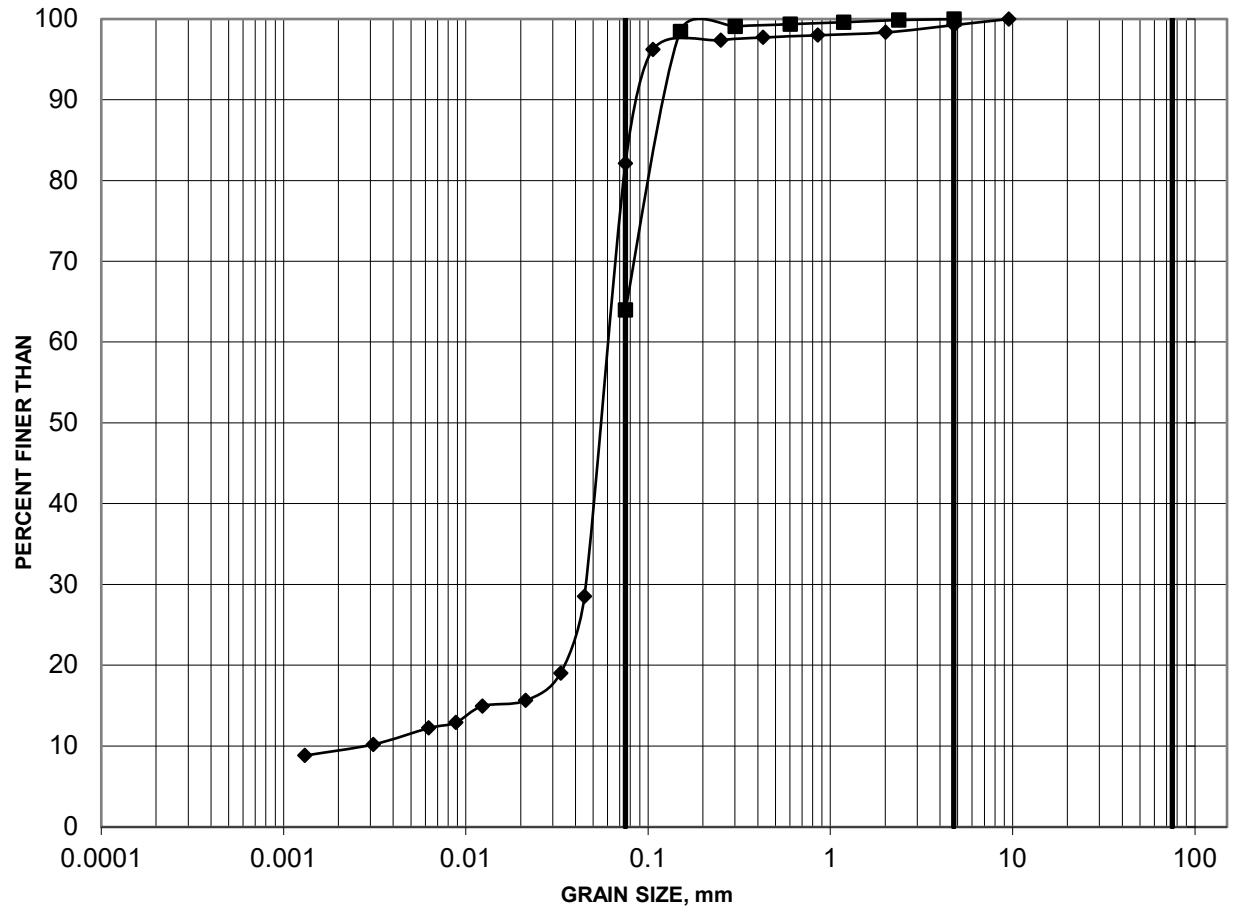
Checked By: KCP



# GRAIN SIZE DISTRIBUTION

FIGURE B4

## SILT (ML) to SILT (ML) and sand



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

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	22-01A	6C	4.27-4.42	0	36	64	
◆	22-03A	7A	4.57-4.88	1	17	72	10

Project: 22513877A



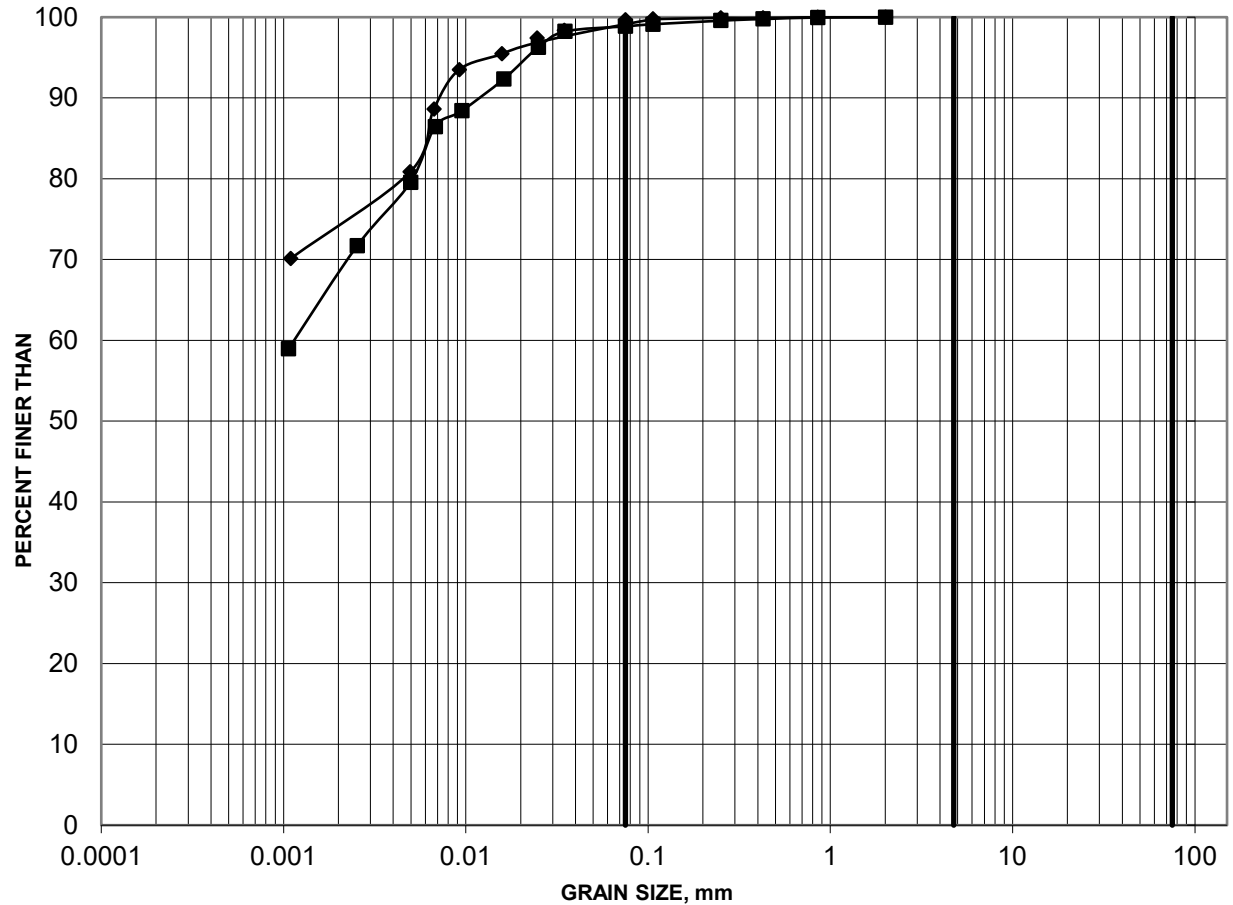
Created by: MI  
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# GRAIN SIZE DISTRIBUTION

FIGURE B5

## CLAY (CH) (Weathered Crust)



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

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	22-02A	12	8.38-8.99	0	1	31	68
◆	22-03A	8	5.33-5.94	0	0	26	74

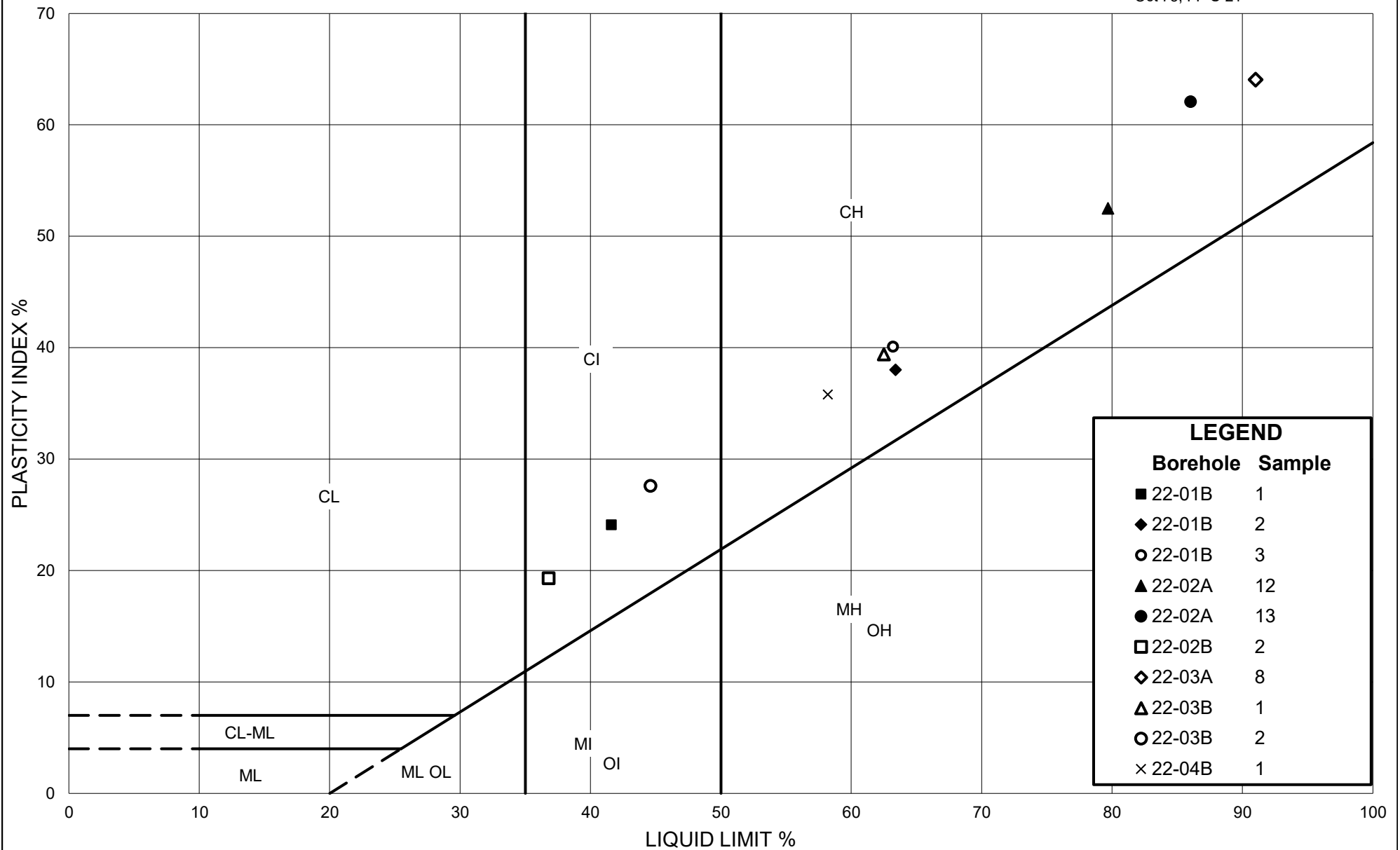
Project: 22513877A



Created by: KG

Checked by: KCP

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# PLASTICITY CHART

## SILTY CLAY (CI) to CLAY (CH) (Weathered Crust)

Figure: B6

Project: 22513877A

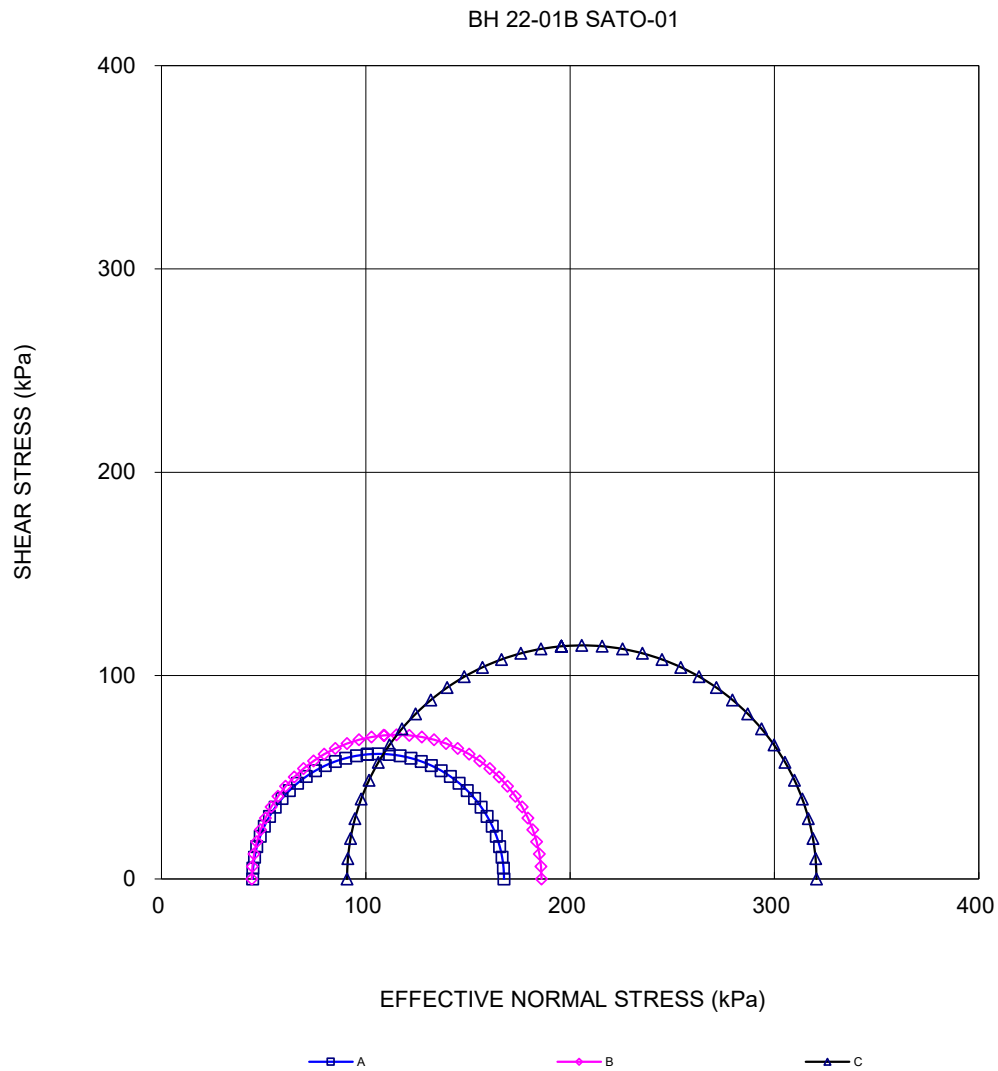
Created By: CW

Checked By: MI

<b>CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS</b> <b>ASTM D4767</b> <b>SHEET 1 OF 4</b>		<b>FIGURE B7</b>	
TEST STAGE	A	B	C
BOREHOLE NUMBER	22-01B		
SAMPLE	TO-01		
DEPTH, m	4.57-5.18		
SPECIMEN DIAMETER, cm	6.92	6.90	6.86
SPECIMEN HEIGHT, cm	14.04	14.08	14.07
NATURAL WATER CONTENT, %	34.3	41.0	48.8
DRY DENSITY, Mg/m <sup>3</sup>	1.40	1.27	1.17
WATER CONTENT AFTER SATURATION, %	35.6	43.1	50.7
CELL PRESSURE, $\sigma_3$ , kPa	320.0	370.0	470.0
BACK PRESSURE, kPa	270.0	270.0	270.0
PORE PRESSURE PARAMETER "B"	0.96	0.96	0.96
EFFECTIVE CONSOLIDATION STRESS, $\sigma_c$ , kPa	50.0	100.0	200.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	1.3	2.4	4.3
WATER CONTENT AFTER CONSOLIDATION, %	34.7	41.2	47.1
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, HOURS	29.8	7.0	5.3
WATER CONTENT AFTER TEST, %	35.3	41.4	48.6
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	123.0	141.8	229.8
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ maximum, %	14.9	3.5	2.6
MAX EFFECTIVE PRINCIPAL STRESS RATIO, $(\sigma'_1 / \sigma'_3)$ maximum	4.7	4.2	3.6
DEVIATOR STRESS AT $(\sigma'_1 / \sigma'_3)$ maximum, kPa	88.2	140.4	229.7
AXIAL STRAIN AT $(\sigma'_1 / \sigma'_3)$ maximum, %	2.3	3.0	2.8
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ maximum	0.04	0.39	0.48
PORE PRESSURE PARAMETER, Af, AT $(\sigma'_1 / \sigma'_3)$ maximum	0.30	0.40	0.48
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:  Effective consolidation stresses are assigned by the client. Specimen A taken 1-16 cm from top of tube. Specimen B taken 16-31cm from top of tube. Specimen C taken 31-46cm from top of tube.			
FAILURE PLANE NUMBER	-	1.0	1.0
ANGLE OF FAILURE PLANE, DEGREES	Bulged	60.0	50.0
Date: 3/3/2023 Project No. 22513877A			
<b>WSP Canada Inc</b>		Prepared By: LH Checked By: MM	

CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
ASTM D4767  
SHEET 2 OF 4

FIGURE B7



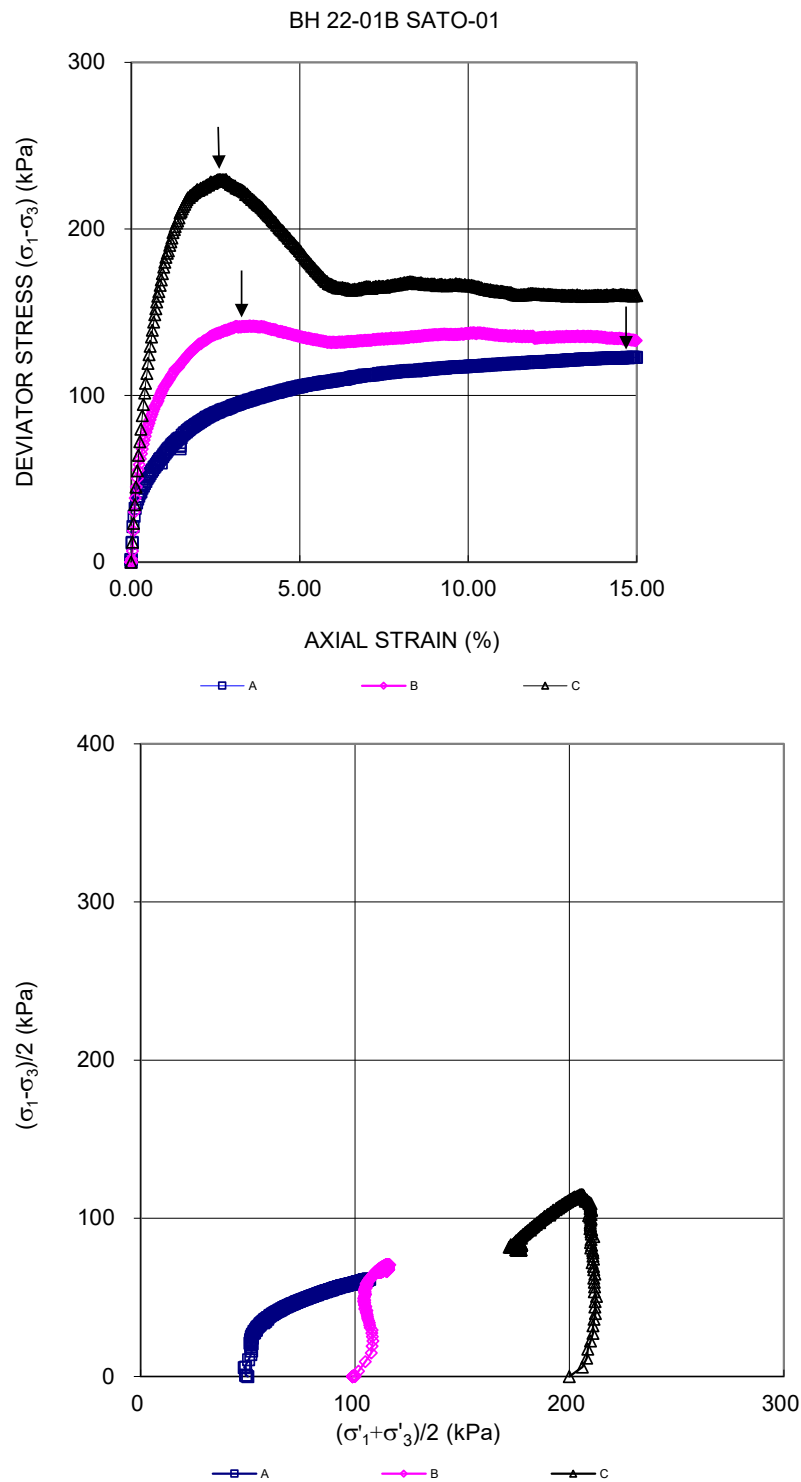
Date: 3/3/2023  
Project No. 22513877A

WSP Canada Inc

Prepared By: LH  
Checked By: MM

**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
ASTM D4767  
SHEET 3 OF 4**

**FIGURE B7**



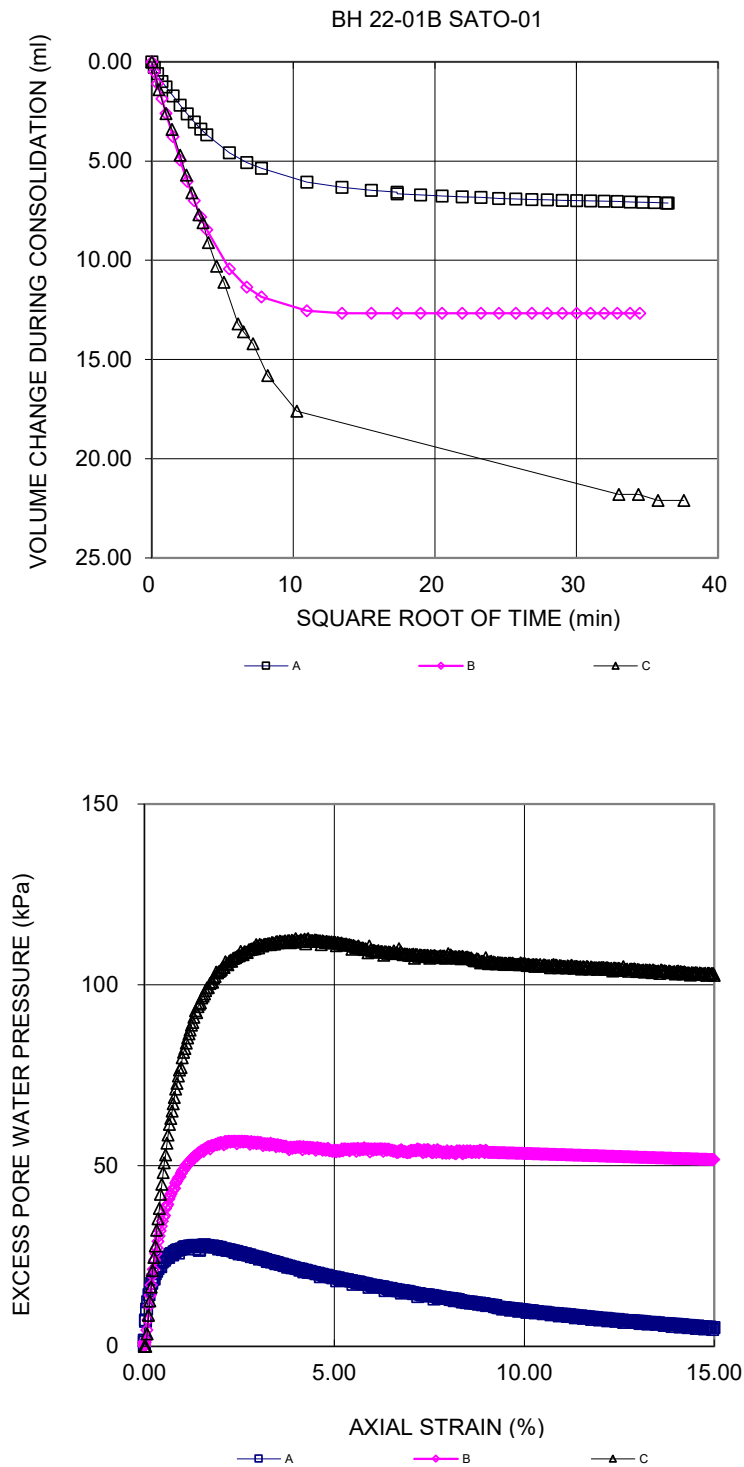
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Project No. 22513877A

**WSP Canada Inc**

Prepared By: LH  
Checked By: MM

**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
ASTM D4767  
SHEET 4 OF 4**

**FIGURE B7**



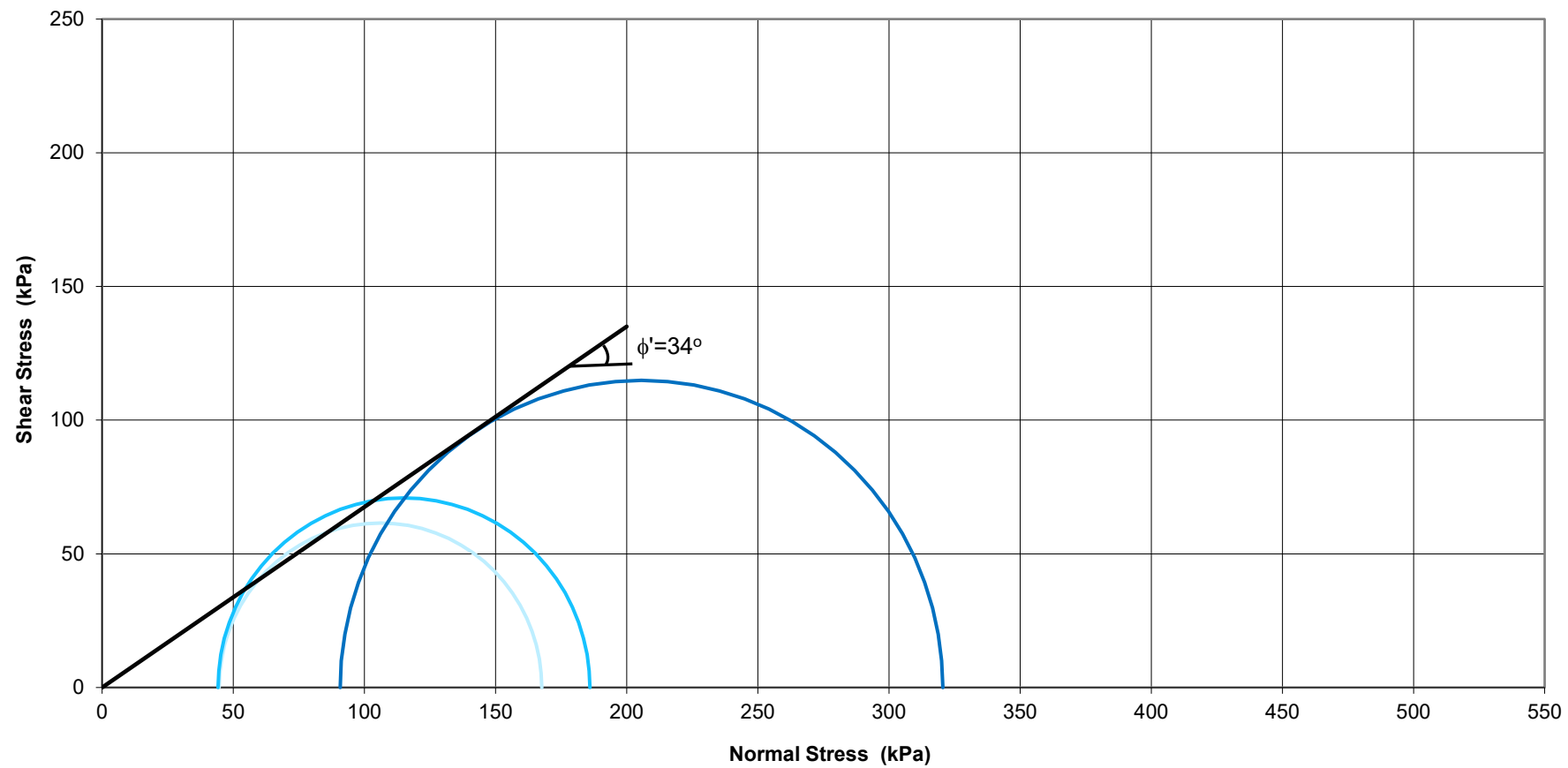
Date: 3/3/2023  
Project No. 22513877A

**WSP Canada Inc**

Prepared By: LH  
Checked By: MM

FIGURE B7

Triaxial Test Results - 22-01B/TO-01  
Peak Shear Strength Envelope



BH22-01B/TO-01/A/4.57-5.18 m

BH22-01B/TO-01/B/4.57-5.18 m

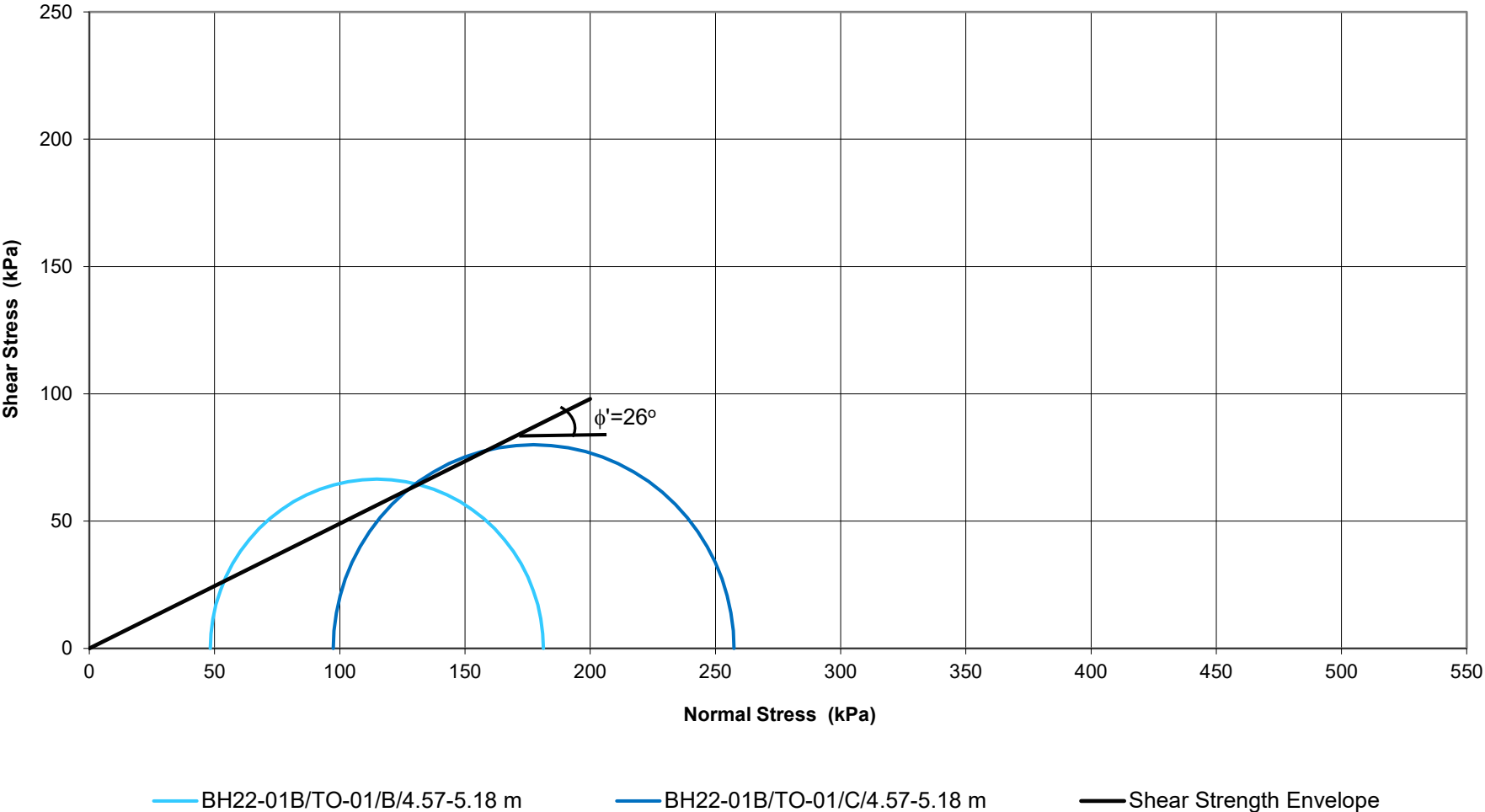
BH22-01B/TO-01/C/4.57-5.18 m

Shear Strength Envelope



FIGURE B7

Triaxial Test Results - 22-01B/TO-01  
Post Peak/ Soften Shear Strength Envelope



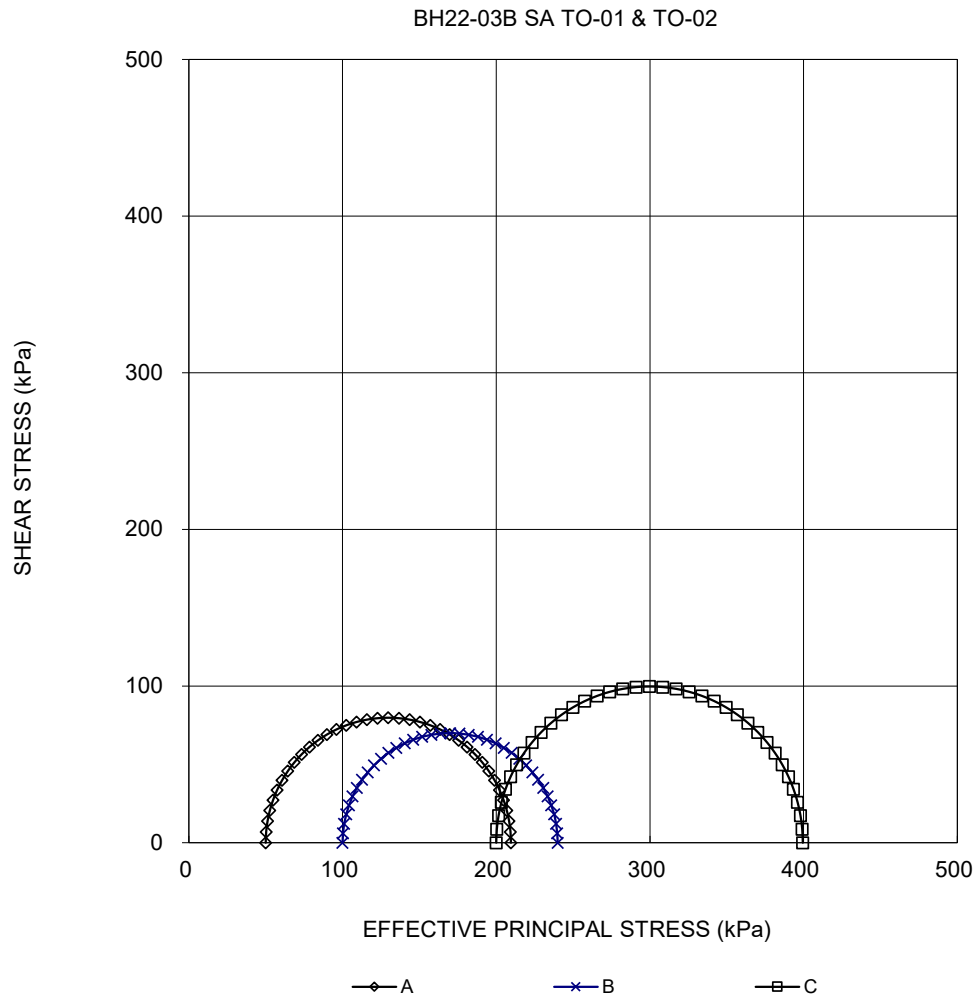
CONSOLIDATED DRAINED TRIAXIAL ASTM D7181 SHEET 1 OF 4		FIGURE B8	
TEST STAGE	A	B	C
BOREHOLE NUMBER	22-03B		
SAMPLE NUMBER	TO-01	TO-02	TO-02
DEPTH, m	4.85-5.49	5.49-6.10	5.49-6.10
SPECIMEN DIAMETER, cm	6.92	6.96	6.94
SPECIMEN HEIGHT, cm	13.98	13.99	13.99
NATURAL WATER CONTENT, %	58.4	75.2	70.0
DRY DENSITY, Mg/m <sup>3</sup>	1.03	0.89	0.93
WATER CONTENT BEFORE CONSOLIDATION, %	61.1	76.3	71.8
CELL PRESSURE, $\sigma_3$ , kPa	460.0	370.0	540.0
BACK PRESSURE, kPa	410.0	270.0	340.0
PORE PRESSURE PARAMETER "B"	0.96	0.96	0.96
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa	50.0	100.0	200.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	1.3	3.3	8.1
WATER CONTENT AFTER CONSOLIDATION, %	59.9	72.6	59.5
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, HOURS	7	8	30
WATER CONTENT AFTER TEST, %	57.9	65.4	53.1
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	159.6	140.0	199.5
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	3.3	3.8	15.0
MAX PRINCIPAL STRESS RATIO, $(\sigma'_1 / \sigma'_3)$ maximum	4.2	2.7	2.1
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
<p>Specimen A taken 24-40cm from top of tube.</p> <p>Specimen B taken 2-17cm from top of tube.</p> <p>Specimen C taken 17-33cm from top of tube.</p>			
FAILURE PLANE NUMBER	1.0	1.0	-
ANGLE OF FAILURE, DEGREES	55.0	60.0	Bulged
<div> <div>Date: 3/13/2023</div> <div>Project No. 22513877A</div> </div> <div> <div>WSP Canada Inc</div> </div> <div> <div>Prepared By: LH</div> <div>Checked By: MM</div> </div>			

CONSOLIDATED DRAINED TRIAXIAL

ASTM D7181

SHEET 2 OF 4

FIGURE B8



Date: 3/13/2023  
Project No. 22513877A

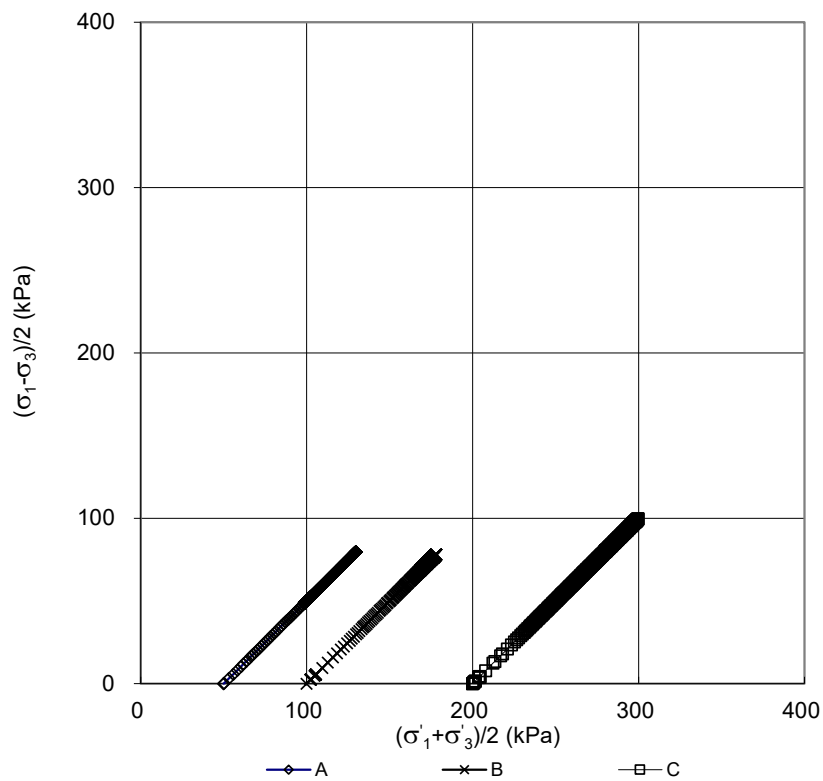
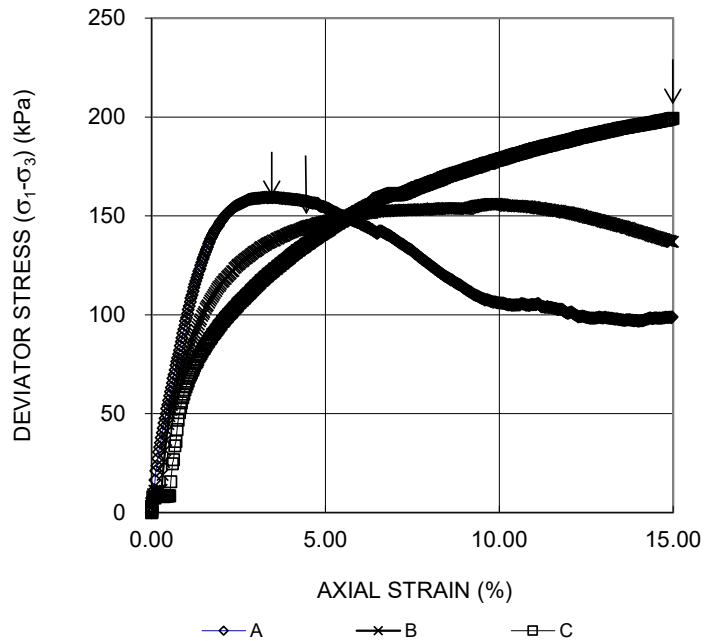
**WSP Canada Inc**

Prepared By: LH  
Checked By: MM

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

**FIGURE B8**

BH22-03B SA TO-01 & TO-02



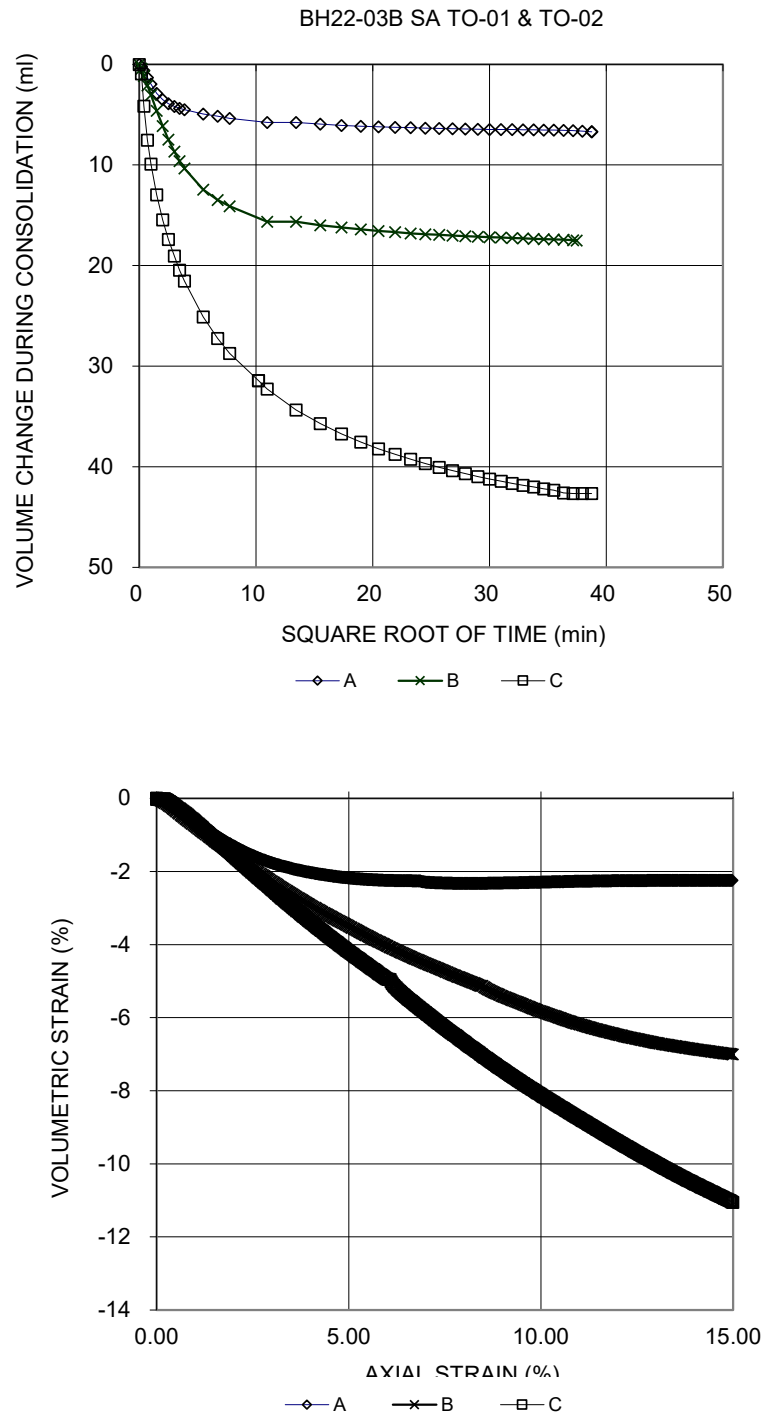
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**CONSOLIDATED DRAINED TRIAXIAL  
ASTM D7181  
SHEET 4 OF 4**

**FIGURE B8**



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

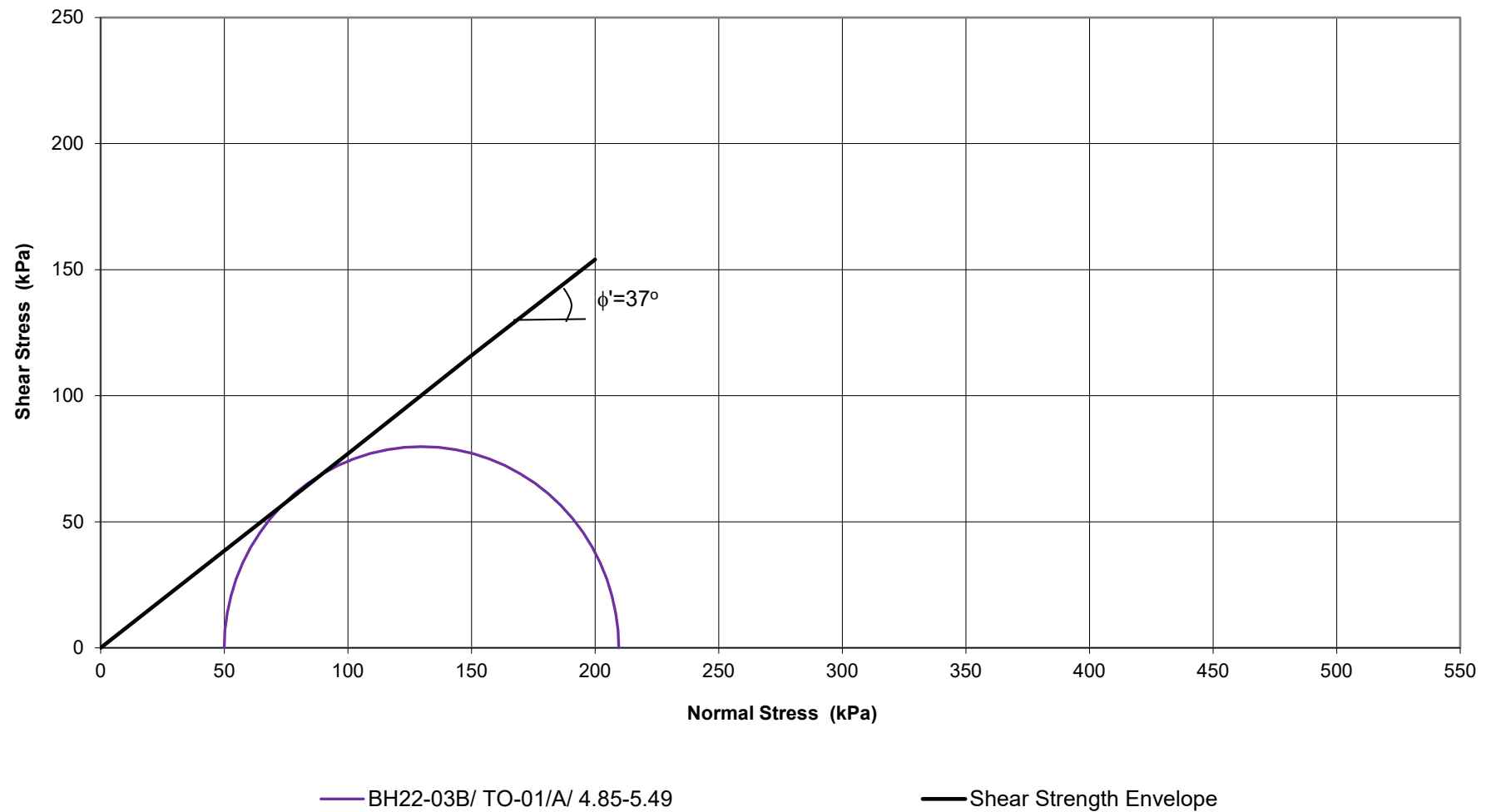
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Project No. 22513877A

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Checked By: MM

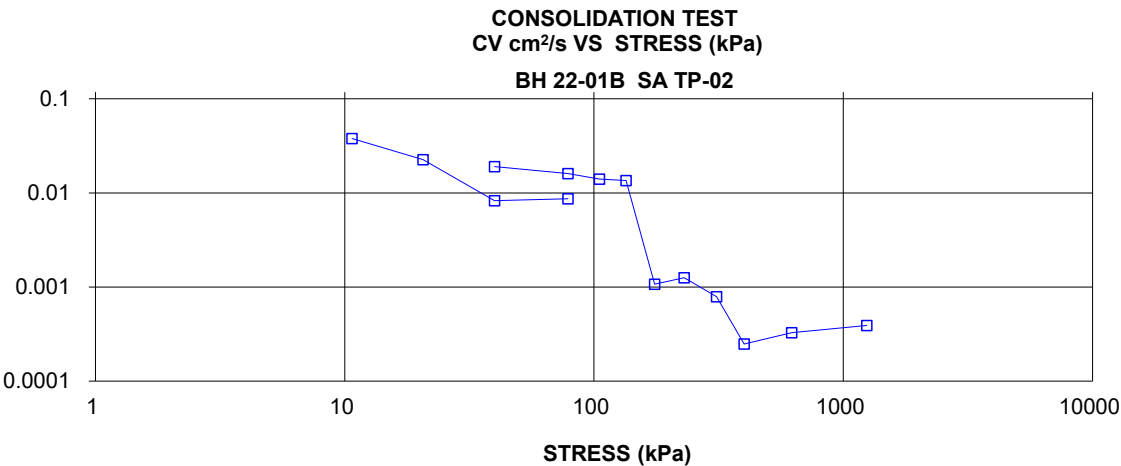
FIGURE B8

Triaxial Test Results - 22-03B/TO-01  
Peak Shear Strength Envelope

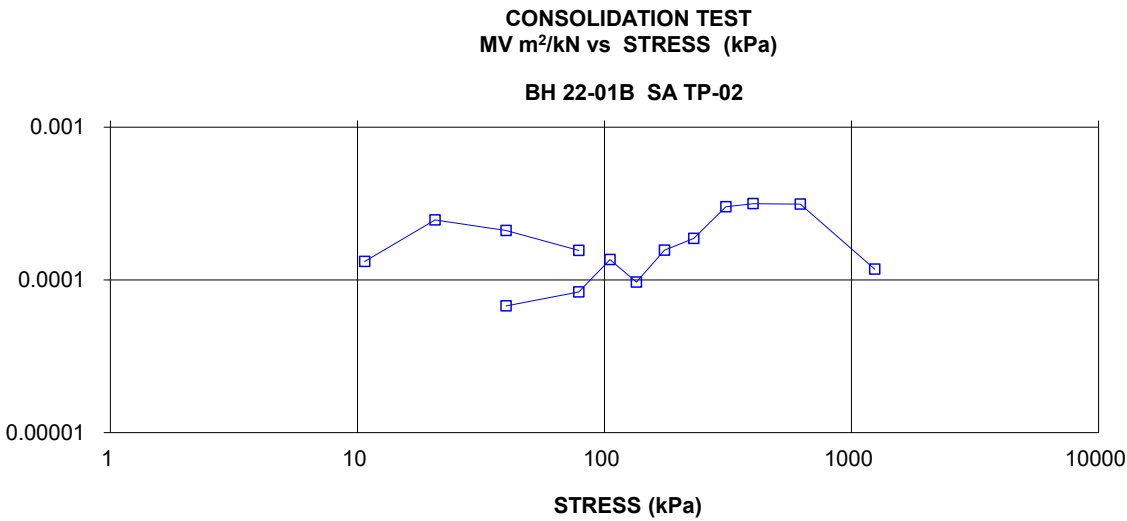


CONSOLIDATION TEST SUMMARY					FIGURE B9		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number		22513877A		Sample Number		TP-02	
Borehole Number		BH22-01B		Sample Depth, m		5.18-5.79	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		1					
Date Started		02/23/2023					
Date Completed		03/16/2023					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		2.53		Unit Weight, kN/m <sup>3</sup>		15.54	
Sample Diameter, cm		6.34		Dry Unit Weight, kN/m <sup>3</sup>		9.30	
Area, cm <sup>2</sup>		31.61		Specific Gravity, measured		2.77	
Volume, cm <sup>3</sup>		80.07		Solids Height, cm		0.868	
Water Content, %		66.98		Volume of Solids, cm <sup>3</sup>		27.42	
Wet Mass, g		126.84		Volume of Voids, cm <sup>3</sup>		52.64	
Dry Mass, g		75.96		Degree of Saturation, %		96.6	
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	1.920	2.533				
5.91	2.535	1.922	2.534				
10.70	2.533	1.920	2.534	36	3.78E-02	1.32E-04	4.89E-07
20.61	2.527	1.913	2.530	60	2.26E-02	2.47E-04	5.47E-07
39.93	2.517	1.901	2.522	163	8.27E-03	2.10E-04	1.71E-07
78.70	2.501	1.883	2.509	154	8.66E-03	1.56E-04	1.32E-07
39.93	2.505	1.887	2.503				
10.70	2.513	1.896	2.509				
39.93	2.508	1.890	2.510	70	1.91E-02	6.75E-05	1.26E-07
78.70	2.499	1.881	2.503	83	1.60E-02	8.35E-05	1.31E-07
105.17	2.490	1.870	2.495	94	1.40E-02	1.36E-04	1.87E-07
134.51	2.483	1.862	2.487	97	1.35E-02	9.69E-05	1.28E-07
175.05	2.467	1.844	2.475	1215	1.07E-03	1.57E-04	1.64E-08
229.89	2.441	1.814	2.454	1017	1.26E-03	1.87E-04	2.30E-08
309.87	2.380	1.743	2.410	1561	7.89E-04	3.02E-04	2.33E-08
399.74	2.308	1.660	2.344	4699	2.48E-04	3.15E-04	7.66E-09
620.78	2.133	1.458	2.220	3197	3.27E-04	3.13E-04	1.00E-08
1241.39	1.948	1.245	2.040	2264	3.90E-04	1.18E-04	4.49E-09
399.70	1.971	1.272	1.960				
174.75	2.006	1.313	1.989				
39.88	2.085	1.403	2.045				
10.65	2.131	1.456	2.108				
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 24-30cm from top of the tube. Specimen swelled under 5.91 KPa.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		2.13		Unit Weight, kN/m <sup>3</sup>		16.93	
Sample Diameter, cm		6.34		Dry Unit Weight, kN/m <sup>3</sup>		11.06	
Area, cm <sup>2</sup>		31.61		Specific Gravity, measured		2.77	
Volume, cm <sup>3</sup>		67.36		Solids Height, cm		0.868	
Water Content, %		53.09		Volume of Solids, cm <sup>3</sup>		27.42	
Wet Mass, g		116.29		Volume of Voids, cm <sup>3</sup>		39.93	
Dry Mass, g		75.96					
Prepared By: LH				WSP Canada Inc		Checked By: MM	

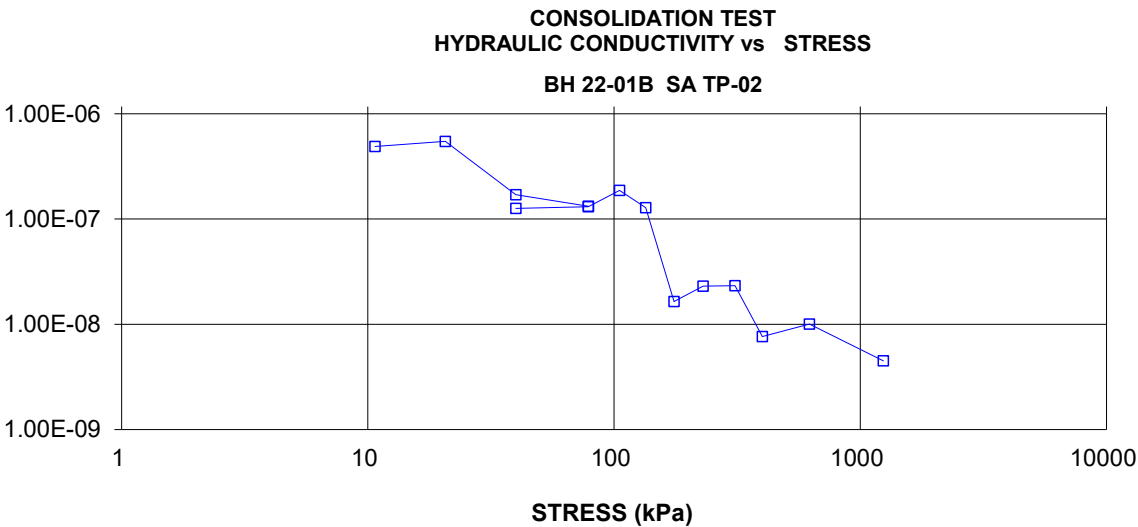
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



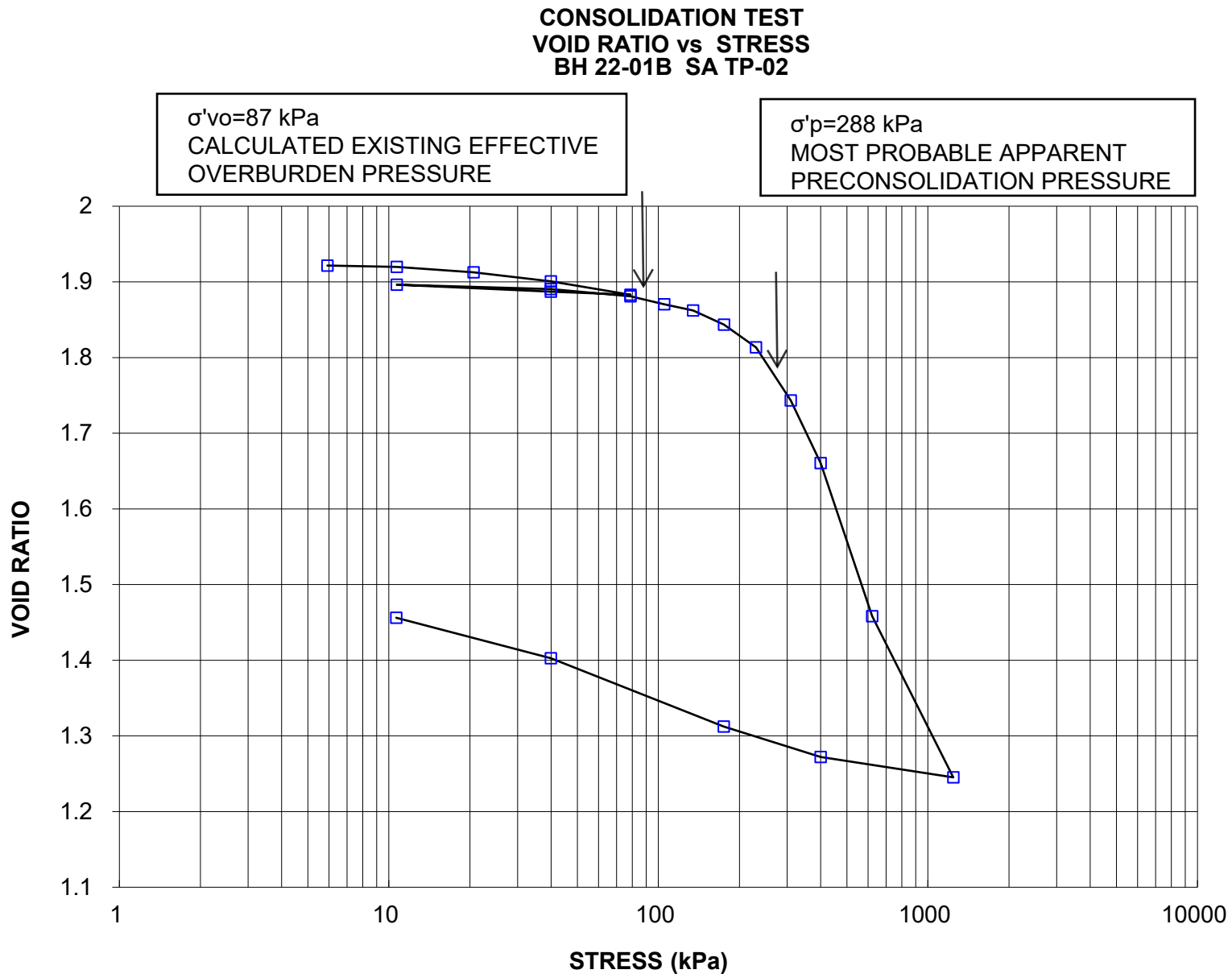
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN

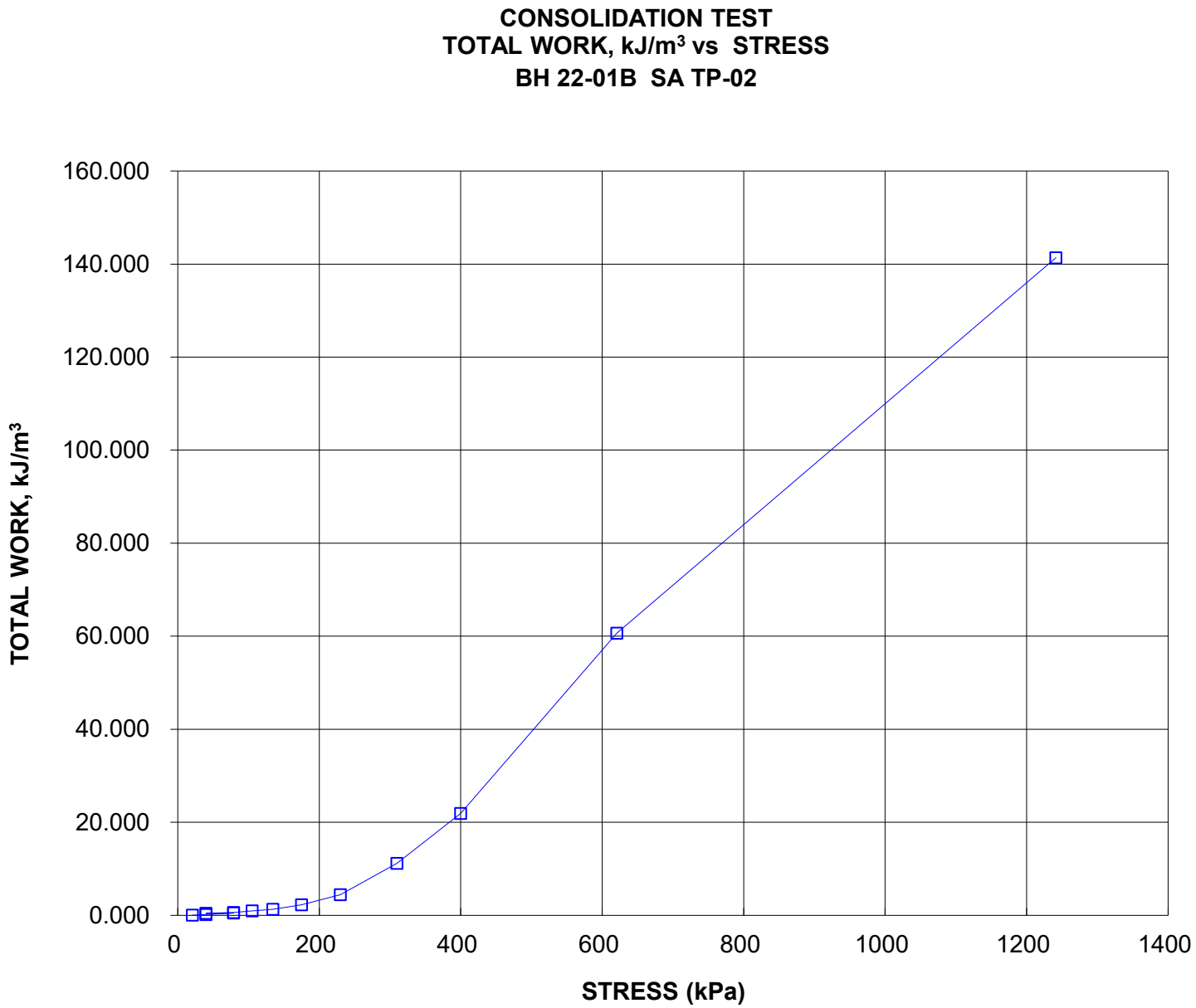


HYDRAULIC CONDUCTIVITY, cm/s

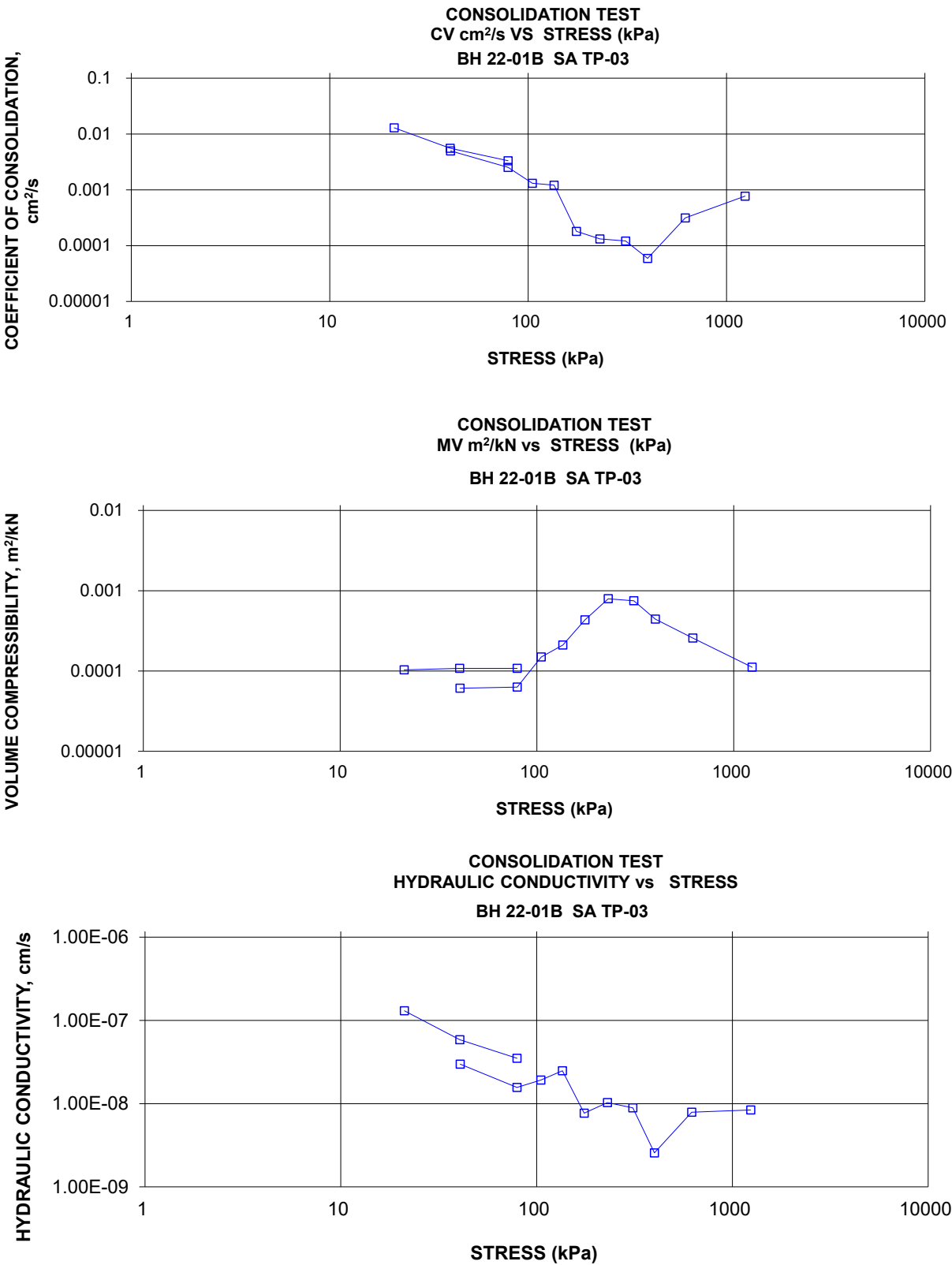


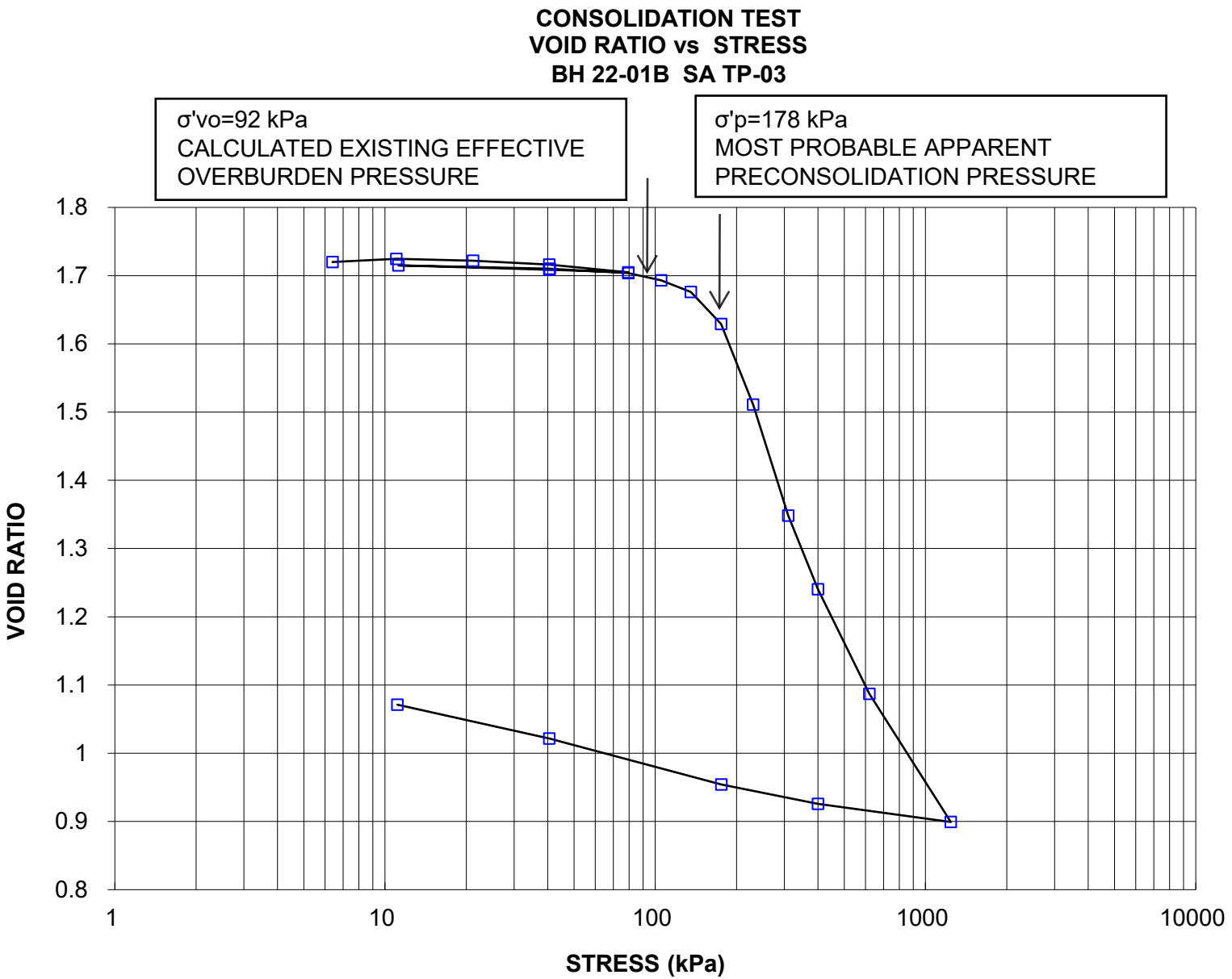






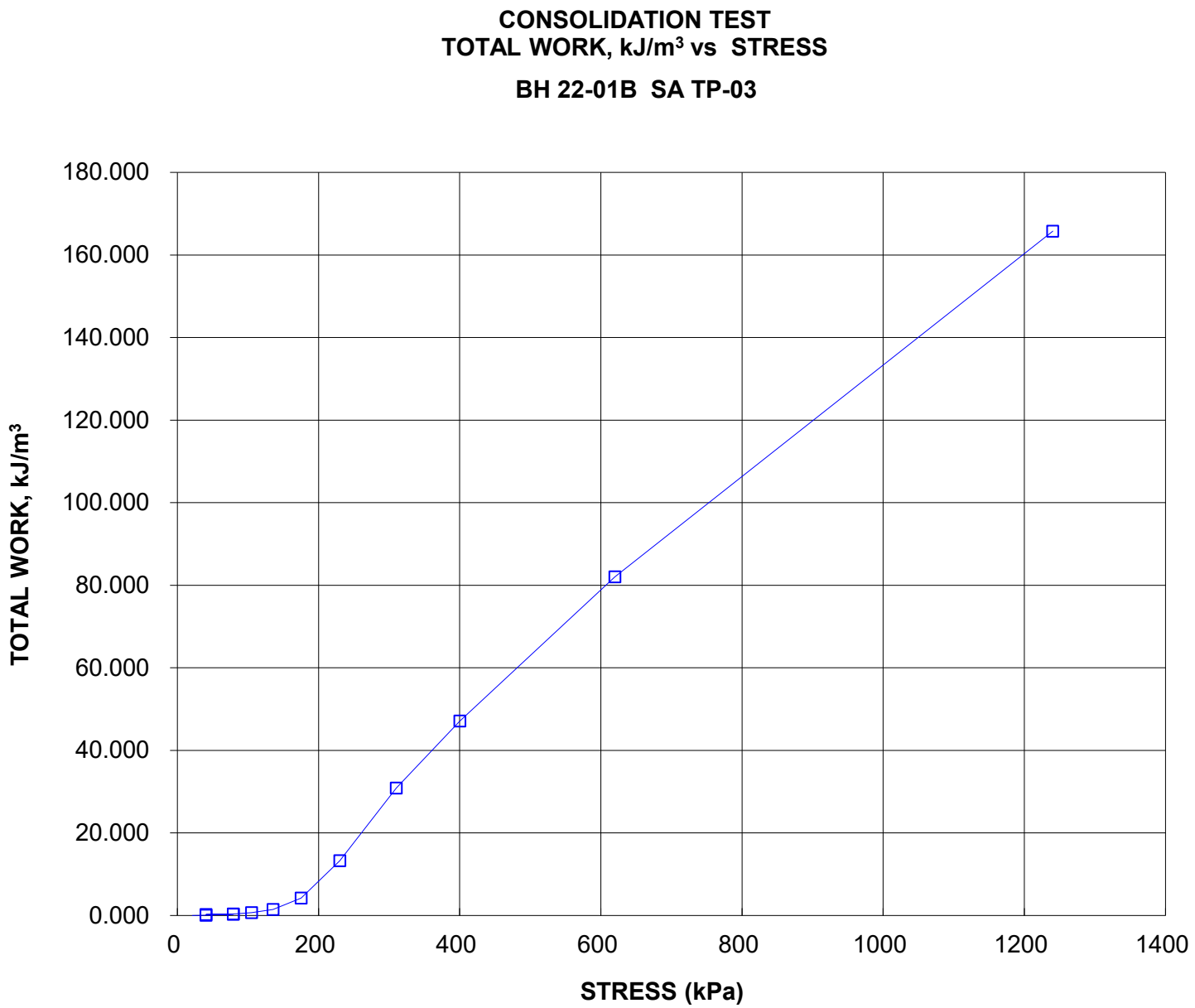
CONSOLIDATION TEST SUMMARY					FIGURE B10		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number		22513877A		Sample Number		TP-03	
Borehole Number		BH22-01B		Sample Depth, m		5.79-6.30	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		9					
Date Started		02/22/2023					
Date Completed		03/15/2023					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		1.91		Unit Weight, kN/m <sup>3</sup>		16.08	
Sample Diameter, cm		6.34		Dry Unit Weight, kN/m <sup>3</sup>		10.01	
Area, cm <sup>2</sup>		31.61		Specific Gravity, measured		2.77	
Volume, cm <sup>3</sup>		60.22		Solids Height, cm		0.702	
Water Content, %		60.73		Volume of Solids, cm <sup>3</sup>		22.18	
Wet Mass, g		98.75		Volume of Voids, cm <sup>3</sup>		38.04	
Dry Mass, g		61.44		Degree of Saturation, %		98.1	
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	1.905	1.715	1.905				
6.39	1.909	1.720	1.907				
11.00	1.912	1.725	1.910				
21.14	1.910	1.722	1.911	60	1.29E-02	1.03E-04	1.30E-07
40.51	1.906	1.716	1.908	139	5.55E-03	1.08E-04	5.87E-08
79.38	1.898	1.705	1.902	231	3.32E-03	1.08E-04	3.51E-08
40.51	1.901	1.709	1.899				
11.20	1.905	1.715	1.903				
40.70	1.902	1.710	1.904	154	4.99E-03	6.10E-05	2.98E-08
79.38	1.897	1.704	1.900	302	2.53E-03	6.31E-05	1.57E-08
105.12	1.890	1.693	1.894	581	1.31E-03	1.49E-04	1.92E-08
135.37	1.878	1.676	1.884	623	1.21E-03	2.10E-04	2.48E-08
175.24	1.845	1.629	1.861	4069	1.81E-04	4.33E-04	7.66E-09
230.03	1.762	1.511	1.803	5205	1.32E-04	7.93E-04	1.03E-08
310.25	1.648	1.348	1.705	5096	1.21E-04	7.49E-04	8.87E-09
400.07	1.572	1.240	1.610	9277	5.92E-05	4.42E-04	2.56E-09
620.00	1.465	1.087	1.518	1559	3.13E-04	2.57E-04	7.89E-09
1240.31	1.333	0.899	1.399	540	7.68E-04	1.12E-04	8.40E-09
400.07	1.351	0.926	1.342				
175.35	1.371	0.954	1.361				
40.51	1.418	1.021	1.395				
11.10	1.453	1.071	1.436				
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 swelled under 11 KPa.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		1.45		Unit Weight, kN/m <sup>3</sup>		18.19	
Sample Diameter, cm		6.34		Dry Unit Weight, kN/m <sup>3</sup>		13.12	
Area, cm <sup>2</sup>		31.61		Specific Gravity, measured		2.77	
Volume, cm <sup>3</sup>		45.94		Solids Height, cm		0.702	
Water Content, %		38.70		Volume of Solids, cm <sup>3</sup>		22.18	
Wet Mass, g		85.22		Volume of Voids, cm <sup>3</sup>		23.76	
Dry Mass, g		61.44					
Prepared By: IR		WSP Canada Inc				Checked By: MM	





**CONSOLIDATION TEST  
TOTAL WORK VS STRESS**

**FIGURE B10**



Project No. 22513877A

Prepared By: IR

**Golder Associates**

Checked By: MM

**CONSOLIDATION TEST SUMMARY**  
**ASTM D2435/D2435M**

**FIGURE B11**

**SAMPLE IDENTIFICATION**

Project Number	22513877A	Sample Number	TO-02
Borehole Number	BH22-02B	Sample Depth, m	7.92-8.53

**TEST CONDITIONS**

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	02/14/2023		
Date Completed	03/02/2023		

**SAMPLE DIMENSIONS AND PROPERTIES - INITIAL**

Sample Height, cm	1.89	Unit Weight, kN/m <sup>3</sup>	18.99
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	14.56
Area, cm <sup>2</sup>	31.71	Specific Gravity, measured	2.72
Volume, cm <sup>3</sup>	59.77	Solids Height, cm	1.029
Water Content, %	30.48	Volume of Solids, cm <sup>3</sup>	32.62
Wet Mass, g	115.76	Volume of Voids, cm <sup>3</sup>	27.15
Dry Mass, g	88.72	Degree of Saturation, %	99.6

**TEST COMPUTATIONS**

Stress kPa	Corr. Height cm	Void Ratio	Average		t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
	Height cm		Height cm					
0.00	1.885	0.832	1.885					
6.19	1.885	0.832	1.885	10	7.53E-02	7.01E-06	5.18E-08	
10.81	1.874	0.822	1.880	20	3.74E-02	1.22E-03	4.47E-06	
20.83	1.866	0.814	1.870	22	3.37E-02	4.65E-04	1.54E-06	
40.00	1.853	0.802	1.859	38	1.93E-02	3.38E-04	6.40E-07	
78.65	1.834	0.783	1.844	58	1.24E-02	2.60E-04	3.16E-07	
40.00	1.837	0.786	1.836					
10.81	1.846	0.795	1.842					
40.19	1.839	0.787	1.843	60	1.20E-02	1.39E-04	1.63E-07	
78.77	1.829	0.778	1.834	60	1.19E-02	1.39E-04	1.62E-07	
155.92	1.802	0.752	1.815	83	8.42E-03	1.83E-04	1.51E-07	
310.25	1.761	0.712	1.781	97	6.94E-03	1.42E-04	9.64E-08	
619.02	1.709	0.662	1.735	89	7.17E-03	8.82E-05	6.20E-08	
1237.16	1.646	0.600	1.678	97	6.15E-03	5.48E-05	3.30E-08	
2472.48	1.573	0.529	1.609	90	6.10E-03	3.11E-05	1.86E-08	
619.02	1.585	0.541	1.579					
156.13	1.609	0.564	1.597					
40.00	1.639	0.593	1.624					
10.89	1.659	0.613	1.649					

Note:

Consolidation loading and unloading schedule assigned by the client.

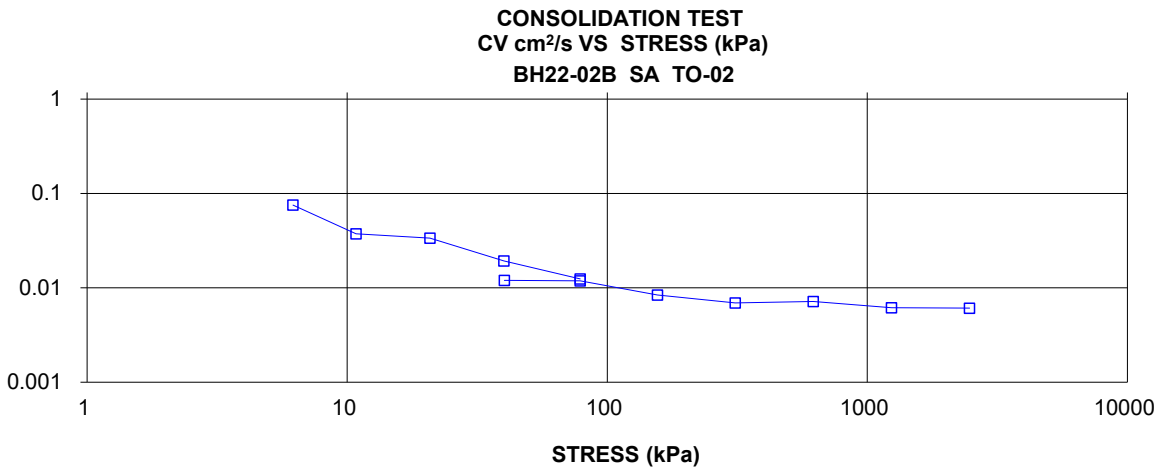
cv and k are approximate only based on t<sub>90</sub> estimated from Square Root of Time Method (ASTMD2435/2435M)

Specimen taken 19cm to 27cm from bottom of the tube.

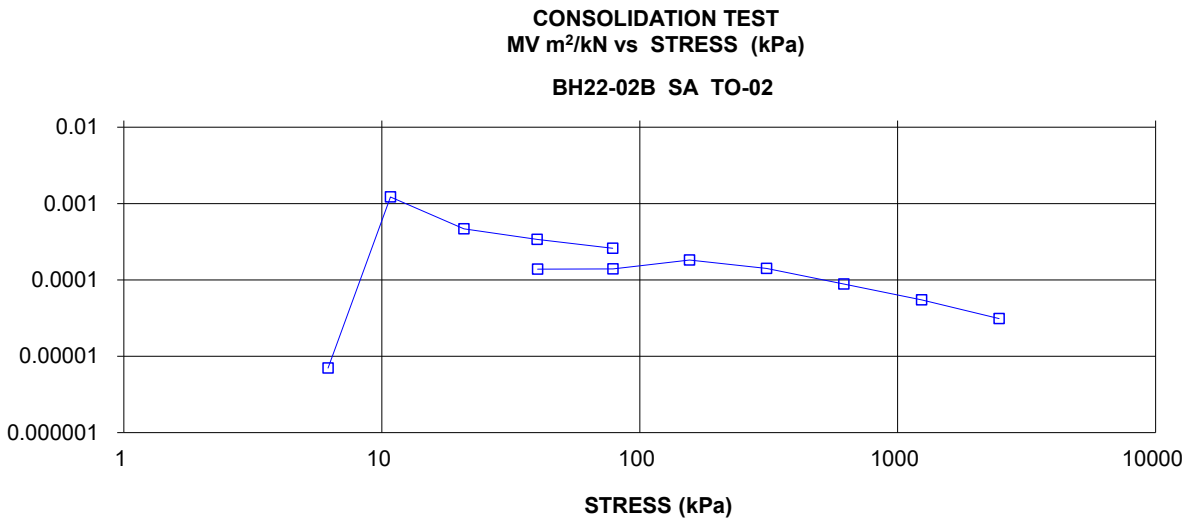
**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	1.66	Unit Weight, kN/m <sup>3</sup>	20.96
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	16.54
Area, cm <sup>2</sup>	31.71	Specific Gravity, measured	2.72
Volume, cm <sup>3</sup>	52.60	Solids Height, cm	1.029
Water Content, %	26.71	Volume of Solids, cm <sup>3</sup>	32.62
Wet Mass, g	112.42	Volume of Voids, cm <sup>3</sup>	19.98
Dry Mass, g	88.72		

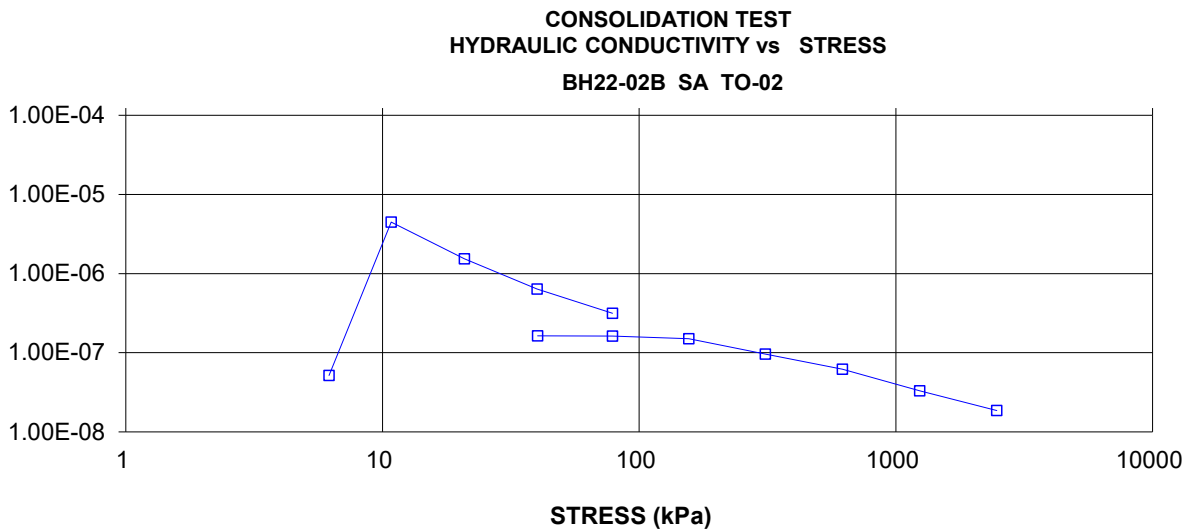
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



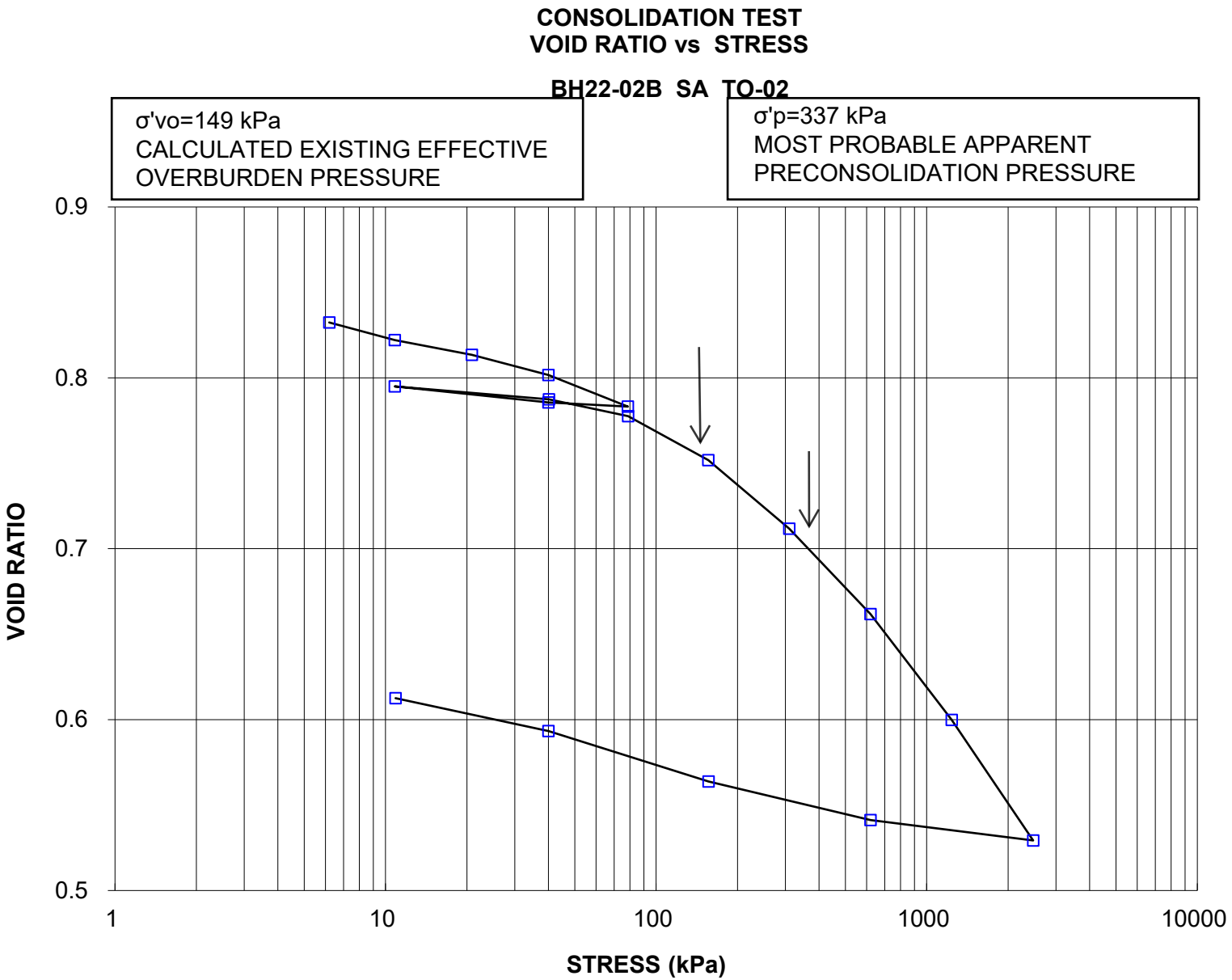
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN

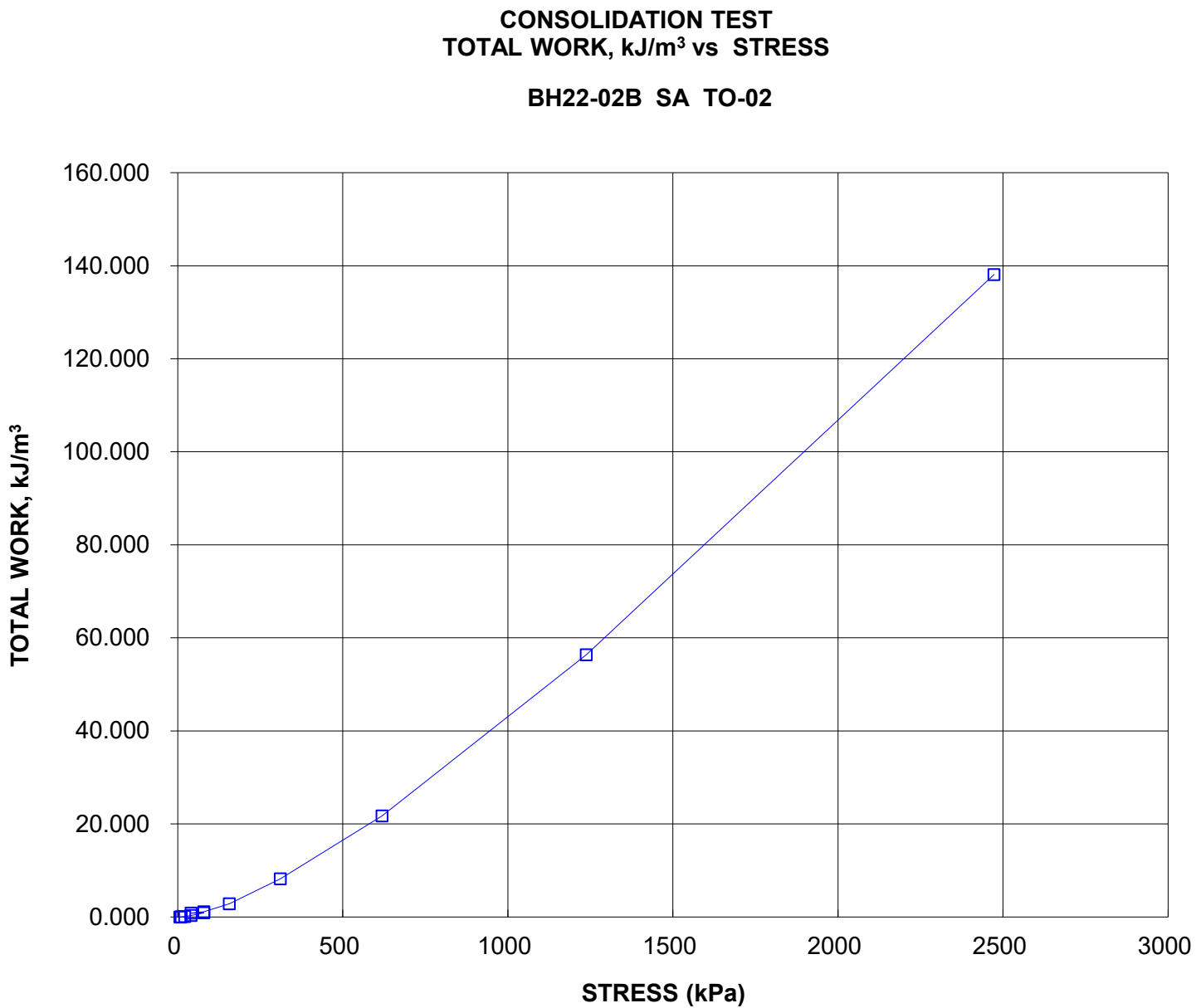


HYDRAULIC CONDUCTIVITY, cm/s

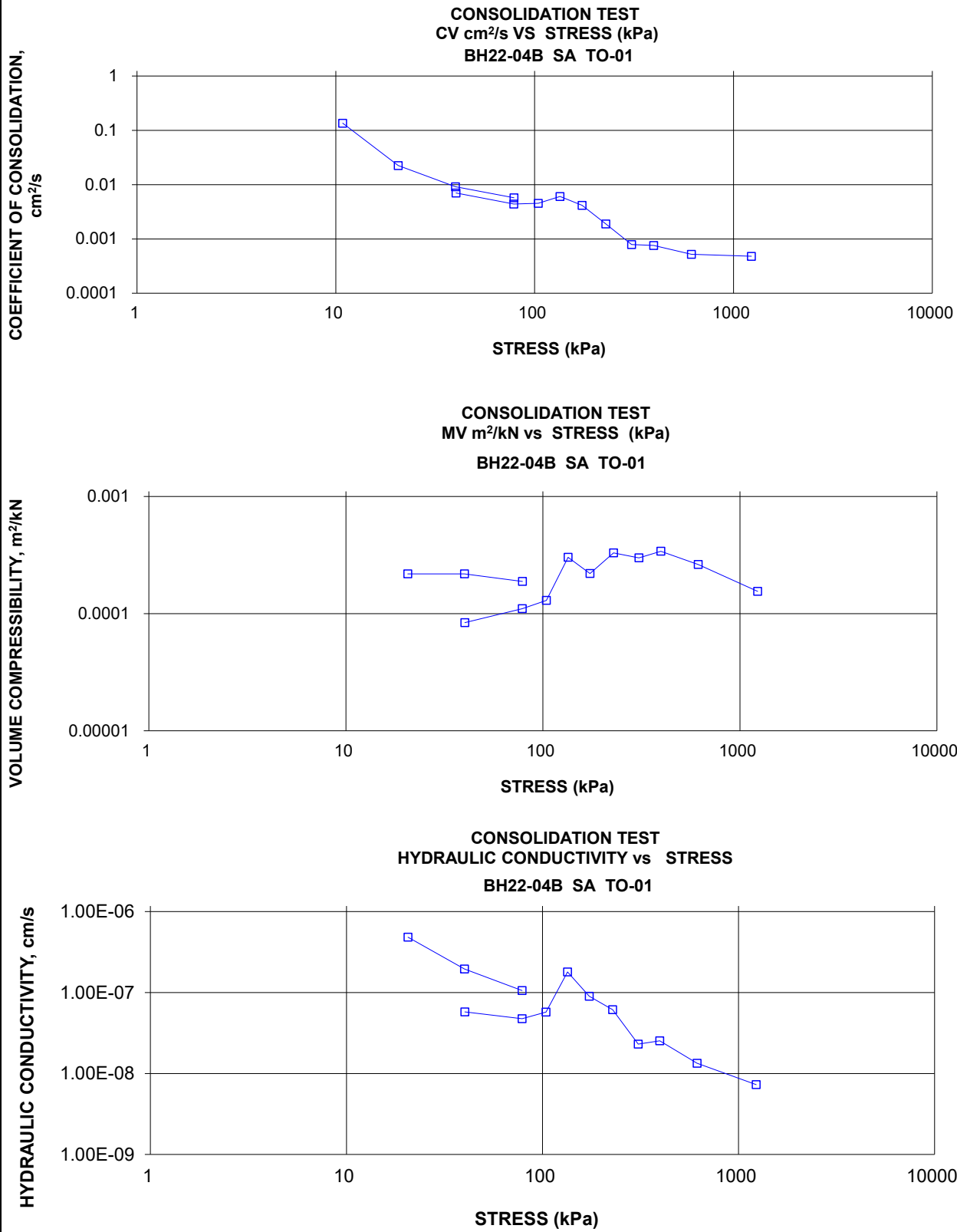


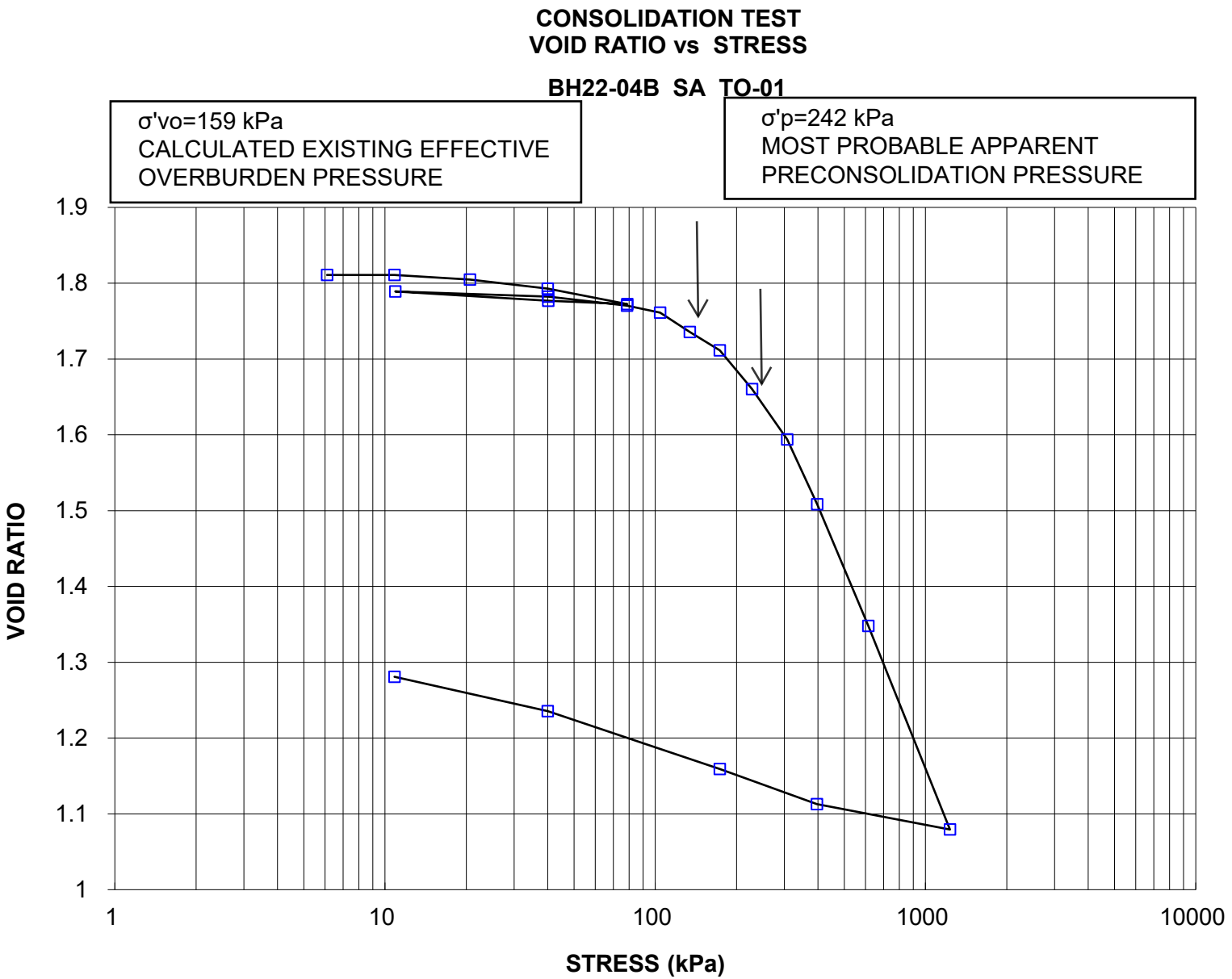


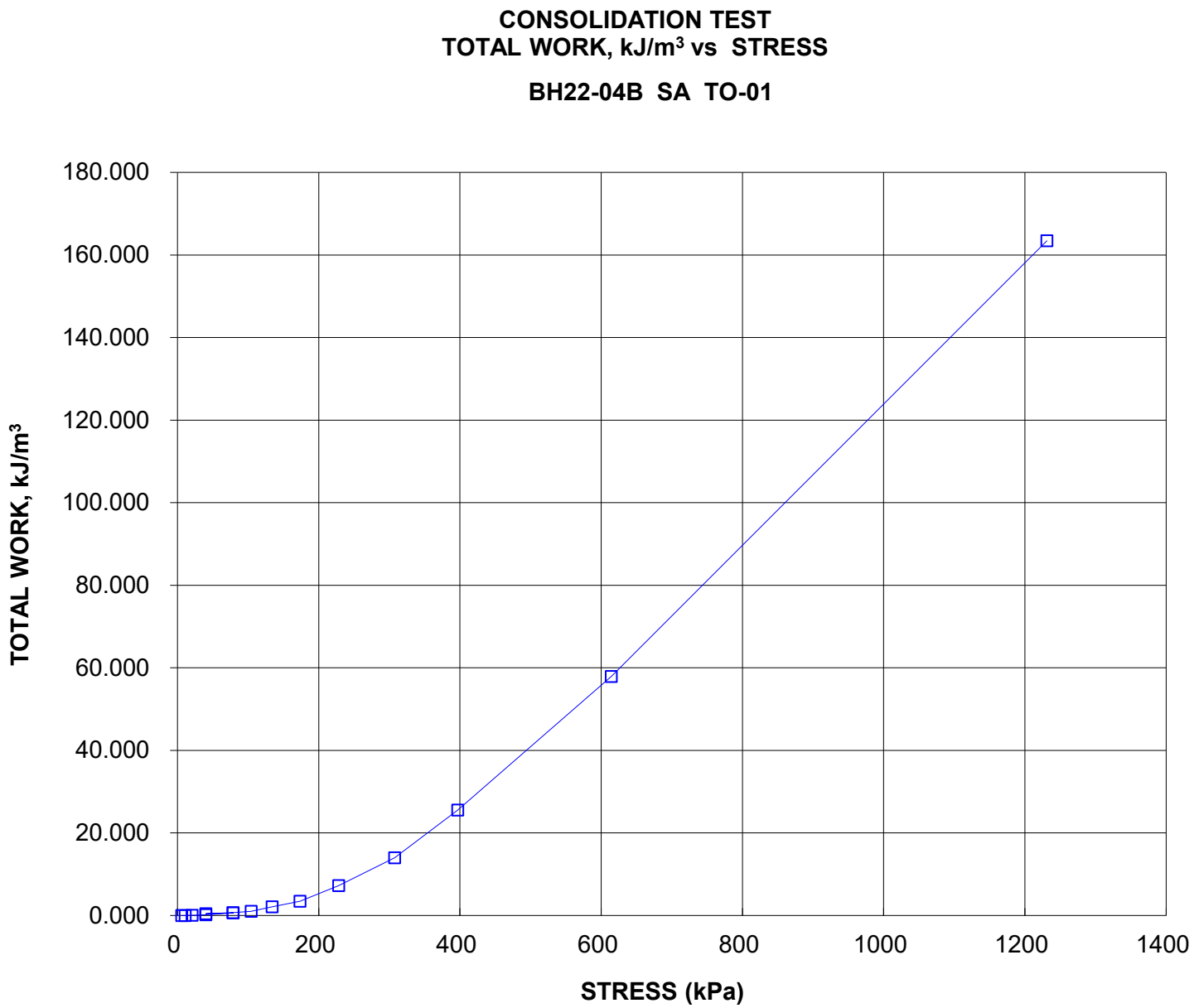




CONSOLIDATION TEST SUMMARY ASTM D2435/D2435M					FIGURE B12		
SAMPLE IDENTIFICATION							
Project Number		22513877A		Sample Number		TO-01	
Borehole Number		BH22-04B		Sample Depth, m		8.38-8.99	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		4					
Date Started		02/14/2023					
Date Completed		03/03/2023					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		2.53		Unit Weight, kN/m <sup>3</sup>		15.66	
Sample Diameter, cm		6.36		Dry Unit Weight, kN/m <sup>3</sup>		9.63	
Area, cm <sup>2</sup>		31.74		Specific Gravity, measured		2.76	
Volume, cm <sup>3</sup>		80.20		Solids Height, cm		0.899	
Water Content, %		62.60		Volume of Solids, cm <sup>3</sup>		28.53	
Wet Mass, g		128.05		Volume of Voids, cm <sup>3</sup>		51.67	
Dry Mass, g		78.75		Degree of Saturation, %		95.4	
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	2.527	1.811	2.527				
6.10	2.527	1.811	2.527				
10.83	2.527	1.811	2.527	10	1.35E-01	0.00E+00	0.00E+00
20.61	2.522	1.805	2.524	60	2.25E-02	2.18E-04	4.82E-07
39.99	2.511	1.793	2.516	147	9.13E-03	2.18E-04	1.95E-07
78.72	2.492	1.772	2.502	231	5.74E-03	1.88E-04	1.06E-07
40.18	2.496	1.777	2.494				
10.90	2.507	1.789	2.502				
40.18	2.501	1.782	2.504	189	7.03E-03	8.38E-05	5.78E-08
78.61	2.491	1.770	2.496	300	4.40E-03	1.10E-04	4.75E-08
104.26	2.482	1.761	2.486	290	4.52E-03	1.30E-04	5.74E-08
134.18	2.459	1.736	2.471	214	6.05E-03	3.03E-04	1.79E-07
173.34	2.437	1.711	2.448	305	4.17E-03	2.20E-04	8.99E-08
228.15	2.392	1.660	2.415	652	1.90E-03	3.30E-04	6.13E-08
307.50	2.332	1.594	2.362	1500	7.88E-04	2.99E-04	2.31E-08
396.75	2.255	1.508	2.293	1467	7.60E-04	3.41E-04	2.54E-08
614.42	2.111	1.348	2.183	1940	5.21E-04	2.63E-04	1.34E-08
1231.07	1.869	1.079	1.990	1750	4.80E-04	1.55E-04	7.28E-09
396.50	1.899	1.113	1.884				
173.34	1.941	1.159	1.920				
39.99	2.010	1.235	1.975				
10.83	2.050	1.281	2.030				
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 24cm to 29cm from top of the tube. Specime swelled under 6.10kPa.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		2.05		Unit Weight, kN/m <sup>3</sup>		17.43	
Sample Diameter, cm		6.36		Dry Unit Weight, kN/m <sup>3</sup>		11.87	
Area, cm <sup>2</sup>		31.74		Specific Gravity, measured		2.76	
Volume, cm <sup>3</sup>		65.08		Solids Height, cm		0.899	
Water Content, %		46.90		Volume of Solids, cm <sup>3</sup>		28.53	
Wet Mass, g		115.68		Volume of Voids, cm <sup>3</sup>		36.55	
Dry Mass, g		78.75					
Prepared By: IR		WSP Canada Inc				Checked By: MM	



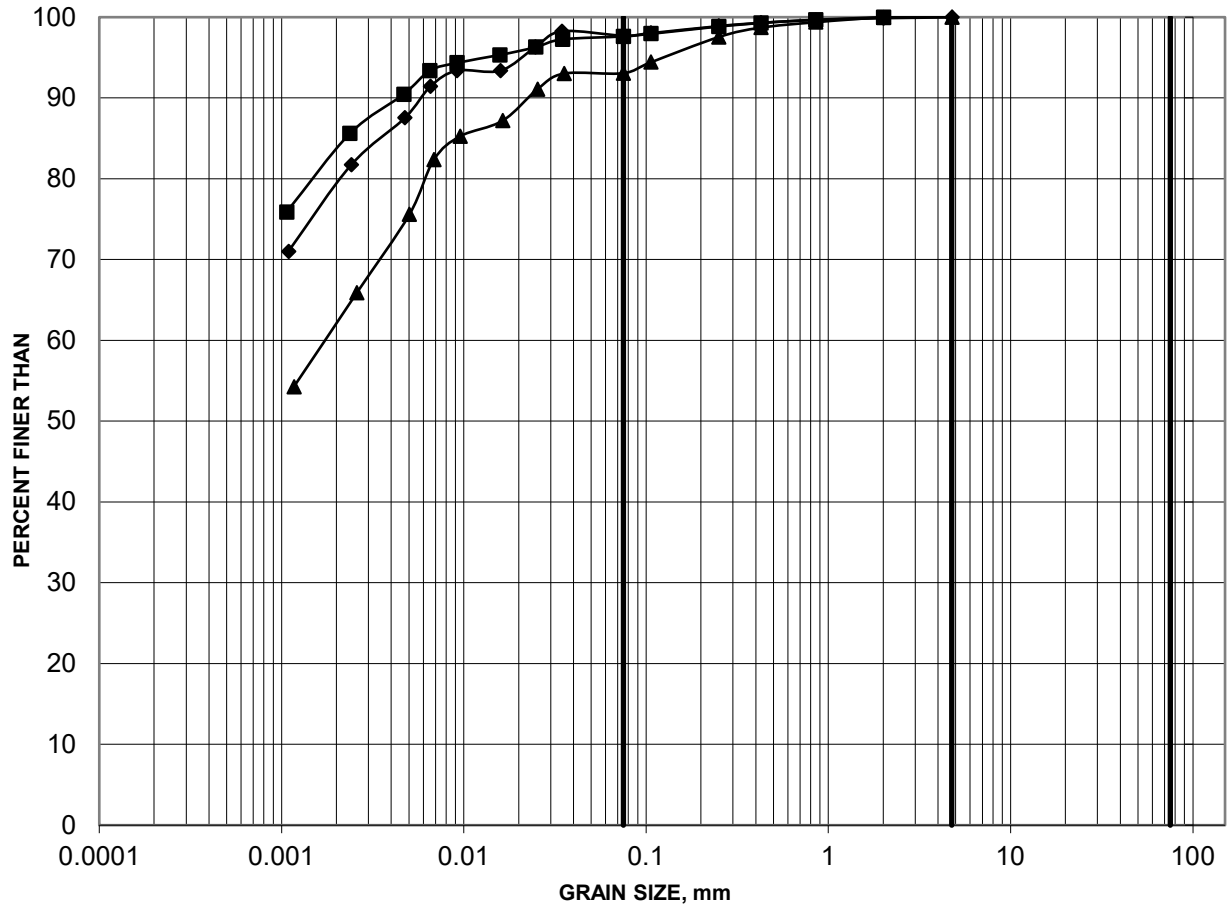




# GRAIN SIZE DISTRIBUTION

FIGURE B13

## CLAY (CH) (Unweathered)



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

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
22-02A	14	10.67-11.28	0	2	14	84
22-04A	14	10.67-11.28	0	2	19	79
22-04A	16	13.72-14.33	0	7	31	62

Project: 22513877A



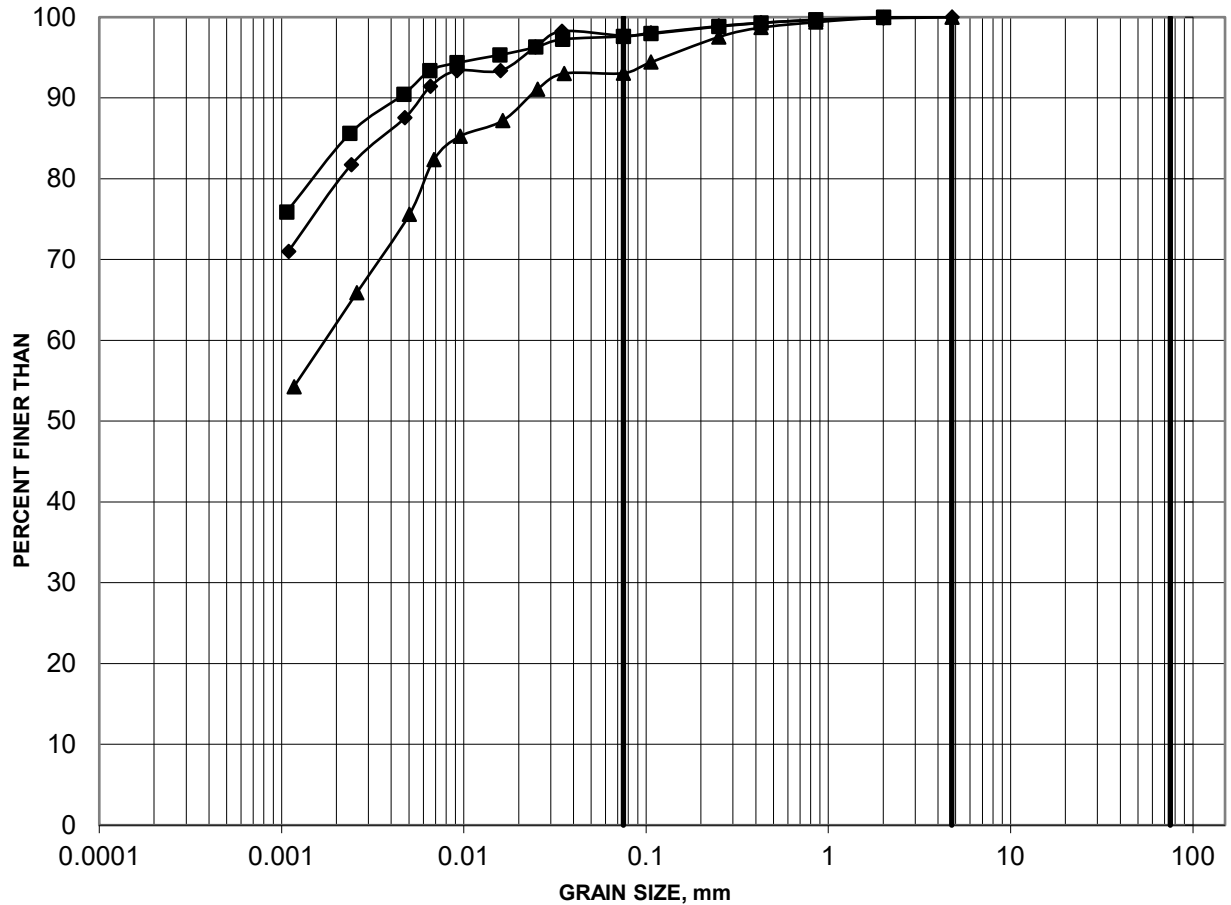
Created by: MI  
Checked by: KCP

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2022/22513877A/Figures/>

# GRAIN SIZE DISTRIBUTION

FIGURE B13

## CLAY (CH) (Below Crust)



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

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	22-02A	14	10.67-11.28	0	2	14	84
◆	22-04A	14	10.67-11.28	0	2	19	79
▲	22-04A	16	13.72-14.33	0	7	31	62

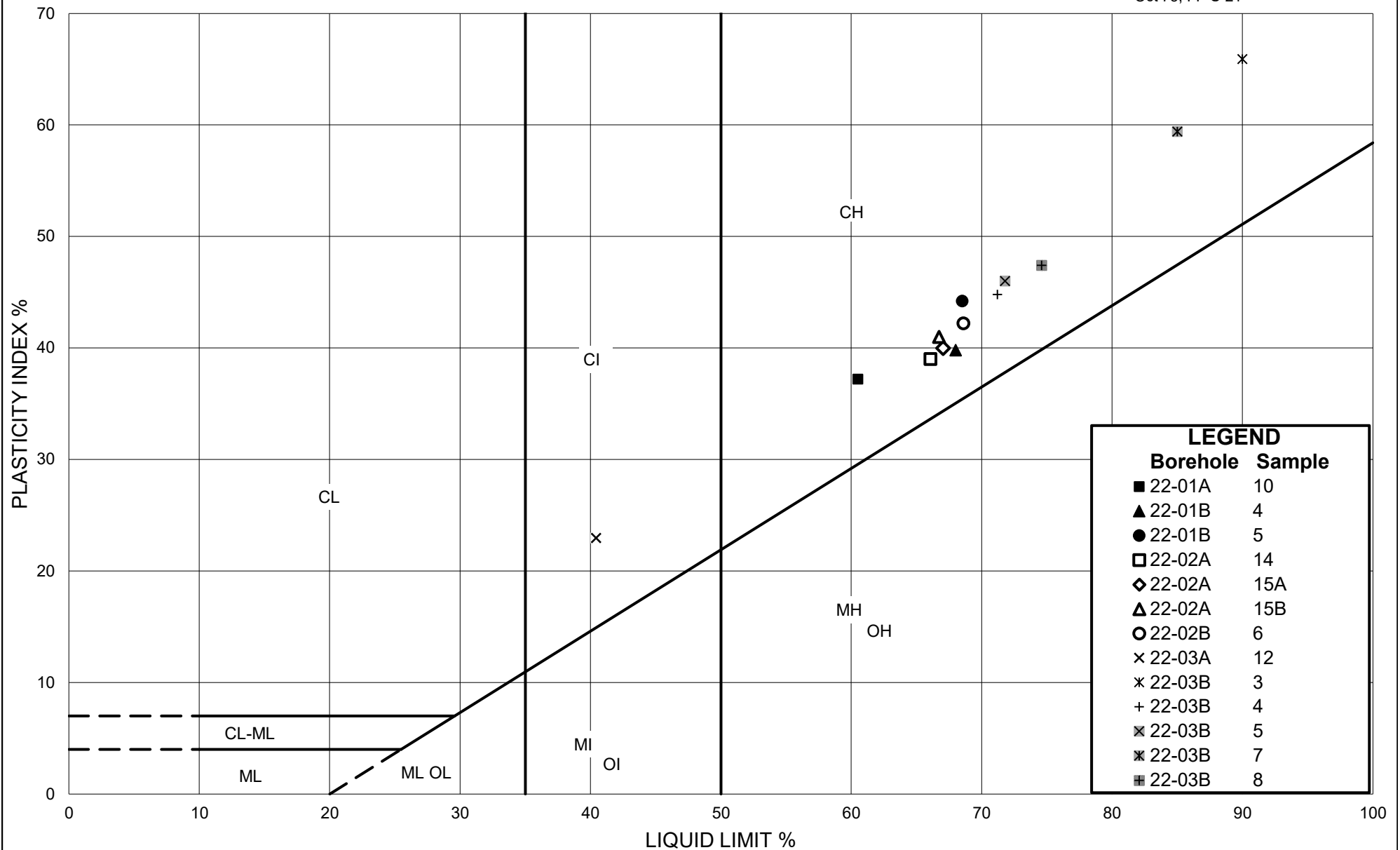
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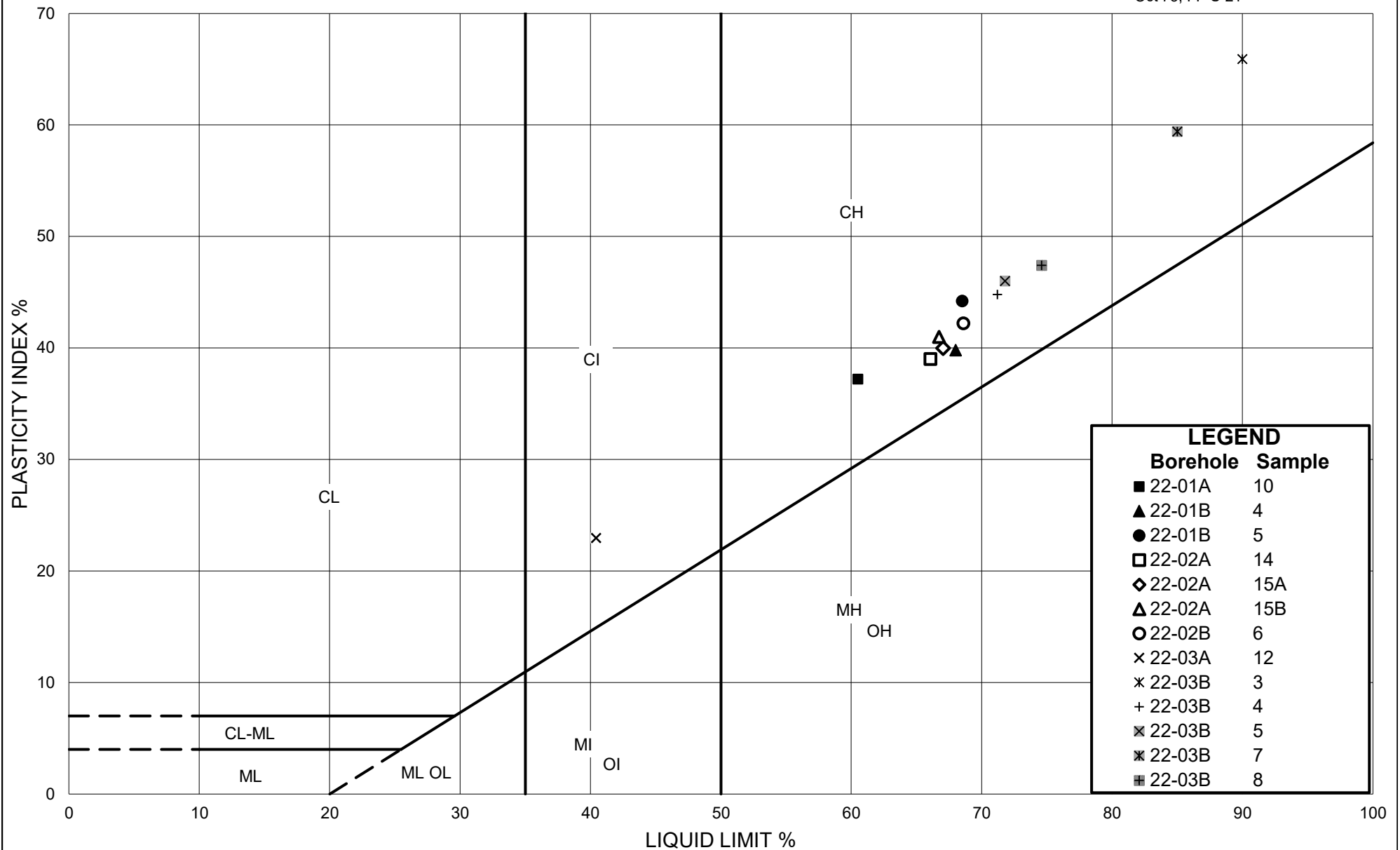
# PLASTICITY CHART

SILTY CLAY (CI) to CLAY (CH) (Unweathered)

Figure: B14

Project: 22513877A

Created By: CW Checked By: MI



Ontario

Ministry of Transportation

# PLASTICITY CHART

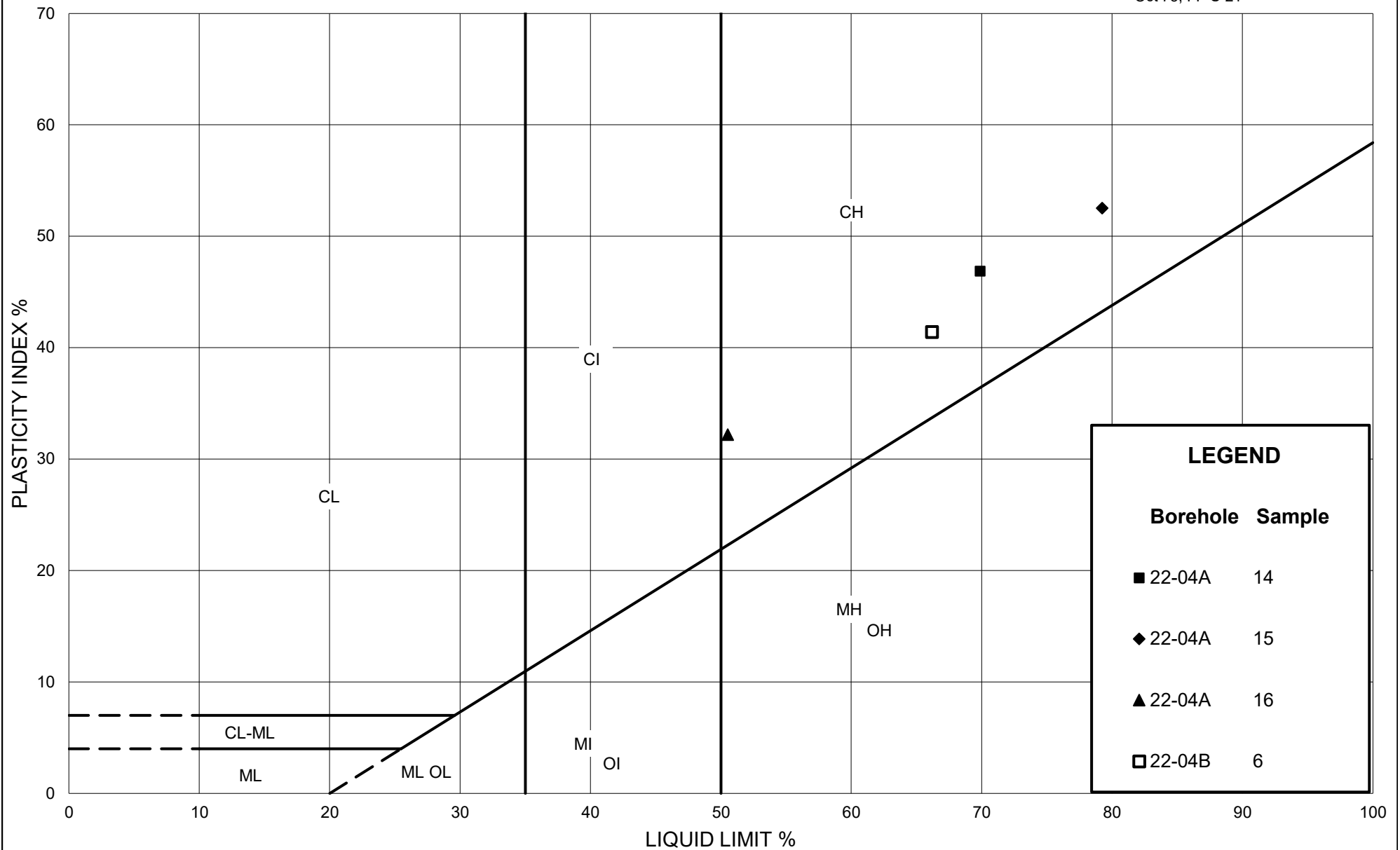
## SILTY CLAY (CI) to CLAY (CH) (Below Crust)

Figure: B14

Project: 22513877A

Created By: CW

Checked By: MI



Ontario

Ministry of Transportation

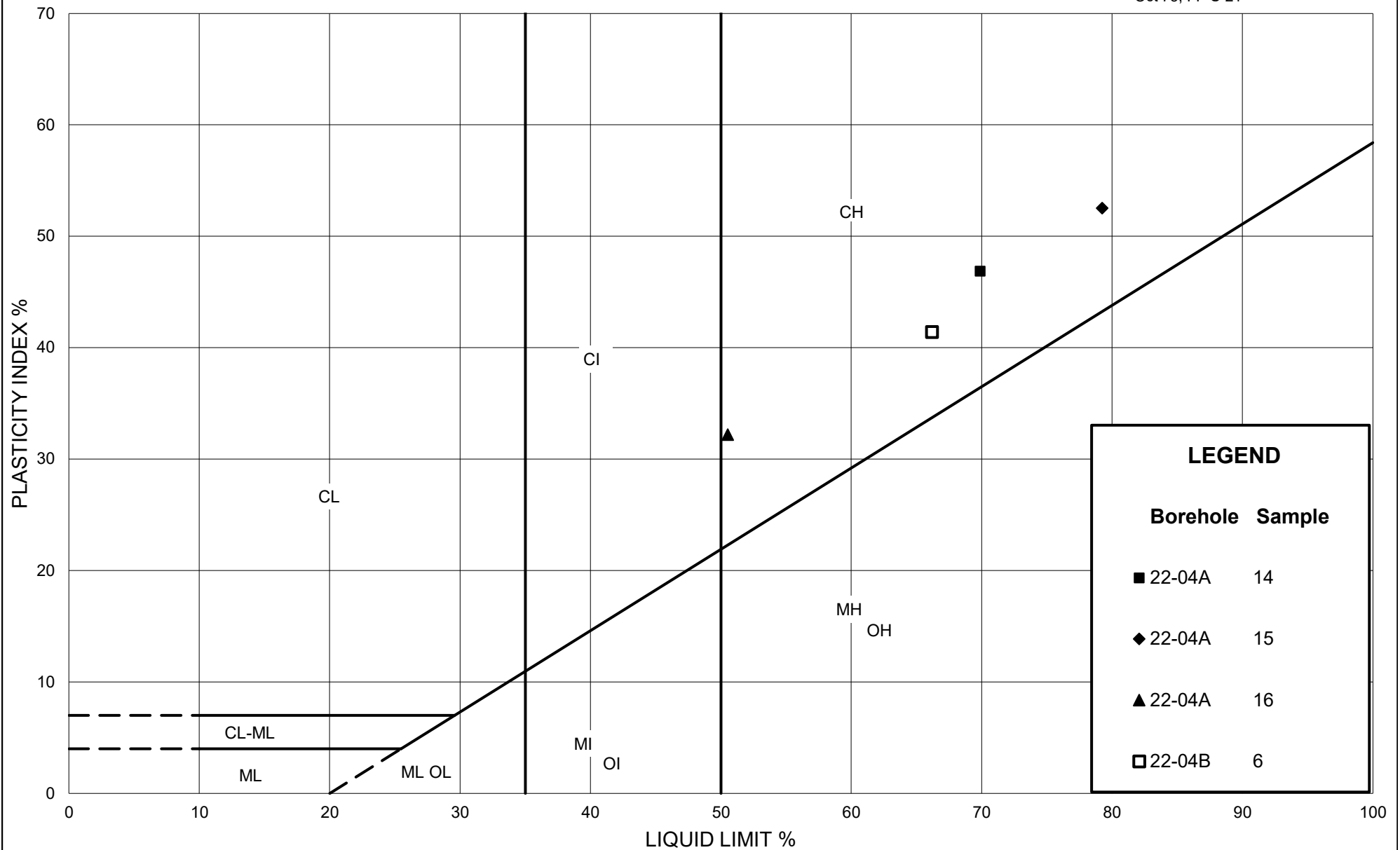
# PLASTICITY CHART

## CLAY (CH) (Unweathered)

Figure: B15

Project: 22513877A

Created By: CW Checked By: MI



Ministry of Transportation

# PLASTICITY CHART

## CLAY (CH) (Below Crust)

Figure: B15

Project: 22513877A

Created By: CW Checked By: MI

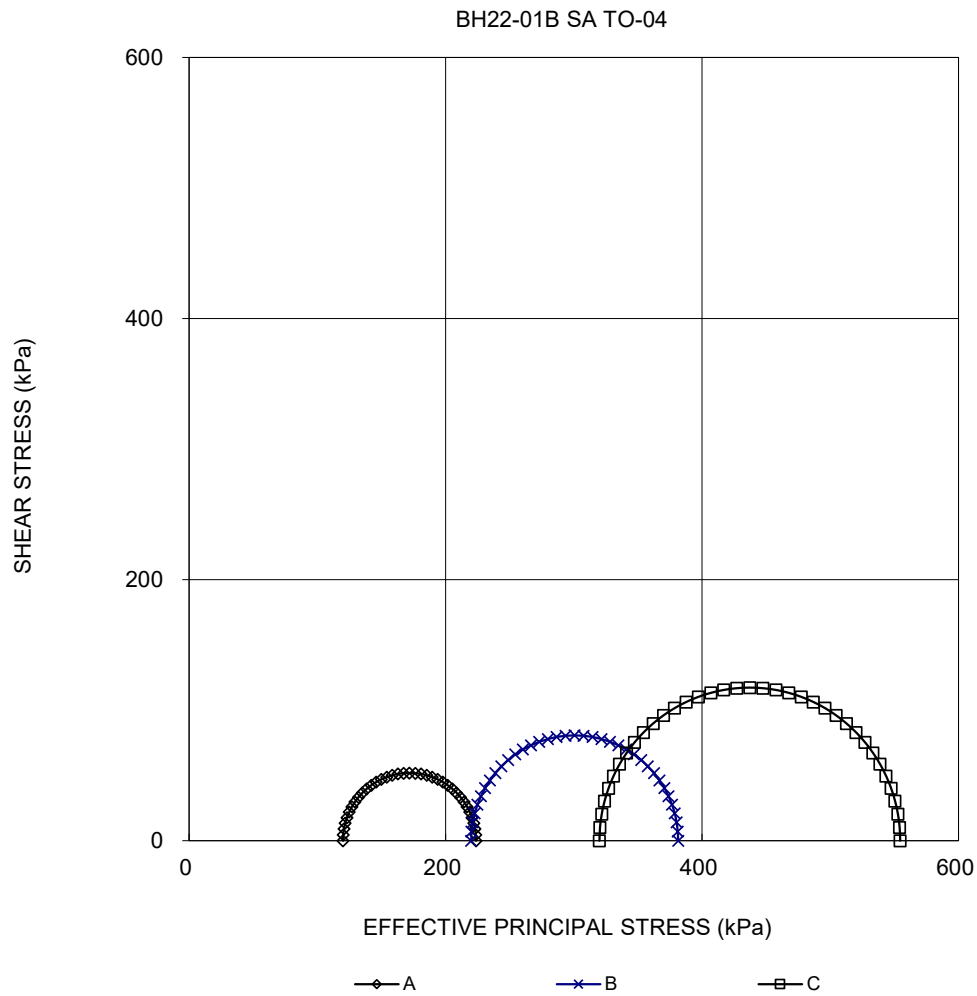
CONSOLIDATED DRAINED TRIAXIAL ASTM D7181 SHEET 1 OF 4		FIGURE B16	
TEST STAGE	A	B	C
BOREHOLE NUMBER	22-01B		
SAMPLE NUMBER	TO-04		
DEPTH, m	6.62-6.91		
SPECIMEN DIAMETER, cm	6.91	6.93	6.94
SPECIMEN HEIGHT, cm	14.02	14.02	13.96
NATURAL WATER CONTENT, %	75.8	78.1	82.5
DRY DENSITY, Mg/m <sup>3</sup>	0.89	0.87	0.85
WATER CONTENT BEFORE CONSOLIDATION, %	77.1	79.8	84.7
CELL PRESSURE, $\sigma_3$ , kPa	320.0	420.0	520.0
BACK PRESSURE, kPa	200.0	200.0	200.0
PORE PRESSURE PARAMETER "B"	0.96	0.97	0.96
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa	120.0	220.0	320.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	6.9	0.0	22.1
WATER CONTENT AFTER CONSOLIDATION, %	69.3	59.6	58.6
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, HOURS	29	30	31
WATER CONTENT AFTER TEST, %	60.2	51.2	49.3
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	103.8	162.2	235.6
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	14.7	15.2	15.3
MAX PRINCIPAL STRESS RATIO, $(\sigma'_1 / \sigma'_3)$ maximum	2.1	1.9	1.8
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
<p>Specimen A taken 3-17cm from top of tube.</p> <p>Specimen B taken 17-31.5cm from top of tube.</p> <p>Specimen C taken 31.5-46.5cm from top of tube.</p>			
FAILURE PLANE NUMBER	-	-	-
ANGLE OF FAILURE, DEGREES	Bulged	Bulged	Bulged
<div> <div>Date: 3/8/2023</div> <div>Project No. 22513877A</div> </div> <div> <div>WSP Canada Inc</div> </div> <div> <div>Prepared By: LH</div> <div>Checked By: MM</div> </div>			

CONSOLIDATED DRAINED TRIAXIAL

ASTM D7181

SHEET 2 OF 4

FIGURE B16



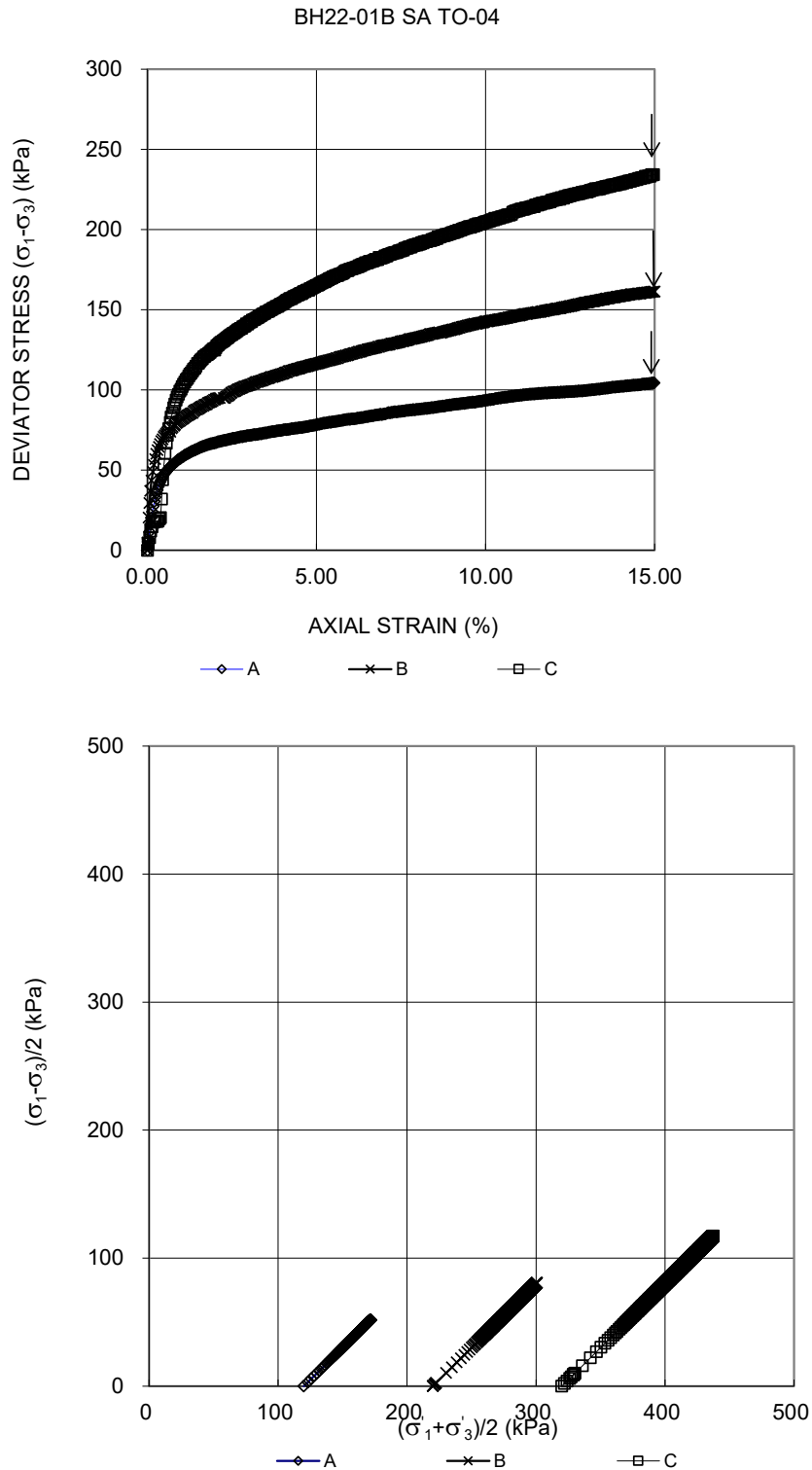
Date: 3/8/2023  
Project No. 22513877A

**WSP Canada Inc**

Prepared By: LH  
Checked By: MM

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

**FIGURE B16**



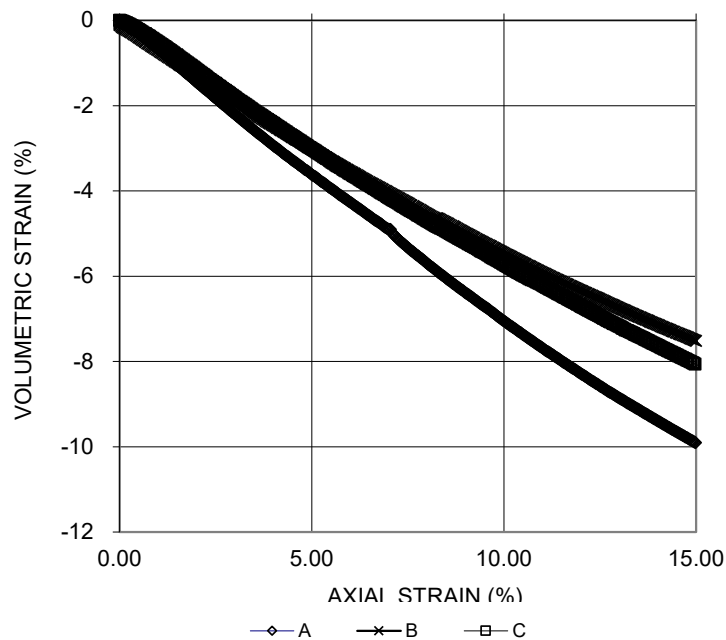
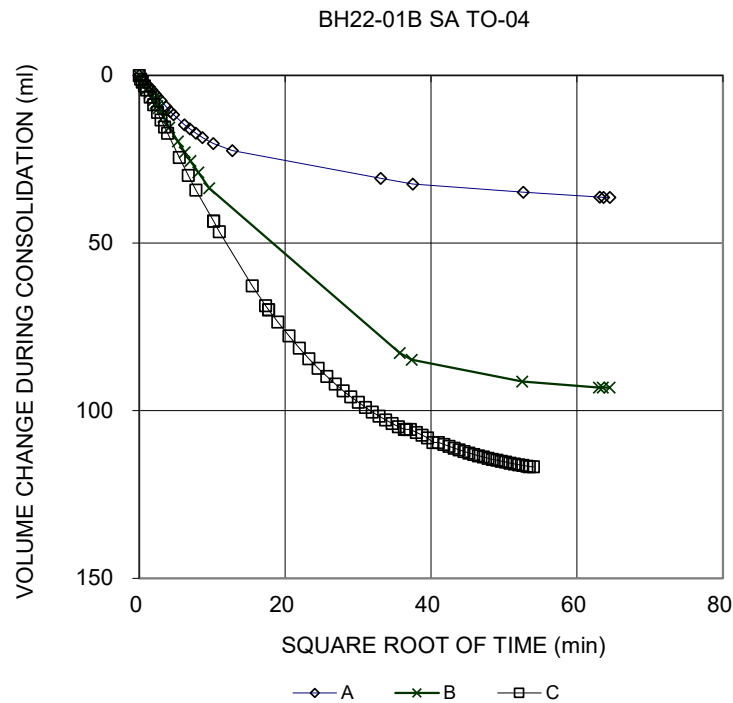
Date: 3/8/2023  
Project No. 22513877A

**WSP Canada Inc**

Prepared By: LH  
Checked By: MM

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

**FIGURE B16**



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

Date: 3/8/2023  
 Project No. 22513877A

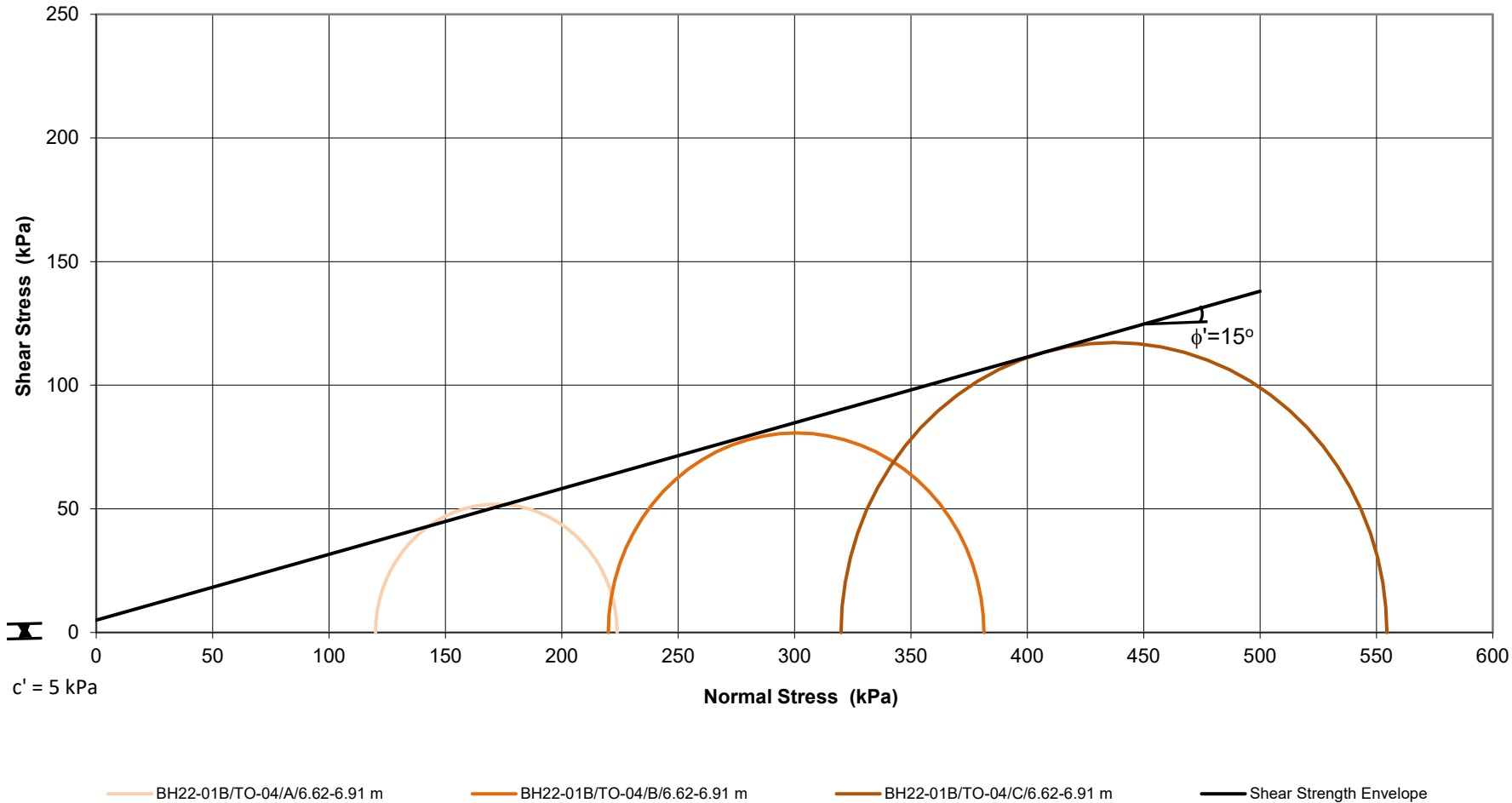
**WSP Canada Inc**

Prepared By: LH  
 Checked By: MM



FIGURE B16

Triaxial Test Results - 22-01B/ TO-04  
Peak Shear Strength Envelope



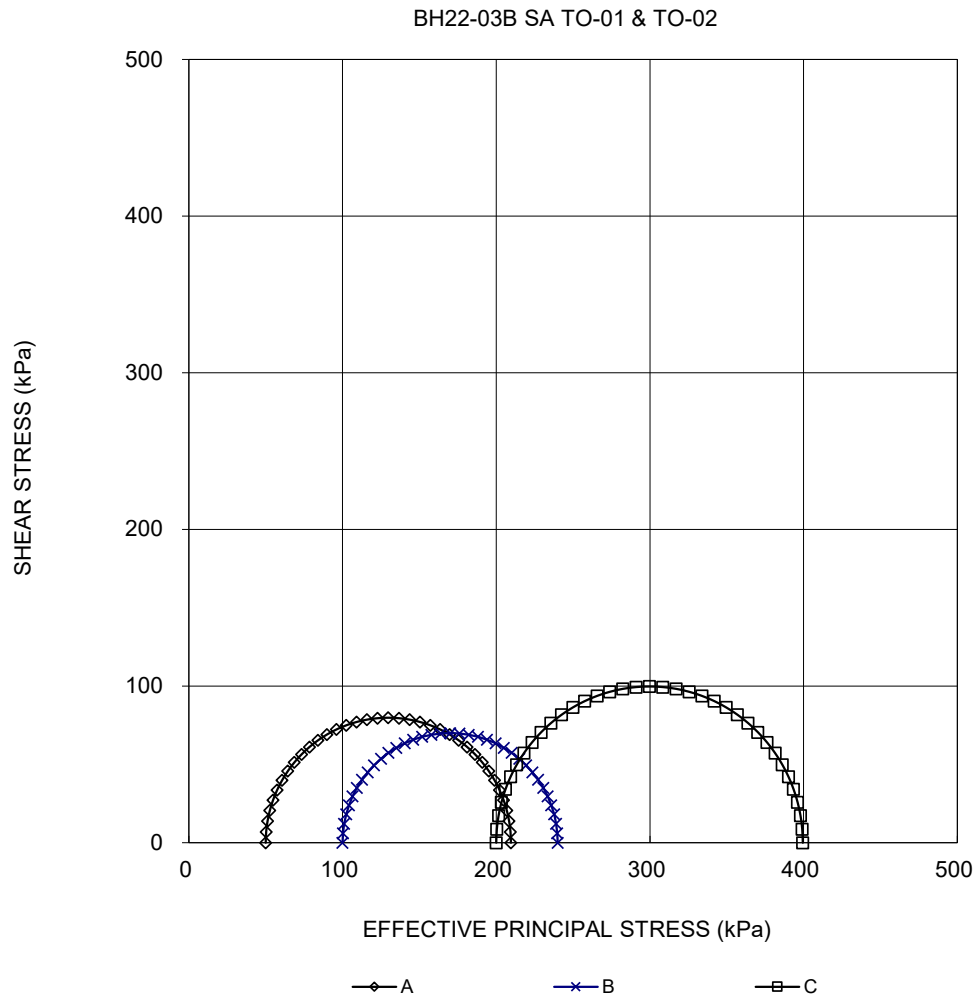
<b>CONSOLIDATED DRAINED TRIAXIAL</b> <b>ASTM D7181</b> <b>SHEET 1 OF 4</b>		<b>FIGURE B17</b>	
TEST STAGE	A	B	C
BOREHOLE NUMBER	22-03B		
SAMPLE NUMBER	TO-01	TO-02	TO-02
DEPTH, m	4.85-5.49	5.49-6.10	5.49-6.10
SPECIMEN DIAMETER, cm	6.92	6.96	6.94
SPECIMEN HEIGHT, cm	13.98	13.99	13.99
NATURAL WATER CONTENT, %	58.4	75.2	70.0
DRY DENSITY, Mg/m <sup>3</sup>	1.03	0.89	0.93
WATER CONTENT BEFORE CONSOLIDATION, %	61.1	76.3	71.8
CELL PRESSURE, $\sigma_3$ , kPa	460.0	370.0	540.0
BACK PRESSURE, kPa	410.0	270.0	340.0
PORE PRESSURE PARAMETER "B"	0.96	0.96	0.96
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa	50.0	100.0	200.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	1.3	3.3	8.1
WATER CONTENT AFTER CONSOLIDATION, %	59.9	72.6	59.5
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, HOURS	7	8	30
WATER CONTENT AFTER TEST, %	57.9	65.4	53.1
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	159.6	140.0	199.5
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	3.3	3.8	15.0
MAX PRINCIPAL STRESS RATIO, $(\sigma'_1 / \sigma'_3)$ maximum	4.2	2.7	2.1
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:  <div> Specimen A taken 24-40cm from top of tube.  Specimen B taken 2-17cm from top of tube.  Specimen C taken 17-33cm from top of tube. </div>			
FAILURE PLANE NUMBER	1.0	1.0	-
ANGLE OF FAILURE, DEGREES	55.0	60.0	Bulged
<div> Date: 3/13/2023  Project No. 22513877A </div> <div> <b>WSP Canada Inc</b> </div> <div> Prepared By: LH  Checked By: MM </div>			

CONSOLIDATED DRAINED TRIAXIAL

ASTM D7181

SHEET 2 OF 4

FIGURE B17



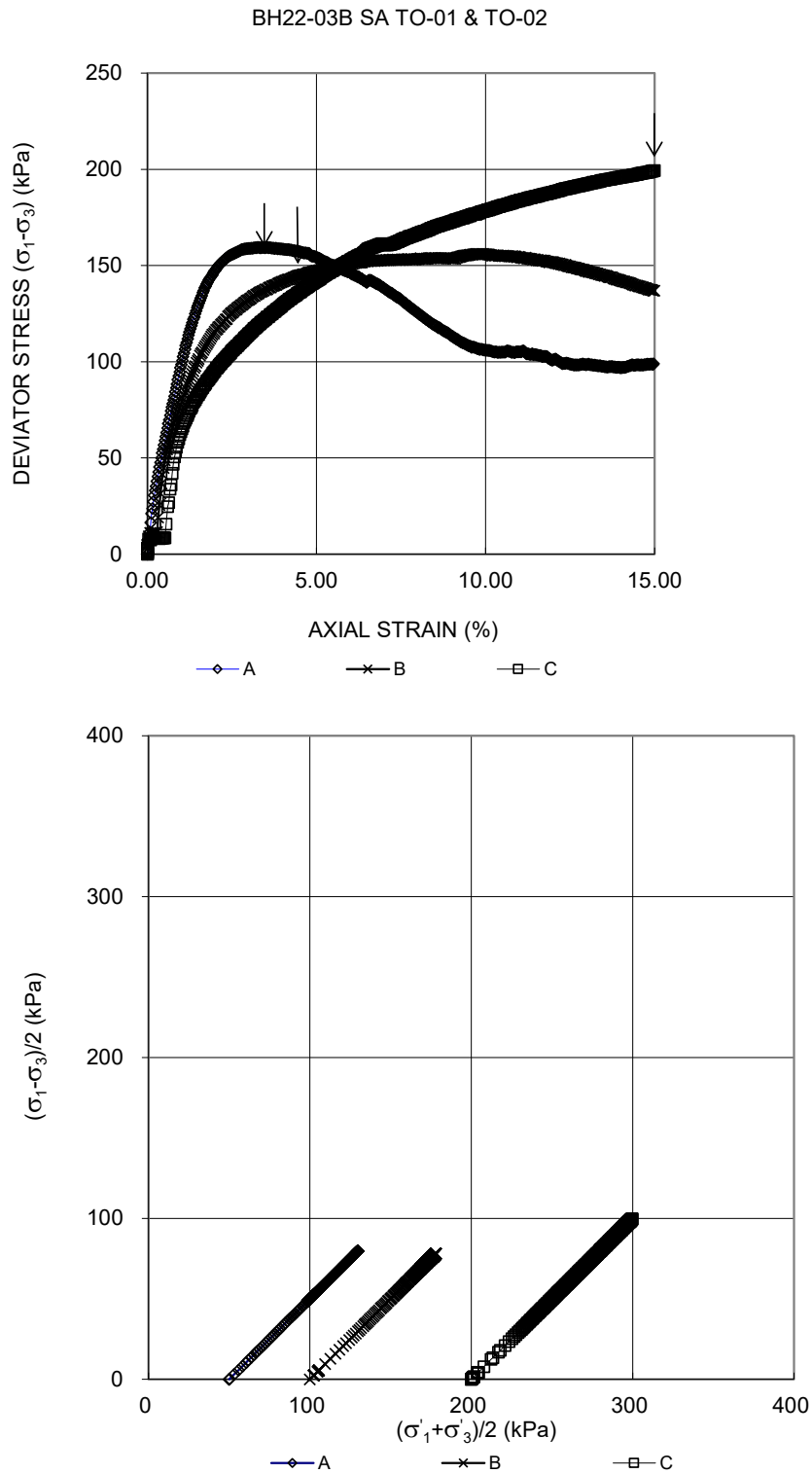
Date: 3/13/2023  
Project No. 22513877A

**WSP Canada Inc**

Prepared By: LH  
Checked By: MM

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

**FIGURE B17**



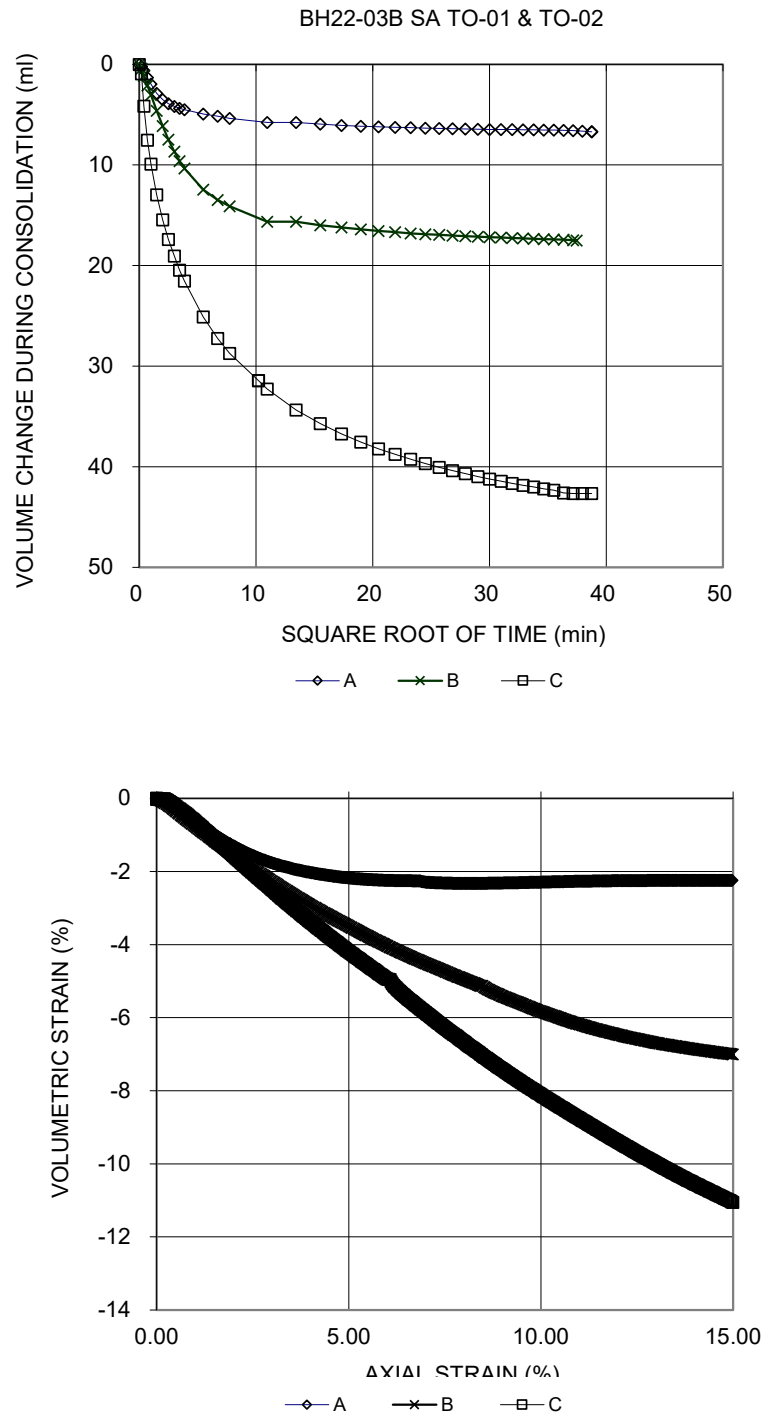
Date: 3/13/2023  
 Project No. 22513877A

**WSP Canada Inc**

Prepared By: LH  
 Checked By: MM

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

**FIGURE B17**



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

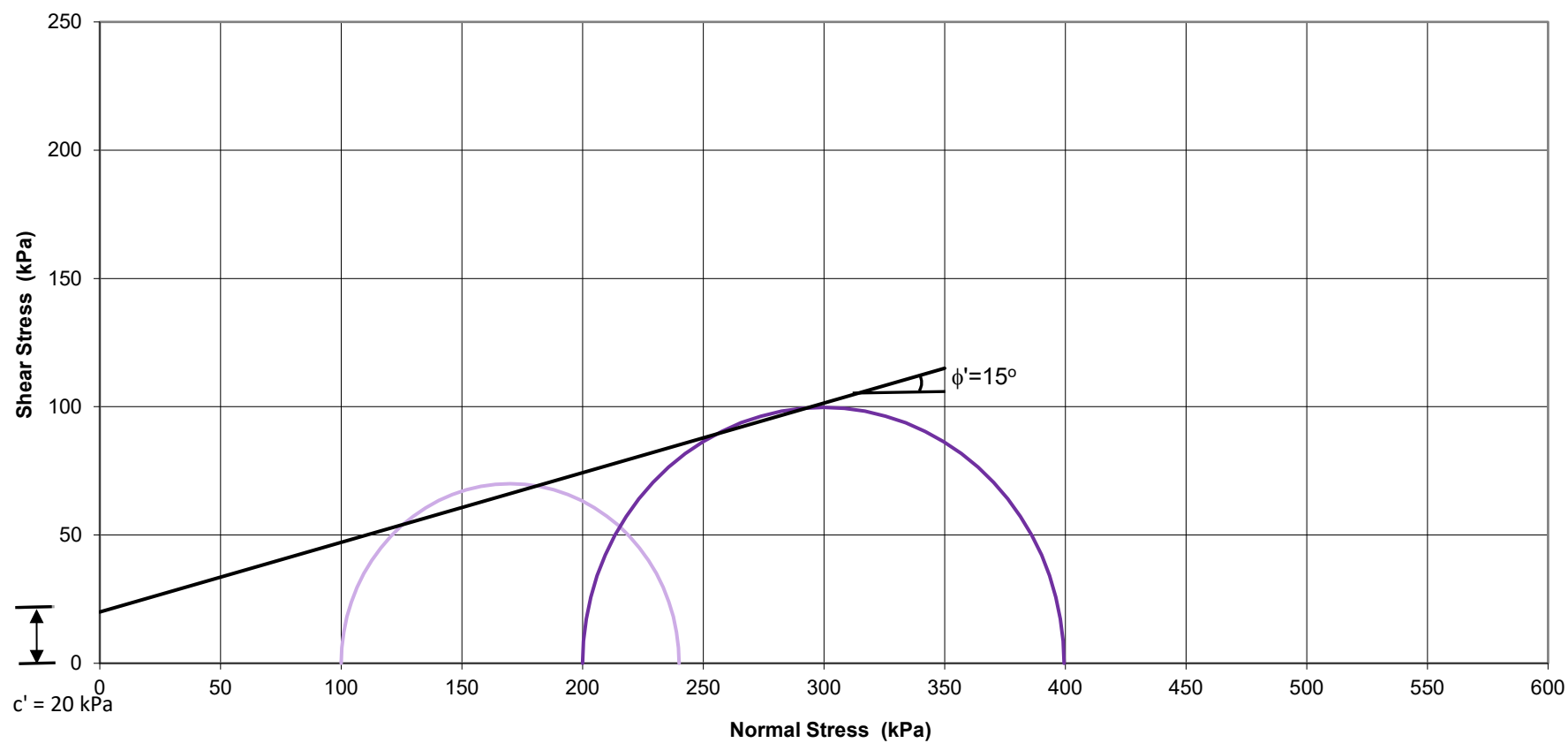
Date: 3/13/2023  
 Project No. 22513877A

**WSP Canada Inc**

Prepared By: LH  
 Checked By: MM

FIGURE B17

# Triaxial Test Results - 22-03B/ TO-02 Peak Shear Strength Envelope



BH22-03B/ TO-02/B/ 5.49-6.10

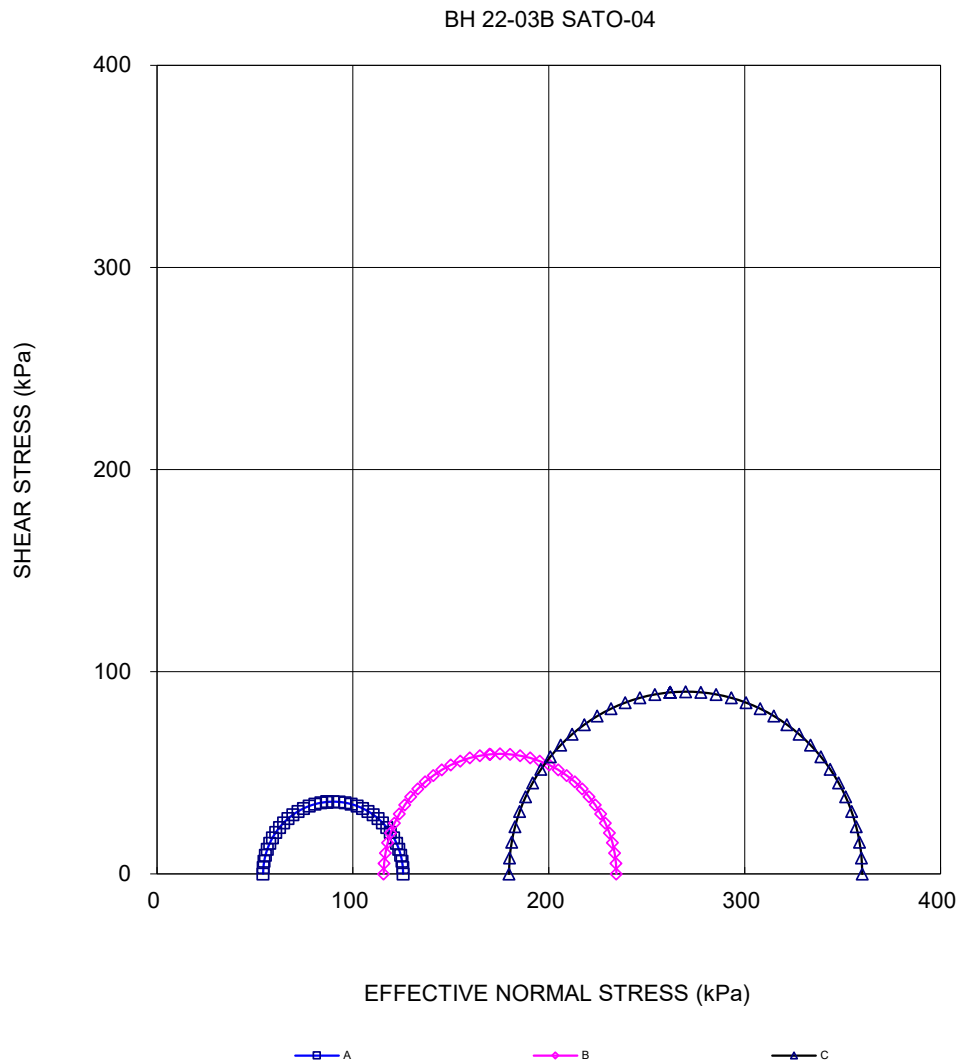
BH22-03B/ TO-02/C/ 5.49-6.10

Shear Strength Envelope

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS ASTM D4767 SHEET 1 OF 4			FIGURE B18
TEST STAGE	A	B	C
BOREHOLE NUMBER	22-03B		
SAMPLE	TO-04		
DEPTH, m	6.71-7.22		
SPECIMEN DIAMETER, cm	6.94	6.95	6.93
SPECIMEN HEIGHT, cm	13.98	14.00	13.98
NATURAL WATER CONTENT, %	86.1	82.1	59.1
DRY DENSITY, Mg/m <sup>3</sup>	0.82	0.85	1.05
WATER CONTENT AFTER SATURATION, %	86.9	84.0	61.0
CELL PRESSURE, $\sigma_3$ , kPa	325.0	450.0	575.0
BACK PRESSURE, kPa	200.0	200.0	200.0
PORE PRESSURE PARAMETER "B"	0.96	0.96	0.96
EFFECTIVE CONSOLIDATION STRESS, $\sigma_c$ , kPa	125.0	250.0	375.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	11.0	21.6	19.8
WATER CONTENT AFTER CONSOLIDATION, %	73.4	58.4	42.2
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, HOURS	5.2	8.1	7.1
WATER CONTENT AFTER TEST, %	72.9	58.3	41.6
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	71.5	118.8	180.3
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ maximum, %	2.6	4.1	3.6
MAX EFFECTIVE PRINCIPAL STRESS RATIO, $(\sigma'_1 / \sigma'_3)$ maximum	2.6	2.2	2.2
DEVIATOR STRESS AT $(\sigma'_1 / \sigma'_3)$ maximum, kPa	64.8	108.8	160.6
AXIAL STRAIN AT $(\sigma'_1 / \sigma'_3)$ maximum, %	6.2	9.3	11.8
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ maximum	0.99	1.13	1.08
PORE PRESSURE PARAMETER, Af, AT $(\sigma'_1 / \sigma'_3)$ maximum	1.32	1.48	1.47
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:  Effective consolidation stresses are assigned by the client. Specimen A taken 12-26cm from top of tube. Specimen B taken 26-40cm from top of tube. Specimen C taken 26-40cm from top of tube.			
FAILURE PLANE NUMBER	1.0	1.0	-
ANGLE OF FAILURE PLANE, DEGREES	60.0	70.0	Bulged
Date: 3/3/2023 Project No. 22513877A <div>             WSP Canada Inc             <div>               Prepared By: LH                Checked By: MM             </div> </div>			

CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
ASTM D4767  
SHEET 2 OF 4

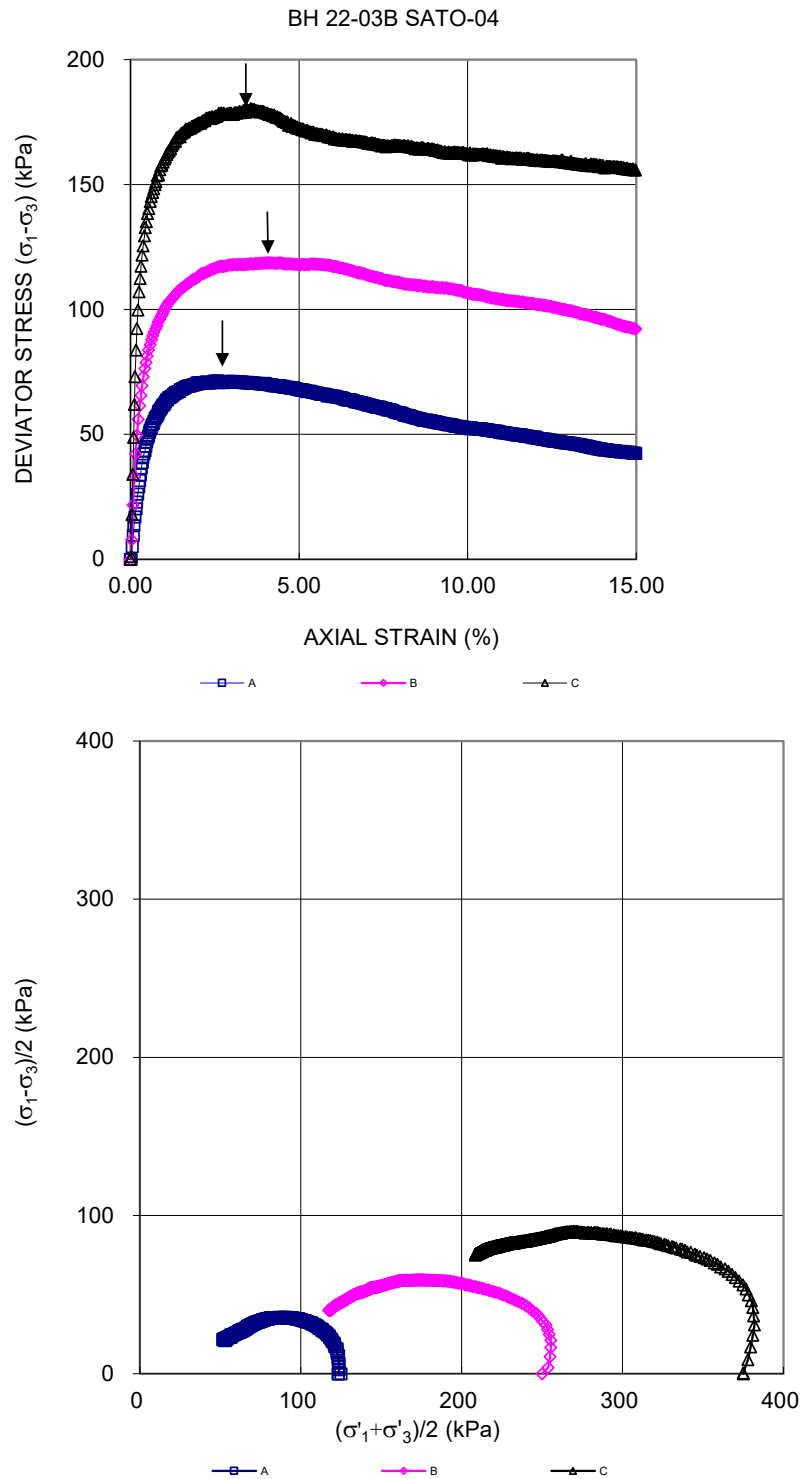
FIGURE B18





**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
ASTM D4767  
SHEET 3 OF 4**

**FIGURE B18**



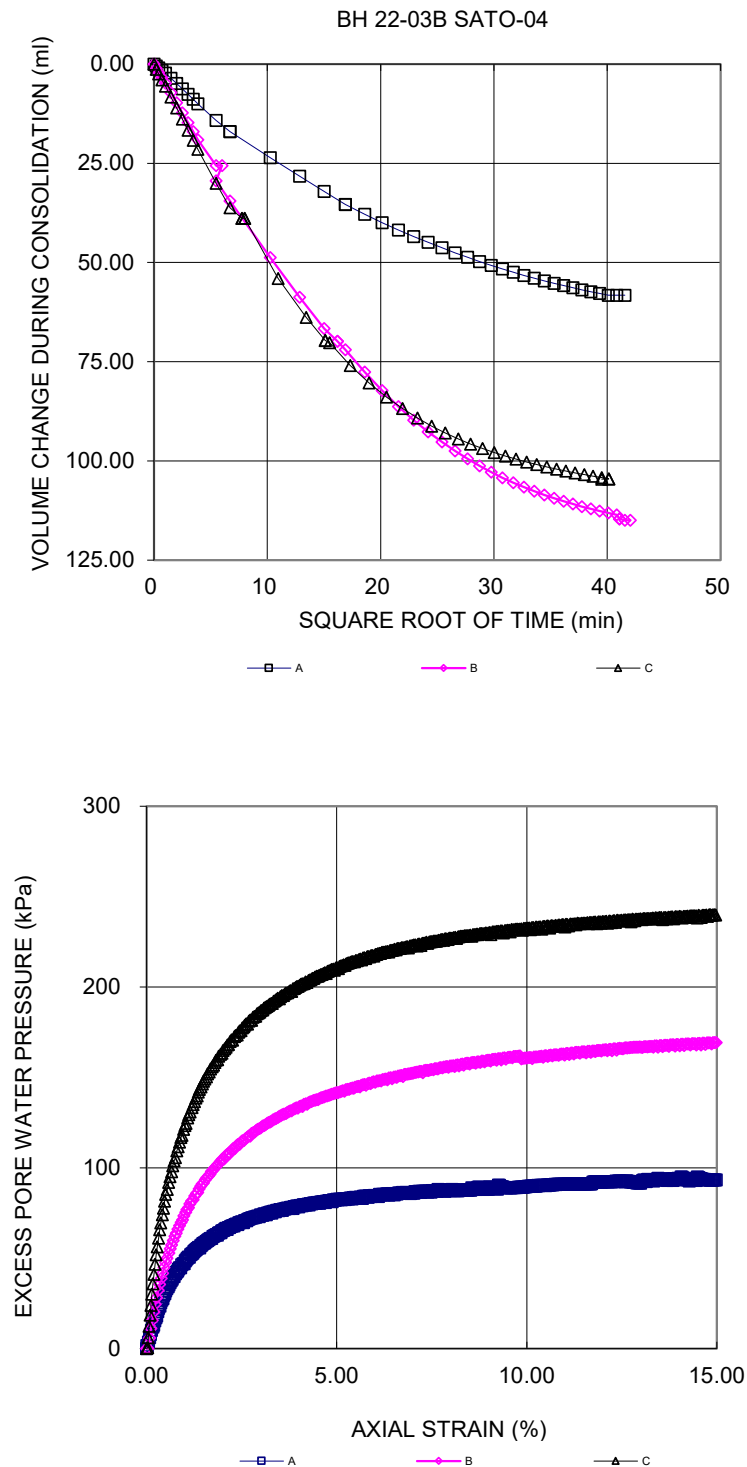
Date: 3/3/2023  
Project No. 22513877A

**WSP Canada Inc**

Prepared By: LH  
Checked By: MM

**CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
ASTM D4767  
SHEET 4 OF 4**

**FIGURE B18**



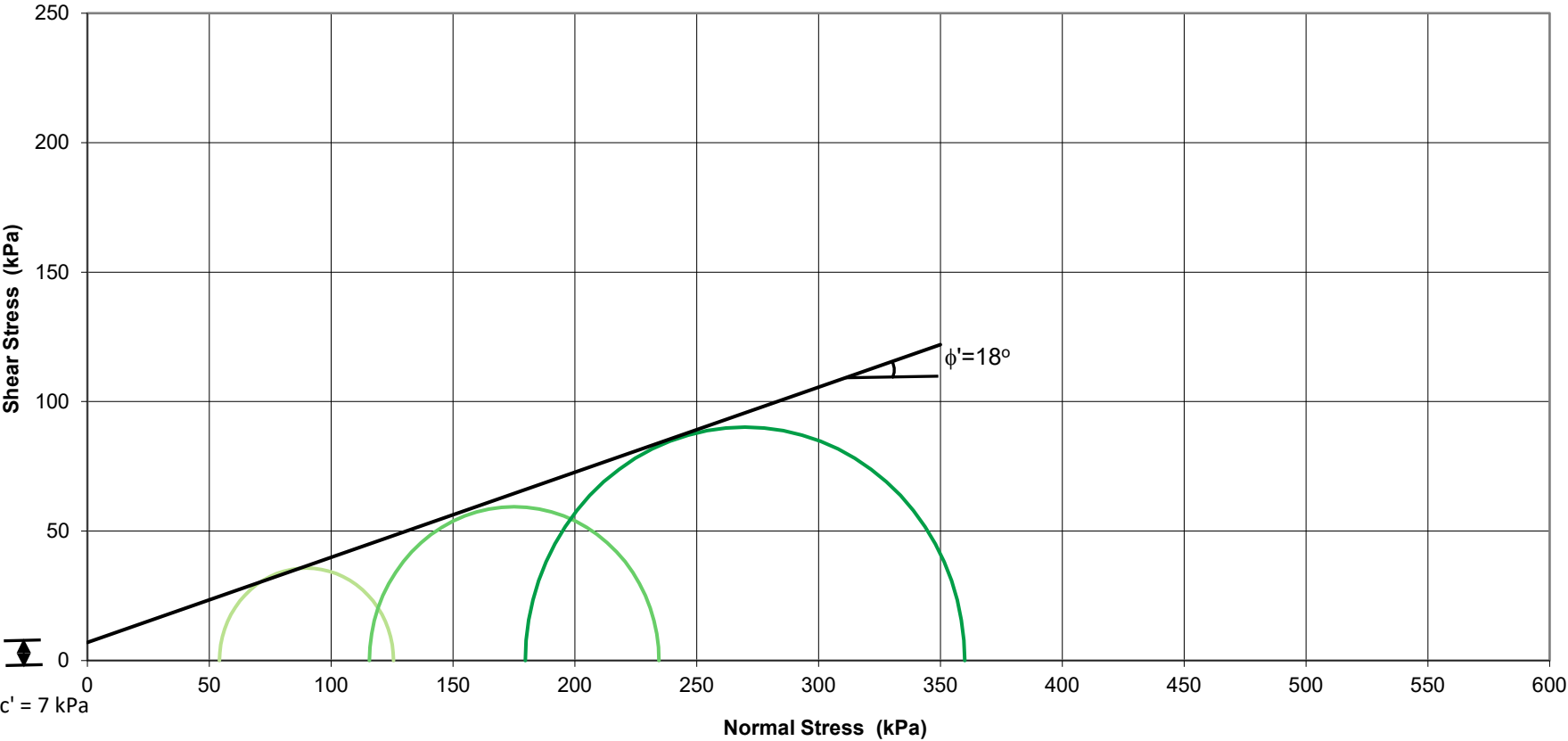
Date: 3/3/2023  
Project No. 22513877A

**WSP Canada Inc**

Prepared By: LH  
Checked By: MM

FIGURE B18

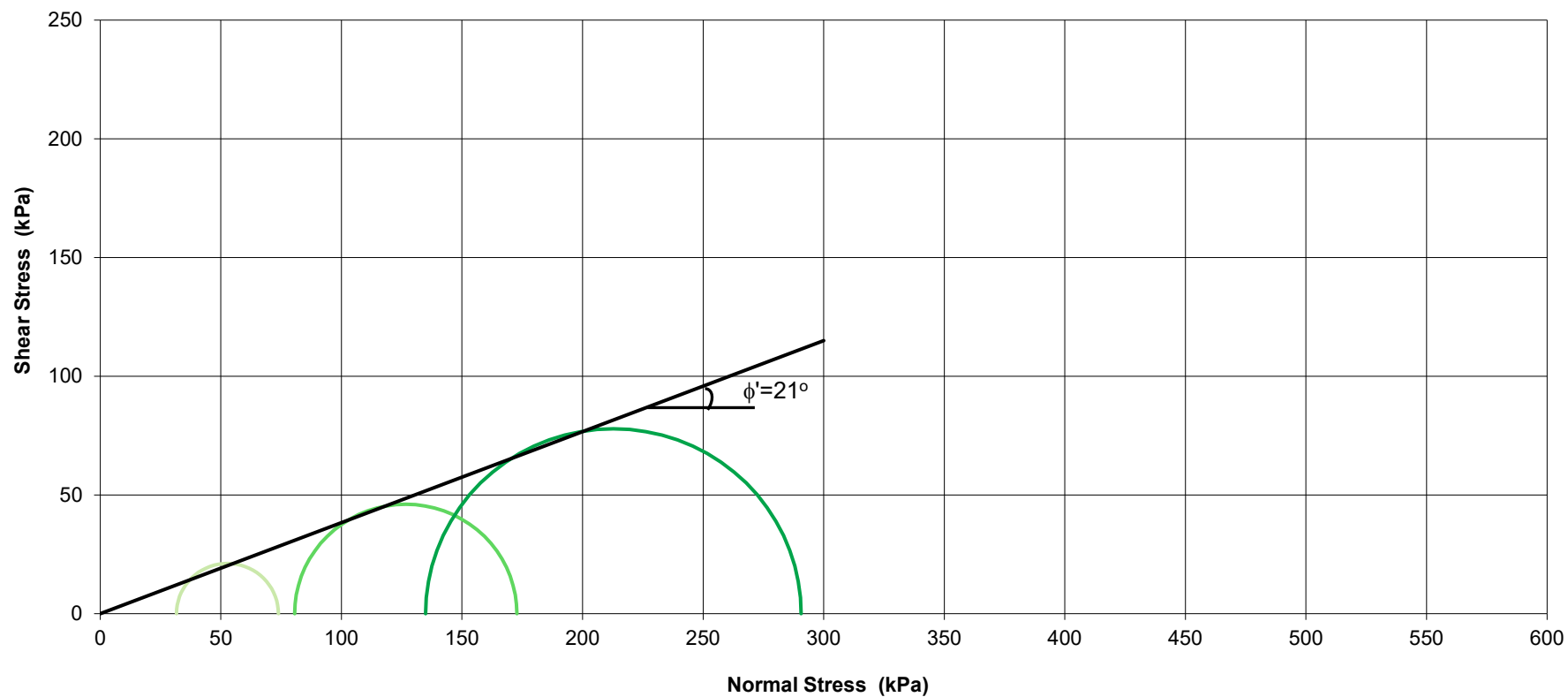
Triaxial Test Results - 22-03B/ TO-04  
Peak Shear Strength Envelope



BH22-03B/TO-04/A/6.71-7.22 m    BH22-03B/TO-04/B/6.71-7.22 m    BH22-03B/TO-04/C/6.71-7.22 m    Shear Strength Envelope

FIGURE B18

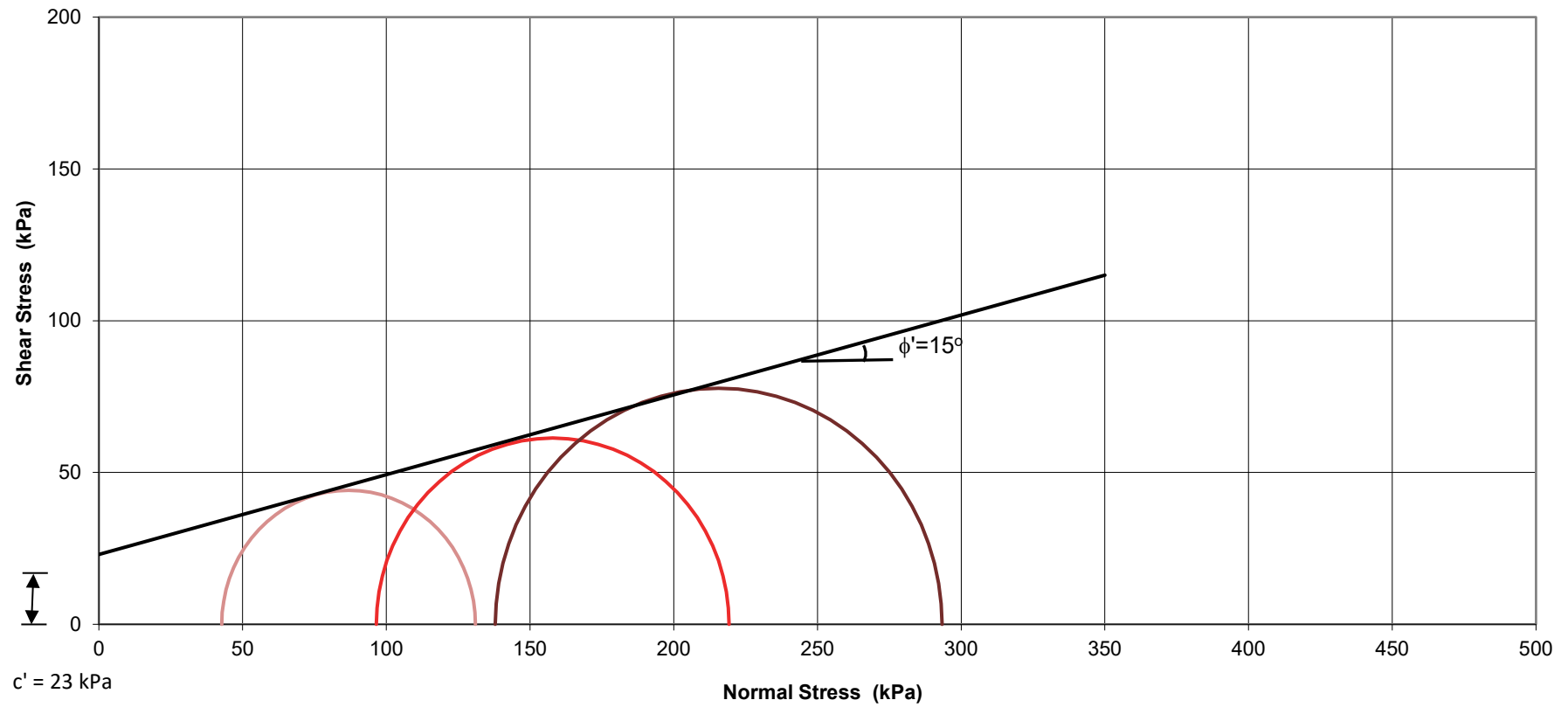
**Traixial Test Results - 22-03B/ TO-04**  
**Post Peak/ Soften Shear Strength Envelope**



— BH22-03B/TO-04/A/6.71-7.22 m   
 — BH22-03B/TO-04/B/6.71-7.22 m   
 — BH22-03B/TO-04/C/6.71-7.22 m   
 — Shear Strength Envelope

FIGURE B19

Triaxial Test Results- Report No. 65135, January 1966  
Peak Shear Strength Envelope



— Report No. 65135, January 1966 / A

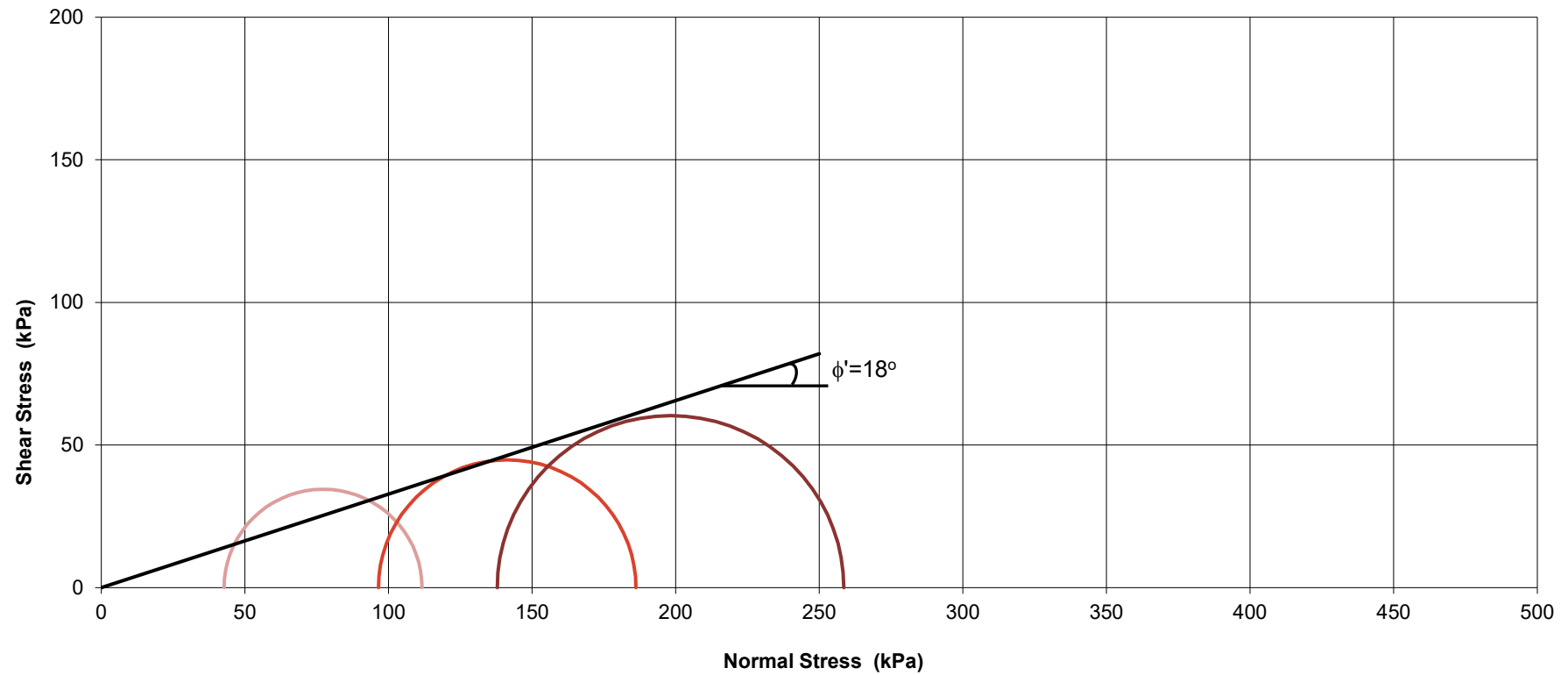
— Report No. 65135, January 1966 / B

— Report No. 65135, January 1966 / C

— Shear Strength Envelope

FIGURE B19

Triaxial Test Results- Report No. 65135, January 1966  
Post Peak/ Soften Shear Strength Envelope



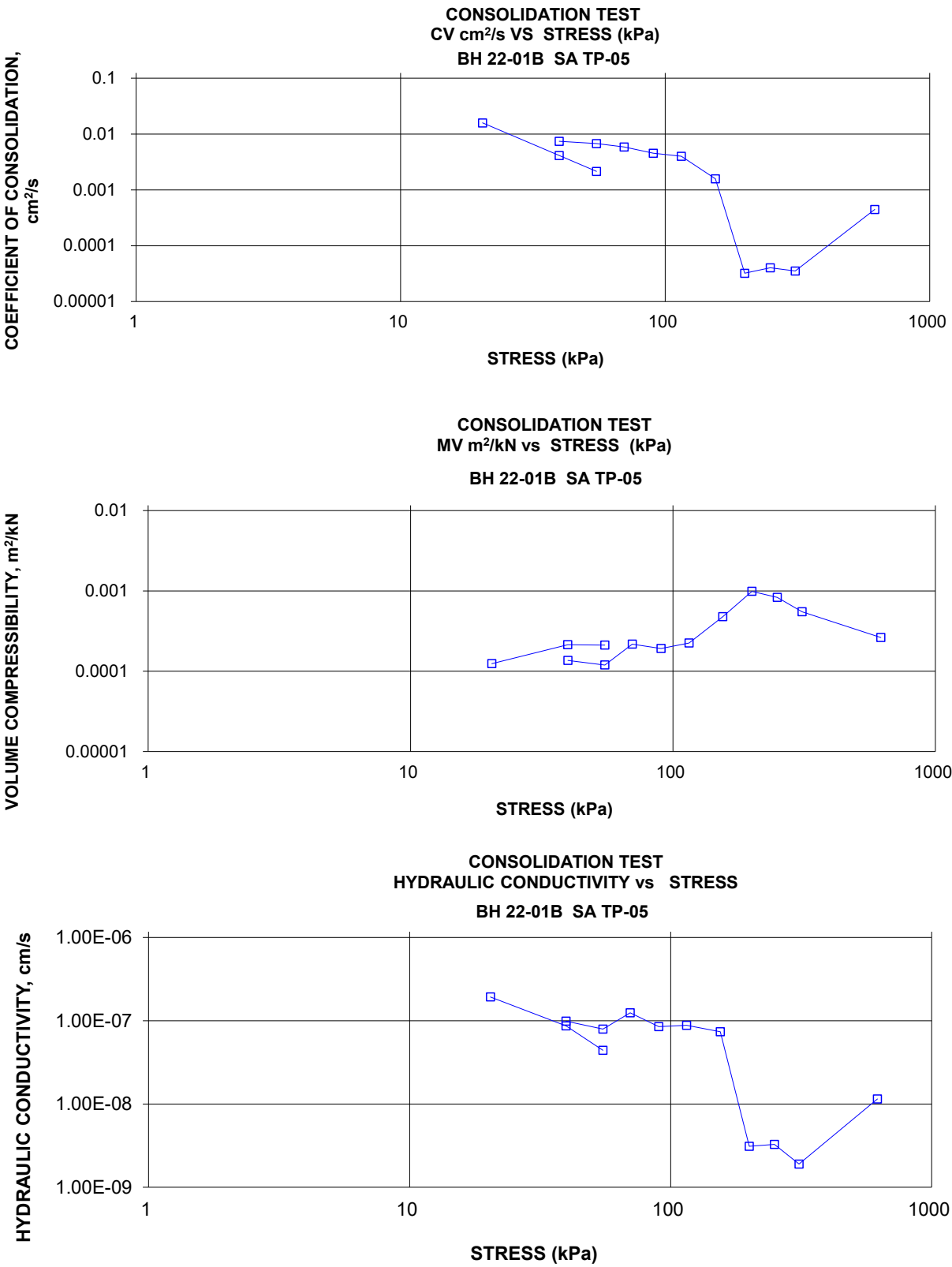
Report No. 65135, January 1966 / A

Report No. 65135, January 1966 / B

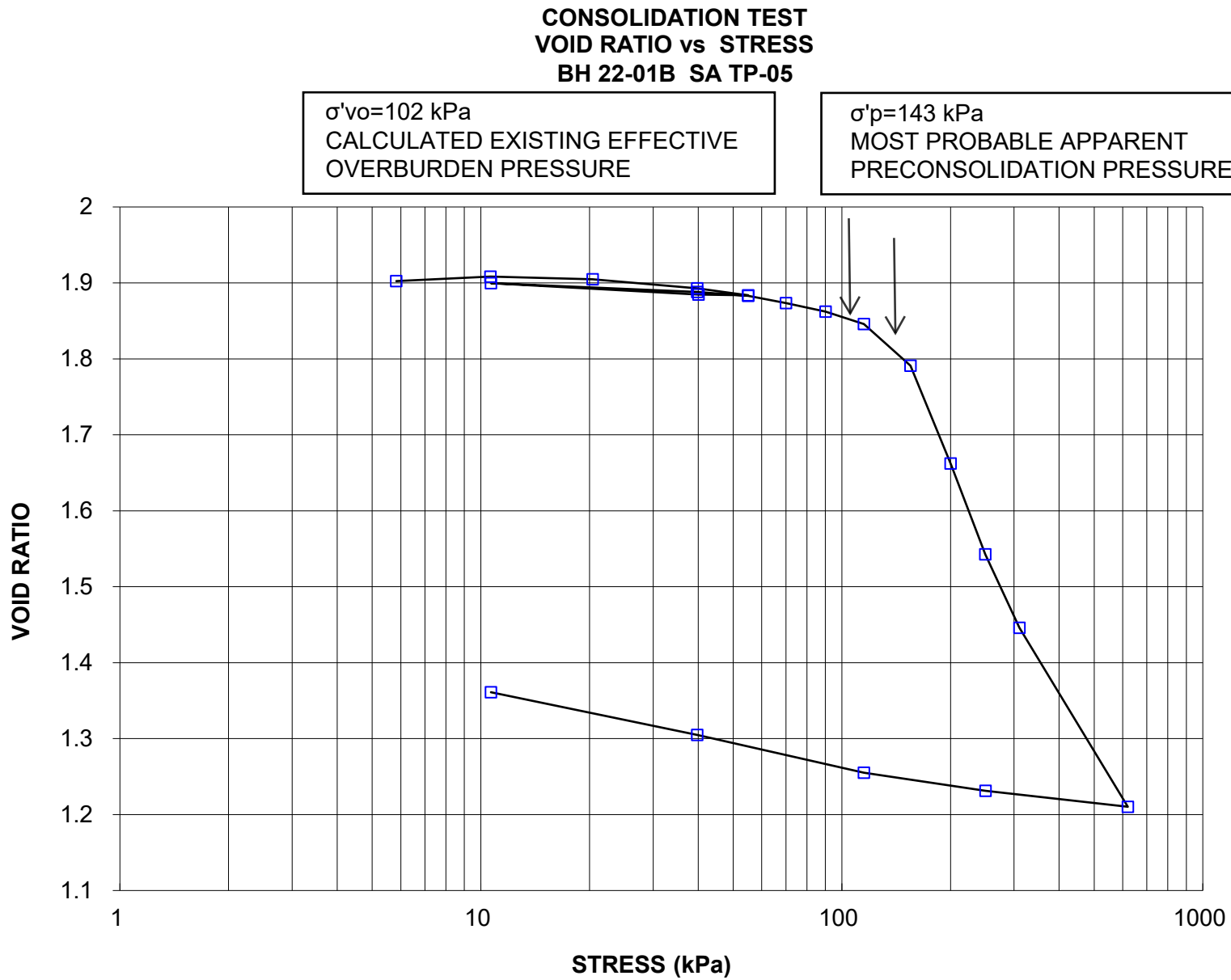
Report No. 65135, January 1966 / C

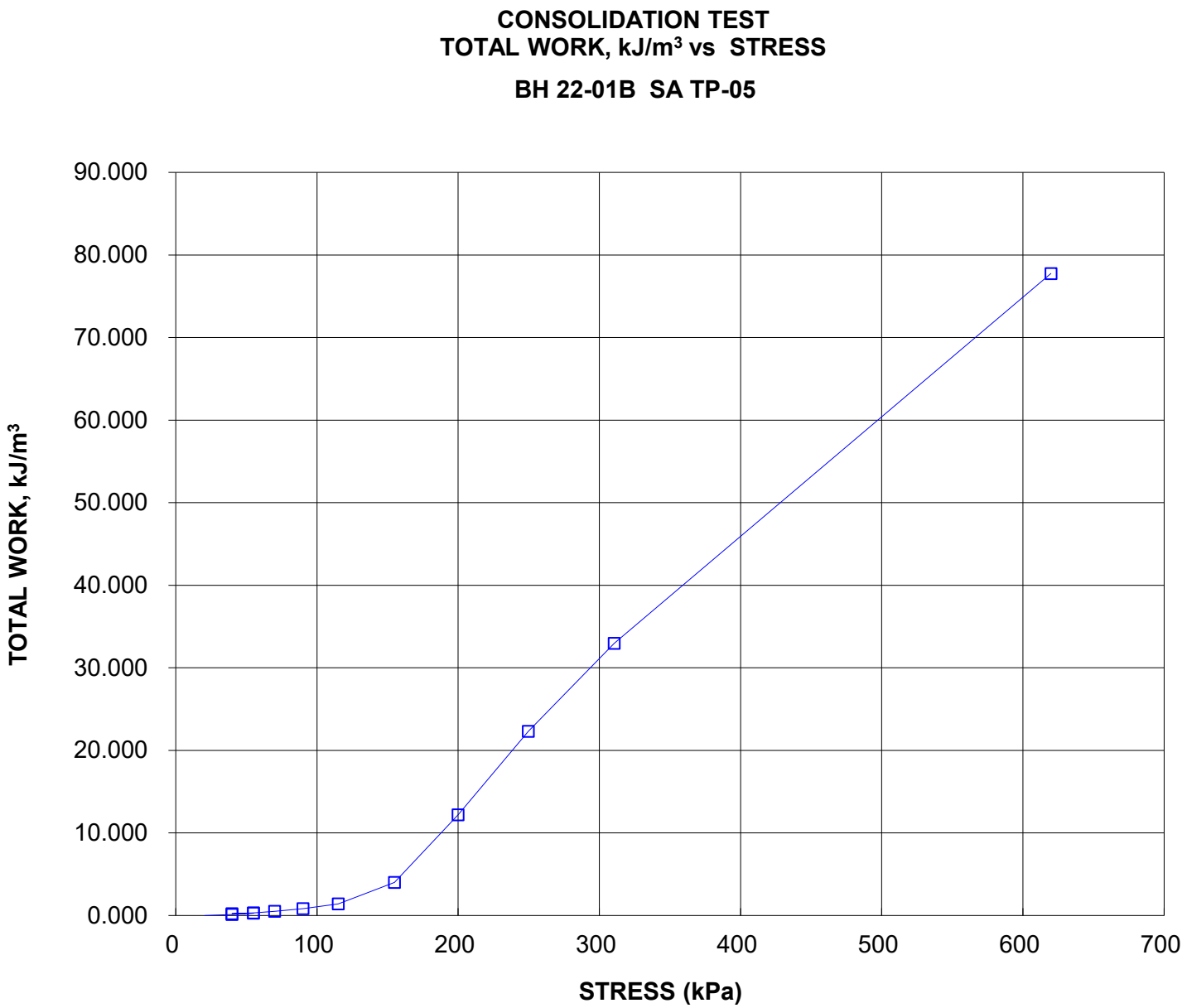
Shear Strength Envelope

CONSOLIDATION TEST SUMMARY					FIGURE B20		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number		22513877A		Sample Number		TP-05	
Borehole Number		BH22-01B		Sample Depth, m		7.01-7.52	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		6					
Date Started		02/22/2023					
Date Completed		03/10/2023					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		1.91		Unit Weight, kN/m <sup>3</sup>		15.70	
Sample Diameter, cm		6.35		Dry Unit Weight, kN/m <sup>3</sup>		9.49	
Area, cm <sup>2</sup>		31.71		Specific Gravity, measured		2.80	
Volume, cm <sup>3</sup>		60.47		Solids Height, cm		0.659	
Water Content, %		65.50		Volume of Solids, cm <sup>3</sup>		20.89	
Wet Mass, g		96.80		Volume of Voids, cm <sup>3</sup>		39.58	
Dry Mass, g		58.49		Degree of Saturation, %		96.8	
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	1.907	1.895	1.907				
5.83	1.912	1.902	1.910				
10.63	1.916	1.908	1.914				
20.40	1.914	1.905	1.915	49	1.59E-02	1.24E-04	1.93E-07
39.73	1.906	1.893	1.910	187	4.13E-03	2.14E-04	8.67E-08
55.00	1.900	1.884	1.903	359	2.14E-03	2.11E-04	4.42E-08
40.06	1.900	1.885	1.900				
10.66	1.910	1.900	1.905				
39.75	1.903	1.888	1.906	104	7.41E-03	1.36E-04	9.91E-08
55.00	1.899	1.883	1.901	113	6.78E-03	1.19E-04	7.93E-08
70.00	1.893	1.873	1.896	130	5.86E-03	2.17E-04	1.25E-07
90.13	1.886	1.862	1.889	167	4.53E-03	1.92E-04	8.51E-08
115.08	1.875	1.846	1.880	187	4.01E-03	2.24E-04	8.82E-08
154.89	1.839	1.791	1.857	463	1.58E-03	4.78E-04	7.39E-08
200.00	1.754	1.662	1.796	21260	3.22E-05	9.86E-04	3.11E-09
249.64	1.675	1.543	1.714	15511	4.02E-05	8.32E-04	3.28E-09
310.48	1.611	1.446	1.643	16225	3.53E-05	5.50E-04	1.90E-09
619.80	1.456	1.210	1.534	1116	4.47E-04	2.63E-04	1.15E-08
249.94	1.470	1.231	1.463				
115.08	1.486	1.255	1.478				
39.78	1.518	1.305	1.502				
10.66	1.555	1.361	1.537				
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 26cm to 33cm from top of the tube. Specimen swelled under 10.63 KPa.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		1.56		Unit Weight, kN/m <sup>3</sup>		17.19	
Sample Diameter, cm		6.35		Dry Unit Weight, kN/m <sup>3</sup>		11.63	
Area, cm <sup>2</sup>		31.71		Specific Gravity, measured		2.80	
Volume, cm <sup>3</sup>		49.32		Solids Height, cm		0.659	
Water Content, %		47.82		Volume of Solids, cm <sup>3</sup>		20.89	
Wet Mass, g		86.46		Volume of Voids, cm <sup>3</sup>		28.43	
Dry Mass, g		58.49					
Prepared By: IR				WSP Canada Inc		Checked By: MM	



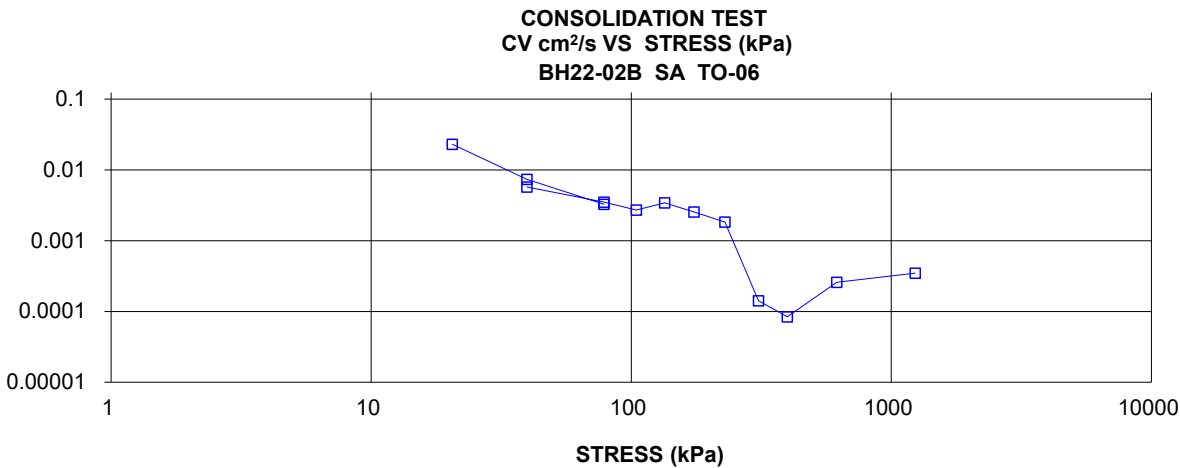




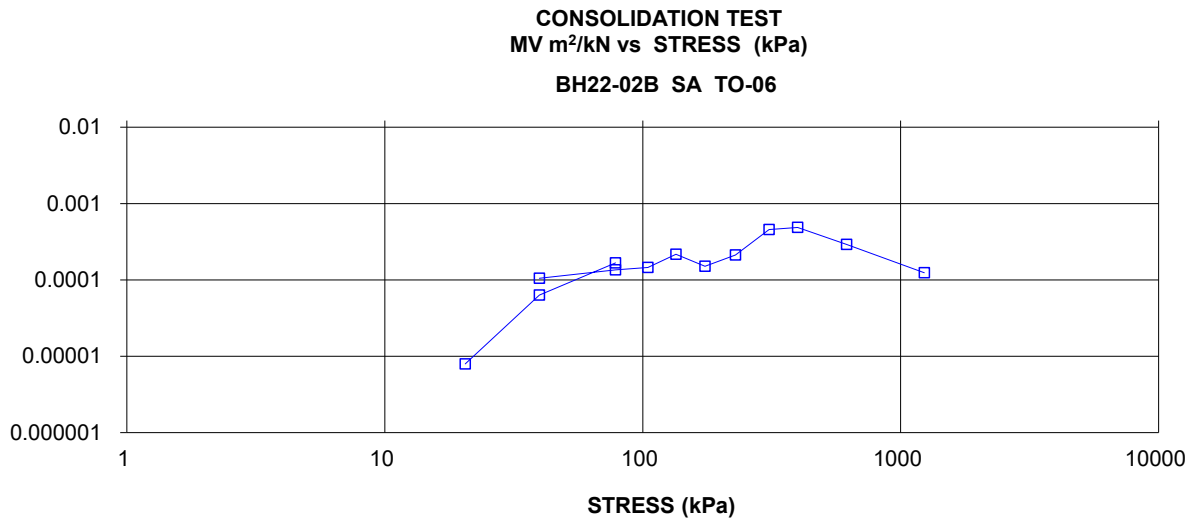


CONSOLIDATION TEST SUMMARY ASTM D2435/D2435M					FIGURE B21		
SAMPLE IDENTIFICATION							
Project Number		22513877A		Sample Number		TO-06	
Borehole Number		BH22-02B		Sample Depth, m		10.67-11.28	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		12					
Date Started		02/14/2023					
Date Completed		03/02/2023					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		2.54		Unit Weight, kN/m <sup>3</sup>		16.09	
Sample Diameter, cm		6.35		Dry Unit Weight, kN/m <sup>3</sup>		9.82	
Area, cm <sup>2</sup>		31.68		Specific Gravity, measured		2.78	
Volume, cm <sup>3</sup>		80.59		Solids Height, cm		0.916	
Water Content, %		63.88		Volume of Solids, cm <sup>3</sup>		29.03	
Wet Mass, g		132.25		Volume of Voids, cm <sup>3</sup>		51.56	
Dry Mass, g		80.7		Degree of Saturation, %		100.0	
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	2.544	1.776	2.544				
5.84	2.548	1.781	2.546				
10.62	2.553	1.786	2.551				
20.51	2.553	1.786	2.553	60	2.30E-02	7.95E-06	1.79E-08
39.73	2.550	1.783	2.551	187	7.38E-03	6.34E-05	4.59E-08
78.47	2.533	1.765	2.542	420	3.26E-03	1.67E-04	5.35E-08
39.73	2.539	1.771	2.536				
10.60	2.554	1.788	2.547				
39.73	2.547	1.779	2.551	240	5.75E-03	1.05E-04	5.93E-08
78.47	2.533	1.765	2.540	390	3.51E-03	1.35E-04	4.64E-08
104.59	2.524	1.754	2.528	501	2.71E-03	1.46E-04	3.87E-08
134.39	2.507	1.736	2.515	392	3.42E-03	2.16E-04	7.25E-08
174.34	2.492	1.719	2.500	519	2.55E-03	1.51E-04	3.77E-08
228.92	2.462	1.687	2.477	712	1.83E-03	2.12E-04	3.80E-08
308.72	2.369	1.586	2.416	8725	1.42E-04	4.58E-04	6.37E-09
398.27	2.259	1.465	2.314	13508	8.40E-05	4.86E-04	4.01E-09
617.12	2.096	1.287	2.177	3878	2.59E-04	2.92E-04	7.41E-09
1234.66	1.901	1.075	1.999	2436	3.48E-04	1.24E-04	4.23E-09
398.13	1.930	1.106	1.915				
174.34	1.956	1.135	1.943				
39.79	2.028	1.213	1.992				
10.57	2.080	1.270	2.054				
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 23cm to 29cm from top of the tube. Specimen swelled under 10.62kPa.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		2.08		Unit Weight, kN/m <sup>3</sup>		17.70	
Sample Diameter, cm		6.35		Dry Unit Weight, kN/m <sup>3</sup>		12.01	
Area, cm <sup>2</sup>		31.68		Specific Gravity, measured		2.78	
Volume, cm <sup>3</sup>		65.89		Solids Height, cm		0.916	
Water Content, %		47.35		Volume of Solids, cm <sup>3</sup>		29.03	
Wet Mass, g		118.91		Volume of Voids, cm <sup>3</sup>		36.86	
Dry Mass, g		80.7					
Prepared By: IR				WSP Canada Inc		Checked By: MM	

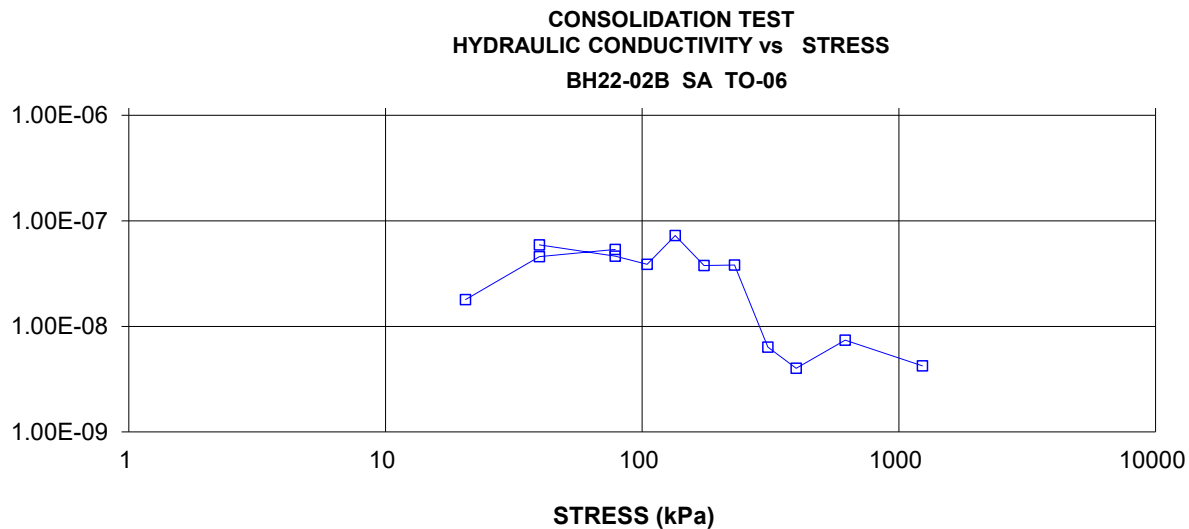
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s

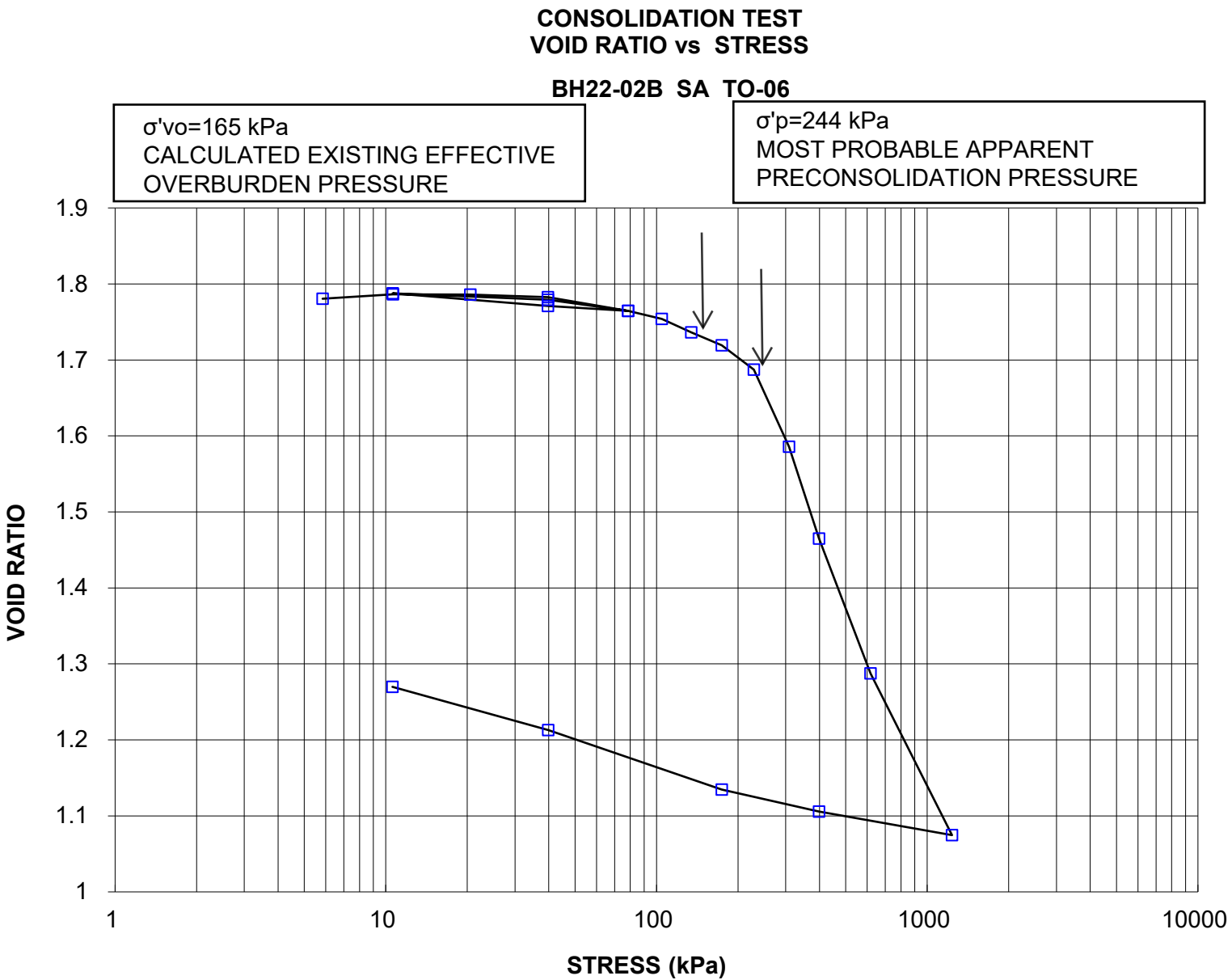


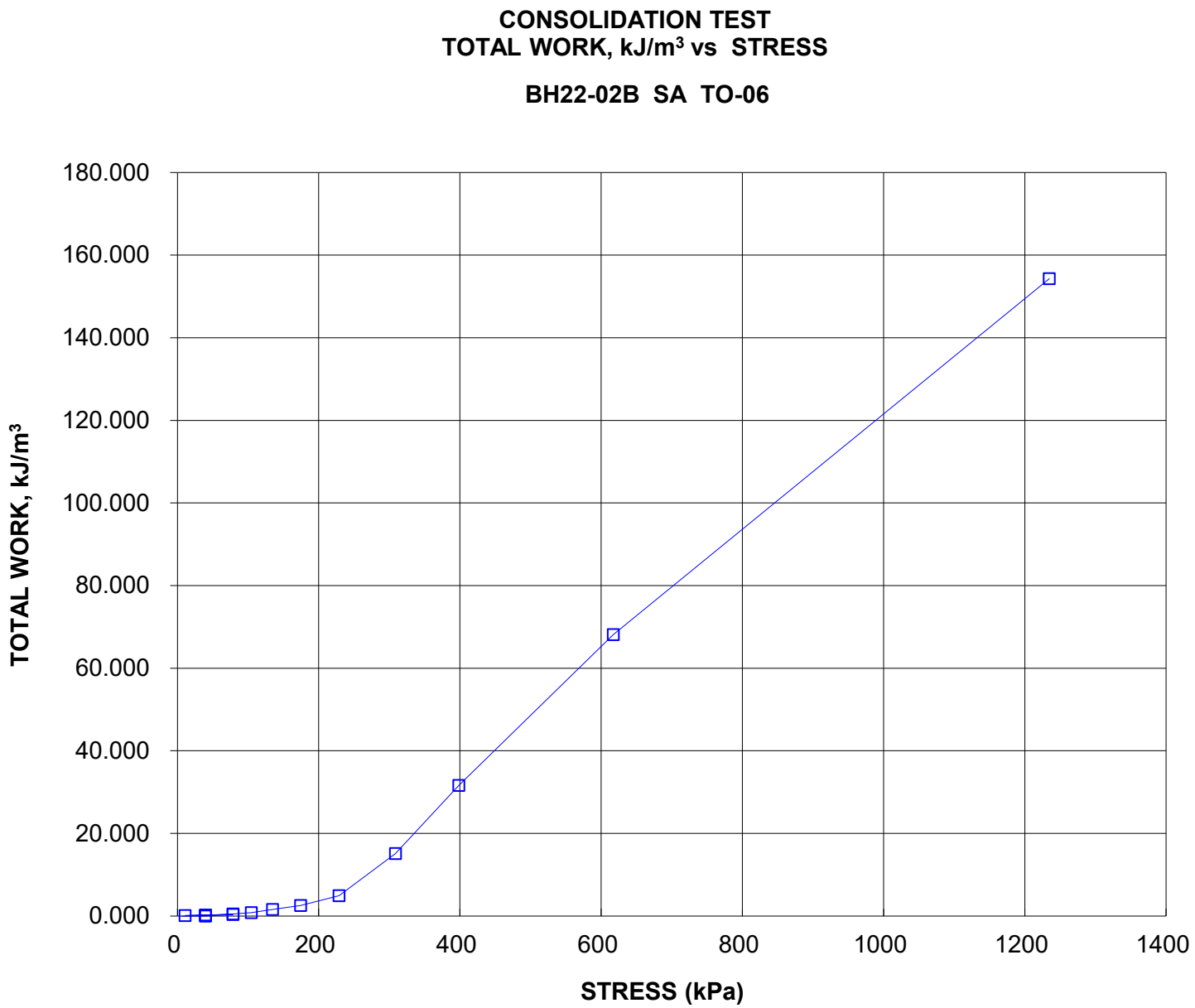
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



HYDRAULIC CONDUCTIVITY, cm/s

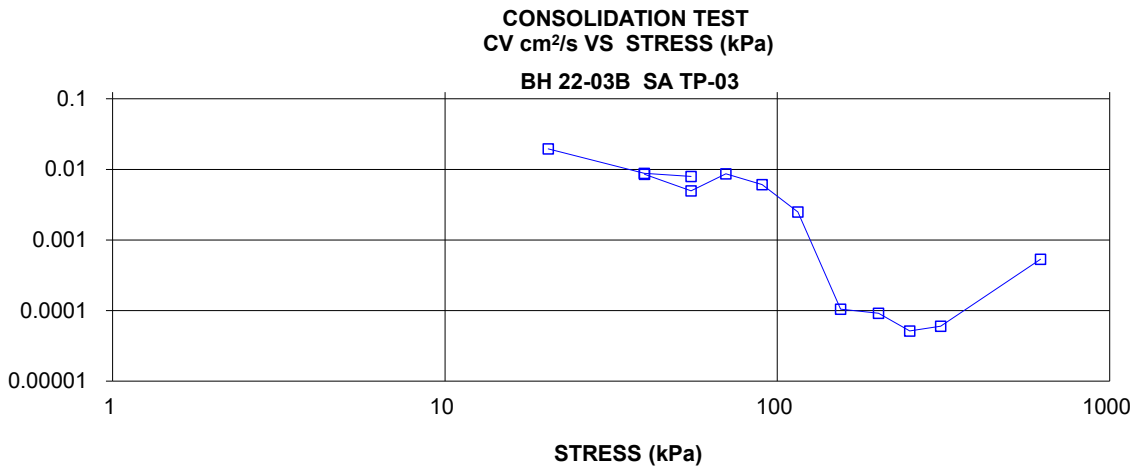




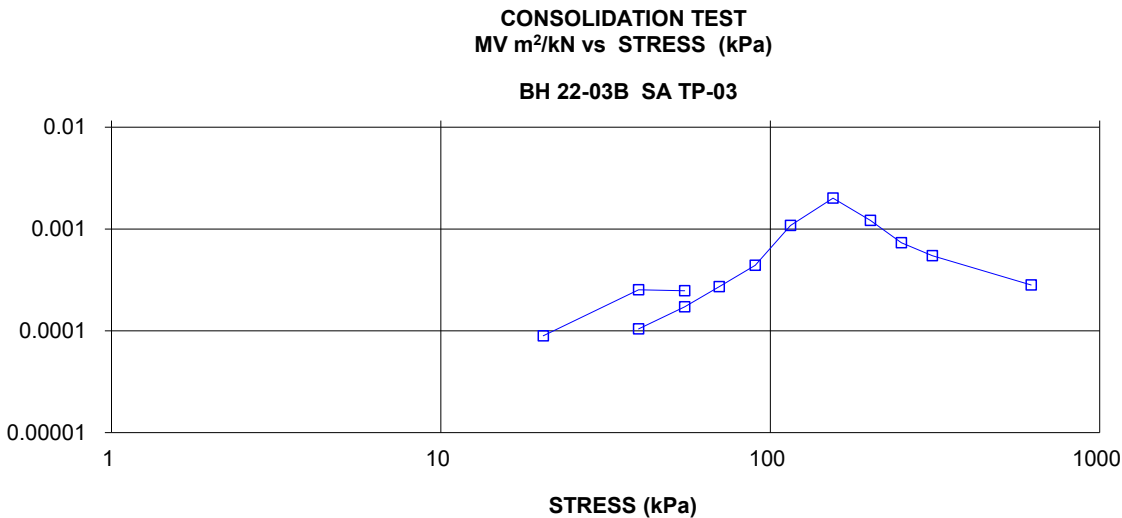


CONSOLIDATION TEST SUMMARY					FIGURE B22		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number		22513877A		Sample Number		TP-03	
Borehole Number		BH22-03B		Sample Depth, m		6.1-6.61	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		3					
Date Started		02/22/2023					
Date Completed		03/15/2023					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		2.53		Unit Weight, kN/m <sup>3</sup>		14.95	
Sample Diameter, cm		6.35		Dry Unit Weight, kN/m <sup>3</sup>		8.13	
Area, cm <sup>2</sup>		31.68		Specific Gravity, measured		2.77	
Volume, cm <sup>3</sup>		80.09		Solids Height, cm		0.757	
Water Content, %		83.81		Volume of Solids, cm <sup>3</sup>		23.97	
Wet Mass, g		122.07		Volume of Voids, cm <sup>3</sup>		56.11	
Dry Mass, g		66.41		Degree of Saturation, %		99.2	
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	2.528	2.340	2.528				
5.87	2.813	2.716	2.670				
10.71	2.814	2.719	2.813				
20.45	2.812	2.716	2.813	86	1.95E-02	8.93E-05	1.71E-07
39.80	2.800	2.699	2.806	190	8.78E-03	2.53E-04	2.18E-07
55.00	2.790	2.687	2.795	208	7.96E-03	2.47E-04	1.93E-07
39.85	2.791	2.688	2.791				
10.71	2.801	2.701	2.796				
39.85	2.793	2.690	2.797	194	8.55E-03	1.05E-04	8.76E-08
55.00	2.786	2.682	2.790	332	4.97E-03	1.72E-04	8.39E-08
70.02	2.776	2.668	2.781	190	8.63E-03	2.71E-04	2.29E-07
90.04	2.754	2.639	2.765	267	6.07E-03	4.41E-04	2.62E-07
115.23	2.685	2.548	2.719	628	2.50E-03	1.08E-03	2.64E-07
155.01	2.483	2.281	2.584	13500	1.05E-04	2.01E-03	2.06E-08
201.43	2.340	2.092	2.411	13500	9.13E-05	1.22E-03	1.09E-08
250.06	2.250	1.973	2.295	21705	5.14E-05	7.35E-04	3.70E-09
310.20	2.167	1.863	2.208	17249	5.99E-05	5.47E-04	3.21E-09
619.28	1.946	1.571	2.056	1682	5.33E-04	2.82E-04	1.47E-08
249.79	1.965	1.596	1.955				
115.23	1.986	1.624	1.975				
39.85	2.018	1.667	2.002				
10.75	2.056	1.717	2.037				
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 36cm to 43cm from top of the tube. Specimen swelled under 10.71 KPa.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		2.06		Unit Weight, kN/m <sup>3</sup>		15.02	
Sample Diameter, cm		6.35		Dry Unit Weight, kN/m <sup>3</sup>		10.00	
Area, cm <sup>2</sup>		31.68		Specific Gravity, measured		2.77	
Volume, cm <sup>3</sup>		65.13		Solids Height, cm		0.757	
Water Content, %		50.20		Volume of Solids, cm <sup>3</sup>		23.97	
Wet Mass, g		99.75		Volume of Voids, cm <sup>3</sup>		41.15	
Dry Mass, g		66.41					
Prepared By: IR				WSP Canada Inc		Checked By: MM	

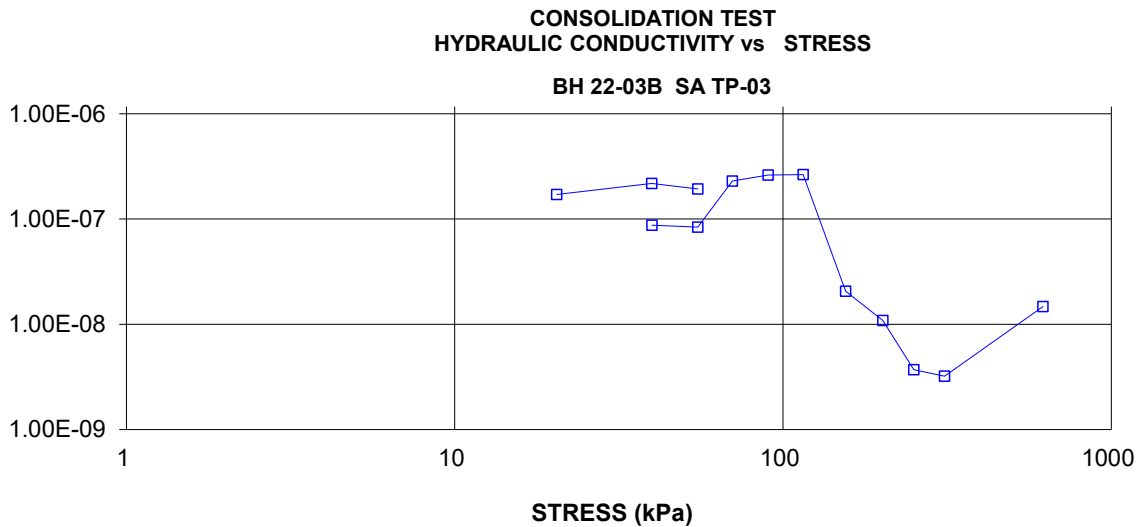
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



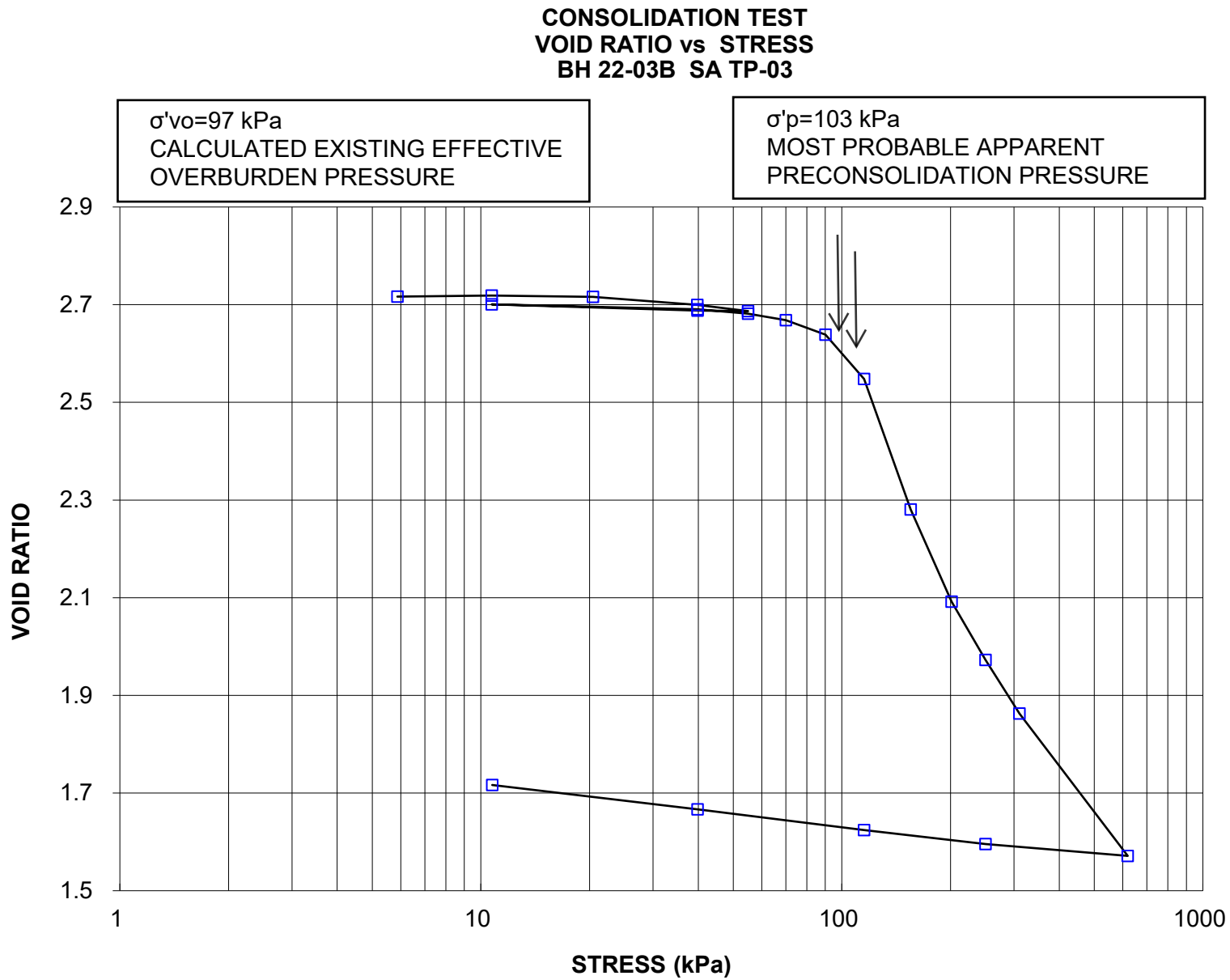
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN

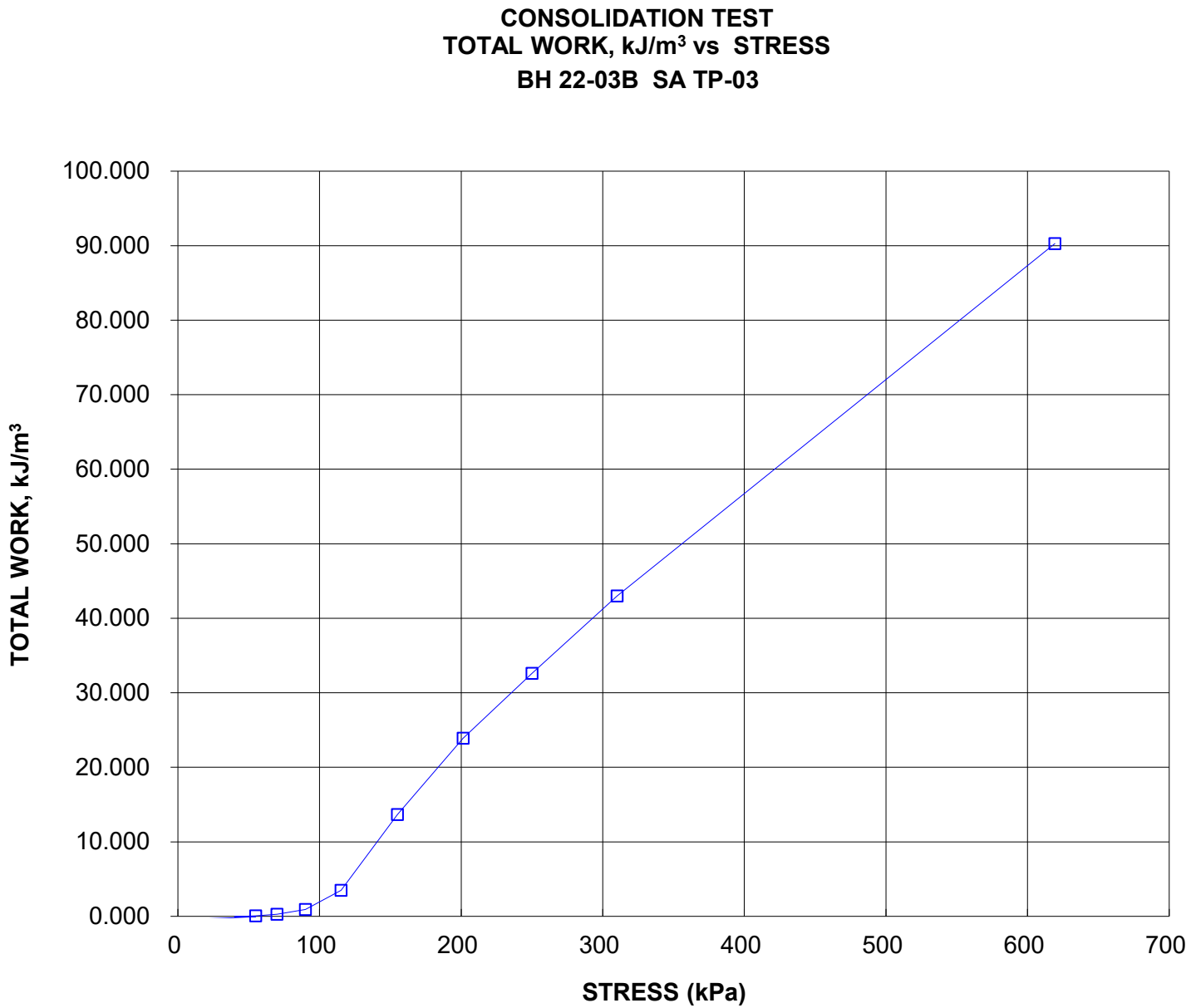


HYDRAULIC CONDUCTIVITY, cm/s

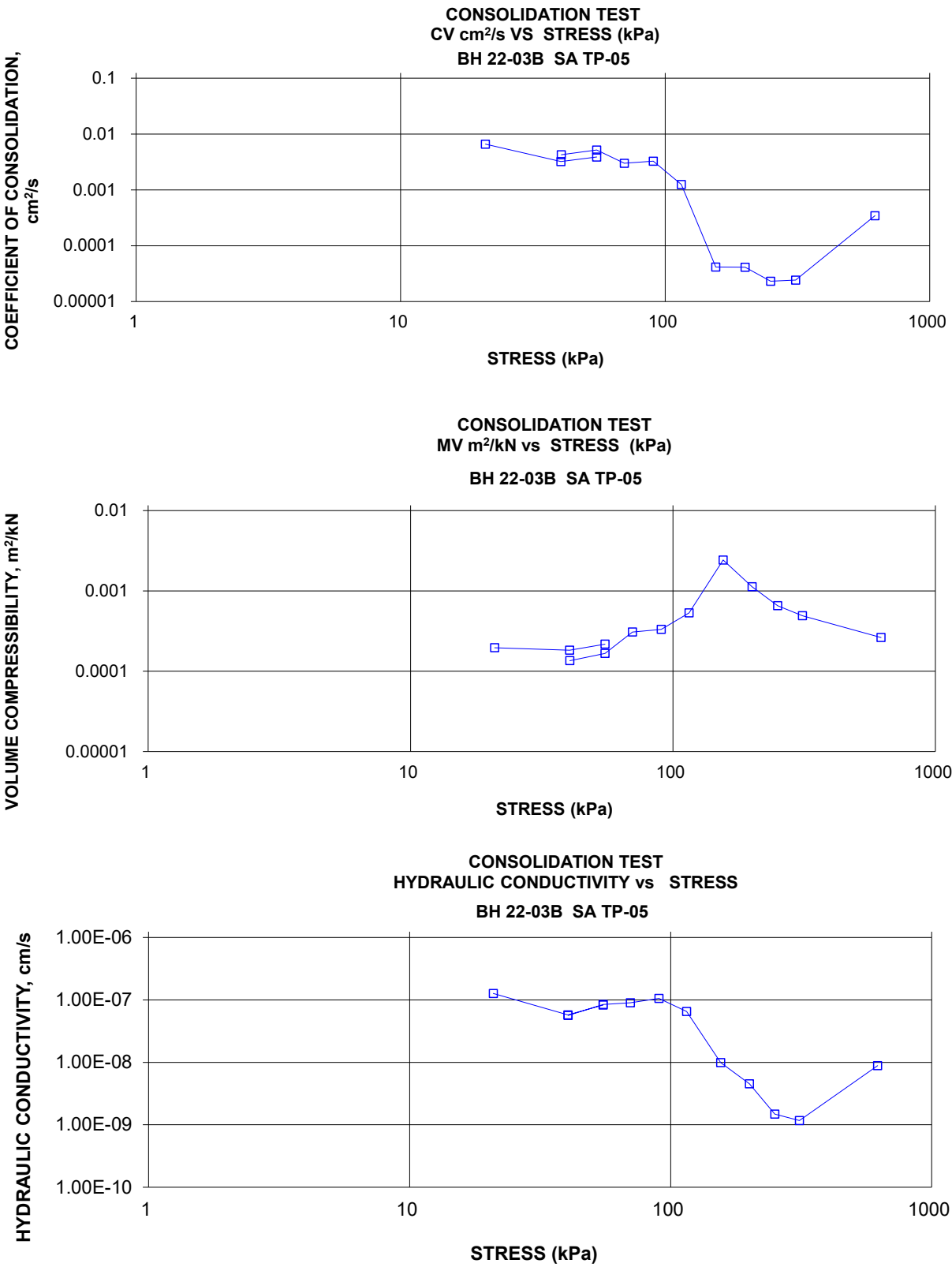


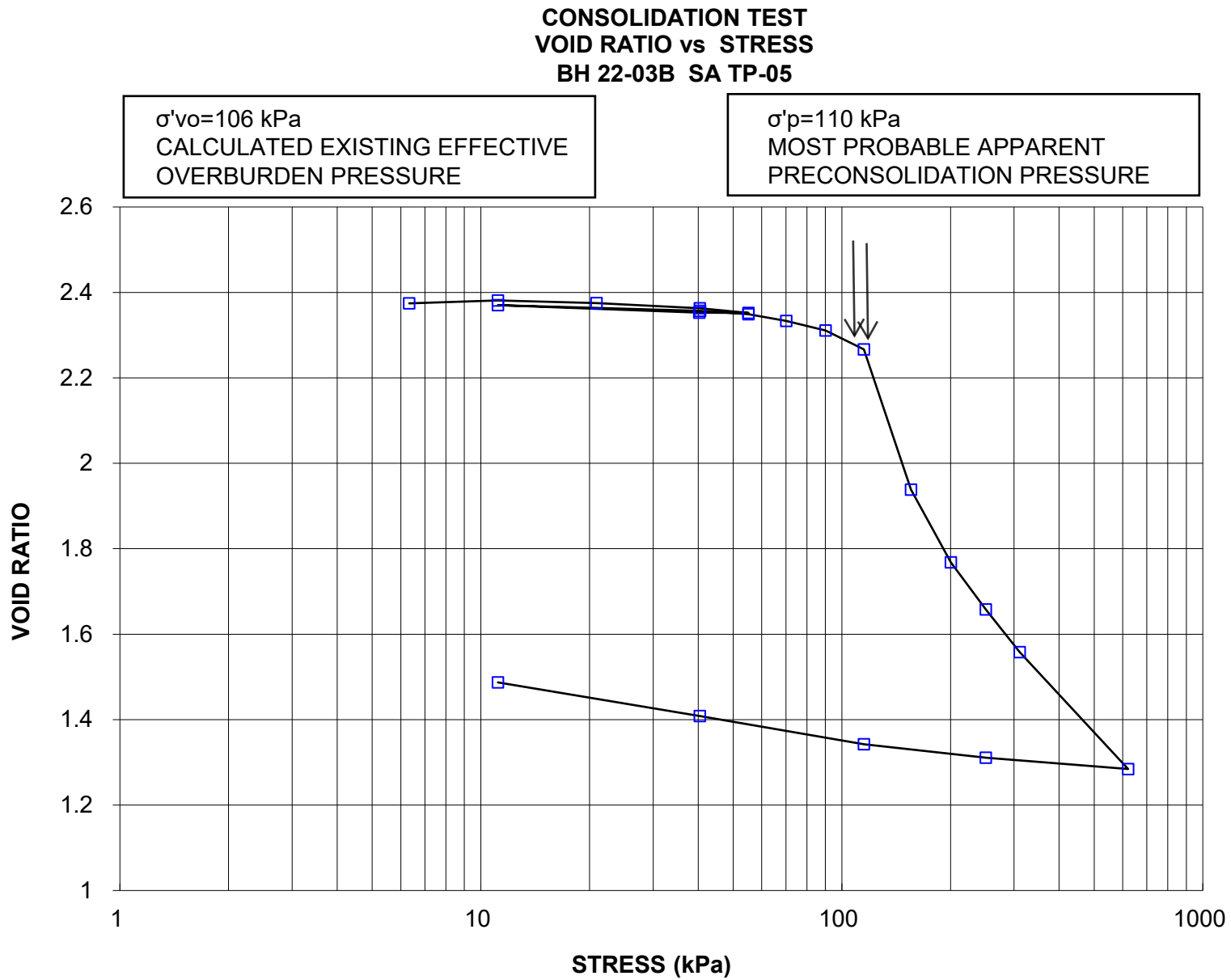


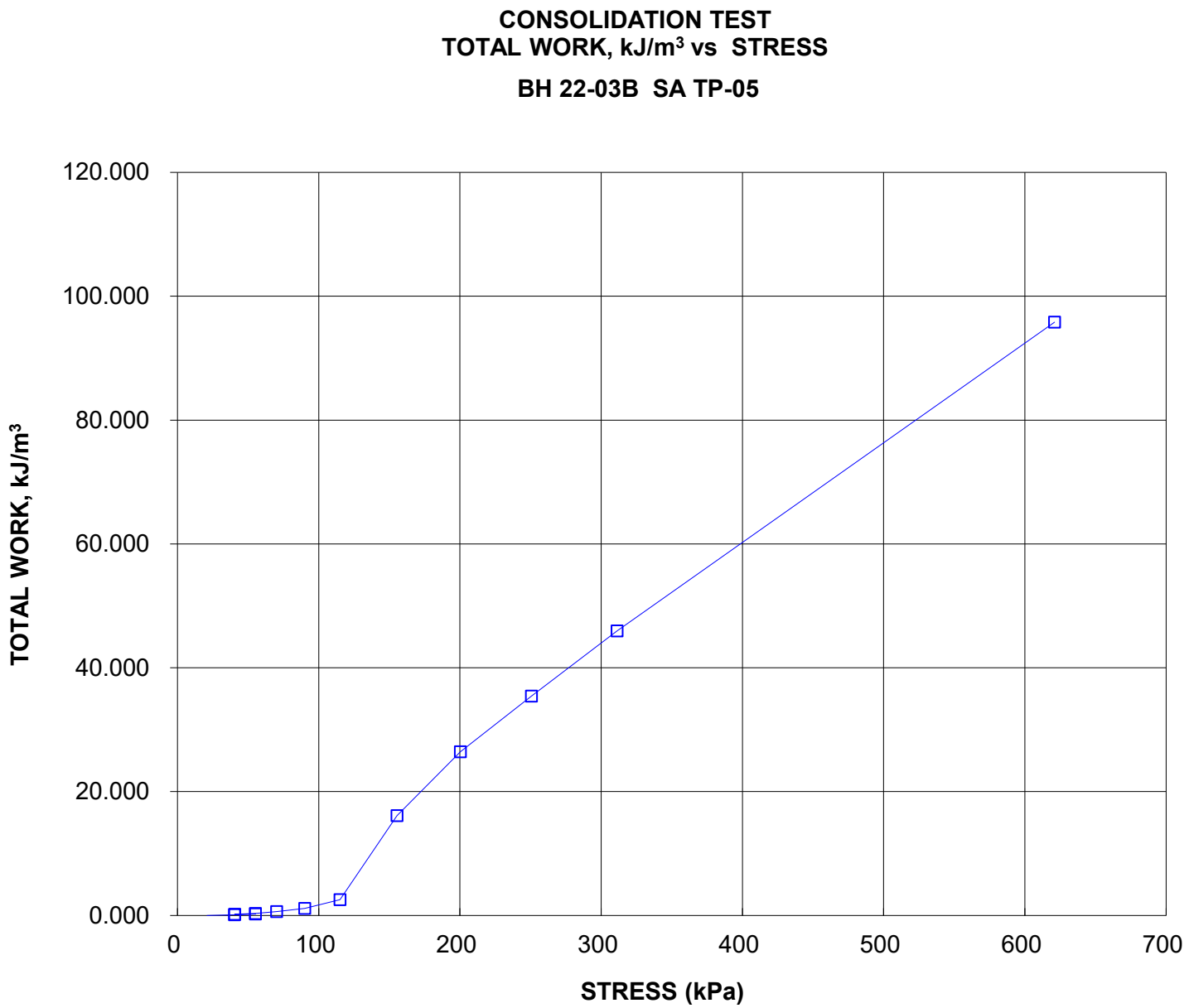




CONSOLIDATION TEST SUMMARY					FIGURE B23		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number		22513877A		Sample Number		TP-05	
Borehole Number		BH22-03B		Sample Depth, m		7.32-7.81	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		8					
Date Started		02/22/2023					
Date Completed		03/10/2023					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		1.91		Unit Weight, kN/m <sup>3</sup>		15.00	
Sample Diameter, cm		6.34		Dry Unit Weight, kN/m <sup>3</sup>		8.12	
Area, cm <sup>2</sup>		31.61		Specific Gravity, measured		2.78	
Volume, cm <sup>3</sup>		60.22		Solids Height, cm		0.567	
Water Content, %		84.79		Volume of Solids, cm <sup>3</sup>		17.92	
Wet Mass, g		92.08		Volume of Voids, cm <sup>3</sup>		42.29	
Dry Mass, g		49.83		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	1.905	2.359	1.905				
6.33	1.914	2.375	1.909				
11.14	1.917	2.381	1.915				
20.92	1.914	2.375	1.916	118	6.59E-03	1.96E-04	1.27E-07
40.38	1.907	2.363	1.910	240	3.22E-03	1.83E-04	5.77E-08
55.12	1.901	2.352	1.904	198	3.88E-03	2.17E-04	8.26E-08
40.31	1.901	2.353	1.901				
11.14	1.911	2.370	1.906				
40.46	1.904	2.357	1.907	181	4.26E-03	1.35E-04	5.65E-08
55.12	1.899	2.349	1.901	148	5.18E-03	1.67E-04	8.47E-08
70.12	1.890	2.333	1.895	254	3.00E-03	3.07E-04	9.01E-08
90.18	1.878	2.311	1.884	231	3.26E-03	3.32E-04	1.06E-07
115.10	1.852	2.266	1.865	591	1.25E-03	5.33E-04	6.52E-08
155.38	1.666	1.938	1.759	15789	4.16E-05	2.43E-03	9.88E-09
200.35	1.570	1.768	1.618	13500	4.11E-05	1.13E-03	4.54E-09
250.61	1.507	1.658	1.538	21705	2.31E-05	6.53E-04	1.48E-09
311.21	1.451	1.558	1.479	19057	2.43E-05	4.91E-04	1.17E-09
620.99	1.295	1.284	1.373	1162	3.44E-04	2.63E-04	8.86E-09
250.38	1.310	1.311	1.303				
115.13	1.328	1.343	1.319				
40.38	1.366	1.409	1.347				
11.14	1.411	1.487	1.388				
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 20cm to 25cm from top of the tube.							
Specimen swelled under 11.14 KPa.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		1.41		Unit Weight, kN/m <sup>3</sup>		16.88	
Sample Diameter, cm		6.34		Dry Unit Weight, kN/m <sup>3</sup>		10.96	
Area, cm <sup>2</sup>		31.61		Specific Gravity, measured		2.78	
Volume, cm <sup>3</sup>		44.59		Solids Height, cm		0.567	
Water Content, %		54.00		Volume of Solids, cm <sup>3</sup>		17.92	
Wet Mass, g		76.74		Volume of Voids, cm <sup>3</sup>		26.66	
Dry Mass, g		49.83					
Prepared By: IR				WSP Canada Inc		Checked By: MM	

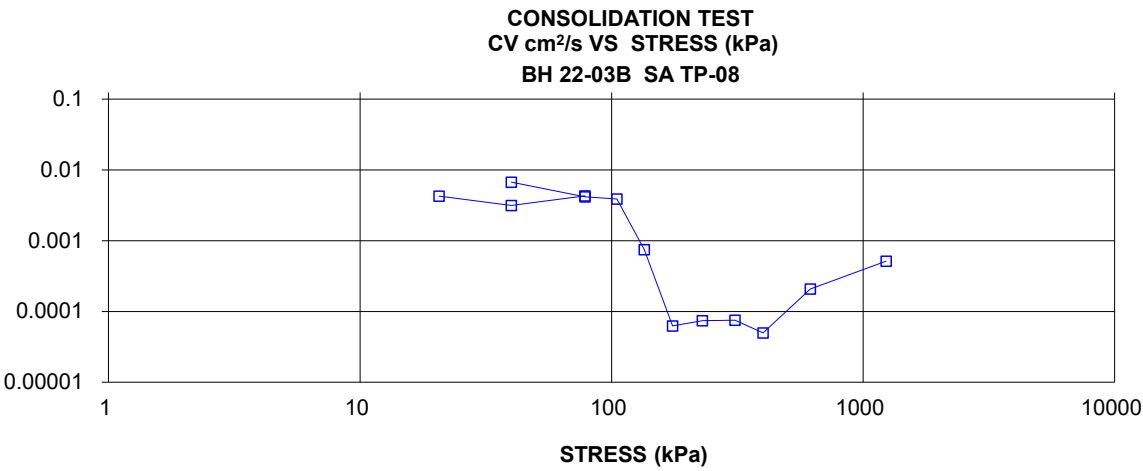




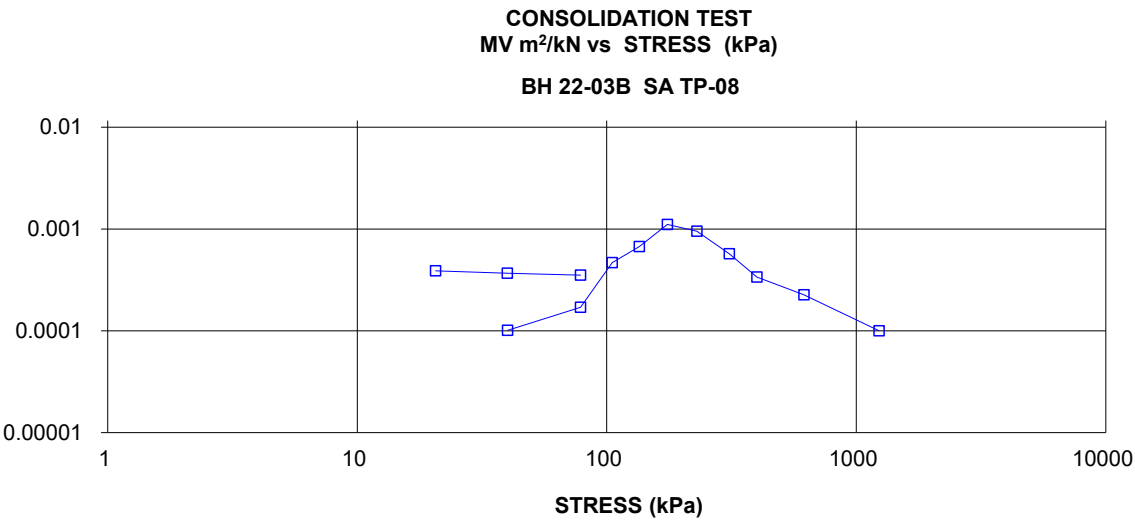


CONSOLIDATION TEST SUMMARY					FIGURE B24		
ASTM D2435/D2435M							
SAMPLE IDENTIFICATION							
Project Number		22513877A		Sample Number		TP-08	
Borehole Number		BH22-03B		Sample Depth, m		9.14-9.65	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		10					
Date Started		02/22/2023					
Date Completed		03/15/2023					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		2.53		Unit Weight, kN/m <sup>3</sup>		15.21	
Sample Diameter, cm		6.36		Dry Unit Weight, kN/m <sup>3</sup>		8.80	
Area, cm <sup>2</sup>		31.77		Specific Gravity, measured		2.79	
Volume, cm <sup>3</sup>		80.38		Solids Height, cm		0.814	
Water Content, %		72.87		Volume of Solids, cm <sup>3</sup>		25.85	
Wet Mass, g		124.69		Volume of Voids, cm <sup>3</sup>		54.52	
Dry Mass, g		72.13		Degree of Saturation, %		96.4	
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	2.530	2.109	2.530				
6.01	2.530	2.109	2.530				
10.67	2.529	2.108	2.530				
20.62	2.520	2.096	2.524	317	4.26E-03	3.89E-04	1.63E-07
39.95	2.502	2.074	2.511	426	3.14E-03	3.68E-04	1.13E-07
78.45	2.467	2.032	2.484	305	4.29E-03	3.52E-04	1.48E-07
39.95	2.472	2.038	2.470				
10.66	2.482	2.050	2.477				
39.99	2.475	2.041	2.479	194	6.71E-03	1.01E-04	6.65E-08
78.45	2.458	2.021	2.467	310	4.16E-03	1.71E-04	6.96E-08
105.06	2.427	1.982	2.443	327	3.87E-03	4.66E-04	1.77E-07
135.05	2.376	1.920	2.401	1637	7.47E-04	6.71E-04	4.91E-08
175.00	2.264	1.782	2.320	18272	6.24E-05	1.11E-03	6.79E-09
229.75	2.132	1.620	2.198	13855	7.39E-05	9.52E-04	6.90E-09
309.85	2.017	1.478	2.074	12116	7.53E-05	5.70E-04	4.20E-09
399.94	1.940	1.384	1.978	16667	4.98E-05	3.37E-04	1.64E-09
617.43	1.816	1.231	1.878	3599	2.08E-04	2.26E-04	4.59E-09
1234.93	1.659	1.039	1.737	1250	5.12E-04	1.00E-04	5.02E-09
400.10	1.684	1.070	1.672				
175.00	1.712	1.104	1.698				
39.95	1.768	1.173	1.740				
10.60	1.808	1.221	1.788				
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 30cm to 37cm from top of the tube. Specimen swelled under 10.67 KPa.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		1.81		Unit Weight, kN/m <sup>3</sup>		17.71	
Sample Diameter, cm		6.36		Dry Unit Weight, kN/m <sup>3</sup>		12.32	
Area, cm <sup>2</sup>		31.77		Specific Gravity, measured		2.79	
Volume, cm <sup>3</sup>		57.42		Solids Height, cm		0.814	
Water Content, %		43.80		Volume of Solids, cm <sup>3</sup>		25.85	
Wet Mass, g		103.72		Volume of Voids, cm <sup>3</sup>		31.57	
Dry Mass, g		72.13					
Prepared By: IR				WSP Canada Inc		Checked By: MM	

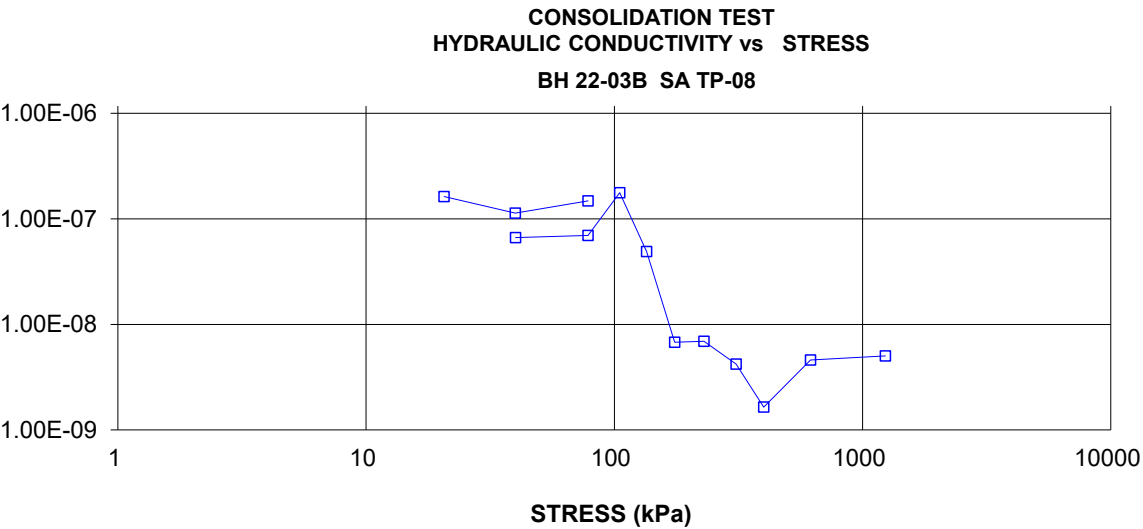
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



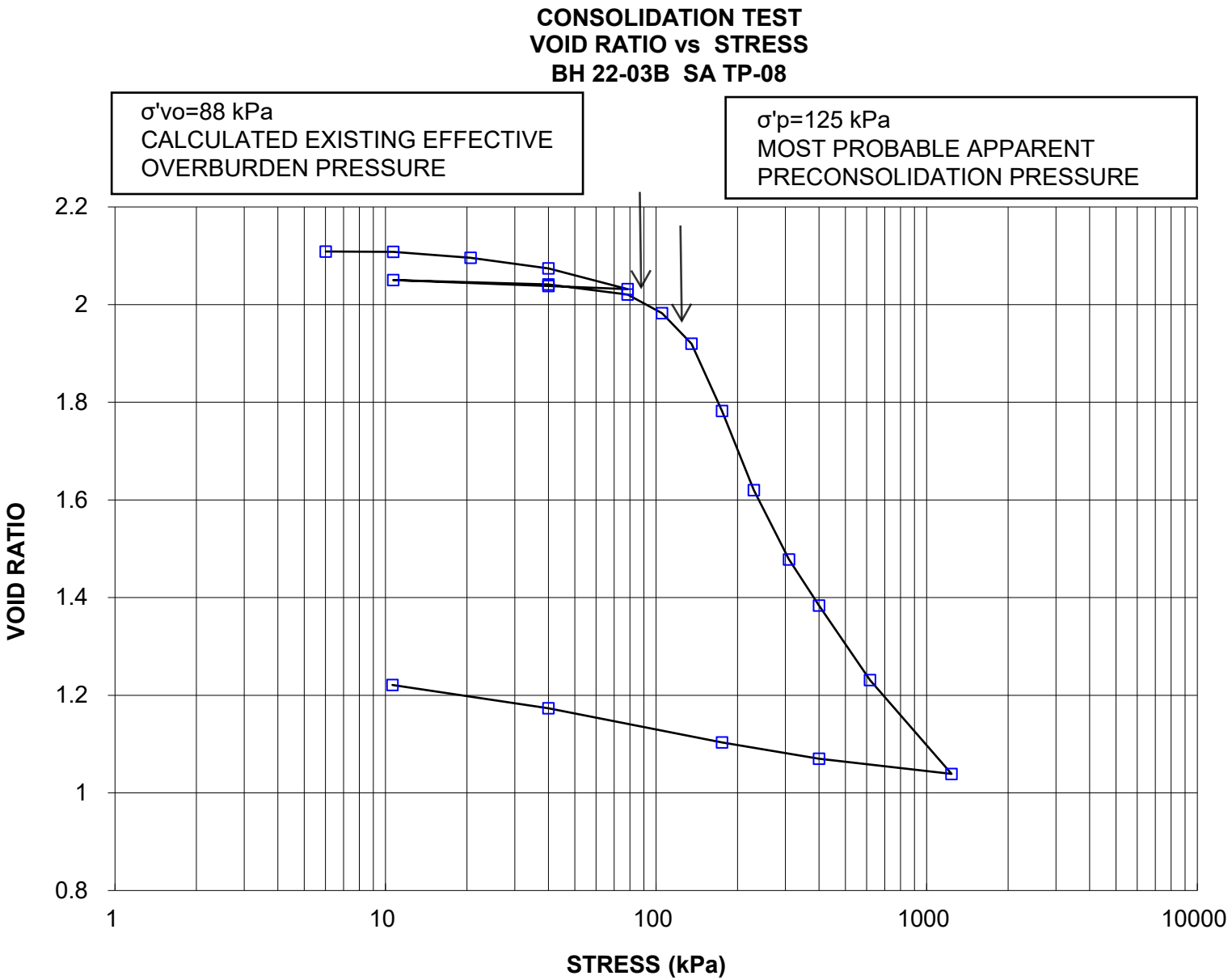
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



HYDRAULIC CONDUCTIVITY, cm/s

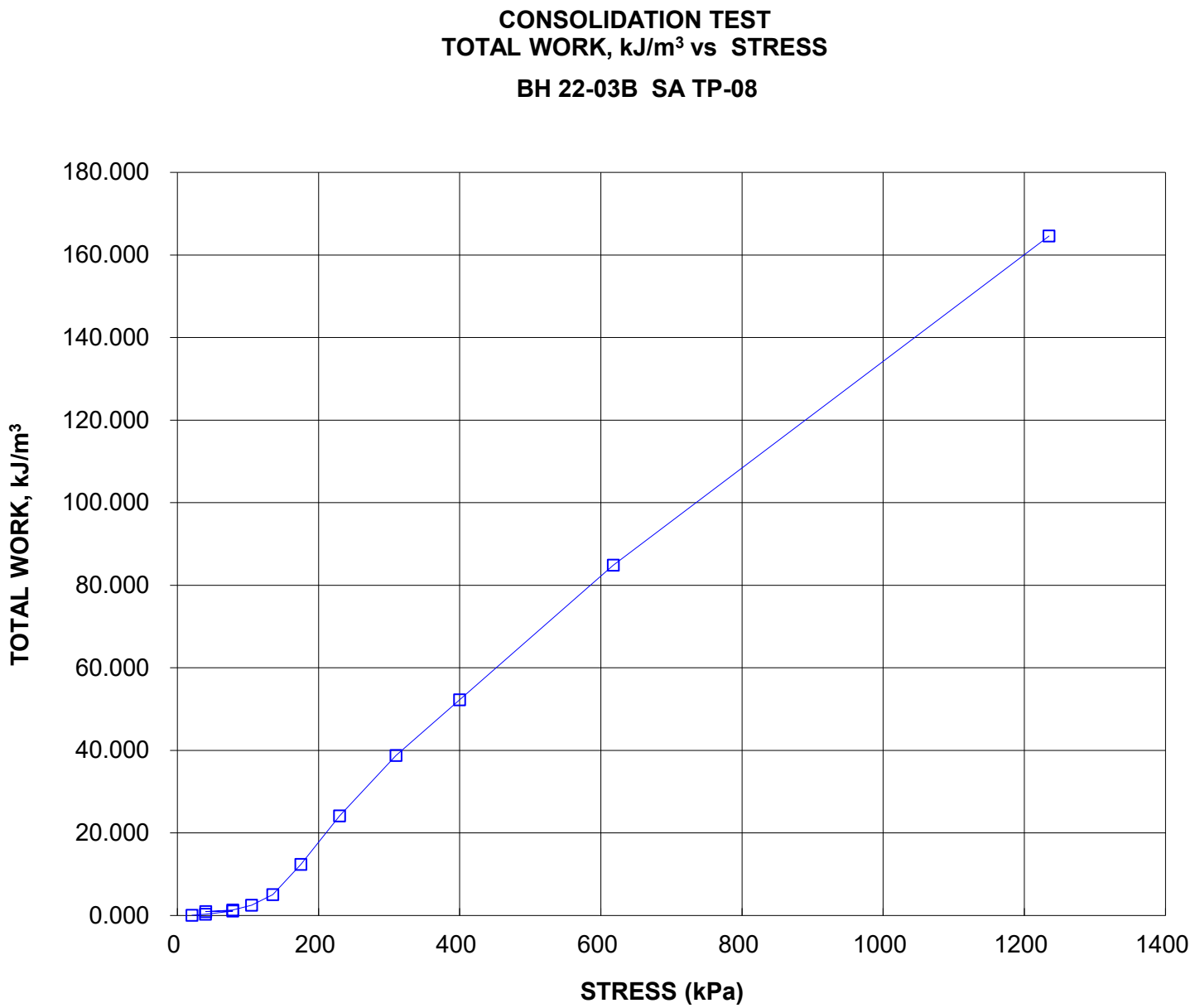






**CONSOLIDATION TEST  
TOTAL WORK VS STRESS**

**FIGURE B24**



Project No. 22513877A

Prepared By: IR

**Golder Associates**

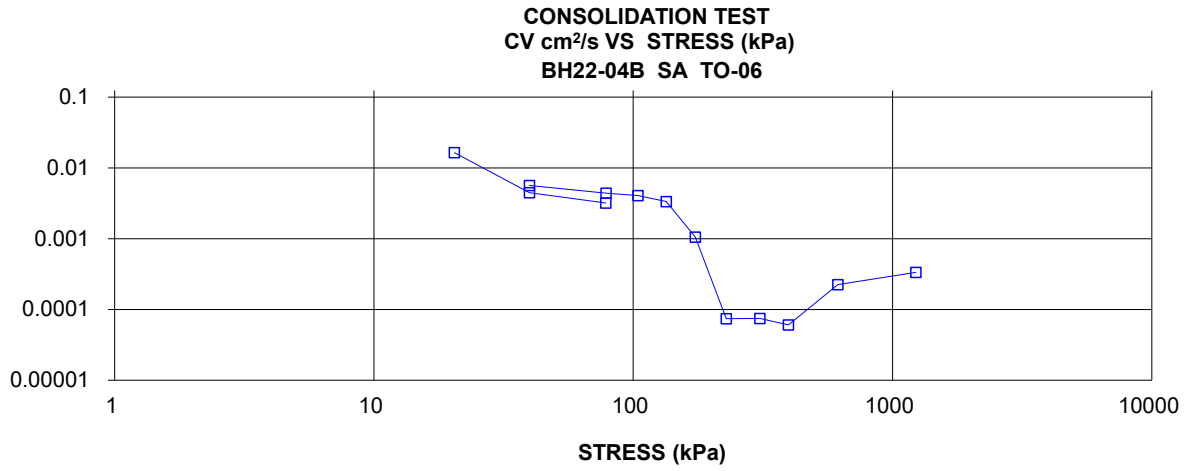
Checked By: MM

CONSOLIDATION TEST SUMMARY ASTM D2435/D2435M					FIGURE B25		
SAMPLE IDENTIFICATION							
Project Number		22513877A		Sample Number		TO-06	
Borehole Number		BH22-04B		Sample Depth, m		11.58-12.19	
TEST CONDITIONS							
Test Type		Laboratory Standard		Load Duration, hr		24	
Oedometer Number		11					
Date Started		02/14/2023					
Date Completed		03/02/2023					
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm		2.54		Unit Weight, kN/m <sup>3</sup>		16.02	
Sample Diameter, cm		6.36		Dry Unit Weight, kN/m <sup>3</sup>		9.82	
Area, cm <sup>2</sup>		31.73		Specific Gravity, measured		2.77	
Volume, cm <sup>3</sup>		80.43		Solids Height, cm		0.916	
Water Content, %		63.12		Volume of Solids, cm <sup>3</sup>		29.08	
Wet Mass, g		131.39		Volume of Voids, cm <sup>3</sup>		51.35	
Dry Mass, g		80.55		Degree of Saturation, %		99.0	
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	2.535	1.766	2.535				
5.91	2.536	1.767	2.535				
10.68	2.536	1.767	2.536				
20.44	2.533	1.764	2.535	83	1.64E-02	1.21E-04	1.95E-07
39.81	2.528	1.758	2.530	305	4.45E-03	1.08E-04	4.71E-08
78.44	2.508	1.737	2.518	420	3.20E-03	1.99E-04	6.25E-08
39.81	2.513	1.742	2.511				
10.63	2.528	1.758	2.521				
39.81	2.519	1.749	2.524	240	5.63E-03	1.12E-04	6.19E-08
78.47	2.508	1.737	2.514	305	4.39E-03	1.15E-04	4.96E-08
104.25	2.496	1.723	2.502	327	4.06E-03	1.87E-04	7.42E-08
133.98	2.475	1.700	2.485	392	3.34E-03	2.81E-04	9.21E-08
173.57	2.440	1.662	2.457	1220	1.05E-03	3.45E-04	3.55E-08
228.23	2.341	1.554	2.390	16302	7.43E-05	7.17E-04	5.22E-09
307.74	2.220	1.422	2.280	14764	7.47E-05	6.00E-04	4.39E-09
396.47	2.142	1.338	2.181	16667	6.05E-05	3.44E-04	2.04E-09
614.92	2.011	1.194	2.077	4069	2.25E-04	2.37E-04	5.23E-09
1230.33	1.843	1.010	1.927	2362	3.33E-04	1.08E-04	3.52E-09
396.76	1.869	1.039	1.856				
173.57	1.890	1.062	1.879				
40.00	1.954	1.132	1.922				
10.71	2.003	1.186	1.979				
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 32cm to 38cm from top of the tube.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm		2.00		Unit Weight, kN/m <sup>3</sup>		17.74	
Sample Diameter, cm		6.36		Dry Unit Weight, kN/m <sup>3</sup>		12.43	
Area, cm <sup>2</sup>		31.73		Specific Gravity, measured		2.77	
Volume, cm <sup>3</sup>		63.55		Solids Height, cm		0.916	
Water Content, %		42.71		Volume of Solids, cm <sup>3</sup>		29.08	
Wet Mass, g		114.95		Volume of Voids, cm <sup>3</sup>		34.47	
Dry Mass, g		80.55					
Prepared By: IR		WSP Canada Inc				Checked By: MM	

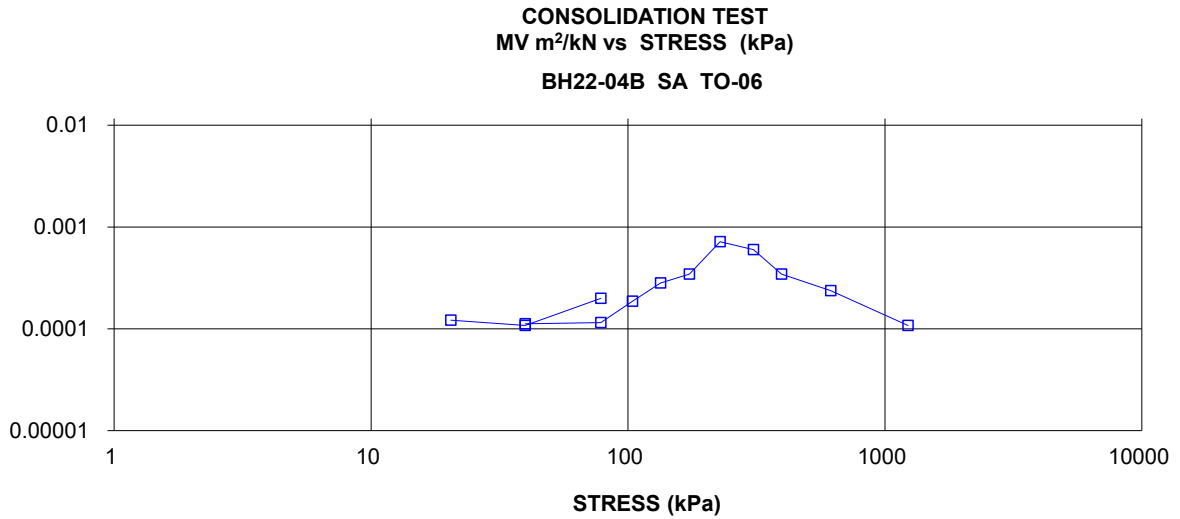
# CONSOLIDATION TEST SUMMARY

FIGURE B25

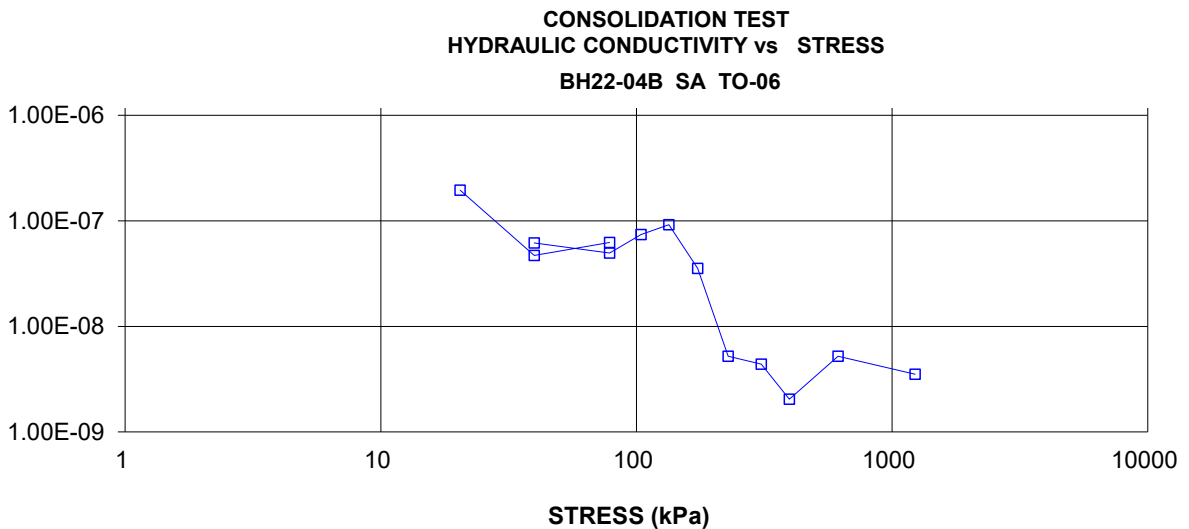
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s

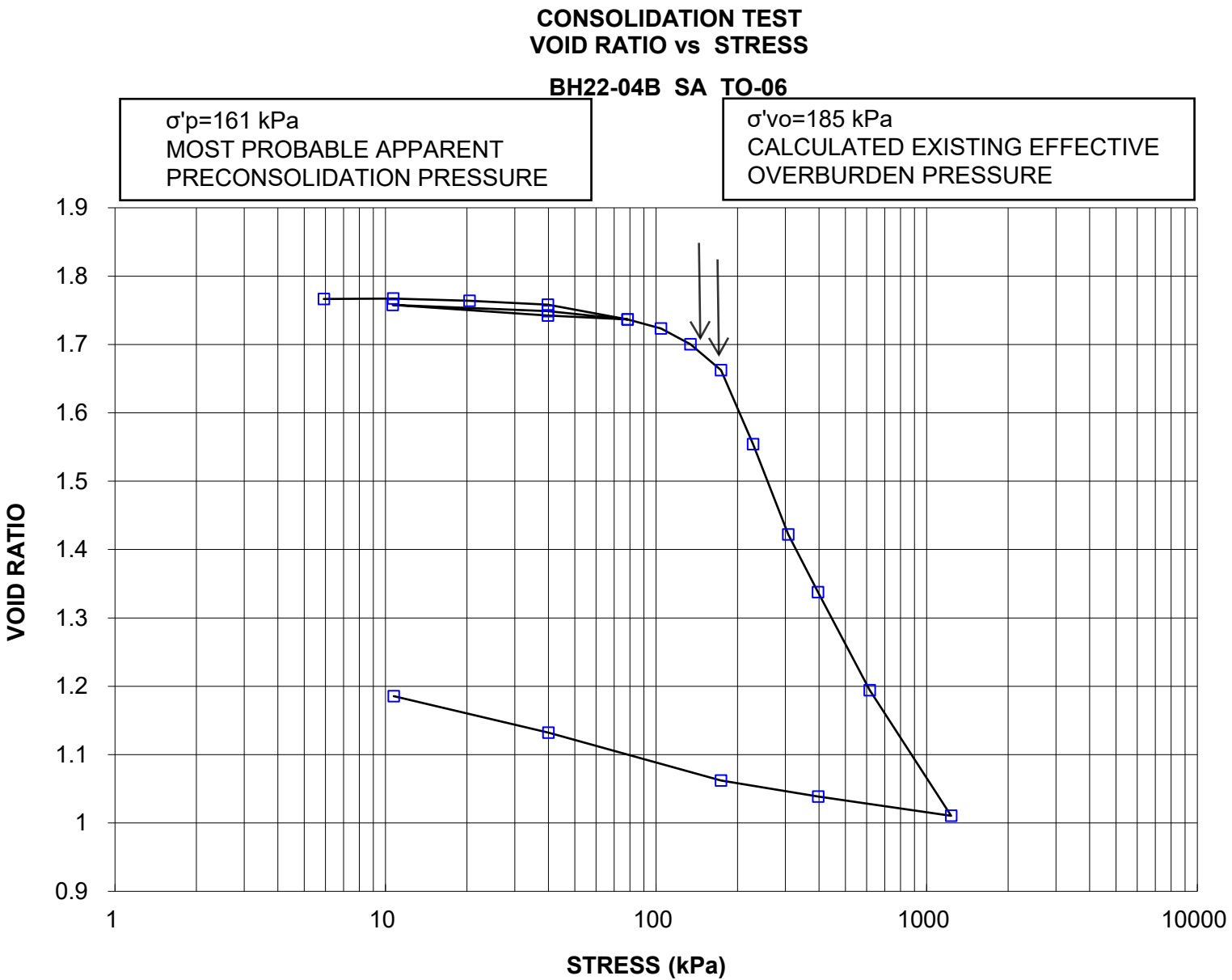


VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



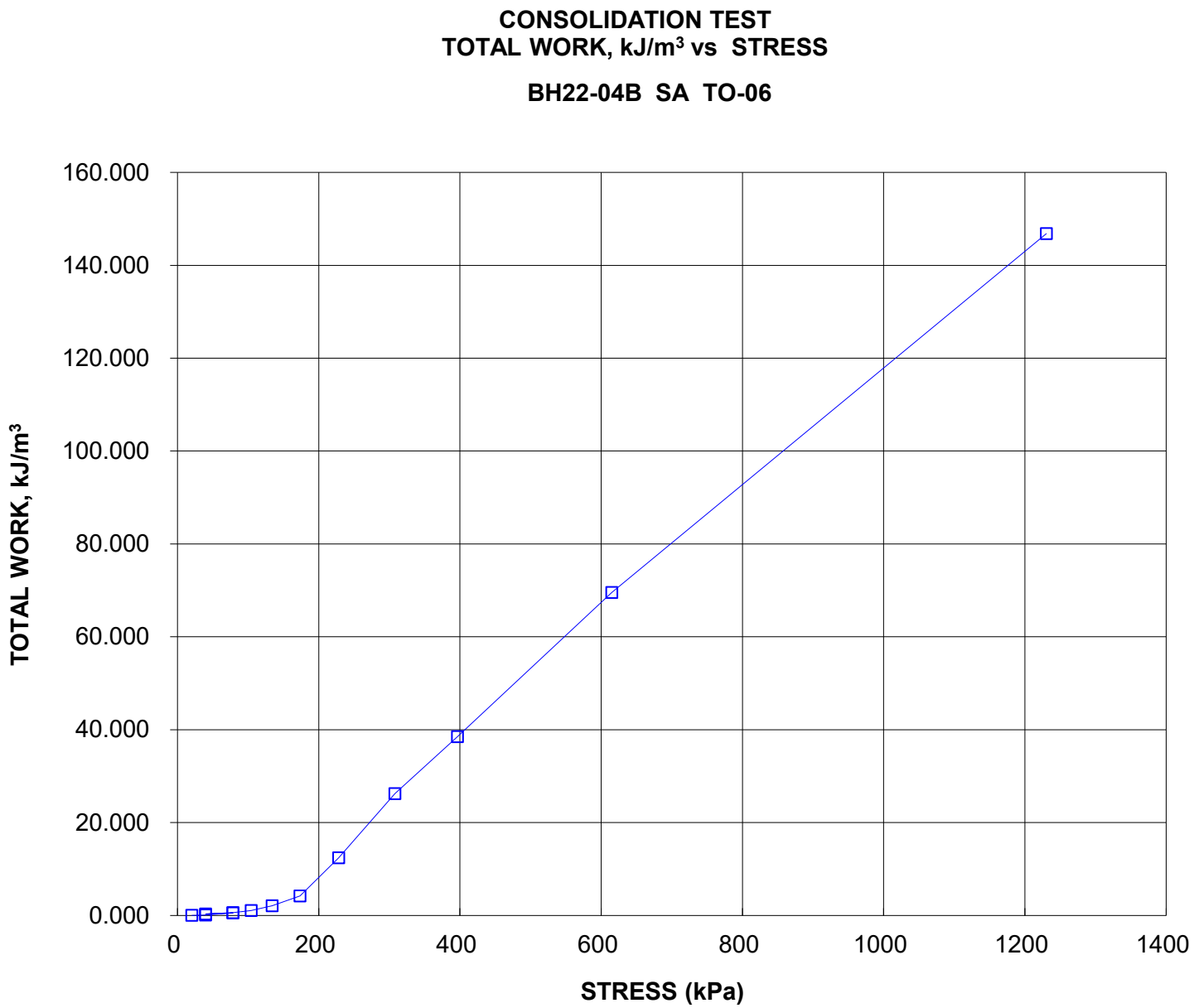
HYDRAULIC CONDUCTIVITY, cm/s





**CONSOLIDATION TEST  
TOTAL WORK VS STRESS**

**FIGURE B25**

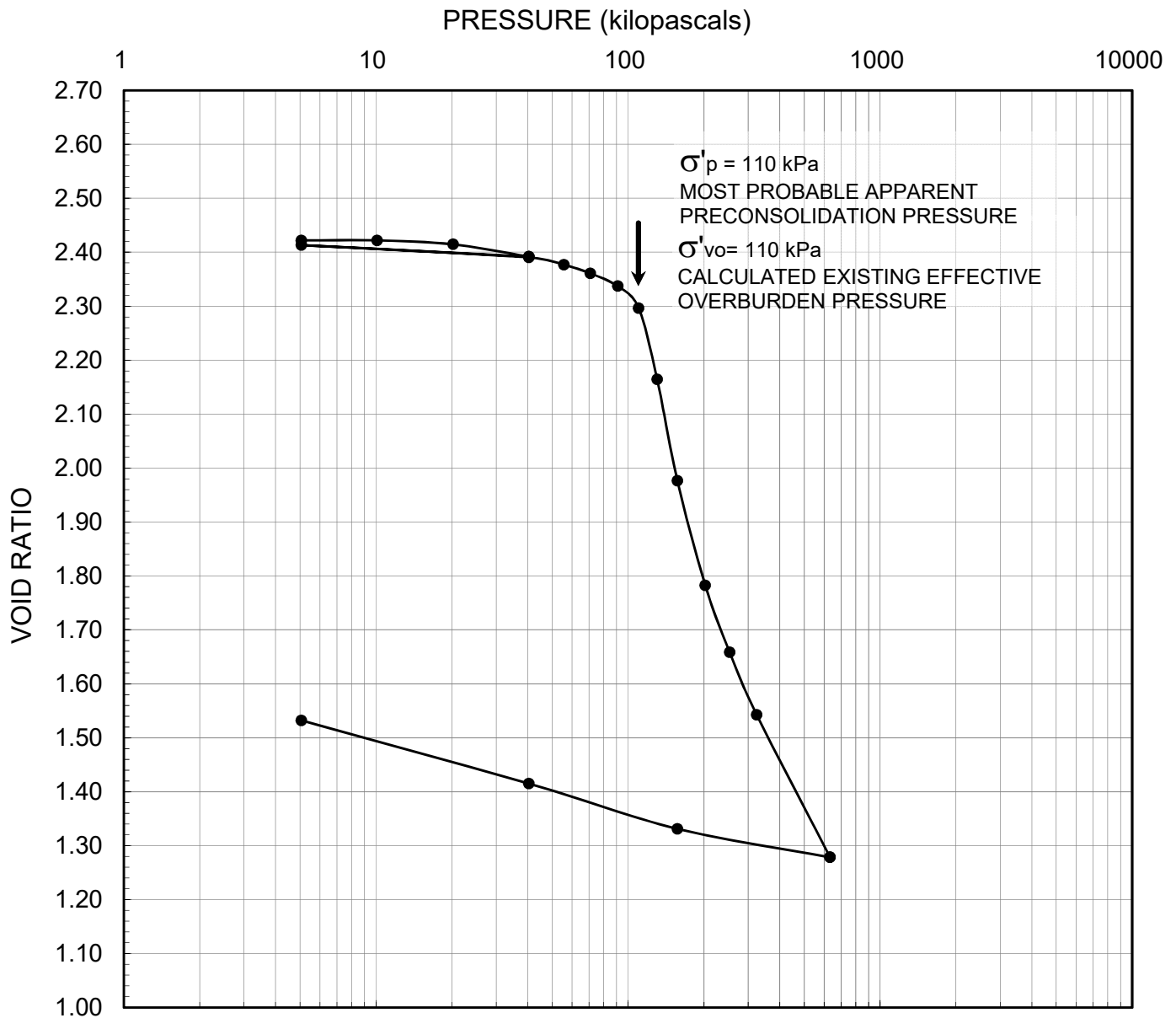


Project No. 22513877A

Prepared By: IR

**Golder Associates**

Checked By: MM



### LEGEND

Borehole: 18-1101	$w_i = 86\%$	$S_o = 99\%$	$g = 14.9 \text{ kN/m}^3$
Sample: 9	$w_f = 56\%$	$e_o = 2.42$	$G_s = 2.80$
Depth (m): 6.6	$w_l = 79\%$	$C_c = 2.29$	
Elevation (m): 46.2	$w_p = 23\%$	$C_r = 0.026$	



**GOLDER**

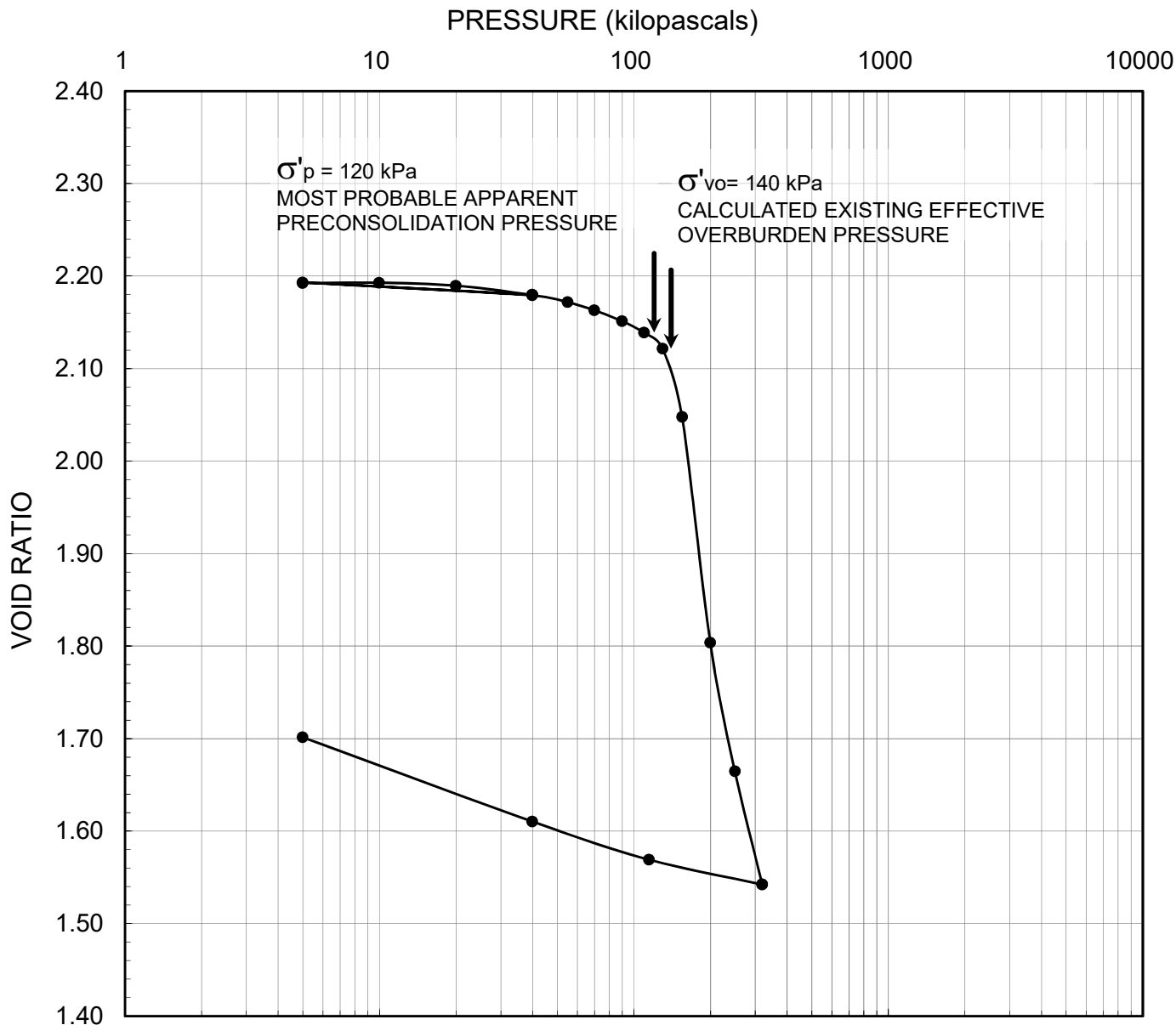
SCALE	AS SHOWN	TITLE
DATE	02-25-19	
CADD	N/A	
ENTERED	MI	
CHECK	CW	
REVIEW	CK	

## CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary  
PROJECT No. 1899802/1100 REV. 0

FIGURE

**B26**



### LEGEND

Borehole:	18-1101	$w_i = 79\%$	$S_o = 100\%$	$g = 15.2 \text{ kN/m}^3$
Sample:	10	$w_f = 50\%$	$e_o = 2.19$	$G_s = 2.76$
Depth (m):	7.9	$w_l = 78\%$	$C_c = 2.20$	
Elevation (m):	44.9	$w_p = 27\%$	$C_r = 0.016$	



**GOLDER**

SCALE	AS SHOWN	TITLE
DATE	02-25-19	
CADD	N/A	
ENTERED	MI	
CHECK	CW	
REVIEW	CK	

## CONSOLIDATION TEST RESULTS

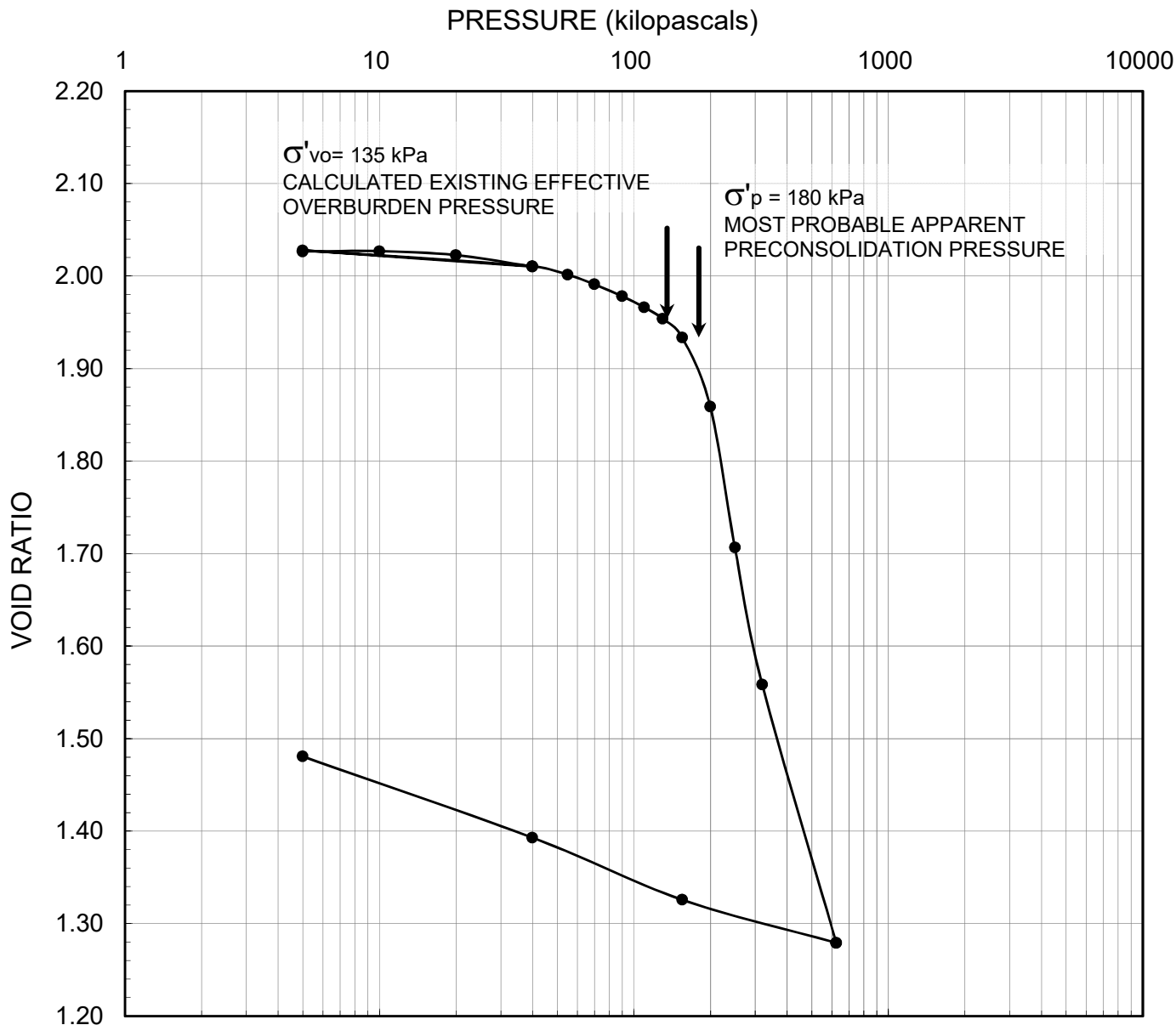
FILE No. Consolidation summary

PROJECT No. 1899802/1100 REV. 0

FIGURE

**B27**





#### LEGEND

Borehole:	18-1103B	$w_i = 72\%$	$S_o = 100\%$	$g = 15.6 \text{ kN/m}^3$
Sample:	1	$w_f = 54\%$	$e_o = 2.02$	$G_s = 2.80$
Depth (m):	7.6	$w_l = 66\%$	$C_c = 1.60$	
Elevation (m):	45.4	$w_p = 22\%$	$C_r = 0.019$	



**GOLDER**

SCALE AS SHOWN TITLE

DATE 02-25-19

CADD N/A

ENTERED MI

CHECK CW

REVIEW CK

## CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

PROJECT No. 1899802/1100 REV. 0

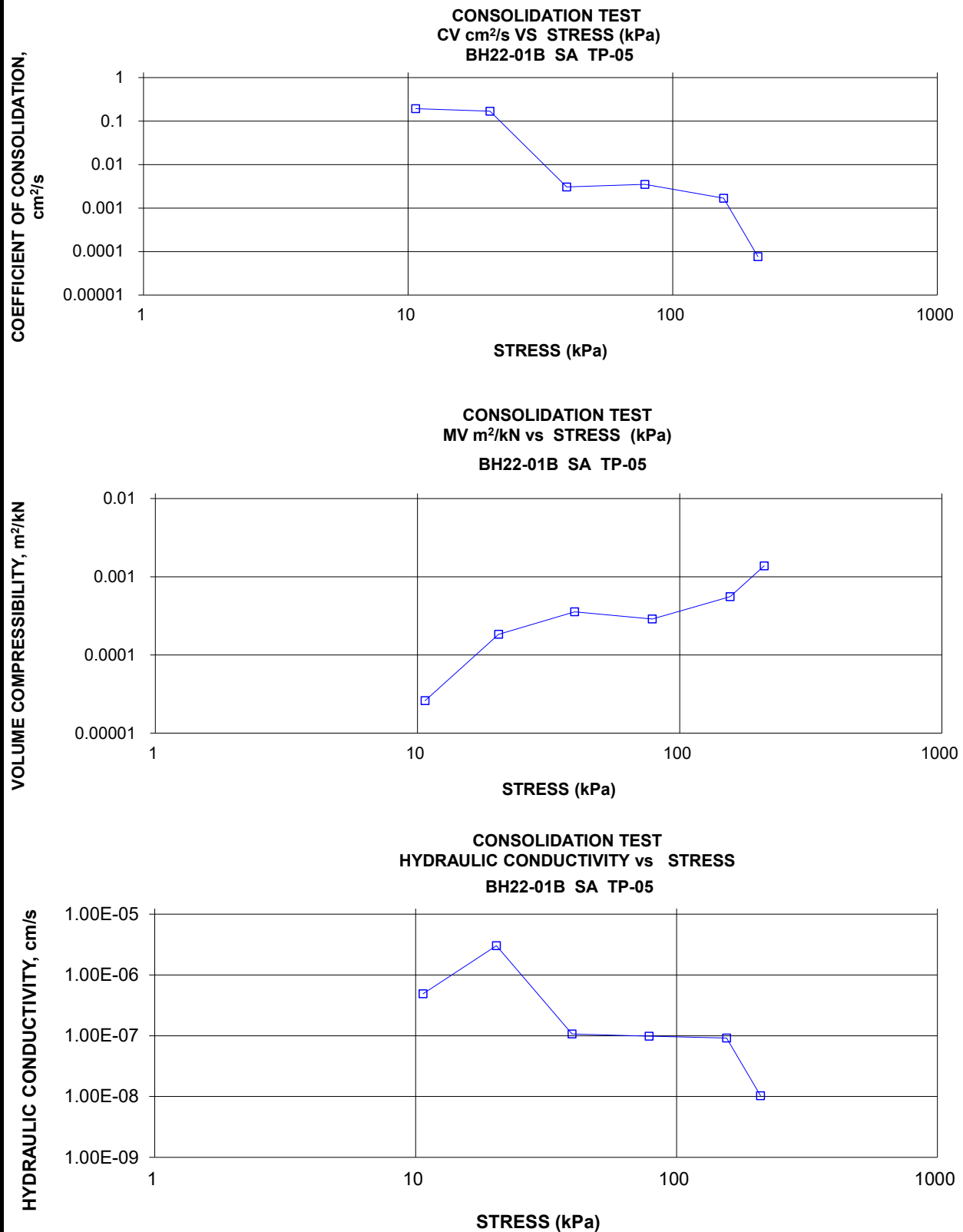
FIGURE

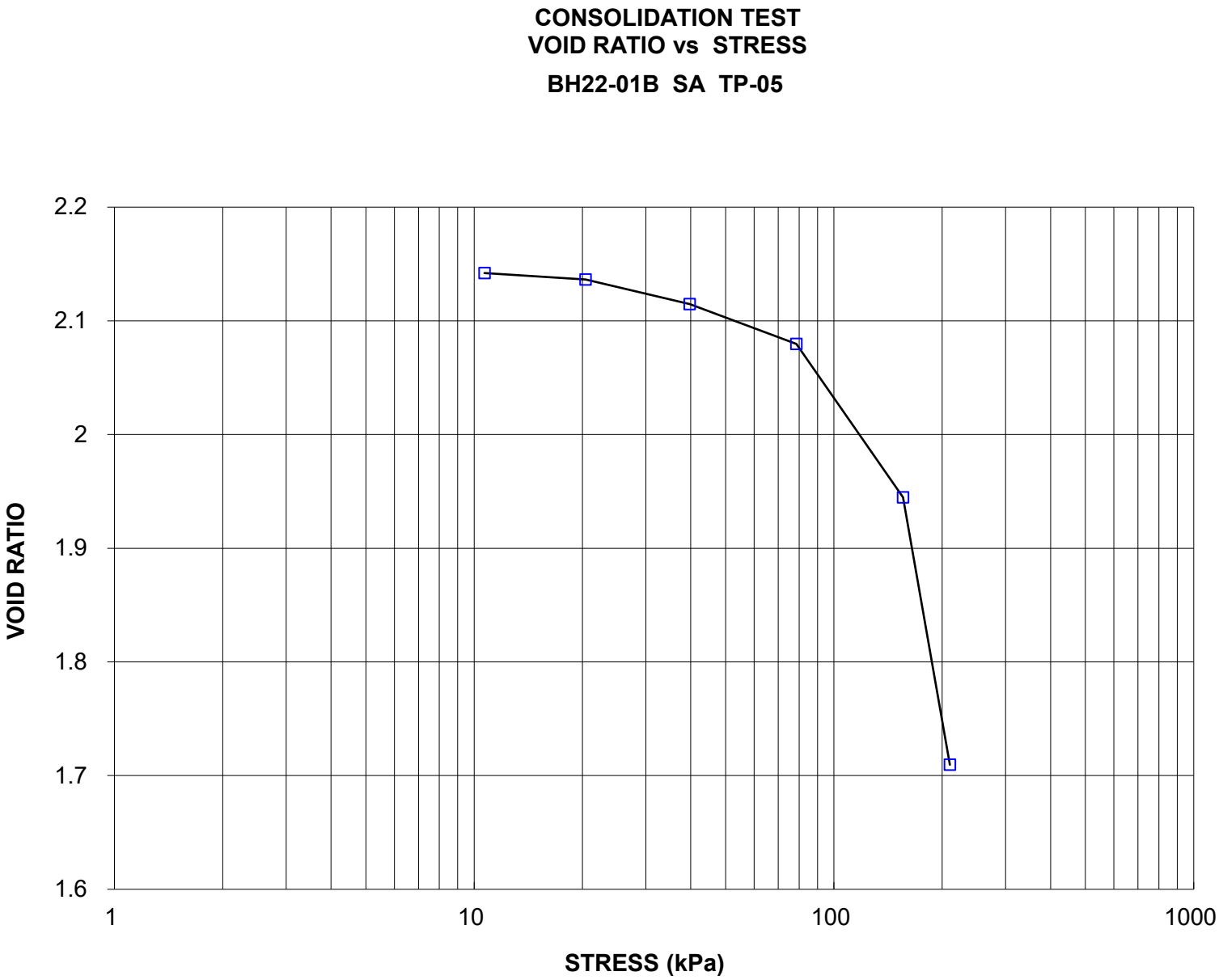
**B28**

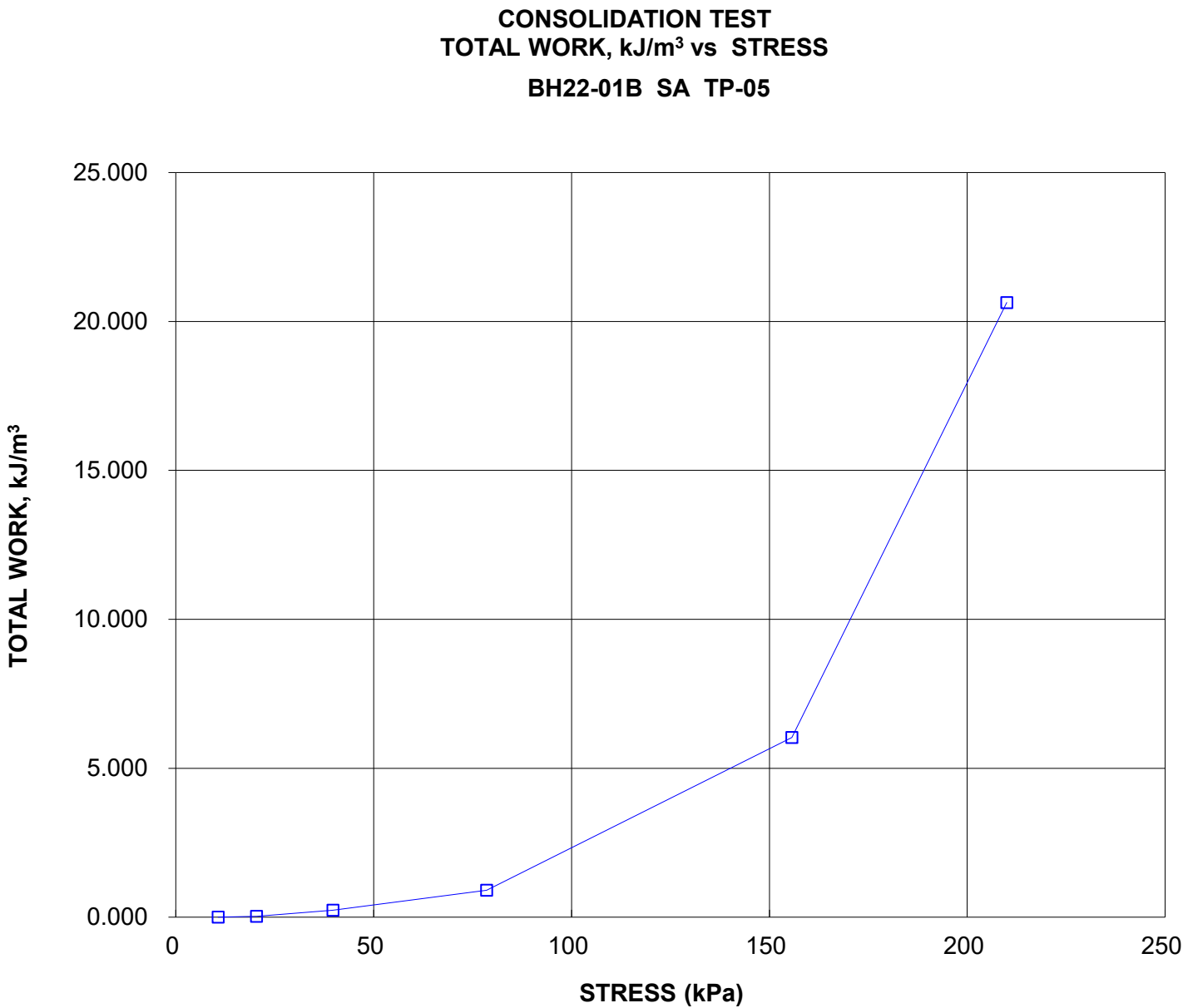
CONSOLIDATION TEST SUMMARY ASTM D2435/D2435M					FIGURE B29		
SAMPLE IDENTIFICATION							
Project Number	22513877A			Sample Number	TP-05		
Borehole Number	BH22-01B			Sample Depth, m	7.01-7.52		
TEST CONDITIONS							
Test Type	Laboratory Standard			Load Duration, hr	-		
Oedometer Number	2						
Date Started	02/22/2023						
Date Completed	03/23/2023						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	2.52			Unit Weight, kN/m <sup>3</sup>	15.26		
Sample Diameter, cm	6.36			Dry Unit Weight, kN/m <sup>3</sup>	8.64		
Area, cm <sup>2</sup>	31.72			Specific Gravity, measured	2.77		
Volume, cm <sup>3</sup>	80.06			Solids Height, cm	0.803		
Water Content, %	76.57			Volume of Solids, cm <sup>3</sup>	25.47		
Wet Mass, g	124.59			Volume of Voids, cm <sup>3</sup>	54.59		
Dry Mass, g,	70.56			Degree of Saturation, %	99.0		
TEST COMPUTATIONS							
Stress	Corr. Height	Void Ratio	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	2.524	2.143	2.524				
10.68	2.523	2.142	2.524	7	1.93E-01	2.60E-05	4.91E-07
20.40	2.519	2.136	2.521	8	1.68E-01	1.83E-04	3.03E-06
39.74	2.501	2.115	2.510	437	3.06E-03	3.56E-04	1.07E-07
78.55	2.473	2.080	2.487	375	3.50E-03	2.88E-04	9.87E-08
155.68	2.365	1.945	2.419	735	1.69E-03	5.56E-04	9.20E-08
209.99	2.176	1.710	2.270	14330	7.63E-05	1.38E-03	1.03E-08
<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 14 to 20cm from top of the tube.</p> <p>Creep test performed on 210kPa for 26 days.</p>							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	2.18			Unit Weight, kN/m <sup>3</sup>	16.44		
Sample Diameter, cm	6.36			Dry Unit Weight, kN/m <sup>3</sup>	10.03		
Area, cm <sup>2</sup>	31.72			Specific Gravity, measured	2.77		
Volume, cm <sup>3</sup>	69.02			Solids Height, cm	0.803		
Water Content, %	64.00			Volume of Solids, cm <sup>3</sup>	25.47		
Wet Mass, g	115.72			Volume of Voids, cm <sup>3</sup>	43.55		
Dry Mass, g	70.56						
Prepared By: LH		WSP Canada Inc			Checked By: MM		

# CONSOLIDATION TEST SUMMARY

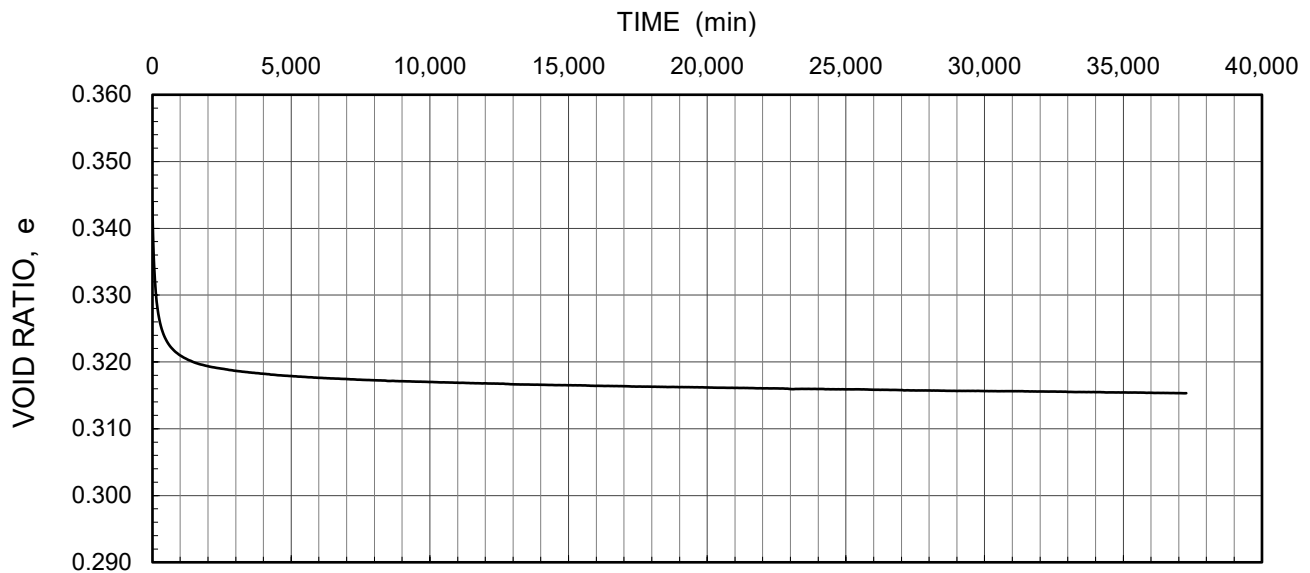
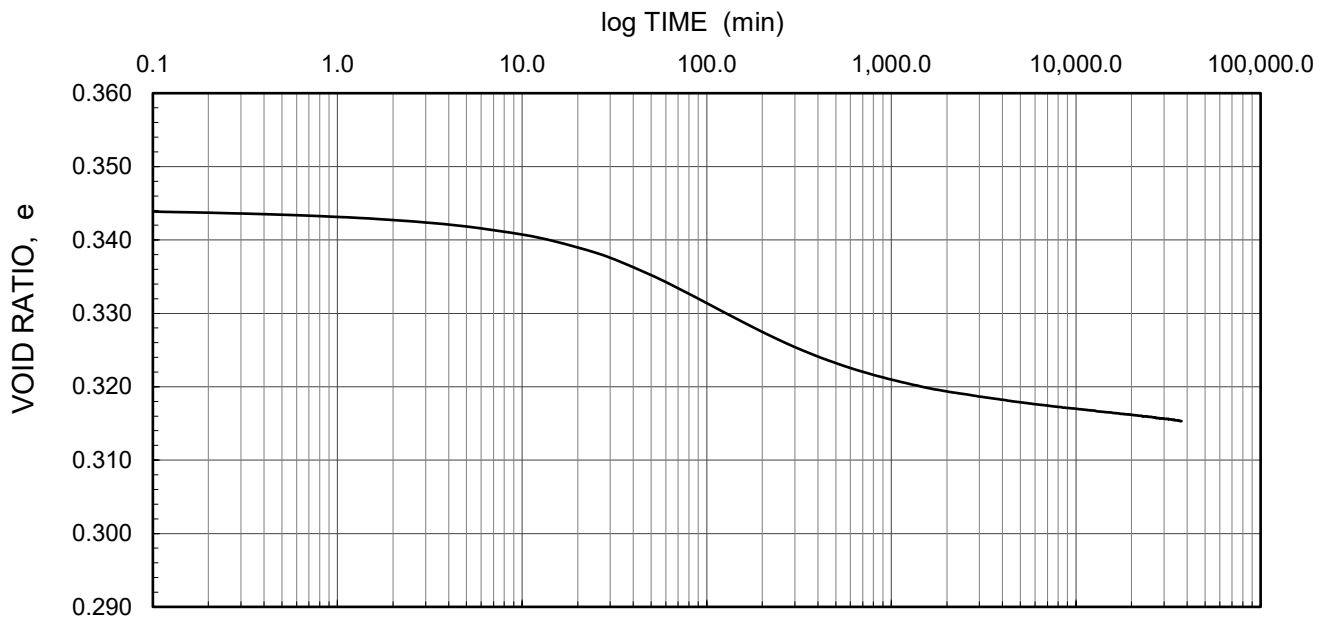
FIGURE B29







# PRESSURE = 210 kPa



## LEGEND

Borehole: 22-01B	$w_i = 77\%$	$S_o = 100\%$
Sample: 5	$w_f = 64\%$	$C_\alpha = 0.20$
Depth (m): 7.01	$w_l = 69\%$	
Elevation (m): 45.87	$w_p = 24\%$	

**wsp** GOLDER

SCALE	AS SHOWN
DATE	12-May-23
DESIGN	KG
CHECK	KCP
REVIEW	JPD

## SUMMARY OF SECONDARY COMPRESSION TEST

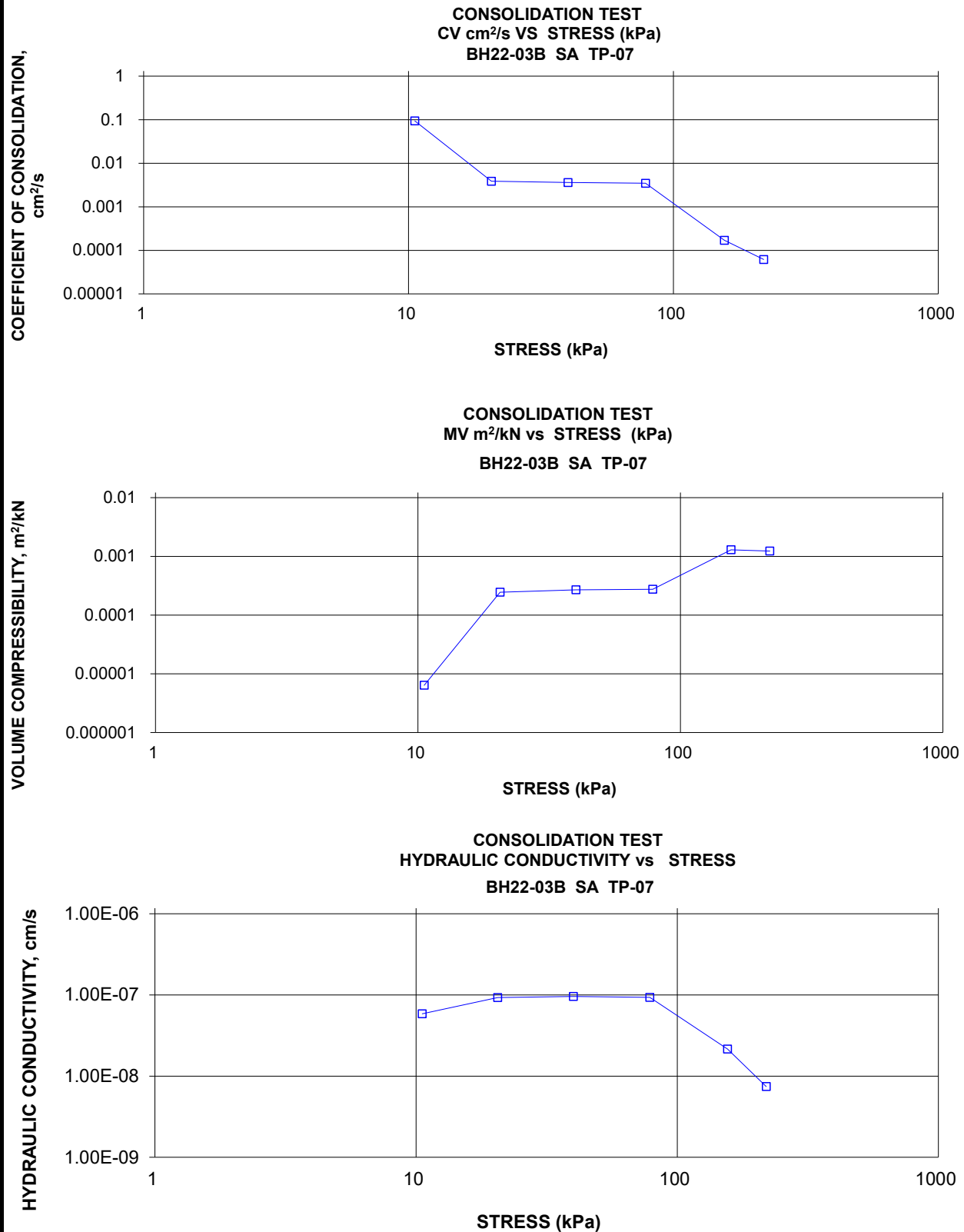
PROJECT No. 22513877A REV. 0

FIGURE B29

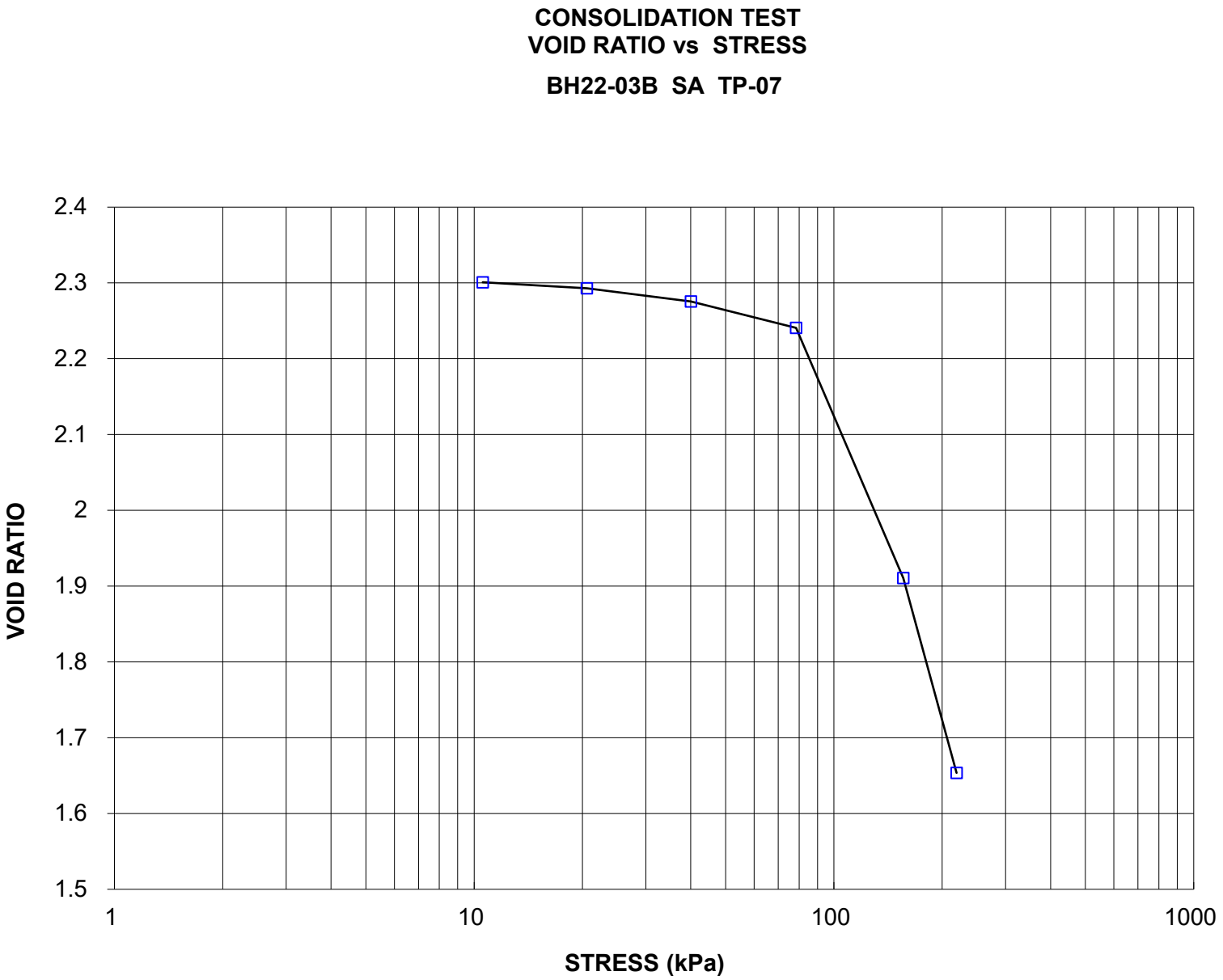
CONSOLIDATION TEST SUMMARY ASTM D2435/D2435M					FIGURE B30			
SAMPLE IDENTIFICATION								
Project Number		22513877A			Sample Number		TP-07	
Borehole Number		BH22-03B			Sample Depth, m		8.54-9.05	
TEST CONDITIONS								
Test Type		Laboratory Standard			Load Duration, hr		-	
Oedometer Number		7						
Date Started		02/22/2023						
Date Completed		04/03/2023						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		1.88			Unit Weight, kN/m <sup>3</sup>		15.09	
Sample Diameter, cm		6.35			Dry Unit Weight, kN/m <sup>3</sup>		8.26	
Area, cm <sup>2</sup>		31.70			Specific Gravity, measured		2.78	
Volume, cm <sup>3</sup>		59.66			Solids Height, cm		0.570	
Water Content, %		82.66			Volume of Solids, cm <sup>3</sup>		18.07	
Wet Mass, g		91.77			Volume of Voids, cm <sup>3</sup>		41.59	
Dry Mass, g,		50.24			Degree of Saturation, %		99.9	
TEST COMPUTATIONS								
Stress	Corr. Height	Void Ratio	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	1.882	2.301	1.882					
10.56	1.882	2.301	1.882	8	9.39E-02	6.37E-06	5.86E-08	
20.55	1.877	2.293	1.880	194	3.86E-03	2.45E-04	9.26E-08	
40.01	1.867	2.276	1.872	205	3.63E-03	2.69E-04	9.57E-08	
78.49	1.847	2.241	1.857	211	3.47E-03	2.75E-04	9.34E-08	
155.83	1.659	1.911	1.753	3840	1.70E-04	1.29E-03	2.15E-08	
219.17	1.513	1.654	1.586	8640	6.17E-05	1.23E-03	7.43E-09	
<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 to 27cm from top of the tube.</p> <p>Creep test performed on 219kPa for 34 days.</p>								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		1.51			Unit Weight, kN/m <sup>3</sup>		16.81	
Sample Diameter, cm		6.35			Dry Unit Weight, kN/m <sup>3</sup>		10.27	
Area, cm <sup>2</sup>		31.70			Specific Gravity, measured		2.78	
Volume, cm <sup>3</sup>		47.96			Solids Height, cm		0.570	
Water Content, %		63.59			Volume of Solids, cm <sup>3</sup>		18.07	
Wet Mass, g		82.19			Volume of Voids, cm <sup>3</sup>		29.88	
Dry Mass, g		50.24						
Prepared By: LH		WSP Canada Inc				Checked By: MM		

# CONSOLIDATION TEST SUMMARY

FIGURE B30

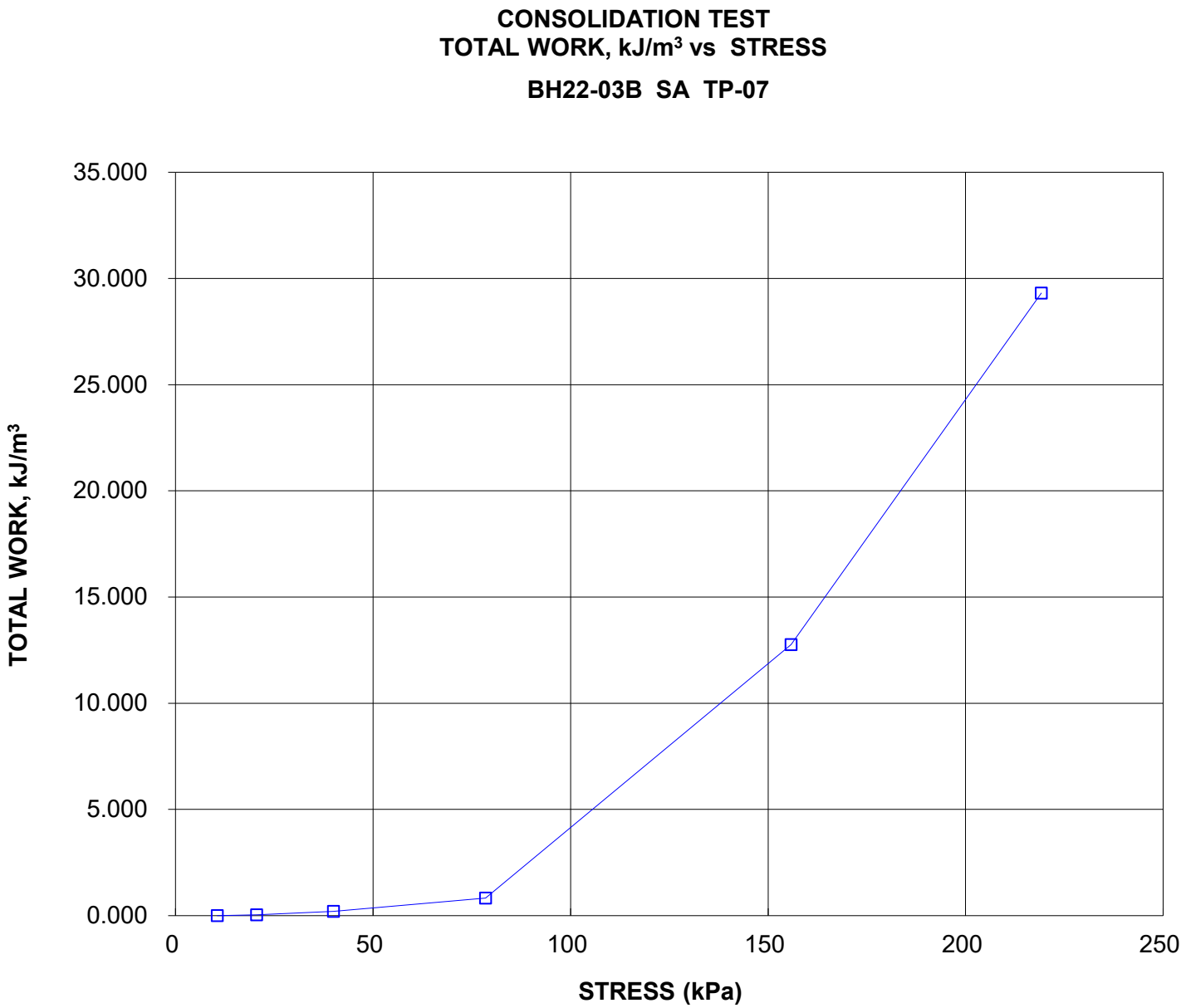






**CONSOLIDATION TEST  
TOTAL WORK VS STRESS**

**FIGURE B30**

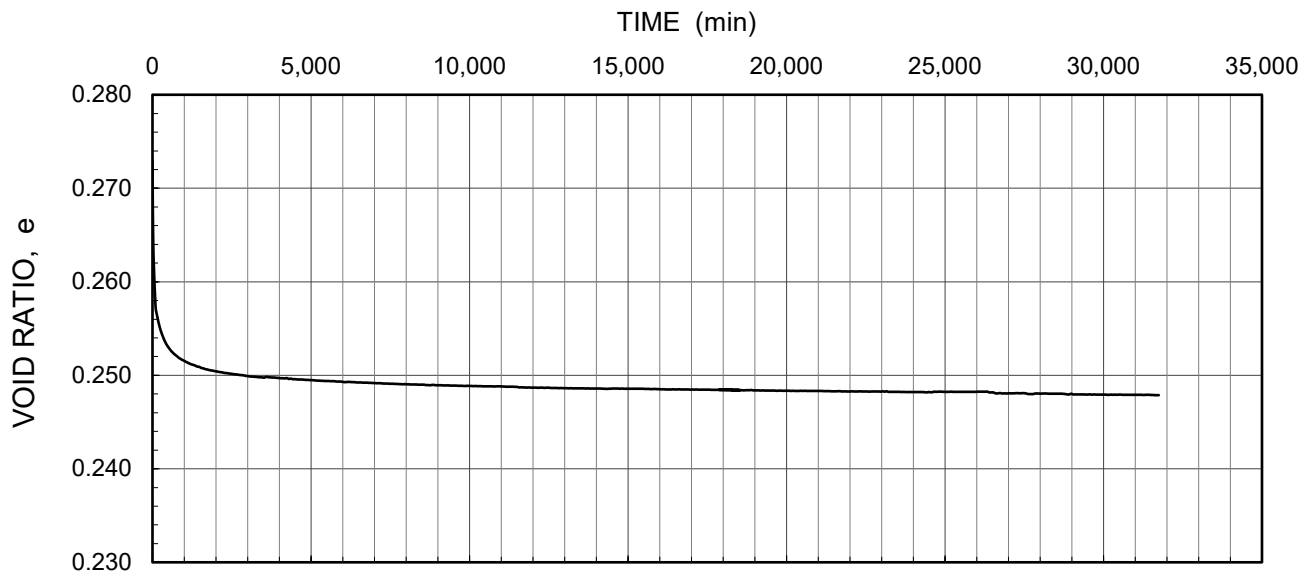
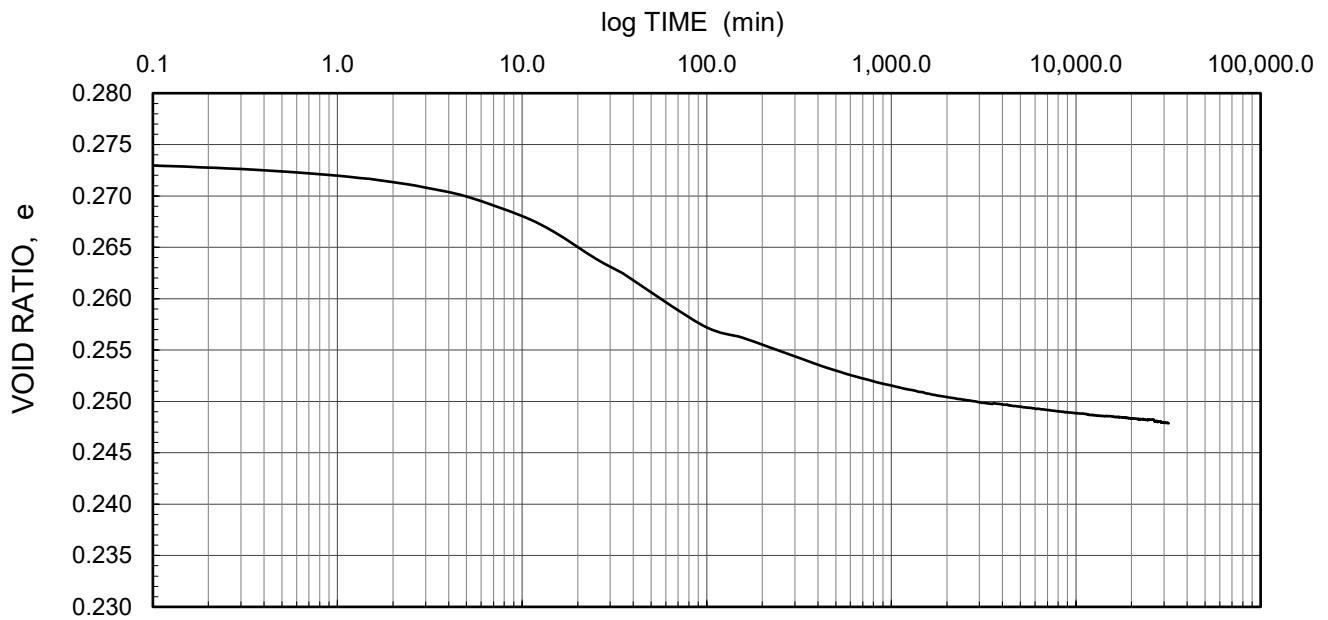


Project No. 22513877A  
Prepared By: LH

**Golder Associates**

Checked By: MM

# PRESSURE = 219 kPa



## LEGEND

Borehole: 22-03B	$w_i = 83\%$	$S_o = 100\%$
Sample: 7	$w_f = 64\%$	$C_\alpha = 0.12$
Depth (m): 8.54	$w_l = 85\%$	
Elevation (m): 44.20	$w_p = 26\%$	

**wsp** GOLDER

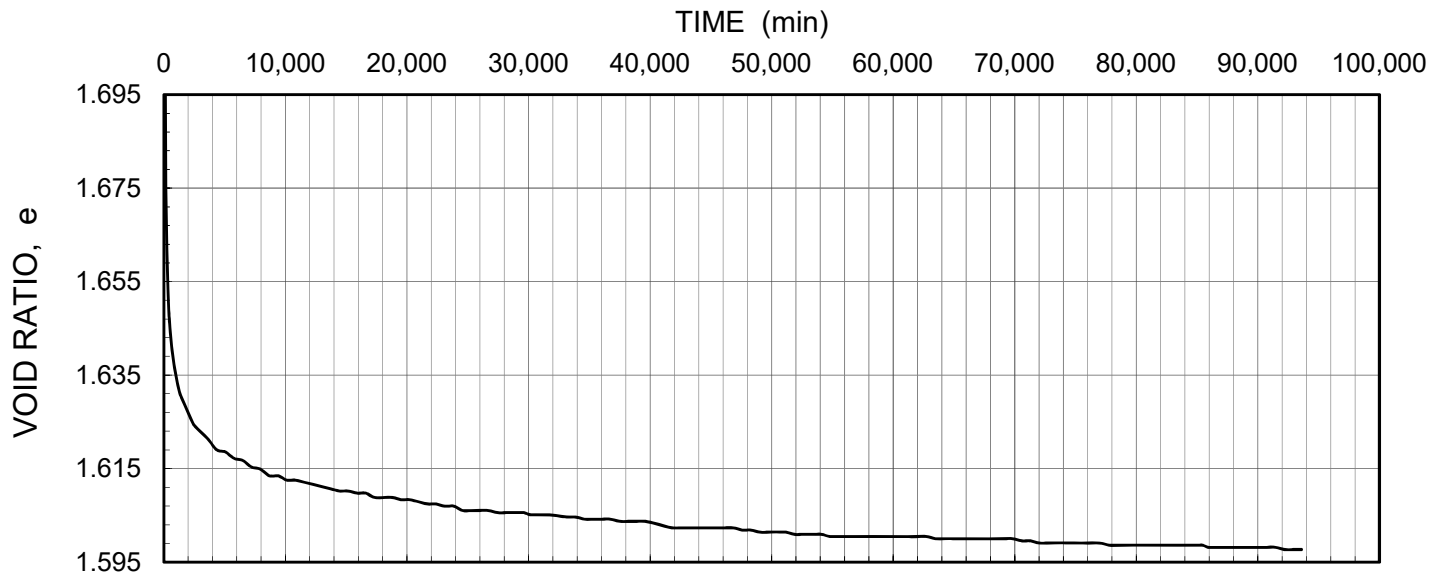
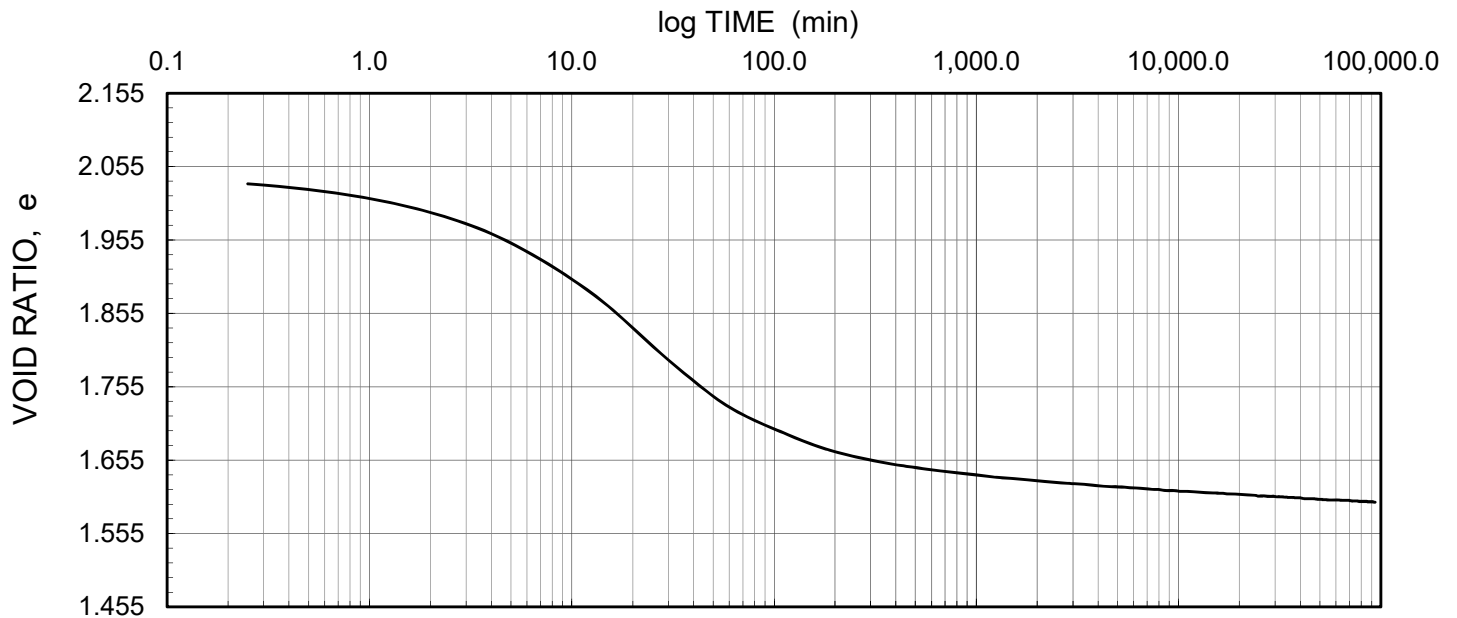
SCALE	AS SHOWN
DATE	12-May-23
DESIGN	KG
CHECK	KCP
REVIEW	JPD

## SUMMARY OF SECONDARY COMPRESSION TEST

PROJECT No. 22513877A REV. 0

FIGURE B30

# **PRESSURE = 245 kPa**



## **LEGEND**

Borehole: 18-1101	$w_i = 83\%$	$S_o = 100\%$
Sample: 10	$w_f = 61\%$	$C_\alpha = 0.027$
Depth (m): 7.9	$w_l = 78\%$	
Elevation (m): 44.9	$w_p = 27\%$	

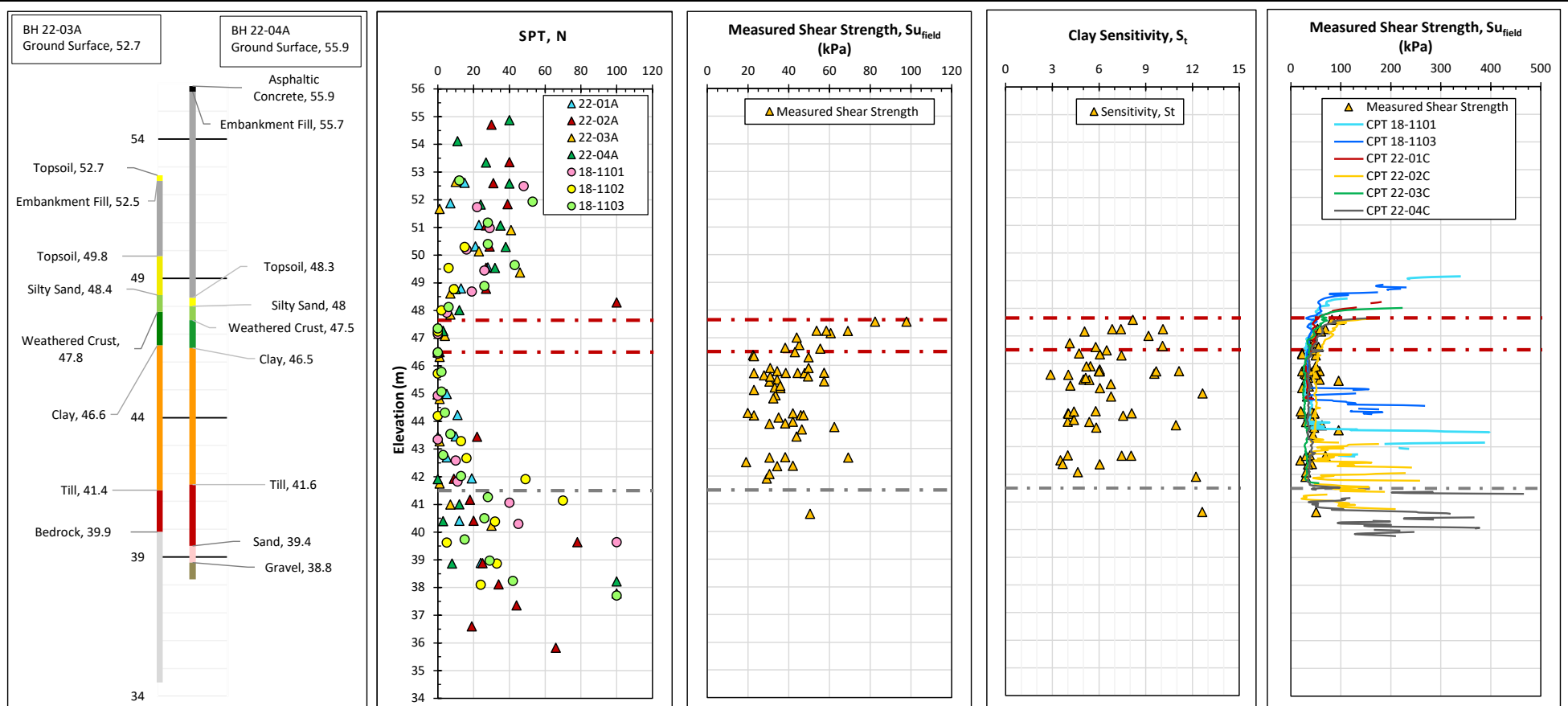


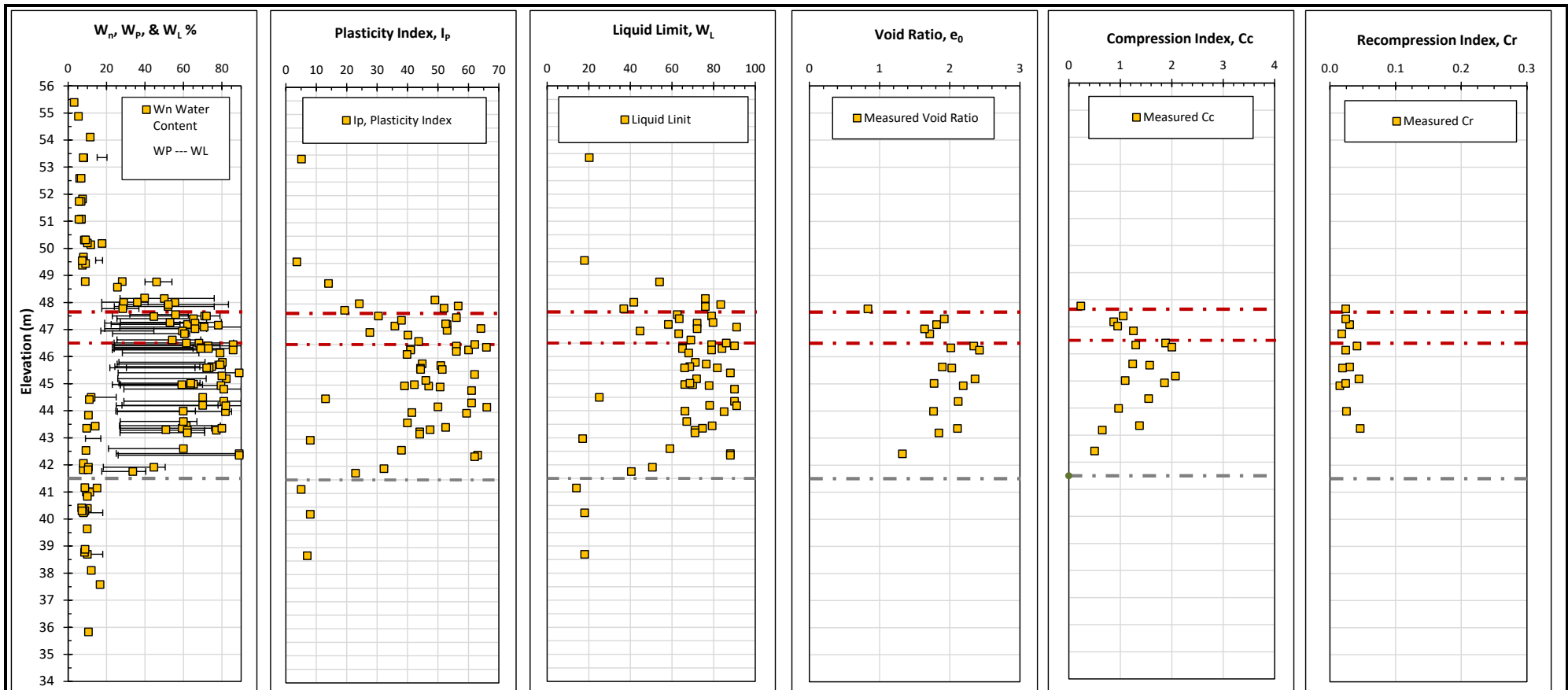
SCALE	AS SHOWN
DATE	25-Feb-19
DESIGN	MI
CHECK	CW
REVIEW	CK

## **SUMMARY OF SECONDARY COMPRESSION TEST**

PROJECT No. 1899802/1100 REV. 0

FIGURE **B31**

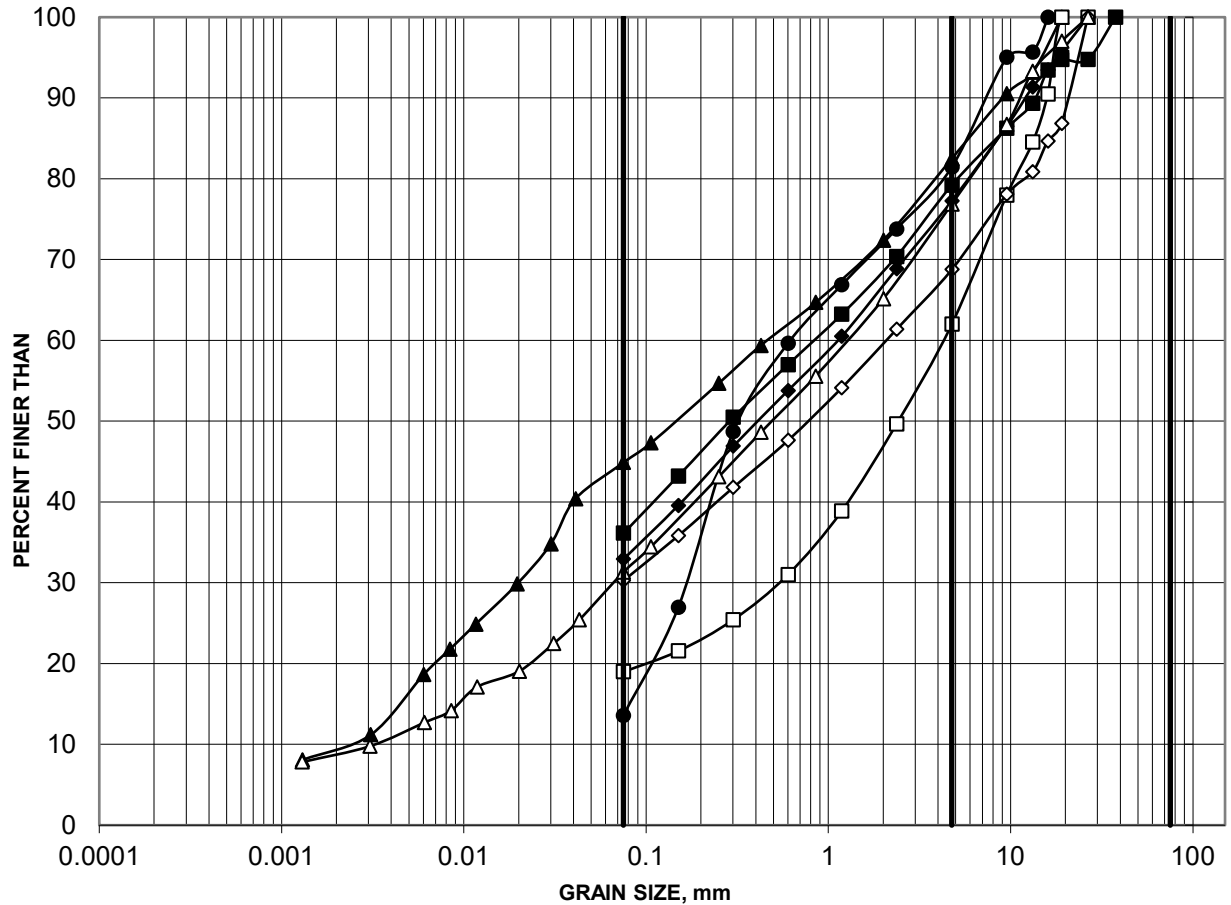




# GRAIN SIZE DISTRIBUTION

FIGURE B34

## Gravelly SILTY SAND (SM) to SILTY SAND (SM/GM) and gravel (TILL)



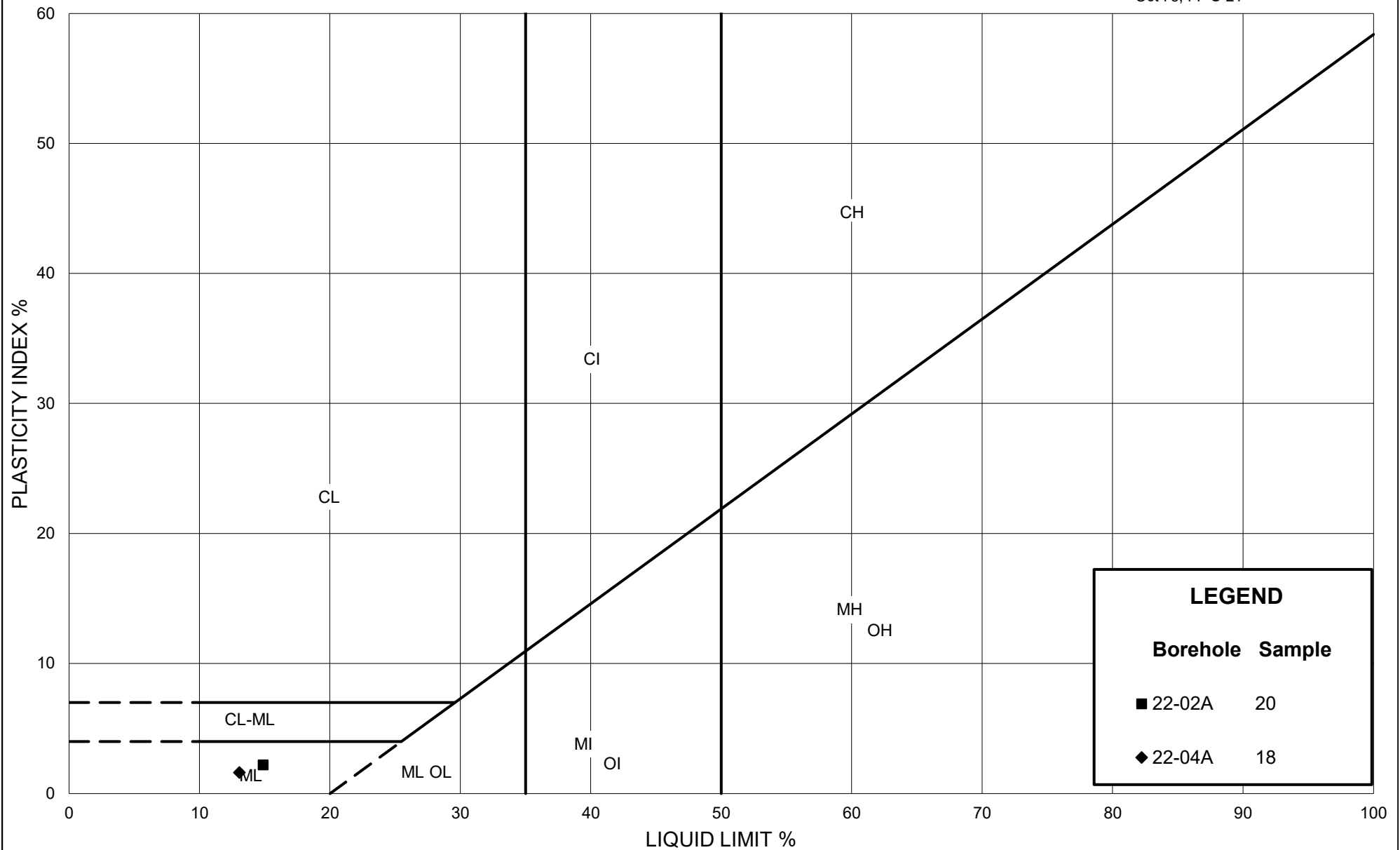
	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	22-01A	11/12	8.38-9.75	21	43		36
◆	22-02A	16	13.11-13.72	23	44		33
▲	22-02A	20	16.00-16.16	18	37	35	10
●	22-02A	23A	18.29-18.44	19	67		14
□	22-02A	25	19.81-20.42	38	43		19
◇	22-03A	13	11.43-12.04	31	39		30
△	22-04A	18	15.24-15.85	23	46	22	9

Project: 22513877A

**wsp** GOLDER

Created by: MI  
Checked by: KCP

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2022/22513877A/Figures/>



Ontario

Ministry of Transportation

# PLASTICITY CHART

SILTY SAND (SM) to Gravelly SILTY SAND (SM) (TILL) with fines of slight plasticity

Figure: B35

Project: 22513877A

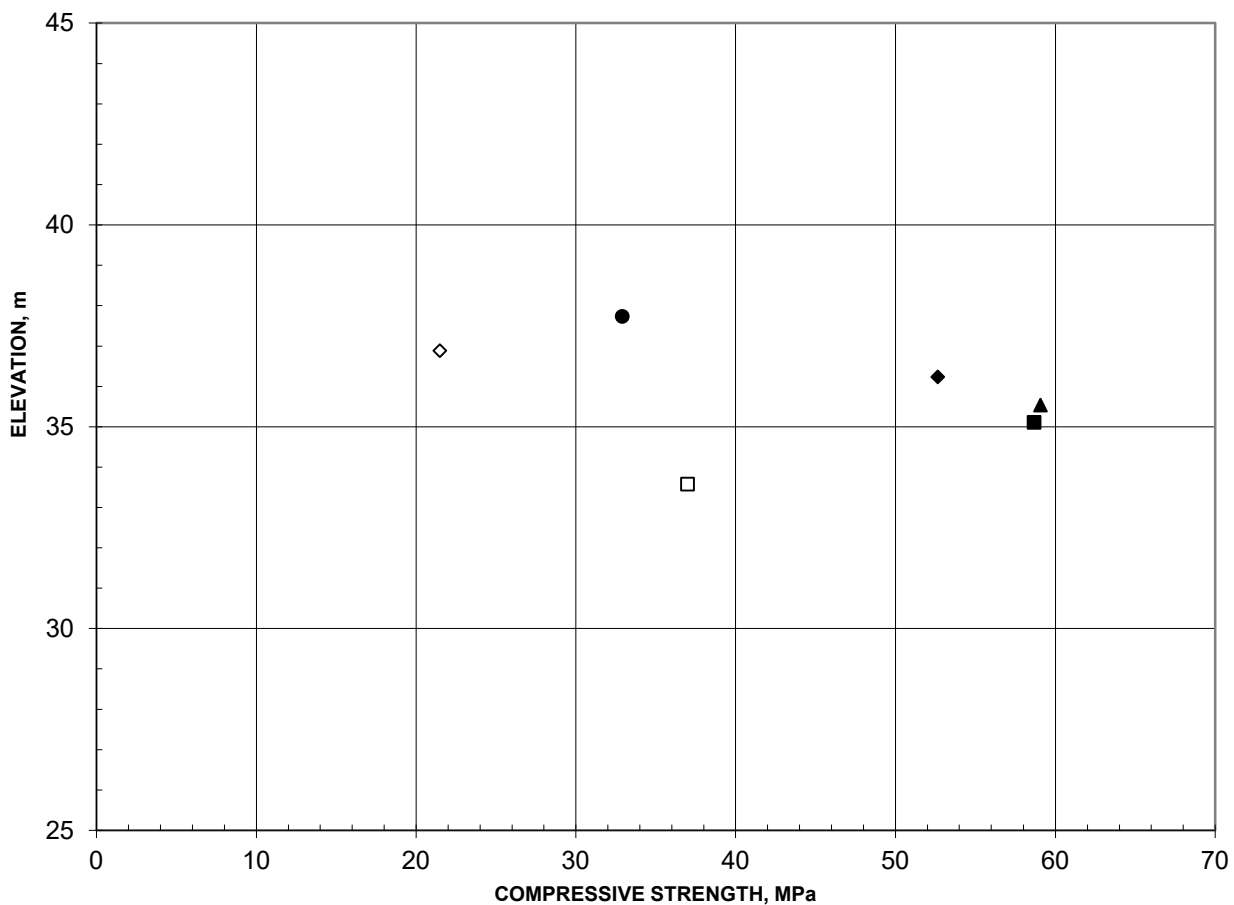
Created By: MI

Checked By: KCP



**ASTM D7012 - Method C**  
**UNIAXIAL COMPRESSIVE STRENGTH OF ROCK CORE**  
**SUMMARY OF LABORATORY TEST RESULTS**

**FIGURE 36**



	Borehole	Depth (m)	L/D	Bulk Density (kg/m <sup>3</sup> )	Lithology	UCS (MPa)	Failure Type
■	BH22-01A RC1	17.8	2.4	2650	Limestone	59	1
◆	BH22-03A RC1	16.5	2.4	2629	Limestone	53	1
▲	BH22-03A RC2	17.2	2.5	2662	Limestone	59	1
●	18-1101 RC1	15.1	2.3	2660	Limestone	33	1
□	18-1102 RC1	17.0	2.4	2688	Limestone	37	1
◇	18-1103 RC1	16.1	2.3	2654	Limestone	22	1

**Notes:**

**Failure Types**

1. Well formed cones on both ends
2. Well formed cones on one end, vertical cracks through cap
3. Columnar vertical cracking through both ends
4. Diagonal fracture with no cracking through ends
5. Side fractures at top or bottom
6. Side fractures at both sides of top or bottom

**Remarks**

- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Time to failure > 2 and < 15 minutes.

**wsp**

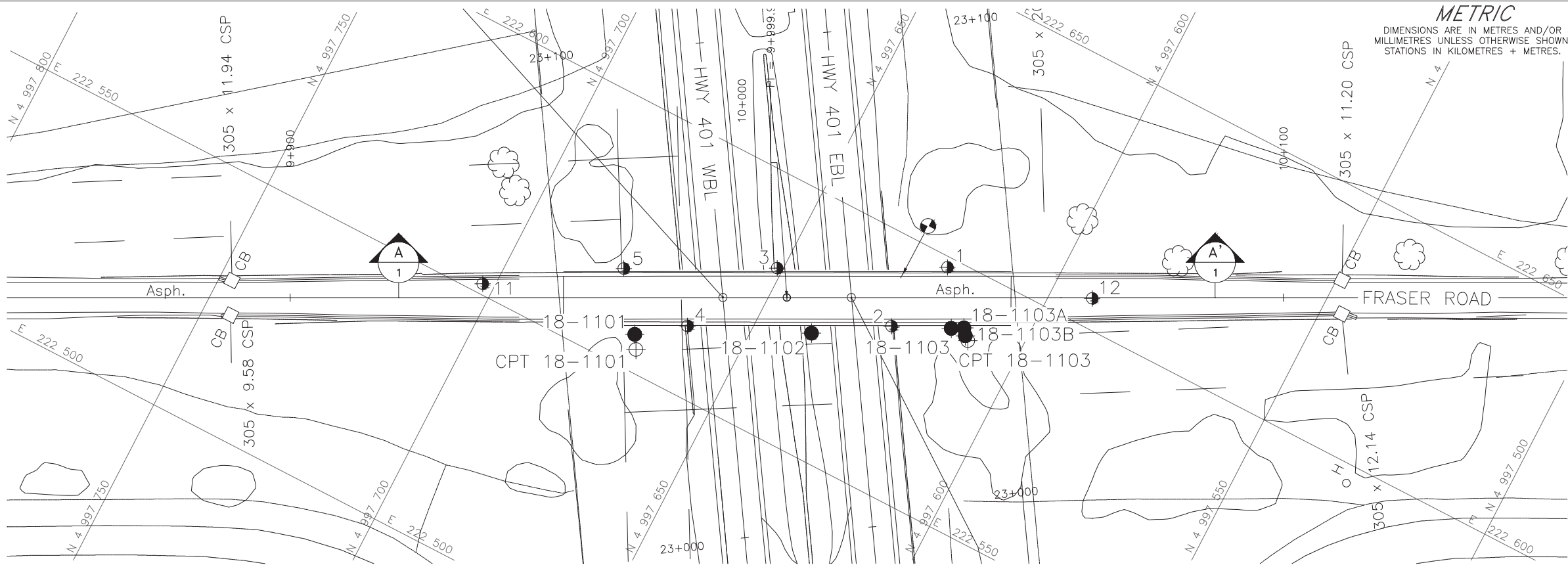
Project: 22513877A

Created by: CW

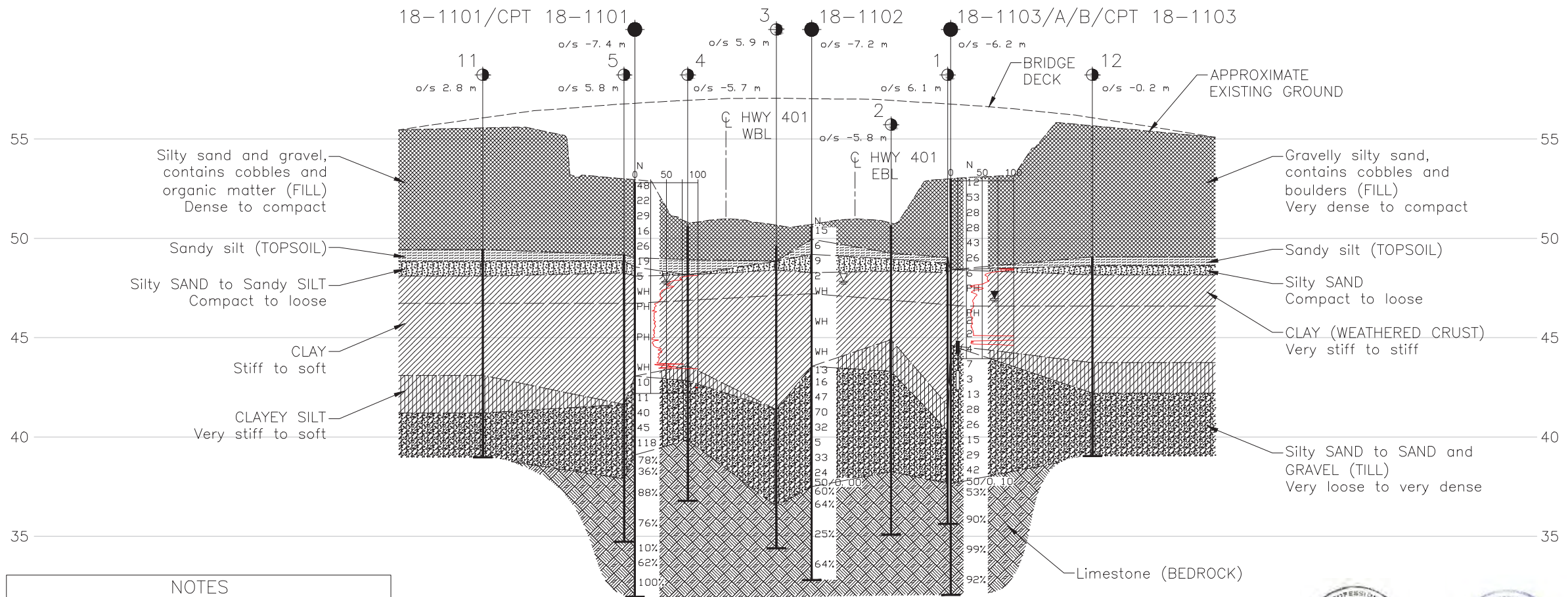
Checked by: MI

**APPENDIX C**

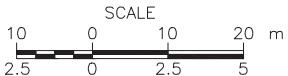
**Record of Boreholes and Laboratory  
Test Results - Previous Investigation**



PLAN



PROFILE A-A'



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.

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

CONT No.  
GWP No. 4248-15-00

FRASER ROAD UNDERPASS  
HIGHWAY 401

BOREHOLE LOCATIONS AND SOIL STRATA  
LAT. 45.114025, LONG. -74.544989



SHEET



KEY PLAN  
SCALE



LEGEND

- Borehole - Current Investigation
- Cone Penetration Test - Current Investigation
- Borehole - Previous Investigation  
Geocres No. 31G00-142
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated  
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on SEPT. 18, 2018
- WL upon completion of drilling
- CPTU Results

BOREHOLE CO-ORDINATES (MTM ZONE 8)

No.	ELEVATION	NORTHING	EASTING
18-1101	52.8	4997670.5	222557.1
18-1102	50.6	4997639.0	222573.6
18-1103	53.0	4997614.4	222587.4
18-1103A	53.1	4997612.2	222588.7
18-1103B	52.9	4997611.2	222587.4
CPT 18-1101	52.8	4997668.9	222554.5
CPT 18-1103	52.9	4997610.3	222586.8
1	49.0	4997620.7	222598.0
2	50.6	4997625.3	222582.3
3	49.6	4997651.1	222582.1
4	50.6	4997661.8	222563.5
5	49.2	4997678.5	222567.8
11	49.4	4997702.4	222552.0
12	49.1	4997592.0	222605.8

REFERENCE

Base plans provided in digital format by Dillon, drawing file no. WP 4328-11-01 - Hwy 401 Charlottenburgh, received JULY 04, 2018.

NO.	DATE	BY	REVISION
Geocres No. 31G5-273			
HWY. 401		PROJECT NO. 1899802-1100	
SUBM'D. CK		DATE: 11/29/2018	
DRAWN: JM		SITE: 31-230	
CHKD. WC		APPD. FJH	
		DWG. 1	



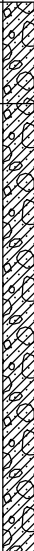

PROJECT 1899802-1100		<b>RECORD OF BOREHOLE No 18-1101</b>		SHEET 1 OF 4		<b>METRIC</b>	
G.W.P. 4248-15-00		LOCATION N 4997670.5; E 222557.1 NAD MTM ZONE 8 (LAT. 45.114230; LONG. -74.545260)		ORIGINATED BY RI			
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ3 Core		COMPILED BY ZS			
DATUM Geodetic		DATE September 4-5, 2018		CHECKED BY CK			

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 (%)  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>				
						20   40   60   80   100					25   50   75						
52.8	GROUND SURFACE					▽											
0.0	(SM) Silty sand, contains organic matter (rootlets) (TOPSOIL/FILL)		1	SS	48												
0.1	Dark brown Moist																
	(SM/GM) Silty sand and gravel, contains cobbles and organic matter (FILL)			2	SS		22										
	Dense to compact																
	Brown Moist																
			3	SS	29												
			4	SS	16												
			5	SS	26												
49.0																	
3.8	(ML) Sandy silt, contains organic matter and silty sand seams (wood/rootlets) (TOPSOIL)		6	SS	19												
48.5	Dark brown to black Moist																
4.3	(SM) Silty SAND, fine, contains organic matter (rootlets/wood)																
48.1	Compact to loose Grey Moist			7	SS	5											
4.7	(CH) CLAY, trace sand, highly fissured, contains silt and sand seams (WEATHERED CRUST)																
	Very stiff to stiff Grey-brown Moist			8	SS	WH											
46.7																	
6.1	(CH) CLAY Firm Grey Moist		9	TP	PH												
				10	TP	PH											
				11	SS	WH											
43.0																	
9.8																	

Continued Next Page

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


GTA-MTO 001 N:\ACTIVE\SPATIAL\_IM\IMTO\HWY401\FRASERROAD\02\_DATA\GINT\1899802.GPJ GAL-GTA.GDT 19-5-23 ZS

PROJECT		1899802-1100		<b>RECORD OF BOREHOLE No 18-1101</b>		SHEET 2 OF 4		<b>METRIC</b>							
G.W.P.		4248-15-00		LOCATION		N 4997670.5; E 222557.1 NAD MTM ZONE 8 (LAT. 45.114230; LONG. -74.545260)		ORIGINATED BY							
DIST		Eastern HWY 401		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ3 Core		COMPILED BY							
DATUM		Geodetic		DATE		September 4-5, 2018		CHECKED BY							
CK															
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		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			20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100		W <sub>p</sub>	W	W <sub>L</sub>	γ	GR SA SI CL	
42.1	(SM) Gravelly Silty SAND (TILL) Loose to compact Grey Wet		12	SS	10										
10.7	(SP/GP) SAND and GRAVEL, some silt, contains cobbles and boulders (TILL) Compact to very dense Grey to dark grey Wet		13	SS	11										33 43 19 5
			14	SS	40										
			15	SS	45										
			16	SS	118										
39.1	Limestone (BEDROCK)		1	RC	REC 79%										RQD = 78%
13.7	Bedrock cored from depths 13.7 m to 20.9 m  For bedrock coring detail refer to Record of Drillhole 18-1101		2	RC	REC 81%										RQD = 36%
			3	RC	REC 100%										RQD = 88%
			4	RC	REC 98%										RQD = 76%
			5	RC	REC 51%										RQD = 10%
			6	RC	REC 90%										RQD = 62%
			7	RC	REC 100%										RQD = 100%

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL\_IMMITO\HWY401\FRASERROAD\02\_DATA\GINT\1899802.GPJ GAL-GTA.GDT 19-5-23 ZS

PROJECT		RECORD OF BOREHOLE No 18-1101				SHEET 3 OF 4		METRIC									
G.W.P. 1899802-1100		LOCATION N 4997670.5; E 222557.1 NAD MTM ZONE 8 (LAT. 45.114230; LONG. -74.545260)				ORIGINATED BY RI											
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ3 Core				COMPILED BY ZS											
DATUM Geodetic		DATE September 4-5, 2018				CHECKED BY CK											
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						
31.9	Limestone (BEDROCK)  Bedrock cored from depths 13.7 m to 20.9 m  For bedrock coring detail refer to Record of Drillhole 18-1101		7	RC	REC 100%												RQD = 100%
20.9	END OF BOREHOLE  NOTES:  1. Water level in open borehole at a depth of 5.0 m below ground surface (Elev. 47.8), measured during drilling.  2. PVC pipe for VSP installed within borehole following drilling.																

GTA-MTO 001 N:\ACTIVE\SPATIAL\_IMMITO\HWY401\FRASERROAD\02\_DATA\GINT\1899802.GPJ GAL-GTA.GDT 19-5-23 ZS

SHEET 4 OF 4

DATUM: Geodetic

DRILLING CONTRACTOR: Downing Drilling

CHECKED: CK

GTA-RCK 031 N:\ACTIVE\SPATIAL IMMTO\HWY401\FRASERROAD\02 DATA\GINT\1899802.GPJ GAL-MISS.GDT 19-5-23 ZS



GTA-MTO 001 N:\ACTIVE\SPATIAL IM\MT01HWY401\FRASERROAD\02 DATA\GINT\1899802.GPJ GAL-GTA.GDT 19-5-23 ZS

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



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

SHEET 3 OF 3

DATUM: Geodetic

DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing Drilling

CHECKED: CK

GTA-RCK 031 N:\ACTIVE\SPATIAL IMMTO\HWY401\FRASERROAD\02 DATA\GINT\1899802.GPJ GAL-MISS.GDT 19-5-23 ZS

PROJECT 1899802-1100		<b>RECORD OF BOREHOLE No 18-1103</b>		SHEET 1 OF 4		<b>METRIC</b>	
G.W.P. 4248-15-00		LOCATION N 4997614.4; E 222587.4 NAD MTM ZONE 8 (LAT. 45.113730; LONG. -74.544860)		ORIGINATED BY RI			
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ3 Core		COMPILED BY ZS			
DATUM Geodetic		DATE September 10-11, 2018		CHECKED BY CK			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL LIMIT   MOISTURE   CONTENT   LIQUID 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									
53.0	GROUND SURFACE						20	40	60	80	100						
0.0	(SM) Silty sand, trace gravel, contains organic matter (rootlets) (TOPSOIL/FILL) Dark brown Moist		1	SS	12												
0.2	(SM) Gravelly silty sand, contains cobbles and boulders (FILL) Very dense to compact Grey-brown Moist		2	SS	53												
			3	SS	28												
			4	SS	28												
			5	SS	43												
			6	SS	26												
48.7																	
4.3	(SM) Silty sand, some gravel, contains organic matter (rootlets) (FILL)																
48.4	Compact																
4.6	Brown to grey Moist		7	SS	6												
	(CH) CLAY, trace sand, highly fissured (WEATHERED CRUST) Very stiff to stiff Grey-brown Moist		8	TP	PH												
46.9																	
6.1	(CH) CLAY, trace sand, trace gravel, contains thin laminations of sand Firm to stiff Grey Wet																
			9	TP	PH												
			10	SS	2												
			11	SS	2												
44.6																	
8.4	(SM) Gravelly Silty SAND (TILL) Loose to very loose Grey Wet		12	SS	4												
			13	SS	7												

Continued Next Page

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


GTA-MTO 001 N:\ACTIVE\SPATIAL\_IM\IMTO\HWY401\FRASERROAD\02\_DATA\GINT\1899802.GPJ GAL-GTA GDT 19-5-23 ZS

PROJECT		1899802-1100		RECORD OF BOREHOLE No 18-1103		SHEET 2 OF 4		METRIC																
G.W.P.		4248-15-00		LOCATION		N 4997614.4; E 222587.4 NAD MTM ZONE 8 (LAT. 45.113730; LONG. -74.544860)		ORIGINATED BY																
DIST		Eastern HWY 401		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ3 Core		COMPILED BY																
DATUM		Geodetic		DATE		September 10-11, 2018		CHECKED BY																
CK																								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			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																			
42.3	(SM) Gravelly Silty SAND (TILL) Loose to very loose Grey Wet		14	SS	3																			
10.7	(SP/GP) SAND and GRAVEL Compact Grey Wet		15	SS	13																			
41.7	(GP) Sandy GRAVEL, some silt (TILL) Compact to dense Grey Wet		16	SS	28																			
11.3			17	SS	26																			
			18	SS	15																			
			19	SS	29																			
			20	SS	42																			
37.7	Limestone (BEDROCK)  Bedrock cored from depths 15.3 m to 21.0 m  For bedrock coring detail refer to Record of Drillhole 18-1103		21	SS	50/0.10																			
15.3			1	RC	REC 75%																			
			2	RC	REC 100%																			
			3	RC	REC 100%																			
			4	RC	REC 100%																			

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL\_IM\MTD\HWY401\FRASERROAD\02\_DATA\GINT\1899802.GPJ GAL-GTA.GDT 19-5-23 ZS

PROJECT		RECORD OF BOREHOLE No 18-1103				SHEET 3 OF 4		METRIC								
G.W.P. 4248-15-00		LOCATION N 4997614.4; E 222587.4 NAD MTM ZONE 8 (LAT. 45.113730; LONG. -74.544860)				ORIGINATED BY RI										
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ3 Core				COMPILED BY ZS										
DATUM Geodetic		DATE September 10-11, 2018				CHECKED BY CK										
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					
32.1	Limestone (BEDROCK)		4	RC	REC 100%											RQD = 92%
21.0	Bedrock cored from depths 15.3 m to 21.0 m  For bedrock coring detail refer to Record of Drillhole 18-1103															
	END OF BOREHOLE															
	NOTES:  1. Water level in open borehole at a depth of 5.7 m below ground surface (Elev. 47.3), measured during drilling.  2. Packer testing was carried out in bedrock.  3. PVC pipe for VSP installed within borehole following drilling.															

SHEET 4 OF 4

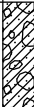

DATUM: Geodetic

DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing Drilling

CHECKED: CK

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

PROJECT		RECORD OF BOREHOLE				No 18-1103A		SHEET 2 OF 2		METRIC						
G.W.P. 4248-15-00		LOCATION				N 4997612.2; E 222588.7 NAD MTM ZONE 8 (LAT. 45.113710; LONG. -74.544840)				ORIGINATED BY PAH						
DIST Eastern HWY 401		BOREHOLE TYPE				Power Auger, 200 mm Diam. (Hollow Stem)				COMPILED BY ZS						
DATUM Geodetic		DATE				September 13, 2018				CHECKED BY CK						
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					
41.8	(SM) Gravelly SILTY SAND, contains cobbles and boulders (TILL) Loose to compact Grey Wet		4	SS	16		43									
11.3	END OF BOREHOLE						42									
NOTES:																
1. Soil stratigraphy from 0.0 to 6.1 m inferred from 18-1103.																
2. Vane refusal was encountered at 6.3 m due possibly to presence of gravel.																
3. Water level in monitoring well at a depth of 6.1 m below ground surface (Elev. 47.0 m), measured on Sept. 18, 2018.																

GTA-MTO 001 N:\ACTIVE\SPATIAL\_IM\IMTO\HWY401\FRASERROAD\02\_DATA\GINT\1899802.GPJ GAL-GTA.GDT 19-5-23 ZS



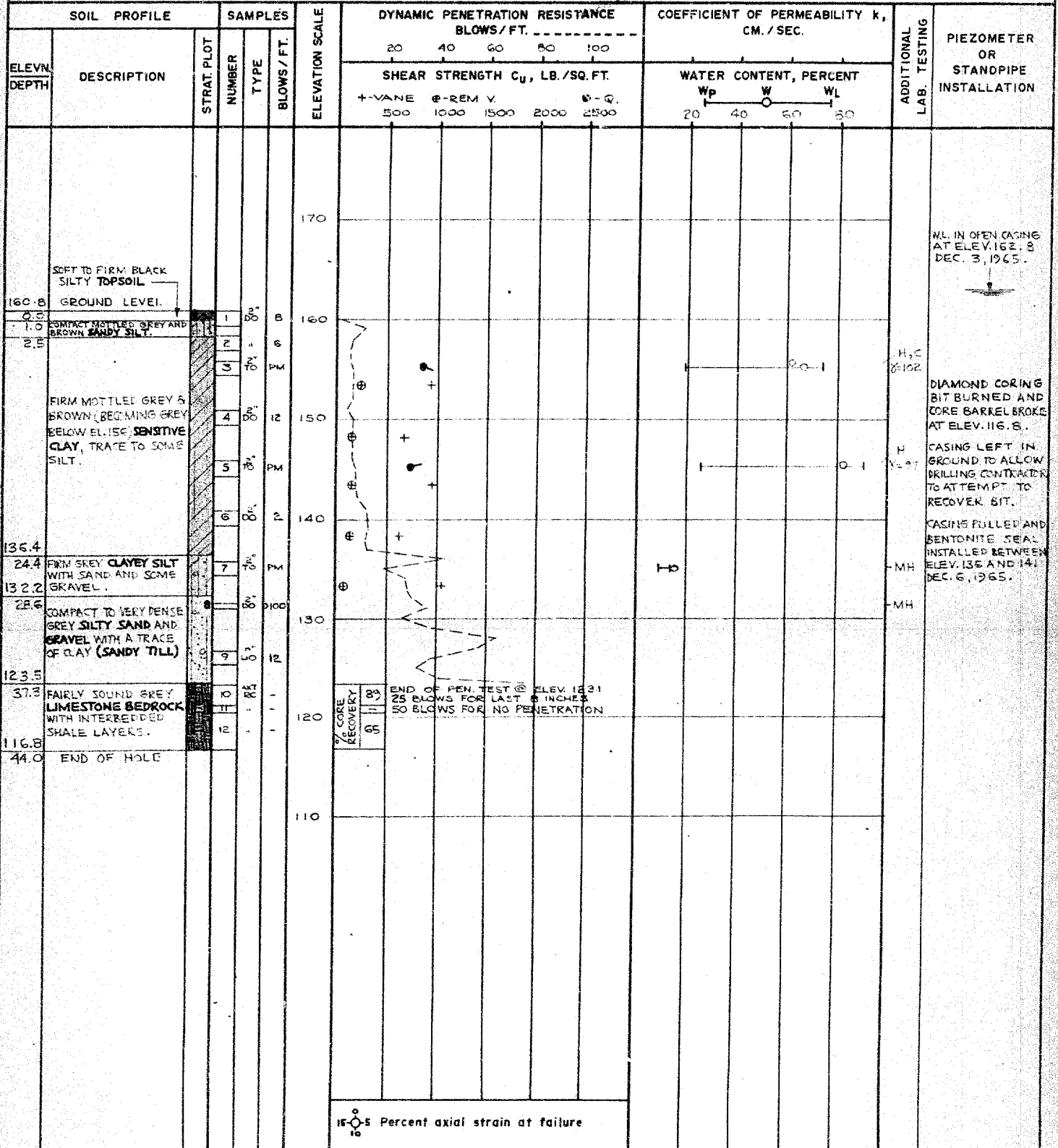
+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

# RECORD OF BOREHOLE 1

LOCATION See Figure 1 BORING DATE NOV 10 - 12, 1965. DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



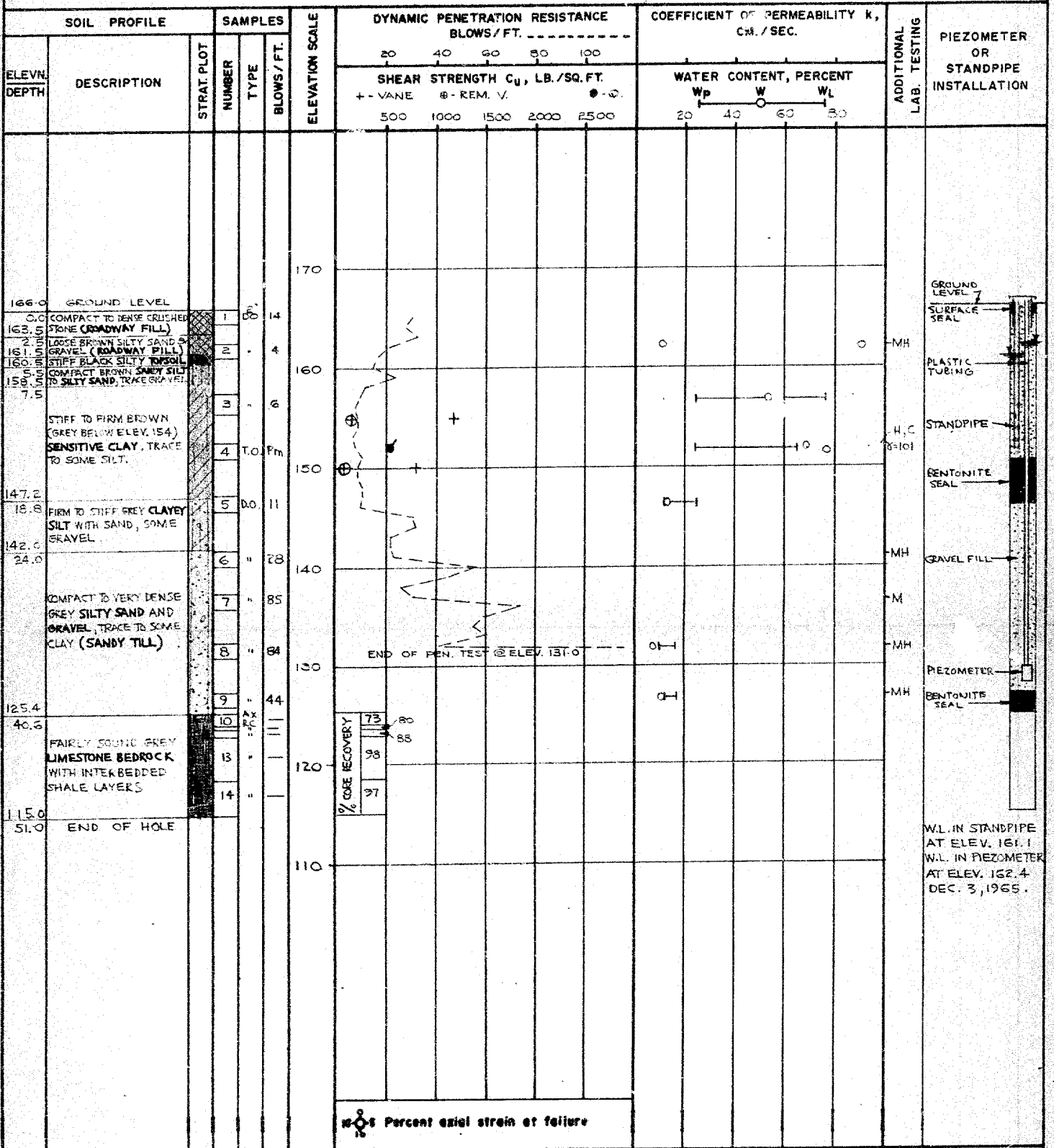
VERTICAL SCALE  
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN R.H. 1/60  
CHECKED JED

# RECORD OF BOREHOLE 2

LOCATION See Figure 1 BORING DATE NOV. 13-19, 1945 DATUM GEODETIC  
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, BX CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE  
 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN R.H. Aldrich  
 CHECKED J.R.D.

# RECORD OF BOREHOLE 3

LOCATION See Figure 1

BORING DATE DEC. 1-2, 1965

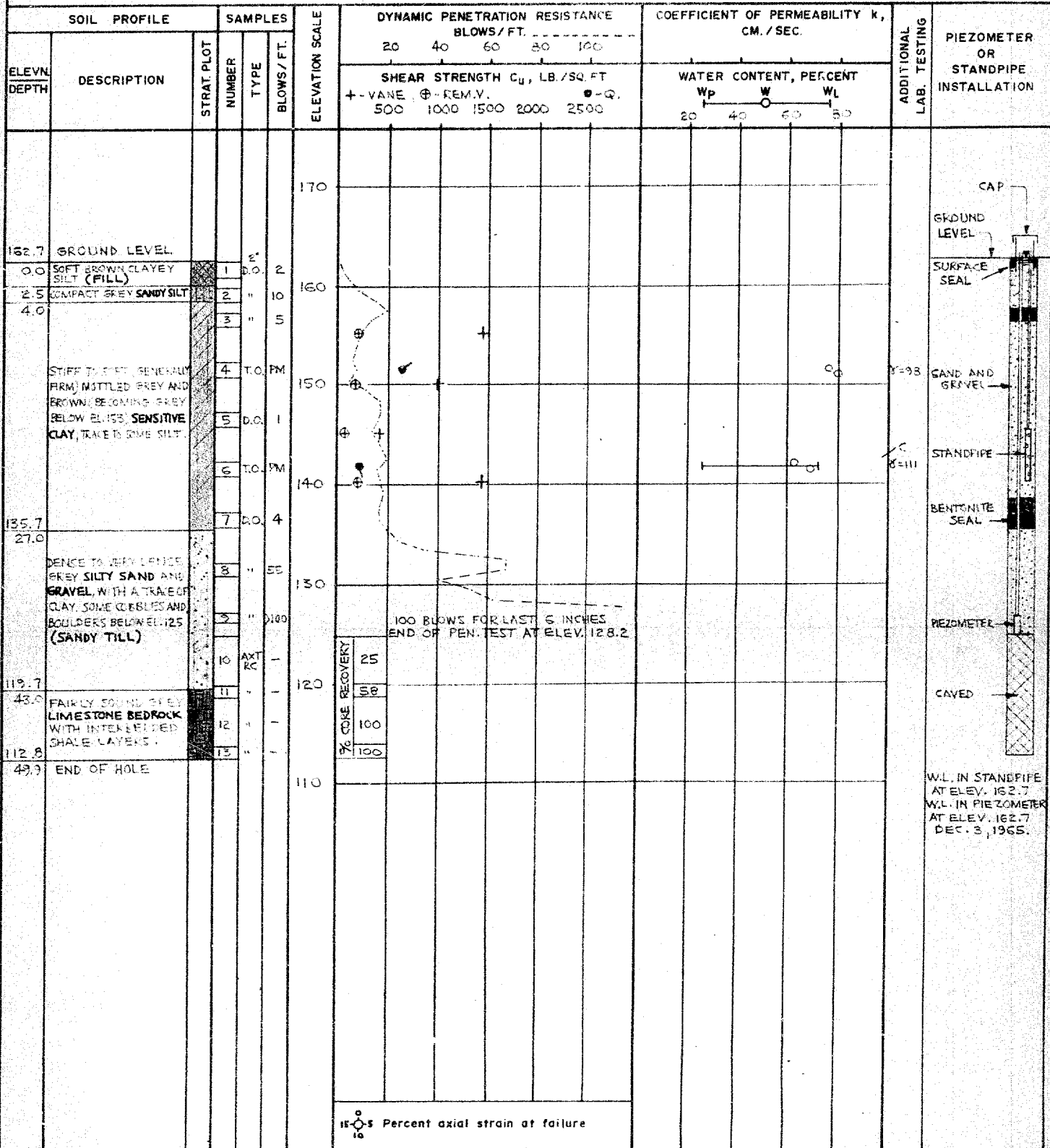
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX, AX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



GOLDER & ASSOCIATES

DRAWN m.w.

CHECKED J.D.

# RECORD OF BOREHOLE 4

LOCATION See Figure 1

BORING DATE NOV. 19-24, 1965

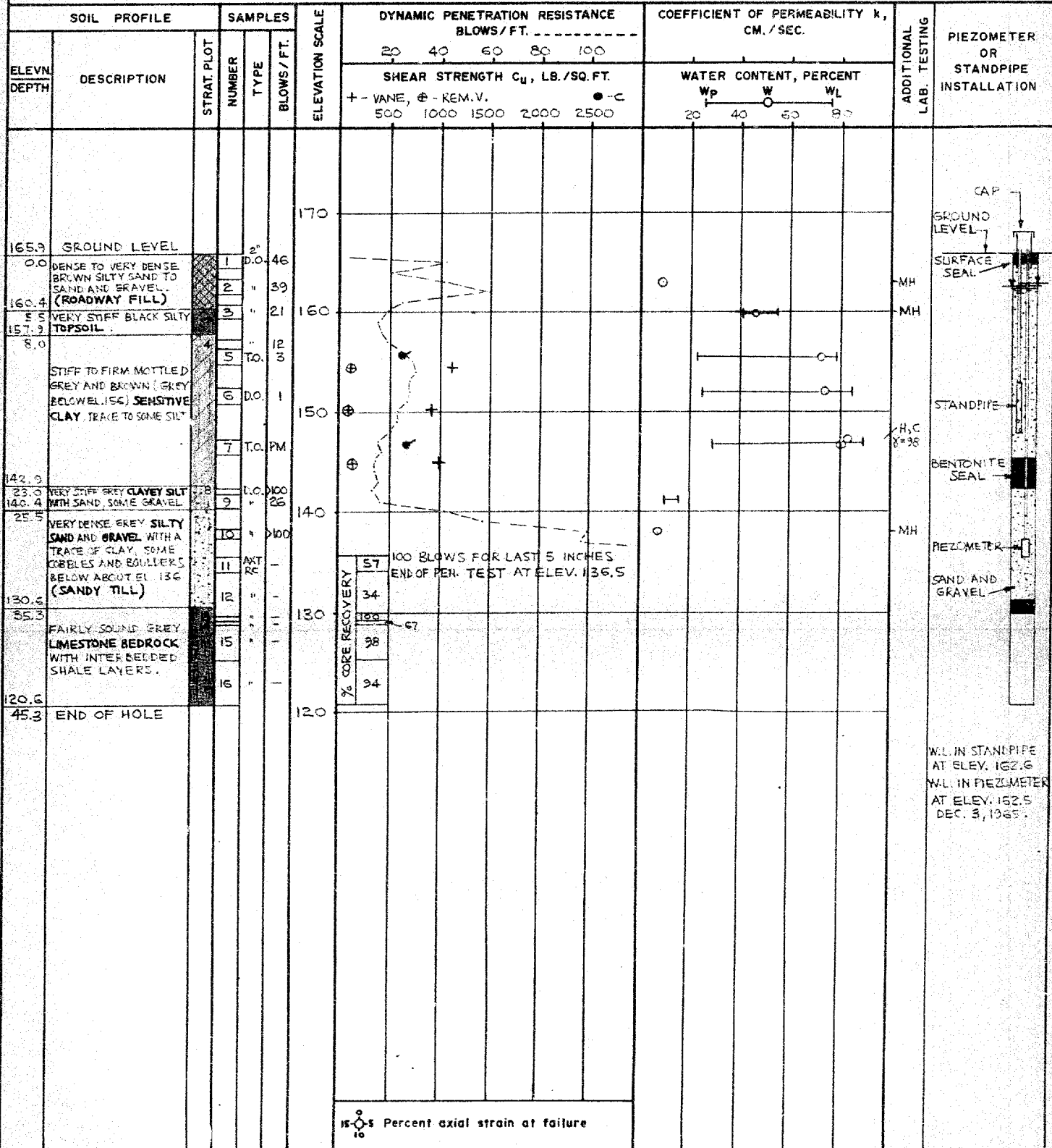
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



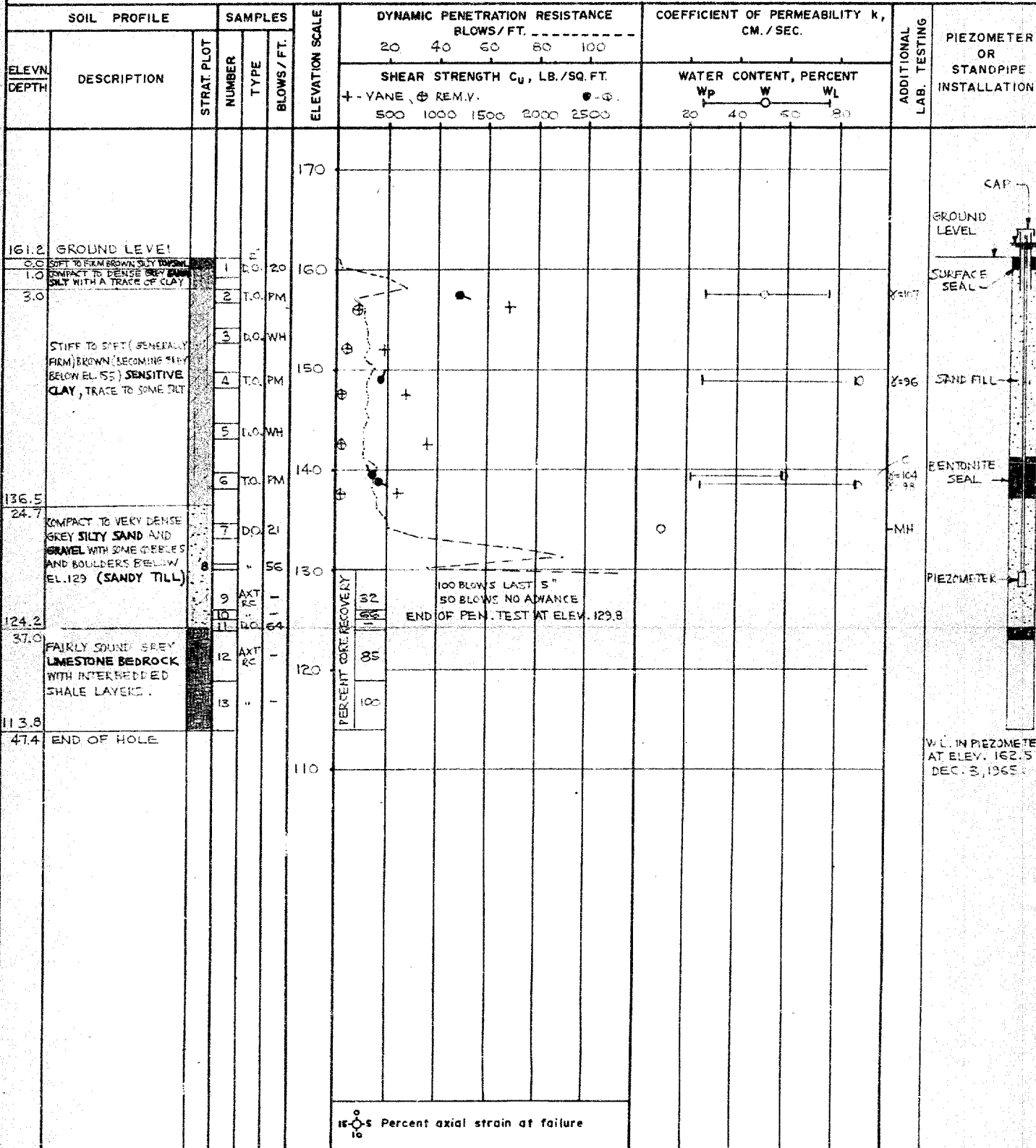
VERTICAL SCALE  
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *MAS*  
CHECKED *MAS*

# RECORD OF BOREHOLE 5

LOCATION See Figure 1 BORING DATE NOV. 25 - 29, 1965 DATUM GEODETIC  
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX-BX CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



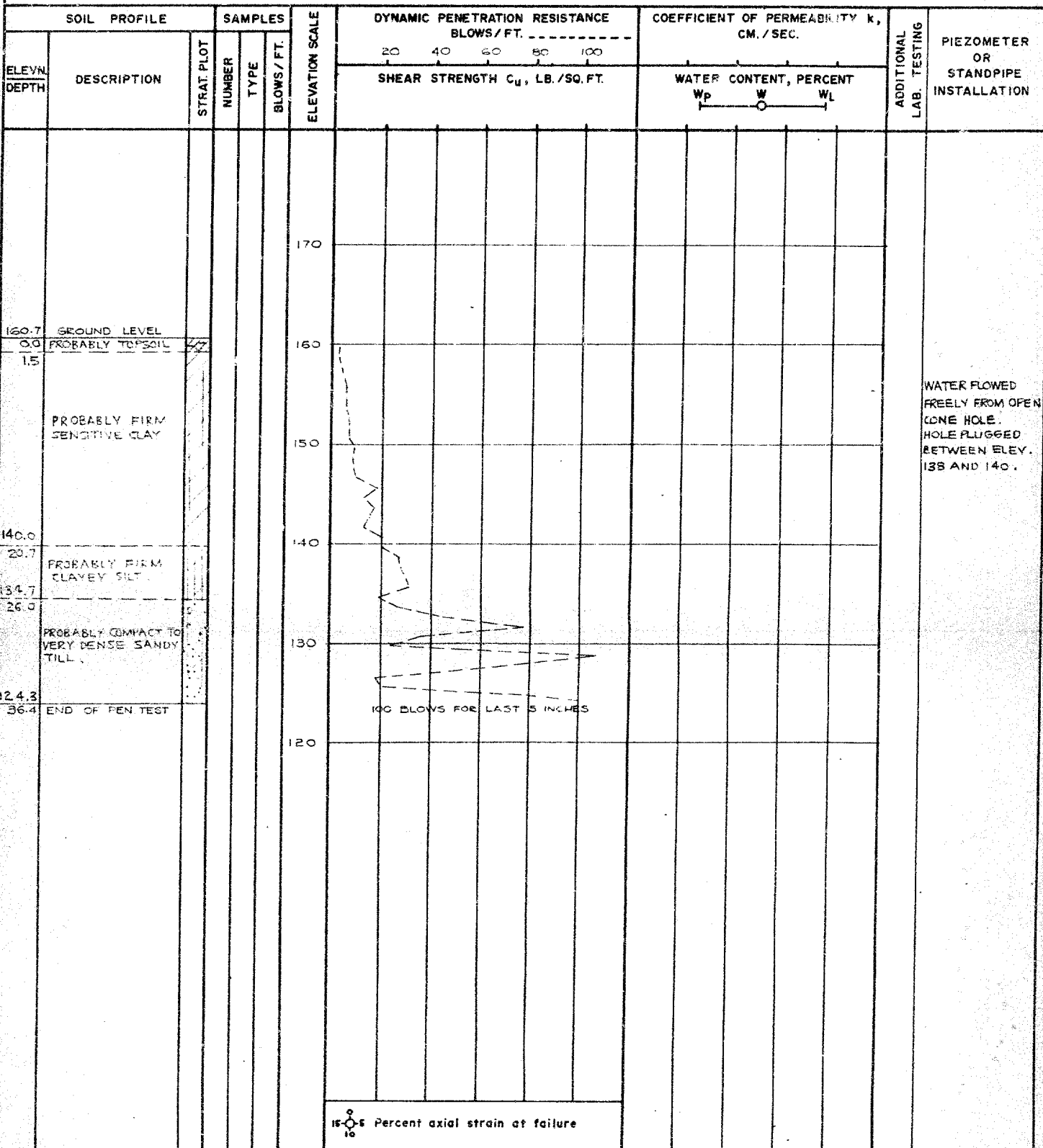
VERTICAL SCALE  
 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *[Signature]*  
 CHECKED *[Signature]*

# PEN TEST RECORD OF BOREHOLE 6

LOCATION **See Figure 1**      BORING DATE **NOV. 3, 1965**      DATUM **GEODETIC**  
 BOREHOLE TYPE **PENETRATION TEST**      BOREHOLE DIAMETER **-**  
 SAMPLER HAMMER WEIGHT **- LB.** DROP **- INCHES**      PEN. TEST HAMMER WEIGHT **140 LB.** DROP **30 INCHES**



VERTICAL SCALE  
 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN RH  
 CHECKED JB





# PEN. TEST RECORD OF BOREHOLE 8

LOCATION See Figure 1

BORING DATE DEC. 1, 1965.

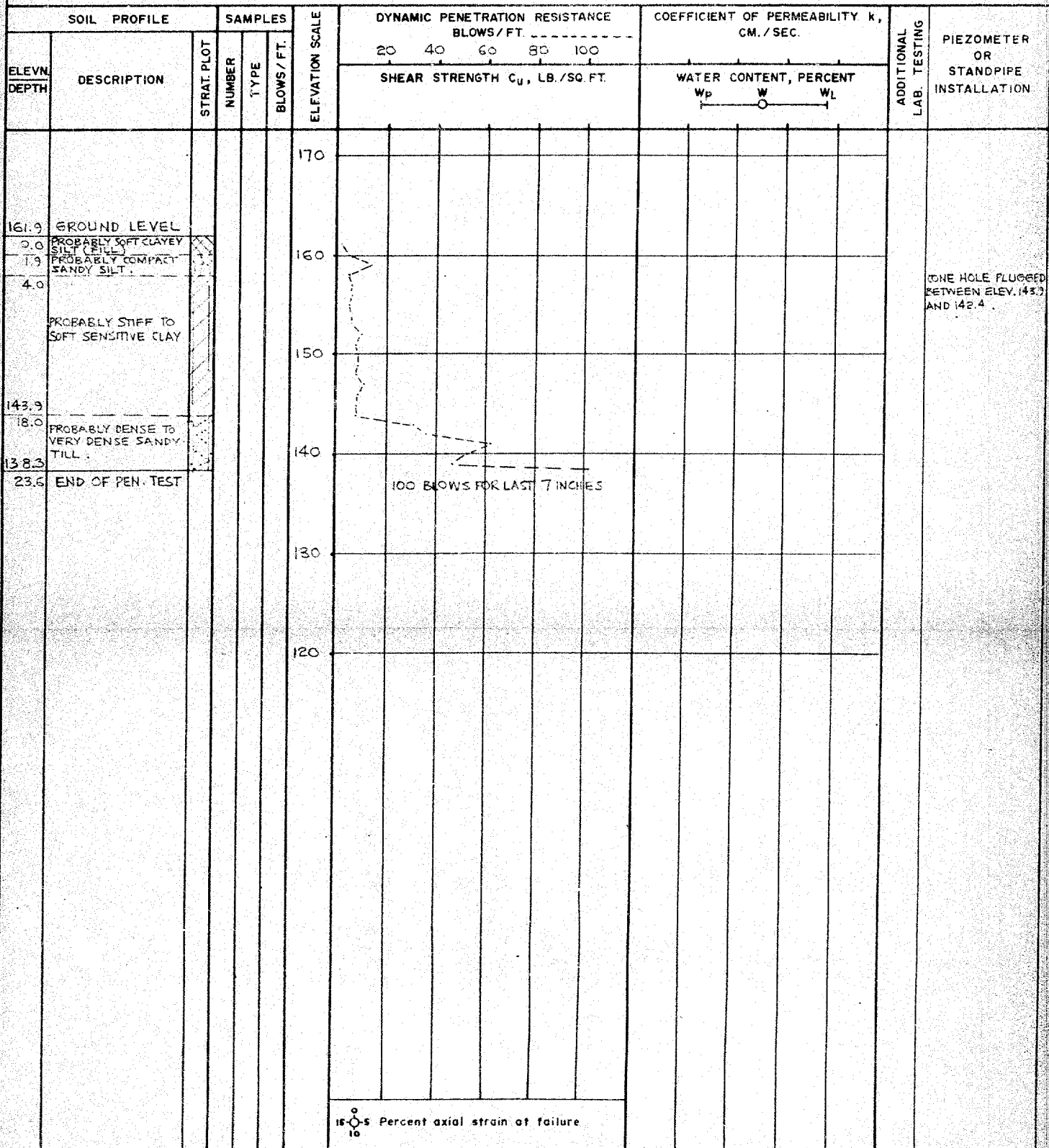
DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER -

SAMPLER HAMMER WEIGHT - LB. DROP - INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES


 VERTICAL SCALE  
 1 INCH TO 10' - 0"

GOLDER &amp; ASSOCIATES

 DRAWN *[Signature]*  
 CHECKED *[Signature]*

PROJECT NO. 44-25-1324

GEODETIC

BOREHOLE DIAMETER

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

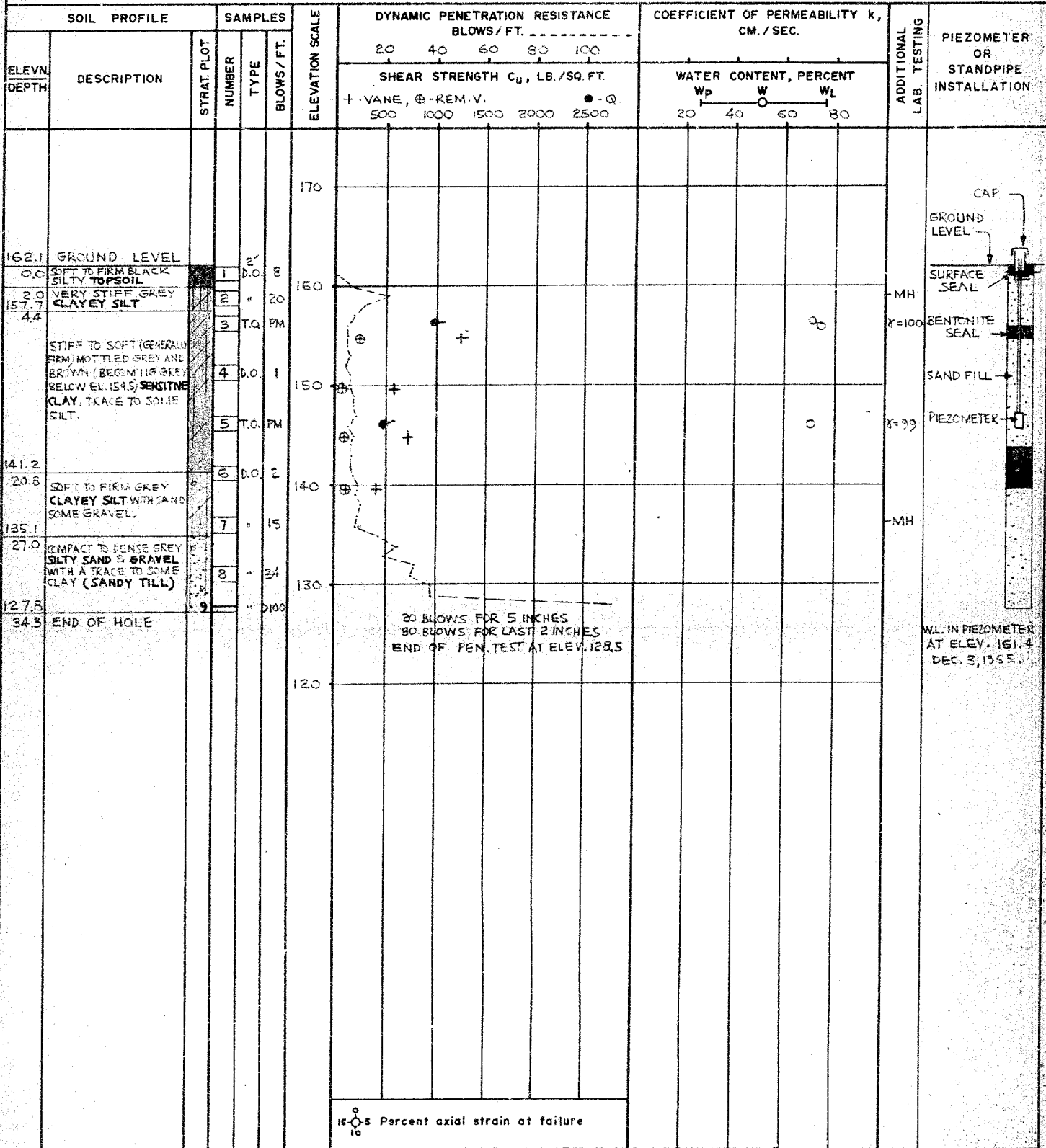
ONE HOLE PLUGGED  
BETWEEN ELEV. 137.9  
AND 136.4.

DRAWN J.A.  
CHECKED TS



# RECORD OF BOREHOLE 11

LOCATION See Figure 1 BORING DATE NOV. 30-DEC. 1965 DATUM GEODETIC  
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE  
 1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN M.W.  
 CHECKED J.B.

# RECORD OF BOREHOLE 12

LOCATION See Figure 1

BORING DATE DEC. 3, 1965

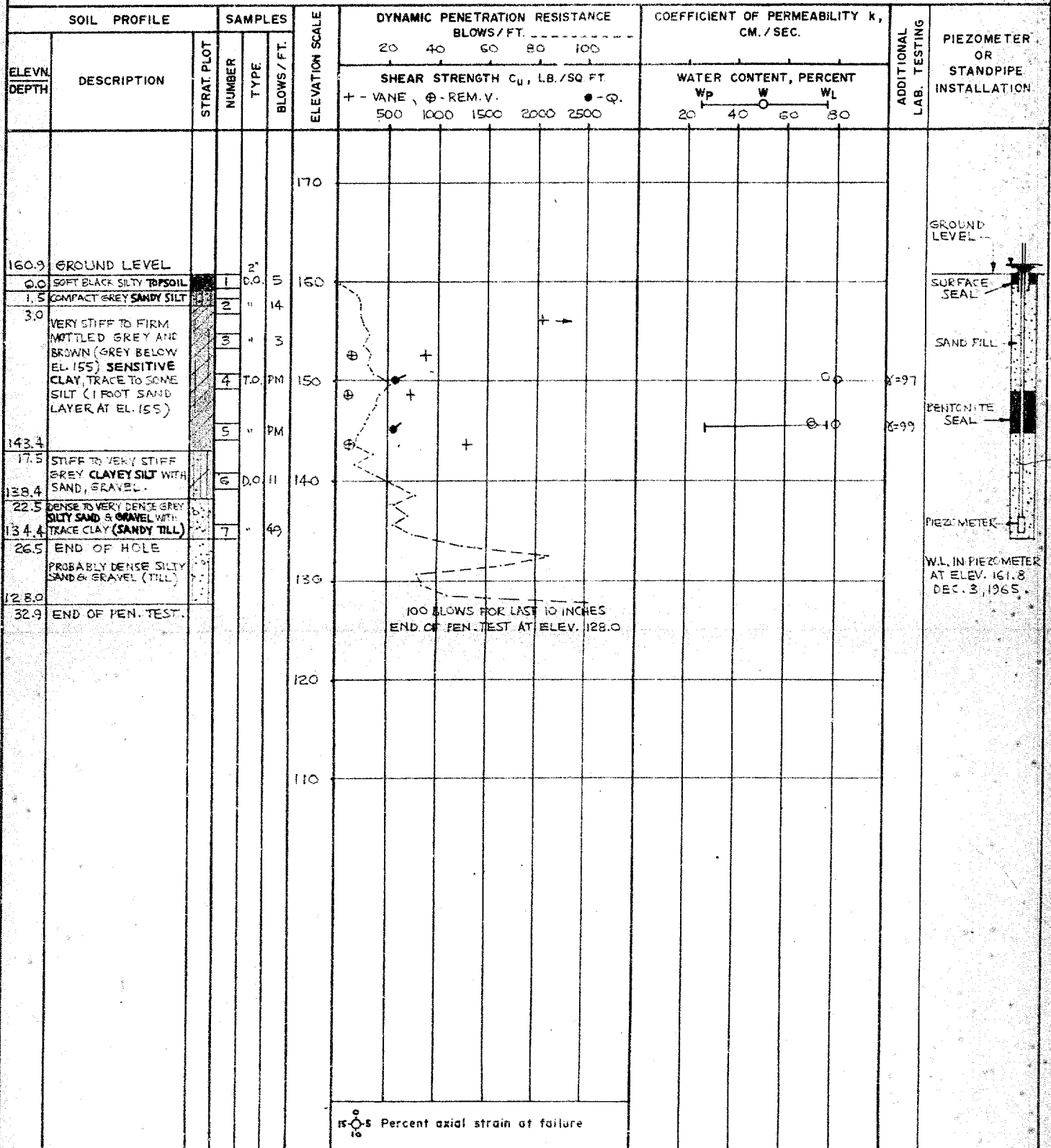
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10'-0"

GOLDER &amp; ASSOCIATES

DRAWN *W.W.*CHECKED *T.S.*

**APPENDIX D**

# Results of Chemical Analysis

**Certificate of Analysis**

Client: Golder Associates Ltd (Ottawa)  
1931 Robertson Road,  
Ottawa, Ontario

Attention: Mr. Kenton Power

PO#:

Invoice to: WSP Canada Inc.

Report Number: 1994271  
Date Submitted: 2023-03-02  
Date Reported: 2023-03-10  
Project: 22513877A  
COC #: 905813

Page 1 of 3

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**Dear Kenton Power:**

**Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).**

Report Comments:

APPROVAL:

\_\_\_\_\_  
Raheleh Zafari, Environmental Chemist

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: <https://directory.cala.ca/>.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

# Certificate of Analysis

Client: Golder Associates Ltd (Ottawa)  
1931 Robertson Road,  
Ottawa, Ontario

Attention: Mr. Kenton Power

PO#:

Invoice to: WSP Canada Inc.

Report Number: 1994271  
Date Submitted: 2023-03-02  
Date Reported: 2023-03-10  
Project: 22513877A  
COC #: 905813

Group	Analyte	MRL	Units	Guideline	Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1676306 Soil  2023-02-06 22-01A Sa3/5-7'	1676307 Soil  2023-02-08 22-01A Sa10/25-27'	1676308 Soil  2023-01-30 22-02A Sa5/10-12'	1676309 Soil  2023-01-31 22-02A Sa21/S5-S7'
Anions	Cl	0.002	%			<0.002	0.011	0.007	0.003
	SO4	0.01	%			0.04	0.02	0.01	0.03
General Chemistry	Electrical Conductivity	0.05	mS/cm			0.46	0.38	0.33	0.43
	pH	2.00				7.46	7.86	7.73	7.71
	Resistivity	1	ohm-cm			2174	2632	3030	2326

Group	Analyte	MRL	Units	Guideline	Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1676310 Soil  2023-02-10 22-03A Sa4/7.5-9.5'	1676311 Soil  2023-02-10 22-03A Sa11/30-32'	1676312 Soil  2023-02-03 22-04A Sa5/10-12'	1676313 Soil  2023-02-06 22-04A Sa17/48-50'
Anions	Cl	0.002	%			0.003	<0.002	0.010	0.004
	SO4	0.01	%			0.03	0.02	0.05	0.02
General Chemistry	Electrical Conductivity	0.05	mS/cm			0.47	0.30	0.62	0.32
	pH	2.00				7.67	8.00	7.58	7.89
	Resistivity	1	ohm-cm			2128	3333	1613	3125

**Guideline =**                      **\* = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.  
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range



# Certificate of Analysis

Client: Golder Associates Ltd (Ottawa)  
1931 Robertson Road,  
Ottawa, Ontario

Attention: Mr. Kenton Power

PO#:

Invoice to: WSP Canada Inc.

Report Number: 1994271  
Date Submitted: 2023-03-02  
Date Reported: 2023-03-10  
Project: 22513877A  
COC #: 905813

## QC Summary

Analyte	Blank	QC % Rec	QC Limits
<b>Run No</b> 438459 <b>Analysis/Extraction Date</b> 2023-03-09 <b>Analyst</b> IP <b>Method</b> Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	90-110
pH	7.01	99	90-110
Resistivity			
<b>Run No</b> 438464 <b>Analysis/Extraction Date</b> 2023-03-09 <b>Analyst</b> IP <b>Method</b> AG SOIL			
SO4	<0.01 %	95	70-130
<b>Run No</b> 438496 <b>Analysis/Extraction Date</b> 2023-03-10 <b>Analyst</b> AET <b>Method</b> C CSA A23.2-4B			
Chloride		107	90-110
<b>Run No</b> 438499 <b>Analysis/Extraction Date</b> 2023-03-10 <b>Analyst</b> IP <b>Method</b> AG SOIL			
SO4	<0.01 %	105	70-130

**Guideline =**      \* = **Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.  
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

**APPENDIX E**

# Selected Site Photographs



*Photograph 1: Fraser Road over Highway 401*



*Photograph 2: Highway 401 looking westbound*





*Photograph 3: Fraser Road looking southbound from BH 22-02*



*Photograph 4: Fraser Road looking northbound from BH 22-02*





*Photograph 5: Looking west from BH 22-03*



Photograph 6: Fraser Road looking southbound from BH 22-04





Photograph 7: Fraser Road looking northbound from BH 22-04



**APPENDIX F**

**ConeTec Investigation Report No: 23-05-25302**

# PRESENTATION OF SITE INVESTIGATION RESULTS

## Fraser Road CPTs

*Prepared for:*

Golder Associates

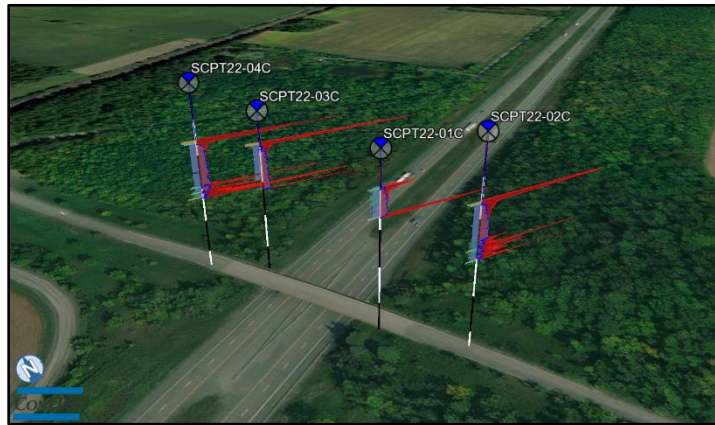
ConeTec Job No: 23-05-25302

Project Start Date: 31-Jan-2023

Project End Date: 08-Feb-2023

Report Date: 15-Feb-2023

Revised Report Date: 16-Feb-2023



*Prepared by:*

ConeTec Investigations Ltd.  
9033 Leslie Street, Unit 15  
Richmond Hill, ON L4B 4K3

Tel: (905) 886-2663

Fax: (905) 886-2664

ConeTecON@conetec.com  
[www.conetec.com](http://www.conetec.com)  
[www.conetecdataservices.com](http://www.conetecdataservices.com)



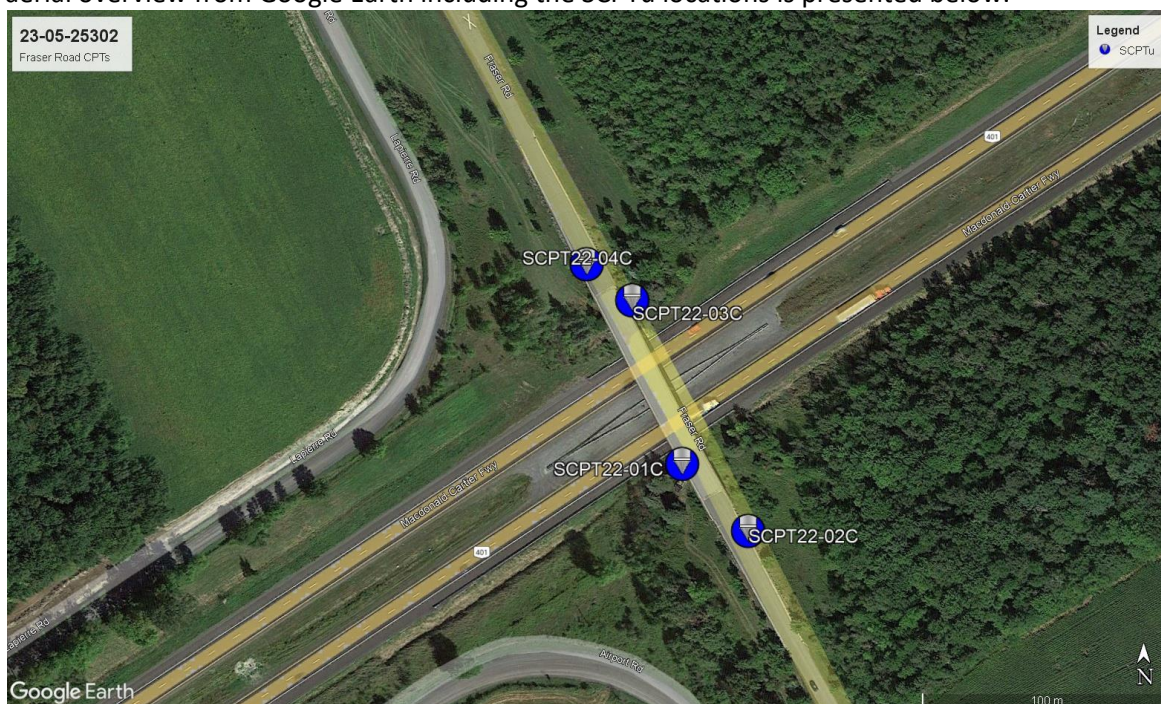
## Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates near Fraser Road and Macdonald-Cartier Freeway, South Glengarry Ontario. The program consisted of four seismic cone penetration tests (SCPTu). Please note that this report, which also includes all accompanying data, are subject to the 3<sup>rd</sup> Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

## Project Information

Project	
Client	Golder Associates
Project	Fraser Road CPTs
ConeTec project number	23-05-25302

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



Rig Description	Deployment System	Test Type
CPT track rig (M5T)	14 ton rig cylinder	SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
SCPTu	Client Provided	2950

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 (bar)
806:T1000F10U35	806	15	225	1000	10	35
Cone 806 was used for all CPTu soundings.						

Cone Penetration Test (CPTu)	
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.
Seismic calculations	Poisson's ratio ( $\nu$ ) was calculated from the shear wave ( $V_s$ ) and compression wave ( $V_p$ ) velocities using the following equation: $\nu = \frac{(V_p/V_s)^2 - 2}{2((V_p/V_s)^2 - 1)}$
Additional plots	<ul style="list-style-type: none"> <li>Advanced plots with <math>I_c</math>, <math>S_u</math>, <math>\Phi</math>, and <math>N1(60)I_c</math></li> <li>Seismic shear and compression wave velocity plots</li> <li>Soil Behaviour Type (SBT) scatter plots</li> </ul>
Additional comments	Seismic compression wave velocity data is not reported for SCPT22-02C and SCPT22-04C due to poor signal quality.

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 CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu 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>).</p> <p>Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile.</p> <p>Soils were classified as either drained or undrained based on the <math>Q_{tn}</math> Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).</p>

## Limitations

### 3rd Party Disclaimer

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### Client Disclaimer

ConeTec was retained by Golder Associates to collect and provide the raw data (“Data”) which is included in this report titled “Fraser Road CPTs”, which is referred to as the (“Report”). ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Any analysis, interpretation, judgment, calculations and/or geotechnical parameters (collectively “Interpretations”) included in the Report, including those based on the Data, are outside the scope of ConeTec’s retainer and are included in the Report as a courtesy only. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

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 two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

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

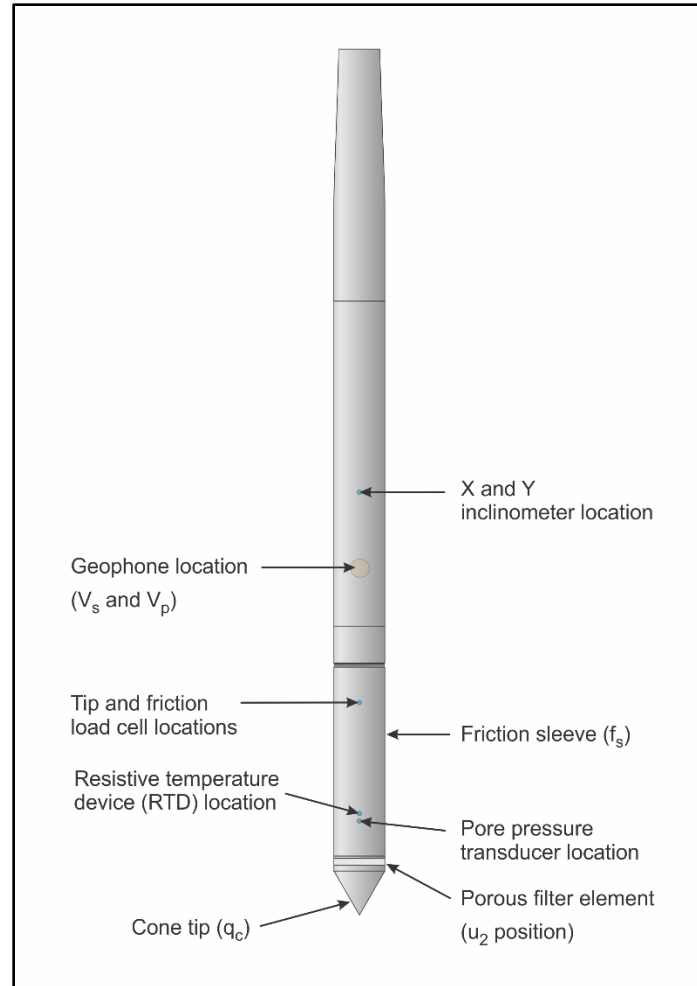


Figure CPTu. Piezocone Penetrometer (15 cm<sup>2</sup>)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. 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 centimeters; 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 CPTu 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 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 two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 38.1 millimeters 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
- 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 \(2012\)](#).

## References

ASTM D5778-20, 2020, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: [10.1520/D5778-20](#).

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: [10.1061/9780784412770.027](#).

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization 4, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158. DOI: [10.1139/T90-014](#).

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: [10.1139/T09-065](#).

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 15 cm<sup>2</sup> piezocone penetrometers are manufactured with one horizontally active geophone (28 hertz) and one vertically active geophone (28 hertz). Both geophones are rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip. The vertically mounted geophone is more sensitive to compression waves; however, it is often affected by the compression wave travelling through the cone rods.

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 may be 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 in the memory of the cone using a fast analog to digital converter. The seismic trace is then transmitted digitally uphole to a Windows based computer through a signal interface box for recording and analysis. An illustration of the shear wave testing configuration is presented in [Figure SCPTu-1](#).

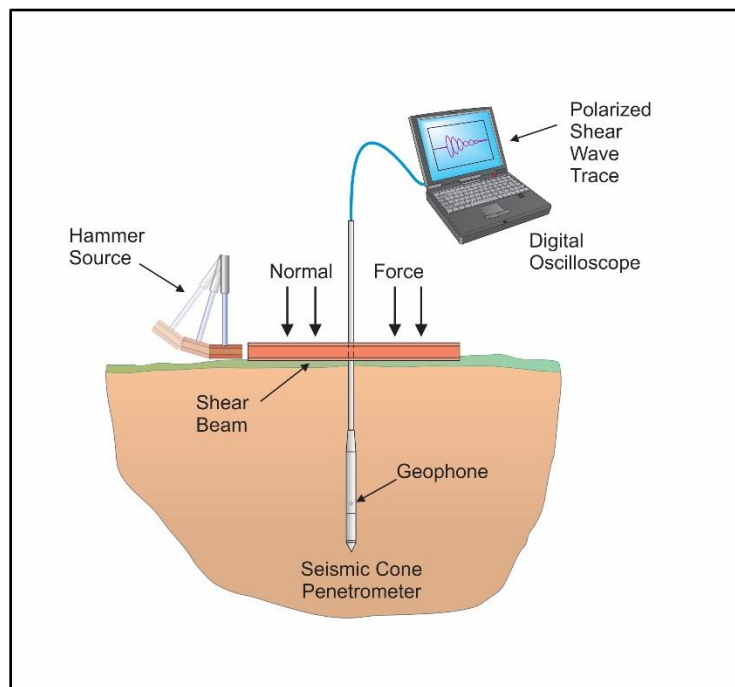


Figure SCPTu-1. Illustration of the SCPTu system

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. \(1986\)](#).

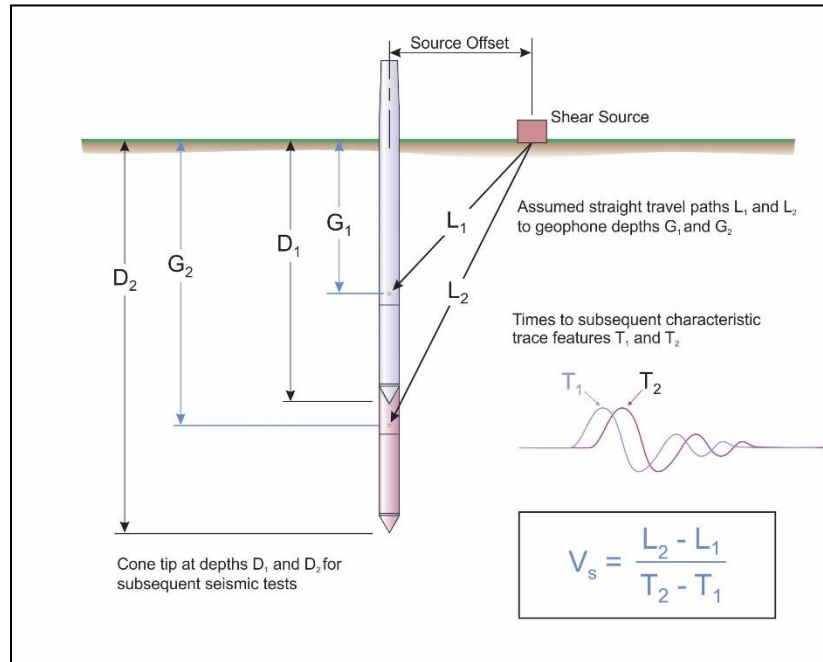


Figure SCPTu-2. Illustration of a seismic cone penetration test

For the determination of interval travel times the wave traces from all depths are displayed in analysis software. The results of the interval picks are supplied in the relevant appendix of this report. Standard practice for ConeTec is to record five wave traces for each source direction at each test depth. Outlier impacts are identified in the field and the impacts are repeated. For the final wave trace profile, the traces are stacked in the time domain to display a single average trace.

Determination of the shear wave interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the trace depths and taking the difference in ray path divided by the time difference between features at subsequent depths. The same process is used for compression waves, however the first break is most commonly used for selecting an arrival time. For velocity calculation, the 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.

In some cases, usually for shear wave velocity testing, more than one characteristic marker may be used. If there is an overlap between different sets of characteristic markers, then the average time value for those sets of interval times is applied to the determination of velocity.

Ideally, all depths are used for the determination of the velocity profile. However, an interval may be skipped if there is some ambiguity or quality concern with a particular depth, resulting in a larger interval.

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

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

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

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

## References

ASTM D5778-20, 2020, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: [10.1520/D5778-20](#).

ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM International, West Conshohocken, PA. DOI: [10.1520/D7400\\_D7400M-19](#).

Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, "National shear wave measurement guidelines for Canadian seismic site assessment", GeoManitoba 2012, Sept 30 to Oct 2, Winnipeg, Manitoba.

Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803. DOI: [10.1061/\(ASCE\)0733-9410\(1986\)112:8\(791\)](#).

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 ( $t$ ).

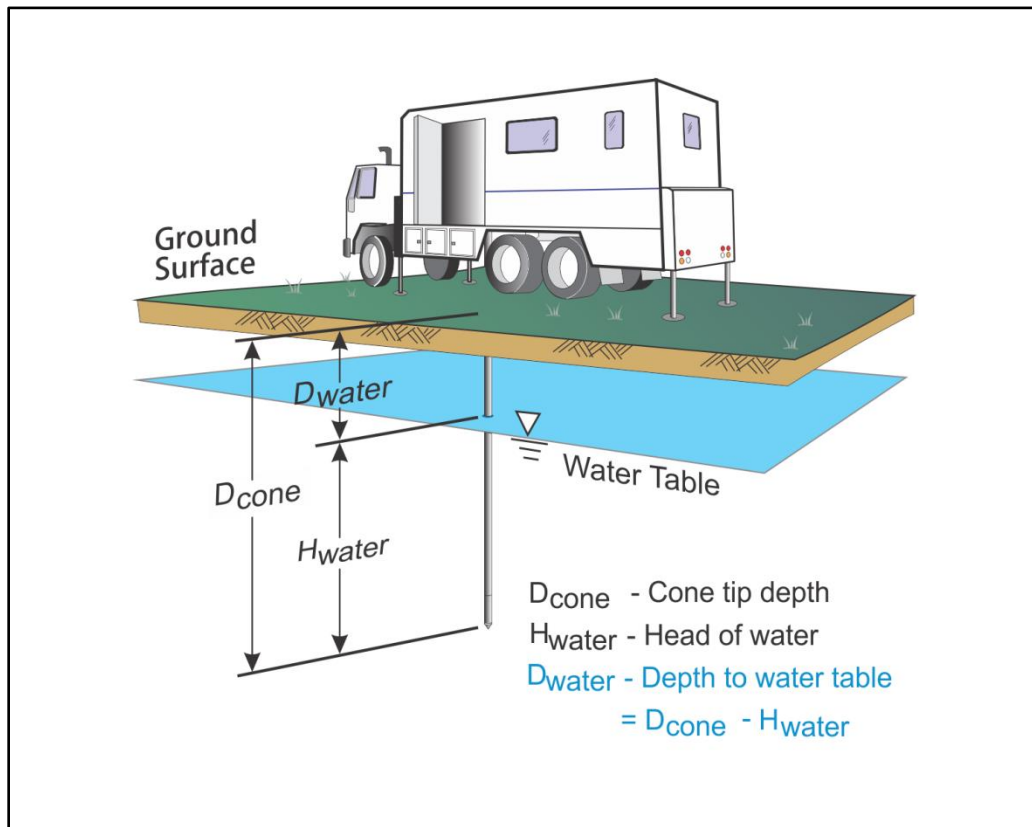


Figure PPD-1. Pore pressure dissipation test setup

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.

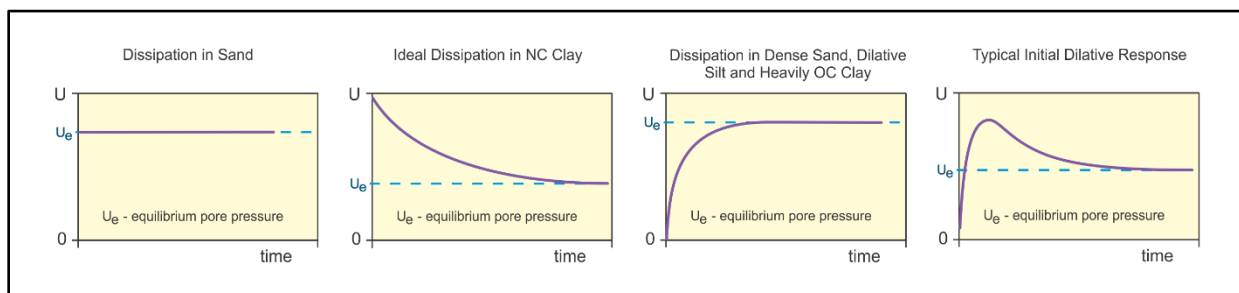


Figure PPD-2. Pore pressure dissipation curve examples

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 dilatatory 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.

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

## References

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", *Geotechnique*, 41(1): 17-34. DOI: [10.1680/geot.1991.41.1.17](https://doi.org/10.1680/geot.1991.41.1.17).

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Tabular Results
- Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Traces
- Seismic Cone Penetration Test Compression Wave ( $V_p$ ) Tabular Results
- Seismic Cone Penetration Test Compression Wave ( $V_p$ ) Traces
- Seismic Cone Penetration Test Poisson's Ratio Tabular Results
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots
- Description of Methods for Calculated CPT Geotechnical Parameters



# Cone Penetration Test Summary and Standard Cone Penetration Test Plots



Job No: 23-05-25302  
Client: Golder Associates  
Project: Fraser Road CPTs  
Start Date: 31-Jan-2023  
End Date: 08-Feb-2023

### CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm <sup>2</sup> )	Assumed Phreatic Surface <sup>1</sup> (m)	Final Depth (m)	Northing <sup>2</sup> (m)	Easting <sup>2</sup> (m)	Elevation <sup>3</sup> (m)	Refer to Notation Number
SCPT22-01C	23-05-25302_SP01	07-Feb-2023	806:T1000F10U35	15	3.2	8.475	4997615.55	222586.22	52.93	
SCPT22-02C	23-05-25302_SP02	03-Feb-2023	806:T1000F10U35	15	6.4	15.175	4997590.74	222609.10	55.85	4
SCPT22-03C	23-05-25302_SP03	08-Feb-2023	806:T1000F10U35	15	3.3	11.100	4997677.91	222569.16	52.71	
SCPT22-04C	23-05-25302_SP04	31-Jan-2023	806:T1000F10U35	15	6.6	16.500	4997696.17	222549.04	56.00	4

1. The assumed phreatic surface was based on pore pressure dissipation tests. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates and elevations were provided by the client in datum NAD 83/ MTM Zone 8 North.
3. Elevations are referenced the to existing surface at the time of testing.
4. No seismic compression velocity data reported due to poor signal quality.



Golder

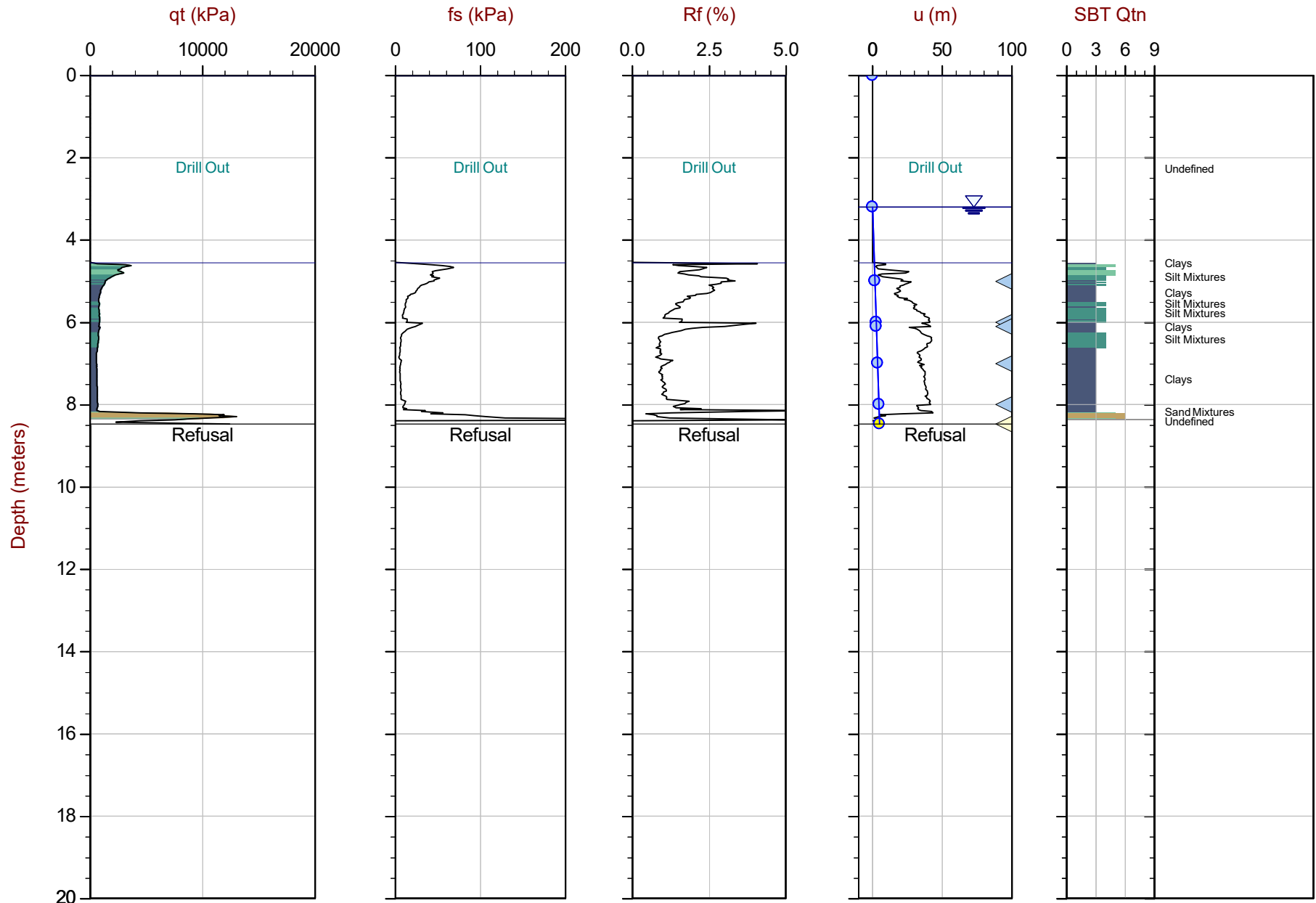
Job No: 23-05-25302

Date: 2023-02-07 10:24

Site: Fraser Road, South Glengarry

Sounding: SCPT22-01C

Cone: 806:T1000F10U35 Area: 15cm<sup>2</sup>



Max Depth: 8.475 m / 27.80 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 23-05-25302\_SP01.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: MTM8NN: 4997615.55m E: 222586.22m Elev: 52.93m

Sheet No: 1 of 1

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

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



Golder

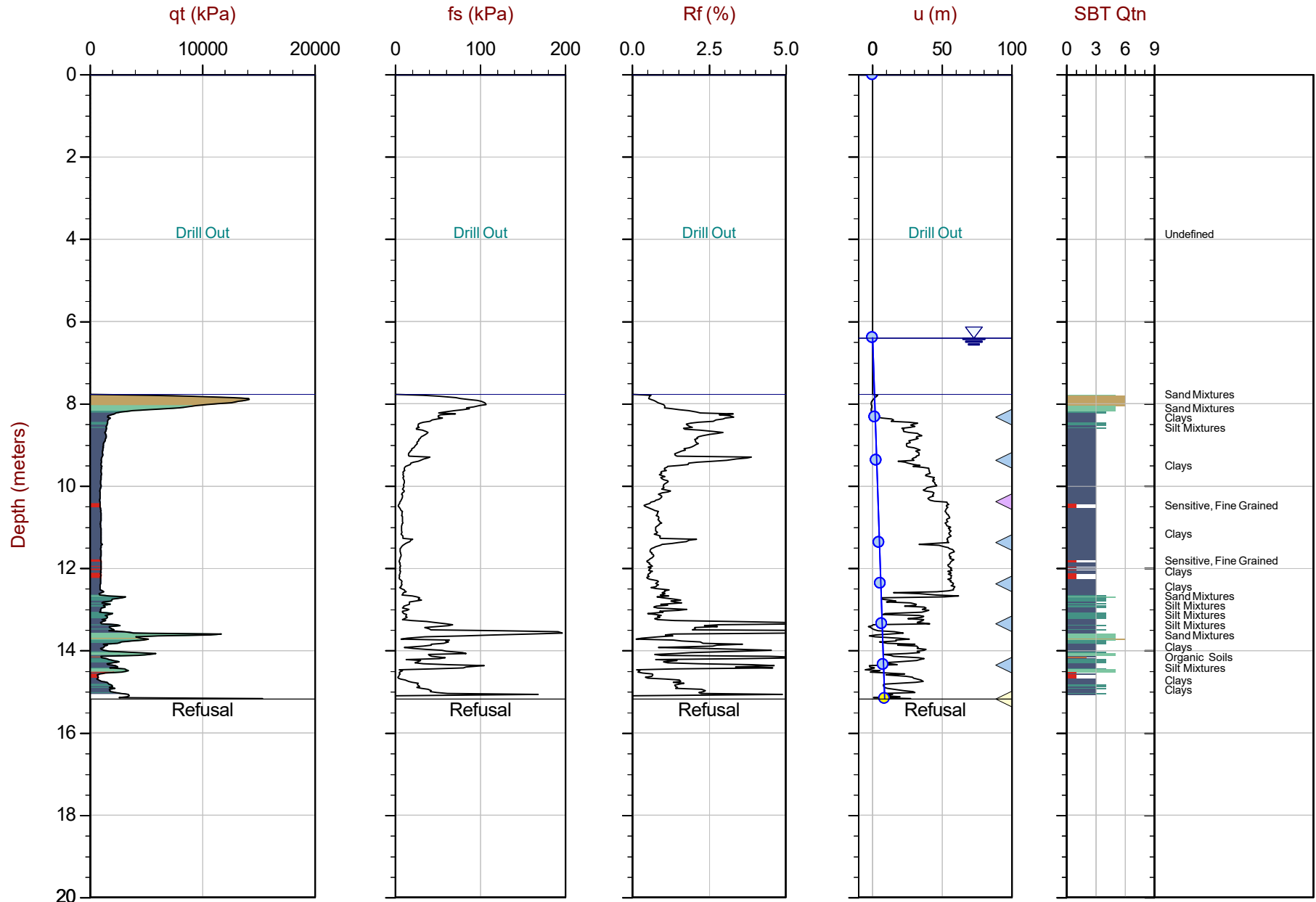
Job No: 23-05-25302

Date: 2023-02-03 10:46

Site: Fraser Road, South Glengarry

Sounding: SCPT22-02C

Cone: 806:T1000F10U35 Area: 15cm<sup>2</sup>



Max Depth: 15.175 m / 49.79 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 23-05-25302\_SP02.COR

Unit Wt: SBTQtn(PKR2009)

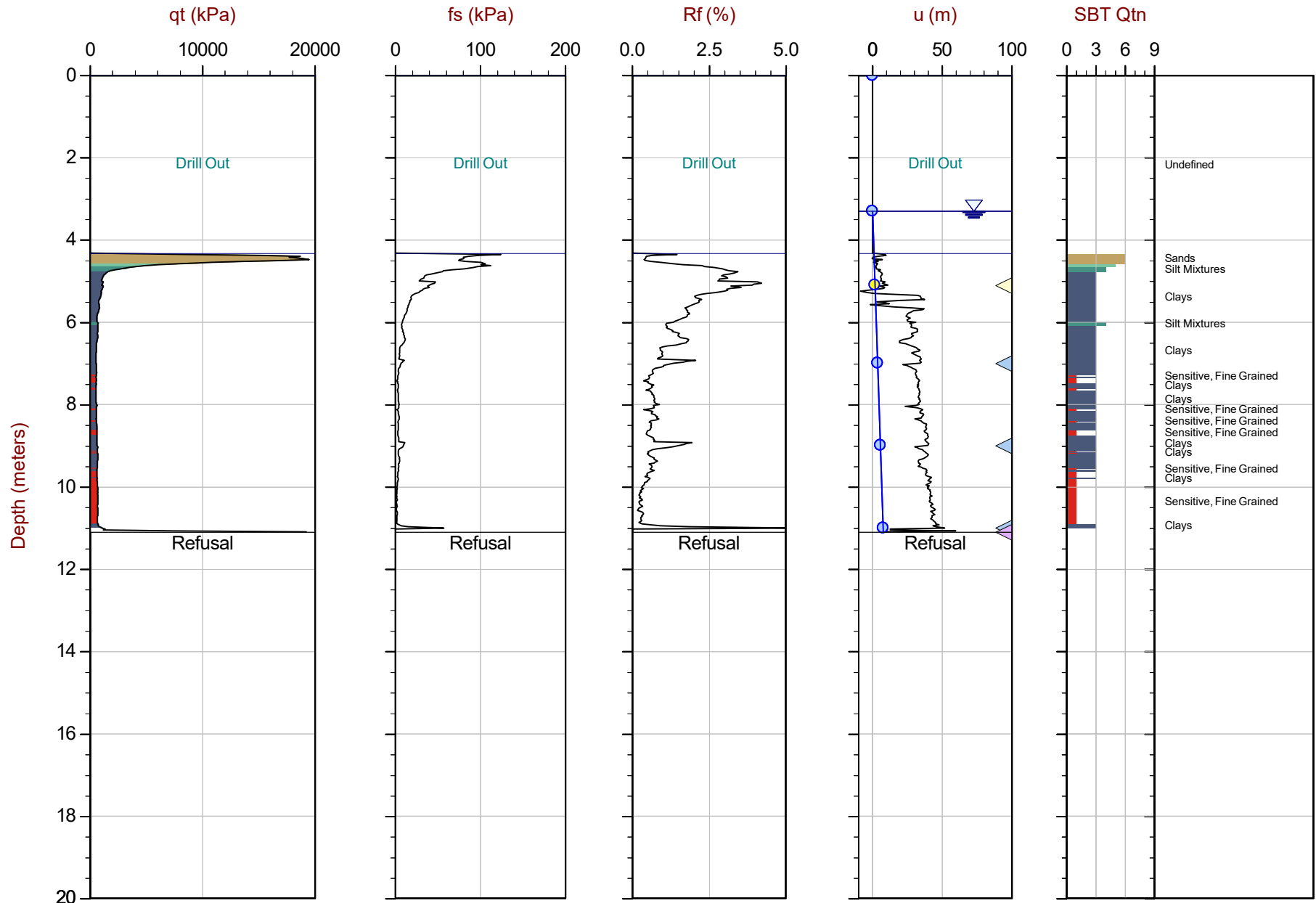
SBT: Robertson, 2009 and 2010

Coords: MTM8N: 4997590.74m E: 222609.10m Elev: 55.85m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▲ Dissipation, Ueq not achieved ▲ Dissipation, Ueq assumed — 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.



Max Depth: 11.100 m / 36.42 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 23-05-25302\_SP03.COR

Unit Wt: SBTQtn(PKR2009)

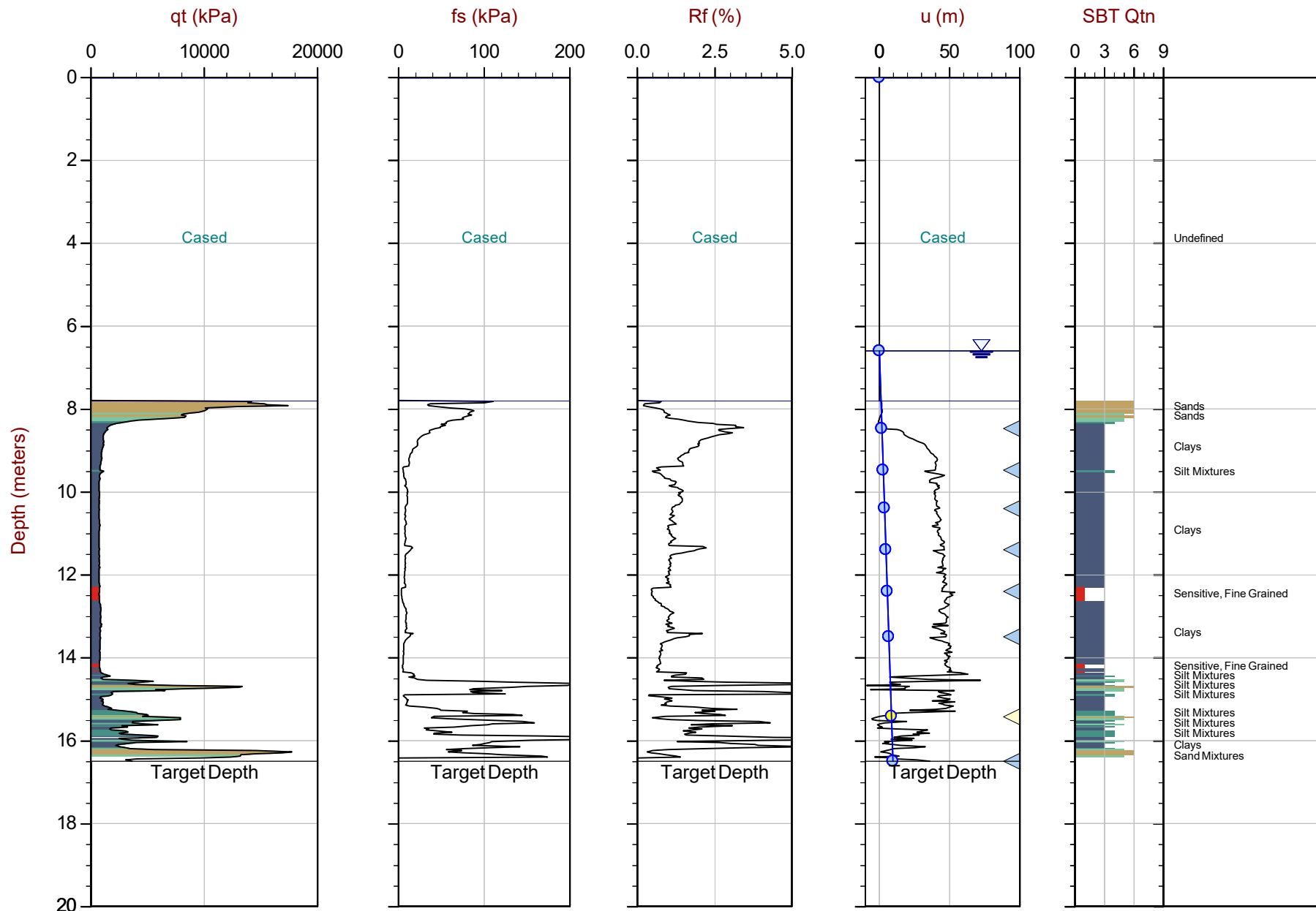
SBT: Robertson, 2009 and 2010

Coords: MTM8NN: 4997677.91m E: 222569.16m Elev: 52.71m

Sheet No: 1 of 1

Overplot Item: ● Ueq    ● Assumed Ueq    ▲ Dissipation, Ueq achieved    ▲ Dissipation, Ueq not achieved    ▲ Dissipation, Ueq assumed    — 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.



Max Depth: 16.500 m / 54.13 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 23-05-25302\_SP04.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: MTM8N: 4997696.17m E: 222549.04m Elev: 56.00m

Sheet No: 1 of 1

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

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

## Advanced Cone Penetration Test Plots



Golder

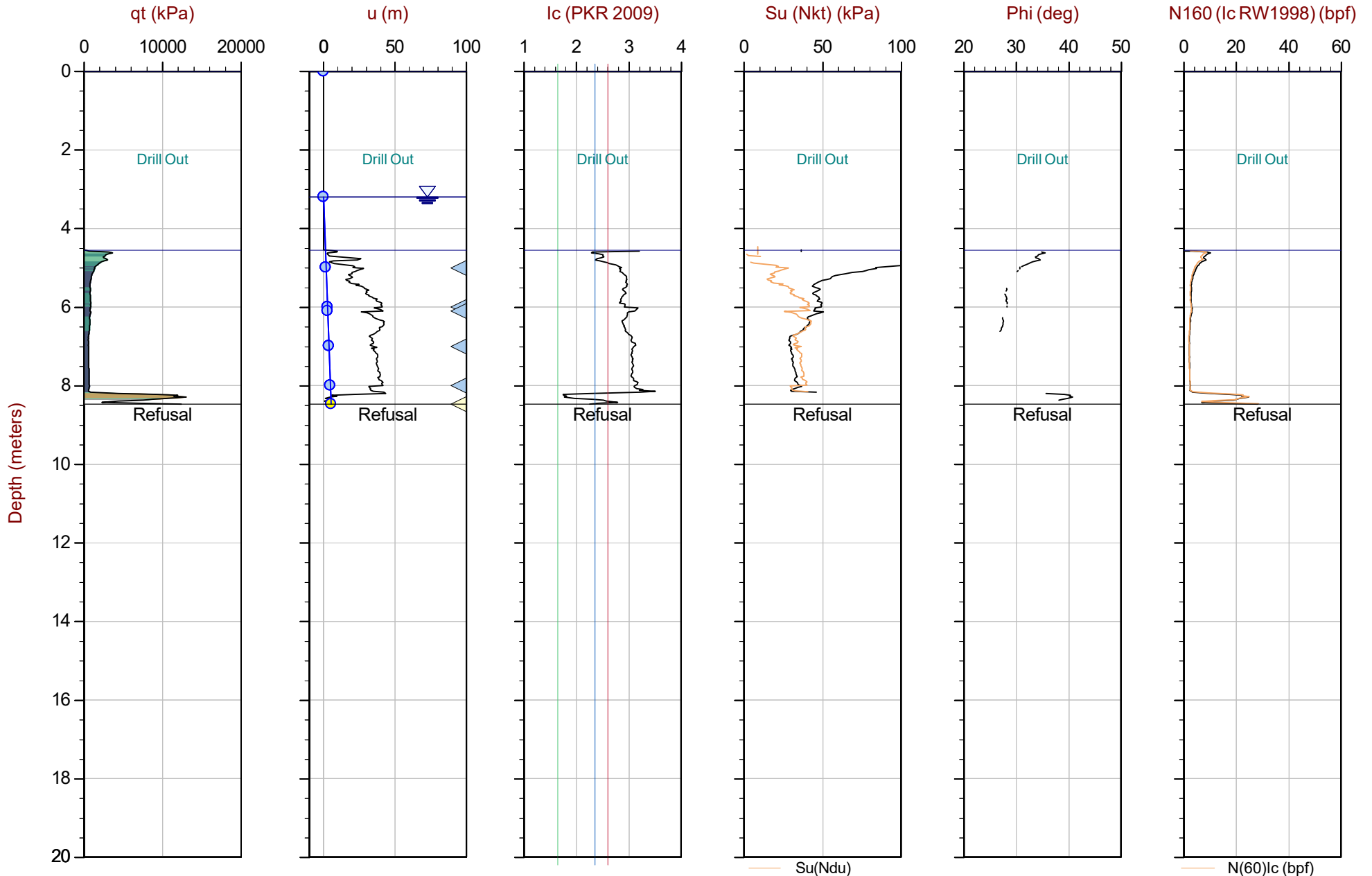
Job No: 23-05-25302

Date: 2023-02-07 10:24

Site: Fraser Road, South Glengarry

Sounding: SCPT22-01C

Cone: 806:T1000F10U35 Area: 15cm<sup>2</sup>



Max Depth: 8.475 m / 27.80 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 23-05-25302\_SP01.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010

Coords: MTM8NN: 4997615.55m E: 222586.22m Elev: 52.93m

Sheet No: 1 of 1

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

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





Golder

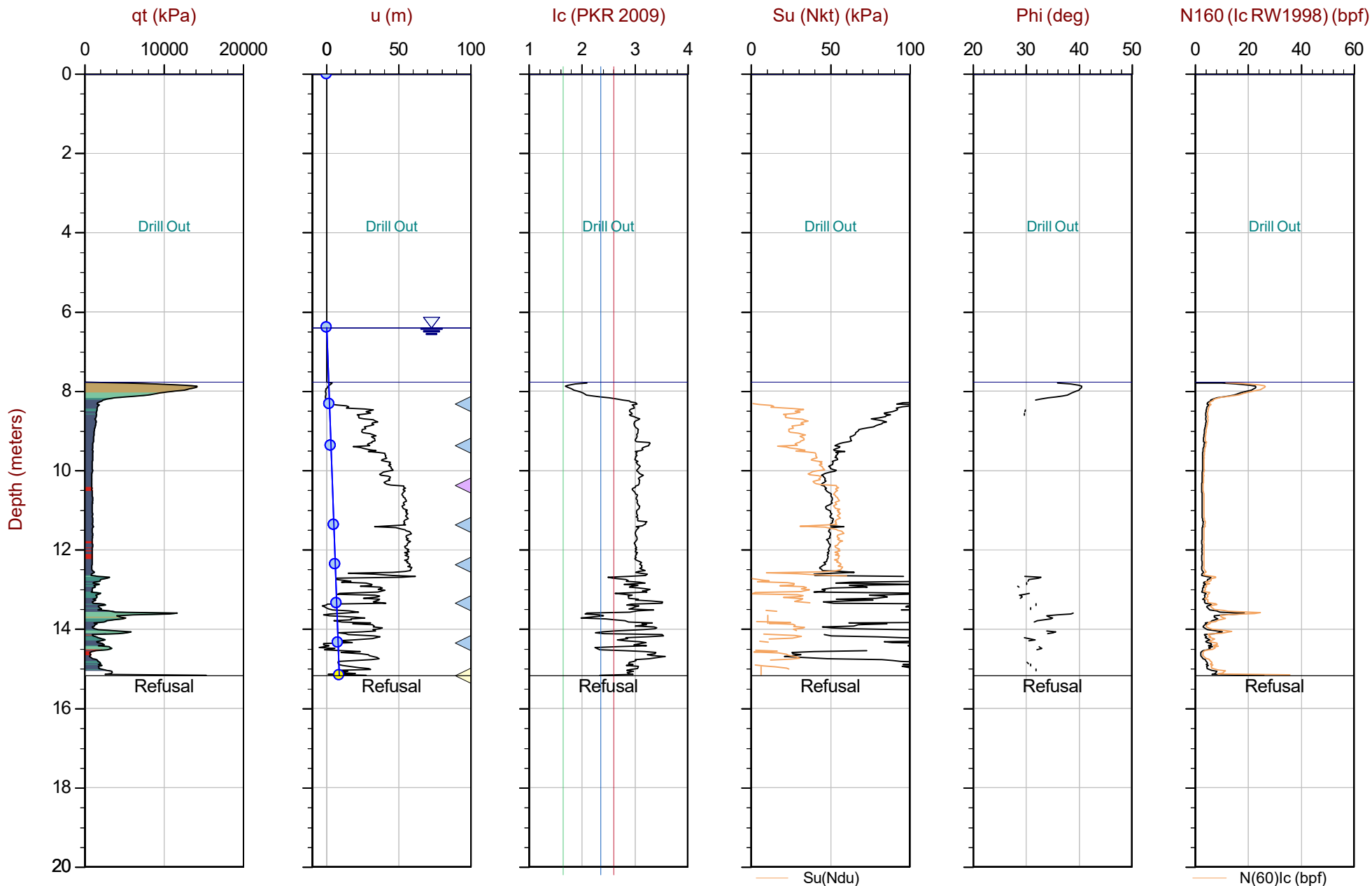
Job No: 23-05-25302

Date: 2023-02-03 10:46

Site: Fraser Road, South Glengarry

Sounding: SCPT22-02C

Cone: 806:T1000F10U35 Area: 15cm2



Max Depth: 15.175 m / 49.79 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 23-05-25302\_SP02.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010

Coords: MTM8NN: 4997590.74m E: 222609.10m Elev: 55.85m

Sheet No: 1 of 1

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

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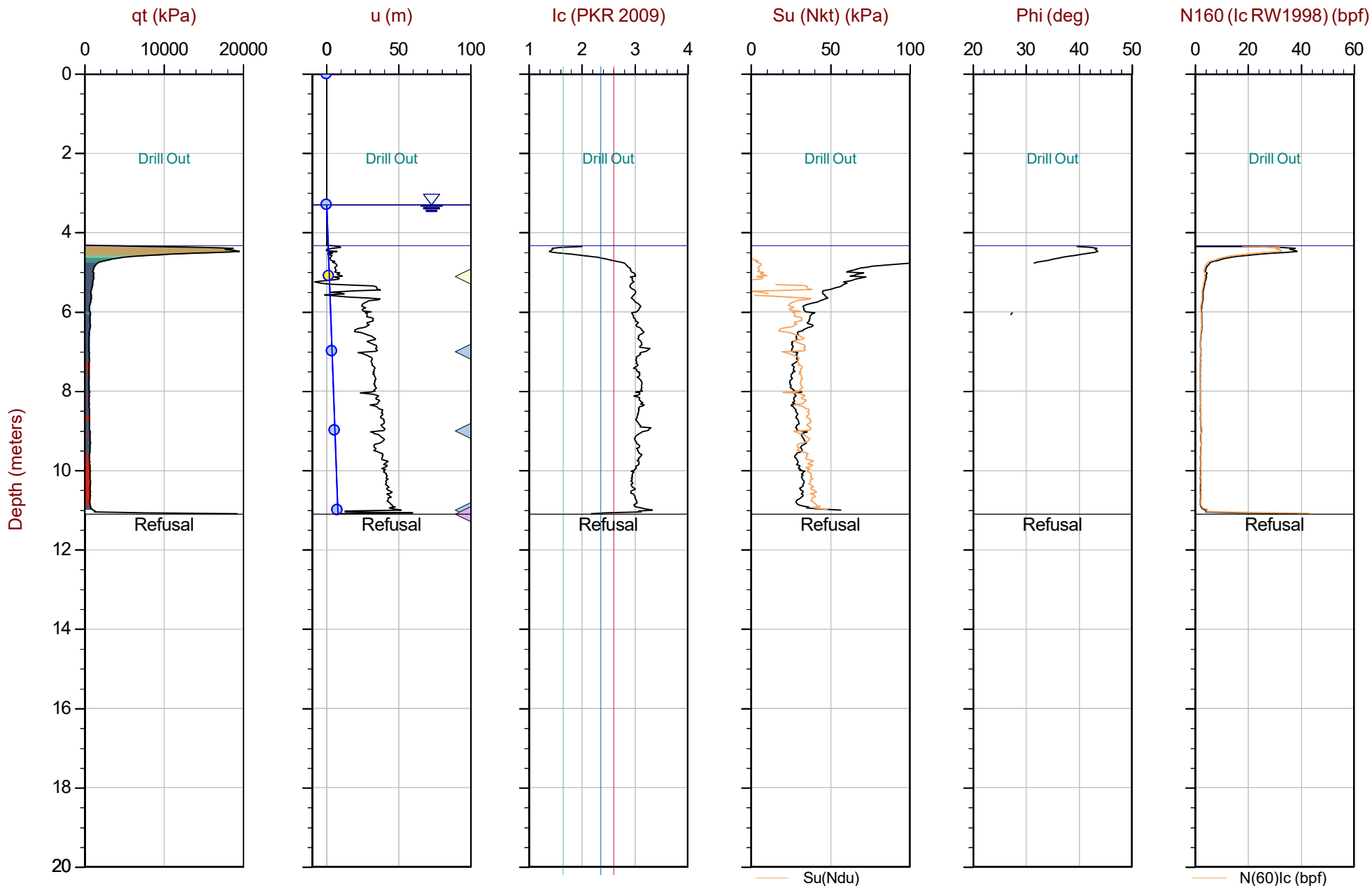
Job No: 23-05-25302

Date: 2023-02-08 08:08

Site: Fraser Road, South Glengarry

Sounding: SCPT22-03C

Cone: 806:T1000F10U35 Area: 15cm2



Max Depth: 11.100 m / 36.42 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 23-05-25302\_SP03.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010

Coords: MTM8NN: 4997677.91m E: 222569.16m Elev: 52.71m

Sheet No: 1 of 1

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

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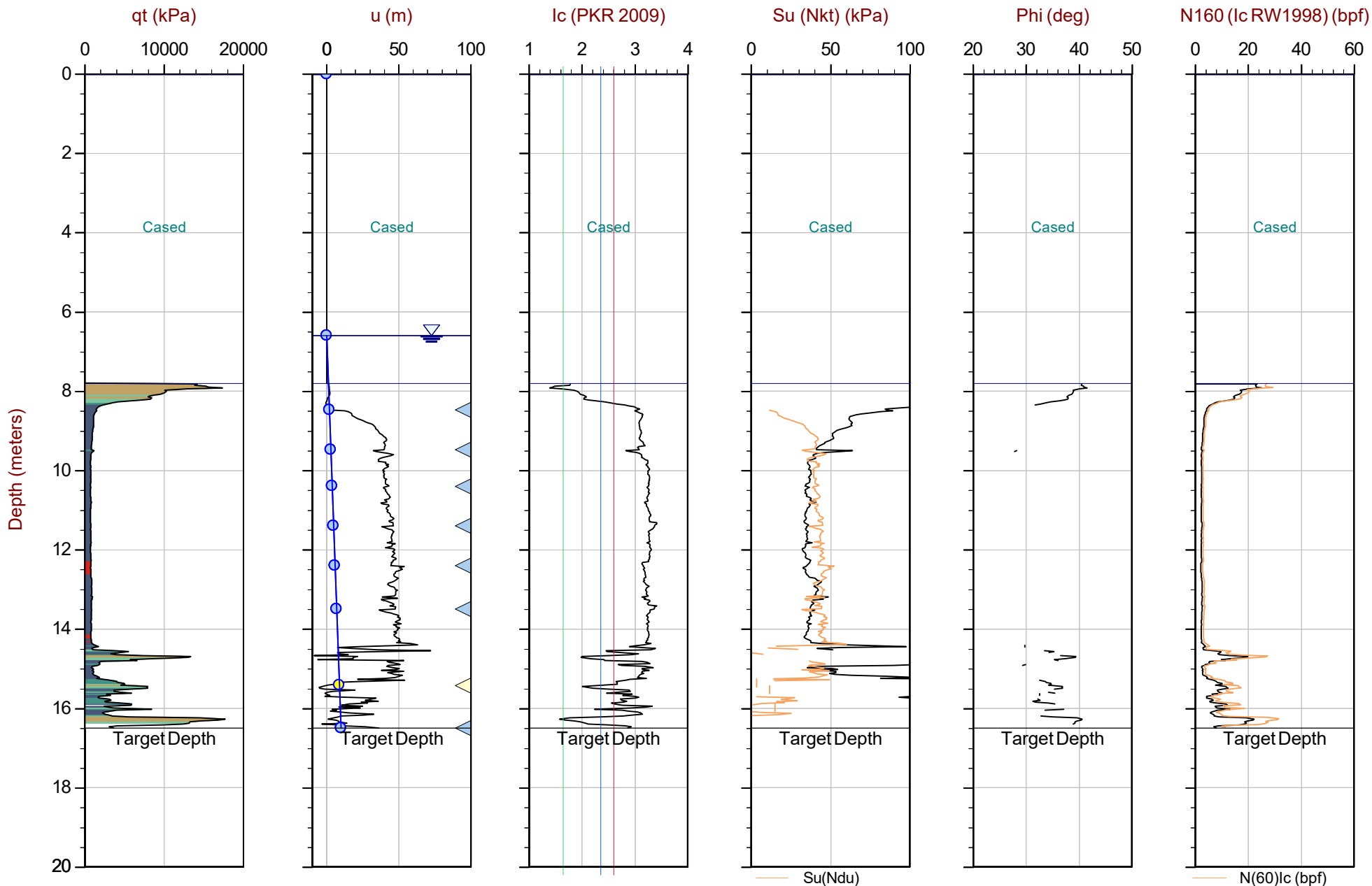
Job No: 23-05-25302

Date: 2023-01-31 12:51

Site: Fraser Road, South Glengarry

Sounding: SCPT22-04C

Cone: 806:T1000F10U35 Area: 15cm2



Max Depth: 16.500 m / 54.13 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 23-05-25302\_SP04.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010

Coords: MTM8NN: 4997696.17m E: 222549.04m Elev: 56.00m

Sheet No: 1 of 1

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

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## Seismic Cone Penetration Test Plots



Golder

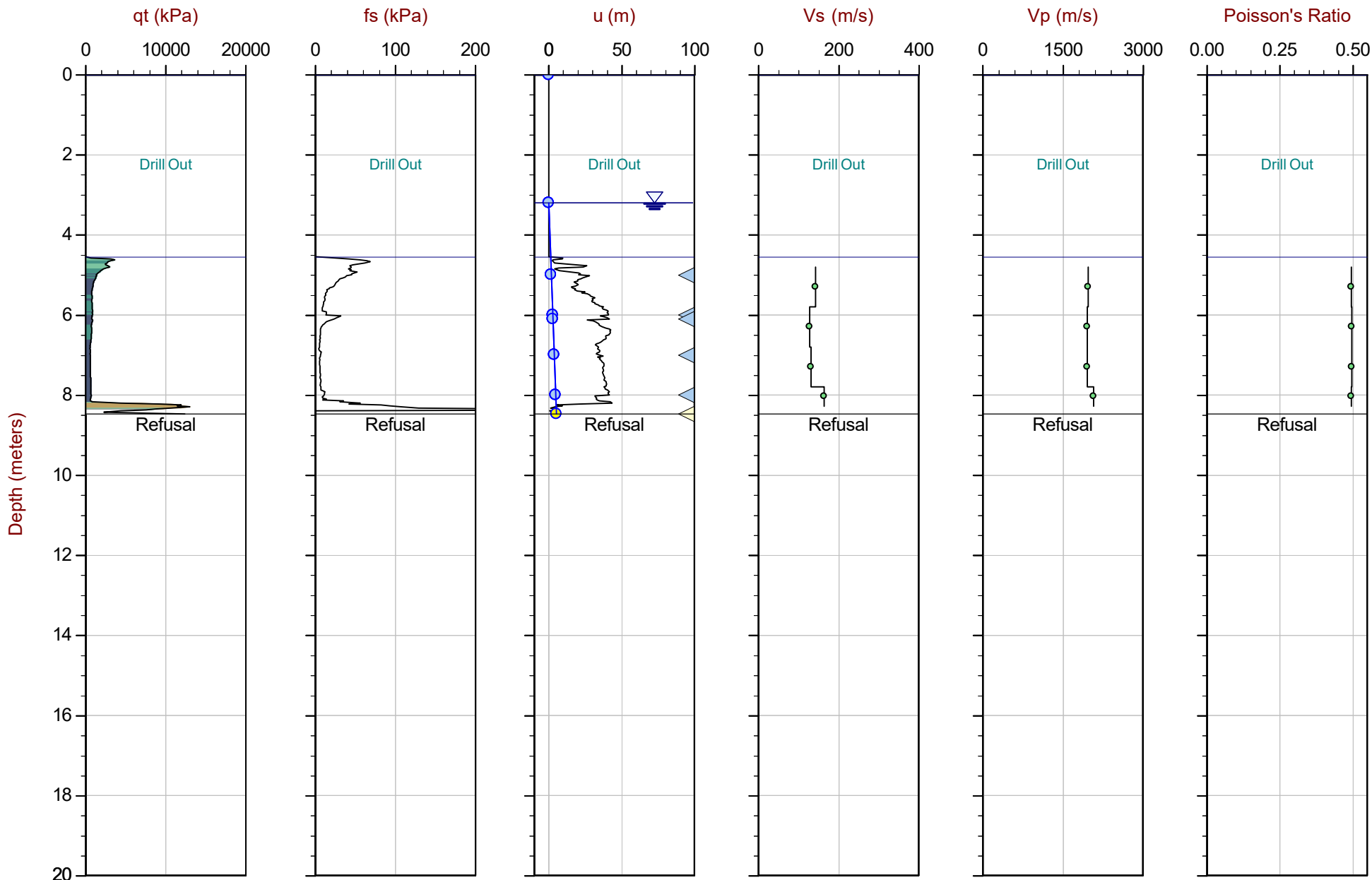
Job No: 23-05-25302

Date: 2023-02-07 10:24

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Sounding: SCPT22-01C

Cone: 806:T1000F10U35 Area: 15cm<sup>2</sup>



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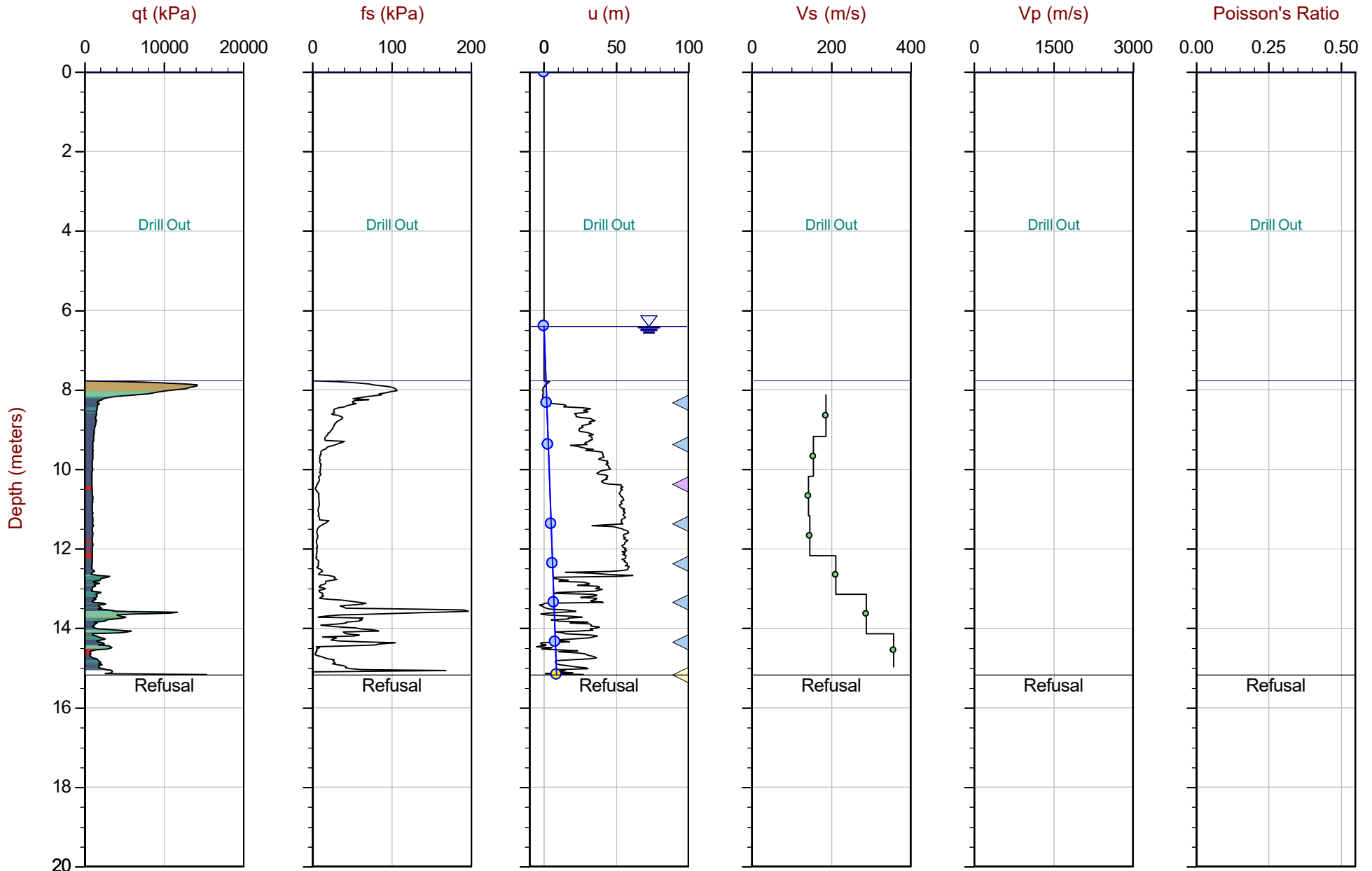
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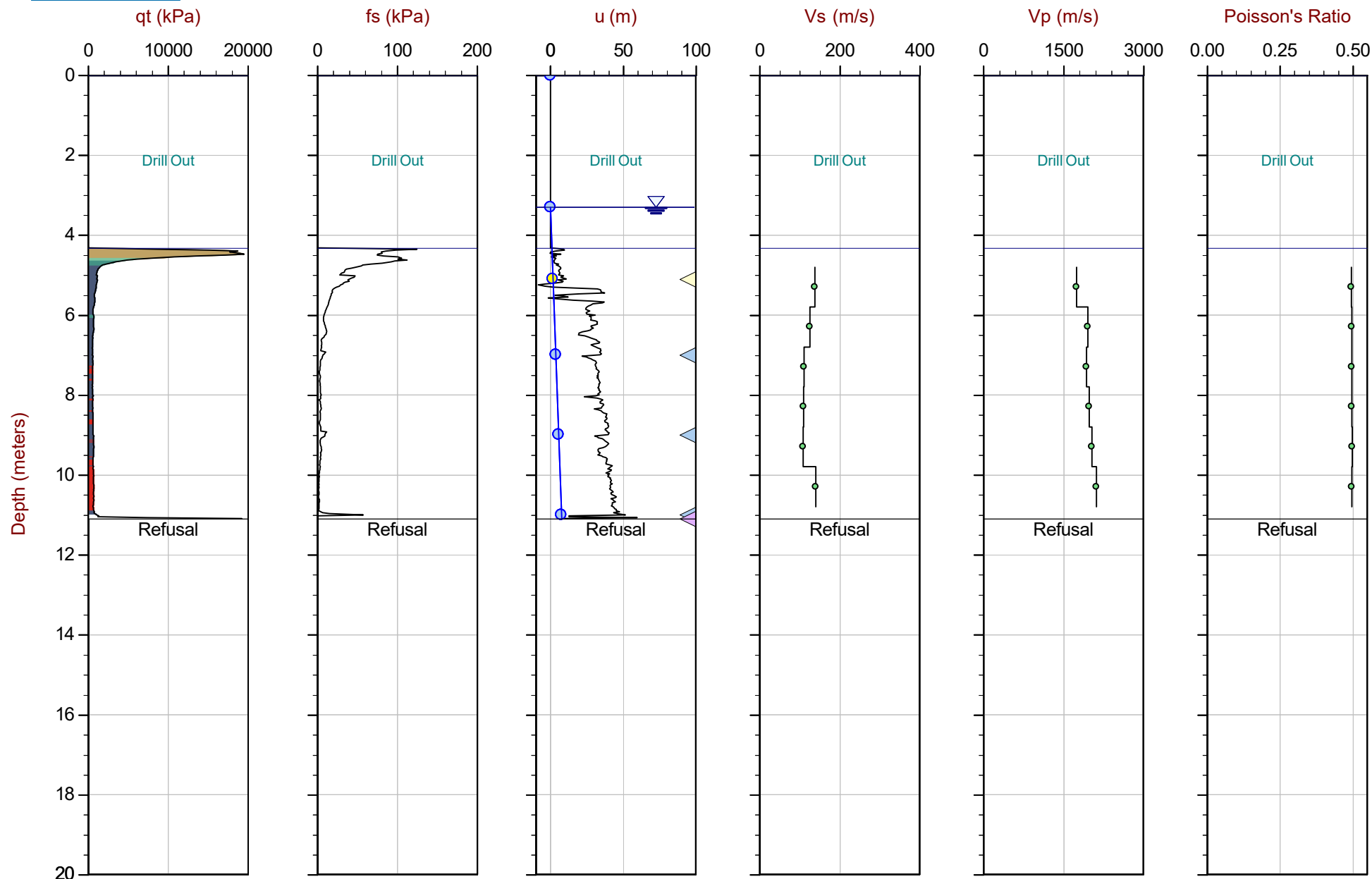
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Date: 2023-02-08 08:08

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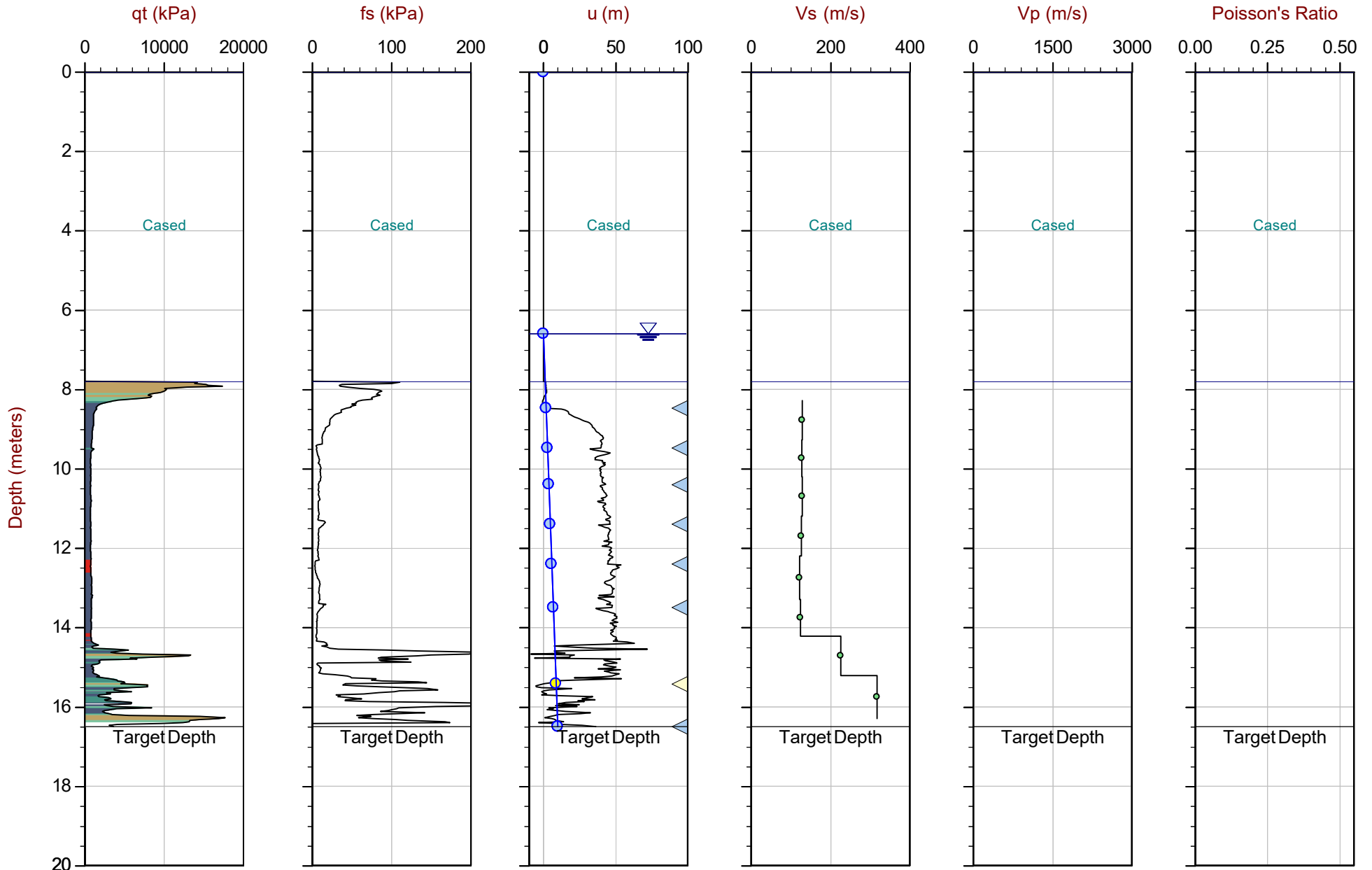
Job No: 23-05-25302

Date: 2023-01-31 12:51

Site: Fraser Road, South Glengarry

Sounding: SCPT22-04C

Cone: 806:T1000F10U35 Area: 15cm<sup>2</sup>



Max Depth: 16.500 m / 54.13 ft

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File: 23-05-25302\_SP04.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

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Sheet No: 1 of 1

OverplotItem: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▲ Dissipation, Ueq not achieved ▲ Dissipation, Ueq assumed — Hydrostatic Line

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## Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Tabular Results



Job No: 23-05-25302  
Client: Golder Associates  
Project: Fraser Road CPTs  
Sounding ID: SCPT22-01C  
Date: 07-Feb-2023

Seismic Source: Beam  
Seismic Offset (m): 0.45  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - $V_s$***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
5.00	4.80	4.82			
6.00	5.80	5.82	1.00	6.99	143
7.00	6.80	6.82	1.00	7.82	128
8.00	7.80	7.81	1.00	7.58	132
8.48	8.28	8.29	0.48	2.92	164



Job No: 23-05-25302  
Client: Golder Associates  
Project: Fraser Road CPTs  
Sounding ID: SCPT22-02C  
Date: 03-Feb-2023

Seismic Source: Beam  
Seismic Offset (m): 0.45  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

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

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
8.33	8.13	8.14			
9.38	9.18	9.19	1.05	5.61	187
10.38	10.18	10.19	1.00	6.45	155
11.38	11.18	11.19	1.00	7.00	143
12.38	12.18	12.19	1.00	6.83	146
13.35	13.15	13.16	0.97	4.57	212
14.35	14.15	14.16	1.00	3.46	289
15.18	14.98	14.99	0.83	2.32	358



Job No: 23-05-25302  
Client: Golder Associates  
Project: Fraser Road CPTs  
Sounding ID: SCPT22-03C  
Date: 08-Feb-2023

Seismic Source: Beam  
Seismic Offset (m): 0.45  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

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

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
5.00	4.80	4.82			
6.00	5.80	5.82	1.00	7.19	139
7.00	6.80	6.82	1.00	7.94	126
8.00	7.80	7.81	1.00	9.01	111
9.00	8.80	8.81	1.00	9.09	110
10.00	9.80	9.81	1.00	9.19	109
11.00	10.80	10.81	1.00	7.10	141



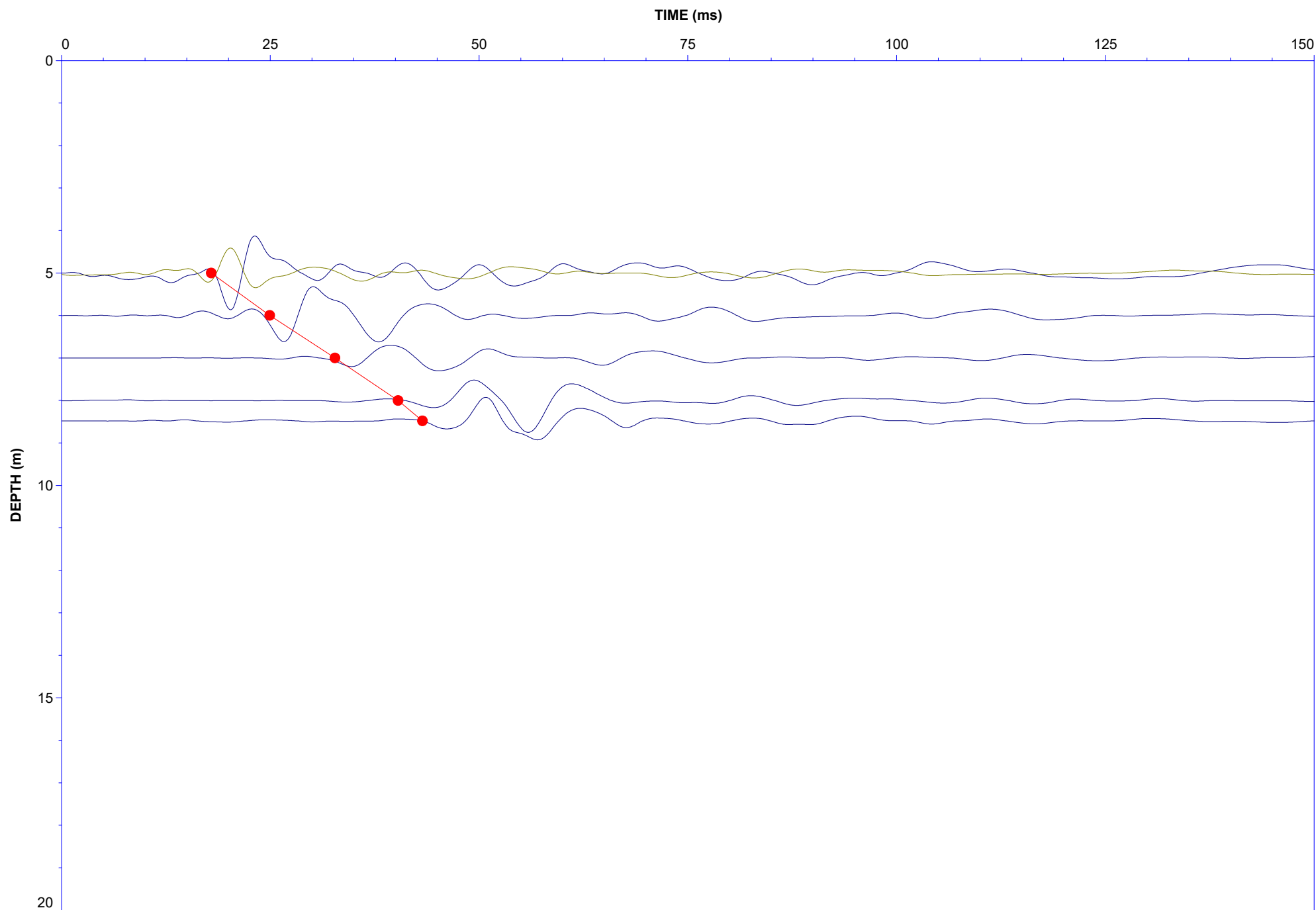
Job No: 23-05-25302  
Client: Golder Associates  
Project: Fraser Road CPTs  
Sounding ID: SCPT22-04C  
Date: 31-Jan-2023

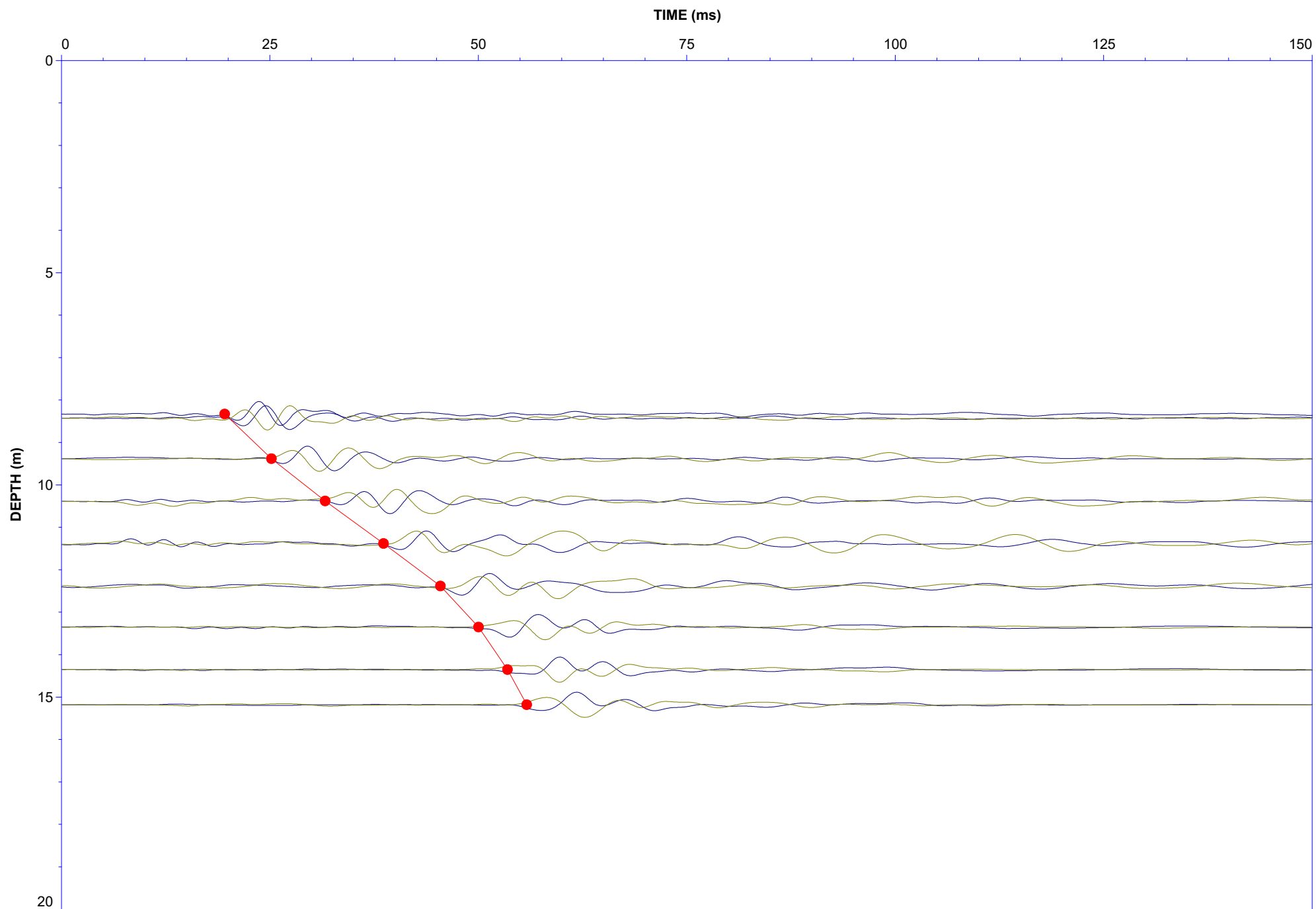
Seismic Source: Beam  
Seismic Offset (m): 0.45  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

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

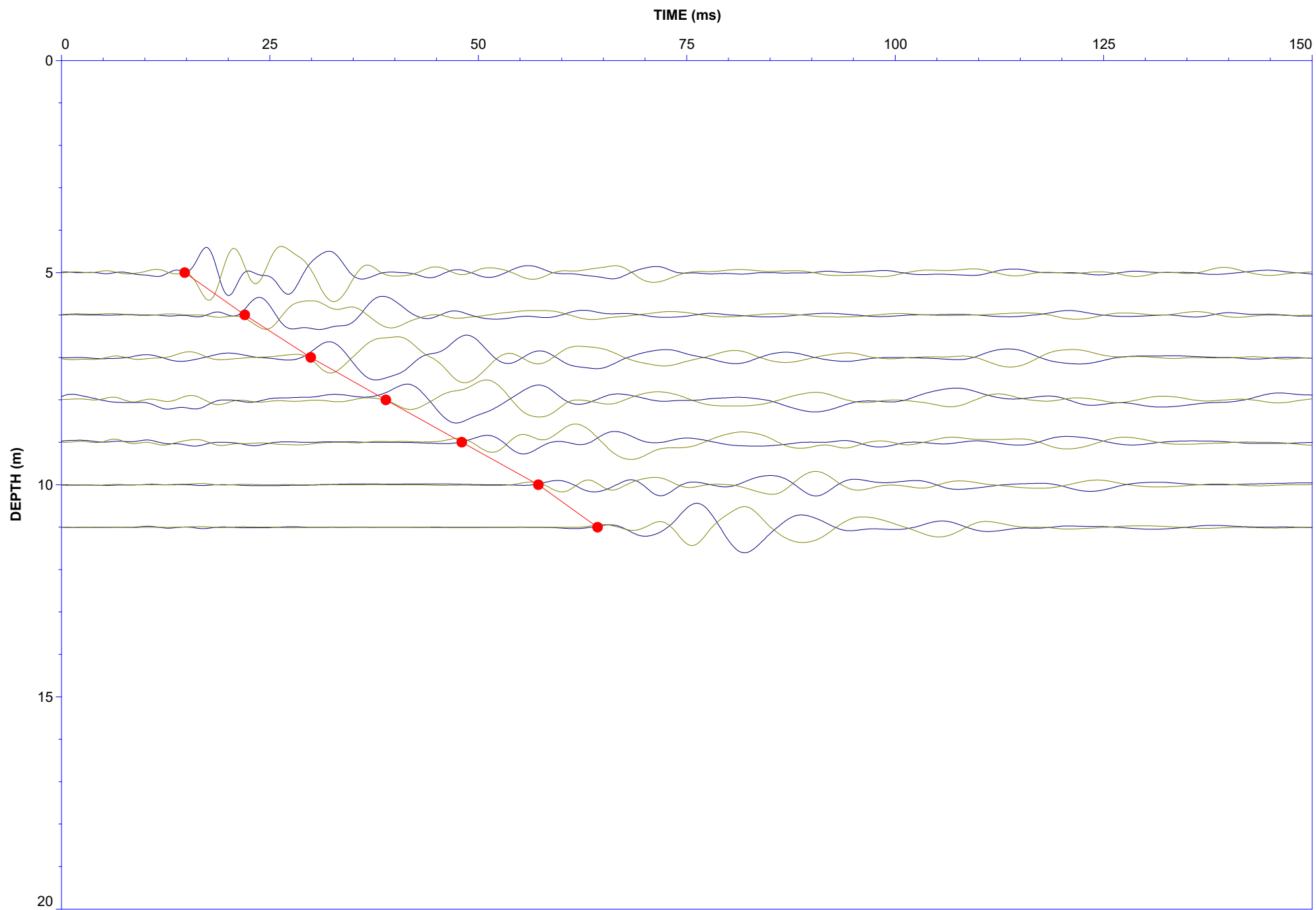
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
8.48	8.28	8.29			
9.48	9.28	9.29	1.00	7.73	129
10.40	10.20	10.21	0.92	7.14	129
11.40	11.20	11.21	1.00	7.74	129
12.40	12.20	12.21	1.00	7.89	127
13.50	13.30	13.31	1.10	8.97	123
14.42	14.22	14.23	0.92	7.39	124
15.42	15.22	15.23	1.00	4.40	227
16.50	16.30	16.31	1.08	3.39	318

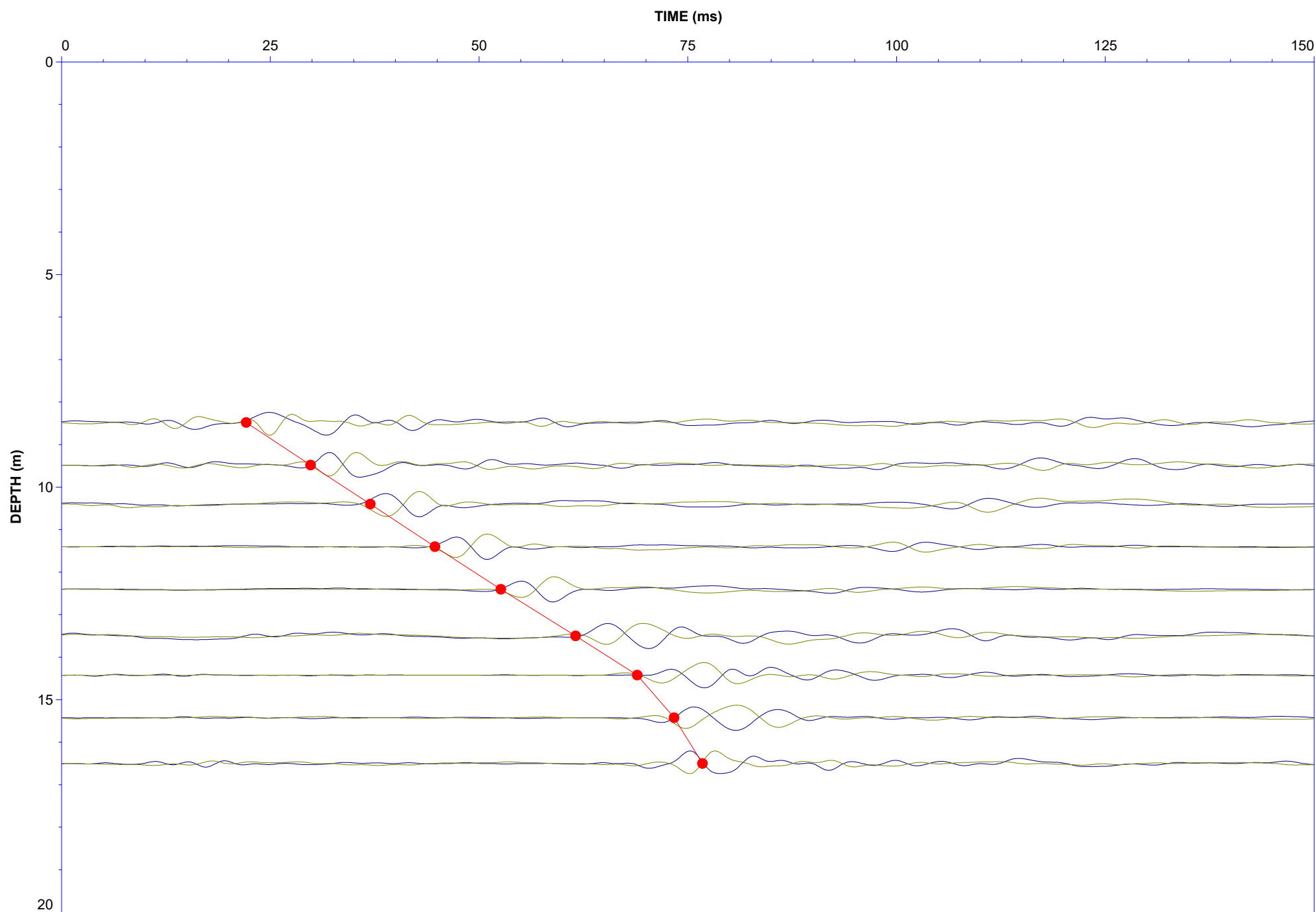
## Seismic Cone Penetration Test Shear Wave ( $V_s$ ) Traces











## Seismic Cone Penetration Test Compression Wave (Vp) Tabular Results



Job No: 23-05-25302  
Client: Golder Associates  
Project: Fraser Road CPTs  
Sounding ID: SCPT22-01C  
Date: 07-Feb-2023

Seismic Source: Plate  
Seismic Offset (m): 1.75  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> COMPRESSION WAVE VELOCITY TEST RESULTS - $V_p$***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
5.00	4.80	5.11			
6.00	5.80	6.06	0.95	0.48	1975
7.00	6.80	7.02	0.96	0.49	1963
8.00	7.80	7.99	0.97	0.50	1961
8.48	8.28	8.46	0.47	0.23	2080



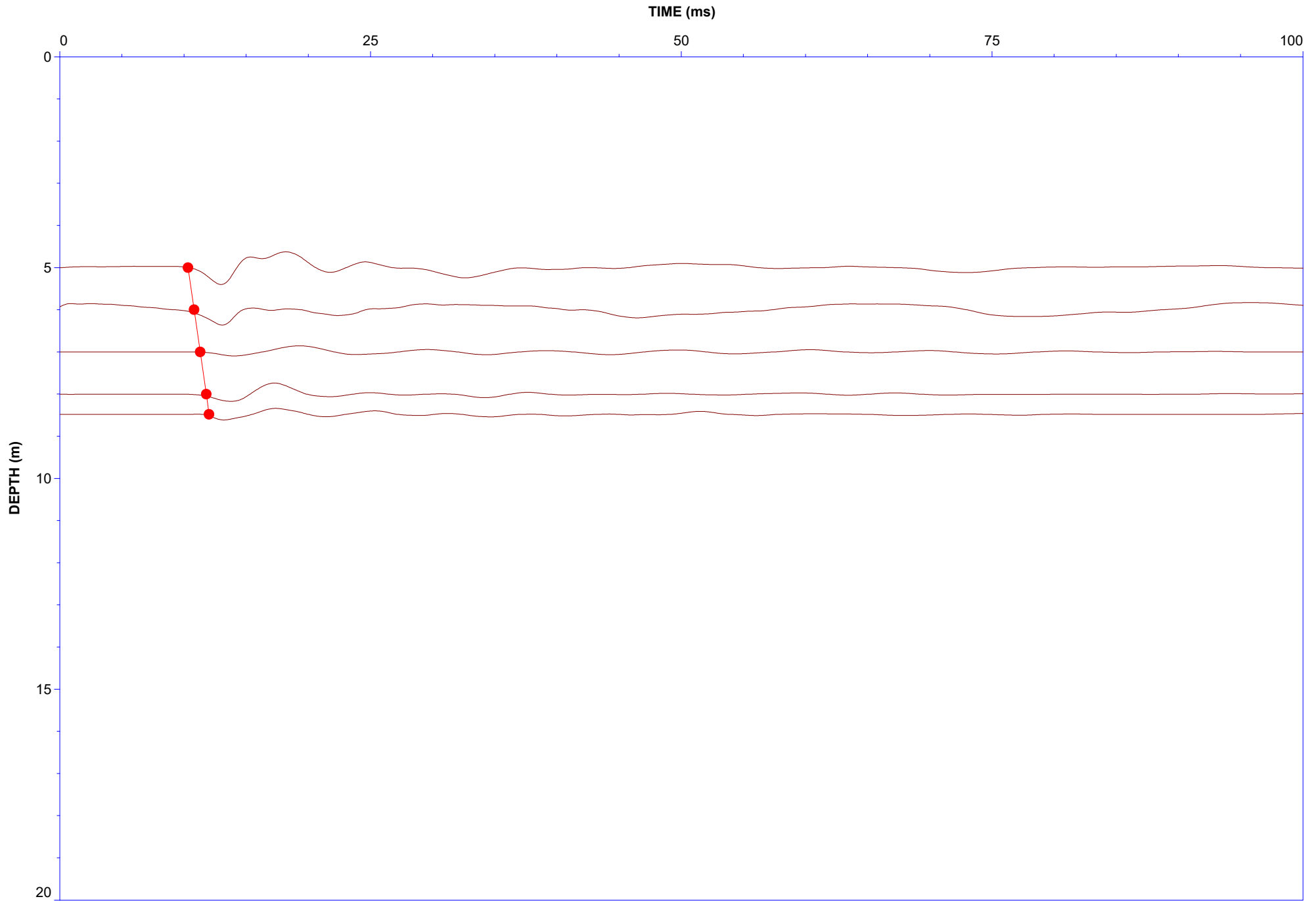
Job No: 23-05-25302  
Client: Golder Associates  
Project: Fraser Road CPTs  
Sounding ID: SCPT22-03C  
Date: 08-Feb-2023

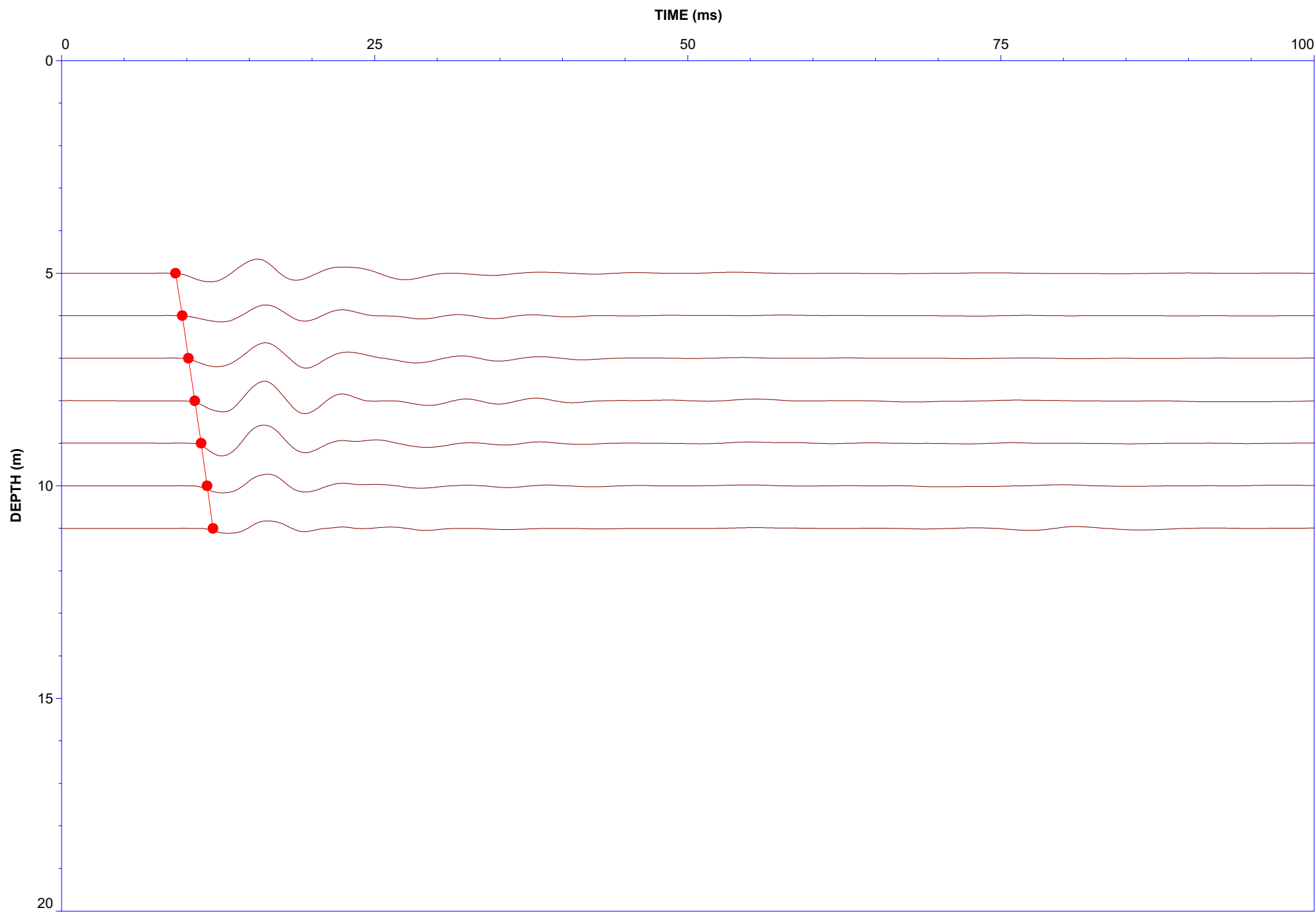
Seismic Source: Plate  
Seismic Offset (m): 1.45  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> COMPRESSION WAVE VELOCITY TEST RESULTS - $V_p$***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
5.00	4.80	5.01			
6.00	5.80	5.98	0.97	0.55	1750
7.00	6.80	6.95	0.97	0.50	1965
8.00	7.80	7.93	0.98	0.51	1938
9.00	8.80	8.92	0.99	0.50	1987
10.00	9.80	9.91	0.99	0.49	2034
11.00	10.80	10.90	0.99	0.47	2124

## Seismic Cone Penetration Test Compression Wave (Vp) Traces







## Seismic Cone Penetration Test Poisson's Ratio Tabular Results



Job No: 23-05-25302  
Client: Golder Associates  
Project: Fraser Road CPTs  
Sounding ID: SCPT22-01C  
Date: 07-Feb-2023

### ***SCPT<sub>u</sub> POISSON'S RATIO RESULTS***

Depth From (m)	Depth To (m)	Vs Interval Velocity (m/s)	Vp Interval Velocity (m/s)	Poisson's Ratio
4.80	5.80	143	1975	0.50
5.80	6.80	128	1963	0.50
6.80	7.80	132	1961	0.50
7.80	8.28	164	2080	0.50

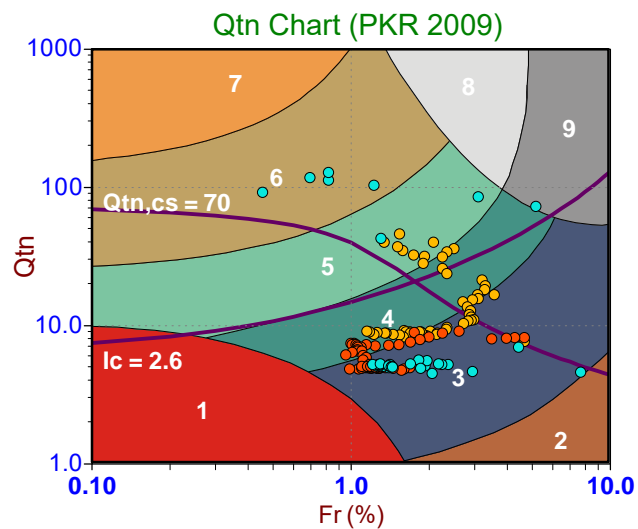


Job No: 23-05-25302  
Client: Golder Associates  
Project: Fraser Road CPTs  
Sounding ID: SCPT22-03C  
Date: 08-Feb-2023

### ***SCPT<sub>u</sub> POISSON'S RATIO RESULTS***

Depth From (m)	Depth To (m)	Vs Interval Velocity (m/s)	Vp Interval Velocity (m/s)	Poisson's Ratio
4.80	5.80	139	1750	0.50
5.80	6.80	126	1965	0.50
6.80	7.80	111	1938	0.50
7.80	8.80	110	1987	0.50
8.80	9.80	109	2034	0.50
9.80	10.80	141	2124	0.50

## Soil Behaviour Type (SBT) Scatter Plots

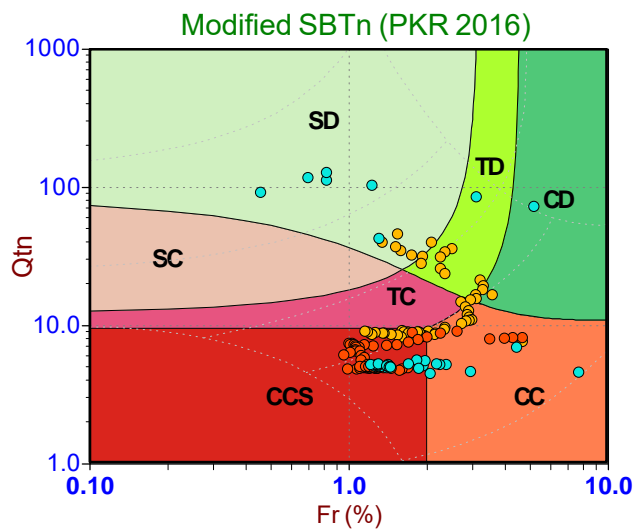


**Depth Ranges**

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

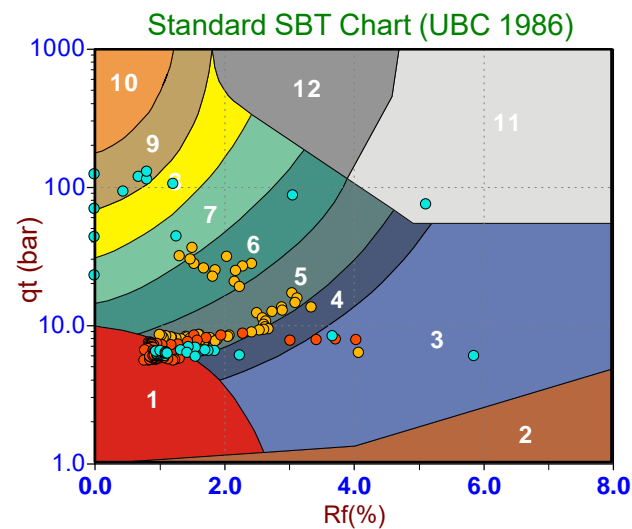
**Legend**

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



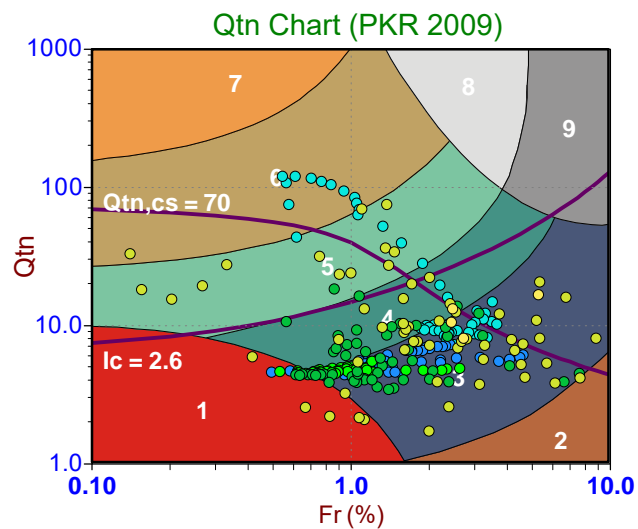
**Legend**

- 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

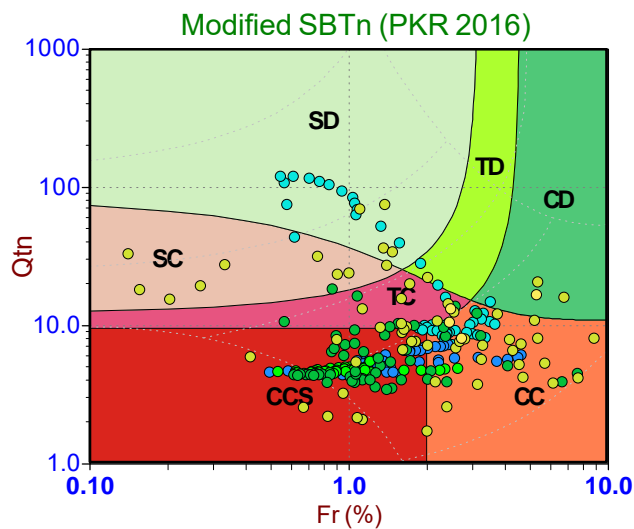


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- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

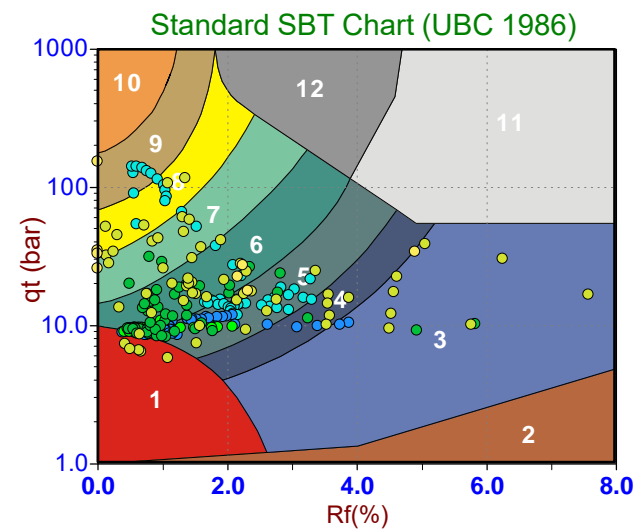
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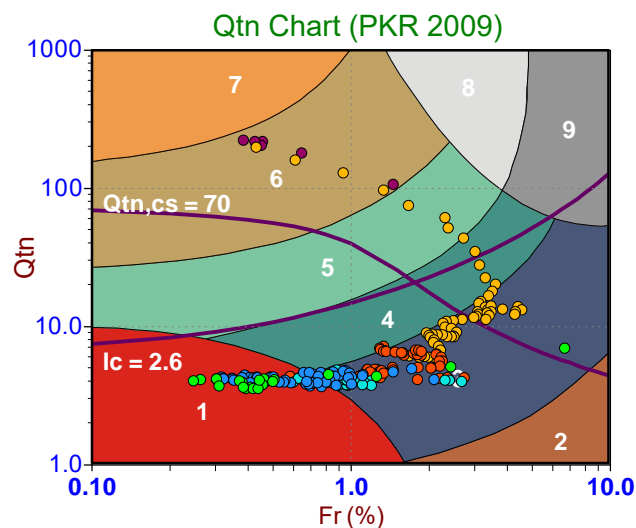
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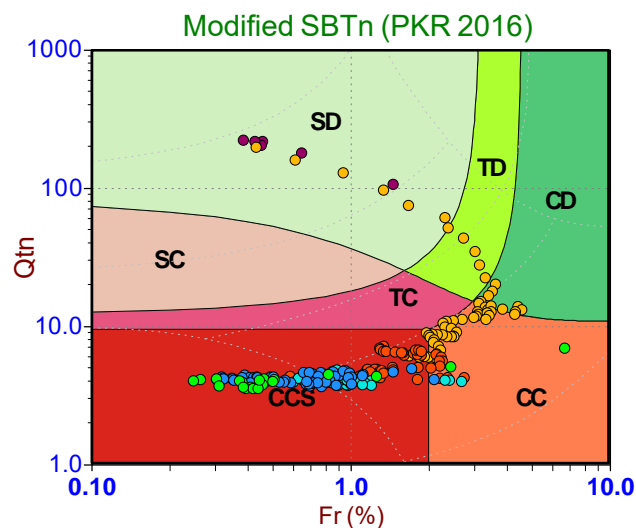


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- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

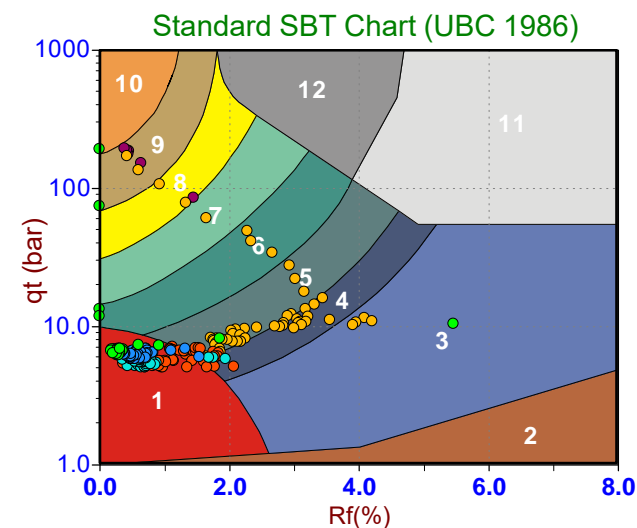
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- Sand Mixtures
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- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained



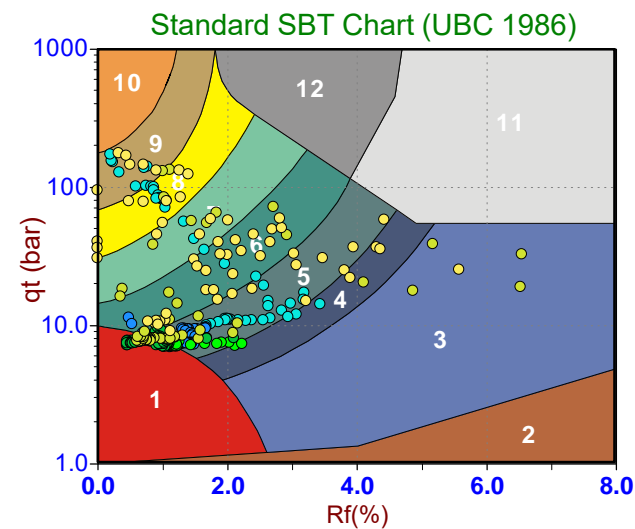
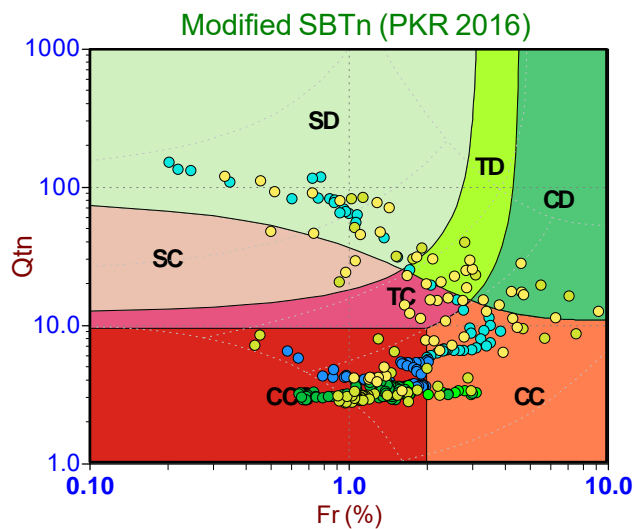
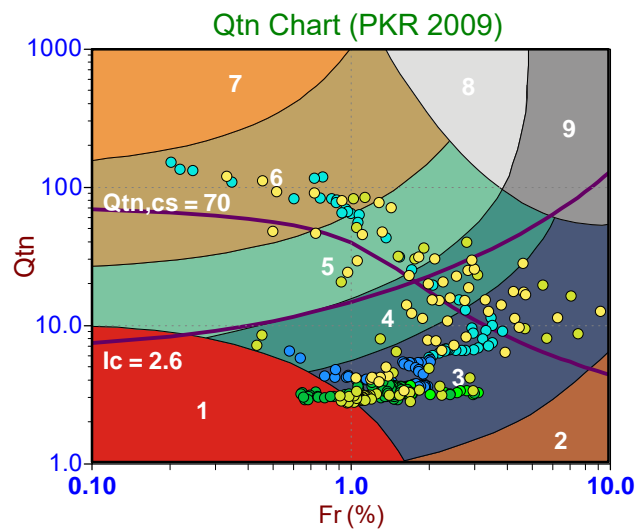
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- Cemented Sand



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

**Legend**

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- 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



## Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



Job No: 23-05-25302  
Client: Golder Associates  
Project: Fraser Road CPTs  
Start Date: 31-Jan-2023  
End Date: 08-Feb-2023

### CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm <sup>2</sup> )	Duration (s)	Test Depth (m)	U <sub>Initial</sub> (m)	U <sub>max</sub> (m)	U <sub>min</sub> (m)	U <sub>final</sub> (m)	Equilibrium Pore Pressure U <sub>eq</sub> (m)	Estimated Equilibrium Pore Pressure U <sub>eq</sub> (m)	Estimated Phreatic Surface (m)	Percent Dissipation (%)	t <sub>50</sub> (s) <sub>1</sub>	Assumed Rigidity Index (I <sub>r</sub> )	c <sub>h</sub> (cm <sup>2</sup> /min) <sub>2</sub>	Refer to Notation Number
SCPT22-01C	23-05-25302_SP01	15	275	5.000	22.1	25.4	11.0	21.0		1.8	3.2	18.7				3
SCPT22-01C	23-05-25302_SP01	15	180	6.000	41.4	41.4	25.8	32.0		2.8	3.2	24.6				3
SCPT22-01C	23-05-25302_SP01	15	3840	6.100	31.7	36.5	4.0	4.0		2.9	3.2	96.7	574	100	1.2	3
SCPT22-01C	23-05-25302_SP01	15	205	7.000	34.6	34.6	24.9	30.6		3.8	3.2	13.0				3
SCPT22-01C	23-05-25302_SP01	15	4680	8.000	31.5	37.3	19.1	19.1		4.8	3.2	56.1	2649	100	0.3	3
SCPT22-01C	23-05-25302_SP01	15	530	8.475	11.4	11.4	-9.5	5.3	5.3			100.0				
SCPT22-02C	23-05-25302_SP02	15	160	8.325	27.1	31.9	25.3	25.3		1.9	6.4	22.0				3
SCPT22-02C	23-05-25302_SP02	15	4080	9.375	31.0	36.2	3.8	3.8		3.0	6.4	97.5	593	100	1.2	3
SCPT22-02C	23-05-25302_SP02	15	160	10.375	40.5	45.4	35.1	43.2								
SCPT22-02C	23-05-25302_SP02	15	6000	11.375	51.0	52.3	25.9	25.9		5.0	6.4	55.7	4617	100	0.2	3
SCPT22-02C	23-05-25302_SP02	15	850	12.375	54.3	54.4	43.7	43.7		6.0	6.4	22.1				3
SCPT22-02C	23-05-25302_SP02	15	4080	13.350	35.5	36.8	9.3	9.3		7.0	6.4	92.2	714	100	1.0	3
SCPT22-02C	23-05-25302_SP02	15	600	14.350	23.1	24.8	8.5	8.5		8.0	6.4	96.6	66	100	10.6	3
SCPT22-02C	23-05-25302_SP02	15	840	15.175	16.3	22.2	-6.3	8.7	8.7			100.0	87	100	8.1	
SCPT22-03C	23-05-25302_SP03	15	3660	5.100	11.3	19.6	1.8	1.8	1.8		3.3	100.0	261	100	2.7	
SCPT22-03C	23-05-25302_SP03	15	6900	7.000	31.4	31.4	14.8	14.9		3.7	3.3	59.6	4512	100	0.2	3
SCPT22-03C	23-05-25302_SP03	15	5400	9.000	35.9	37.3	18.4	18.7		5.7	3.3	58.9	3434	100	0.2	3
SCPT22-03C	23-05-25302_SP03	15	1100	11.000	37.8	40.8	16.3	16.4		7.7	3.3	73.7	527	100	1.3	3
SCPT22-03C	23-05-25302_SP03	15	210	11.100	7.7	10.8	-9.5	10.5								
SCPT22-04C	23-05-25302_SP04	15	270	8.475	0.8	19.9	0.8	17.0		1.9	6.6	16.2				3
SCPT22-04C	23-05-25302_SP04	15	7740	9.475	37.9	37.9	4.6	4.6		2.9	6.6	95.0	1125	100	0.6	3
SCPT22-04C	23-05-25302_SP04	15	155	10.400	43.4	43.4	38.3	38.3		3.8	6.6	12.9				3
SCPT22-04C	23-05-25302_SP04	15	5520	11.400	45.9	47.2	23.1	23.1		4.8	6.6	56.8	3745	100	0.2	3
SCPT22-04C	23-05-25302_SP04	15	780	12.400	46.1	48.0	36.5	36.5		5.8	6.6	27.4				3
SCPT22-04C	23-05-25302_SP04	15	6660	13.500	47.9	48.0	23.9	23.9		6.9	6.6	58.7	4327	100	0.2	3
SCPT22-04C	23-05-25302_SP04	15	8760	15.425	0.5	19.5	-9.9	8.8	8.8		6.6	100.0	790	100	0.9	
SCPT22-04C	23-05-25302_SP04	15	565	16.500	41.0	43.2	16.8	16.8		9.9	6.6	79.2	203	100	3.5	3

1. Time for 50 percent dissipation based in U<sub>max</sub>, U<sub>min</sub>, and the applied U<sub>eq</sub>. Note the time is relative to where U<sub>max</sub> occurred.

2. Houlby and Teh, 1991.

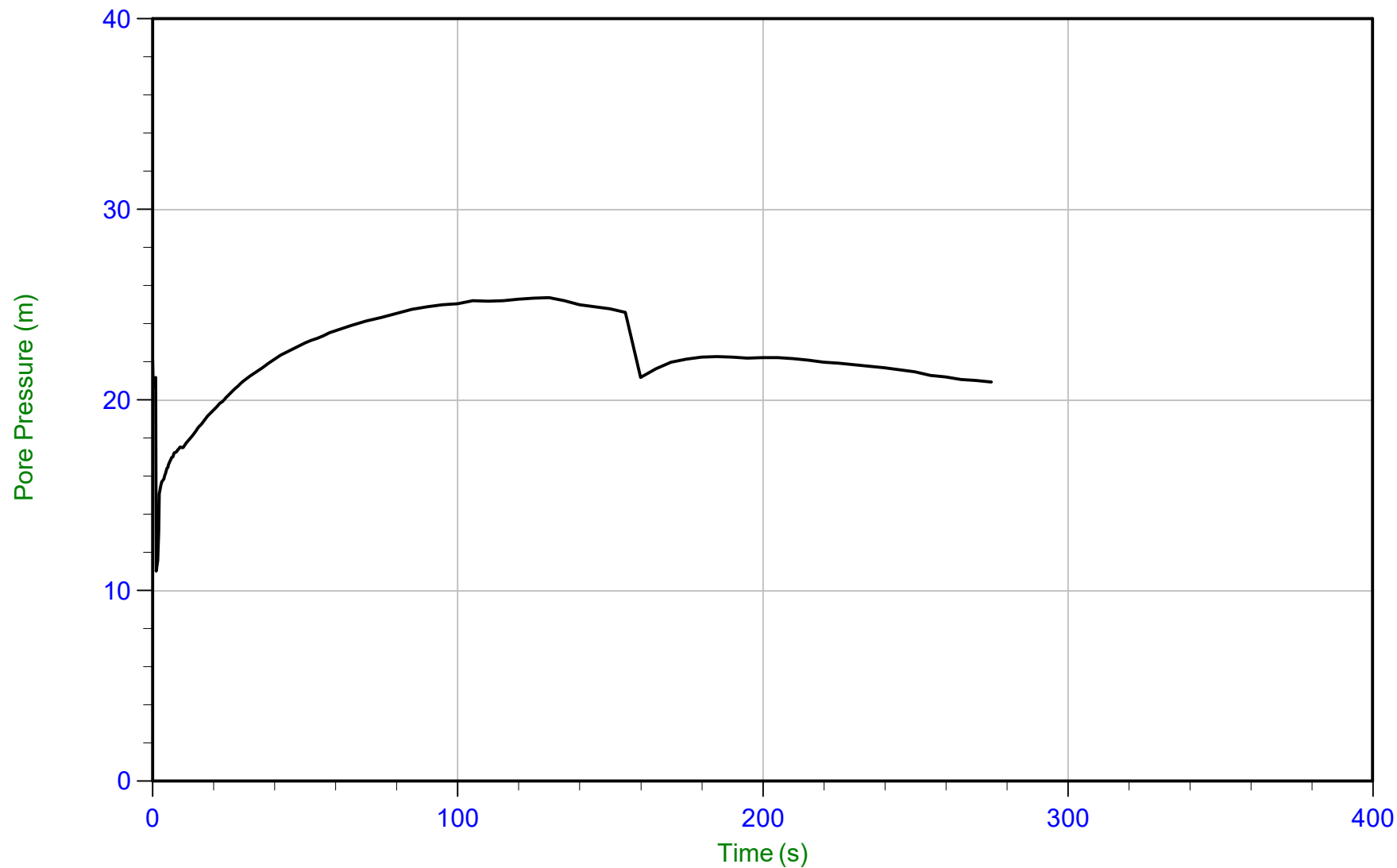
3. Equilibrium pore pressure estimated based on a hydrostatic assumption from the assumed phreatic surface.



*Golder*

Job No: 23-05-25302  
Date: 02/07/2023 10:24  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-01C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP01.PPF2  
Depth: 5.000 m / 16.404 ft  
Duration: 275.0 s

u Min: 11.0 m  
u Max: 25.4 m  
u Final: 21.0 m

WT: 3.200 m / 10.499 ft  
Ueq: 1.8 m



*Golder*

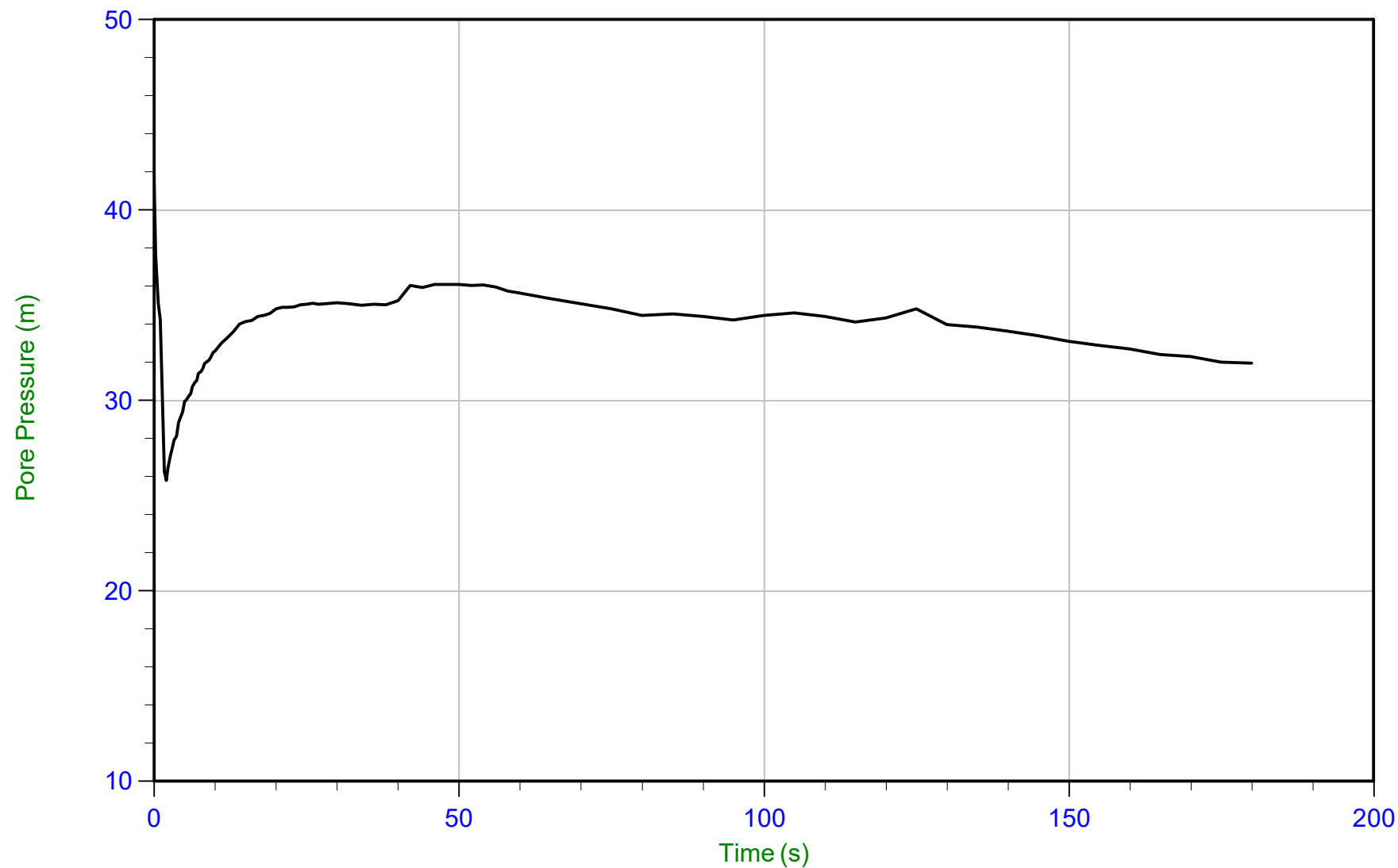
Job No: 23-05-25302

Date: 02/07/2023 10:24

Site: Fraser Road, South Glengarry

Sounding: SCPT22-01C

Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP01.PPF2

Depth: 6.000 m / 19.685 ft

Duration: 180.0 s

u Min: 25.8 m

u Max: 41.4 m

u Final: 32.0 m

WT: 3.200 m / 10.499 ft

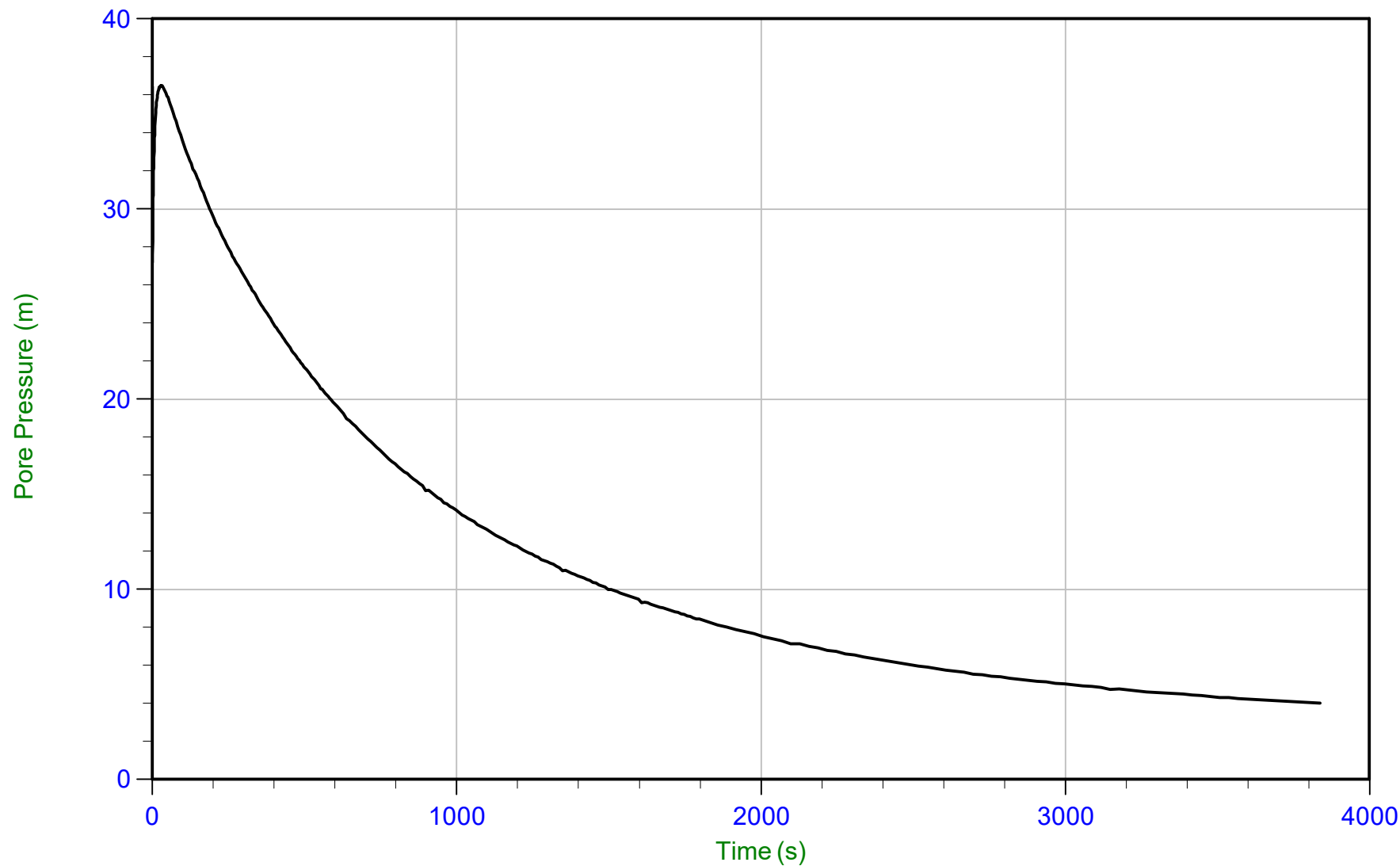
Ueq: 2.8 m



*Golder*

Job No: 23-05-25302  
Date: 02/07/2023 10:24  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-01C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP01.PPF2  
Depth: 6.100 m / 20.013 ft  
Duration: 3840.0 s

u Min: 4.0 m  
u Max: 36.5 m  
u Final: 4.0 m

WT: 3.200 m / 10.499 ft  
Ueq: 2.9 m  
U(50): 19.70 m

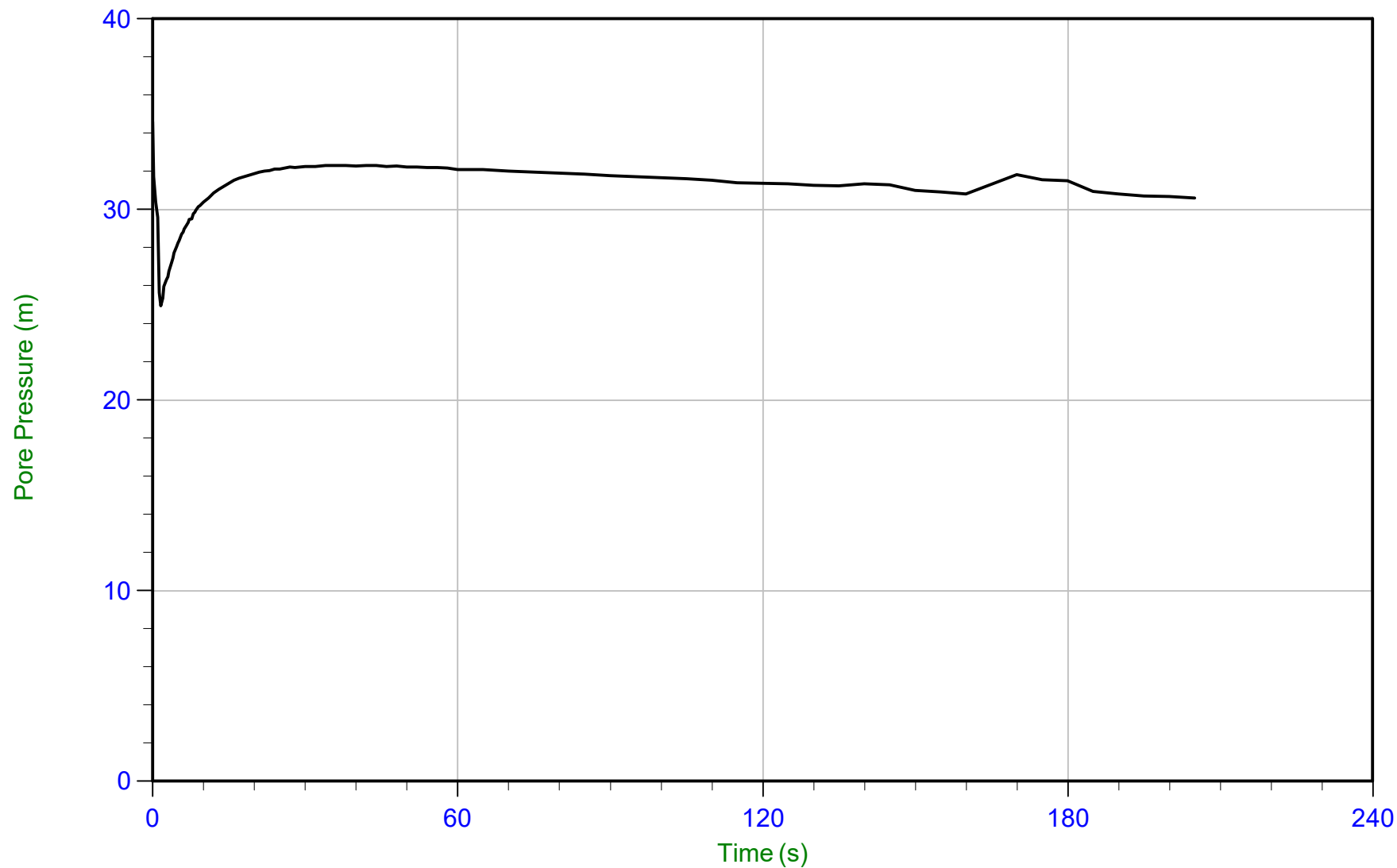
T(50): 574.1 s  
Ir: 100  
Ch: 1.2 cm<sup>2</sup>/min



*Golder*

Job No: 23-05-25302  
Date: 02/07/2023 10:24  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-01C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP01.PPF2  
Depth: 7.000 m / 22.966 ft  
Duration: 205.0 s

u Min: 24.9 m  
u Max: 34.6 m  
u Final: 30.6 m

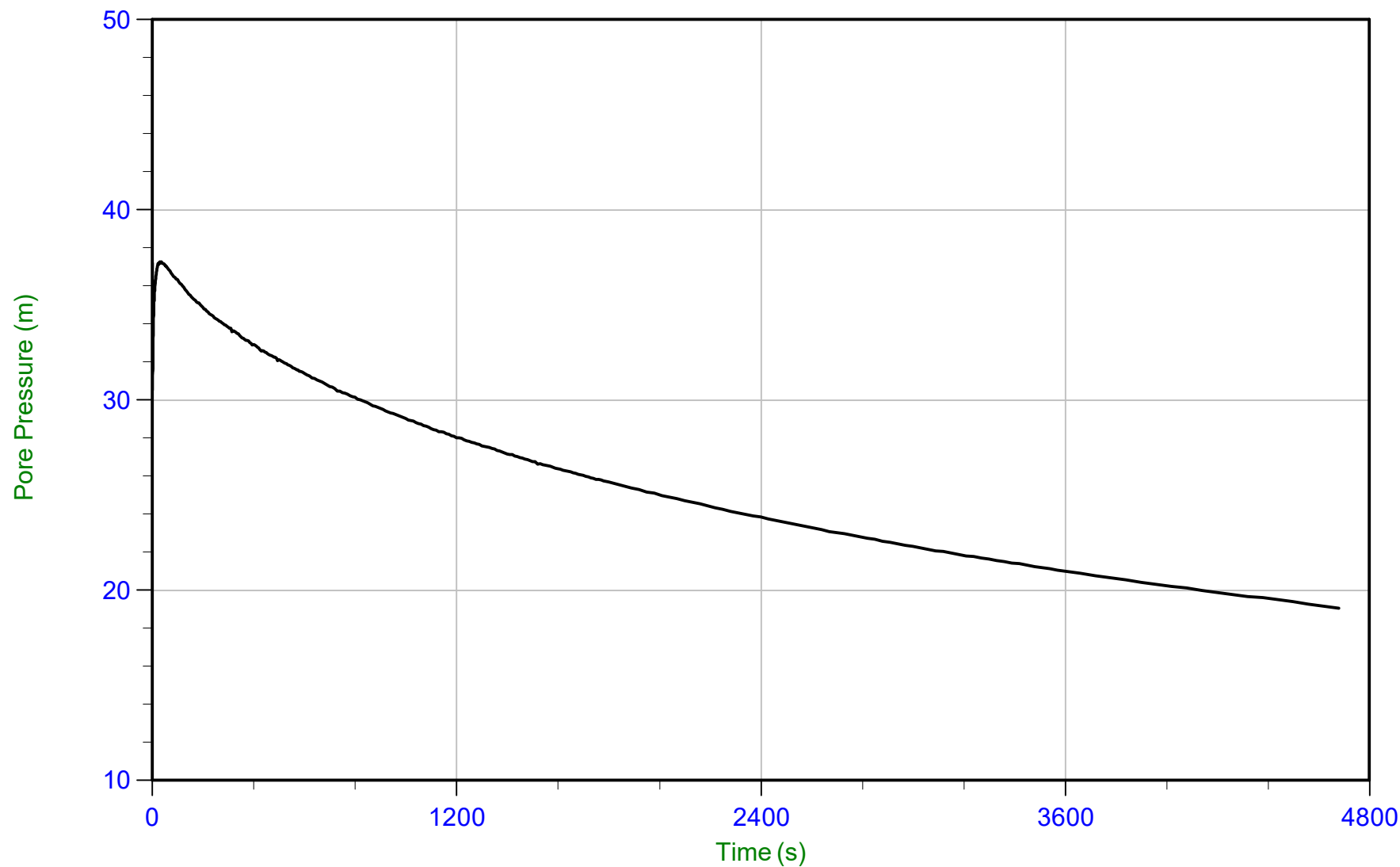
WT: 3.200 m / 10.499 ft  
Ueq: 3.8 m



*Golder*

Job No: 23-05-25302  
Date: 02/07/2023 10:24  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-01C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP01.PPF2  
Depth: 8.000 m / 26.246 ft  
Duration: 4680.0 s

u Min: 19.1 m  
u Max: 37.3 m  
u Final: 19.1 m

WT: 3.200 m / 10.499 ft  
Ueq: 4.8 m  
U(50): 23.15 m

T(50): 2649.2 s  
Ir: 100  
Ch: 0.3 cm<sup>2</sup>/min



*Golder*

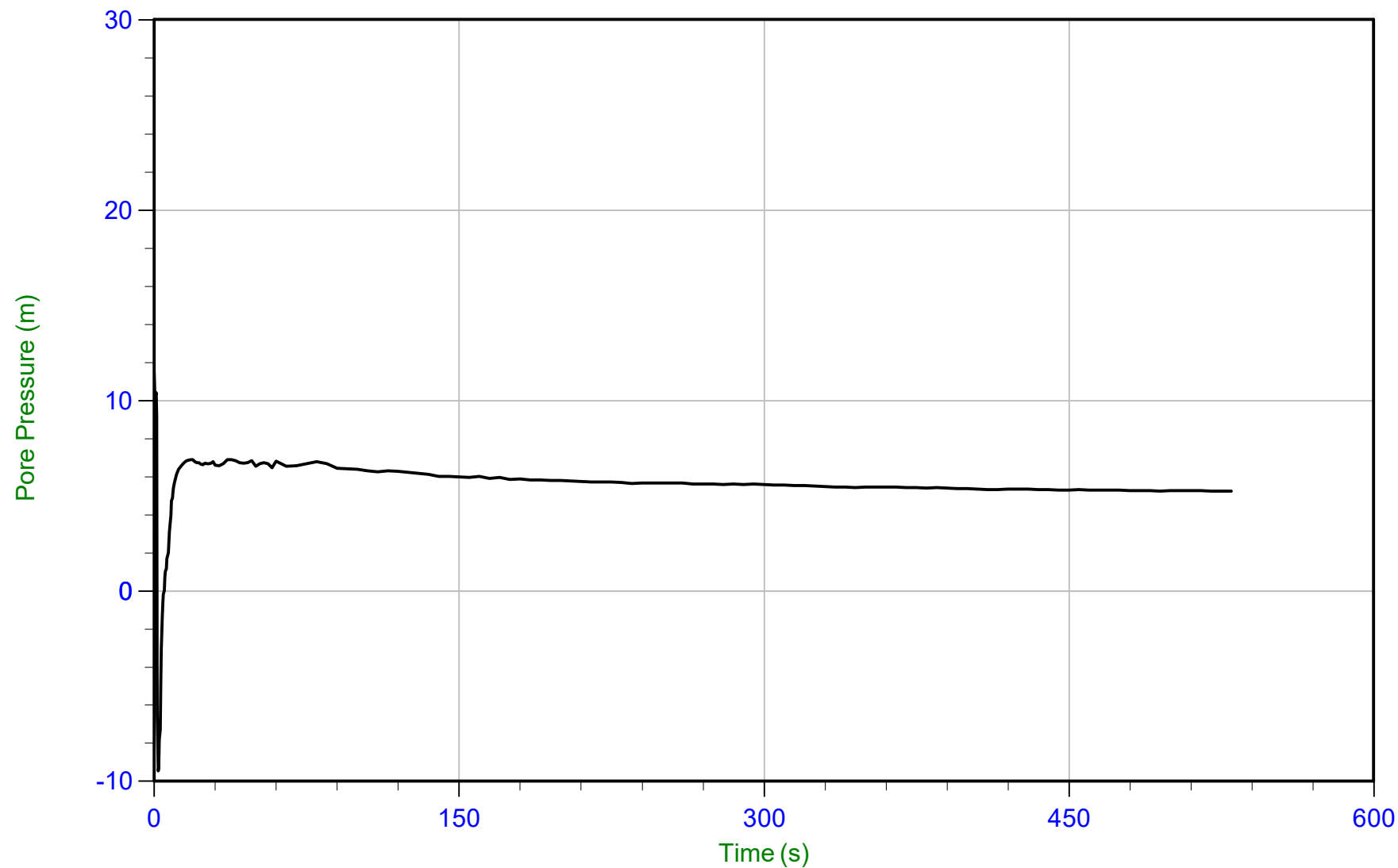
Job No: 23-05-25302

Date: 02/07/2023 10:24

Site: Fraser Road, South Glengarry

Sounding: SCPT22-01C

Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP01.PPF2

Depth: 8.475 m / 27.805 ft

Duration: 530.0 s

u Min: -9.5 m

u Max: 11.4 m

u Final: 5.3 m

WT: 3.212 m / 10.538 ft

Ueq: 5.3 m

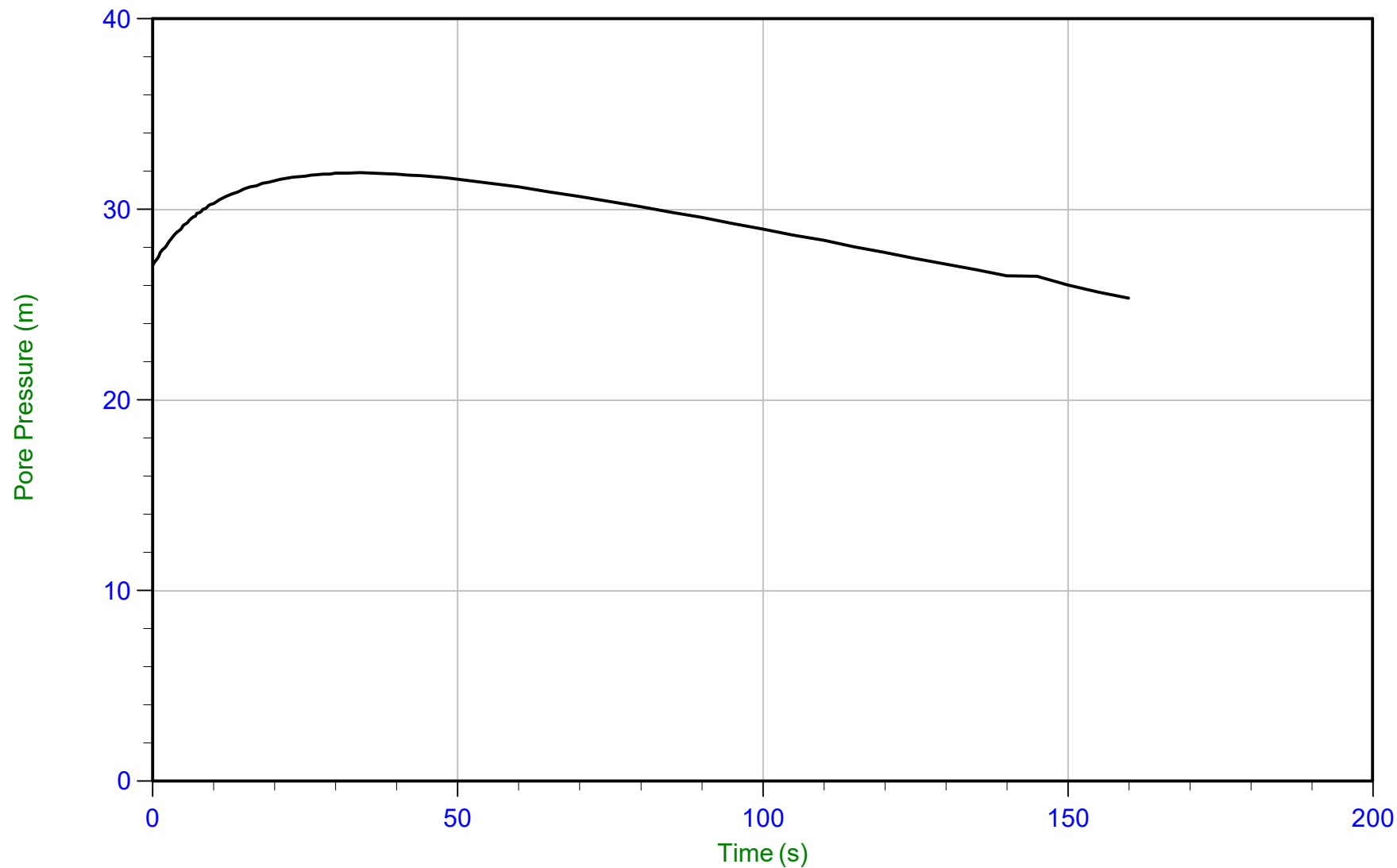




*Golder*

Job No: 23-05-25302  
Date: 02/03/2023 10:46  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-02C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP02.PPF2  
Depth: 8.325 m / 27.313 ft  
Duration: 160.0 s

u Min: 25.3 m  
u Max: 31.9 m  
u Final: 25.3 m

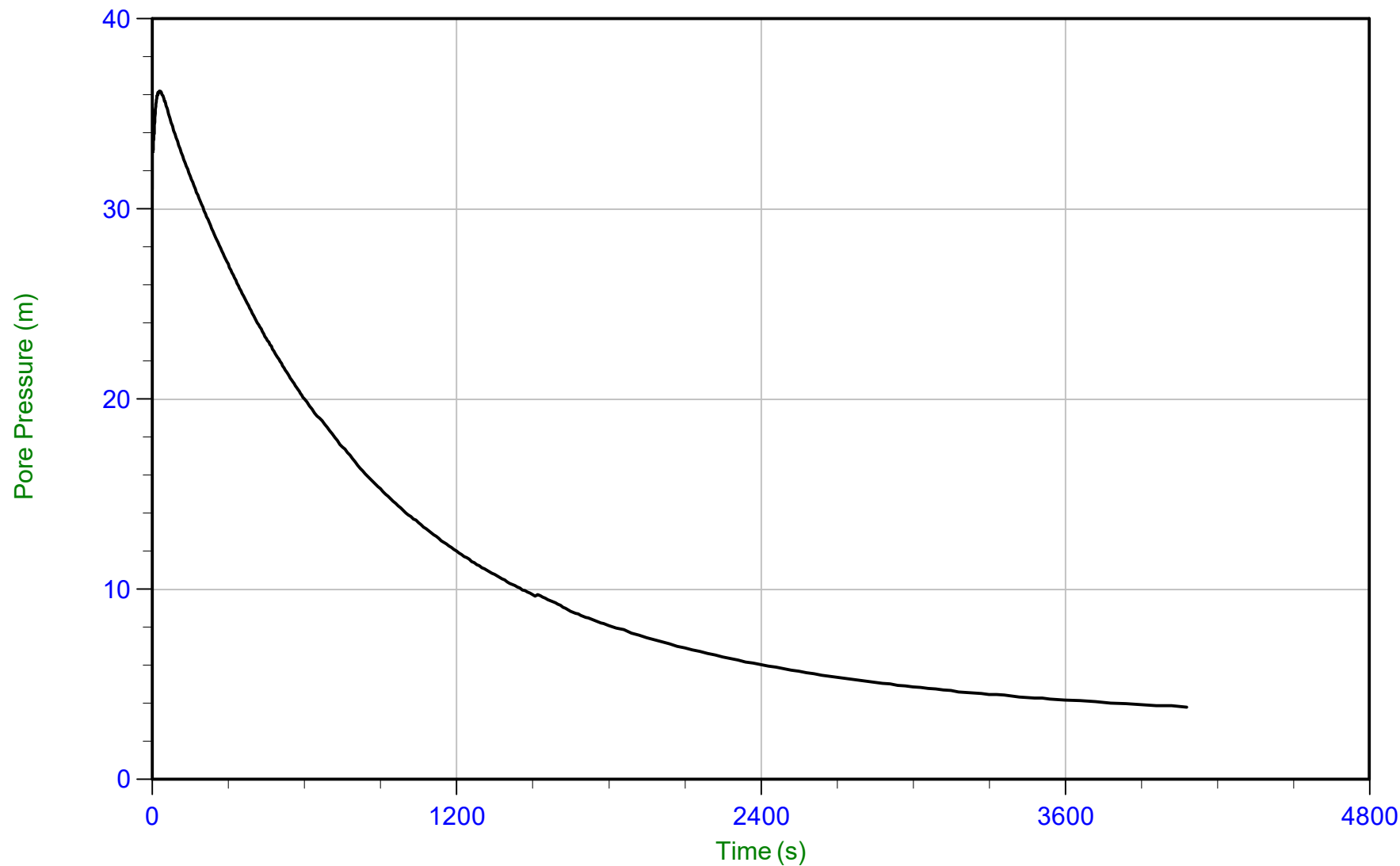
WT: 6.400 m / 20.997 ft  
Ueq: 1.9 m



*Golder*

Job No: 23-05-25302  
Date: 02/03/2023 10:46  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-02C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP02.PPF2  
Depth: 9.375 m / 30.758 ft  
Duration: 4080.0 s

u Min: 3.8 m  
u Max: 36.2 m  
u Final: 3.8 m

WT: 6.400 m / 20.997 ft  
Ueq: 3.0 m  
U(50): 19.59 m

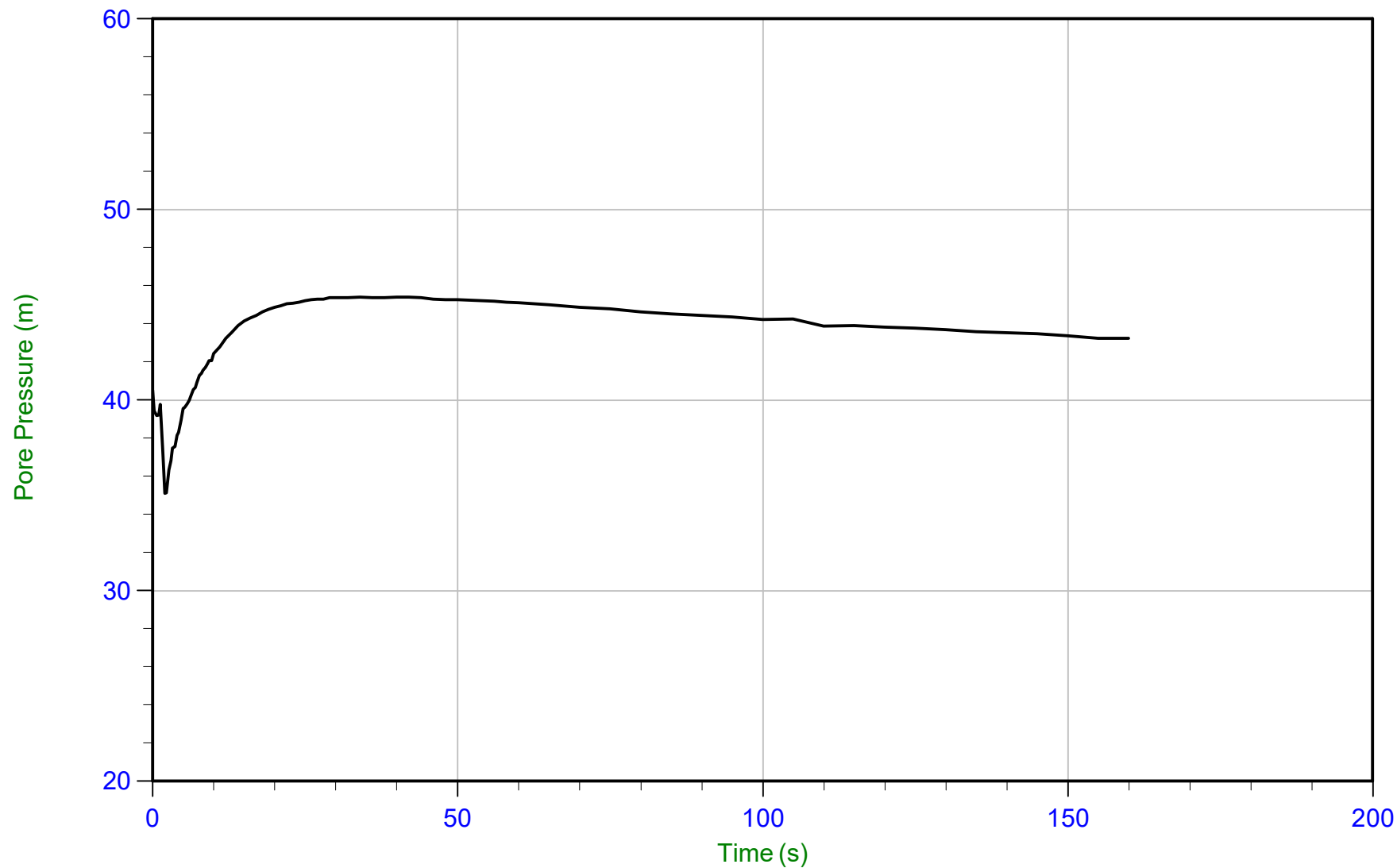
T(50): 592.8 s  
Ir: 100  
Ch: 1.2 cm<sup>2</sup>/min



*Golder*

Job No: 23-05-25302  
Date: 02/03/2023 10:46  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-02C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP02.PPF2  
Depth: 10.375 m / 34.038 ft  
Duration: 160.0 s

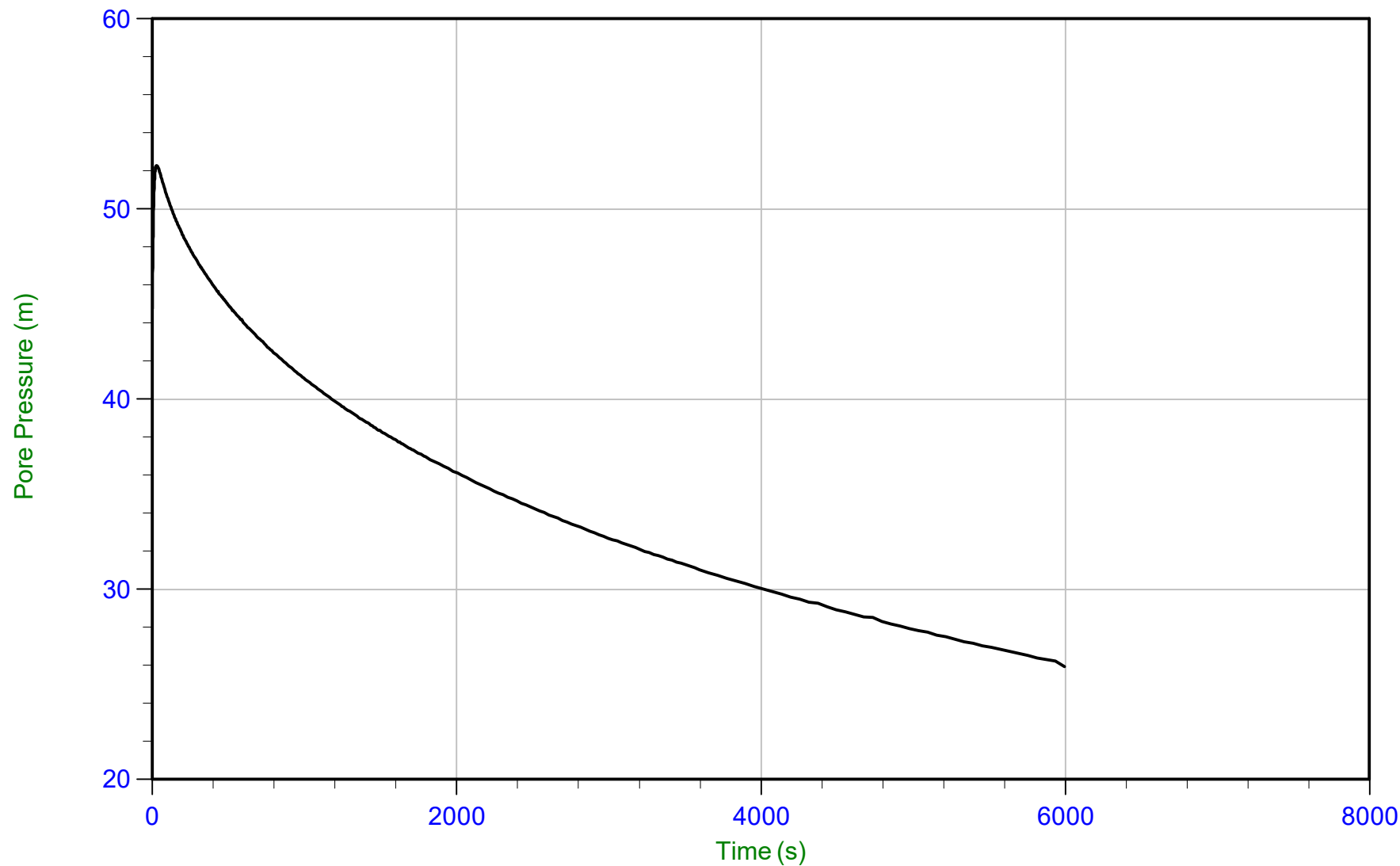
u Min: 35.1 m  
u Max: 45.4 m  
u Final: 43.2 m



*Golder*

Job No: 23-05-25302  
Date: 02/03/2023 10:46  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-02C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP02.PPF2  
Depth: 11.375 m / 37.319 ft  
Duration: 6000.0 s

u Min: 25.9 m  
u Max: 52.3 m  
u Final: 25.9 m

WT: 6.400 m / 20.997 ft  
Ueq: 5.0 m  
U(50): 28.63 m

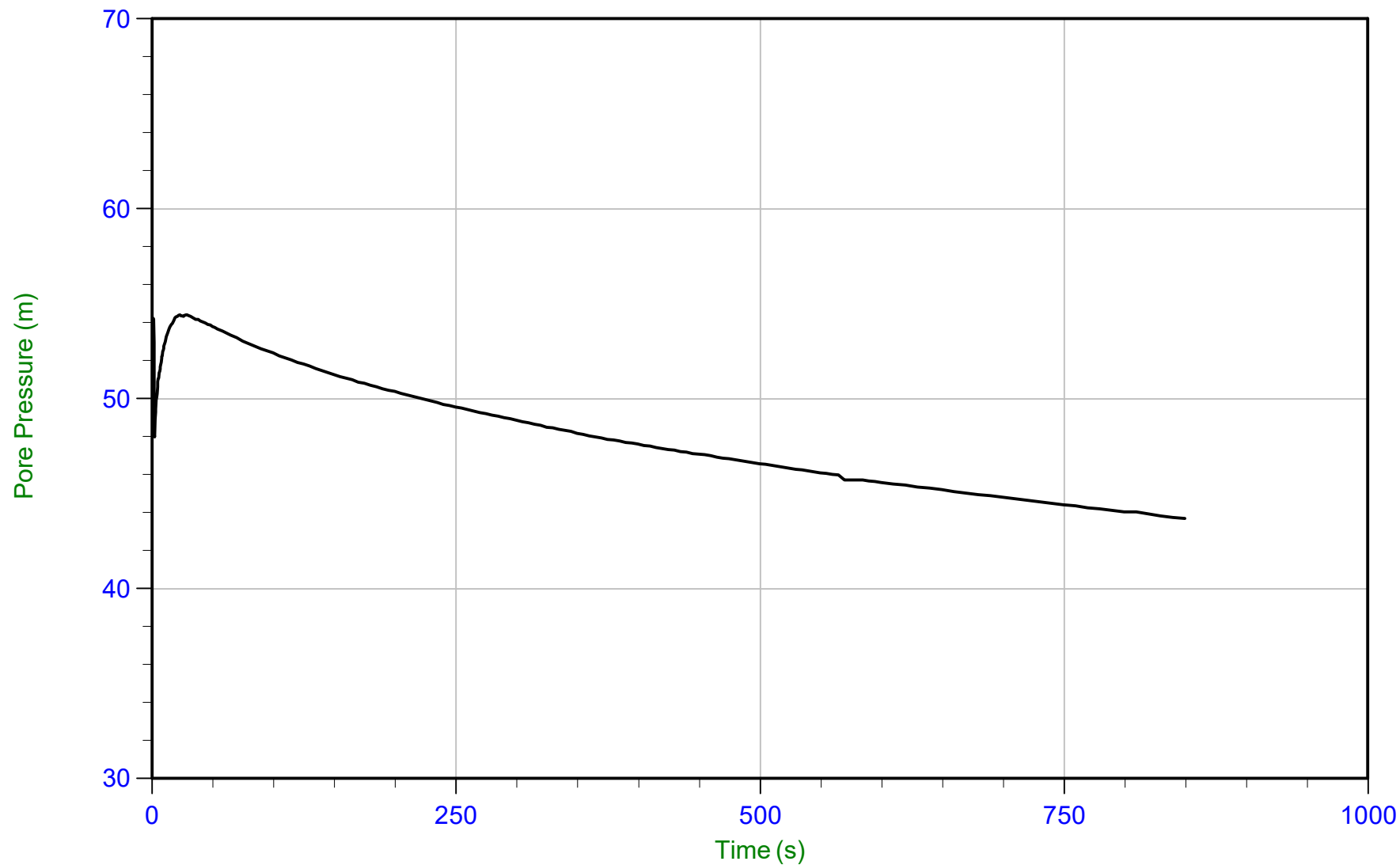
T(50): 4616.6 s  
Ir: 100  
Ch: 0.2 cm<sup>2</sup>/min



*Golder*

Job No: 23-05-25302  
Date: 02/03/2023 10:46  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-02C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP02.PPF2  
Depth: 12.375 m / 40.600 ft  
Duration: 850.0 s

u Min: 43.7 m  
u Max: 54.4 m  
u Final: 43.7 m

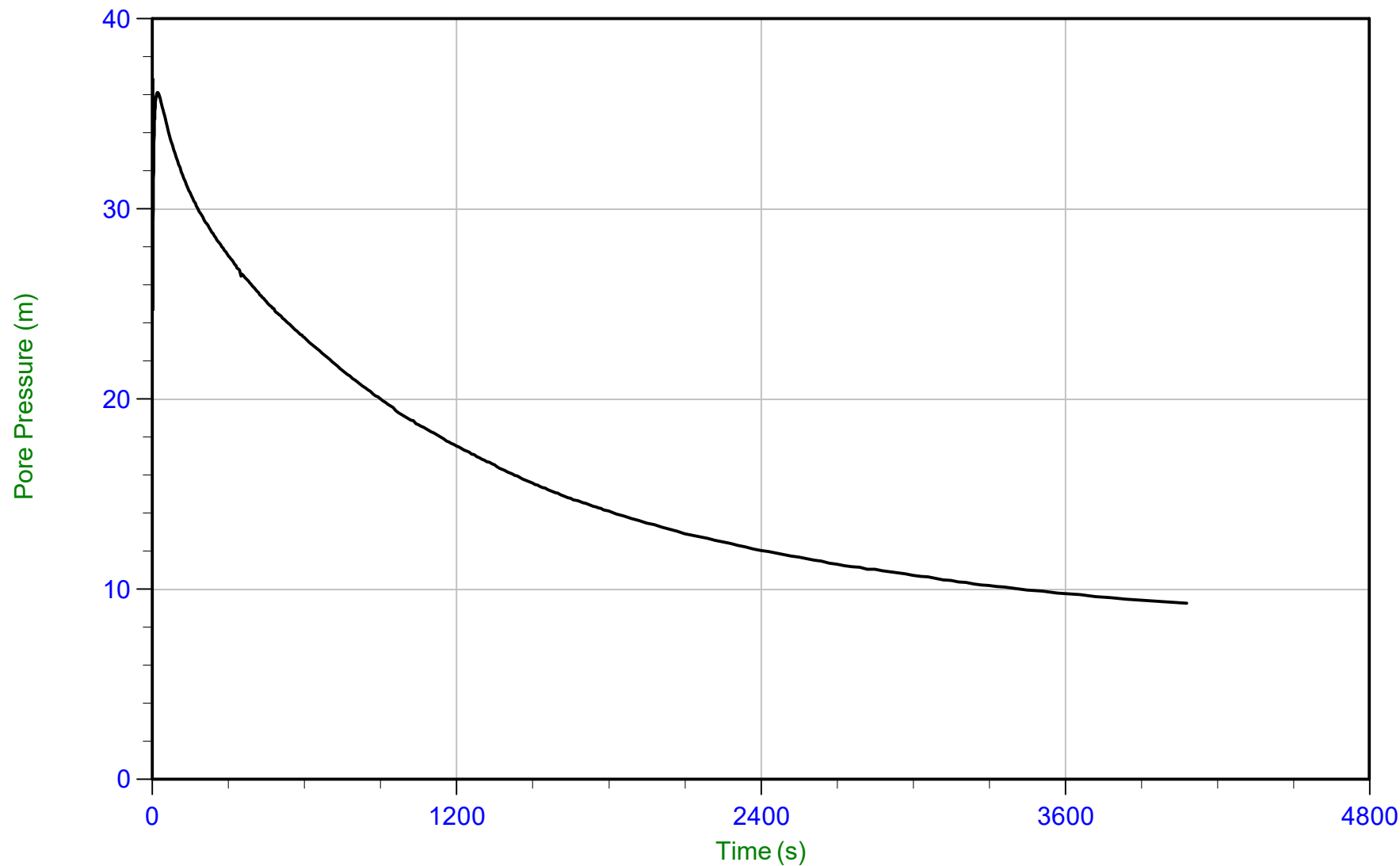
WT: 6.400 m / 20.997 ft  
Ueq: 6.0 m



*Golder*

Job No: 23-05-25302  
Date: 02/03/2023 10:46  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-02C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP02.PPF2  
Depth: 13.350 m / 43.799 ft  
Duration: 4080.0 s

u Min: 9.3 m  
u Max: 36.8 m  
u Final: 9.3 m

WT: 6.400 m / 20.997 ft  
Ueq: 7.0 m  
U(50): 21.88 m

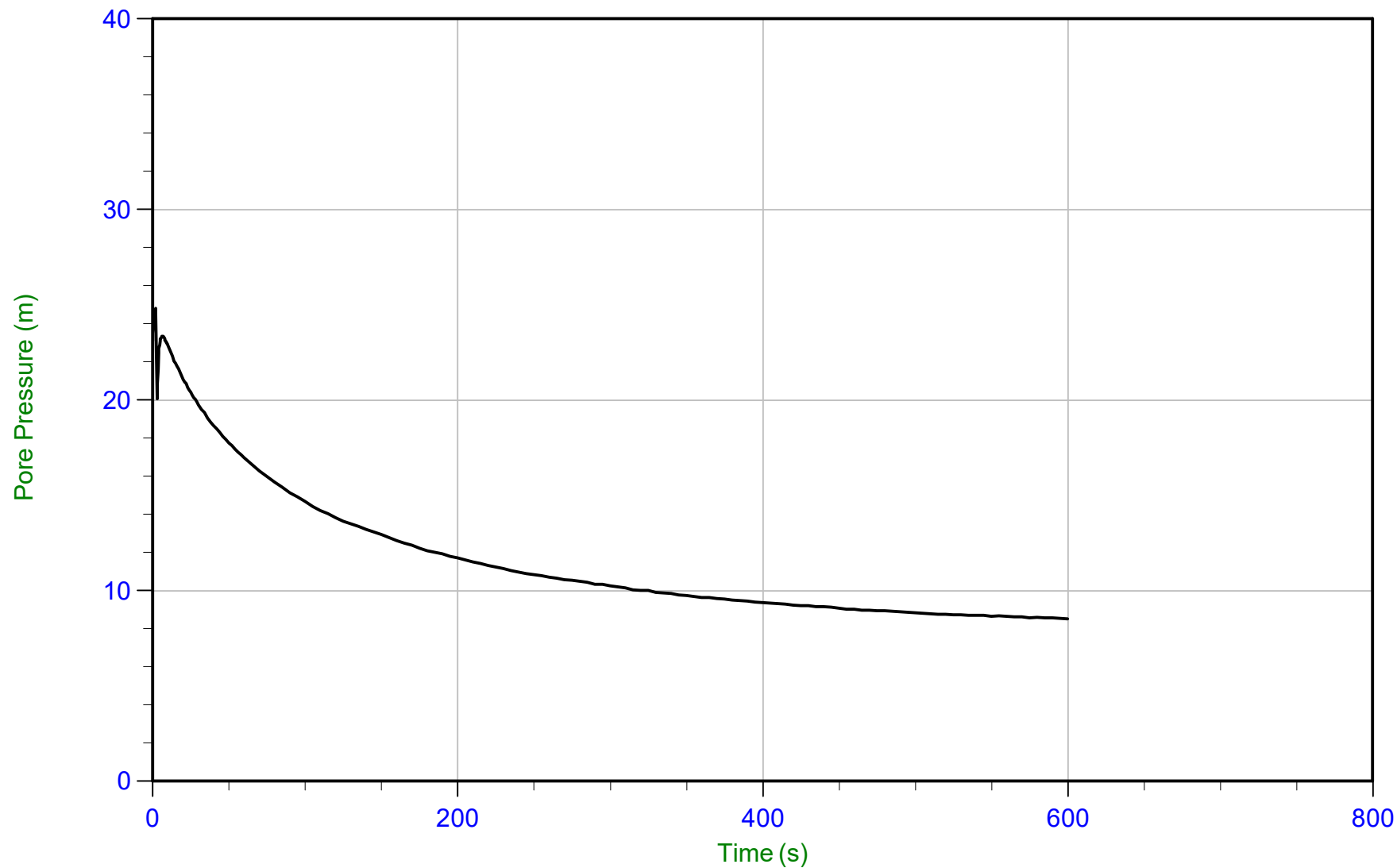
T(50): 714.4 s  
Ir: 100  
Ch: 1.0 cm<sup>2</sup>/min



*Golder*

Job No: 23-05-25302  
Date: 02/03/2023 10:46  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-02C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP02.PPF2  
Depth: 14.350 m / 47.079 ft  
Duration: 600.0 s

u Min: 8.5 m  
u Max: 24.8 m  
u Final: 8.5 m

WT: 6.400 m / 20.997 ft  
Ueq: 8.0 m  
U(50): 16.39 m

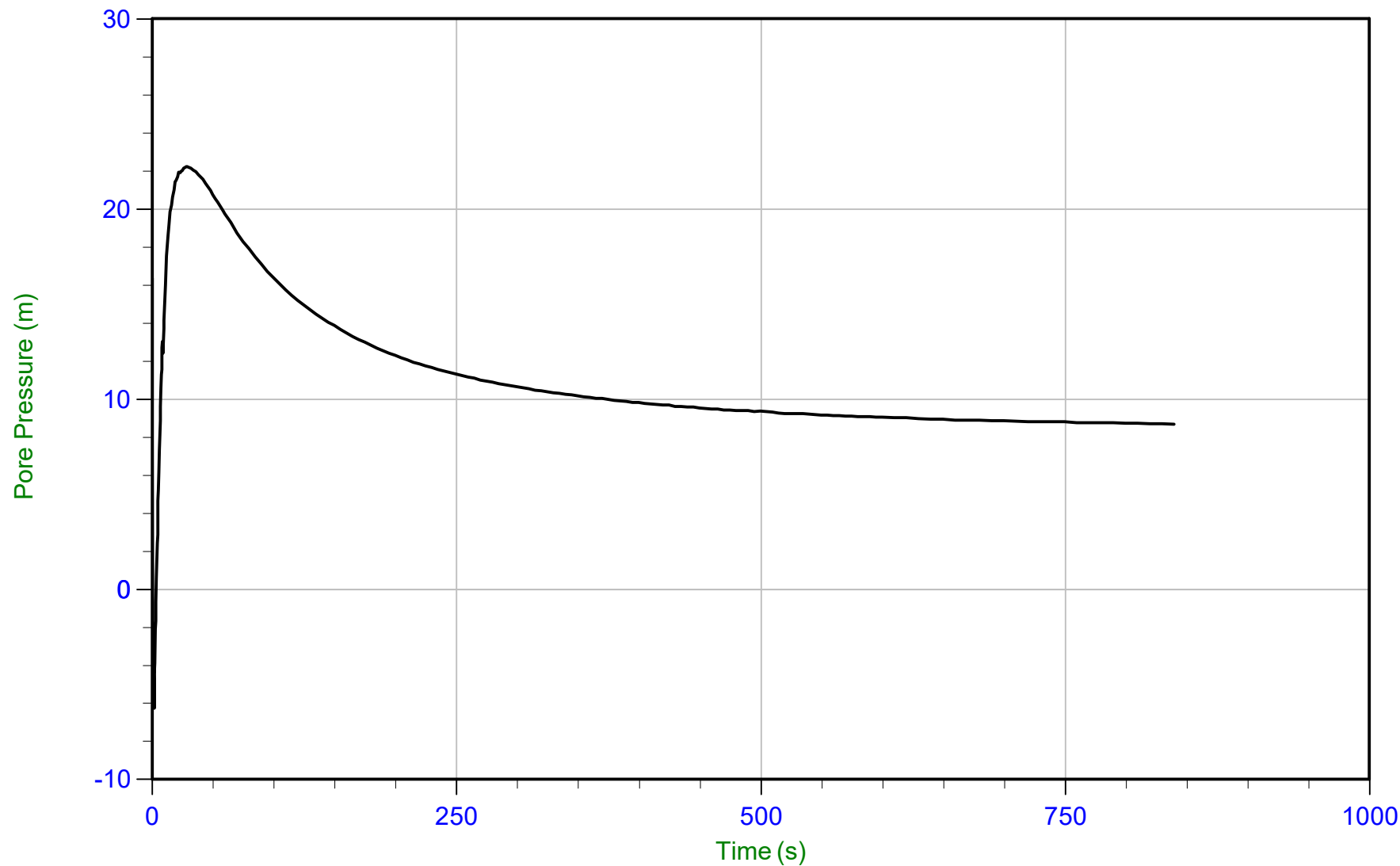
T(50): 66.4 s  
Ir: 100  
Ch: 10.6 cm<sup>2</sup>/min



*Golder*

Job No: 23-05-25302  
Date: 02/03/2023 10:46  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-02C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP02.PPF2  
Depth: 15.175 m / 49.786 ft  
Duration: 840.0 s

u Min: -6.3 m  
u Max: 22.2 m  
u Final: 8.7 m

WT: 6.438 m / 21.122 ft  
Ueq: 8.7 m  
U(50): 15.48 m

T(50): 86.6 s  
Ir: 100  
Ch: 8.1 cm<sup>2</sup>/min

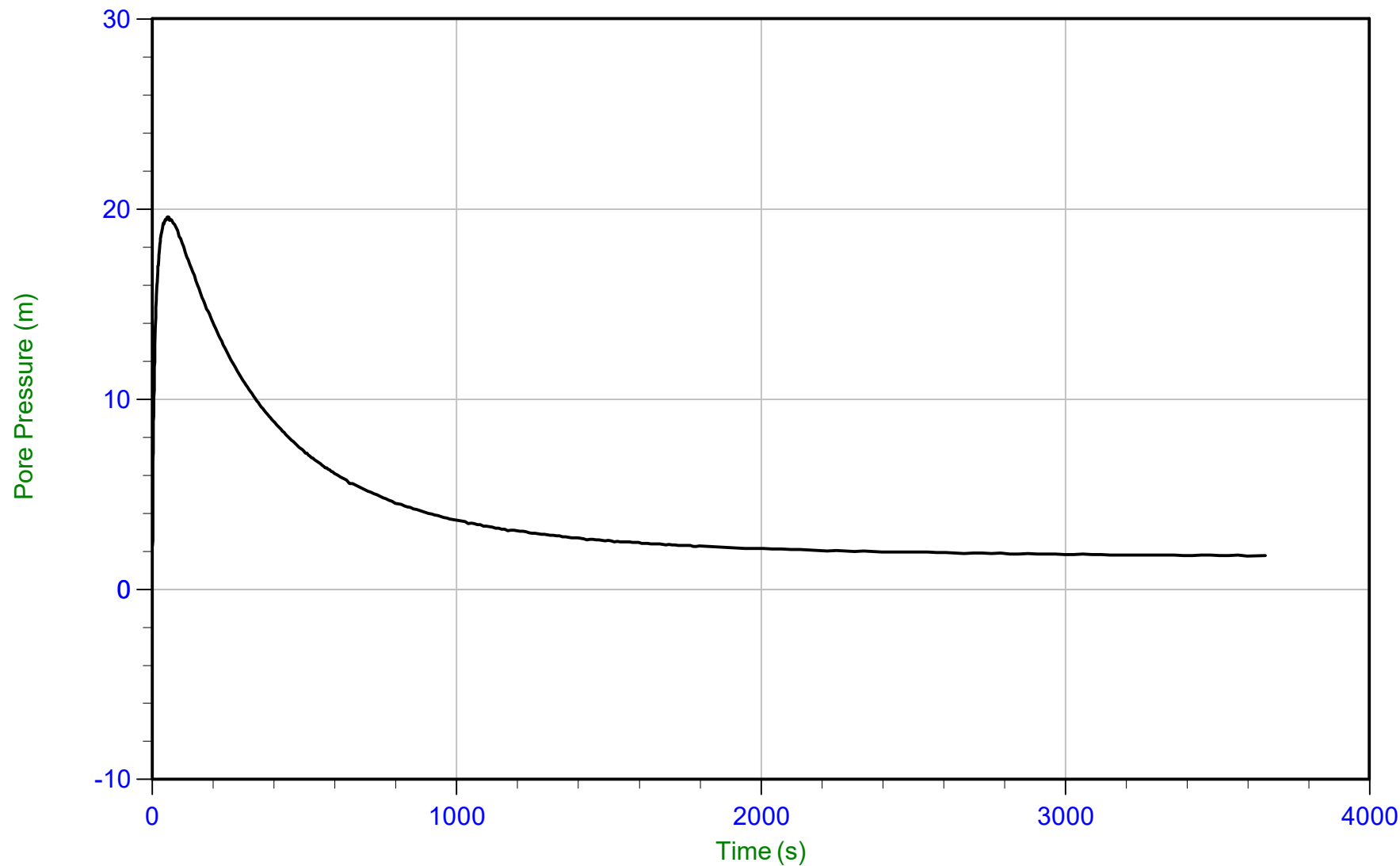




*Golder*

Job No: 23-05-25302  
Date: 02/08/2023 08:08  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-03C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP03.PPF2  
Depth: 5.100 m / 16.732 ft  
Duration: 3660.0 s

u Min: 1.8 m  
u Max: 19.6 m  
u Final: 1.8 m

WT: 3.311 m / 10.863 ft  
Ueq: 1.8 m  
U(50): 10.68 m

T(50): 260.5 s  
Ir: 100  
Ch: 2.7 cm<sup>2</sup>/min



*Golder*

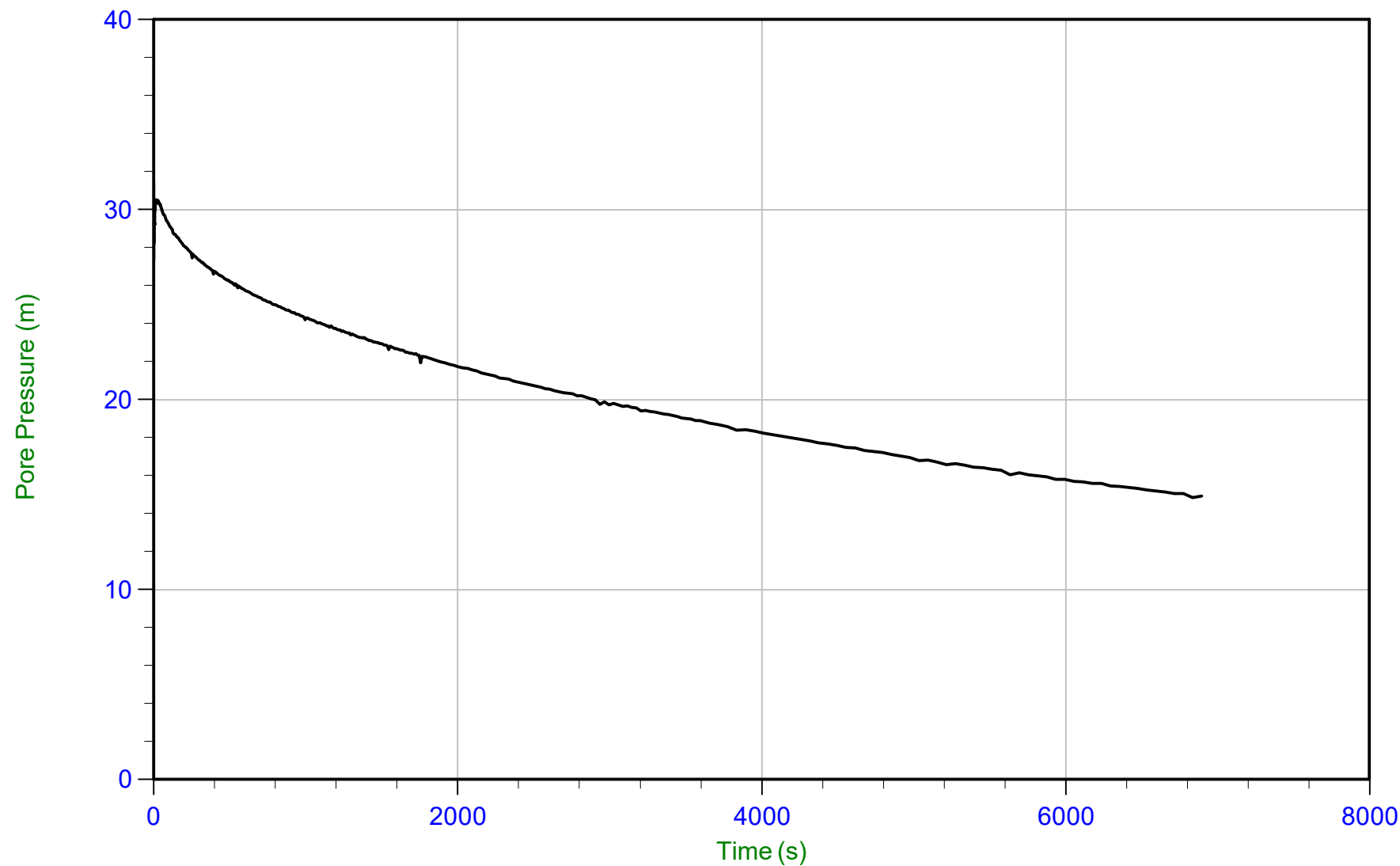
Job No: 23-05-25302

Date: 02/08/2023 08:08

Site: Fraser Road, South Glengarry

Sounding: SCPT22-03C

Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP03.PPF2

Depth: 7.000 m / 22.966 ft

Duration: 6900.0 s

u Min: 14.8 m

u Max: 31.4 m

u Final: 14.9 m

WT: 3.300 m / 10.827 ft

Ueq: 3.7 m

U(50): 17.57 m

T(50): 4511.6 s

Ir: 100

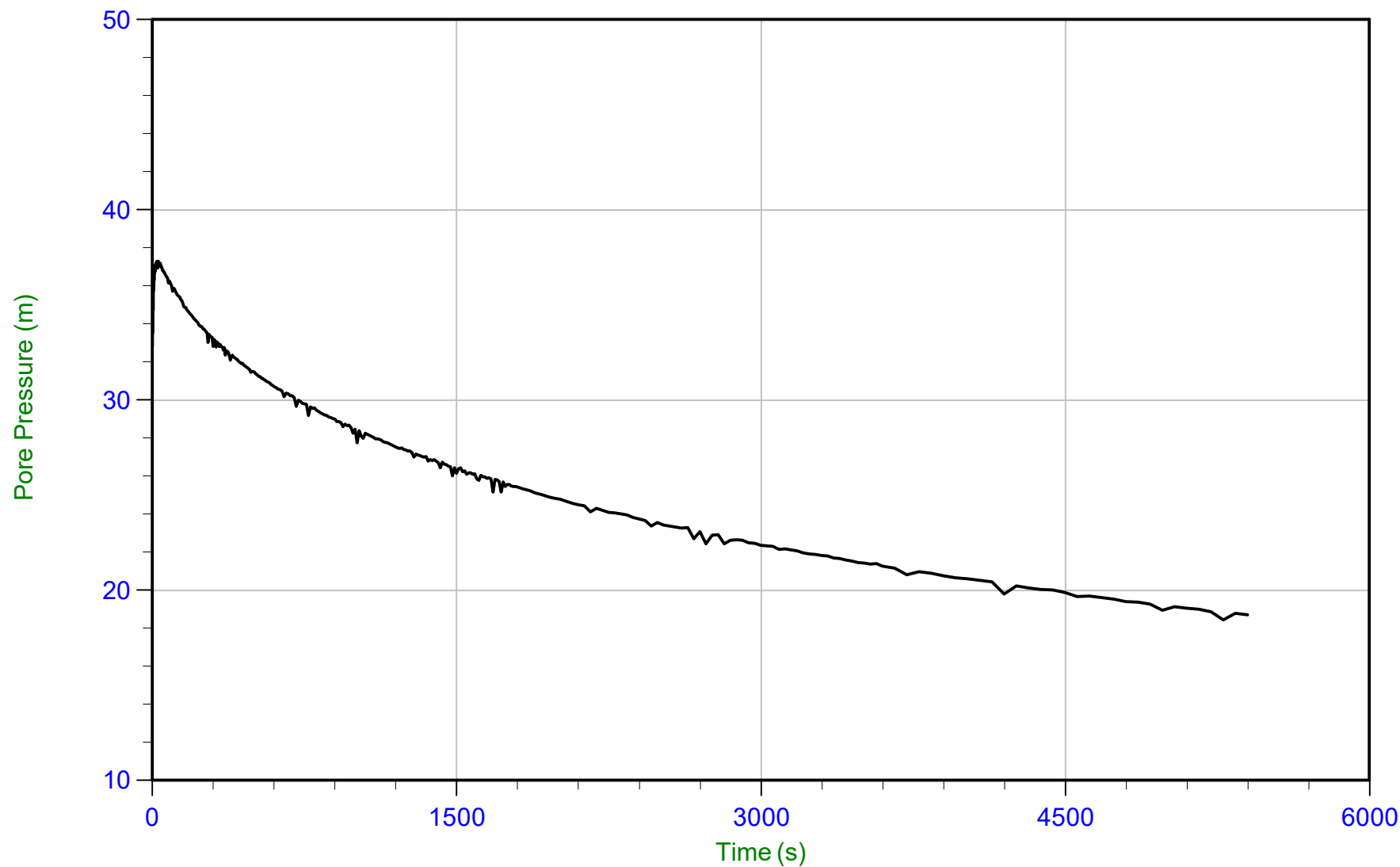
Ch: 0.2 cm<sup>2</sup>/min



*Golder*

Job No: 23-05-25302  
Date: 02/08/2023 08:08  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-03C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP03.PPF2  
Depth: 9.000 m / 29.527 ft  
Duration: 5400.0 s

u Min: 18.4 m  
u Max: 37.3 m  
u Final: 18.7 m

WT: 3.300 m / 10.827 ft  
Ueq: 5.7 m  
U(50): 21.49 m

T(50): 3434.2 s  
Ir: 100  
Ch: 0.2 cm<sup>2</sup>/min



*Golder*

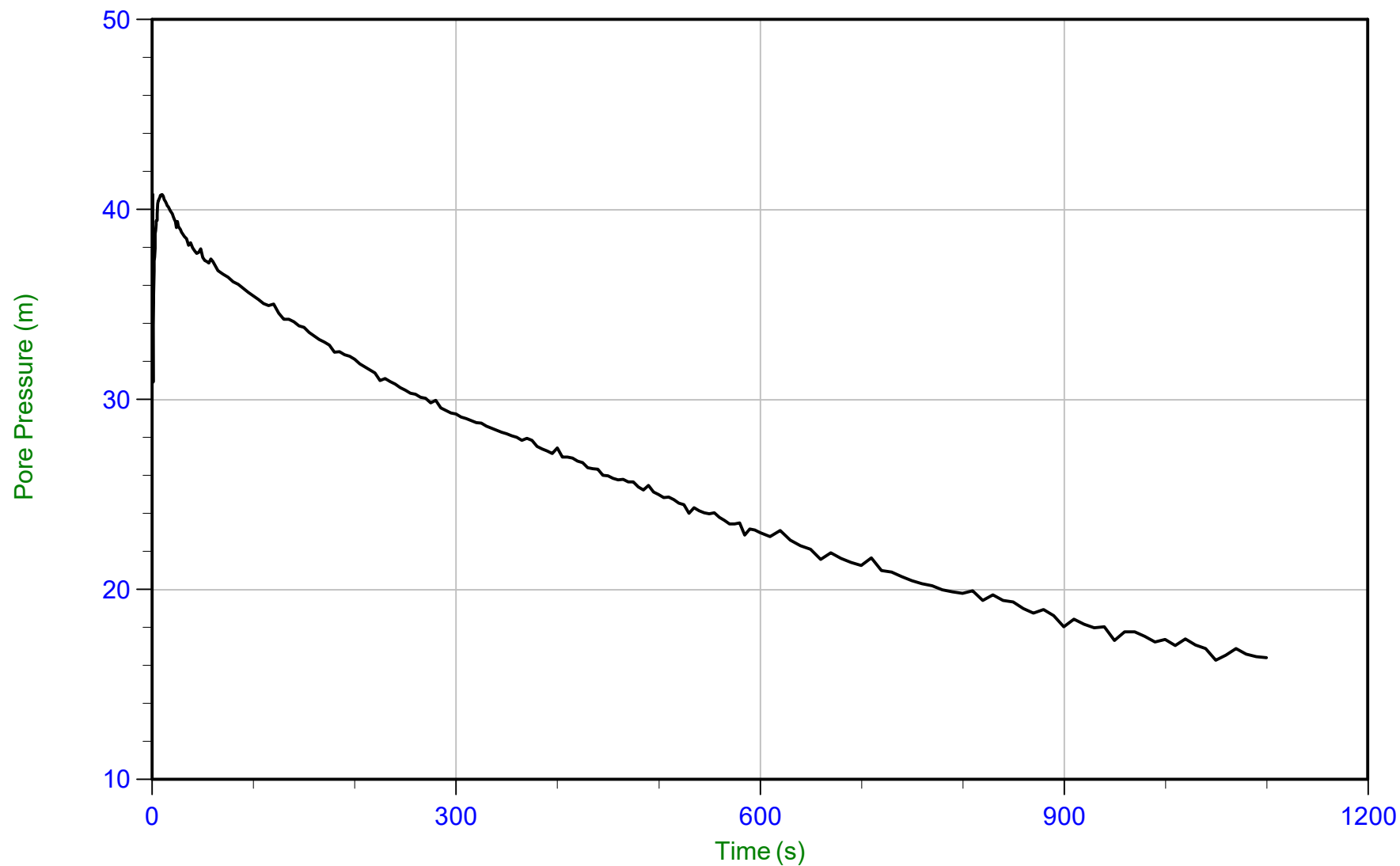
Job No: 23-05-25302

Date: 02/08/2023 08:08

Site: Fraser Road, South Glengarry

Sounding: SCPT22-03C

Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP03.PPF2

Depth: 11.000 m / 36.089 ft

Duration: 1100.0 s

u Min: 16.3 m

u Max: 40.8 m

u Final: 16.4 m

WT: 3.300 m / 10.827 ft

Ueq: 7.7 m

U(50): 24.26 m

T(50): 526.6 s

Ir: 100

Ch: 1.3 cm<sup>2</sup>/min



*Golder*

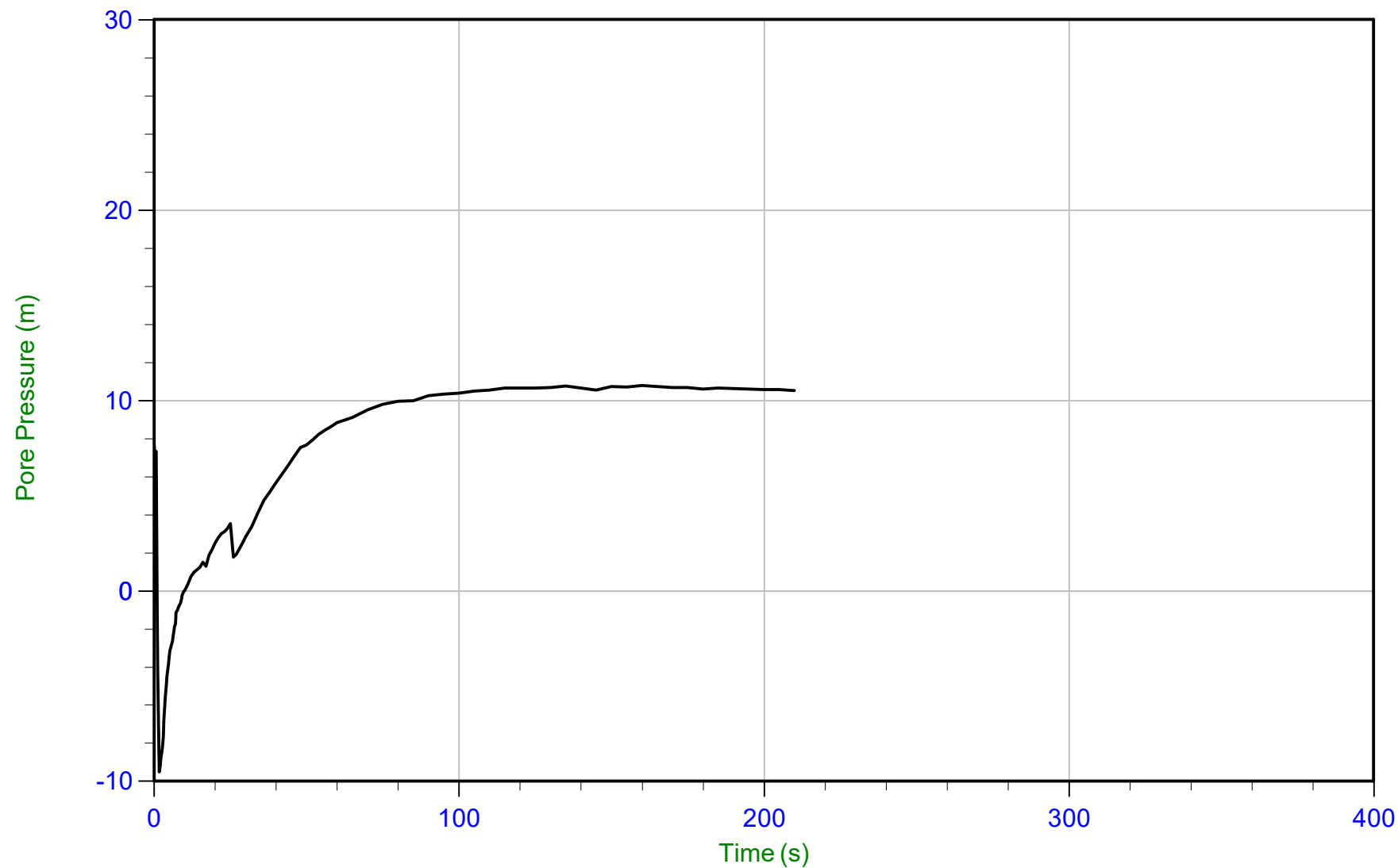
Job No: 23-05-25302

Date: 02/08/2023 08:08

Site: Fraser Road, South Glengarry

Sounding: SCPT22-03C

Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP03.PPF2

Depth: 11.100 m / 36.417 ft

Duration: 210.0 s

u Min: -9.5 m

u Max: 10.8 m

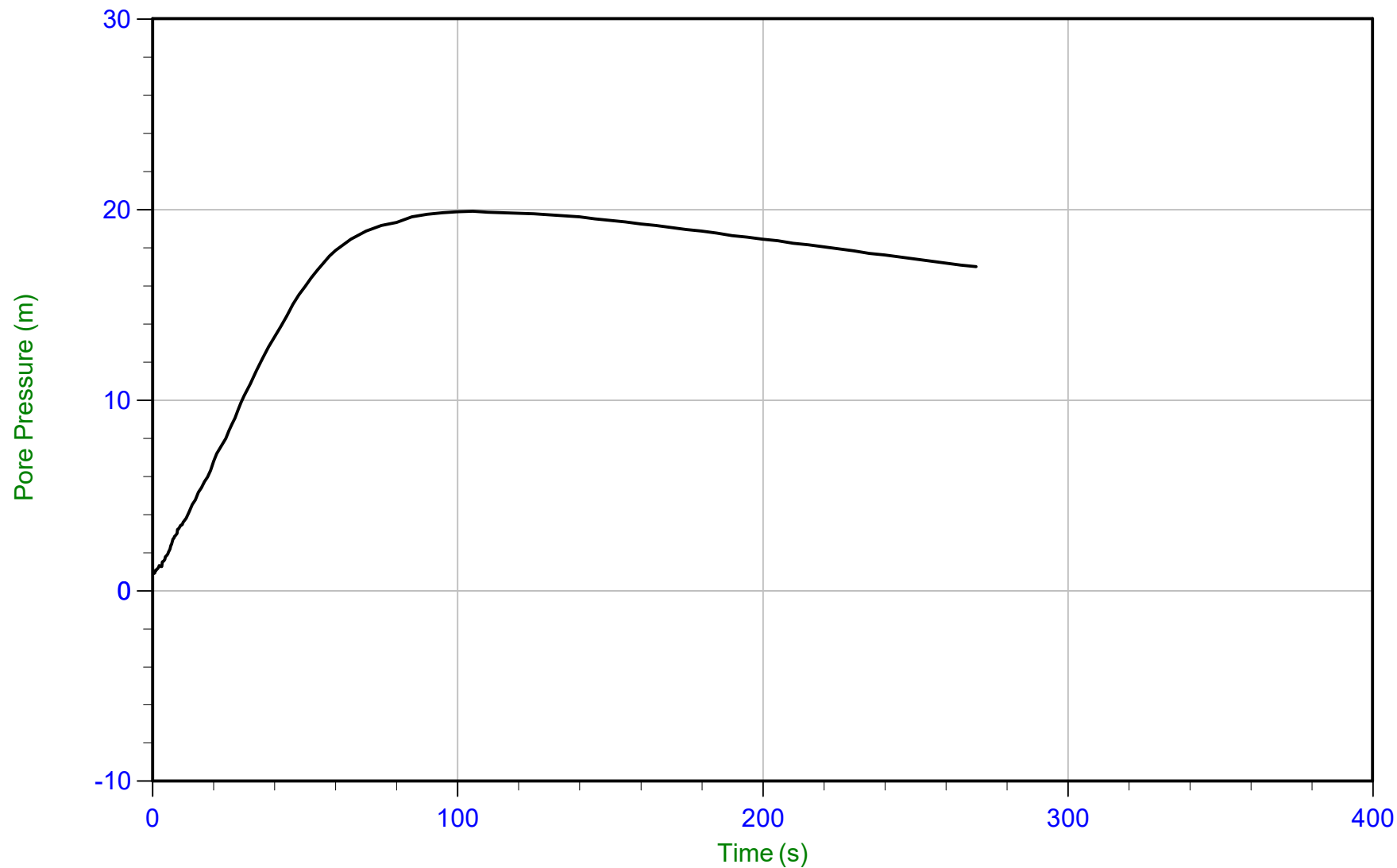
u Final: 10.5 m



*Golder*

Job No: 23-05-25302  
Date: 01/31/2023 12:51  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-04C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP04.PPF2  
Depth: 8.475 m / 27.805 ft  
Duration: 270.0 s

u Min: 0.8 m  
u Max: 19.9 m  
u Final: 17.0 m

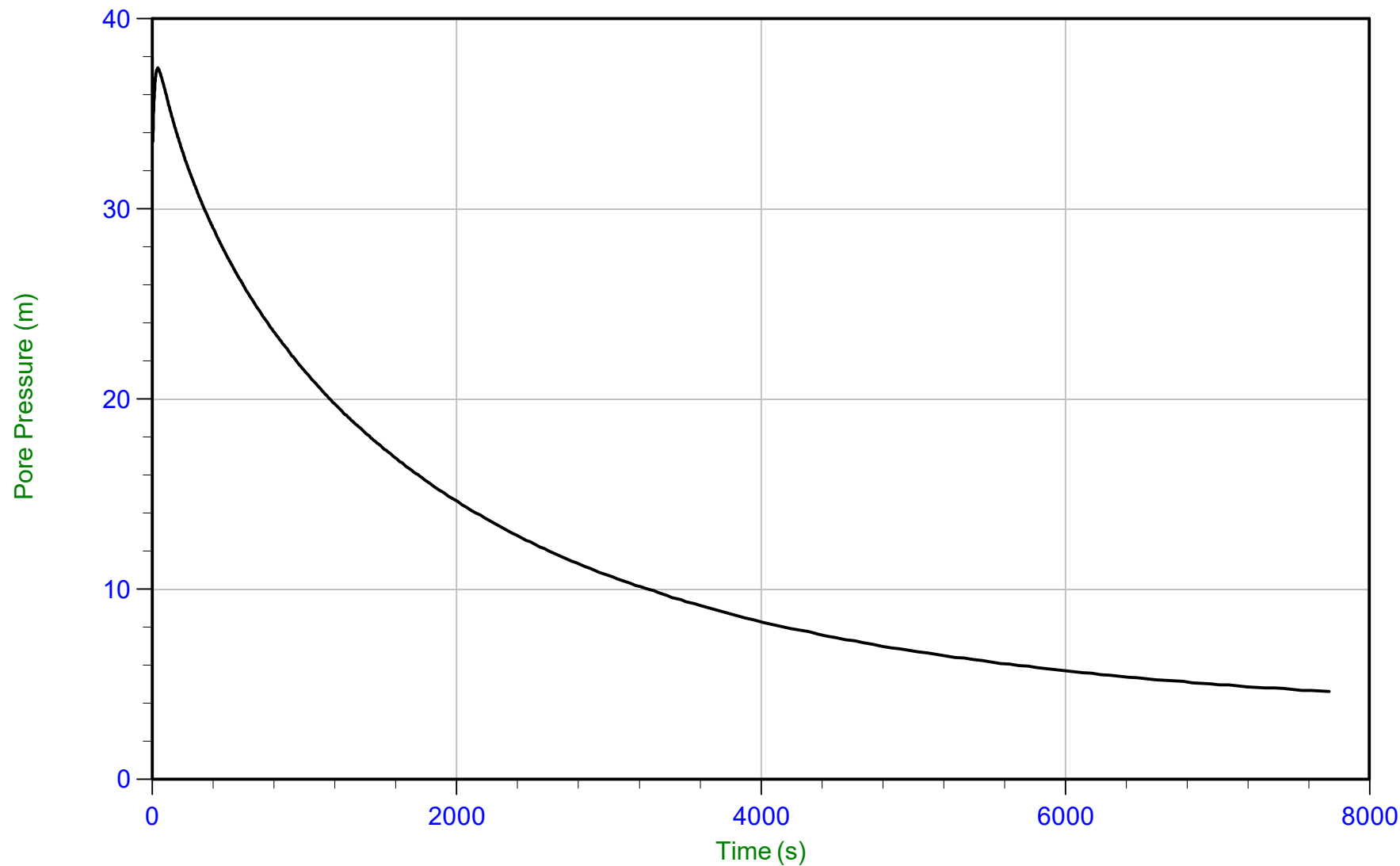
WT: 6.600 m / 21.653 ft  
Ueq: 1.9 m



*Golder*

Job No: 23-05-25302  
Date: 01/31/2023 12:51  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-04C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP04.PPF2  
Depth: 9.475 m / 31.086 ft  
Duration: 7740.0 s

u Min: 4.6 m  
u Max: 37.9 m  
u Final: 4.6 m

WT: 6.600 m / 21.653 ft  
Ueq: 2.9 m  
U(50): 20.40 m

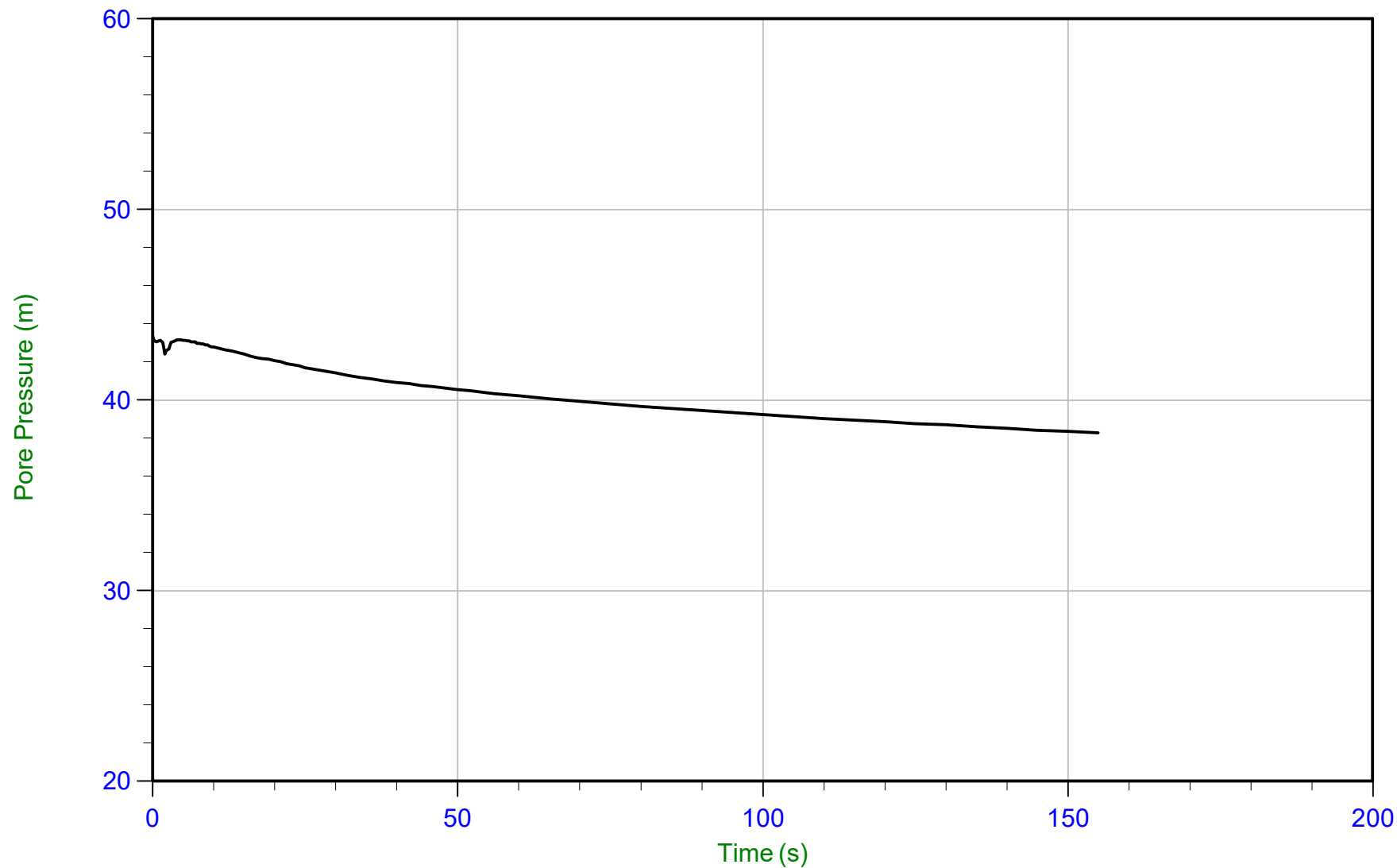
T(50): 1124.5 s  
Ir: 100  
Ch: 0.6 cm<sup>2</sup>/min



*Golder*

Job No: 23-05-25302  
Date: 01/31/2023 12:51  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-04C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP04.PPF2  
Depth: 10.400 m / 34.120 ft  
Duration: 155.0 s

u Min: 38.3 m  
u Max: 43.4 m  
u Final: 38.3 m

WT: 6.600 m / 21.653 ft  
Ueq: 3.8 m

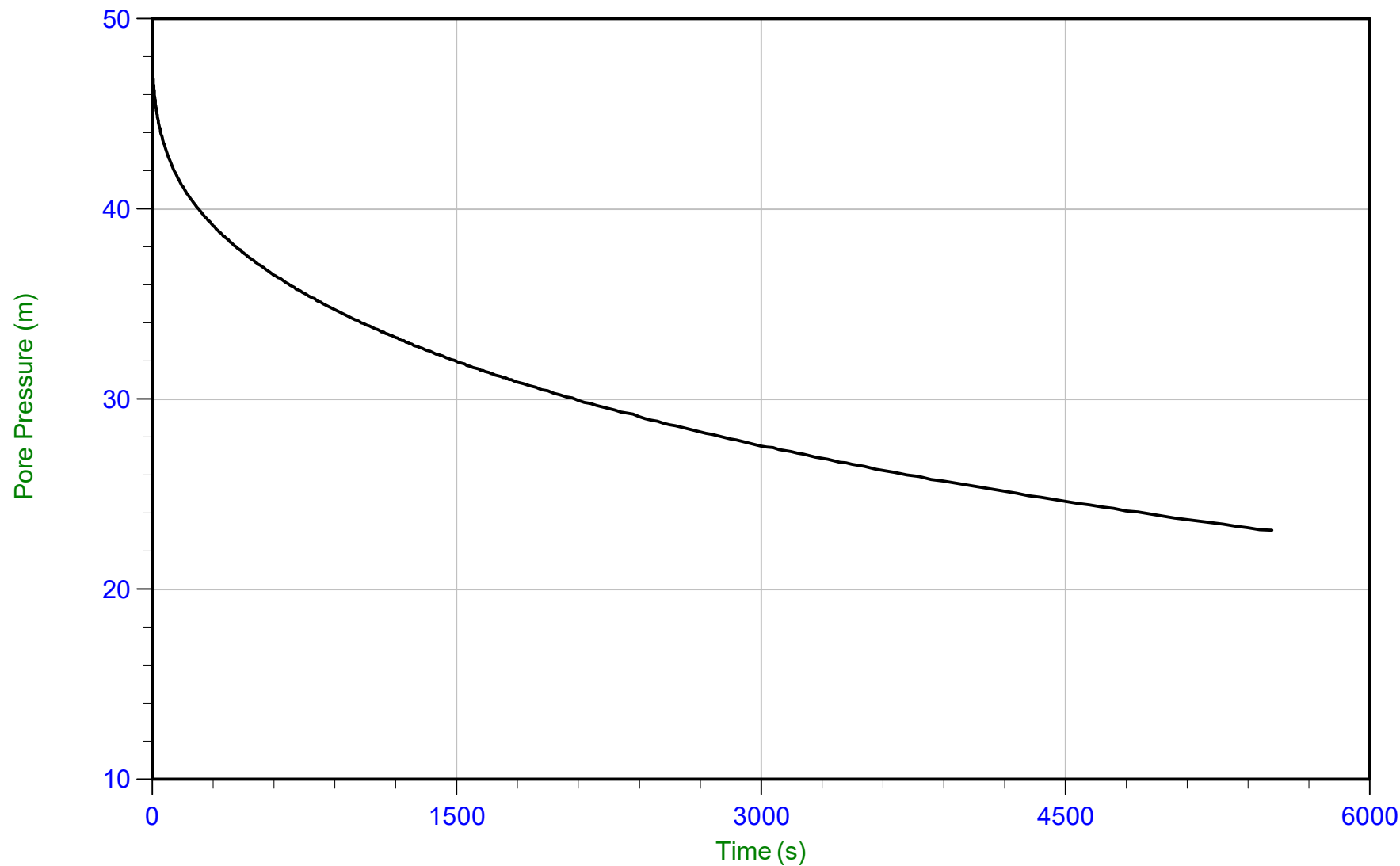




*Golder*

Job No: 23-05-25302  
Date: 01/31/2023 12:51  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-04C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP04.PPF2  
Depth: 11.400 m / 37.401 ft  
Duration: 5520.0 s

u Min: 23.1 m  
u Max: 47.2 m  
u Final: 23.1 m

WT: 6.600 m / 21.653 ft  
Ueq: 4.8 m  
U(50): 25.98 m

T(50): 3745.2 s  
Ir: 100  
Ch: 0.2 cm<sup>2</sup>/min



*Golder*

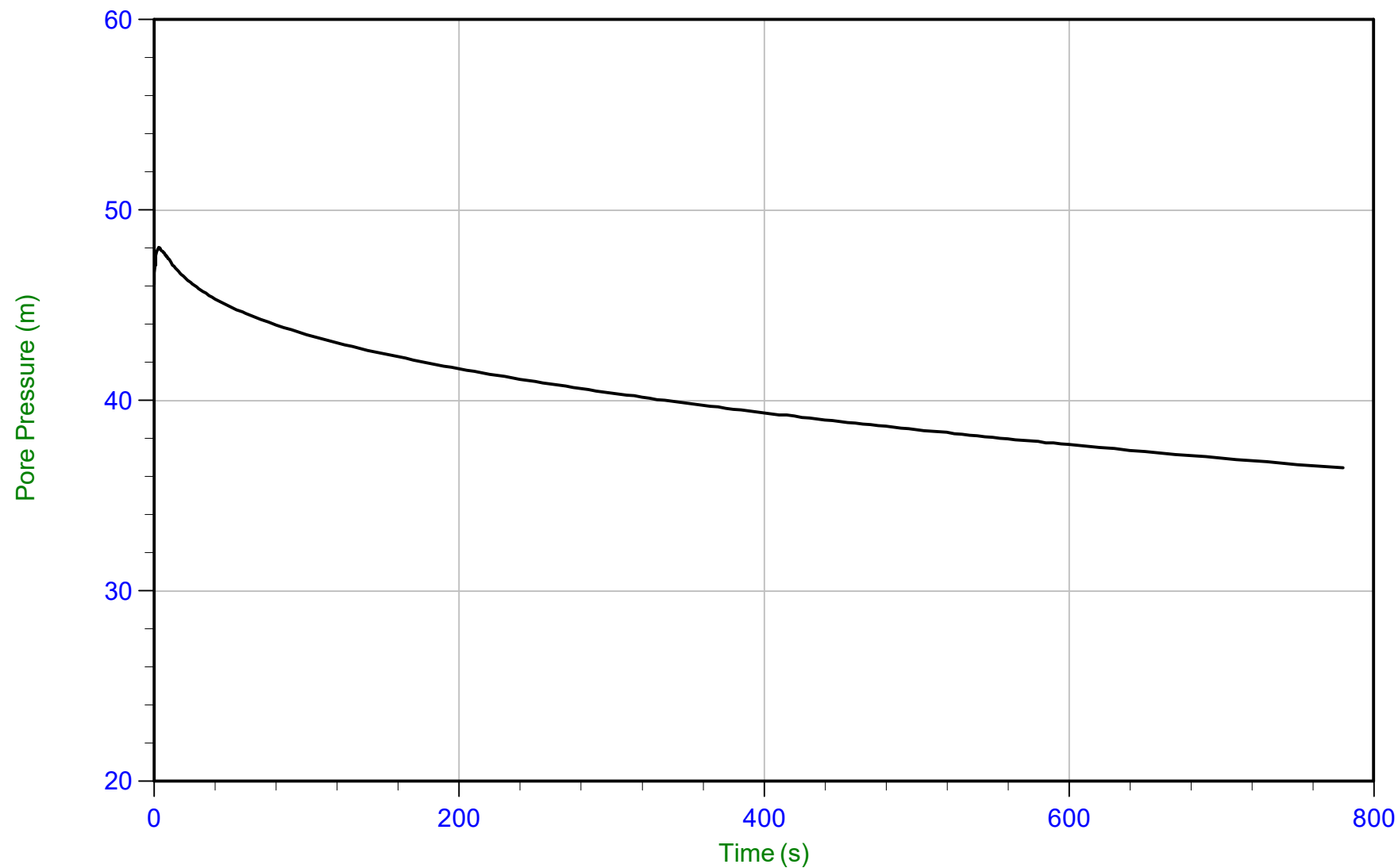
Job No: 23-05-25302

Date: 01/31/2023 12:51

Site: Fraser Road, South Glengarry

Sounding: SCPT22-04C

Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP04.PPF2

Depth: 12.400 m / 40.682 ft

Duration: 780.0 s

u Min: 36.5 m

u Max: 48.0 m

u Final: 36.5 m

WT: 6.600 m / 21.653 ft

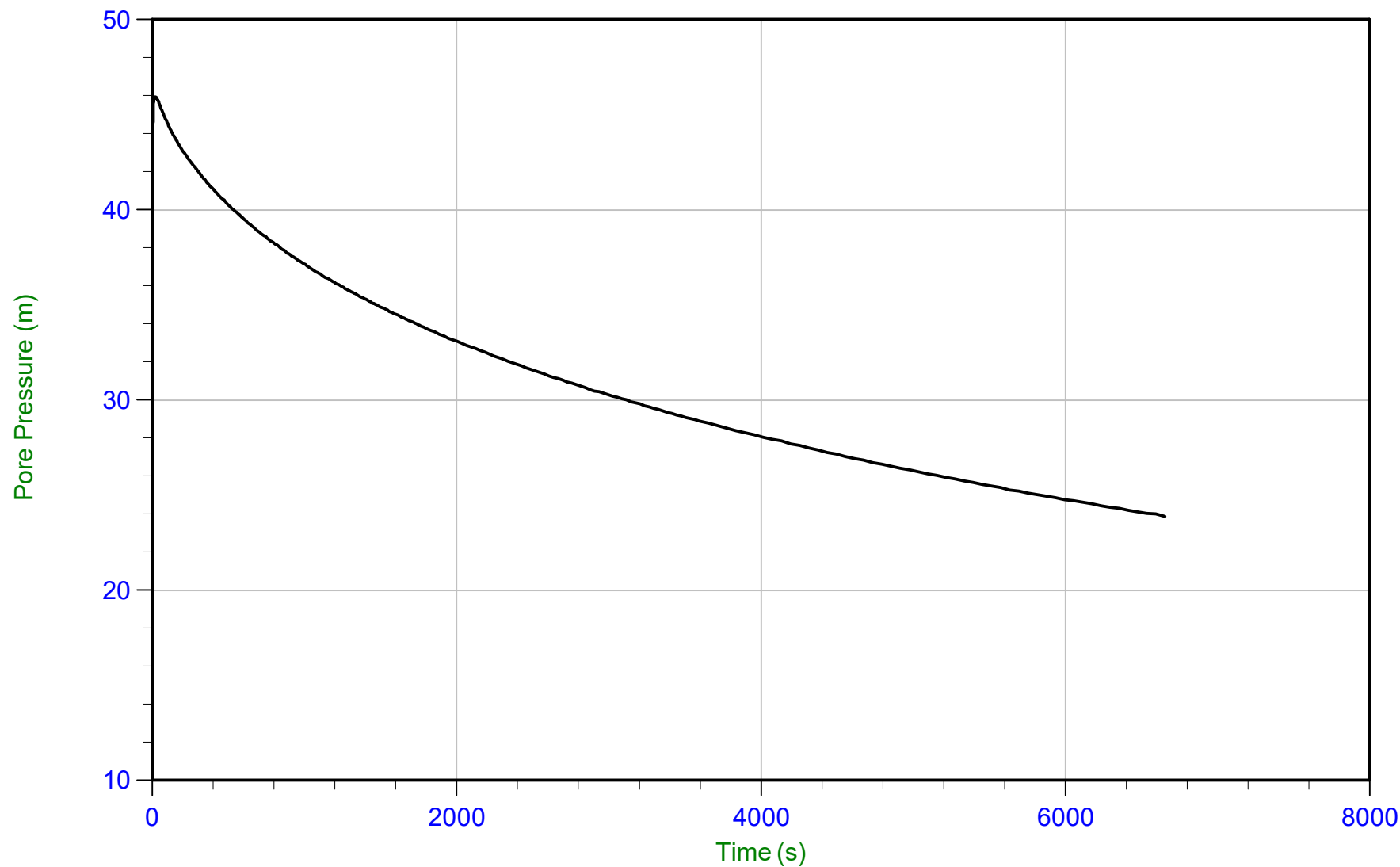
Ueq: 5.8 m



*Golder*

Job No: 23-05-25302  
Date: 01/31/2023 12:51  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-04C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP04.PPF2  
Depth: 13.500 m / 44.291 ft  
Duration: 6660.0 s

u Min: 23.9 m  
u Max: 48.0 m  
u Final: 23.9 m

WT: 6.600 m / 21.653 ft  
Ueq: 6.9 m  
U(50): 27.46 m

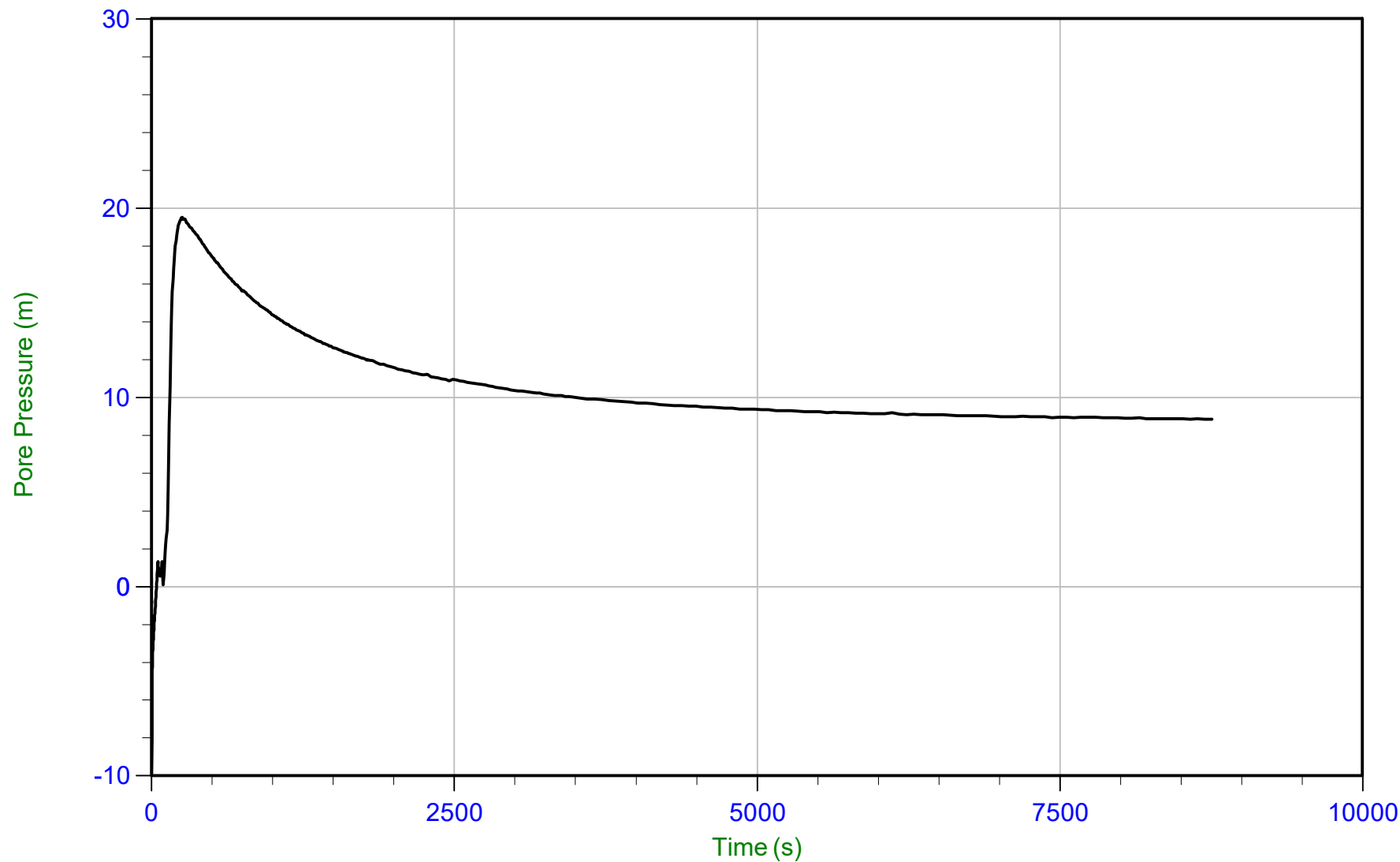
T(50): 4327.0 s  
Ir: 100  
Ch: 0.2 cm<sup>2</sup>/min



*Golder*

Job No: 23-05-25302  
Date: 01/31/2023 12:51  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-04C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP04.PPF2  
Depth: 15.425 m / 50.606 ft  
Duration: 8760.0 s

u Min: -9.9 m  
u Max: 19.5 m  
u Final: 8.8 m

WT: 6.583 m / 21.598 ft  
Ueq: 8.8 m  
U(50): 14.17 m

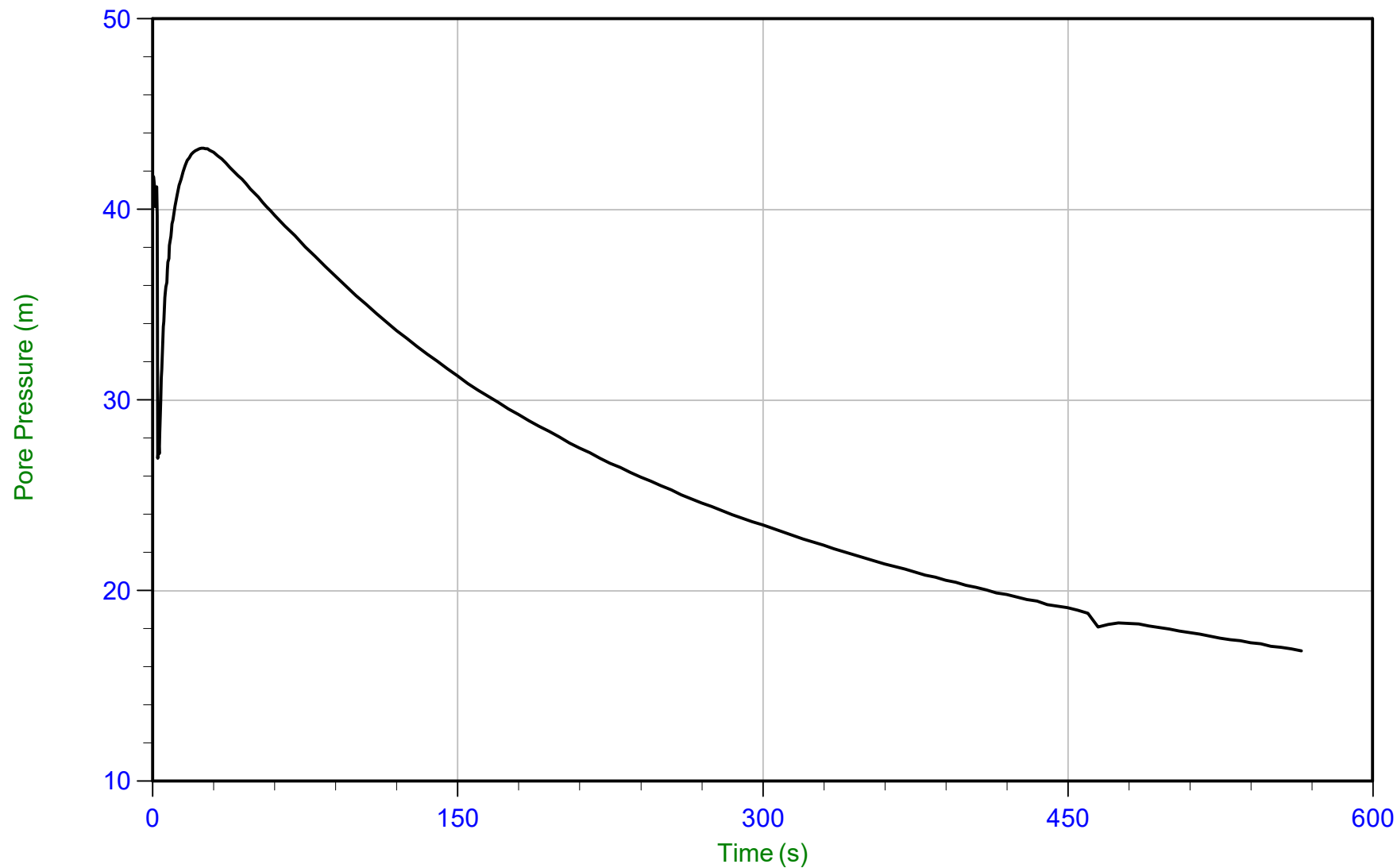
T(50): 790.2 s  
Ir: 100  
Ch: 0.9 cm<sup>2</sup>/min



*Golder*

Job No: 23-05-25302  
Date: 01/31/2023 12:51  
Site: Fraser Road, South Glengarry

Sounding: SCPT22-04C  
Cone: 806:T1000F10U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 23-05-25302\_SP04.PPF2  
Depth: 16.500 m / 54.133 ft  
Duration: 565.0 s

u Min: 16.8 m  
u Max: 43.2 m  
u Final: 16.8 m

WT: 6.600 m / 21.653 ft  
Ueq: 9.9 m  
U(50): 26.56 m

T(50): 202.7 s  
Ir: 100  
Ch: 3.5 cm<sup>2</sup>/min

## Description of Methods for Calculated CPT Geotechnical Parameters

# CALCULATED CPT GEOTECHNICAL PARAMETERS

## A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software



Revision SZW-Rev 14

Revised November 26, 2019

Prepared by Jim Greig, M.A.Sc, P.Eng (BC)



### Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

## ConeTec's Calculated CPT Geotechnical Parameters as of November 26, 2019

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g. 0.20 m). Note that  $q_t$  is the tip resistance corrected for pore pressure effects and  $q_c$  is the recorded tip resistance. The corrected tip resistance (corrected using  $u_2$  pore pressure values) is used for all of the calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction,  $f_s$ , are not required.

The tip correction is:  $q_t = q_c + (1-a) \cdot u_2$  (consistent units are implied)

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 cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline have been taken into account as has the appropriate unit weight of water. How this is done depends on where the instruments were zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 5. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBT chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The





Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter,  $I_c$ . Please note that the  $I_c$  parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that used by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the  $B_q$  parameter. The normalized  $Q_{tn}$  SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent,  $n$ , for normalization based on a slightly modified redefinition and iterative approach for  $I_c$ . The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson.

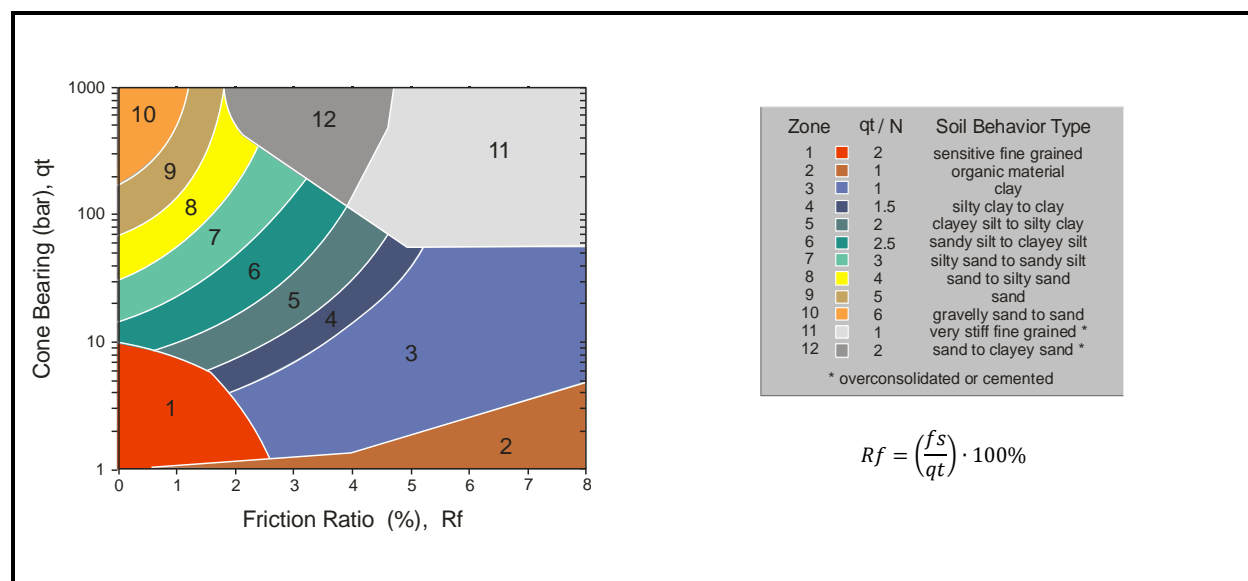


Figure 1. Non-Normalized Soil Behavior Type Classification Chart (SBT)

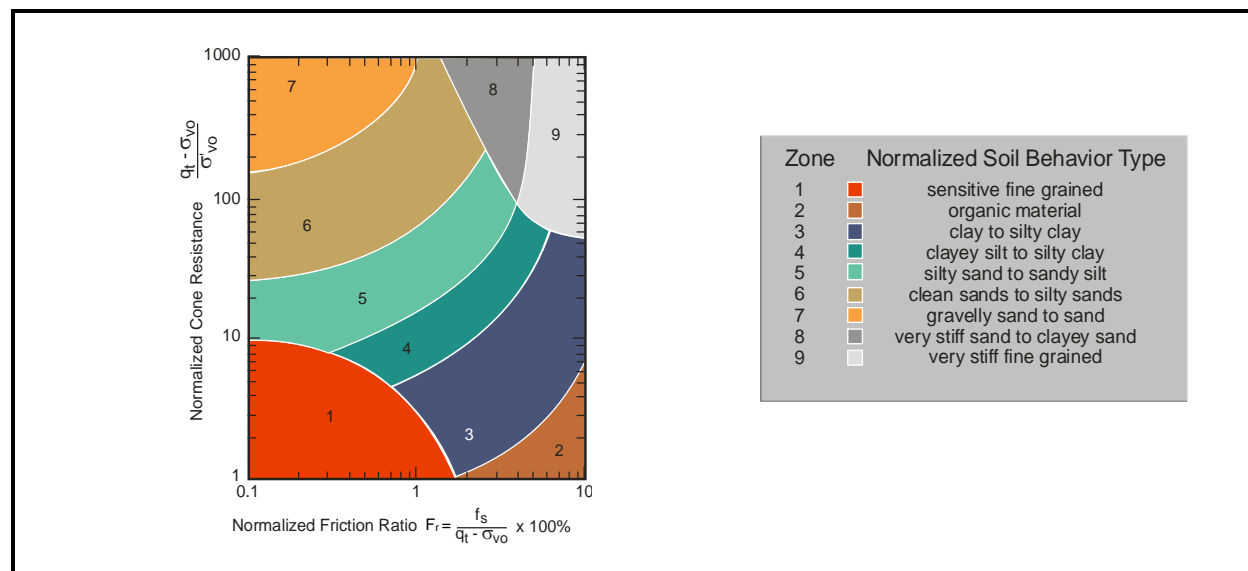


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

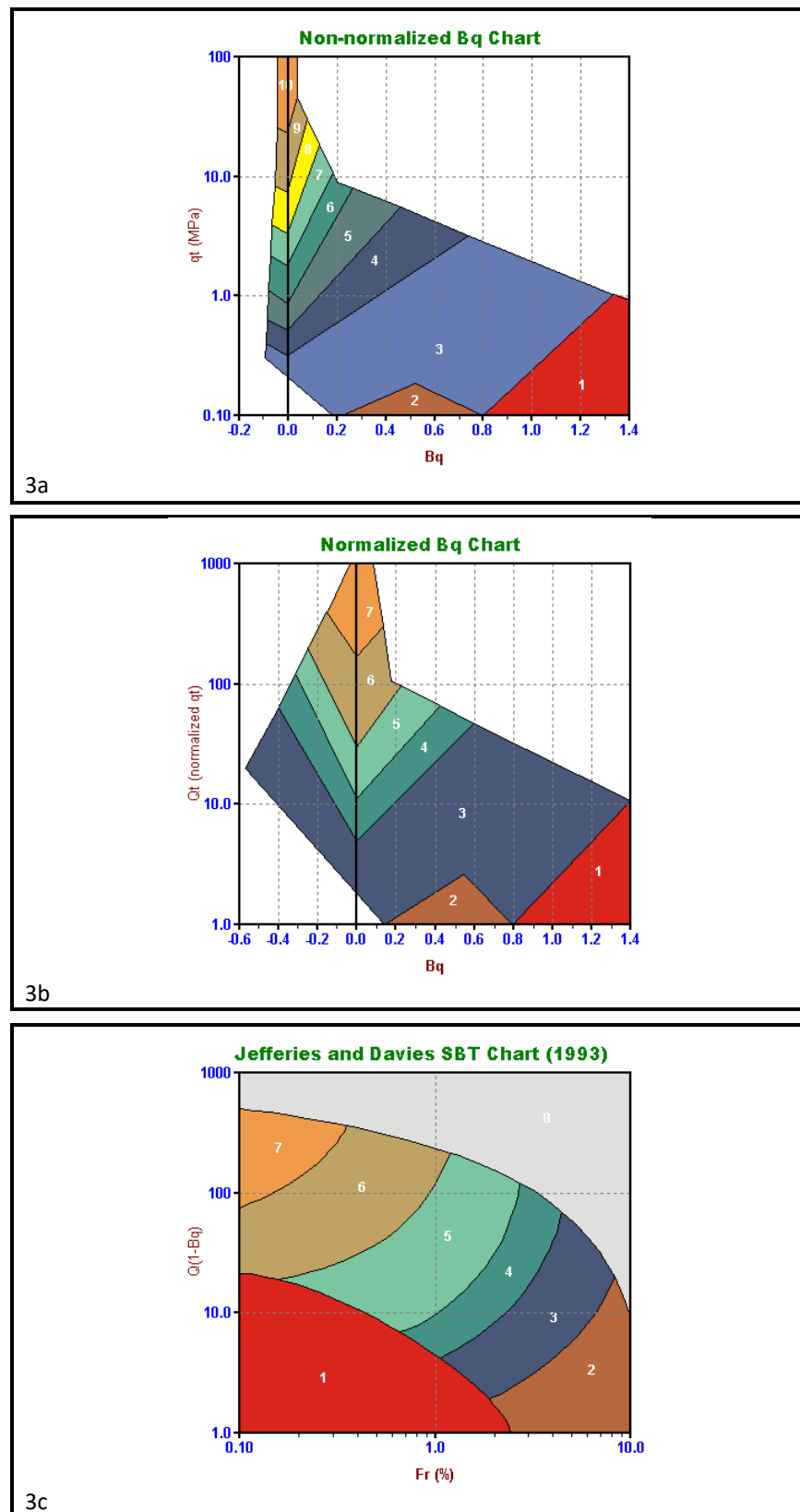


Figure 3. Alternate Soil Behavior Type Charts

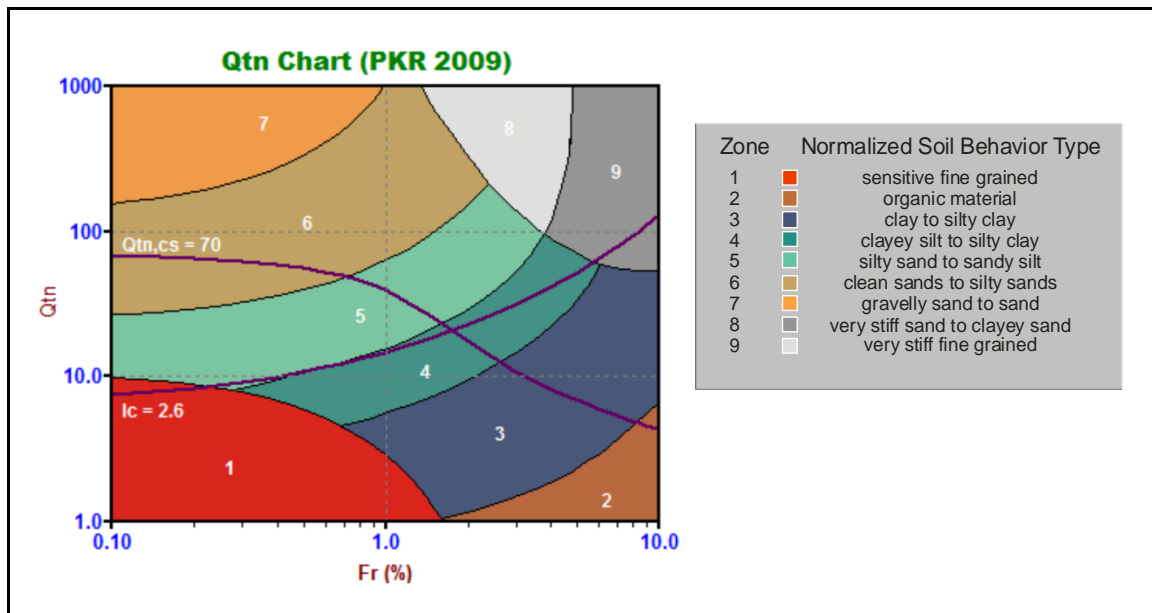
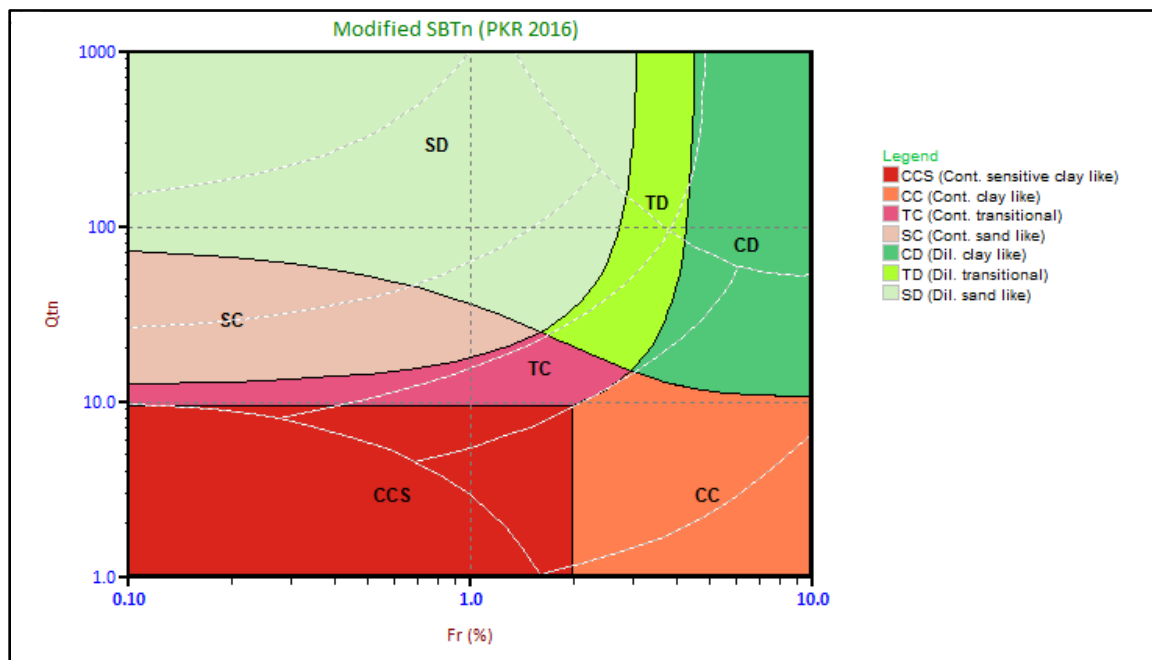
Figure 4. Normalized Soil Behavior Type Chart using  $Q_{tn}$  (SBT  $Q_{tn}$ )

Figure 5. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary the user should refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed 'invalid' the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g. drilled out section or data gap).
2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving as an undrained material (and vice versa).
3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.
4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Table 1 may be included in the output files delivered with this report.

The output files are typically provided in Microsoft Excel XLS or XLSX format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or requested by the client. Each output file is named using the original COR file base name followed by a three or four letter indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

**Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters**

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth <i>(where calculations are done at each point then Mid Layer Depth = Recorded Depth)</i>	$[Depth (Layer Top) + Depth (Layer Bottom)] / 2.0$	CK*
Elevation	Elevation of Mid Layer based on sounding collar elevation supplied by client or through site survey	Elevation = Collar Elevation - Depth	CK*
Avg qc	Averaged recorded tip value ( $q_c$ )	$Avg qc = \frac{1}{n} \sum_{i=1}^n q_c$ <i>n=1 when calculations are done at each point</i>	CK*
Avg qt	Averaged corrected tip ( $q_t$ ) where: $q_t = q_c + (1 - \alpha) \bullet u_2$	$Avg qt = \frac{1}{n} \sum_{i=1}^n q_t$ <i>n=1 when calculations are done at each point</i>	1
Avg fs	Averaged sleeve friction ( $f_s$ )	$Avg fs = \frac{1}{n} \sum_{i=1}^n f_s$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Rf	Averaged friction ratio ( $R_f$ ) where friction ratio is defined as: $R_f = 100\% \bullet \frac{f_s}{q_t}$	$Avg Rf = 100\% \bullet \frac{Avg fs}{Avg qt}$ <i>n=1 when calculations are done at each point</i>	CK*
Avg u	Averaged dynamic pore pressure ( $u$ )	$Avg u = \frac{1}{n} \sum_{i=1}^n u_i$ <i>n=1 when calculations are done at each point</i>	CK*

Calculated Parameter	Description	Equation	Ref
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n} \sum_{i=1}^n Resistivity_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n} \sum_{i=1}^n UVIF_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Temp	Averaged Temperature (this data is not always available since it requires specialized calibrations)	$AvgTemp = \frac{1}{n} \sum_{i=1}^n Temperature_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n} \sum_{i=1}^n Gamma_i$ <i>n=1 when calculations are done at each point</i>	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization)	See Figure 2	2, 5
SBT-Bq	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	1, 2, 5
SBT-Bqn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on $I_c$	See Figure 4	15
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior.	See Figure 5	30
Unit Wt.	<p>Unit Weight of soil determined from one of the following user selectable options:</p> <ol style="list-style-type: none"> <li>1) uniform value</li> <li>2) value assigned to each SBT zone</li> <li>3) value assigned to each SBTn zone</li> <li>4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on <math>q_{c1n}</math></li> <li>5) values assigned to SBT Qtn zones</li> <li>6) Mayne <math>f_s</math> (sleeve friction) method</li> <li>7) Robertson 2010 method</li> <li>8) user supplied unit weight profile</li> </ol> <p>The last option may co-exist with any of the other options</p>	See references	3, 5, 15, 21, 24, 29

Calculated Parameter	Description	Equation	Ref
TStress  $\sigma_v$	<p>Total vertical overburden stress at Mid Layer Depth</p> <p><i>A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.</i></p> <p><i>For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point.</i></p> <p><i>Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point.</i></p> <p><i>For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.</i></p>	$TStress = \sum_{i=1}^n \gamma_i h_i$ <p>where <math>\gamma_i</math> is layer unit weight <math>h_i</math> is layer thickness</p>	CK*
EStress $\sigma_v'$	Effective vertical overburden stress at mid-layer depth	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u $u_{eq}$ or $u_0$	<p>Equilibrium pore pressure determined from one of the following user selectable options:</p> <ol style="list-style-type: none"> <li>1) hydrostatic below water table</li> <li>2) user supplied profile</li> <li>3) combination of those above</li> </ol> <p>When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used.</p> <p>Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.</p>	<p>For hydrostatic option:</p> $u_{eq} = \gamma_w \cdot (D - D_{wt})$ <p>where <math>u_{eq}</math> is equilibrium pore pressure <math>\gamma_w</math> is unit weight of water <math>D</math> is the current depth <math>D_{wt}</math> is the depth to the water table</p>	CK*
$K_0$	Coefficient of earth pressure at rest, $K_0$	$K_0 = (1 - \sin \Phi') OCR^{\sin \Phi'}$	17
$C_n$	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters	$C_n = (P_a / \sigma_v')^{0.5}$ <p>where <math>0.0 &lt; C_n &lt; 2.0</math> (user adjustable, typically 1.7) <math>P_a</math> is atmospheric pressure (100 kPa)</p>	12
$C_q$	Overburden stress normalizing factor	$C_q = 1.8 / (0.8 + (\sigma_v' / P_a))$ <p>where <math>0.0 &lt; C_q &lt; 2.0</math> (user adjustable) <math>P_a</math> is atmospheric pressure (100 kPa)</p>	3, 12

Calculated Parameter	Description	Equation	Ref
$N_{60}$	SPT N value at 60% energy calculated from $q_t/N$ ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	<i>See Figure 1</i>	5
$(N_1)_{60}$	SPT $N_{60}$ value corrected for overburden pressure	$(N_1)_{60} = C_n \cdot N_{60}$	4
$N_{60lc}$	SPT $N_{60}$ values based on the $I_c$ parameter [as defined by Roberston and Wride 1998 (5), or by Robertson 2009 (15)].	$(q_t/P_a)/N_{60} = 8.5 (1 - I_c/4.6)$ $(q_t/P_a)/N_{60} = 10^{(1.1268 - 0.2817I_c)}$ $P_a$ being atmospheric pressure	5 15, 31
$(N_1)_{60lc}$	SPT $N_{60}$ value corrected for overburden pressure (using $N_{60lc}$ ). User has 3 options.	1) $(N_1)_{60lc} = C_n \cdot (N_{60lc})$ 2) $q_{c1n}/(N_1)_{60lc} = 8.5 (1 - I_c/4.6)$ 3) $(Q_{tn})/(N_1)_{60lc} = 10^{(1.1268 - 0.2817I_c)}$	4 5 15, 31
$S_u$ or $S_u$ (Nkt)	Undrained shear strength based on $q_t$ $S_u$ factor $N_{kt}$ is user selectable	$S_u = \frac{q_t - \sigma_v}{N_{kt}}$	1, 5
$S_u$ or $S_u$ (Ndu)	Undrained shear strength based on pore pressure $S_u$ factor $N_{du}$ is user selectable	$S_u = \frac{u_2 - u_{eq}}{N_{du}}$	1, 5
$D_r$	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Høksund Sand c) Schmertmann (1978) d) Jamiolkowski (1985) - All Sands e) Jamiolkowski et al (2003) (various compressibilities, $K_o$ )	See reference (methods a through d) Jamiolkowski et al (2003) reference	5 14
$\Phi$ $\phi$	Friction Angle determined from one of the following user selectable options (methods a through d are for sands and method e is for silts and clays): a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne e) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/ $q_t$	Differential pore pressure ratio (older parameter used before $B_q$ was established)	$= \frac{\Delta u}{q_t}$  where: $\Delta u = u - u_{eq}$ and $u = \text{dynamic pore pressure}$ $u_{eq} = \text{equilibrium pore pressure}$	CK*
$B_q$	Pore pressure parameter	$B_q = \frac{\Delta u}{q_t - \sigma_v}$  where: $\Delta u = u - u_{eq}$ and $u = \text{dynamic pore pressure}$ $u_{eq} = \text{equilibrium pore pressure}$	1, 2, 5
Net $q_t$ or $q_{tNet}$	Net tip resistance (used in many subsequent correlations)	$q_t - \sigma_v$	CK*
$q_e$	Effective tip resistance (using the dynamic pore pressure $u_2$ and not equilibrium pore pressure)	$q_t - u_2$	CK*

Calculated Parameter	Description	Equation	Ref
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v}$	CK*
$Q_t$ or Norm: $Q_t$	Normalized $q_t$ for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from $Q_{tn}$ .	$Q_t = \frac{qt - \sigma_v}{\sigma_v}$	2, 5
$F_r$ or Norm: $F_r$	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
$Q(1-Bq)$	$Q(1-Bq)$ grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their $I_c$ parameter	$Q \cdot (1 - Bq)$ <i>where <math>Bq</math> is defined as above and <math>Q</math> is the same as the normalized tip resistance, <math>Q_t</math>, defined above</i>	6, 7
qc1	Normalized tip resistance, $q_{c1}$ , using a fixed stress ratio exponent, $n$ (this method has stress units)	$q_{c1} = q_t \cdot (P_a / \sigma_v')^{0.5}$ where: $P_a$ = atmospheric pressure	21
qc1 (0.5)	Normalized tip resistance, $q_{c1}$ , using a fixed stress ratio exponent, $n$ (this method is unit-less)	$q_{c1} (0.5) = (q_t / P_a) \cdot (P_a / \sigma_v')^{0.5}$ where: $P_a$ = atmospheric pressure	5
qc1 ( $C_n$ )	Normalized tip resistance, $q_{c1}$ , based on $C_n$ (this method has stress units)	$q_{c1}(C_n) = C_n \cdot q_t$	5, 12
qc1 ( $C_q$ )	Normalized tip resistance, $q_{c1}$ , based on $C_q$ (this method has stress units)	$q_{c1}(C_q) = C_q \cdot q_t$ (some papers use $q_c$ )	5, 12
qc1n	normalized tip resistance, $q_{c1n}$ , using a variable stress ratio exponent, $n$ (where $n=0.0, 0.70, 1.0$ ) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a / \sigma_v')^n$ where: $P_a$ = atm. Pressure and $n$ varies as described below	3, 5
$I_c$ or $I_c$ (RW1998)	Soil Behavior Type Index as defined by Robertson and Fear (1995) and Robertson and Wride (1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart	$I_c = [(3.47 - \log_{10} Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$  Where: $Q = \left( \frac{qt - \sigma_v}{P_a} \right) \left( \frac{P_a}{\sigma_v'} \right)^n$  Or $Q = q_{c1n} = \left( \frac{qt}{P_a} \right) \left( \frac{P_a}{\sigma_v'} \right)^n$  <i>depending on the iteration in determining <math>I_c</math></i>  And $Fr$ is in percent $P_a$ = atmospheric pressure  <i><math>n</math> varies between 0.5, 0.70 and 1.0 and is selected in an iterative manner based on the resulting <math>I_c</math></i>	3, 5, 21
$I_c$ (PKR 2009)	Soil Behavior Type Index, $I_c$ (PKR 2009) based on a variable stress ratio exponent $n$ , which itself is based on $I_c$ (PKR 2009). An iterative calculation is required to determine $I_c$ (PKR 2009) and its corresponding $n$ (PKR 2009).	$I_c \text{ (PKR 2009)} = [(3.47 - \log_{10} Q_{tn})^2 + (1.22 + \log_{10} Fr)^2]^{0.5}$	15



Calculated Parameter	Description	Equation	Ref
n (PKR 2009)	Stress ratio exponent n, based on $I_c$ (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding $I_c$ (PKR 2009).	$n \text{ (PKR 2009)} = 0.381 (I_c) + 0.05 (\sigma_v'/P_a) - 0.15$	15
Qtn (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on $I_c$ (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Qtn (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ where $P_a$ = atmospheric pressure (100 kPa) $n$ = stress ratio exponent described above	15
FC	Apparent fines content (%)	$FC = 1.75(I_c^{3.25}) - 3.7$ $FC = 100$ for $I_c > 3.5$ $FC = 0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r < 0.5$	3
$I_c$ Zone	This parameter is the Soil Behavior Type zone based on the $I_c$ parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
State Param or State Parameter or $\psi$	The state parameter index, $\psi$ , is defined as the difference between the current void ratio, $e$ , and the critical void ratio, $e_c$ . Positive $\psi$ - contractive soil Negative $\psi$ - dilative soil  This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992)  - vertical effective stress is used rather than a mean normal stress	See reference	6, 8
Yield Stress $\sigma_p'$	Yield stress is calculated using the following methods  a) General method  b) 1 <sup>st</sup> order approximation using $q_t$ Net (clays) c) 1 <sup>st</sup> order approximation using $\Delta u_2$ (clays) d) 1 <sup>st</sup> order approximation using $q_e$ (clays)	All stresses in kPa  a) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)^{m'} (\sigma_{atm}/100)^{1-m'}$  where $m' = 1 - \frac{0.28}{1 + (I_c / 2.65)^{2.5}}$  b) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)$ c) $\sigma_p' = 0.54 \cdot (\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ d) $\sigma_p' = 0.60 \cdot (q_t - u_2)$	19  20 20 20
OCR  OCR(JS1978)  OCR(Mayne2014) OCR (qtNet) OCR (deltaU) OCR (qe) OCR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on  a) Schmertmann (1978) method involving a plot of $S_u/\sigma_v' / (S_u/\sigma_v')_{NC}$ and OCR  b) based on Yield stresses described above c) approximate version based on qtNet d) approximate version based on $\Delta u$ e) approximate version based on effective tip, $q_e$ f) approximate version based on shear wave velocity, $V_s$ g) based on $Q_t$	a) requires a user defined value for NC $S_u/P_c'$ ratio  b through f) based on yield stresses  g) $OCR = 0.25 \cdot (Q_t)^{1.25}$	9  19 20 20 20 18 32

Calculated Parameter	Description	Equation	Ref
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Es Young's Modulus E	<p>Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from:</p> <p>a) OC Sands b) Aged NC Sands c) Recent NC Sands</p> <p>Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the Es/qt chart. Es is evaluated for an axial strain of 0.1%.</p>	<p>Mean normal stress is evaluated from:</p> $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)$ <p>where <math>\sigma'_v</math> = vertical effective stress <math>\sigma'_h</math> = horizontal effective stress</p> <p>and <math>\sigma'_h = K_o \cdot \sigma'_v</math> with <math>K_o</math> assumed to be 0.5</p>	5
Delta U/TStress	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ where: $\Delta u = u - u_{eq}$	CK*
Delta U/Estress, P Value, Excess Pore Pressure Ratio	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ where: $\Delta u = u - u_{eq}$	25, 25a, CK*
Su/Estress	Undrained shear strength ratio with respect to vertical effective overburden stress using the Su (N <sub>kt</sub> ) method	$= Su (N_{kt}) / \sigma'_v$	CK*
Gmax	G <sub>max</sub> determined from SCPT shear wave velocities (not estimated values)	$G_{max} = \rho V_s^2$ where $\rho$ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/Gmax	Net tip resistance ratio with respect to the small strain modulus G <sub>max</sub> determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and $\rho$ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30

\*CK – common knowledge

**Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters**

Calculated Parameter	Description	Equation	Ref
$K_{SPT}$	Equivalent clean sand factor for $(N_1)_{60}$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	10
$K_{CPT}$ or $K_c$ (RW1998)	Equivalent clean sand correction for $q_{c1N}$	$K_{cpt} = 1.0$ for $l_c \leq 1.64$ $K_{cpt} = f(l_c)$ for $l_c > 1.64$ (see reference) $K_c = -0.403 l_c^4 + 5.581 l_c^3 - 21.63 l_c^2 + 33.75 l_c - 17.88$	3, 10
$K_c$ (PKR 2010)	Clean sand equivalent factor to be applied to $Q_{tn}$	$K_c = 1.0$ for $l_c \leq 1.64$ $K_c = -0.403 l_c^4 + 5.581 l_c^3 - 21.63 l_c^2 + 33.75 l_c - 17.88$ for $l_c > 1.64$	16
$(N_1)_{60cs} I_c$	Clean sand equivalent SPT $(N_1)_{60} I_c$ . User has 3 options.	1) $(N_1)_{60cs} I_c = \alpha + \beta((N_1)_{60} I_c)$ 2) $(N_1)_{60cs} I_c = K_{SPT} \cdot ((N_1)_{60} I_c)$ 3) $(q_{c1ncs}) / ((N_1)_{60cs} I_c) = 8.5 (1 - I_c/4.6)$  FC $\leq$ 5%: $\alpha = 0, \beta = 1.0$ FC $\geq$ 35%: $\alpha = 5.0, \beta = 1.2$ 5% < FC < 35%: $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
$q_{c1ncs}$	Clean sand equivalent $q_{c1n}$	$q_{c1ncs} = q_{c1n} \cdot K_{cpt}$	3
$Q_{tn,cs}$ (PKR 2010)	Clean sand equivalent for $Q_{tn}$ described above - $Q_{tn}$ being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
$Su(Liq)/ESv$	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$  Note: $\sigma_v'$ and $s_v'$ are synonymous	13
$Su(Liq)/ESv$ (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{Su(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
$Su(Liq)$ (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress		16
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4} [(N_1)_{60}]^{4.79}$ $qc1$ is calculated from specified qt(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$ : $CRR_{7.5} = 0.833 [q_{c1ncs}/1000] + 0.05$  $50 \leq q_{c1ncs} < 160$ : $CRR_{7.5} = 93 [q_{c1ncs}/1000]^3 + 0.08$	10
$K_g$	Small strain Stiffness Ratio Factor, $K_g$	$[G_{max}/qt]/[q_{c1n}^{-m}]$ $m$ = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Qtn chart from plotted point to state parameter $\Psi = -0.05$ curve	25
URS NP Fr	Normalized friction ratio point on $\Psi = -0.05$ curve used in SP Distance calculation		25
URS NP Qtn	Normalized tip resistance (Qtn) point on $\Psi = -0.05$ curve used in SP Distance calculation		25

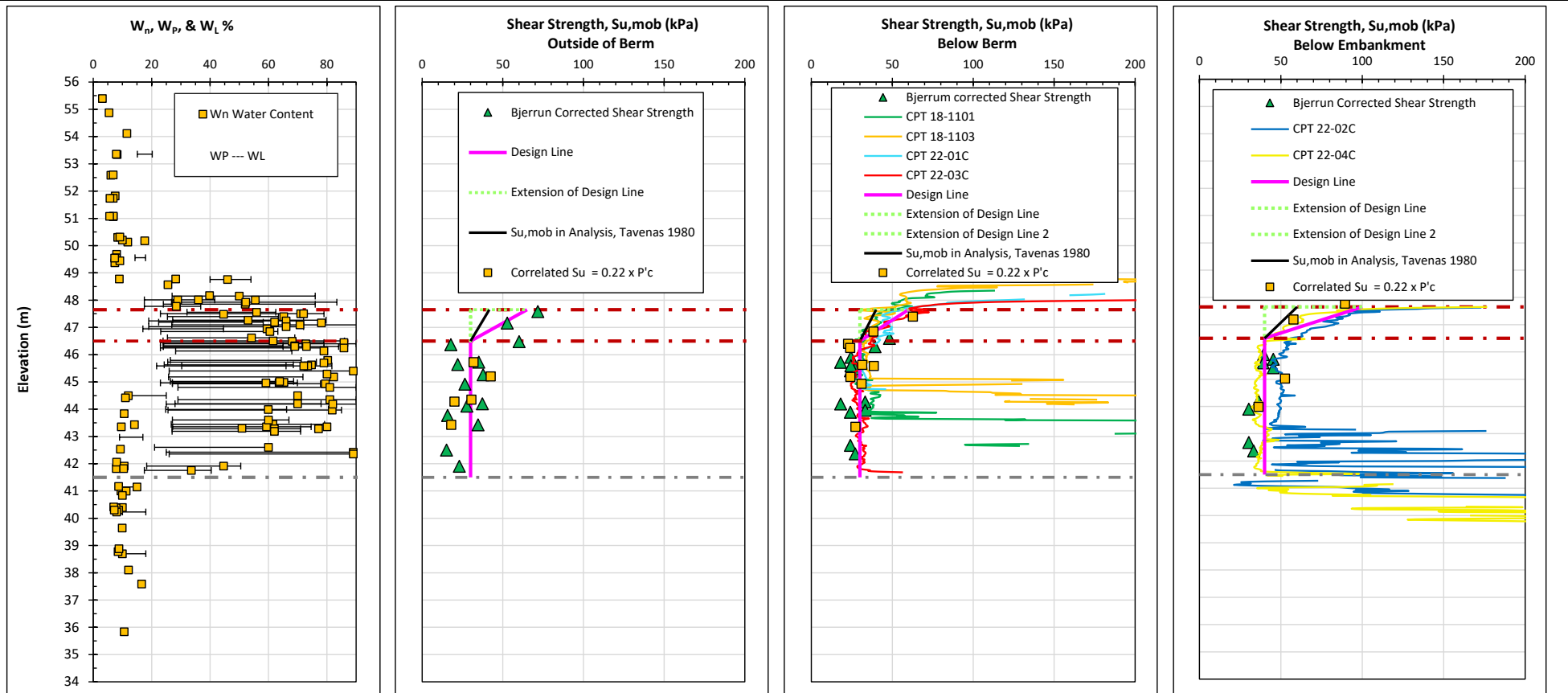
**Table 2. References**

No.	Reference
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**APPENDIX G**

# Slope Stability Figures



Site 31-230 - Highway 401 Underpass at Fraser Road  
United Counties of Stormont, Dundas and Glengarry, Ontario  
Agreement No 4021-E-0021; Assignment 1

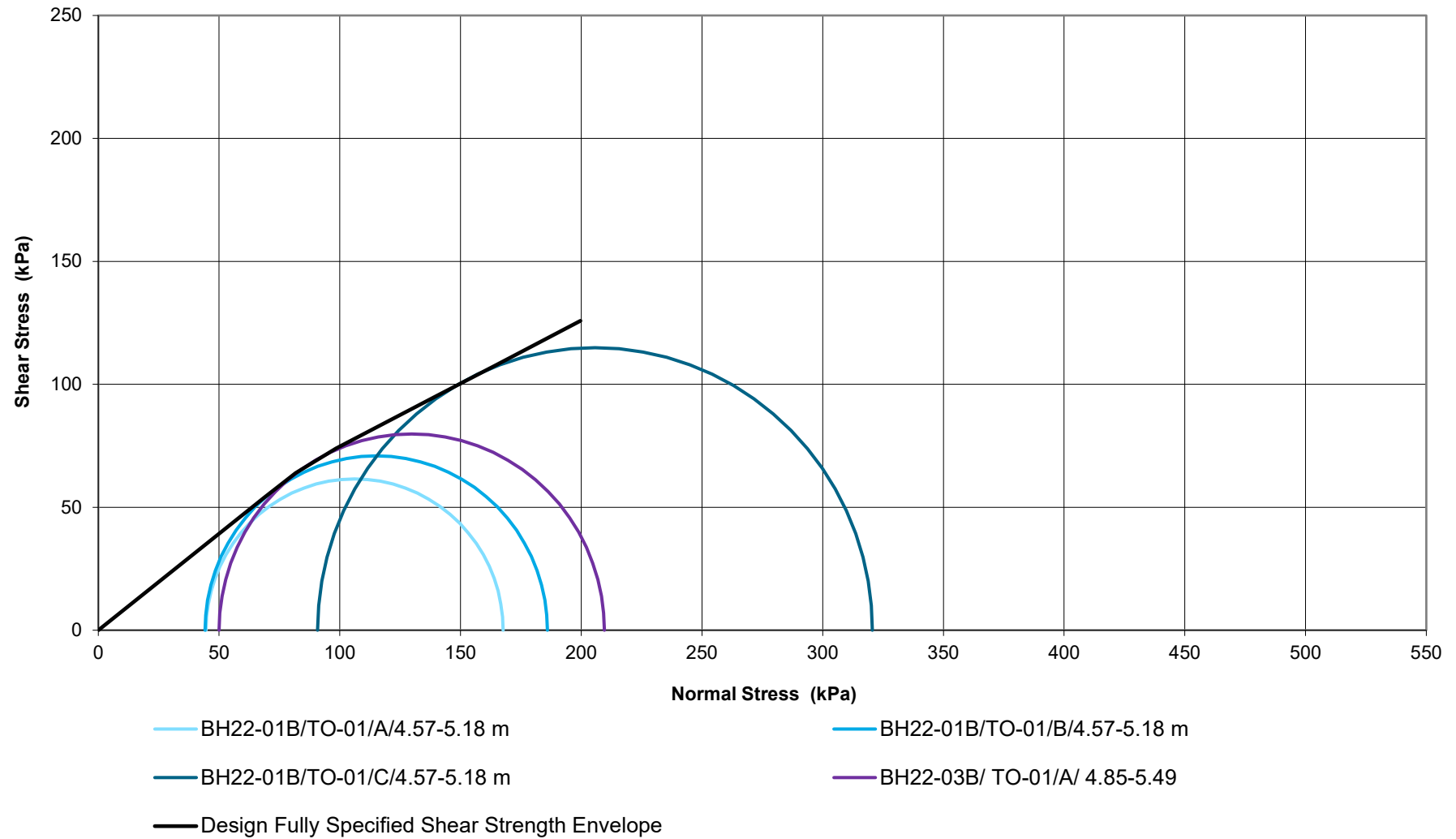
Shear Strength Design Parameters

Project No.: 22513877A  
Date: March 20, 2023  
Prepared by: KG  
Review by: KCP  
Approved by: JPD

FIGURE G1



**FIGURE G2 - Triaxial Test Results - Weathered Crust Peak Fully Specified Shear Strength Envelope**



**FIGURE G3 - Triaxial Test Results - Weathered Crust Post Peak/  
Soften Design Shear Strength Envelope**

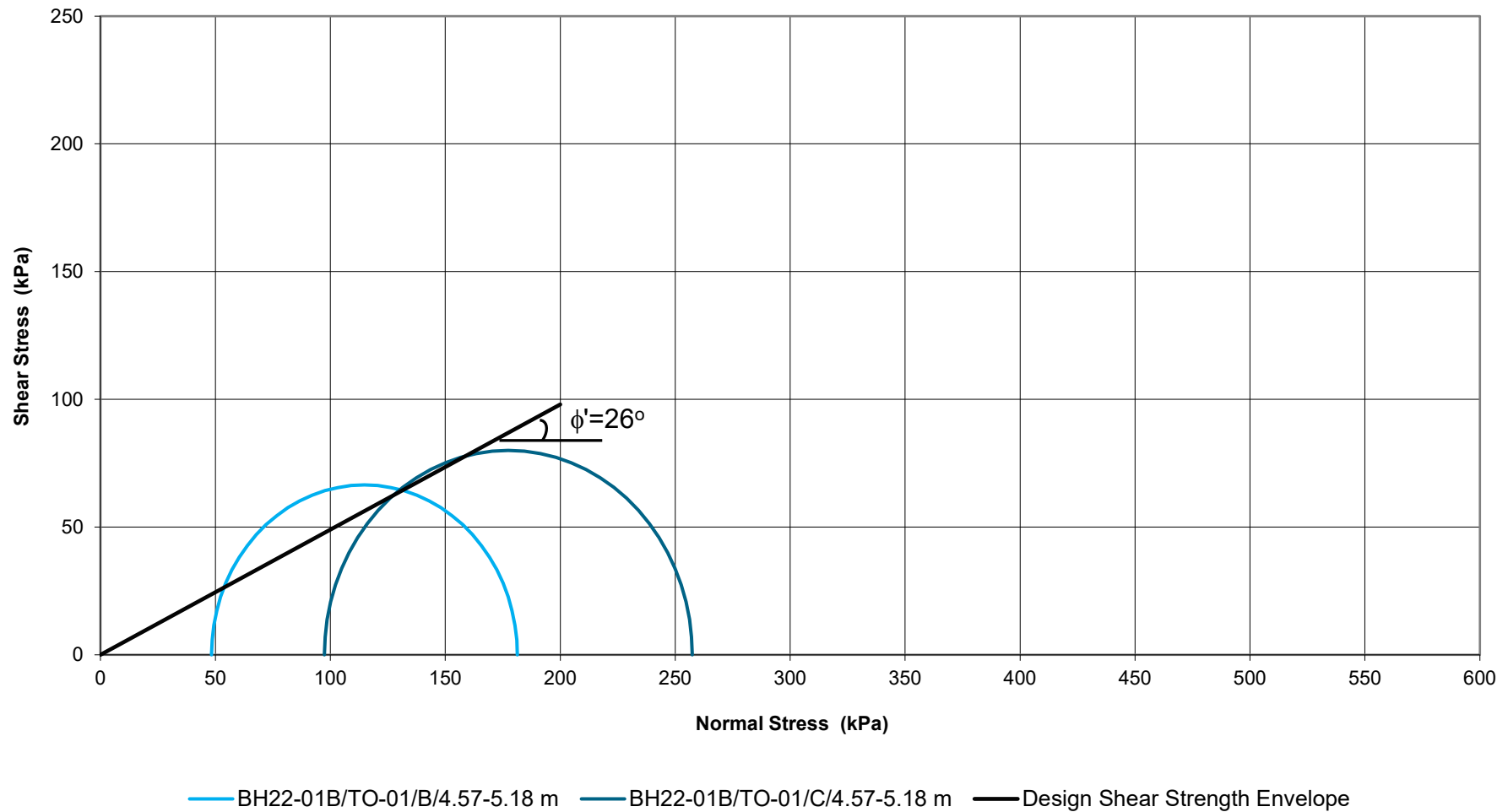
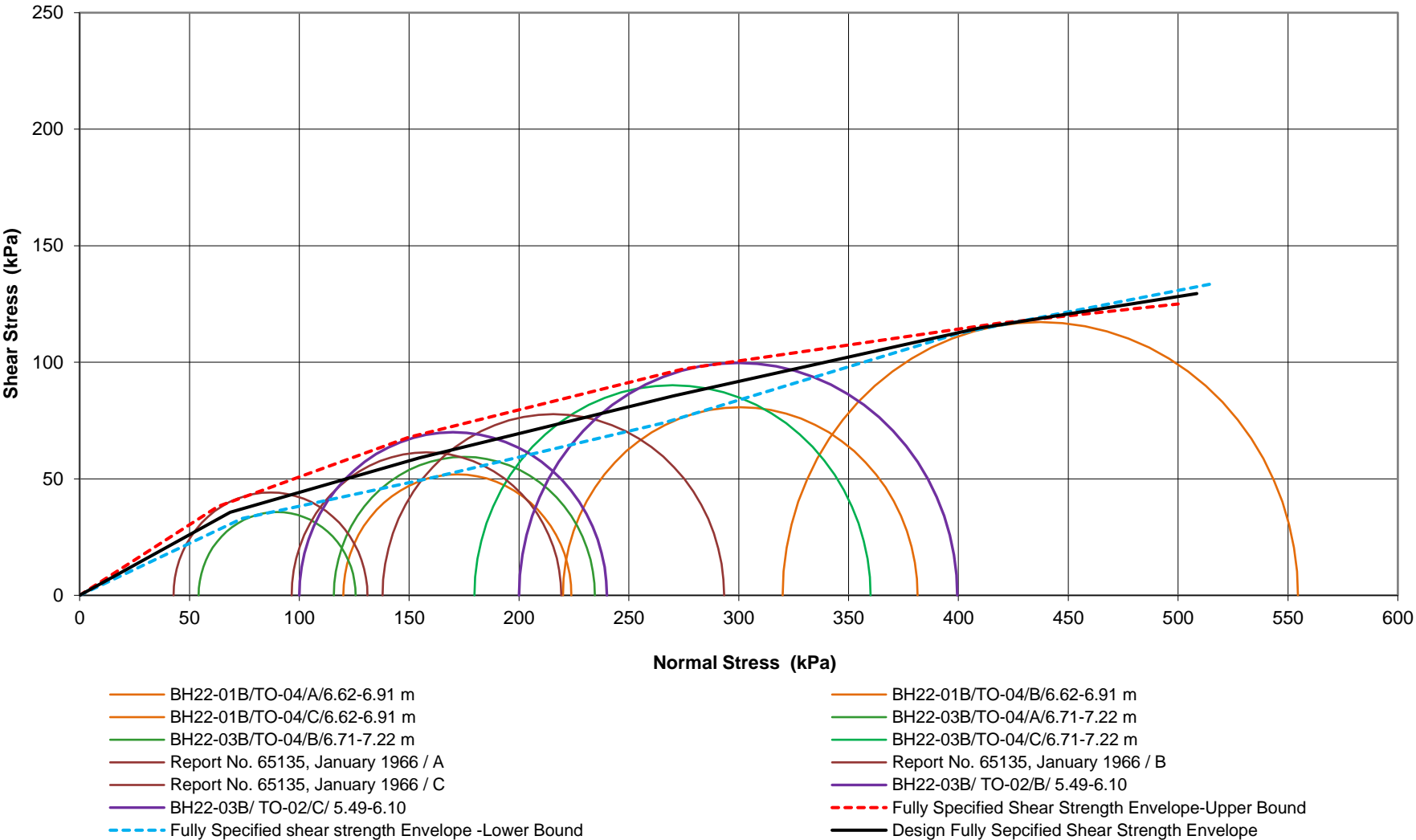
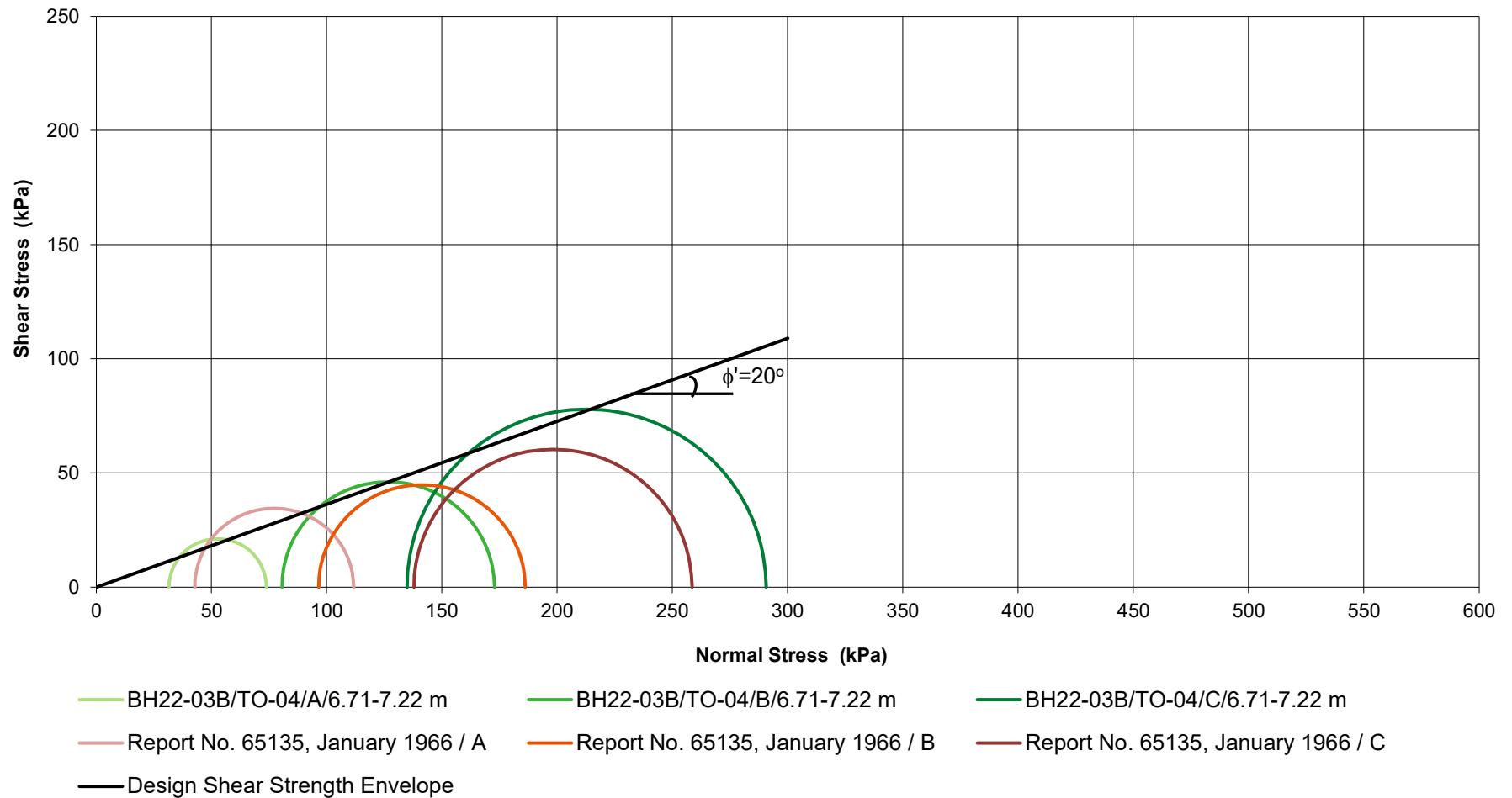


FIGURE G4 - Triaxial Test Results - Grey Clay Peak Fully Specified shear Strength Envelope



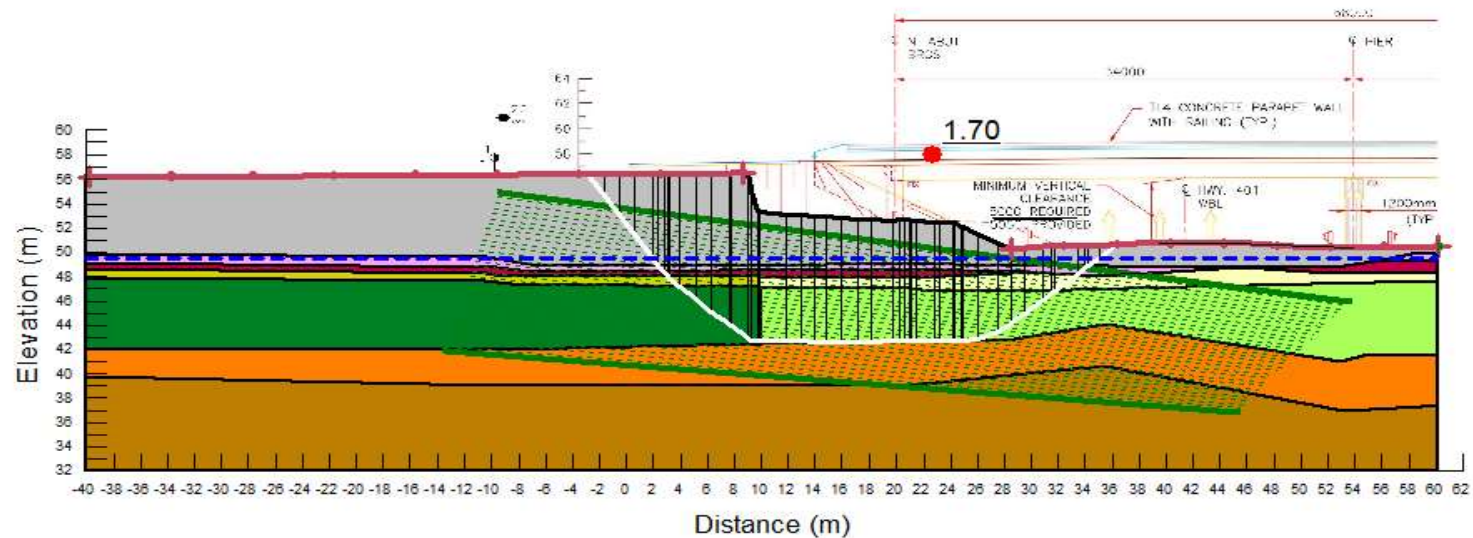
# FIGURE G5 - Triaxial Test Results - Grey Clay

## Post Peak/ Soften Design Shear Strength Envelope



Name: Approach Embankment - Undrained Analysis - Static  
 Analysis Type: Morgenstern-Price  
 Horiz Seismic Coef.:  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m³)/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (impenetrable)							1
	Clay - Below and Outside of Berm	Undrained (Phi=0)	15.25					30	1
	Clay - Below Embankment	Undrained (Phi=0)	15.25					40	1
	Existing Fill	Mohr-Coulomb	21.5			0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Crust - Below and Outside of Berm	S=f(depth)	15.75	40	-8.69				1
	Weathered Crust - Below Embankment	S=f(depth)	15.75	60	-17.4				1



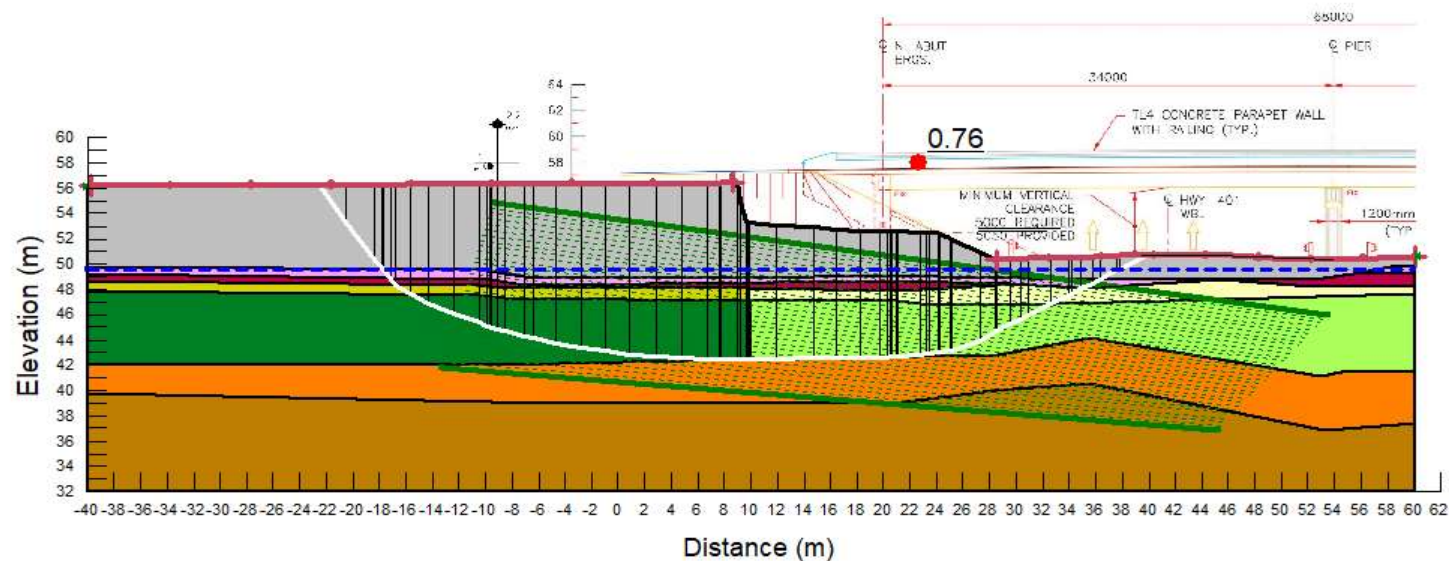
Site 31-230 - Highway 401 Underpass at Fraser Road,  
 United Counties of Stormont, Dundas and Glengarry, Ontario,  
 Agreement No 4021-E-0021; Assignment 1  
 Approach Embankment - Undrained Slope Stability Analysis - Static

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G6

Name: Approach Embankment - Undrained Analysis - Pseudo Static  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.: 0.23  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	C-Top of Layer (kPa)	C-Rate of Change ((kN/m <sup>3</sup> )/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)							1
	Clay - Below and Outside of Berm	Undrained (Phi=0)	15.25					30	1
	Clay - Below Embankment	Undrained (Phi=0)	15.25					40	1
	Existing Fill	Mohr-Coulomb	21.5			0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Crust - Below and Outside of Berm	S=f(depth)	15.75	40	-8.69				1
	Weathered Crust - Below Embankment	S=f(depth)	15.75	60	-17.4				1



Site 31-230 - Highway 401 Underpass at Fraser Road,  
 United Counties of Stormont, Dundas and Glengarry, Ontario,  
 Agreement No 4021-E-0021; Assignment 1  
 Approach Embankment - Undrained Slope Stability Analysis - Pseudo Static

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G7










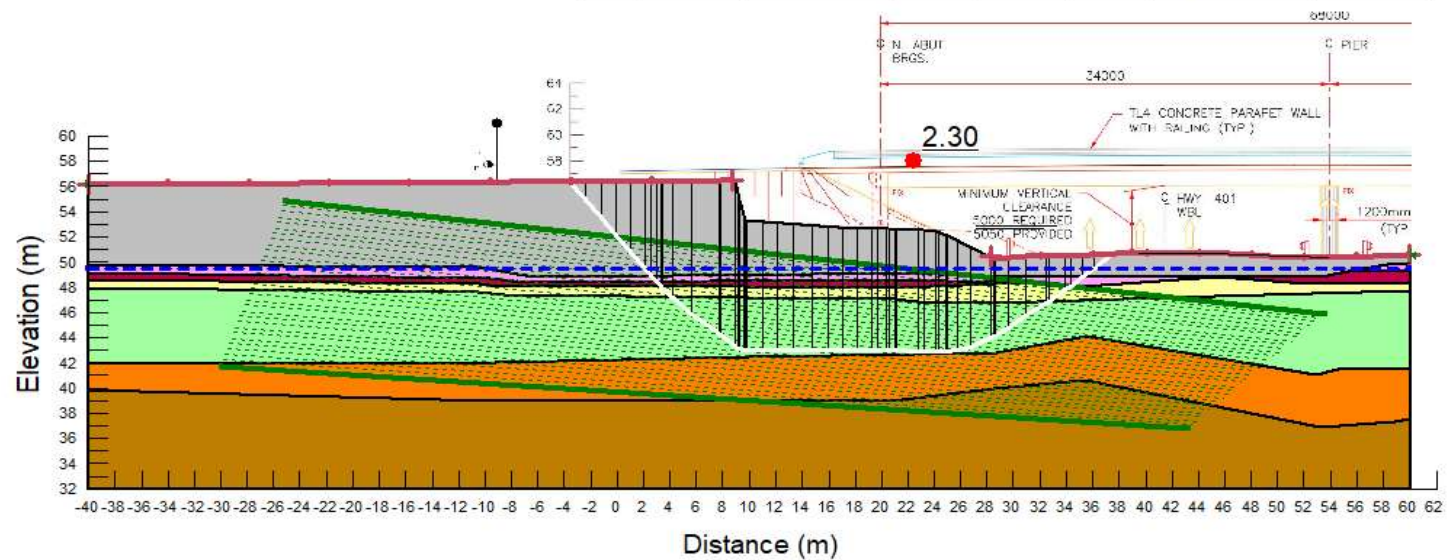
Name: Approach Embankment - Drained Analysis - Peak

Analysis Type: Morgenstern-Price

Horz Seismic Coef.:

Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	Effective Cohesion (kPa)	Effective Friction Angle (°)	Strength Function	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)					1
	Clay	Shear/Normal Fn.	15.25			Design Fully Specified Shear Strength Envelope - Clay	1
	Existing Fill	Mohr-Coulomb	21.5	0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19	0	32		1
	Till	Mohr-Coulomb	22	0	35		1
	Topsoil	Mohr-Coulomb	15	0	25		1
	Weathered Crust	Shear/Normal Fn.	15.75			Design Fully Specified Shear Strength Envelope - Weathered Crust	1



Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1  
Approach Embankment - Drained Slope Stability Analysis - Peak Parameters

Project No: 22513877A  
Drawn: KG  
Date: June 7, 2023  
Checked: KCP  
Review: JPD








FIGURE G8

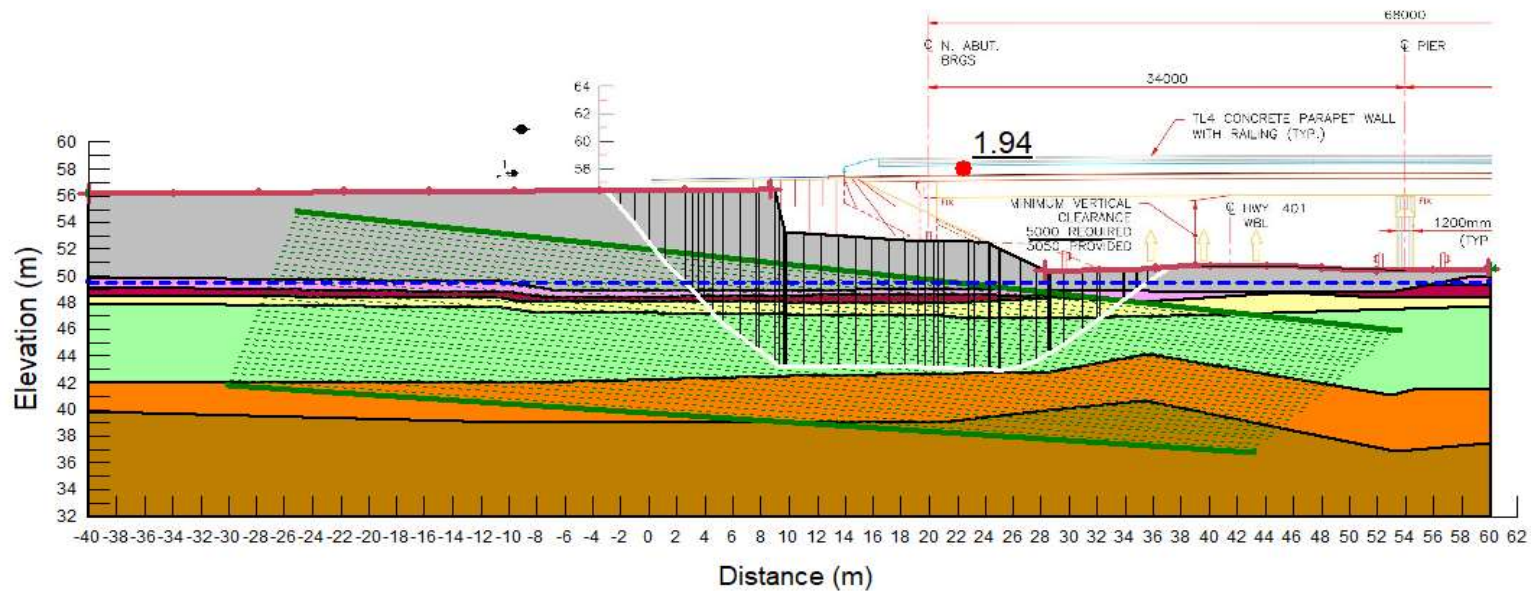
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Analysis Type: Morgenstern-Price

Horz Seismic Coef.:

Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)				1
	Clay	Mohr-Coulomb	15.25	0	20	1
	Existing Fill	Mohr-Coulomb	21.5	0	35	1
	Silt to Sandy Silt	Mohr-Coulomb	19	0	32	1
	Till	Mohr-Coulomb	22	0	35	1
	Topsoil	Mohr-Coulomb	15	0	25	1
	Weathered Crust	Mohr-Coulomb	15.75	0	26	1



Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1

Approach Embankment - Drained Slope Stability Analysis - Post Peak/ Soften Parameters

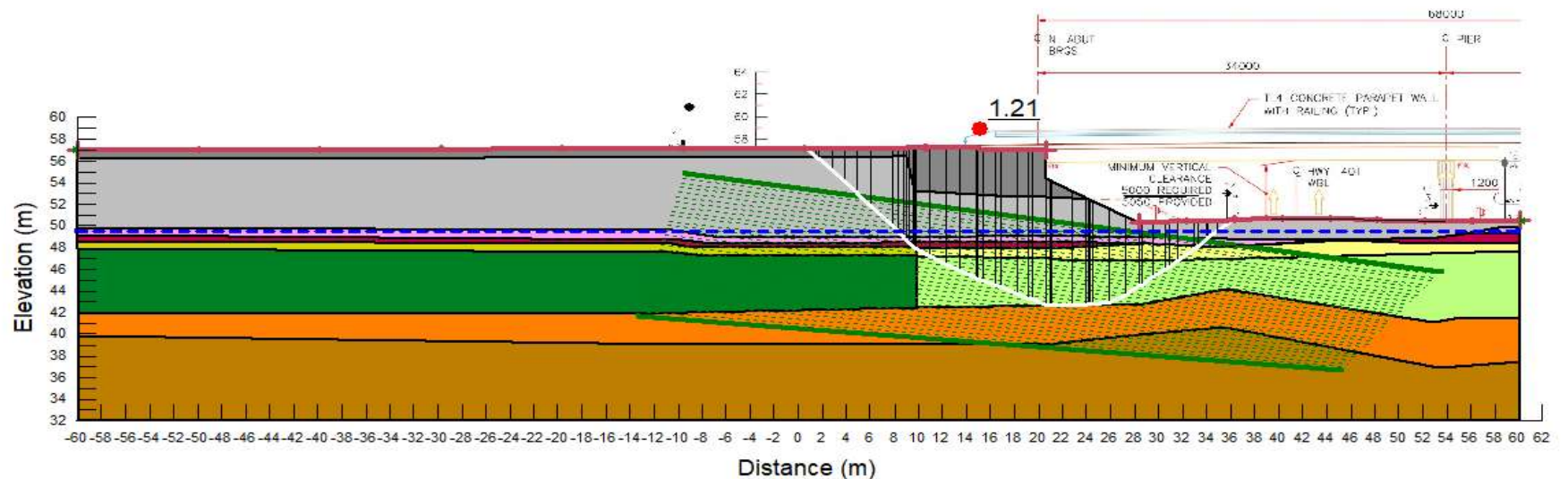
Project No: 22513877A  
Drawn: KG  
Date: June 7, 2023  
Checked: KCP  
Review: JPD

FIGURE G9



Name: Approach Embankment with New Fill- Undrained Analysis -Static  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.: 0  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	C-Top of Layer (kPa)	C-Rate of Change ((kN/m <sup>3</sup> )/m)	Effective Cohesion (kPa)	Effective Friction Angle(°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)							1
	Clay - Below and outside of Berm	Undrained (Phi=0)	1525					30	1
	Clay - Below Embankment	Undrained (Phi=0)	1525					40	1
	Existing Fill	Mohr-Coulumb	21.5			0	35		1
	New Fill	Mohr-Coulumb	21.5			0	35		1
	Silt to Sandy Silt	Mohr-Coulumb	19			0	32		1
	Till	Mohr-Coulumb	22			0	35		1
	Topsoil	Mohr-Coulumb	15			0	25		1
	Weathered Crust Clay - Below and outside of Berm	S=f(depth)	1575	40	-8.69				1
	Weathered Crust Clay - Below Embankment	S=f(depth)	1575	60	-17.4				1



Site 31-230 - Highway 401 Underpass at Fraser Road,  
 United Counties of Stormont, Dundas and Glengarry, Ontario,  
 Agreement No 4021-E-0021; Assignment 1

Approach Embankment with New Fill- Undrained Slope Stability Analysis - Static

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD











FIGURE G10

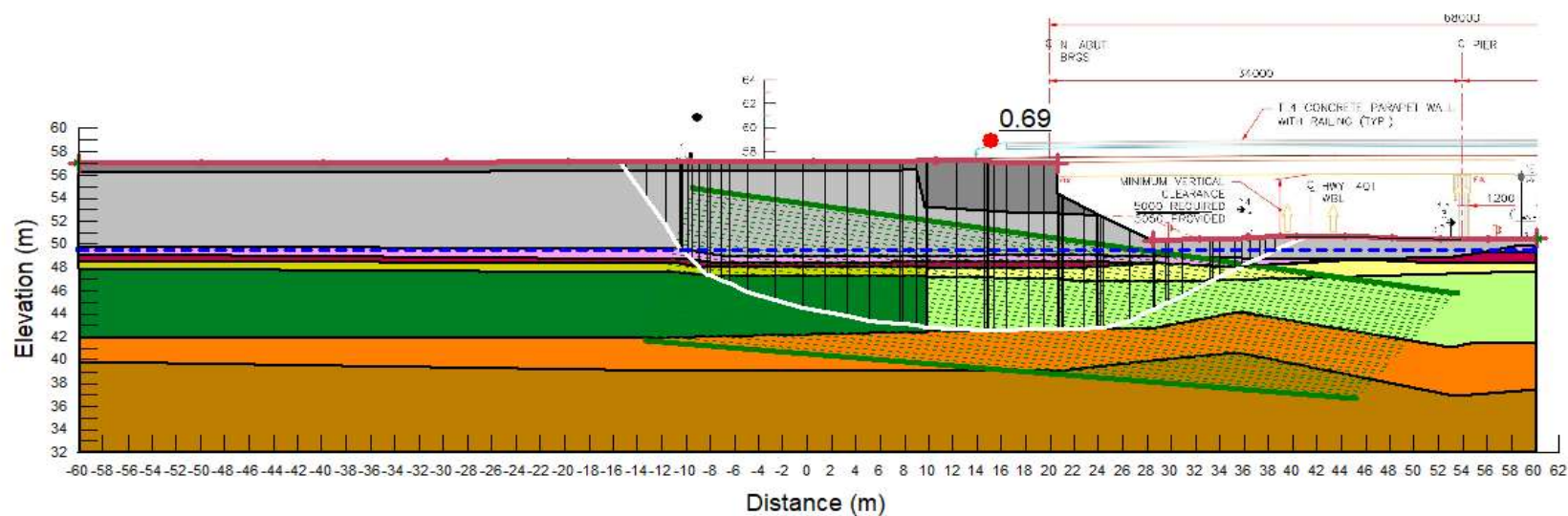
Name: Approach Embankment with New Fill- Undrained Analysis -Pseudo Static

Analysis Type: Morgenstern-Price

Horz Seismic Coef.: 0.23

Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	C-Top of Layer (kPa)	C-Rate of Change ((kN/m <sup>3</sup> )/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)							1
	Clay - Below and outside of Berm	Undrained (Ph=0)	1525					30	1
	Clay - Below Embankment	Undrained (Ph=0)	1525					40	1
	Existing Fill	Mohr-Coulomb	215			0	35		1
	New Fill	Mohr-Coulomb	215			0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Crust Clay - Below and outside of Berm	S=f(depth)	1575	40	-8.69				1
	Weathered Crust Clay - Below Embankment	S=f(depth)	1575	60	-17.4				1



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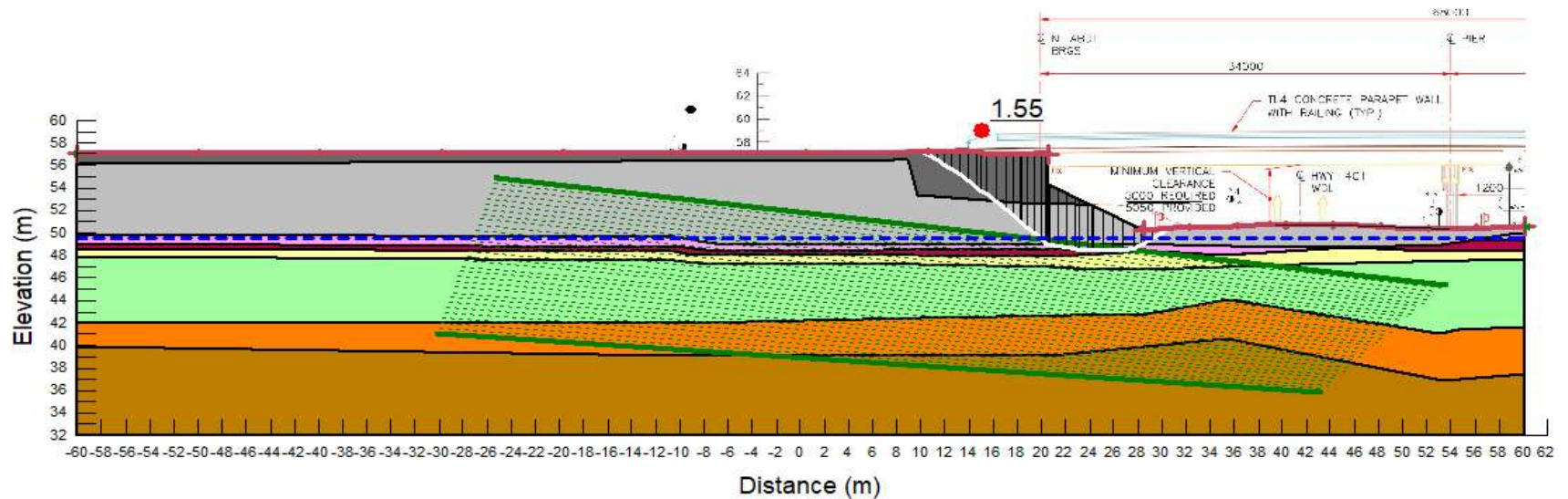
Approach Embankment with New Fill- Undrained Slope Stability Analysis - Pseudo Static

Project No: 22513877A  
Drawn: KG  
Date: June 7, 2023  
Checked: KCP  
Review: JPD

FIGURE G11

Name: Approach Embankment with Fill- Drained Analysis-Peak  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.:  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	Effective Cohesion (kPa)	Effective Friction Angle (°)	Strength Function	Piezometric Surface
	Bedrock	Bedrock (Impermeable)					1
	Clay	Shear/Normal Fr.	1525			Design Fully Specified Shear Strength Envelope - Clay	1
	Existing Fill	Mohr-Coulomb	215	0	35		1
	New Fill	Mohr-Coulomb	215	0	35		1
	Silty/Sandy Silt	Mohr-Coulomb	19	0	32		1
	Till	Mohr-Coulomb	22	0	35		1
	Topsoil	Mohr-Coulomb	15	0	25		1
	Weathered Crust	Shear/Normal Fr.	1575			Design Fully Specified Shear Strength Envelope - Weathered Crust	1



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 United Counties of Stormont, Dundas and Glengarry, Ontario,  
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Approach Embankment with New Fill- Drained Slope Stability Analysis - Peak Parameters

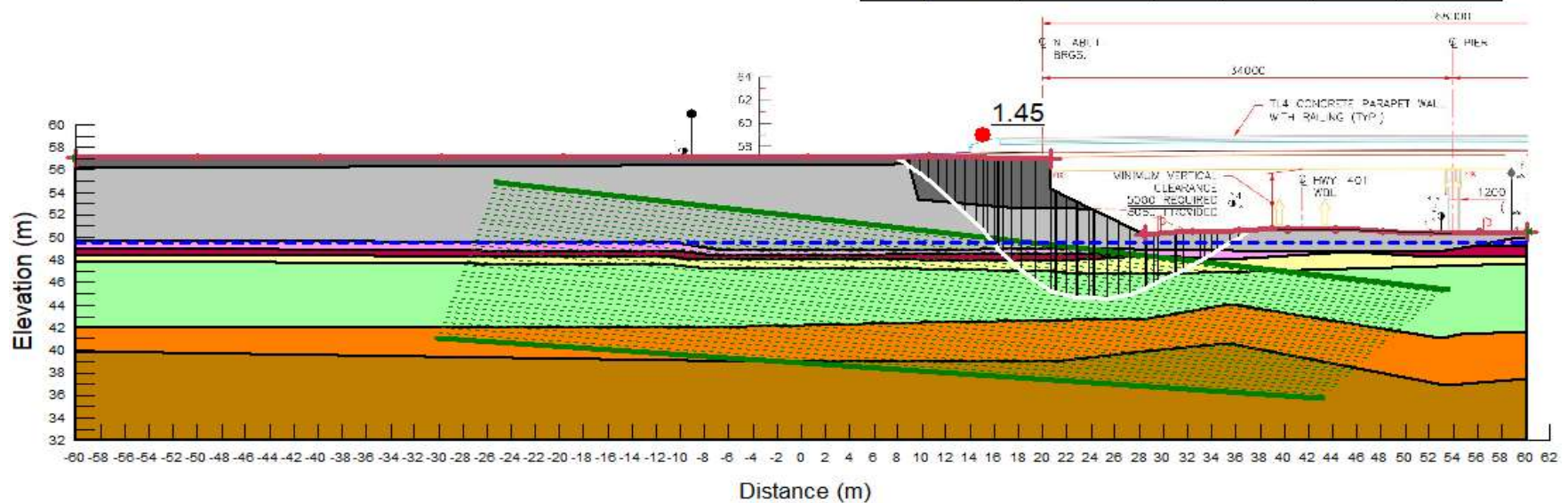
Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G12



Name: Approach Embankment with Fill- Drained Analysis-Post Peak/ Soften  
Analysis Type: Morgenstern-Price  
Horz Seismic Coef.:  
Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Bed rock	Bedrock (Impenetrable)				1
	Clay	Mohr-Coulomb	15.25	0	20	1
	Existing Fill	Mohr-Coulomb	21.5	0	35	1
	New Fill	Mohr-Coulomb	21.5	0	35	1
	Silt to Sandy Silt	Mohr-Coulomb	19	0	32	1
	Till	Mohr-Coulomb	22	0	35	1
	Topsoil	Mohr-Coulomb	15	0	25	1
	Weathered Crust	Mohr-Coulomb	15.75	0	28	1



**Site 31-230 - Highway 401 Underpass at Fraser Road,  
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Agreement No 4021-E-0021; Assignment 1**

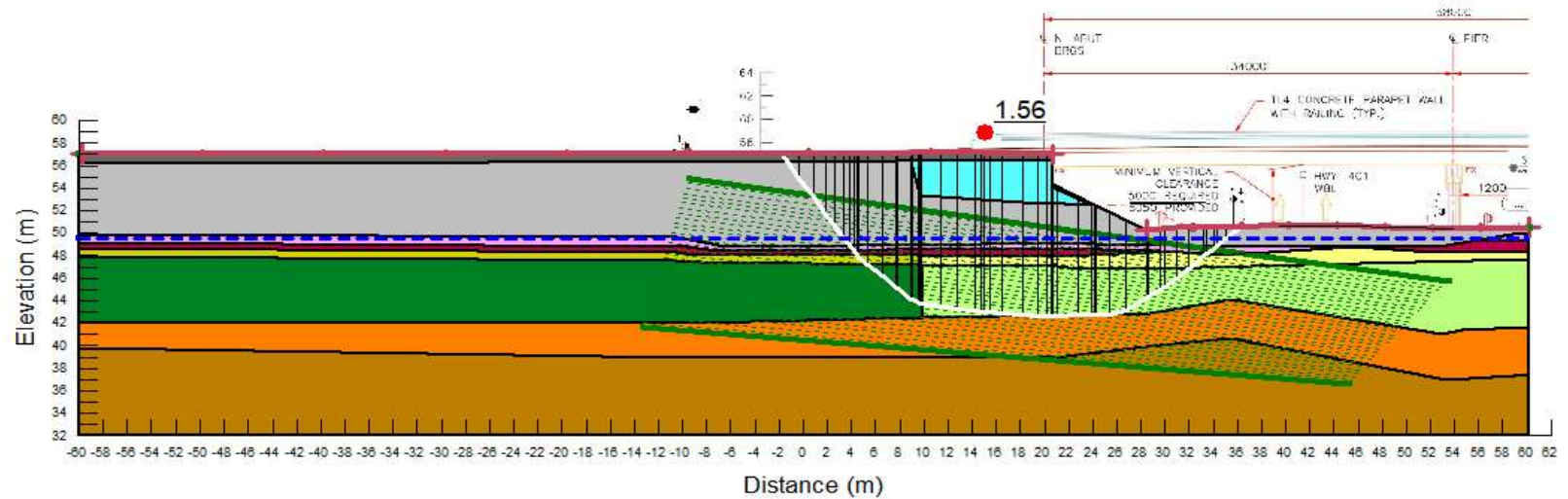
### Approach Embankment with New Fill- Drained Slope Stability Analysis - Post Peak/ Soften Parameters

Project No:	22513877A
Drawn:	KG
Date:	June 7, 2023
Checked:	KCP
Review:	JPD

**FIGURE G13**

Name: Approach Embankment with New Fill and EPS- Undrained Analysis - Static  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.: 0  
 Groundwater Elevation Y: 49.5 m

Cobr	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	C-Top of Layer (kPa)	C-Rate of Change ((kN/m <sup>3</sup> )/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)							1
	Clay - Below and outside of Berm	Undrained (Phi=0)	1525					30	1
	Clay - Below Embankment	Undrained (Phi=0)	1525					40	1
	EPS	Undrained (Phi=0)	1					15	1
	Existing Fill	Mohr-Coulomb	21.5			0	35		1
	New Fill	Mohr-Coulomb	21.5			0	35		1
	Silty/Sandy Silt	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Crust Clay - Below and outside of Berm	S=(depth)	1575	40	-8.69				1
	Weathered Crust Clay - Below Embankment	S=(depth)	1575	60	-17.4				1



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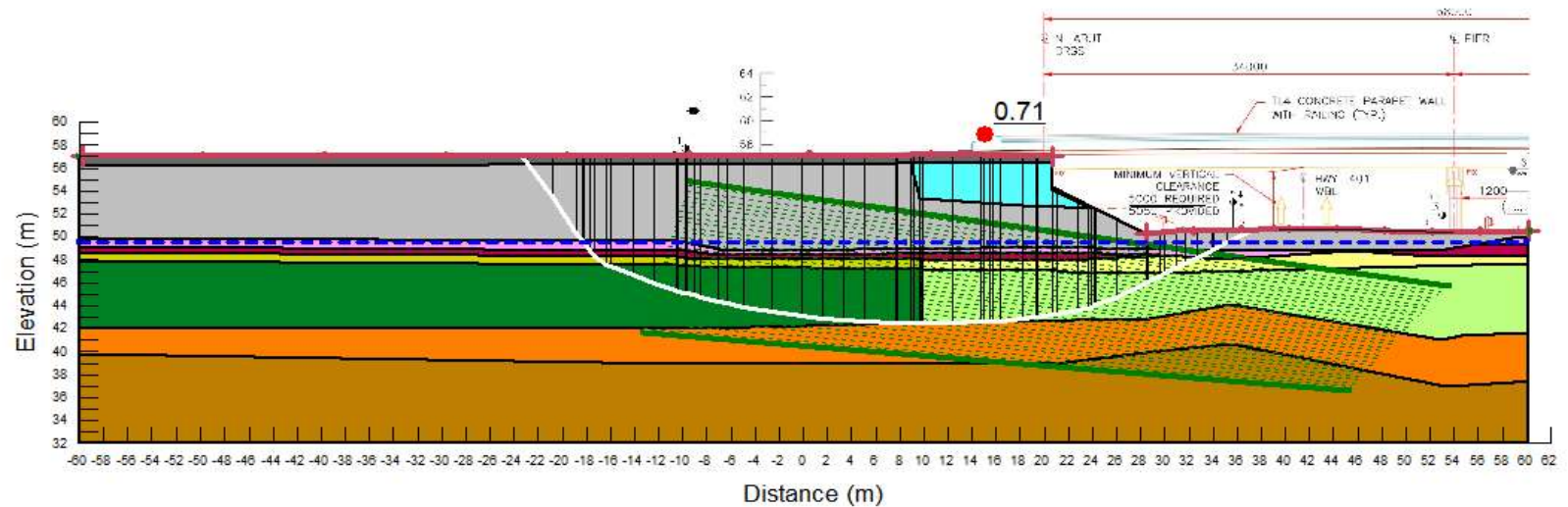
Approach Embankment with New Fill and EPS- Undrained Slope Stability Analysis - Static

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G14

Name: Approach Embankment with New Fill and EPS- Undrained Analysis - Pseudo Static  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.: 0.23  
 Groundwater Elevation Y: 49.5 m

Cobr	Name	Slope Stability Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change (kN/m³/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impermeable)							1
	Clay - Below and outside of Berm	Undrained (Phi=0)	15.25					30	1
	Clay - Below Embankment	Undrained (Phi=0)	15.25					40	1
	EPS	Undrained (Phi=0)	1					15	1
	Existing Fill	Mohr-Coulomb	21.5			0	35		1
	New Fill	Mohr-Coulomb	21.5			0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Crust Clay - Below and outside of Berm	Strain(depth)	15.75	40	-8.89				1
	Weathered Crust Clay - Below Embankment	Strain(depth)	15.75	60	-17.4				1



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Approach Embankment with New Fill and EPS- Undrained Slope Stability Analysis - Pseudo Static

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G15



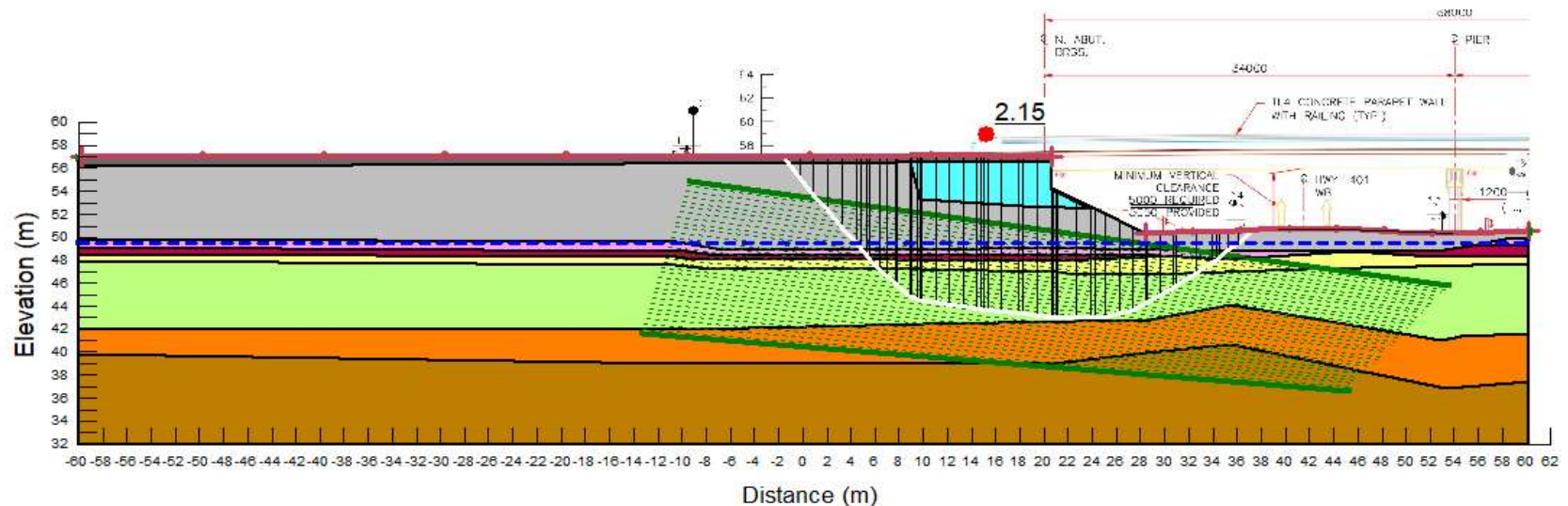
Name: Approach Embankment with New Fill and EPS- Drained Analysis - Peak

Analysis Type: Morgenstern-Price

Horz Seismic Coef.:

Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Strength Function	Piezometric Surface
	Bedrock	Bedrock (Impermeable)						1
	Clay	Shear/Normal Fn	1525				Design/Fully Specified Shear Strength Envelope - Clay	1
	EPS	Undrained (Phi=0)	1			15		1
	Existing Fill	Mohr-Coulomb	21.5	0	35			1
	New Fill	Mohr-Coulomb	21.5	0	35			1
	Silt to Sandy Silt	Mohr-Coulomb	19	0	32			1
	Till	Mohr-Coulomb	22	0	35			1
	Topsoil	Mohr-Coulomb	15	0	25			1
	Weathered Crust	Shear/Normal Fn	1575				Design/Fully Specified Shear Strength Envelope - Weathered Crust	1



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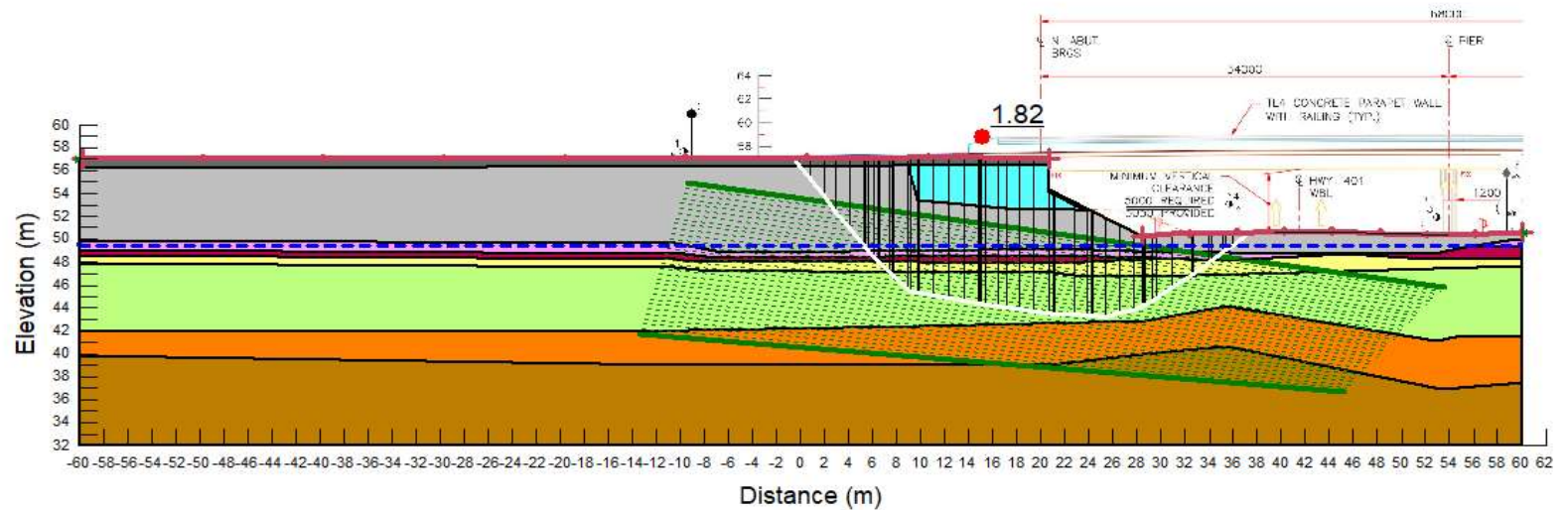
Approach Embankment with New Fill and EPS- Drained Slope Stability Analysis - Peak

Project No: 22513877A  
Drawn: KG  
Date: June 7, 2023  
Checked: KCP  
Review: JPD

FIGURE G16

Name: Approach Embankment with New Fill and EPS- Drained Analysis - Post Peak/ Soften  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.:  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kNm <sup>3</sup> )	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)					1
	Clay	Mohr-Coulomb	15.25		0	20	1
	EPS	Undrained (Phi=0)	1	15			1
	Existing Fill	Mohr-Coulomb	21.5		0	35	1
	New Fill	Mohr-Coulomb	21.5		0	35	1
	Silt to Sandy Silt	Mohr-Coulomb	19		0	32	1
	Till	Mohr-Coulomb	22		0	35	1
	Topsoil	Mohr-Coulomb	15		0	25	1
	Weathered Crust	Mohr-Coulomb	15.75		0	26	1



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Approach Embankment with New Fill and EPS- Drained Slope Stability Analysis - Post Peak/ Soften

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G17



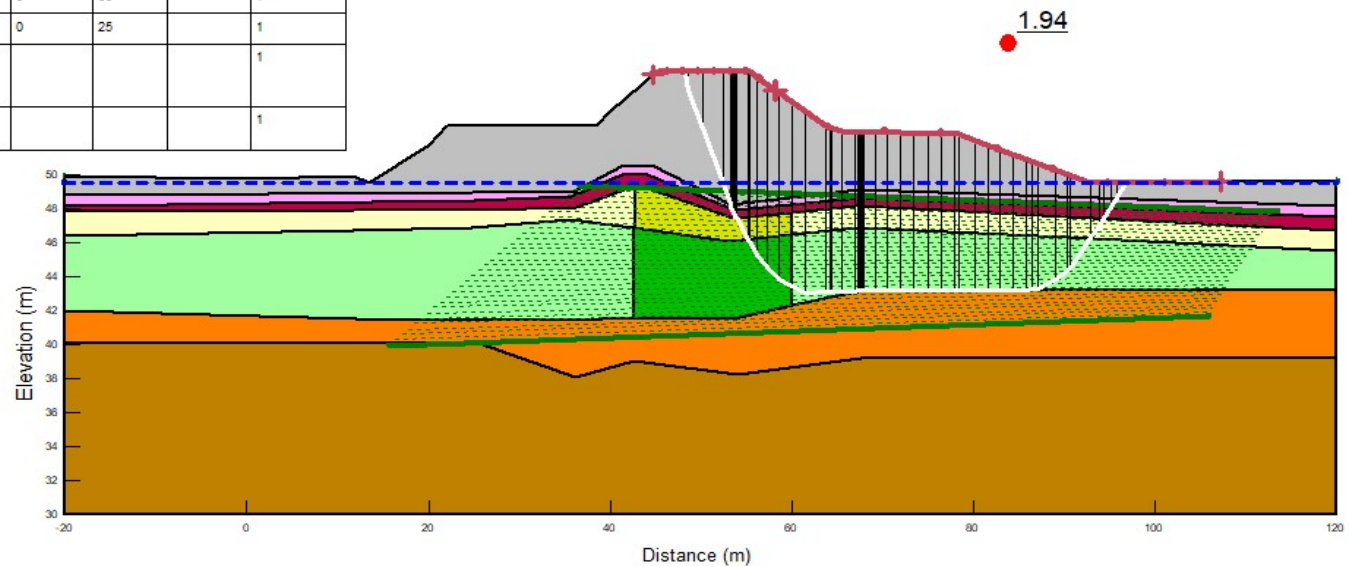
Name: East West Side Slope - Undrained Analysis - Static

Analysis Type: Morgenstern-Price

Horz Seismic Coef.:

Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	C-Top of Layer (kPa)	C-Rate of Change ((kN/m <sup>3</sup> )/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)							1
	Clay - Below and Outside of Berm	Undrained (Phi=0)	15.25					30	1
	Clay - Below Embankment	Undrained (Phi=0)	15.25					40	1
	Existing Fill	Mohr-Coulomb	21.5			0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Crust Clay - Below and Outside of Berm	S=f(depth)	15.75	40	-8.89				1
	Weathered Crust Clay - Below Embankment	S=f(depth)	15.75	60	-17.39				1



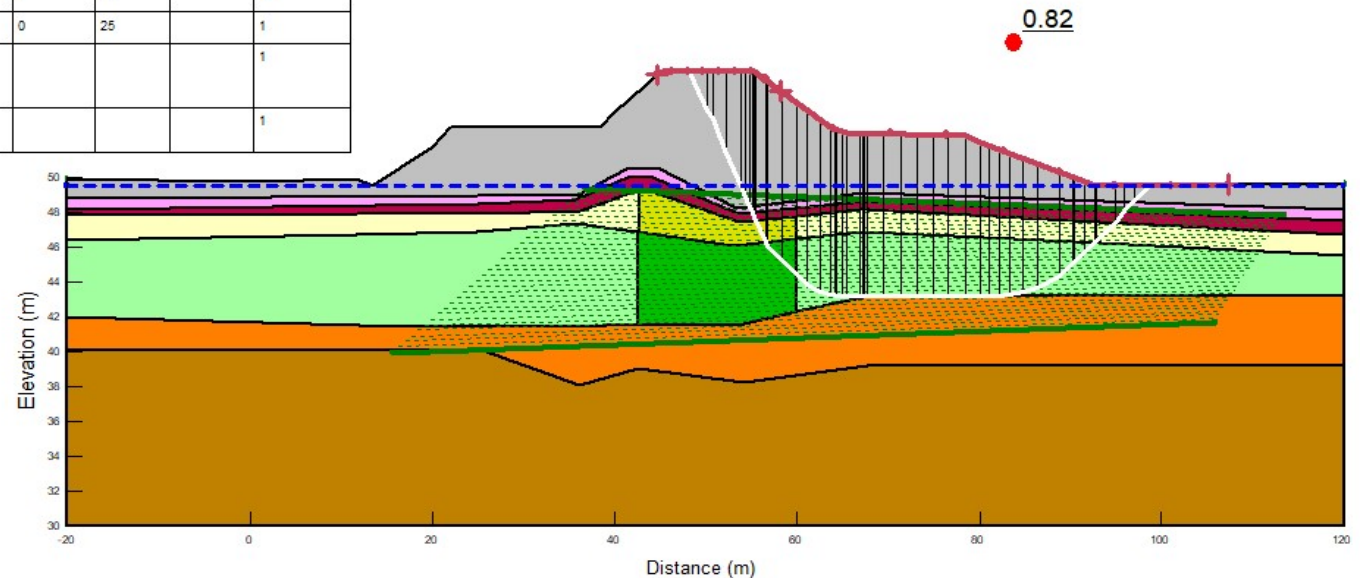
Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1  
Existing East West Side Slope - Undrained Slope Stability Analysis - Static

Project No: 22513877A  
Drawn: KG  
Date: June 7, 2023  
Checked: KCP  
Review: JPD

FIGURE G18

Name: East West Side Slope - Undrained Analysis - Psuedo Static  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.: 0.23  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	C-Top of Layer (kPa)	C-Rate of Change ((kN/m <sup>3</sup> )/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)							1
	Clay - Below and Outside of Berm	Undrained (Ph=0)	15.25					30	1
	Clay - Below Embankment	Undrained (Ph=0)	15.25					40	1
	Existing Fill	Mohr-Coulomb	21.5			0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Crust Clay - Below and Outside of Berm	S=f(depth)	15.75	40	-8.89				1
	Weathered Crust Clay - Below Embankment	S=f(depth)	15.75	80	-17.39				1



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 United Counties of Stormont, Dundas and Glengarry, Ontario,  
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 Existing East West Side Slope - Undrained Slope Stability Analysis - Pseudo Static

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD








FIGURE G19

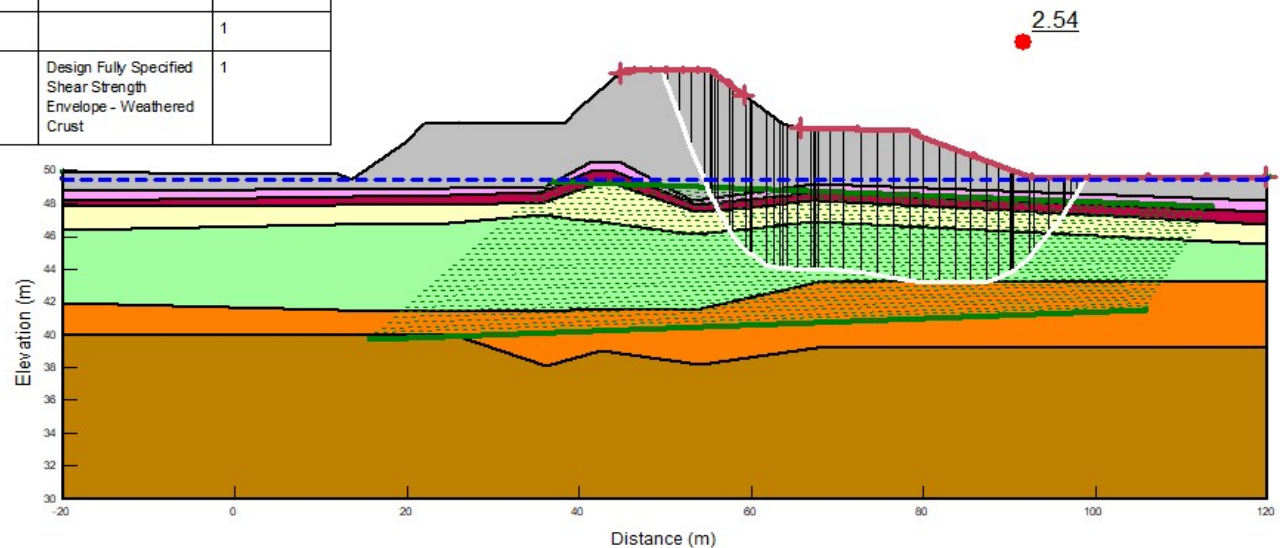
Name: East West Side Slope - Drained Analysis-Peak

Analysis Type: Morgenstern-Price

Horz Seismic Coef.:

Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kNm <sup>3</sup> )	Effective Cohesion (kPa)	Effective Friction Angle (°)	Strength Function	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)					1
	Clay	Shear/Normal Fn.	15.25			Design Fully Specified Shear Strength Envelope - Clay	1
	Existing Fill	Mohr-Coulomb	21.5	0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19	0	32		1
	Till	Mohr-Coulomb	22	0	35		1
	Topsoil	Mohr-Coulomb	15	0	25		1
	Weathered Crust	Shear/Normal Fn.	15.75			Design Fully Specified Shear Strength Envelope - Weathered Crust	1










Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1

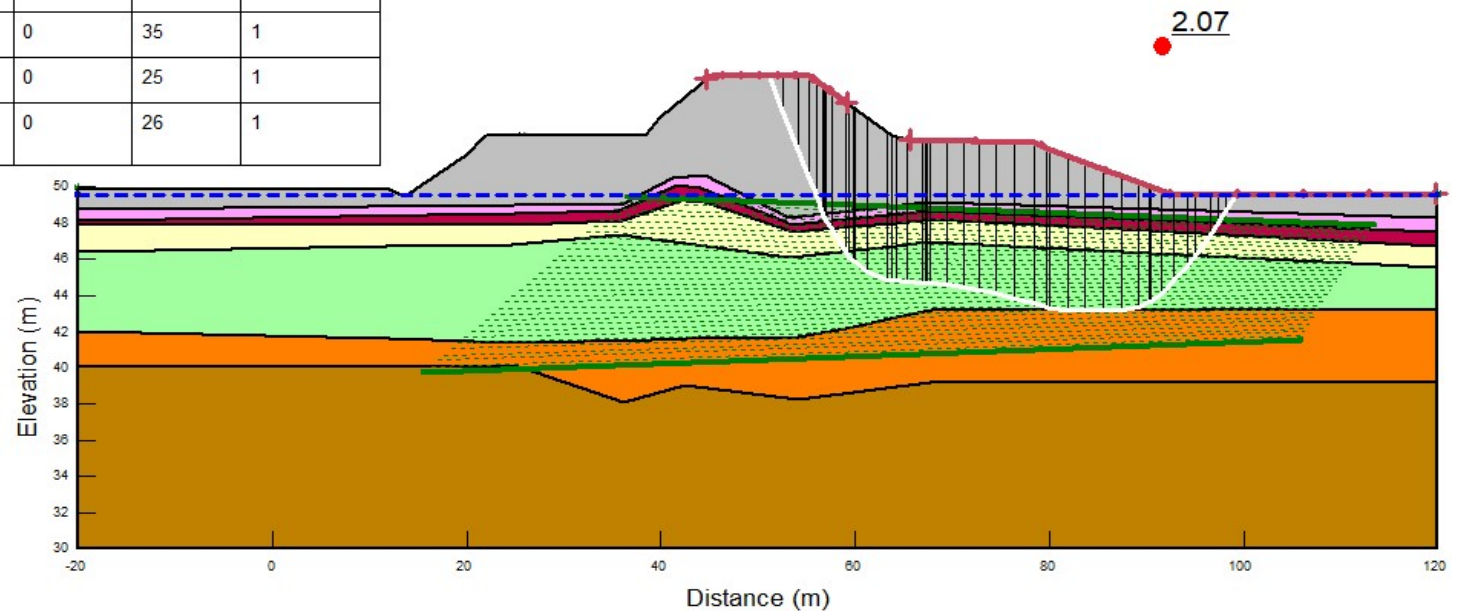
Existing East West Side Slope - Drained Slope Stability Analysis - Peak Parameters

Project No: 22513877A  
Drawn: KG  
Date: June 7, 2023  
Checked: KCP  
Review: JPD

FIGURE G20

Name: East West Side Slope - Drained Analysis - Post Peak/ Soften  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.:  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)				1
	Clay	Mohr-Coulomb	15.25	0	20	1
	Existing Fill	Mohr-Coulomb	21.5	0	35	1
	Silt to Sandy Silt	Mohr-Coulomb	19	0	32	1
	Till	Mohr-Coulomb	22	0	35	1
	Topsoil	Mohr-Coulomb	15	0	25	1
	Weathered Crust	Mohr-Coulomb	15.75	0	26	1



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 United Counties of Stormont, Dundas and Glengarry, Ontario,  
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East West Side Slope - Drained Slope Stability Analysis - Post Peak/ Soften Parameters

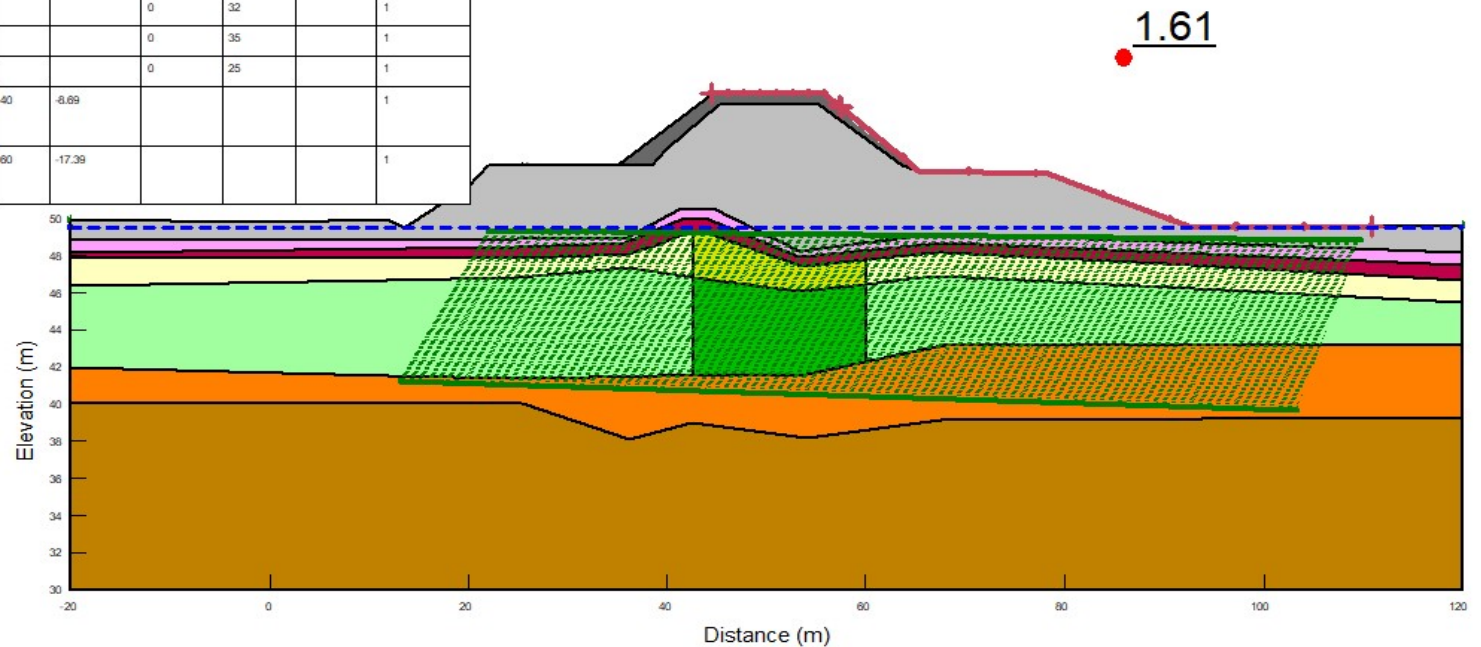
Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G21



Name: 2.75H:1V East West Side Slope with New Fill- Undrained Analysis - Static  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.:  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m³)/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impermeable)							1
	Clay - Below and outside of Berm	Undrained (P <sub>u</sub> =0)	1525					30	1
	Clay - Below Embankment	Undrained (P <sub>u</sub> =0)	1525					40	1
	Existing Fill	Mohr-Coulomb	21.5			0	35		1
	New Fill	Mohr-Coulomb	21.5			0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Crust Clay - Below and outside of Berm	S=f(depth)	1525	40	-6.69				1
	Weathered Crust Clay - Below Embankment	S=f(depth)	1575	60	-17.39				1



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2.75H:1V East West Side Slope with New Fill- Undrained Slope Stability Analysis - Static

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G22

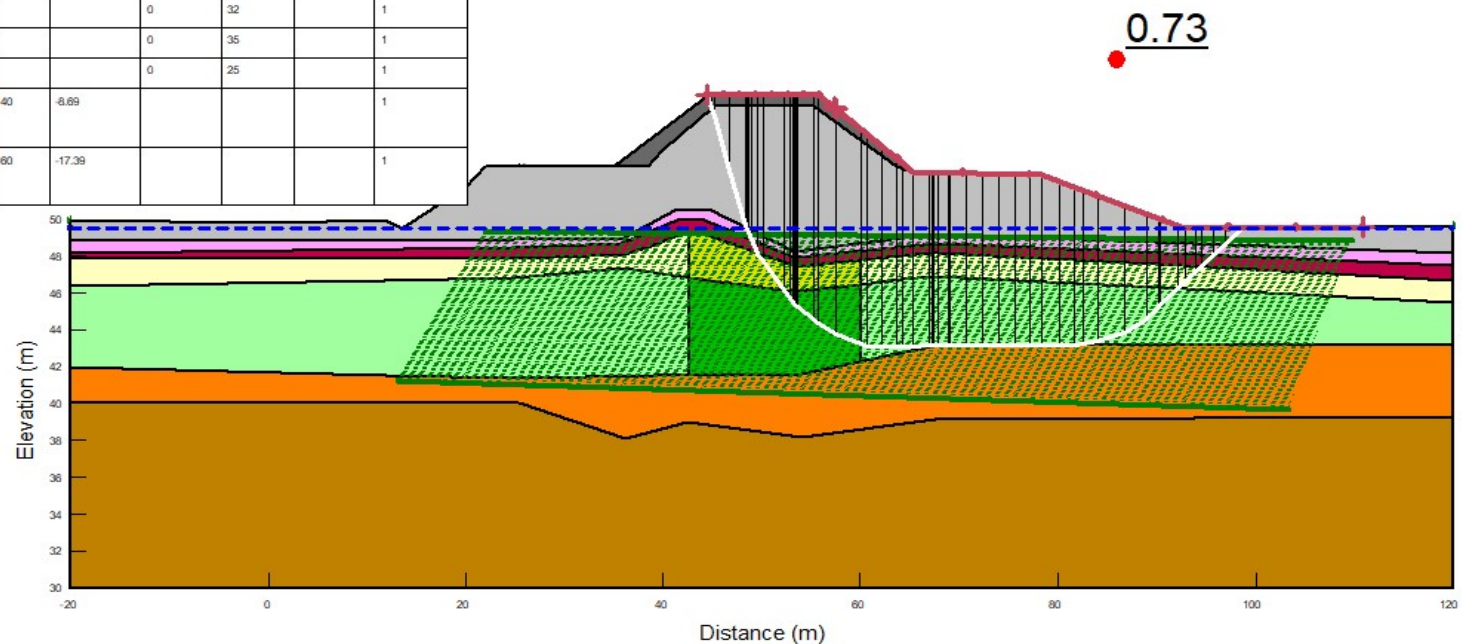
Name: 2.75H:1V East West Side Slope with New Fill- Undrained Analysis - Pseudo Static

Analysis Type: Morgenstern-Price

Horz Seismic Coef.: 0.23

Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	C-Top of Layer (kPa)	C-Rate of Change (kN/m <sup>2</sup> /m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impervious)							1
	Clay - Below and outside of Berm	Undrained (Phi=0)	15.25					30	1
	Clay - Below Embankment	Undrained (Phi=0)	15.25					40	1
	Existing Fill	Mohr-Coulomb	21.5			0	35		1
	New Fill	Mohr-Coulomb	21.5			0	35		1
	Silty Sandy Silt	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Crust Clay - Below and outside of Berm	S=f(depth)	15.25	40	-6.69				1
	Weathered Crust Clay - Below Embankment	S=f(depth)	15.75	60	-17.39				1



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2.75H:1V East West Side Slope with New Fill- Undrained Slope Stability Analysis - Pseudo Static

Project No: 22513877A  
Drawn: KG  
Date: June 7, 2023  
Checked: KCP  
Review: JPD









FIGURE G23

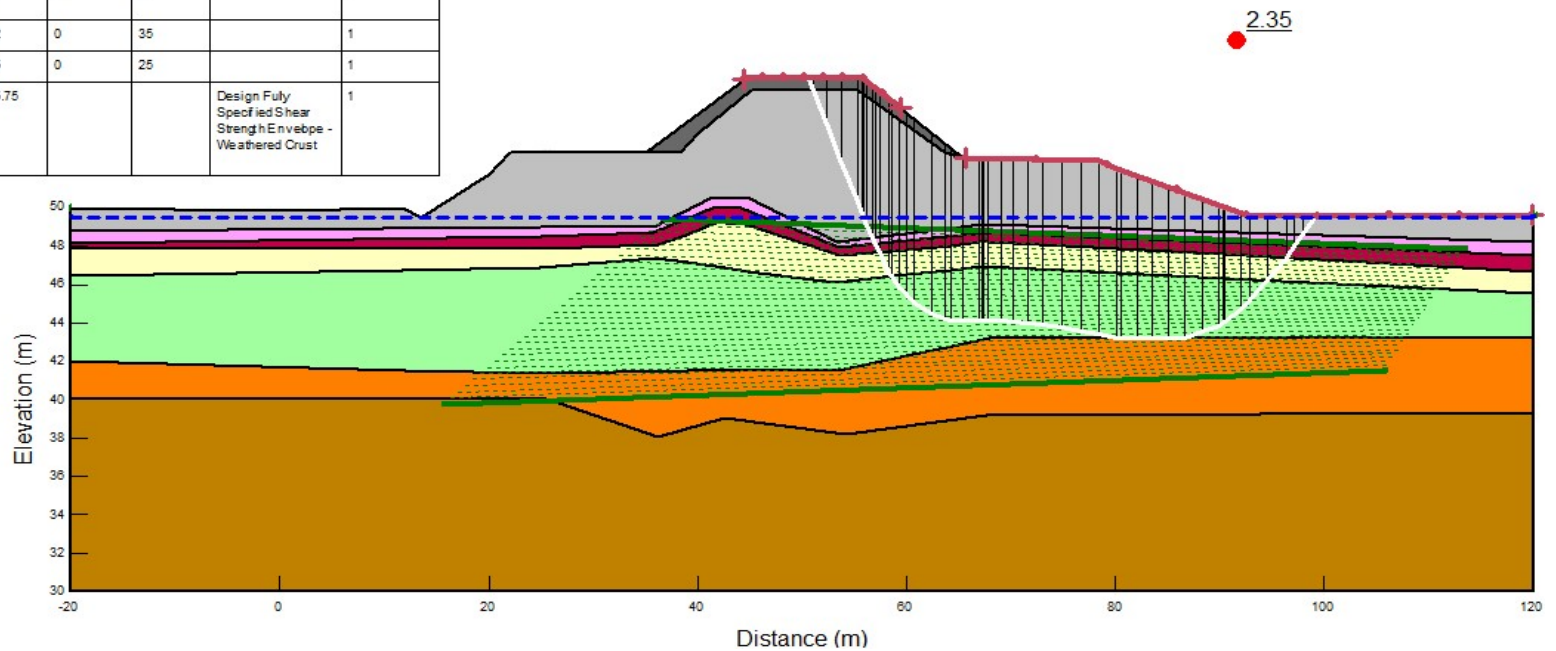
Name: 2.75H:1V East West Side Slope with New Fill - Drained Analysis-Peak

Analysis Type: Morgenstern-Price

Horz Seismic Coef.:

Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	Effective Cohesion (kPa)	Effective Friction Angle (°)	Strength Function	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)					1
	Clay	ShearNormal Fn.	15.25			Design Fully Specified Shear Strength Envelope - Clay	1
	Existing Fill	Mohr-Coulomb	21.5	0	35		1
	New Fill	Mohr-Coulomb	21.5	0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19	0	32		1
	Till	Mohr-Coulomb	22	0	35		1
	Topsoil	Mohr-Coulomb	15	0	25		1
	Weathered Crust	ShearNormal Fn.	15.75			Design Fully Specified Shear Strength Envelope - Weathered Crust	1



Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1

2.75H:1V East west Side Slope with New Fill- Drained Slope Stability Analysis - Peak Parameters

Project No: 22513877A  
Drawn: KG  
Date: June 7, 2023  
Checked: KCP  
Review: JPD









FIGURE G24

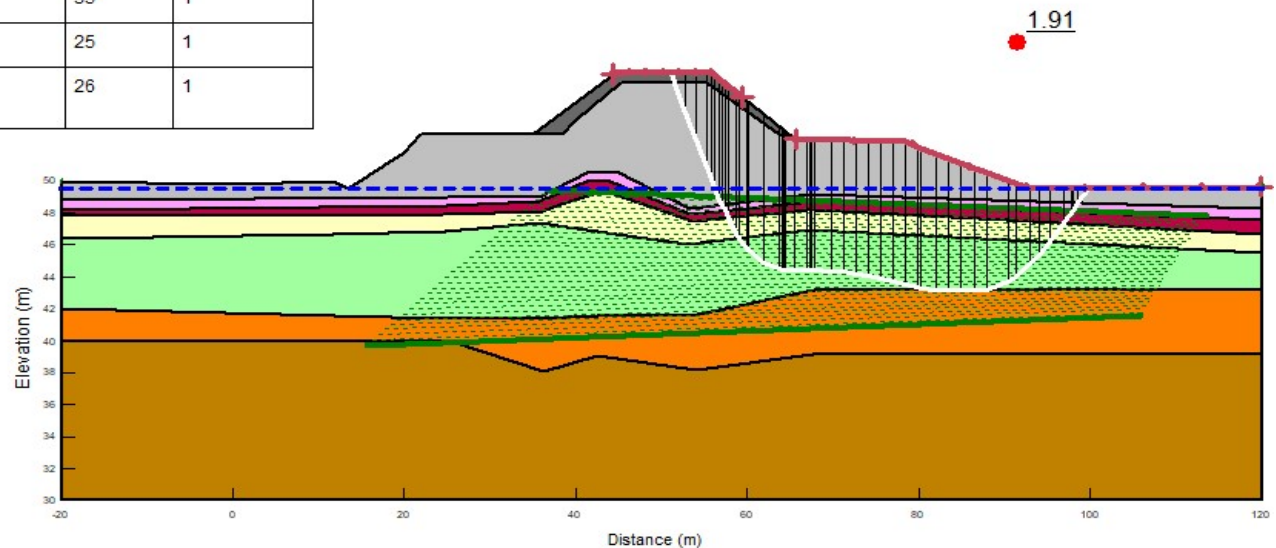
Name: 2.75H:1V East West Side Slope with New Fill - Drained Analysis-Post Peak/ Soften

Analysis Type: Morgenstern-Price

Horz Seismic Coef.:

Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)				1
	Clay	Mohr-Coulomb	15.25	0	20	1
	Existing Fill	Mohr-Coulomb	21.5	0	35	1
	New Fill	Mohr-Coulomb	21.5	0	35	1
	Silt to Sandy Silt	Mohr-Coulomb	19	0	32	1
	Till	Mohr-Coulomb	22	0	35	1
	Topsoil	Mohr-Coulomb	15	0	25	1
	Weathered Crust	Mohr-Coulomb	15.75	0	26	1



Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1

2.75H:1V East west Side Slope with New Fill- Drained Slope Stability Analysis - Post Peak/ Soften Parameters

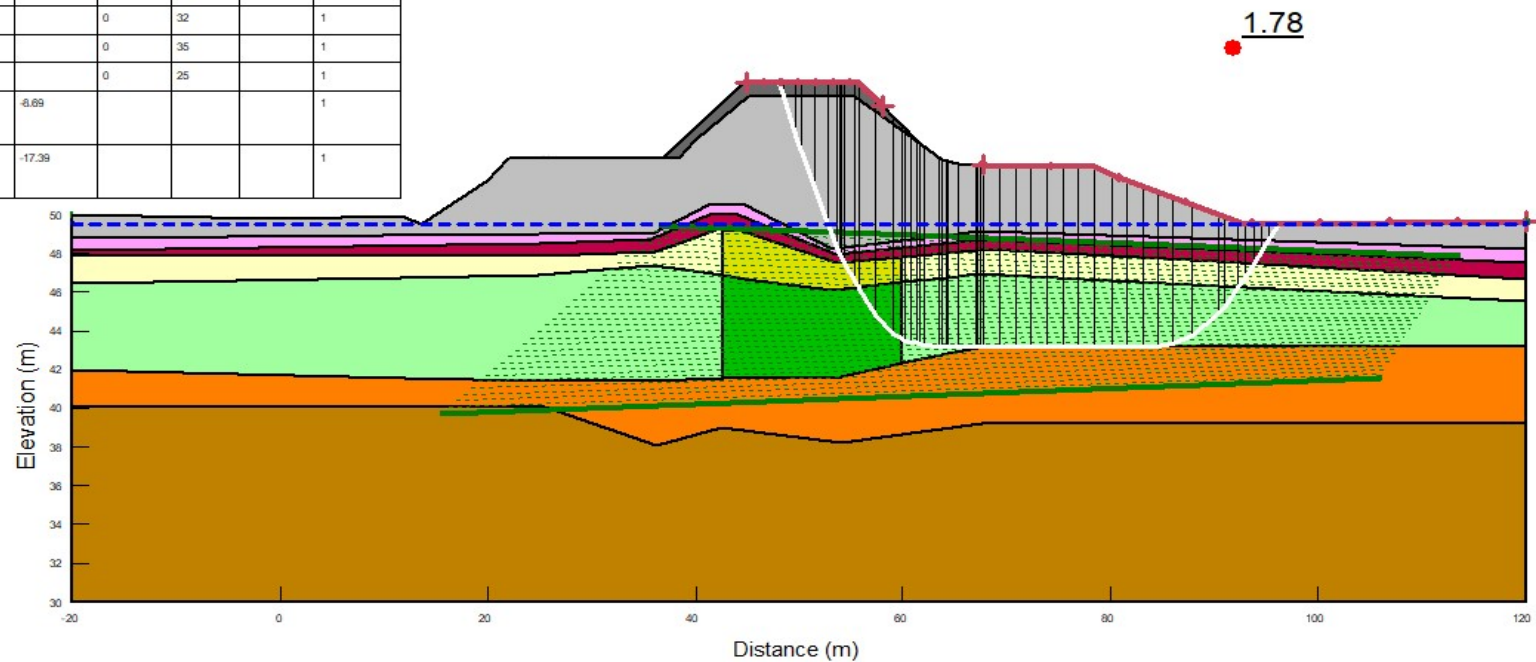
Project No: 22513877A  
Drawn: KG  
Date: June 7, 2023  
Checked: KCP  
Review: JPD

FIGURE G25



Name: 2H:1V East West Side Slope with New Fill - Undrained Analysis - Static  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.:  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m³)/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)							1
	Clay - Below and outside of Berm	Undrained (Phi=0)	15.25					30	1
	Clay - Below Embankment	Undrained (Phi=0)	15.25					40	1
	Existing Fill	Mohr-Coulomb	21.5			0	35		1
	New Fill	Mohr-Coulomb	21.5			0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Clay - Below and outside of Berm	S=(depth)	15.75	40	-6.69				1
	Weathered Clay - Below Embankment	S=(depth)	15.75	60	-17.39				1



Site 31-230 - Highway 401 Underpass at Fraser Road,  
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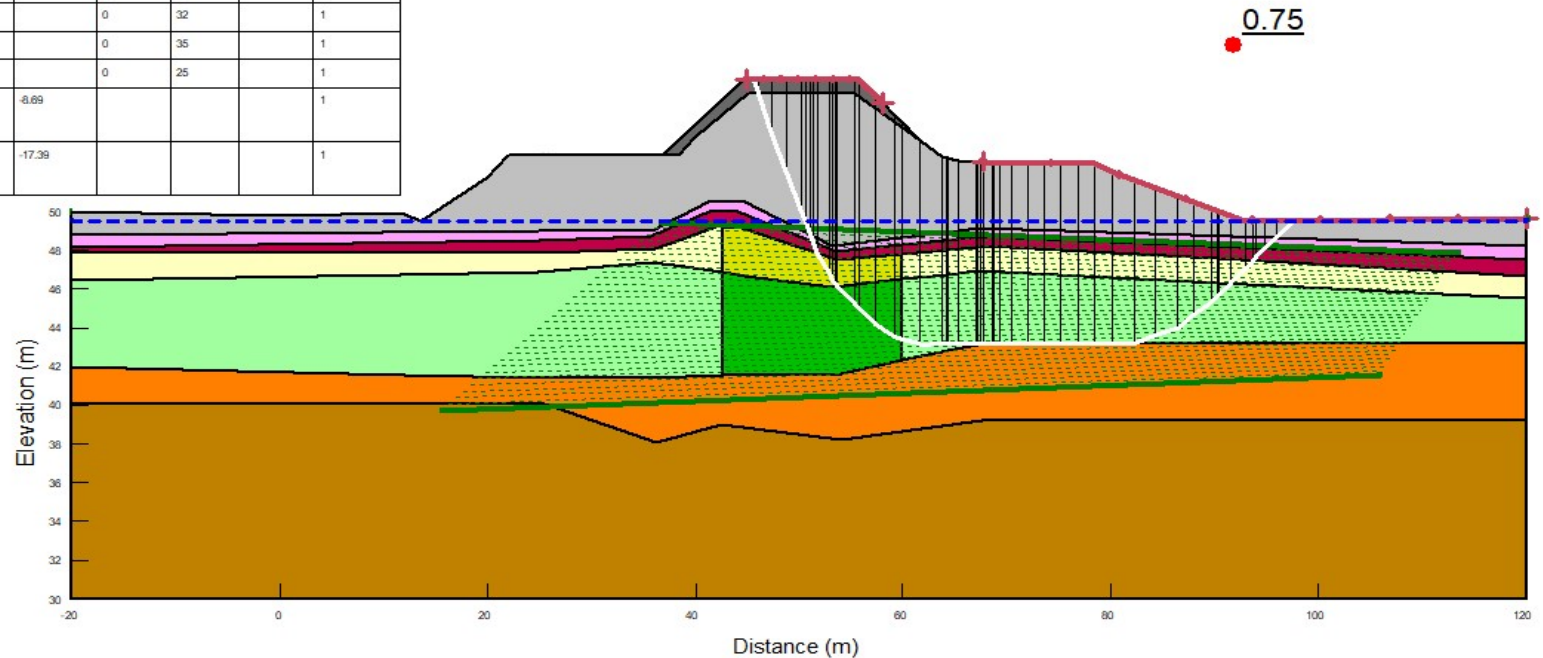
2H:1V East west Side Slope with New Fill- Undrained Slope Stability Analysis - Static

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G26

Name: 2H:1V East West Side Slope with New Fill - Undrained Analysis - Pseudo Static  
 Analysis Type: Morgenstem-Price  
 Horz Seismic Coef.: 0.23  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m³)/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Total Cohesion (kPa)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)							1
	Clay - Below and outside of Berm	Undrained (Phi=0)	15.25					30	1
	Clay - Below Embankment	Undrained (Phi=0)	15.25					40	1
	Existing Fill	Mohr-Coulomb	21.5			0	35		1
	New Fill	Mohr-Coulomb	21.5			0	35		1
	Silt to Sandy Sil	Mohr-Coulomb	19			0	32		1
	Till	Mohr-Coulomb	22			0	35		1
	Topsoil	Mohr-Coulomb	15			0	25		1
	Weathered Crust Clay - Below and outside of Berm	Sr(depth)	15.75	40	-6.69				1
	Weathered Crust Clay - Below Embankment	Sr(depth)	15.75	60	-17.39				1



Site 31-230 - Highway 401 Underpass at Fraser Road,  
 United Counties of Stormont, Dundas and Glengarry, Ontario,  
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2H:1V East west Side Slope with New Fill- Undrained Slope Stability Analysis - Pseudo Static

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G27

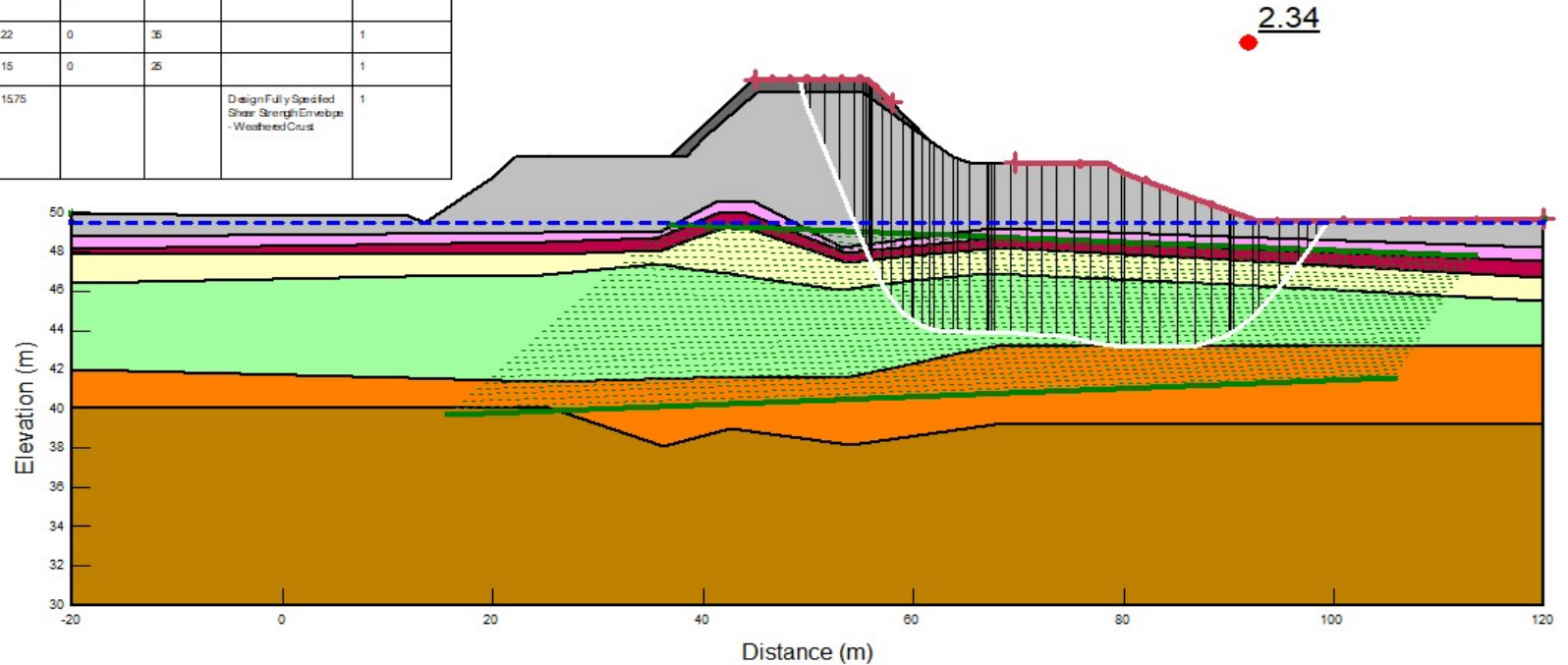
Name: 2H:1V East West Side Slope with New Fill - Undrained Analysis - Drained - Peak

Analysis Type: Morgenstern-Price

Horz Seismic Coef.:

Groundwater Elevation Y: 49.5 m

Cobr	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Strength Function	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)					1
	Clay	Shear Normal Fn.	1525			Design Fully Specified Shear Strength Envelope - Clay	1
	Existing Fill	Mohr-Coulomb	21.5	0	35		1
	New Fill	Mohr-Coulomb	21.5	0	35		1
	Silt to Sandy Silt	Mohr-Coulomb	19	0	32		1
	Till	Mohr-Coulomb	22	0	35		1
	Topsoil	Mohr-Coulomb	15	0	25		1
	Weathered Crust	Shear Normal Fn.	1575			Design Fully Specified Shear Strength Envelope - Weathered Crust	1











Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1

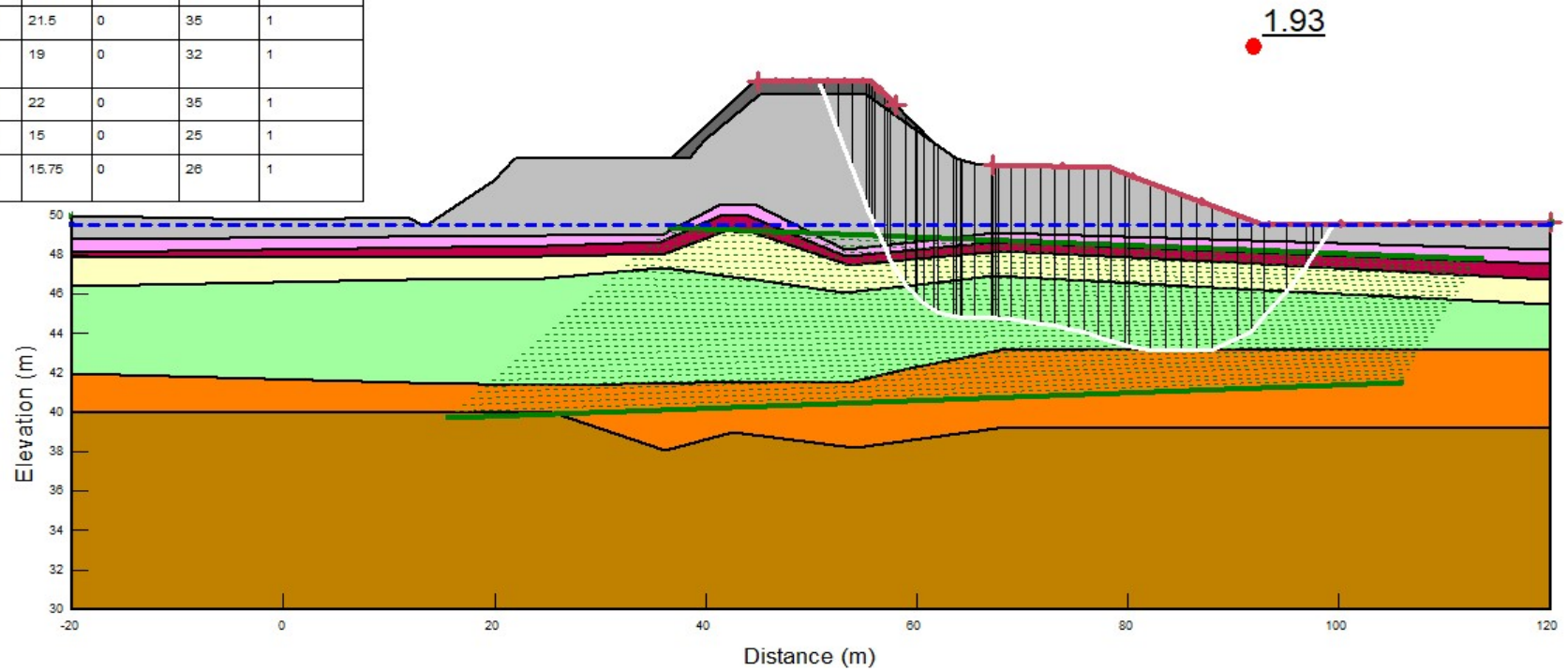
2H:1V East west Side Slope with New Fill- Drained Slope Stability Analysis - Peak Parameters

Project No: 22513877A  
Drawn: KG  
Date: June 7, 2023  
Checked: KCP  
Review: JPD

FIGURE G28

Name: 2H:1V East West Side Slope with New Fill - Drained Analysis - Post Peak/ Soften  
 Analysis Type: Morgenstern-Price  
 Horz Seismic Coef.:  
 Groundwater Elevation Y: 49.5 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m <sup>3</sup> )	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Bedrock	Bedrock (Impenetrable)				1
	Clay	Mohr-Coulomb	15.25	0	20	1
	Existing Fill	Mohr-Coulomb	21.5	0	35	1
	New Fill	Mohr-Coulomb	21.5	0	35	1
	Silt to Sandy Silt	Mohr-Coulomb	19	0	32	1
	Till	Mohr-Coulomb	22	0	35	1
	Topsoil	Mohr-Coulomb	15	0	25	1
	Weathered Crust	Mohr-Coulomb	15.75	0	28	1



Site 31-230 - Highway 401 Underpass at Fraser Road,  
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2H:1V East west Side Slope with New Fill- Drained Slope Stability Analysis - Post Peak/ Soften Parameters

Project No: 22513877A  
 Drawn: KG  
 Date: June 7, 2023  
 Checked: KCP  
 Review: JPD

FIGURE G29

**APPENDIX H**

# Settlement Figures

## Introduction

Figure 1 shows the proposed grade raise and changes in geometry of the existing approach embankment located at Fraser Road, Ottawa. Quantification of the impact of the construction of the new approach embankment was carried out through settlement analysis to determine the short term and long-term settlements of approach fills. These were compared with typical MTO allowable limits of settlement for local road to evaluate whether or not remedial measures would be required.

Based on the available information it is understood that the original bridge was constructed in 1968. The existing embankment currently has about 4.0 m high berms in the front and both sides and the berms are approximately 16 to 18 m long. The maximum height of the approach embankment is approximately 7.6 m (El. ~56.3 m). Settlement of the approach embankment in the order of 0.6 m were predicted during the original investigation period. Settlement readings on the approach embankment pavement was carried out for few years after construction and up to about 0.3 m of settlement was measured at that time. Repair of the pavement structure at the approaches and restoration of the pavement structures was completed in 1971 due to consolidation of underlying sensitive clay. No further additional settlement records are available since 1971.

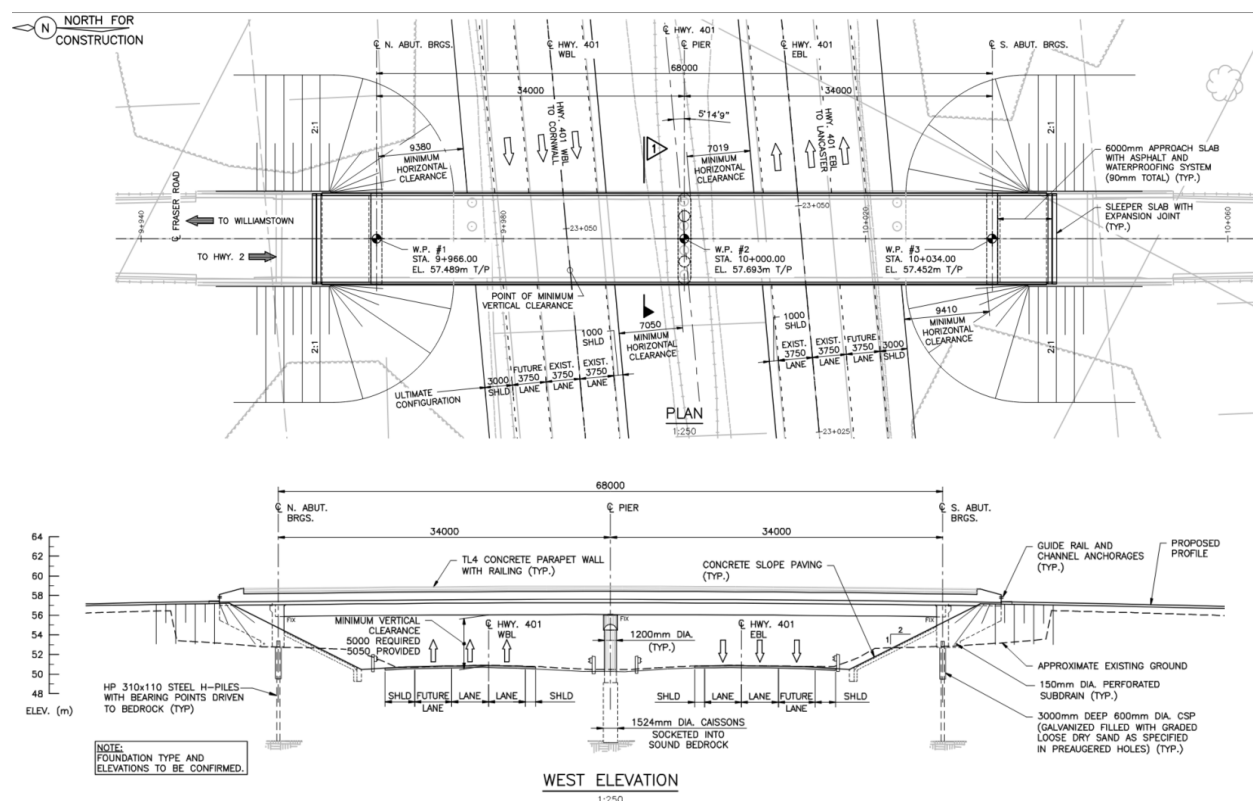


Figure 1: Plan view and cross section showing the existing approach embankment location and proposed changes in geometry for future embankment (Ref: Dillon's General Arrangement Drawing)



The new bridge geometry and construction of approach embankment propose an approximately 4.8 to 5.3 m maximum raise in grade in front of the existing approach embankment and minimum of about 0.7 m on top of the entire existing approach embankment. The following sections provide description of the analysis methodology considered to estimate the settlement, subsurface conditions considered for the analysis and calibration, analysis results and recommendations to minimize settlement, where deemed required.

## Analysis Methodology

The analysis was carried out using Settle3 software. Only north/northwest side of the embankment was considered for analysis. Based on WSP's 2023 base plan, borehole locations and cross section drawings (see Figure 2), the footprint of the embankment including toe berm was approximately 100 m long with an irregular shape. The width of the toe berm varied from north to south with approximately 72 m maximum width at location where the height of the approach fills was maximum. The berm side slopes were approximately 16° for east side and 23° for west side, respectively. To capture the irregular shape of the existing approach fills and toe berm of the north embankment (adjacent to HWY 401 WBL), the model geometry considered followings:

- The footprint of the entire approach embankment including toe berm (100 m in length) was divided into 21 slices/segments which width varied between 1.5 m and 6 m to capture the irregular shape.
- Each segment was modeled as individual embankment with berm side slopes using Embankment Load option/tool available in Settle3. Each embankment was 3.7 m high. This simulated the toe berm with side slopes as described above.
- To simulate the approach fills on top of the toe berm, 10.5 m wide second embankment was modeled on top of toe berm. Similar to toe berm construction in the model, 14 slices/segments were considered to capture the footprint of 78 m long approach embankment constructed on top of the toe berm. A side slope of 2H:1V was considered for this second embankment. The height of these 14 slices (second embankments) was gradually decreased from 4.1 m to 0.7 m to capture the road grade.

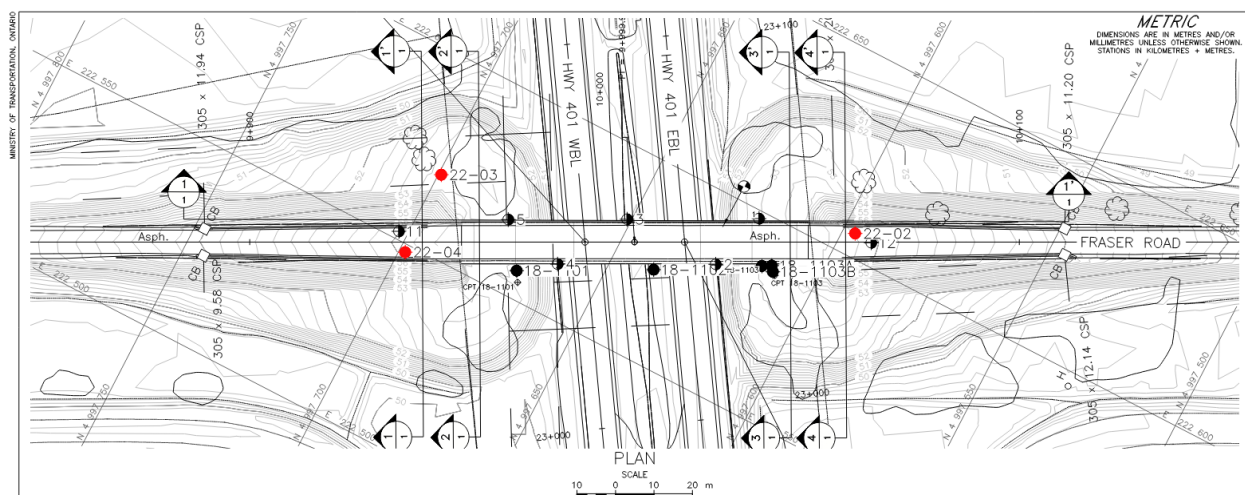


Figure 2: Plan view showing geometry of existing embankment with borehole locations (Ref: WSP's Drawing No. 1, GWP No. 4248-15-00)

- Model calibration was completed to compare the maximum settlements measured on site for the original embankment between the year of 1968 and 1971.
- Additional two segments of embankment load were added in the model in front of the existing approach fills and on top of previously constructed toe berm to simulate the footprint of 11.0 m long and 10.5 m wide new approach embankment with side slopes of 2H:1V (extension of existing embankment).

Geotechnical design parameters considered in settlement analysis are provided in Figures 3 and 4 below. The design lines (pink color) considered in this study were based on the subsurface information and lab test results obtained from recent and historical boreholes, information available for similar soils from published literature and WSP's past experiences. Note that parameters were also calibrated to match with the settlements measured on site during and after the construction of original approach embankment.

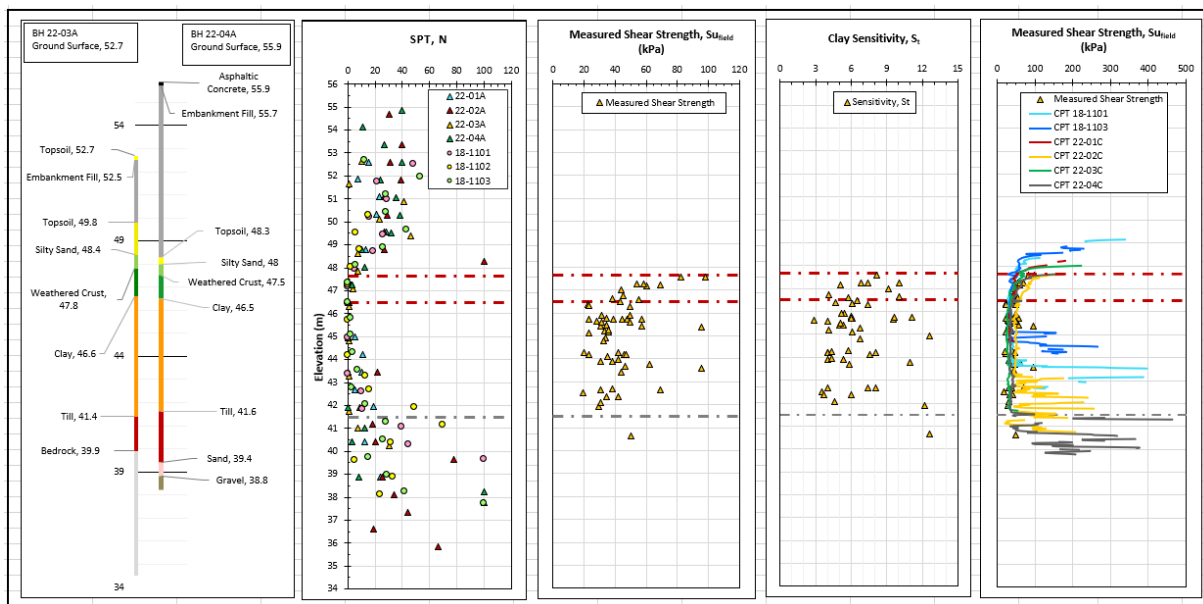


Figure 3: Soil stratigraphy and results obtained from field investigation program

Accordingly, soil stratigraphy below the existing approach embankment was considered in our model to consist of the following soil/rock layers in descending order:

- Compact granular fill with an average thickness of about 0.8 m;
- Stiff weathered crust with an average thickness of about 1.6 m;
- Firm sensitive clay with an average thickness of about 1.5 m of top layer and 5.2 m of bottom layer;
- Compact till (silty sand) with an average thickness of about 3.1 m; and
- Bedrock.



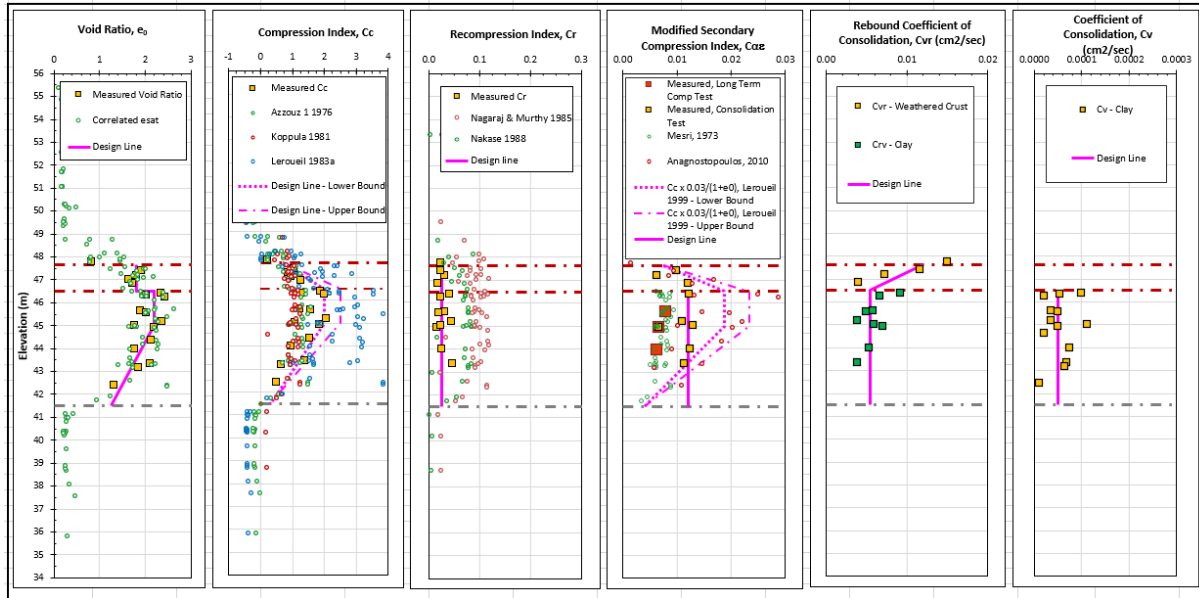


Figure 4: Summary of clay consolidation parameters and design line considered for analysis  
Geotechnical design parameters considered in settlement analysis are provided in Table 1.

Table 1: Geotechnical design parameters considered in the settlement analysis

Soil Layer Type	Unit Weight, $\text{kN/m}^3$	Modulus of Elasticity, MPa	Consolidation Parameters
Granular Fill	20	20	NA
Weathered Crust	17	N/A	$c_c = 0.7$ to $2$ ; $c_r = 0.018$ ; $e_0 = 1.8$ $P'_c = 250$ to $135$ kPa; $c_v = 0.00875$ $\text{cm}^2/\text{s}$ ; $c_{vr} = 0.012$ to $0.0055$ $\text{cm}^2/\text{s}$ ;
Sensitive Clay Top 1.5 m Layer	15.5	NA	$c_{\alpha} = 0.0384$ ; $c_c = 2$ ; $c_r = 0.025$ ; $e_0 = 2.2$ $P'_c = 130$ to $120$ kPa; $c_{\alpha r} = 0.00384$ ; $c_v = 5 \times 10^{-05}$ $\text{cm}^2/\text{s}$ ; $c_{vr} = 0.0055$ $\text{cm}^2/\text{s}$ ;
Sensitive Clay Bottom 5.2 m Layer	15.5	NA	$c_c = 2$ to $0.323$ ; $c_r = 0.025$ ; $e_0 = 2.2$ to $1.25$ $P'_c = 120$ to $75$ kPa; $c_{\alpha} = 0.0384$ ; $c_{\alpha r} = 0.00384$ ; $c_v = 5 \times 10^{-05}$ $\text{cm}^2/\text{s}$ ; $c_{vr} = 5 \times 10^{-05}$ $\text{cm}^2/\text{s}$ ;
Till	22	60	NA

## Design Assumptions

The settlement analysis carried out in this study was based on the following assumed construction sequences:

- Stage 1: Original approach embankment was constructed on soil stratigraphy described above at ground elevation 48.9 m;
- Stage 2: Construct initial toe berm of 3.7 m height within 2 years;
- Stage 3: Construct additional approach fills (height gradually decreases from 4.1 m to 0.7 m) on top of toe berm in year 3 (see model screenshot in Figure 5);
- Stage 4: Calculate settlement after 5 and 50 years of construction to simulate the measured settlement after construction;
- Stage 5: Place new fill (maximum ~ 4.3 m grade raise) in front of the existing embankment as a part of extension of existing approach embankment in 3 months;
- Stage 6: Additional 0.65 m grade raise on top of existing embankment and maximum 1.0 m of additional grade raise on top of newly built approach fills to construct the road structure (maximum El. 57.5 m) after 3 months;
- Stage 7: Estimate settlement after 2.5 and 25 years of construction of new approach embankment.

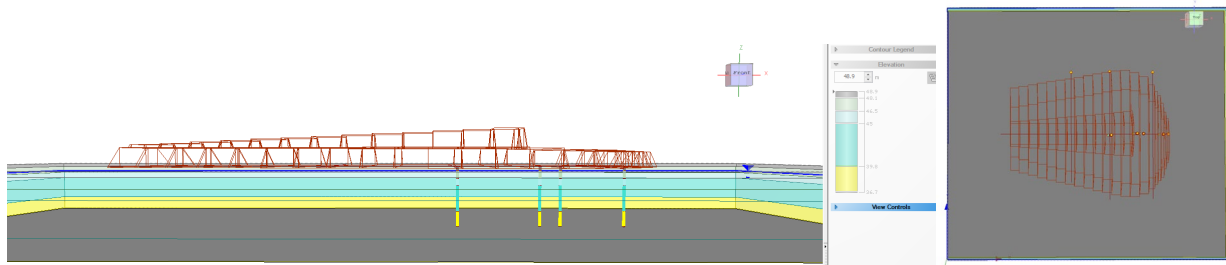


Figure 5: Model geometry considered for settlement analysis in Settle3

Other assumptions considered in the settlement analysis can be summarized as follows:

- The initial grade elevation was at 48.9 m before the construction of approach embankment in 1968;
- Ground water table located at a depth of approximately 1.9 m below grade (El. 47.0 m), based on available information from historic report;
- Dimension of the footprint of north side embankment was about 102 m long, width varied from 82 m to 30 m and height varied from 7.8 m to 4.4 m;
- Although there was variability in thickness of sensitive clay layer encountered in different boreholes, the model considered a maximum 6.7 m thick clay layer under the footprint of embankment as worst-case scenario.
- For extension of existing approach embankment (new approach fills as grade raise), settlement analysis was completed considering two scenarios: i) grade raise using granular fill and ii) grade raise using lightweight fill such as EPS Geofoam. A unit weight of 0.5 kN/m<sup>3</sup> was assumed for EPS geofoam blocks.

## Results

Table 2 provides the maximum consolidation and total settlement obtained from the Settle3 model due to construction of the existing embankment. The results presented in Table 2 were compared to the reported settlements measured on site between the year of 1968 and 1971. Figure 6 provides the total settlement contour plot for Stage 4 (present condition).

Table 2: Calculated settlement during and after construction of existing embankment

Loading Stage	Total Consolidation Settlement, mm	Total Settlement, mm
Stage 1	0	0
Stage 2	0	5 - 7
Stage 3	41 - 44	50 - 52
Stage 4a (after 5 years of construction)	290 - 300	300 - 310
Stage 4b (after 50 years of construction)	550 - 560	580 - 590

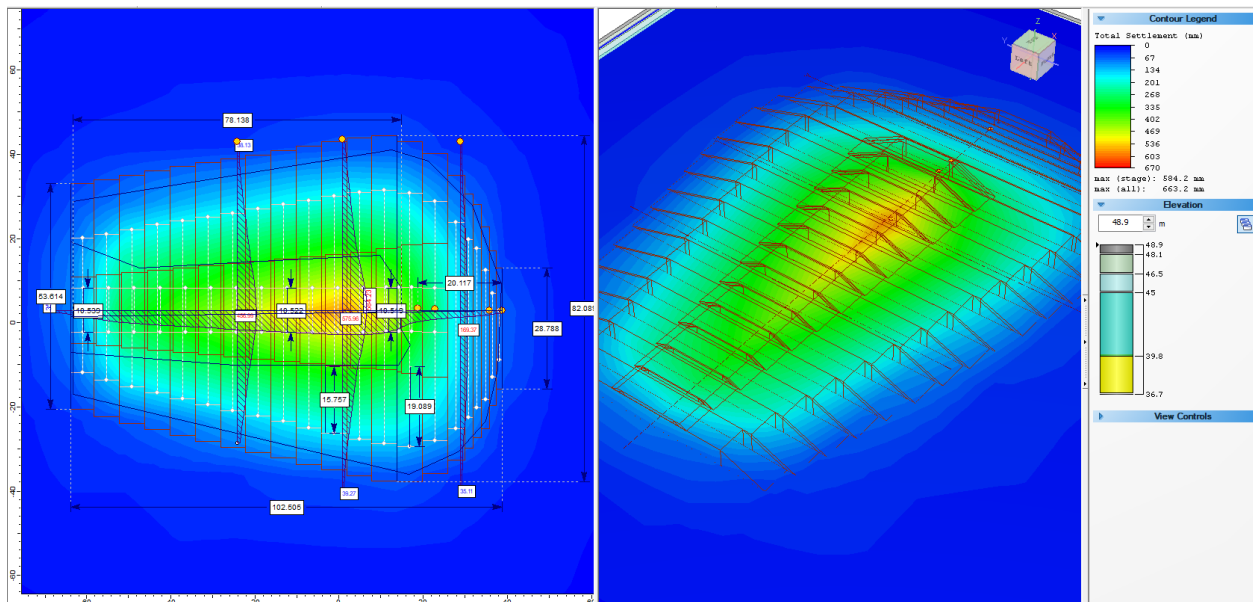


Figure 6: Total settlement contour plot after 50 years of construction of existing embankment

Table 3 below provides the maximum short term and long-term total and differential settlement anticipated from the Settle3 model due to construction of proposed extension of the existing embankment (grade raise due to new approach fills and road structure) as described in previous section. Differential settlement was calculated comparing settlements between two points: maximum height of existing embankment (point B)

and maximum height of new approach fills (Point A) as shown in Figure 7. The results are also presented considering both granular fill and EPS Geofoam as materials for proposed grade raise. Figure 7 provides the total settlement contour plot for Stage 7.

Table 3: Calculated settlement for construction of new embankment

Loading Stage	Total Maximum Settlement, mm		Maximum Differential Settlement, mm	
	Regular Granular Fill	EPS Geofoam	Regular Granular Fill	EPS Geofoam
Stage 5	3 - 4	0.5 - 1	1 – 1.5	0.5 - 1
Stage 6	50 - 55	2.5 - 3	12 - 15	0.5 - 1
Stage 7a (after 2.5 years of construction)	200 - 205	33 - 36	50 - 65	16 - 25
Stage 7b (after 25 years of construction)	360 - 370	80 - 85	90 - 120	25 - 35

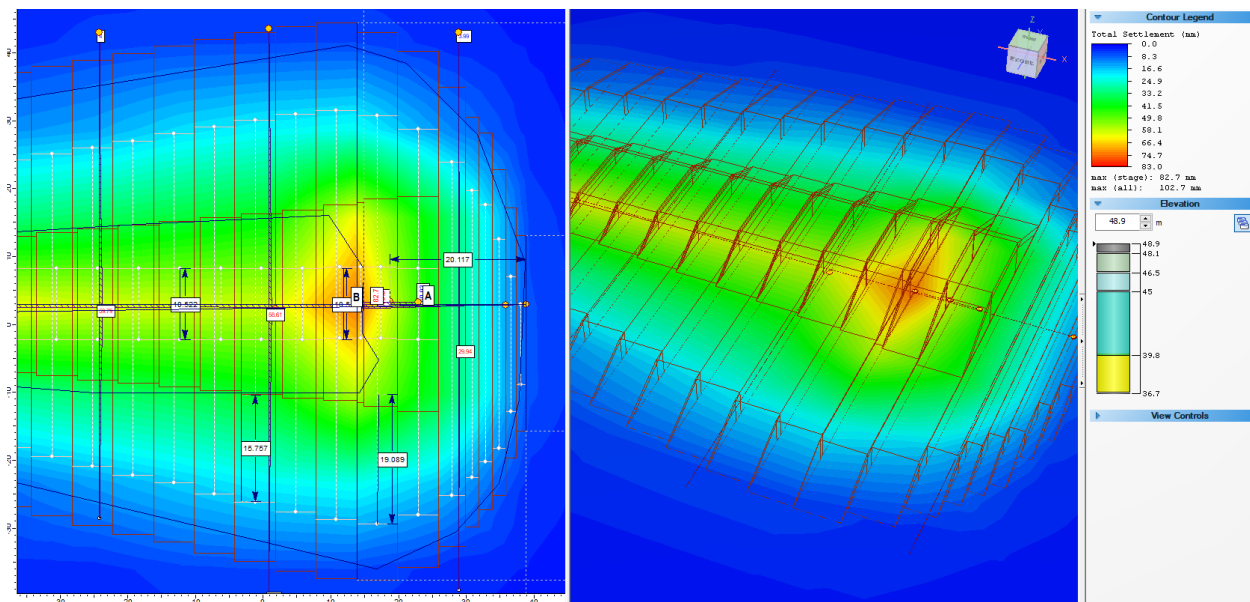


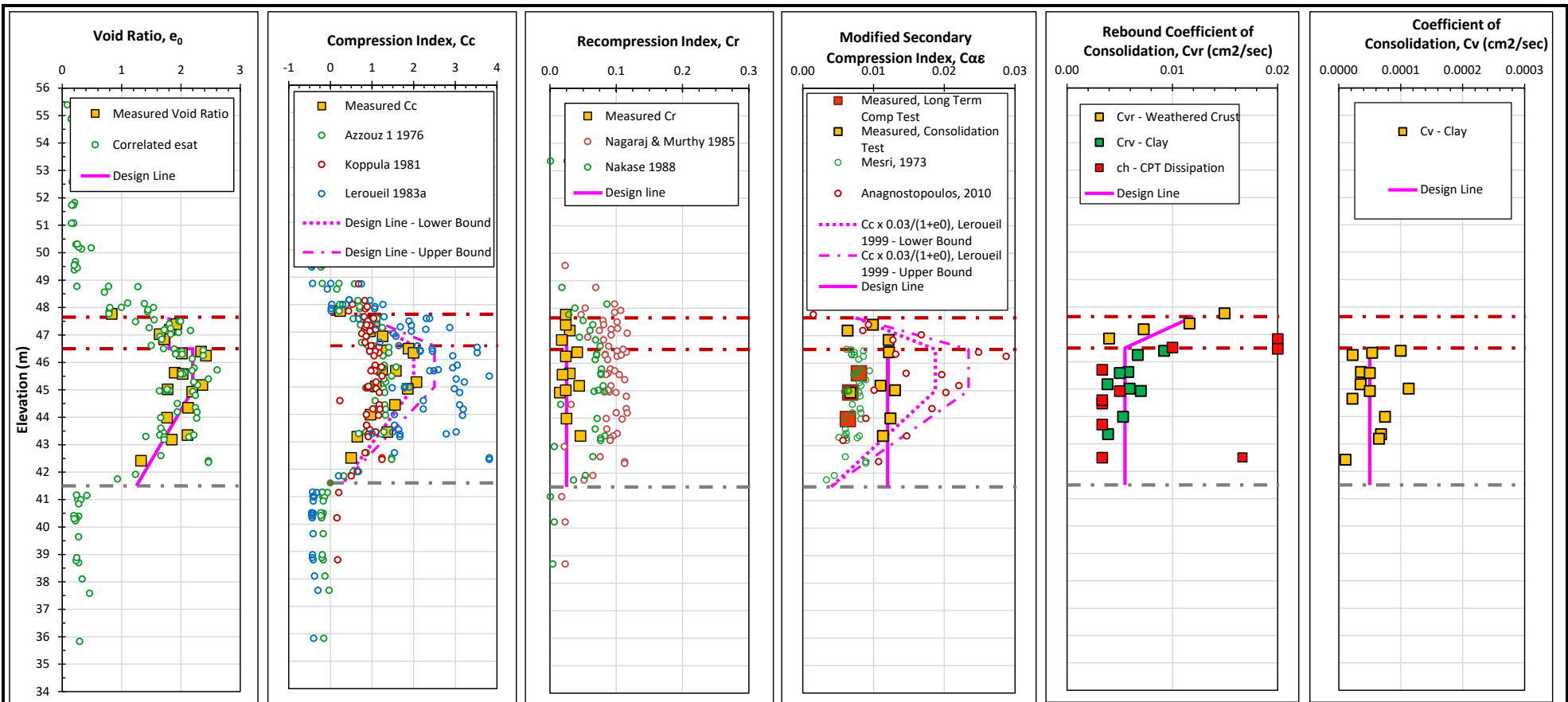
Figure 7: Settlement contour plot after 25 years of construction of proposed new embankment

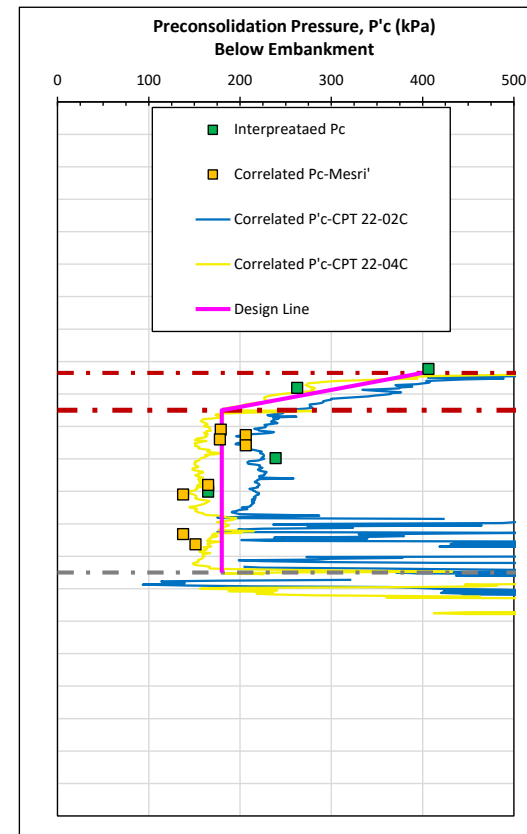
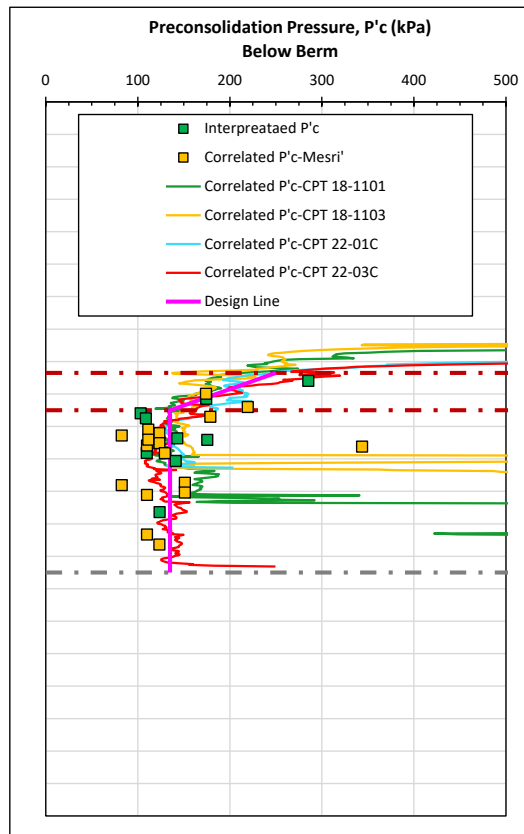
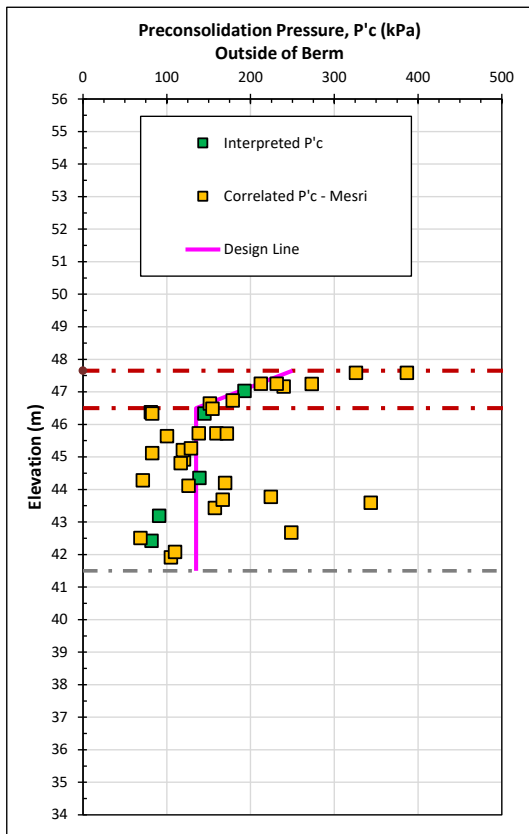
## Comments and Recommendations

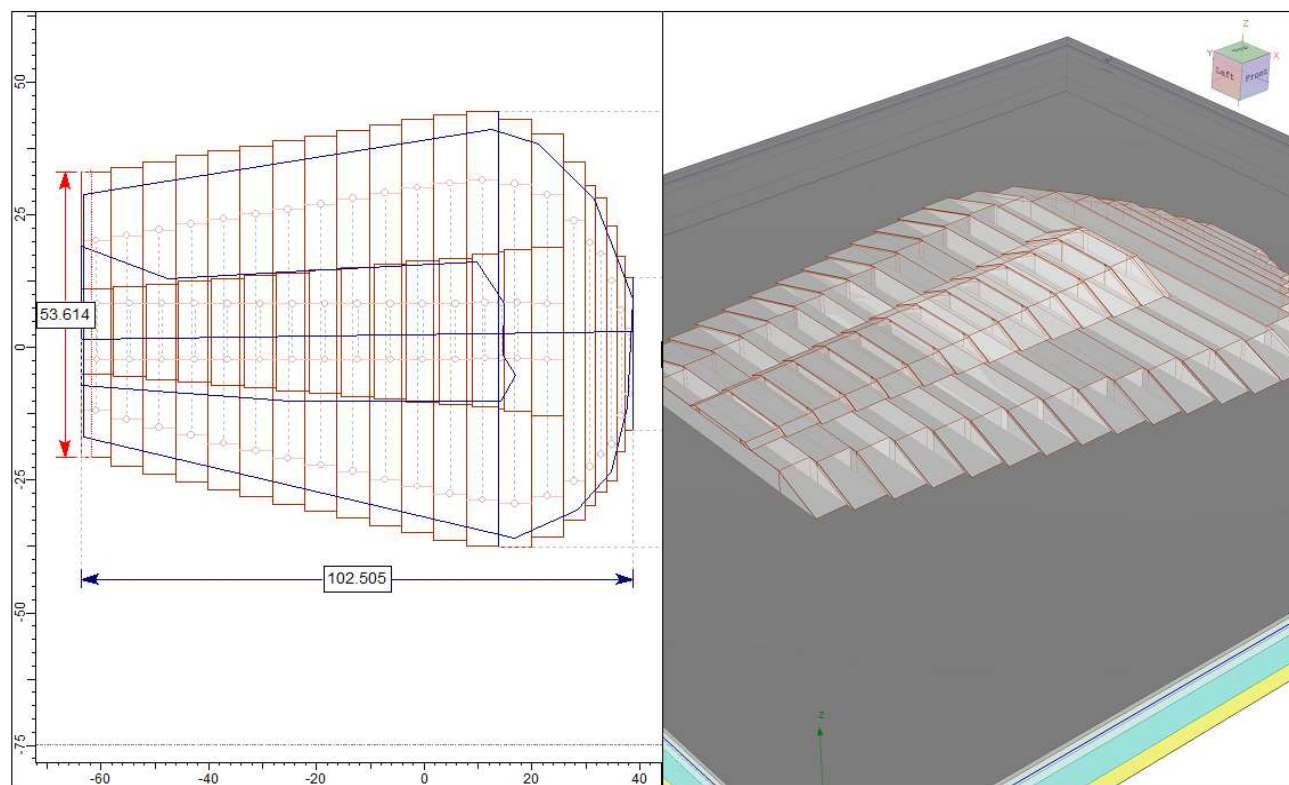
Based on the results obtained from the settlement analysis, following comments and recommendations are made:

- EPS Geofoam materials shall be used for proposed grade raise in front of existing approach embankment.

- Grade raise (including new pavement structure) on top of existing embankment shall be limited to 0.65 m only.
- Thickness of pavement structure on top of EPS Geofoam blocks shall be less than 1.0 m.
- The maximum settlements due to grade raise will occur at location (point B as shown in Figure 7) where the height of the existing embankment is highest.
- A short-term and long-term differential settlements of 25 mm and 15 mm respectively, can be expected between point A and B (transition between existing and new embankment)
- A short-term and long-term total settlement of 35 mm and less than 50 mm respectively, can be expected due to proposed new embankment construction.
- Although these settlements are within MTO's tolerable limit of local roadway, some maintenance in the form of asphalt resurfacing may be required.





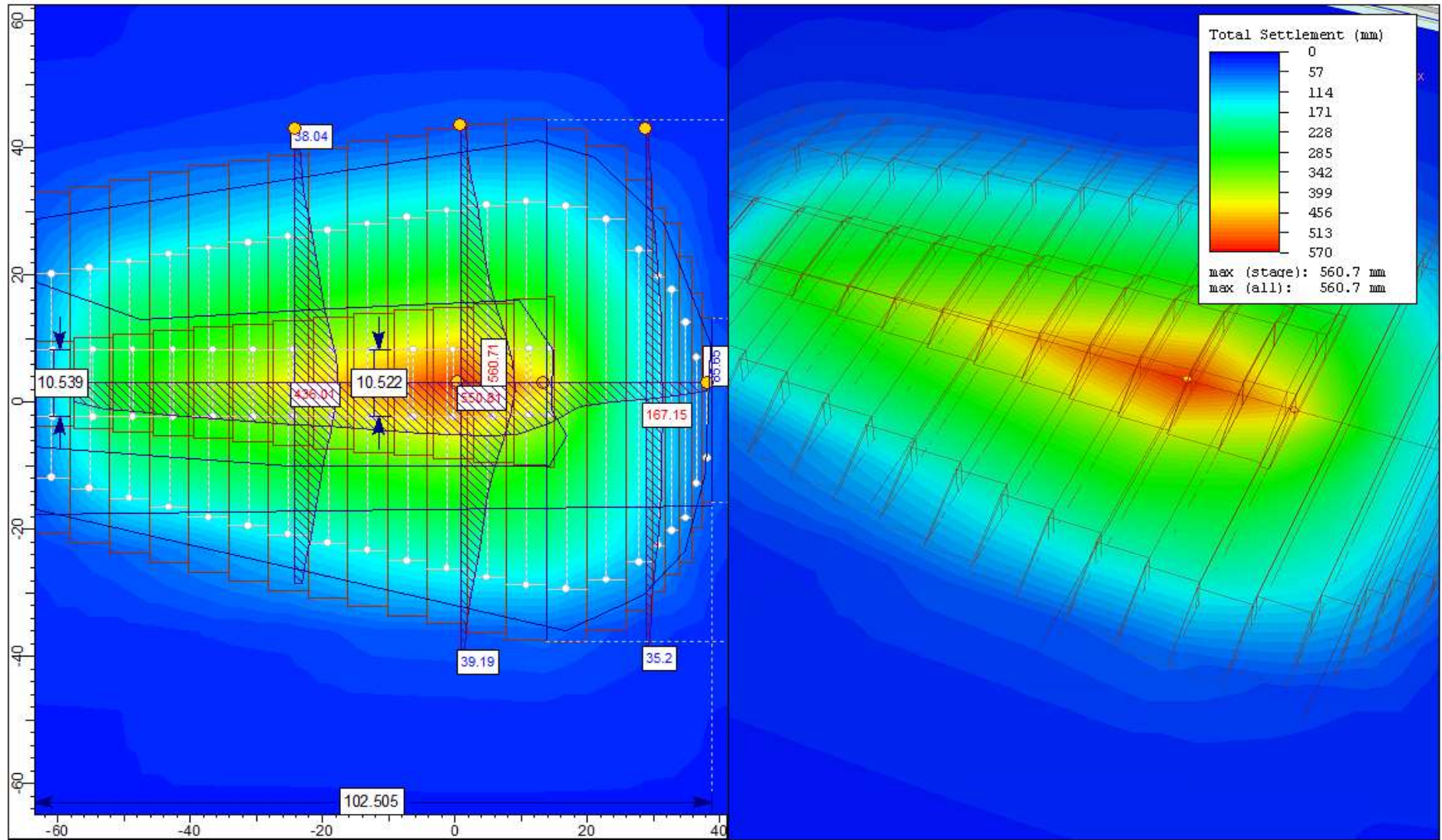


Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1  
Approach Embankment - Settle3 Model Geometry

Project No:	22513877A
Drawn:	RD/AKP
Date:	June 7, 2023
Checked:	KCP
Review:	JPD

FIGURE H3



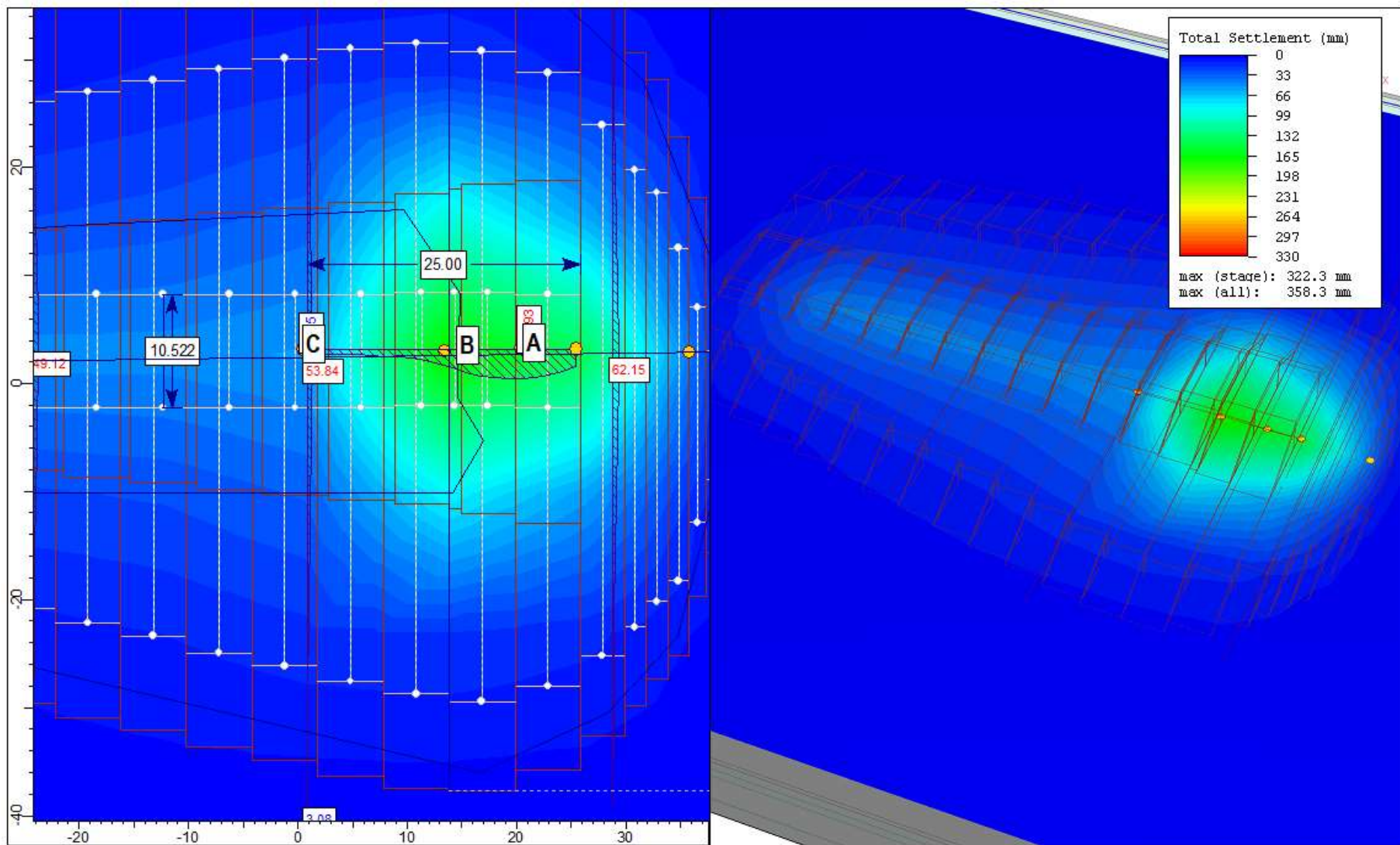


Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1

Existing Embankment - Total settlement contour plot 50 years after construction

Project No:	22513877A
Drawn:	RD/AKP
Date:	June 7, 2023
Checked:	KCP
Review:	JPD

FIGURE H4

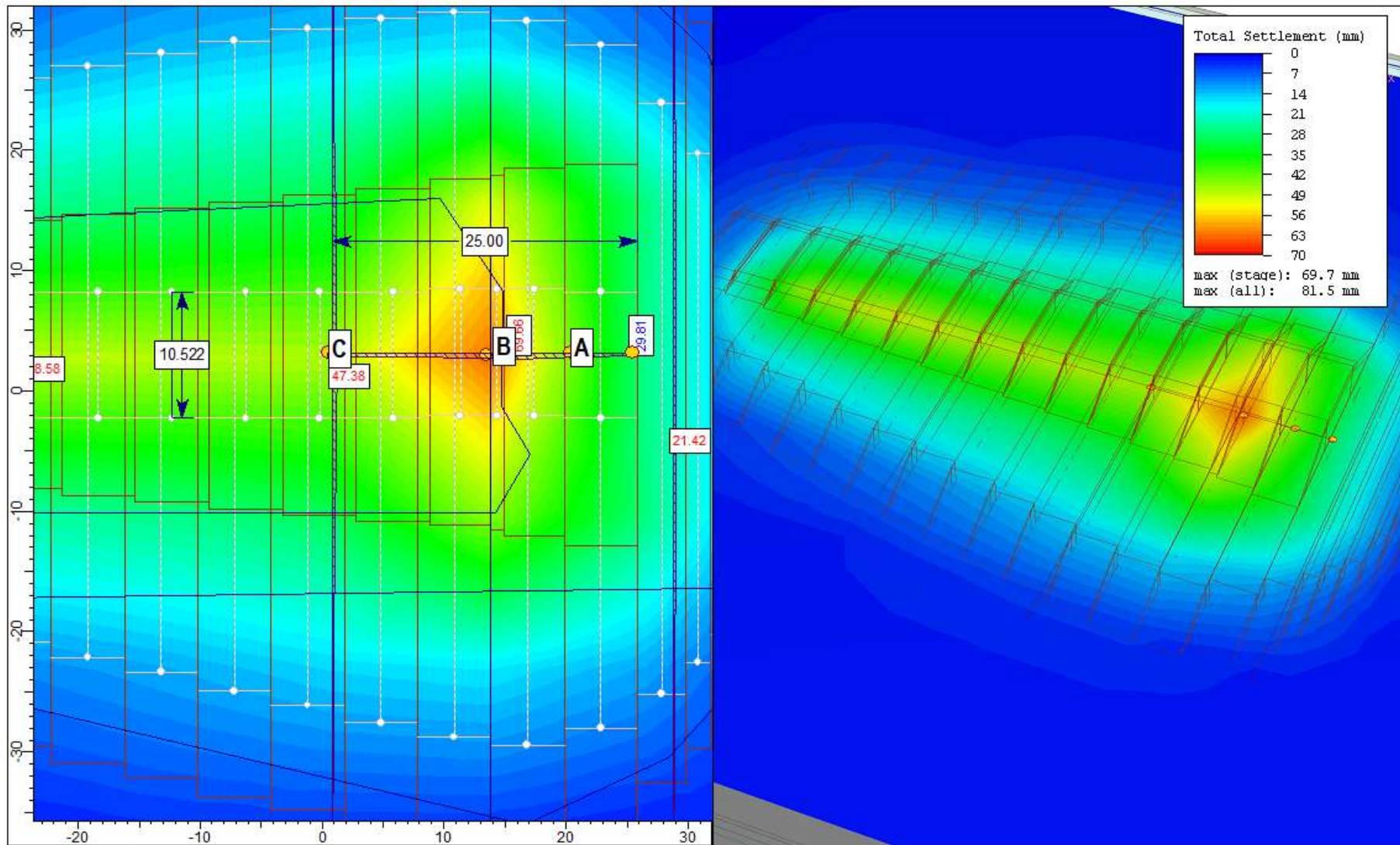


Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1  
Settlement Contour Plot 25 Years after Construction Case 1 - Granular Fill

Project No: 22513877A  
Drawn: RD/AKP  
Date: June 7, 2023  
Checked: KCP  
Review: JPD

FIGURE H5



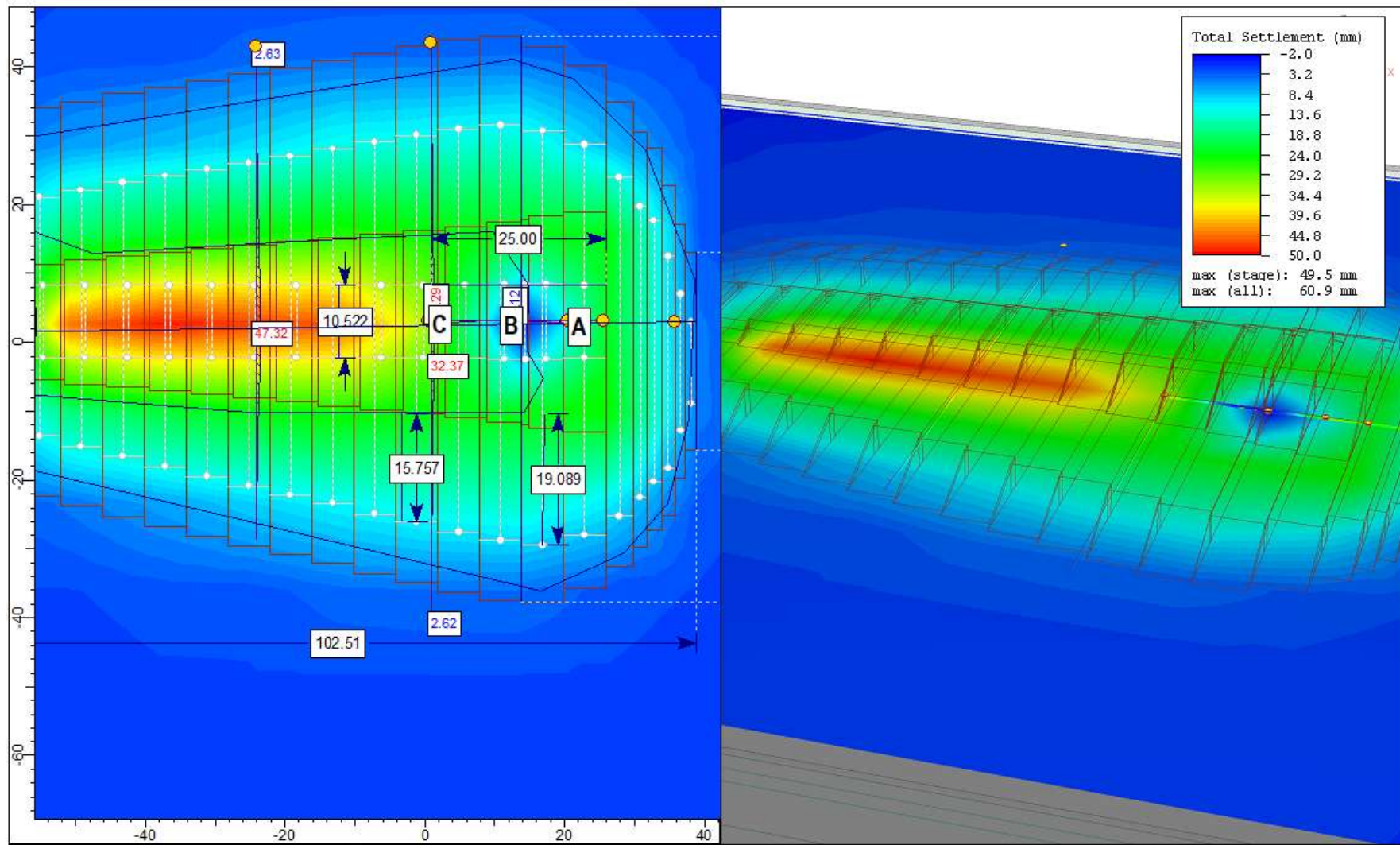


Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1

Settlement Contour Plot 25 Years after Construction Case 2 - EPS GEOFOAM 10 m Length

Project No:	22513877A
Drawn:	RD/AKP
Date:	June 7, 2023
Checked:	KCP
Review:	JPD

FIGURE H6



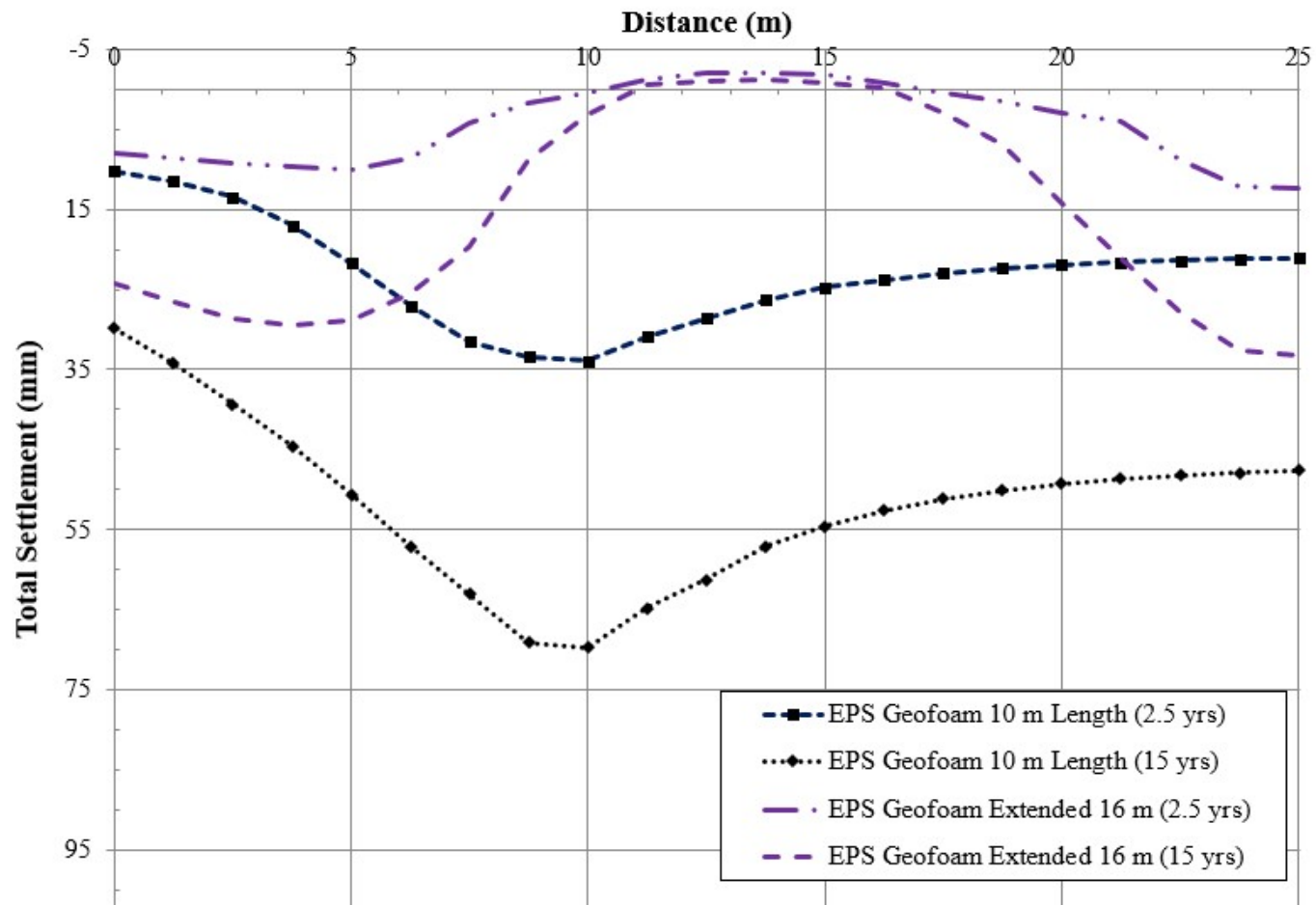
Site 31-230 - Highway 401 Underpass at Fraser Road,  
United Counties of Stormont, Dundas and Glengarry, Ontario,  
Agreement No 4021-E-0021; Assignment 1

Settlement Contour Plot 25 Years after Construction Case 3 - EPS GEOFOAM 16 m Length

Project No:	22513877A
Drawn:	RD/AKP
Date:	June 7, 2023
Checked:	KCP
Review:	JPD

FIGURE H7

# Distance from New Abutment vs. Total Settlement



Site 31-230 - Highway 401 Underpass at Fraser Road,  
 United Counties of Stormont, Dundas and Glengarry, Ontario,  
 Agreement No 4021-E-0021; Assignment 1  
 Settlement Comparision for different lengths of GEOFOAM EPS

Project No:	22513877A
Drawn:	RD /AKP
Date:	June 7, 2023
Checked:	KCP
Review:	JPD

FIGURE H8

**APPENDIX I**

# 2018 VSP Investigation Technical Golder Report



## TECHNICAL MEMORANDUM

**DATE** November 5, 2018

**Project No.** 1899802

**TO** Christine Ko, Golder Associates Ltd

**FROM** Stephane Sol, Christopher Phillips

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

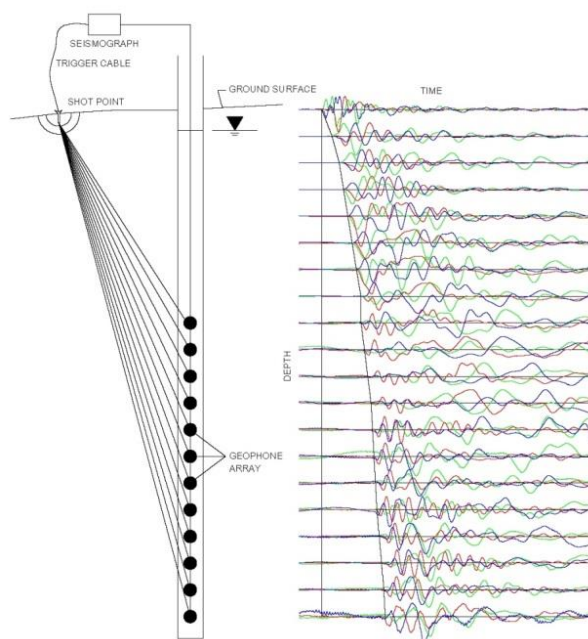
### **VERTICAL SEISMIC PROFILING TEST RESULTS HWY 401 FRASER ROAD UNDERPASS, LANCASTER, ONTARIO**

This memorandum presents the results of two Vertical Seismic Profiling (VSP) testing carried out at the Fraser Road underpass located along Highway 401 near Lancaster, Ontario. VSP testing was carried out on September 18, 2018. Borehole 18-1101, located north of the overpass, was drilled to an approximate depth of 20.8 m below the existing ground surface and then cased with a 2.5 inch PVC pipe grouted in place. The borehole consisted of approximately 5.8 m of silty sand, 3.9 m of silty clay, 3.8 m of silty sand and then limestone bedrock to the bottom of the borehole. Borehole 18-1103, located south of the overpass, was drilled to an approximate depth of 20.95 m below the existing ground surface and then cased with a 2.5 inch PVC pipe grouted in place. The borehole consisted of approximately 4.6 m of silty sand, 3.8 m of silty clay, 7 m of silty sand, and then limestone bedrock to the bottom of the borehole.

### **Methodology**

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

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



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

## Field Work

The field work was carried out on September 18, 2018, by personnel from the Golder Mississauga office.

At BH18-1101, the compression and shear-wave seismic sources were used, and they were located 2.11 m from the borehole. The seismic source for the compression wave test consisted of a 9.9 kilogram sledge hammer vertically impacted on a metal plate. The seismic source for the shear-wave test consisted of a 2.4-metre-long, 150 millimetre by 150 millimetre wooden beam, weighted by a vehicle and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced at 1-metre intervals below the ground surface to the maximum depth of the casing (18.3 m).

At BH18-1102, the compression and shear-wave seismic sources were used and they were located 2.03 m from the borehole. The seismic source for the shear-wave test consisted of a 2.4 metre long, 150 millimetre by 150 millimetre wooden beam, weighted by a vehicle and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. The shear source was coupled to the ground surface by parking a vehicle on top of it. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced at 1-metre intervals below the ground surface to the maximum depth of the casing (20.4 m).

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

## Data Processing

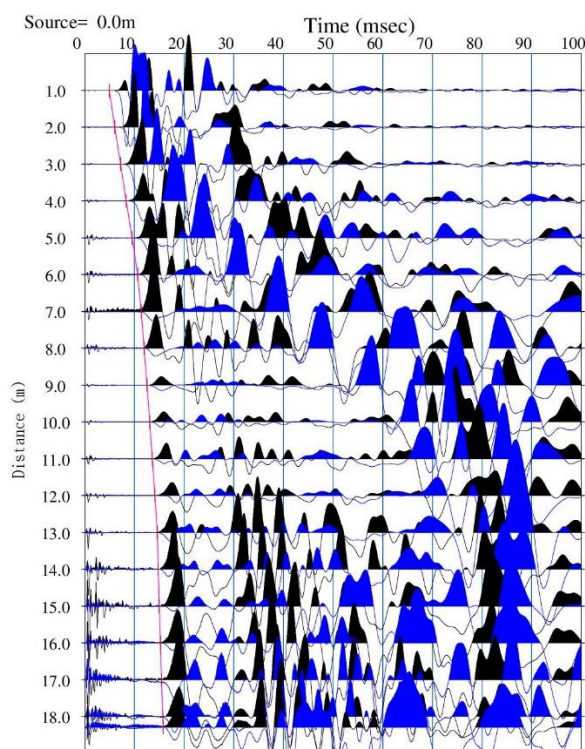
Processing of the VSP test results consisted of the following main steps:

- 1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;

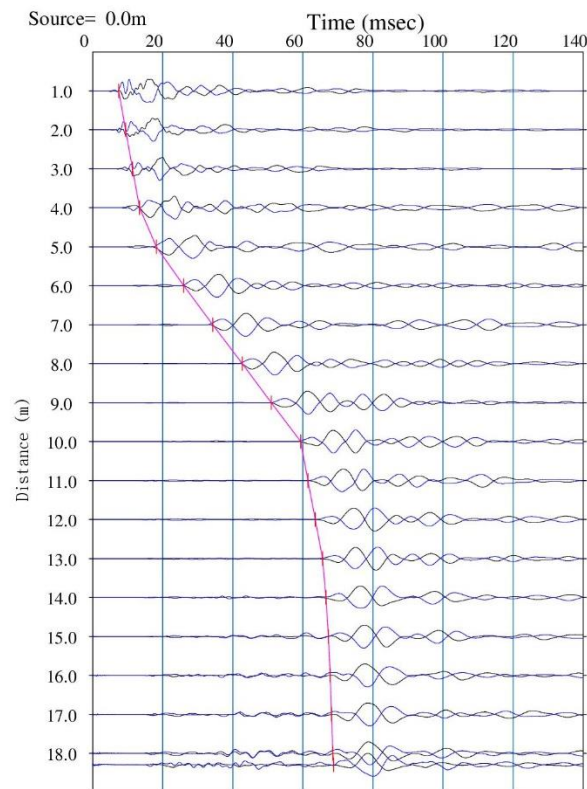


- 2) Low Pass Filtering of data to remove spurious high frequency noise;
- 3) First break picking of the compression and shear-wave arrivals; and,
- 4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records at BH18-1101 are presented on the following two plots and show the first break picks of the compression wave (Figure 1) and shear wave arrivals (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

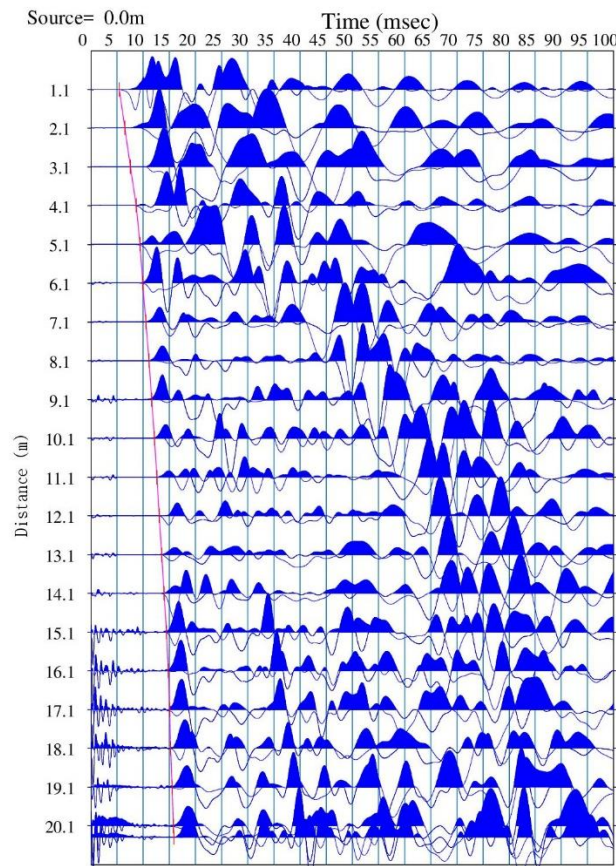


**Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole BH18-1101.**

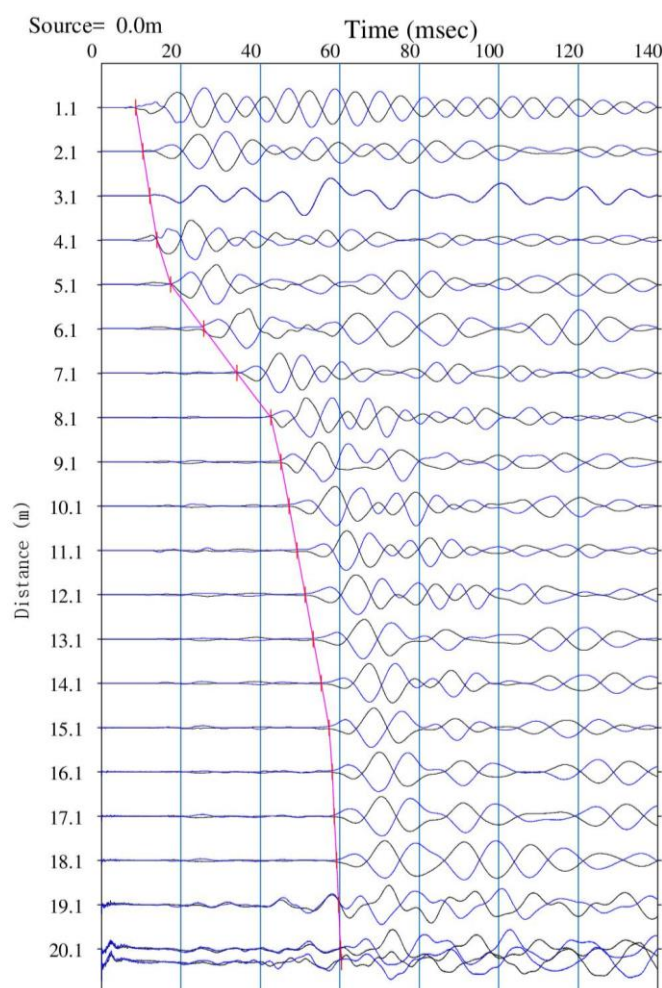


**Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 18-1101.**

The seismic records at BH18-1103 are presented on the following two plots and show the first break picks of the compression wave (Figure 3) and shear wave arrivals (Figure 4) overlaid on the seismic waveform traces recorded at the different geophone depths. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.



**Figure 3: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 18-1103.**



**Figure 4: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 18-1103.**

## Results

The VSP results at BH18-1101 and BH18-1103 are summarized in Tables 1, and Table 2, respectively. The shear wave and compression wave layer velocities were calculated by best fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Tables 1 and 2. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. At boreholes 18-1101 and 18-1103, an estimated bulk density of 2000 kg/m<sup>3</sup> was used for silty sand, 1,550 kg/m<sup>3</sup> for silty clay, and an estimated bulk density of 2,600 kg/m<sup>3</sup> was used for the limestone bedrock.

At borehole 18-1101, the average shear wave velocity from ground surface to a depth of 30 metres was measured to be 404 metres per second. The average velocity at 18-01 was calculated assuming that the velocity from 18.3 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 2,000 m/s which is equal to the velocity at the bottom of the borehole.

At borehole 18-1103, the average shear wave velocity from ground surface to a depth of 30 metres was measured to be 459 metres per second. The average velocity at 18-1103 was calculated assuming that the velocity from 20.4 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 1,800 m/s which is equal to the velocity at the bottom of the borehole.

## Limitations

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

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

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

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

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

## Closure

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

### GOLDER ASSOCIATES LTD.



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



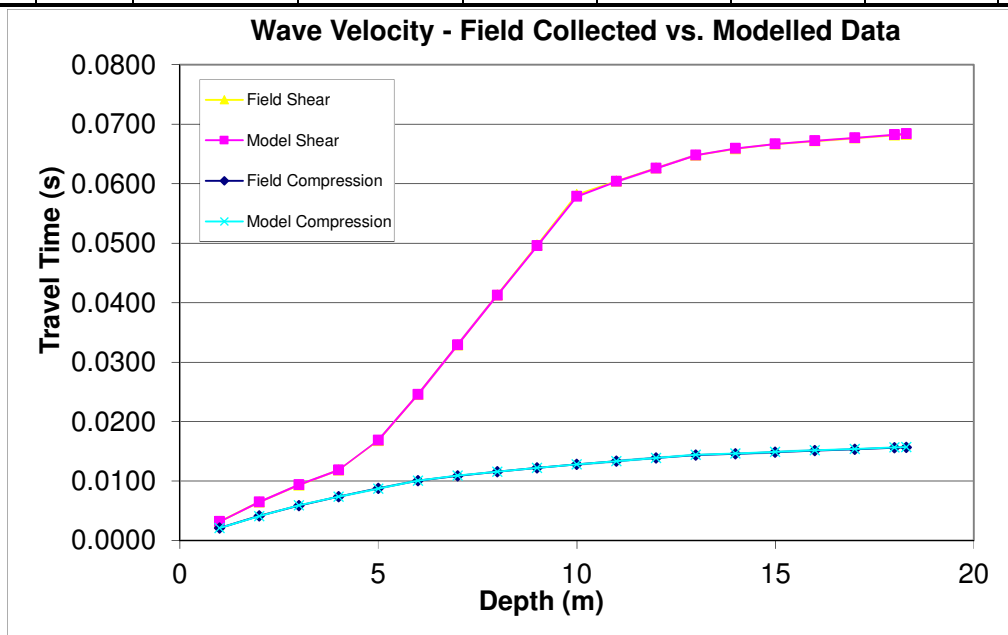
Christopher Phillips, M.Sc., P. Geo.  
*Senior Geophysicist, Principal*

Attach: Tables 1 & 2

SS/CRP/jl

**TABLE 1**  
**SHEAR WAVE VELOCITY PROFILE AT BOREHOLE BH18-1101**

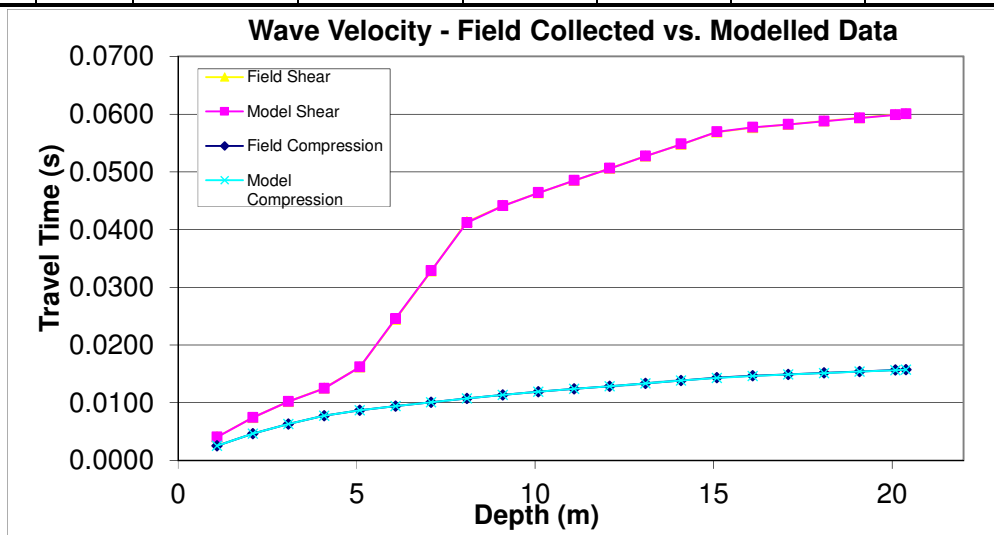
Layer Depth (m)		Velocities (m/s)		Estimated Bulk Density (kg/m <sup>3</sup> )	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1.0	480	316	2000	0.12	200	446	195
1.0	2.0	500	305	2000	0.20	186	448	252
2.0	3.0	550	345	2000	0.18	238	560	288
3.0	4.0	680	400	2000	0.24	320	791	498
4.0	5.0	730	200	2000	0.46	80	234	959
5.0	6.0	770	130	1550	0.49	26	78	884
6.0	7.0	1160	120	1550	0.49	22	67	2056
7.0	8.0	1480	120	1550	0.50	22	67	3365
8.0	9.0	1510	120	1550	0.50	22	67	3504
9.0	10.0	1800	120	1550	0.50	22	67	4992
10.0	11.0	1800	400	2000	0.47	320	943	6053
11.0	12.0	1800	450	2000	0.47	405	1188	5940
12.0	13.0	2000	450	2000	0.47	405	1193	7460
13.0	14.0	3900	900	2600	0.47	2106	6200	36738
14.0	15.0	4000	1300	2600	0.44	4394	12663	35741
15.0	16.0	4000	1900	2600	0.35	9386	25423	29085
16.0	17.0	4000	2000	2600	0.33	10400	27733	27733
17.0	18.0	4000	2000	2600	0.33	10400	27733	27733
18.0	18.3	4000	2000	2600	0.33	10400	27733	27733

**Notes**

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

**TABLE 2**  
**SHEAR WAVE VELOCITY PROFILE AT BOREHOLE BH18-1103**

Layer Depth (m)		Velocities (m/s)		Estimated Bulk Density (kg/m <sup>3</sup> )	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1.1	430	270	2000	0.17	146	342	175
1.1	2.1	475	295	2000	0.19	174	413	219
2.1	3.1	595	365	2000	0.20	266	639	353
3.1	4.1	700	430	2000	0.20	370	885	487
4.1	5.1	1050	270	2000	0.46	146	427	2011
5.1	6.1	1400	120	1550	0.50	22	67	3008
6.1	7.1	1500	120	1550	0.50	22	67	3458
7.1	8.1	1550	120	1550	0.50	22	67	3694
8.1	9.1	1600	345	2000	0.48	238	703	4803
9.1	10.1	1880	440	2000	0.47	387	1139	6553
10.1	11.1	1970	470	2000	0.47	442	1299	7173
11.1	12.1	2060	480	2000	0.47	461	1356	7873
12.1	13.1	2100	470	2000	0.47	442	1302	8231
13.1	14.1	2100	480	2000	0.47	461	1357	8206
14.1	15.1	2100	470	2000	0.47	442	1302	8231
15.1	16.1	2900	1300	2000	0.37	3380	9290	12313
16.1	17.1	4000	1900	2600	0.35	9386	25423	29085
17.1	18.1	4000	1800	2600	0.37	8424	23133	30368
18.1	19.1	4000	1800	2600	0.37	8424	23133	30368
19.1	20.1	4000	1800	2600	0.37	8424	23133	30368
20.1	20.4	4000	1800	2600	0.37	8424	23133	30368

**Notes**

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



**APPENDIX J**

**Seismic Response Analysis  
Technical Memorandum**





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## TECHNICAL MEMORANDUM

**DATE** 12 June 2023

**Reference No.** 22513877-TM-RevA

**TO** Kenton Power  
Paul Dittrich, PhD, PEng  
WSP Canada Inc. – Toronto/Ottawa

**FROM** Mahdi Shahrabi, PEng  
Upul Atukorala, PhD, PEng  
WSP Canada Inc. - Vancouver

**EMAIL** mahdi.shahrabi@wsp.com

### **SEISMIC RESPONSE ANALYSIS OF PROPOSED BRIDGE EMBANKMENTS PRELIMINARY DESIGN OF REPLACEMENT OF FRASER ROAD UNDERPASS AT HIGHWAY 401 UNITED COUNTIES OF STORMONT, ONTARIO**

Dear Kenton/Paul,

WSP Canada Inc. – Vancouver Office has been commissioned to carry out seismic response analysis of the embankments of the proposed replacement bridge for the existing Fraser Road Bridge (Site No. 31-230) over Highway 401. The bridge is located in the United Counties of Stormont, Dundas and Glengarry (SDG), Ontario.

This technical memorandum provides the results of the seismic analyses completed as input to the detail design of the proposed replacement Fraser Road Bridge embankment. All analyses completed herein correspond to ground motions with a return period of 2,475 years, equivalent to having a 2% chance of being exceeded in 50 years.

The memorandum should be read in conjunction with the “**Information and Limitations of This Technical Memorandum**” which is included following the text of this memorandum. The reader’s attention is specifically drawn to this information, as it is essential that it is followed for the proper use and interpretation of this memorandum.

## **1.0 GENERAL AND GEOTECHNICAL SITE CONDITIONS**

The Fraser Road Bridge is located over Highway 401 in the United Counties of SDG, Ontario. The existing bridge (Site No. 31-230) is located at about Station 23+050 on Highway 401. The new replacement bridge will be designed to accommodate the future widening of Highway 401. It is understood that the new bridge will be replaced on the existing abutment.

This section of Highway 401 lies within the major physiographic region known as the Lancaster Flats (Golder 2019). The Lancaster Flats region is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock. This region is underlain by a series of sedimentary rocks, consisting of limestones and shales that are, in turn, underlain at depth by igneous and metamorphic bedrock of the Precambrian Shield.

A series of historical and complementary subsurface site investigations (Golder 2019, WSP 2023) have been carried out to identify the geotechnical site conditions. A detailed discussion of the results of past and recent site investigations has been provided elsewhere (WSP 2023) and is outside of the scope of this memorandum.

The geometry of the proposed bridge approach embankment and soil layers were established based on the recently updated geotechnical cross section and discussions with yourselves and the details are provided in Attachment 1.

**Table 1-1** summarizes the soil profile and characteristic parameters at the north approach embankment established based on the available geotechnical information. For the purposes of seismic site response analysis, the design shear wave velocity profile was selected based on field measurements and geotechnical interpretations as provided in Figure 1-1.

**Table 1-1: Soil Profile and Characteristic Parameters (North Bridge Abutment)**

Soil Layer	Elevation Range <sup>[1]</sup> (m)	Unit Weight (kN/m <sup>3</sup> )	V <sub>s</sub> <sup>[2]</sup> (m/sec)	Effective Friction Angle (degrees)	Undrained Shear Strength (kPa)	OCR	Plasticity Index
Fill (SM)	56.0 to 48.5	21.5	200 to 350	35	N/A	N/A	N/A
Topsoil (ML)	48.5 to 48.0	17	150	30	N/A	N/A	N/A
Silty Sand (SM)	48.0 to 47.5	19	120	32	N/A	N/A	N/A
Clay Crust (CH)	47.5 to 46.5	17	120	N/A	55	2	50
Clay (CH)	46.5 to 42.5	16	120	N/A	35 to 40	1	50
Till (SP/GP)	42.5 to 39.0	22	350 to 450	35	N/A	N/A	N/A
Bedrock	From 39.0	22	2000	N/A	N/A	N/A	N/A

<sup>[1]</sup> Relative to the Geodetic Datum, as used in one dimensional seismic site response analysis described in Section 3.

<sup>[2]</sup> Design V<sub>s</sub> profile shown in Figure 1-1.

It is noted that the one dimensional seismic site response analysis presented here (Section 3.1) was carried out at the north side of proposed bridge abutment for comparison with the two dimensional analysis results (Section 3.2). Based on our review of the recently gathered and historical geotechnical testhole information, while soil layer thicknesses and elevations vary across the site, geotechnical properties of soil layers are not significantly different on the south and north sides of the bridge location. It is noted that for the clay layer, the undrained shear strength is different in different areas due to the effects of the consolidation and strength gain that has occurred in the clay soils following the original construction of the embankment. The variation in the undrained shear strength of the clays has been reviewed and commented on in Section 4.0.

## 2.0 GROUND MOTION TIME-HISTORIES

Seismic site response analyses require representative earthquake time-histories as input. Existing time histories from past earthquakes are commonly modified so that the response spectra approximate the design response spectrum. The seismic design provisions for this project were based on the 6<sup>th</sup> generation seismic hazard model developed for the National Building Code of Canada of 2020 (NBC 2020). The ground motions used in the analyses presented herein were provided by the WSP design team for use as representative input ground motions for this project, which are understood to represent the site seismicity and the period range of interest. The details of time-history development are outside our scope of work and are documented elsewhere.

For the subject project, 12 single component ground motion time-histories were used as shown in **Table 2-1**. Spectral acceleration plots of the input ground motions and the acceleration time-history plots are provided in Figures 2-1 and 2-2, respectively.

**Table 2-1: List of Ground Motion Time-Histories Used**

Earthquake Record	PEER NGA Database ID	Moment Magnitude	Duration (sec)	Scaled Peak Ground Acceleration (g) <sup>[1]</sup>
San Fernando, CA	RSN72 (H1)	6.6	35	0.38
San Fernando, CA	RSN72 (H2)	6.6	35	0.28
Friuli, Italy	RSN123	6.5	38	0.21
Coyote Lake, CA	RSN146	5.7	27	0.28
Livermore, CA	RSN210	5.8	40	0.23
Trinidad	RSN281	7.0	22	0.15
Coalinga, CA	RSN357	6.4	60	0.23
Borah, ID	RSN438	6.9	24	0.21
Big Bear, CA	RSN934	6.5	40	0.23
Northridge, CA	RSN1011	6.7	30	0.21
Sierra Madre, CA	RSN1643	5.6	65	0.24
L'Aquila, Italy	RSN4472	6.3	60	0.30

<sup>[1]</sup> Time histories linearly scaled to match NBCC2020 (2,475-year return period) spectral accelerations within the spectral period range of 0.2 to 2 sec.

## 3.0 SITE RESPONSE ANALYSES

Seismic site response analyses were carried out using both one dimensional (1D) and two dimensional (2D) models. The results of 1D analyses were used primarily to estimate the likely magnitude of the shear strains induced in the clayey soils (during upward propagation of seismic waves) that have a potential for strain softening. Ground surface response spectra computed using the 1D and 2D models were also compared with each other for checking of results from the 2D model.

### 3.1 One Dimensional (1D) Seismic Site Response Analysis

The nonlinear 1D seismic site response analysis was carried out using the computer software DEEPSOIL 7.0 to assess the propagation of seismic waves generated from bedrock at about elevation 39 m, and to estimate maximum cyclic shear strains developed in the clay layer. The General Quadratic/Hyperbolic soil model as well as non-Masing hysteretic re/unloading formulation in DEEPSOIL were used for the nonlinear 1D seismic site response analyses. Based on available Vs measurements, bedrock at this site corresponds to Class A, as per criteria outlined in 2020 NBCC.

A generalized soil column, provided in Table 1-1, was analyzed based on the soil profile along north side of the proposed bridge approach embankment. The modulus reduction and damping curves used in the 1D site response analyses are summarized as follows:

- Clays (CH): Vucetic and Dobry (1991) curves based on the plasticity index (PI) values from available laboratory test results were used. A PI value of 50 was considered for the clay layer between about El. 47.5 m and 42.5 m.
- Topsoil Silts (ML): Sy et al. (1991) curves for modulus reduction and damping were used for the for silty material.
- Sandy soils: EPRI (1993) curves for modulus reduction and damping were used for the embankment fill and native granular soils.

The surface response spectra at El. 56 m were obtained for comparison with the 2D model discussed later in Section 4.0.

### 3.2 Two Dimensional (2D) Seismic Site Response Analysis

2D seismic site response analyses were carried out to assess free field seismic deformations of the proposed bridge approach embankment and slope subject to the seismic time histories discussed in Section 2.0. The response of the proposed bridge approach embankment under dynamic shaking was evaluated using the program PLAXIS2D (Version 21.01). PLAXIS2D is a public domain Finite Element (FE) computer program that can perform both static and seismic site response analysis of soil-structure systems.

A longitudinal section encompassing both the north and south bridge embankments were developed for the subject project to assess seismic response. The geometry of the proposed bridge approach embankment and soil layers were based on the recently updated geotechnical cross section, provided in Attachment 1. The model geometry and FE mesh developed in PLAXIS2D for the purpose of this study has been provided in Figure 3-1. In this model, it has been assumed that both embankments have been constructed using granular fills. As per our approved scope of work, analyzing a transverse section was outside the mandate for this assignment.

It is understood that lightweight (EPS) fill will be required to be installed between the old and new abutment areas in order to satisfy the long-term static settlement criteria, and that the new approach embankments will be contained behind the vertical abutment wall with a 2H:1V slope in front of the abutment. Herein, the free field

seismic deformations of the proposed bridge approach embankment are modeled with a 2H:1V slope profile. As further discussed in Section 4.0, it is observed that the seismic displacement of the embankment is driven by a deep seating movement within the underlying clay layer just under the toe of the embankment slope.

The input ground motion time-histories developed for Site Class 'A' conditions were directly applied on top of bedrock in the FE model using a compliant base boundary condition.

PLAXIS2D includes the PM4Sand model (Boulanger & Ziotopoulou, 2017) as a User Defined Soil Model for simulating soil liquefaction. Materials characterized as exhibiting a *sand-like* behaviour were modelled using PM4Sand. Another model included in PLAXIS2D is the HS-Small (Benz, 2007) which can model the non-linear stress-strain response of clayey soils including stiffness at small strains. The HS-Small model was used to simulate the cyclic behaviour of soils characterized as exhibiting a *clay-like* behaviour with no strength reduction during earthquake shaking due to development of excess pore water pressure.

Primary model parameters established for the two constitutive models PM4SAND and HS-Small used in the FE analysis are summarized in Table 3-1 and 3-2, respectively.

**Table 3-1: PM4SAND Model Parameters <sup>[1]</sup>**

Soil Layer	Design ( $N_1$ ) <sub>60</sub> (Blows/0.3 m)	Apparent Relative Density, $D_R$	Shear Modulus Coefficient, $G_0$	Contraction Rate Parameter, $h_{p0}$
Fill (SM)	35	0.87	2700	2.64
Topsoil (ML)	15	0.57	383	0.40
Silty Sand (SM)	15	0.57	271	0.41
Till (SP/GP)	25	0.74	3751	0.25

<sup>[1]</sup> Default values were selected for other PM4SAND constitutive parameters.

**Table 3-2: HS-Small Model Parameters <sup>[1]</sup>**

Soil Layer	$G_0^{ref}$ (kPa)	$\gamma_{0.7}$ (%) <sup>[2]</sup>	$E_{ur}^{ref}$ (kPa)	$E_{50}^{ref}$ ( $=E_{oed}^{ref}$ ) (kPa)	Undrained Shear Strength, $s_u$ (kPa) <sup>[3]</sup>
Clay Crust (CH)	23,100	9.0E-4	7500	1900	55
Clay (CH)	25,400	9.0E-4	8250	2100	35 to 40

<sup>[1]</sup> Default values were selected for other HS-Small constitutive parameters.

<sup>[2]</sup> Shear strain at which shear modulus degrades to about 70% of initial value.

<sup>[3]</sup> Zoning of undrained shear strength properties for the clay layer discussed in Section 4.0.

## 4.0 SUMMARY OF RESULTS

To verify the calibration of constitutive model parameters used in the FE analysis, the surface response spectra obtained from the PLAXIS2D model at approximately 30 m away from the crest of north bridge abutment slope (ground surface elevation of 56 m), were compared with those obtained from the nonlinear 1D seismic site response analysis described in Section 3.1. It is noted that the results of the 1D non-linear site response analysis indicate that the maximum cyclic shear strains developed in the clay layer are generally 1% or less, while the 2D model shows that the maximum strains in the clays under the bridge embankment are below about 3%, which is mainly driven by the geometry of the slope. Based on the results of available triaxial test data, no appreciable strain softening is expected for this level of straining induced by the upward propagating seismic waves.

Figure 4-1 provides the response spectra of the 12 input ground motions used in the analysis, as well as the ground surface response spectra obtained using the nonlinear 1D and 2D FE analysis approaches. It is noted that given the differences in mathematical and physical approaches of the two analysis approaches, the 1D and 2D ground surface response spectra, especially the peak ground accelerations, compare well.

The computed transient and permanent ground displacements at the crest of the north and south bridge abutments are provided in Figure 4-2. Herein, transient refers to the maximum instantaneous displacements and permanent refers to residual displacements in the ground following the earthquake. While slightly larger at the north abutment, the earthquake-induced displacements are found to be on average less than 120 mm. It is noted that the FE model indicates that maximum displacements generally occur within the clay layer due to its relatively low undrained shear strength. Also, it is expected that the earthquake-induced displacements increase by about 5 to 10% considering the effect of variation in undrained soil strength of the clay layer across the site, by assuming that the undrained shear strength of the clay and clay crust layers varies between 30 to 40 kPa below and outside of the berm in between the bridge abutments.

Figure 4-3 provides an example of the total displacement profiles (Friuli earthquake motion) at the location of the proposed south and north bridge abutments. As shown in this figure, maximum displacements (largest displacement arrows) take place within the soft clay layer, where the horizontal component of displacements are predominant. However, the displacement arrows indicate that horizontal and vertical components of seismic displacement within the embankment fills are approximately equal. It is also observed that total displacements at the north bridge abutment are slightly greater than those on the south side.

Further to the discussion in Section 3.2 on the use of lightweight fill, we re-ran the 2D FE model considering the EPS fill at the front of the embankment (using a 2H:1V slope) using two of the earthquake time histories (Friuli and Northridge). We found that for the analyzed ground motions, the maximum ground displacements at the crest of the bridge abutments were lower by up to approximately 25-35% when lightweight fill was used. We also note based on the FE analysis that while some excess pore water pressure build up occurs within the Silty Sand layer under the embankment during the seismic shaking, it does not liquefy under the design ground level of shaking.

## 5.0 CLOSURE

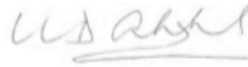
We trust that the information contained in this Technical Memorandum is sufficient for your immediate requirements. Should you have any questions regarding the above, please do not hesitate to contact us.

Yours truly,

**WSP CANADA INC.**



Mahdi Shahrabi, PEng  
*Senior Geotechnical Engineer*



Upul Atukorala, PhD, PEng  
*Principal Senior Geotechnical Engineer*

MS/UA/lih

Attachments: Attachment 1 – Geotechnical Design Cross Section

\\Golder Associates\22513877, MTO 4021 E 0021 0022 Foundation Rtnr - WO 1 22513877A Fraser Rd\Analysis\FEM - Seismic\04. Reporting

## Important Information and Limitations of this Technical Memorandum

**Standard of Care:** WSP Canada Inc. (WSP) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

**Basis and Use of the Report:** This report has been prepared for the specific site, design objective, development and purpose described to WSP by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. WSP can not be responsible for use of this report, or portions thereof, unless WSP is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without WSP's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, WSP may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to WSP. The report, all plans, data, drawings and other documents as well as all electronic media prepared by WSP are considered its professional work product and shall remain the copyright property of WSP, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of WSP. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of WSP's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to WSP by the Client, communications between WSP and the Client, and to any other reports prepared by WSP for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. WSP can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

**Soil, Rock and Groundwater Conditions:** Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, WSP does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions



that WSP interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

**Sample Disposal:** WSP will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

**Follow-Up and Construction Services:** All details of the design were not known at the time of submission of WSP's report. WSP should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of WSP's report.

During construction, WSP should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of WSP's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in WSP's report. Adequate field review, observation and testing during construction are necessary for WSP to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, WSP's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

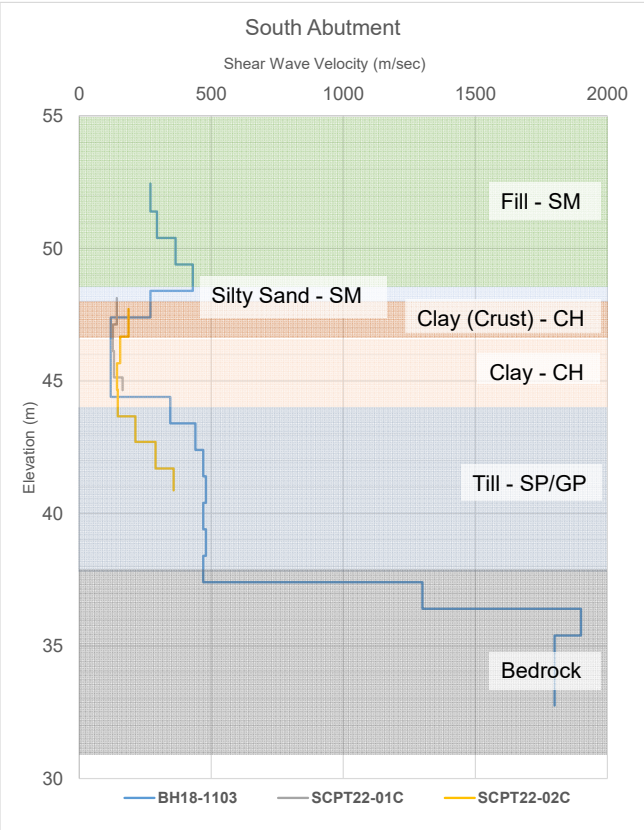
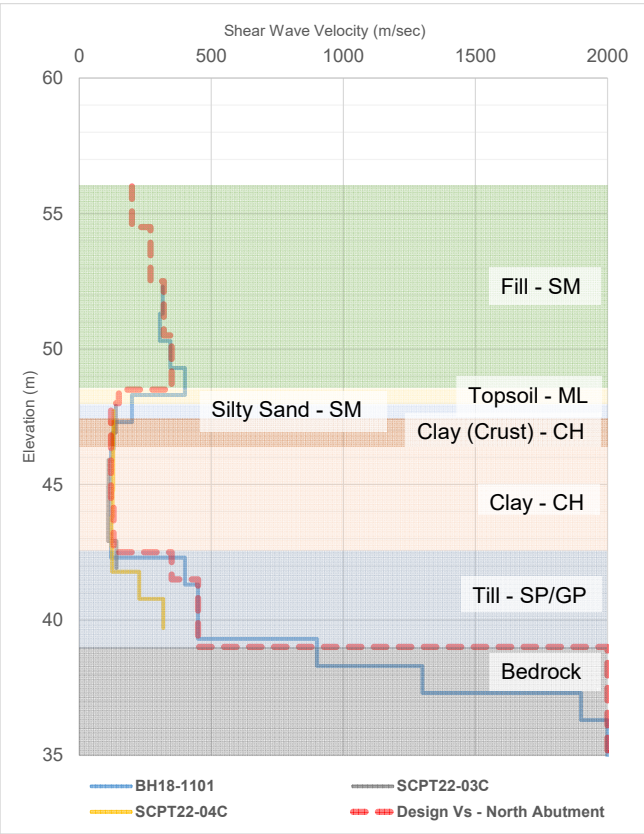
**Changed Conditions and Drainage:** Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that WSP be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that WSP be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. WSP takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

**ATTACHMENT 1**

# Geotechnical Design Cross Section

Path: \\Golder Associates\22513877\_MTO 401 E 0021 0022 Foundation Rmr - W01 22513877A Fraser Rd\Analysis\FEM - Seismic04\_ Reporting



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MINISTRY OF TRANSPORT ONTARIO (MTO)

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DESIGN MS

REVIEW

APPROVED

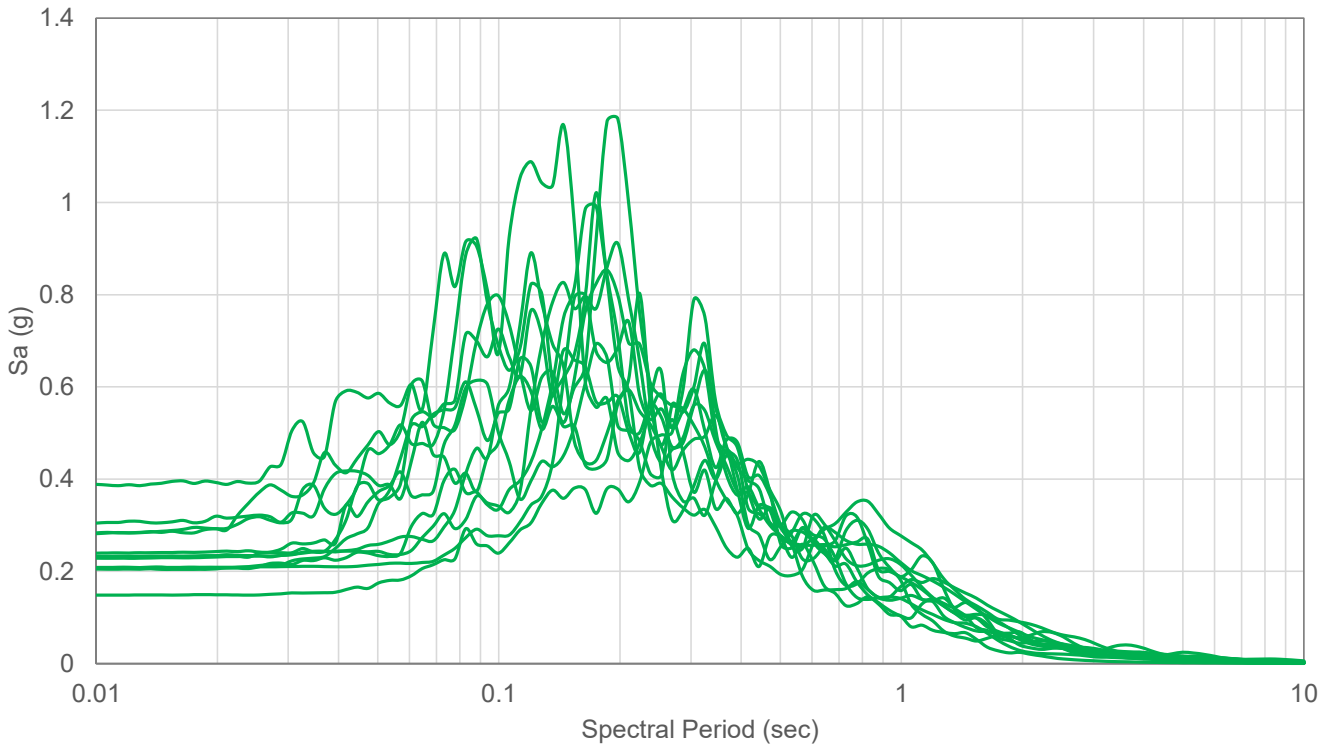
PROJECT  
REPLACEMENT OF HIGHWAY 401 AT FRASER ROAD UNDERPASS  
SEISMIC FINITE ELEMENT ANALYSIS

TITLE  
**SHEAR WAVE VELOCITY MEASUREMENTS AND DESIGN PROFILE**

PROJECT No.  
**22513877**

Rev  
**A**

FIGURE  
**1-1**



— Suite of Input Ground Motions - Site Class A (2,475 Year Return Period)

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MINISTRY OF TRANSPORT ONTARIO (MTO)

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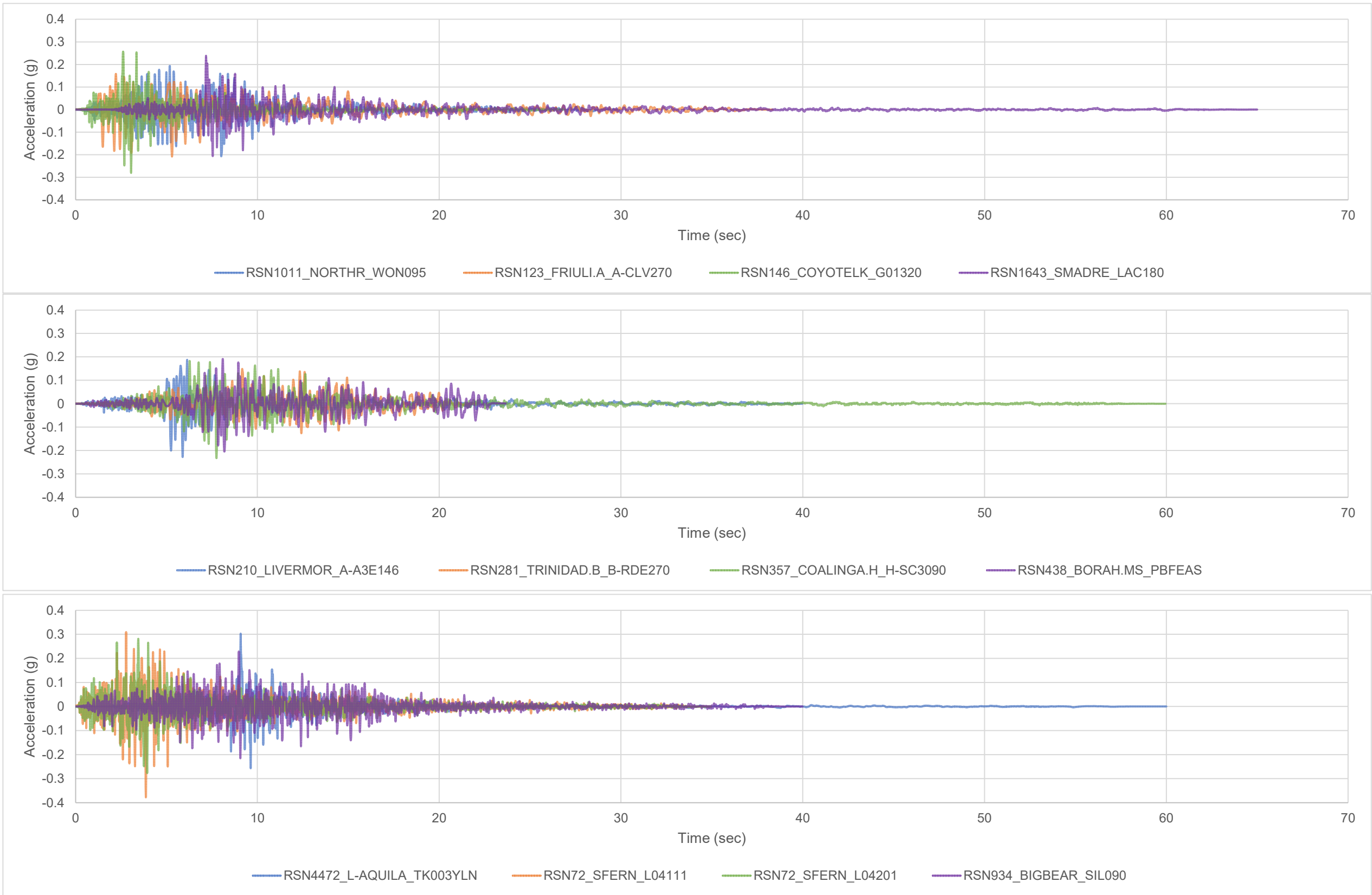
PROJECT  
REPLACEMENT OF HIGHWAY 401 AT FRASER ROAD UNDERPASS  
SEISMIC FINITE ELEMENT ANALYSIS

TITLE  
**INPUT SEISMIC GROUND MOTIONS  
SITE CLASS 'A' – 2,475 YEAR RETURN PERIOD**

PROJECT No.  
**22513877**

Rev  
**A**

FIGURE  
**2-1**



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MINISTRY OF TRANSPORT ONTARIO (MTO)

PROJECT  
REPLACEMENT OF HIGHWAY 401 AT FRASER ROAD UNDERPASS  
SEISMIC FINITE ELEMENT ANALYSIS

CONSULTANT



YYYY-MM-DD 2023-05-26

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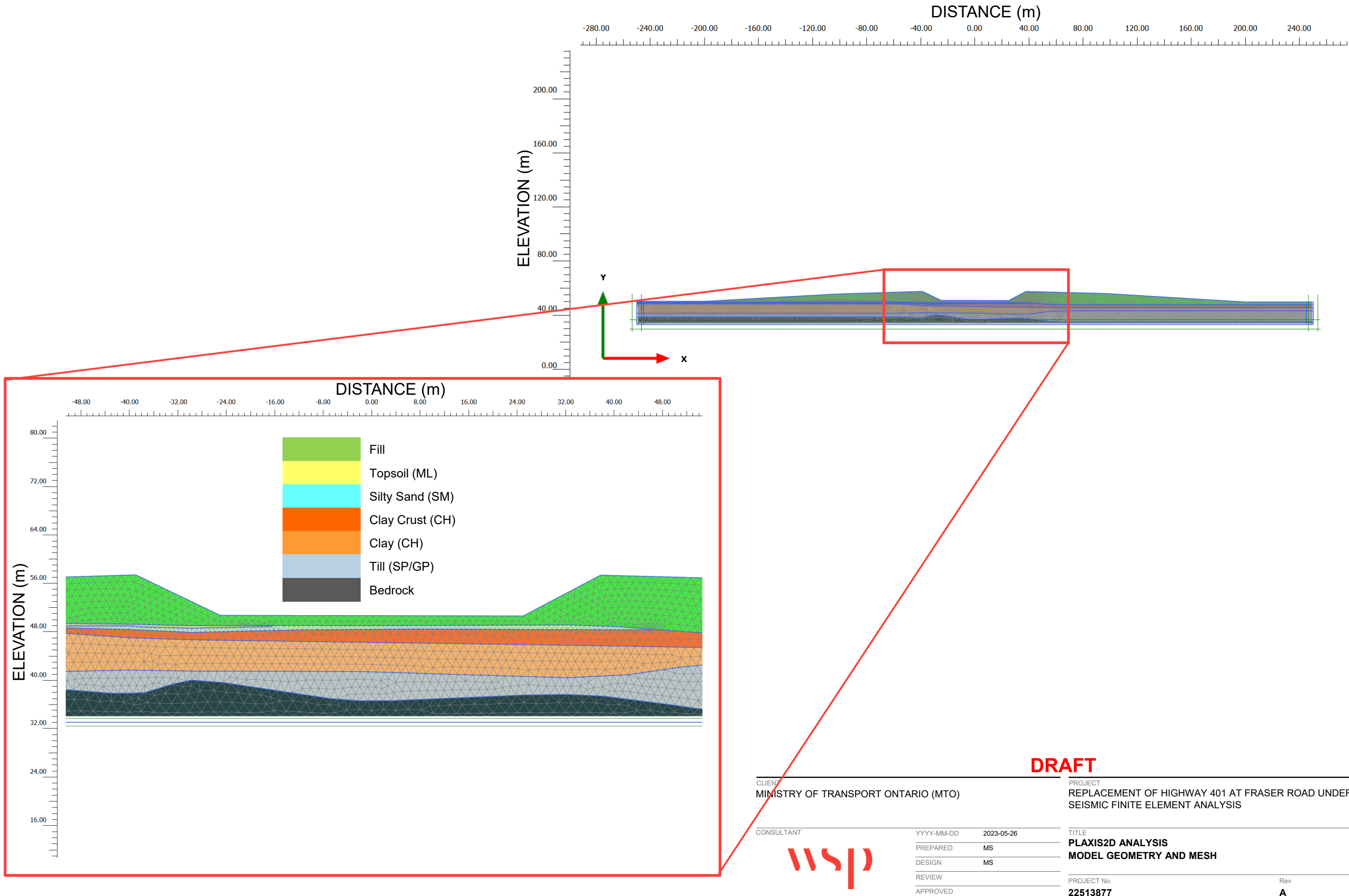
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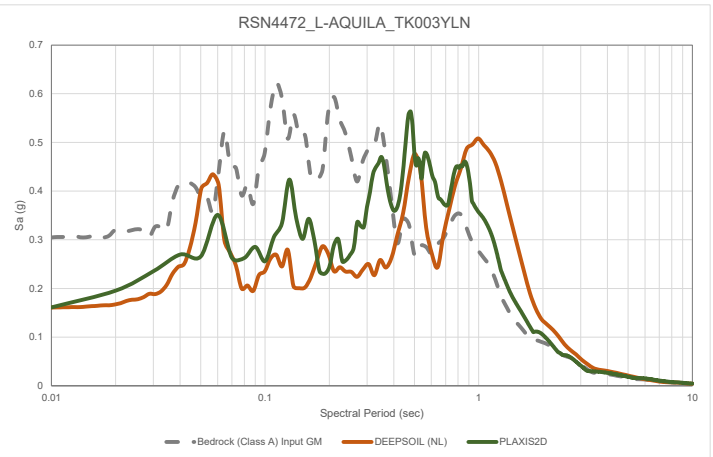
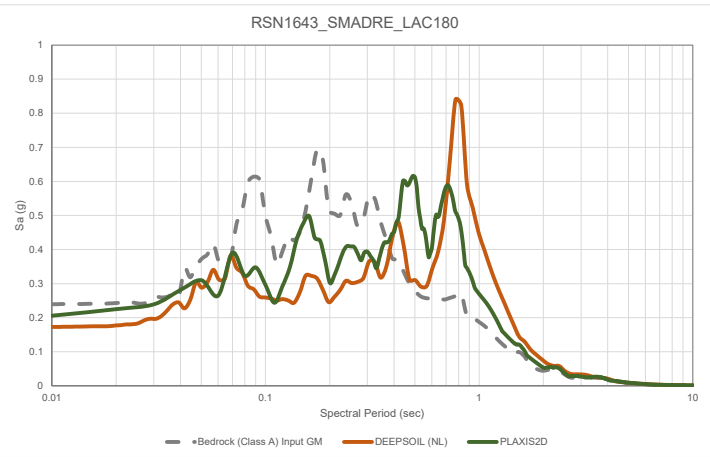
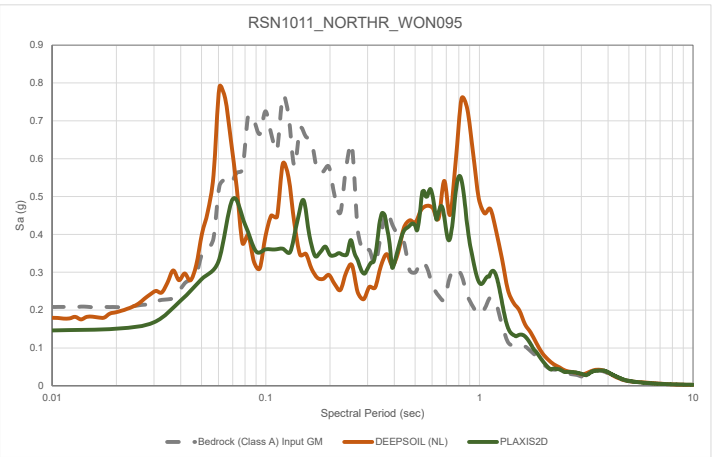
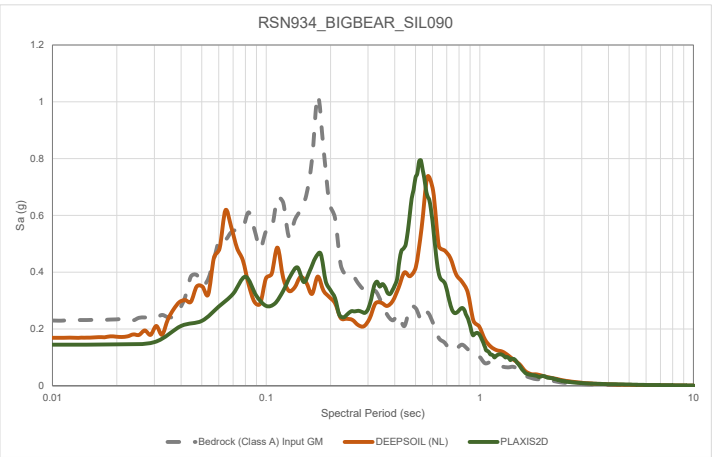
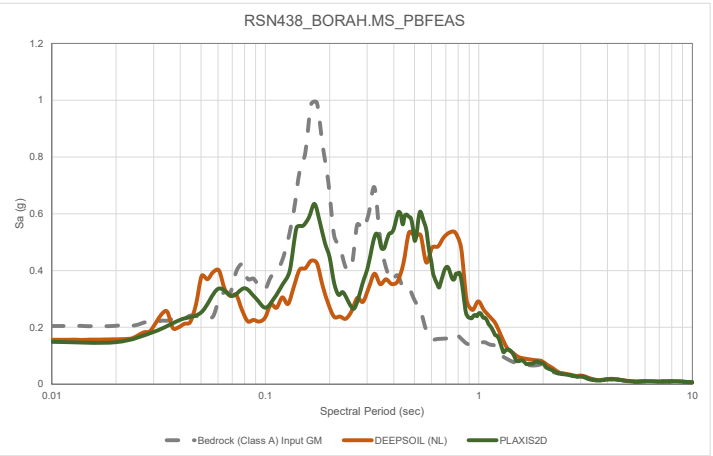
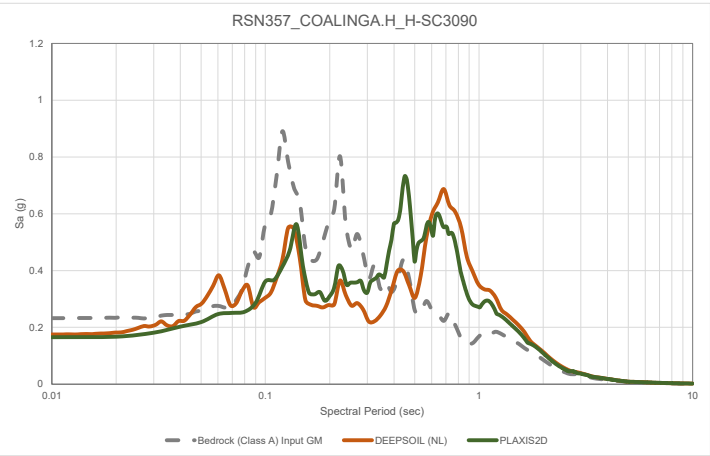
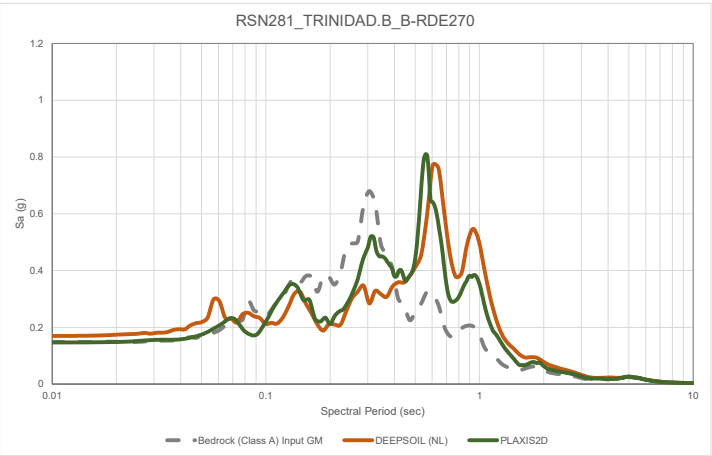
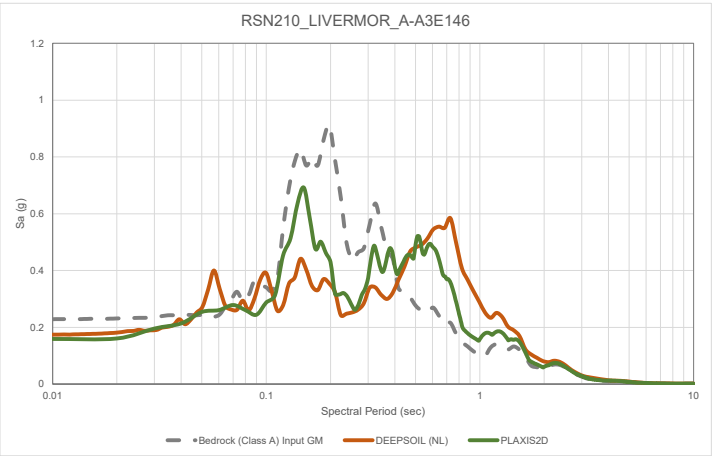
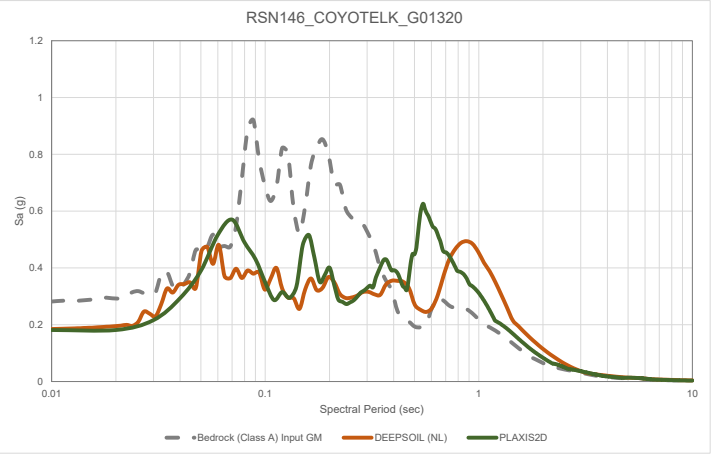
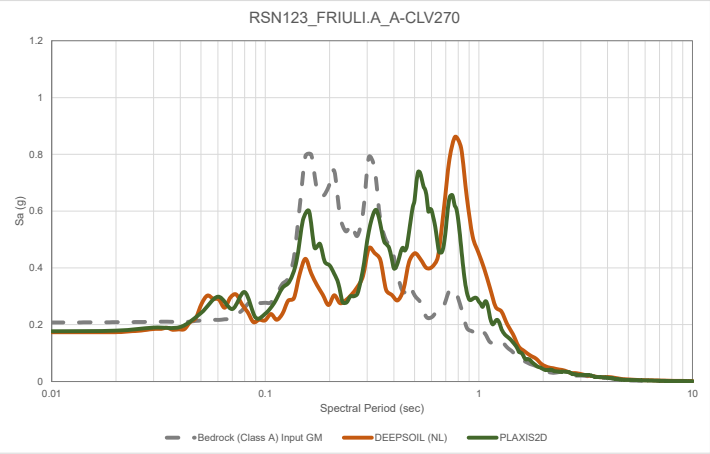
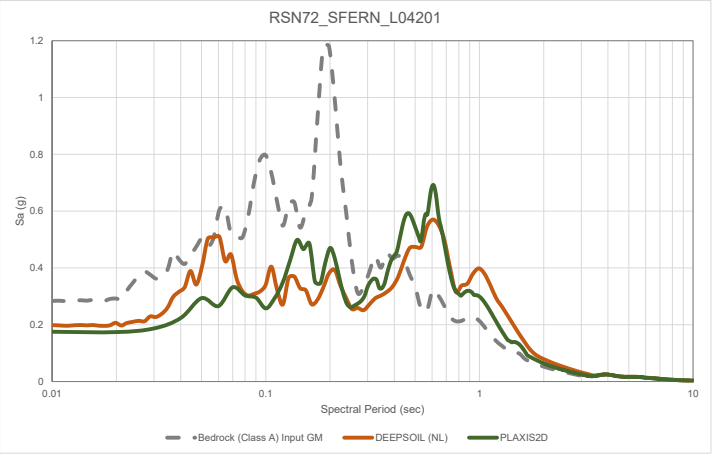
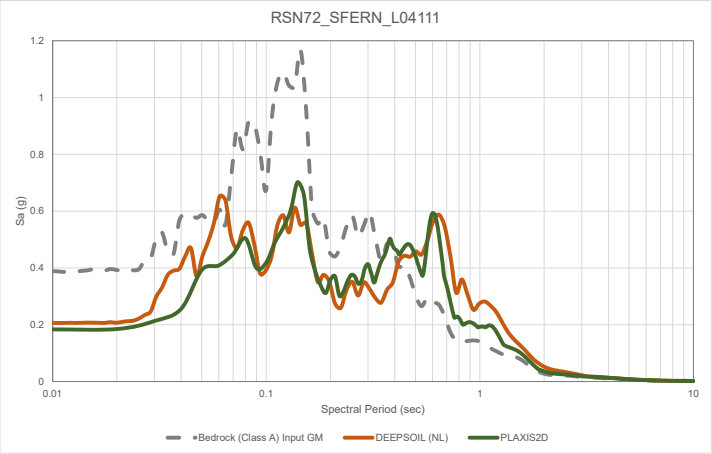
PROJECT No.  
**22513877**

Rev  
**A**

FIGURE  
**2-2**







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MINISTRY OF TRANSPORT ONTARIO (MTO)

PROJECT  
REPLACEMENT OF HIGHWAY 401 AT FRASER ROAD UNDERPASS  
SEISMIC FINITE ELEMENT ANALYSIS

CONSULTANT



YYYY-MM-DD 2023-05-26

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DESIGN MS

REVIEW

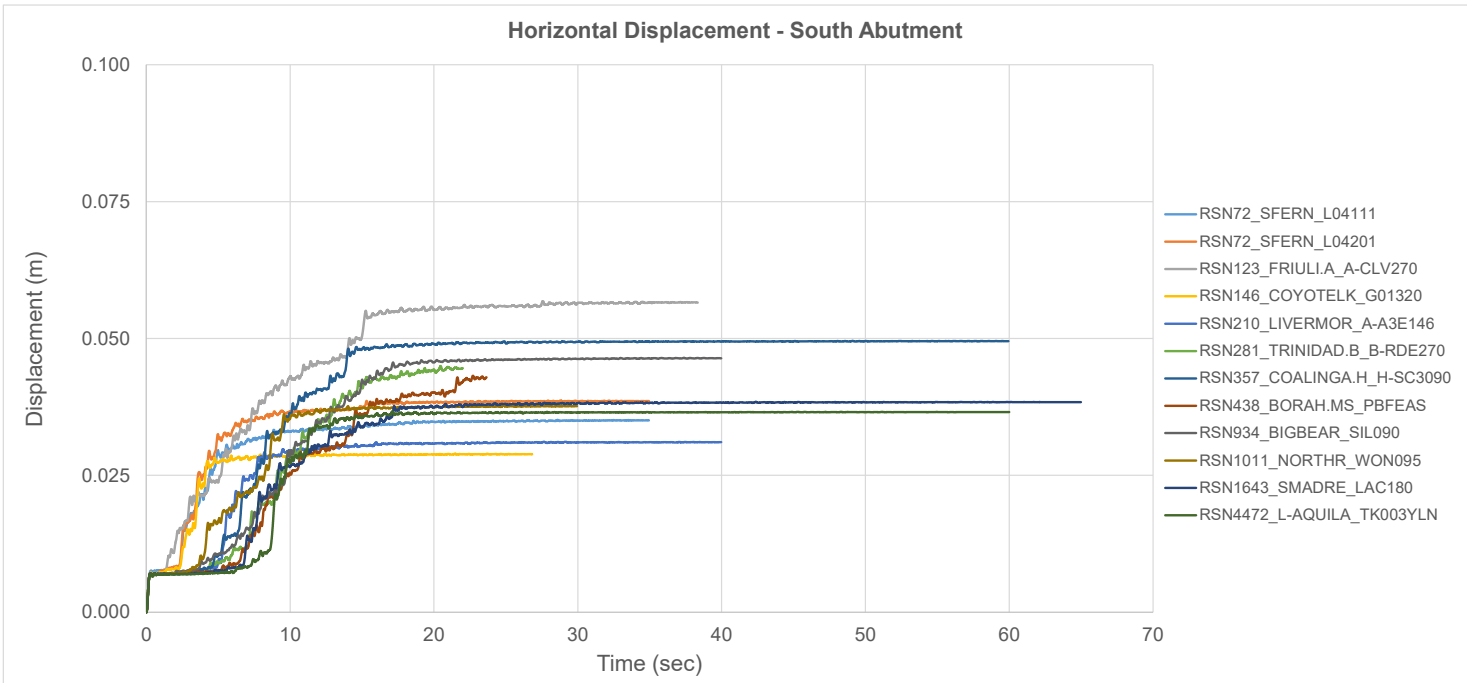
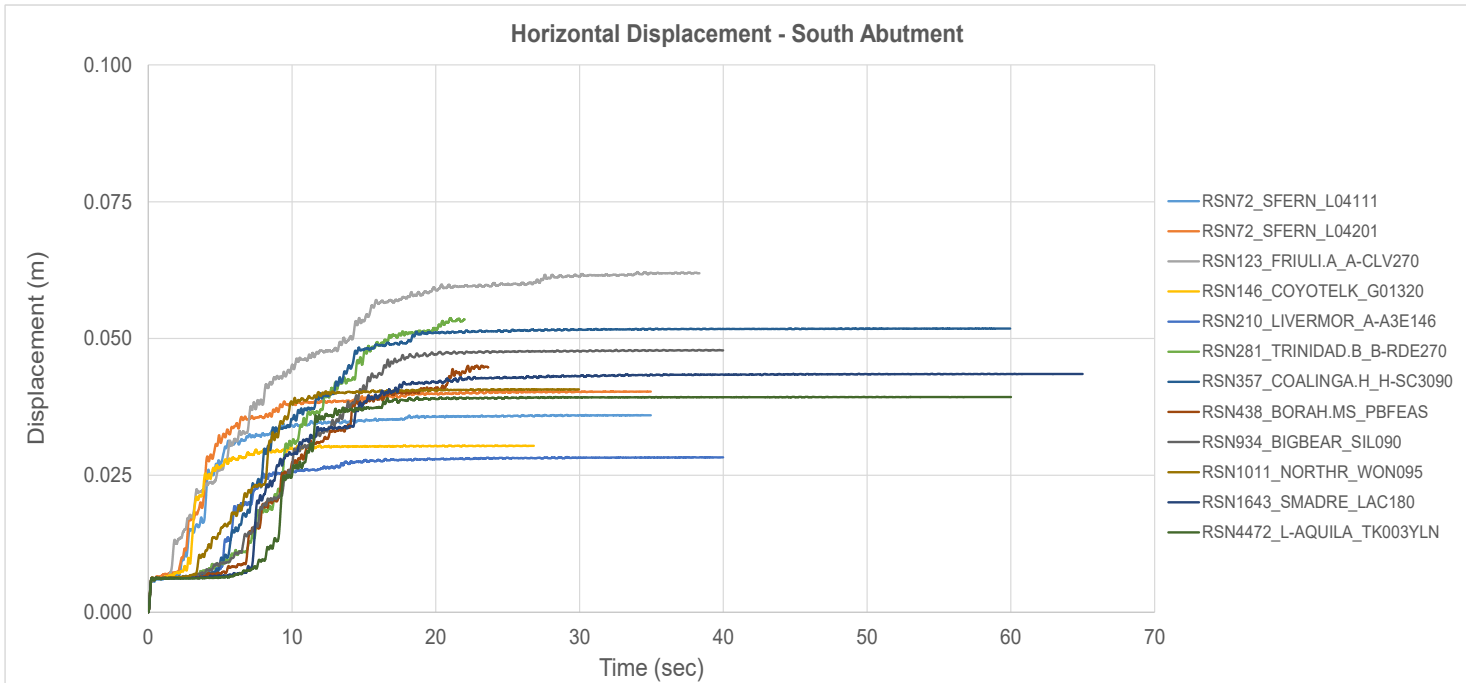
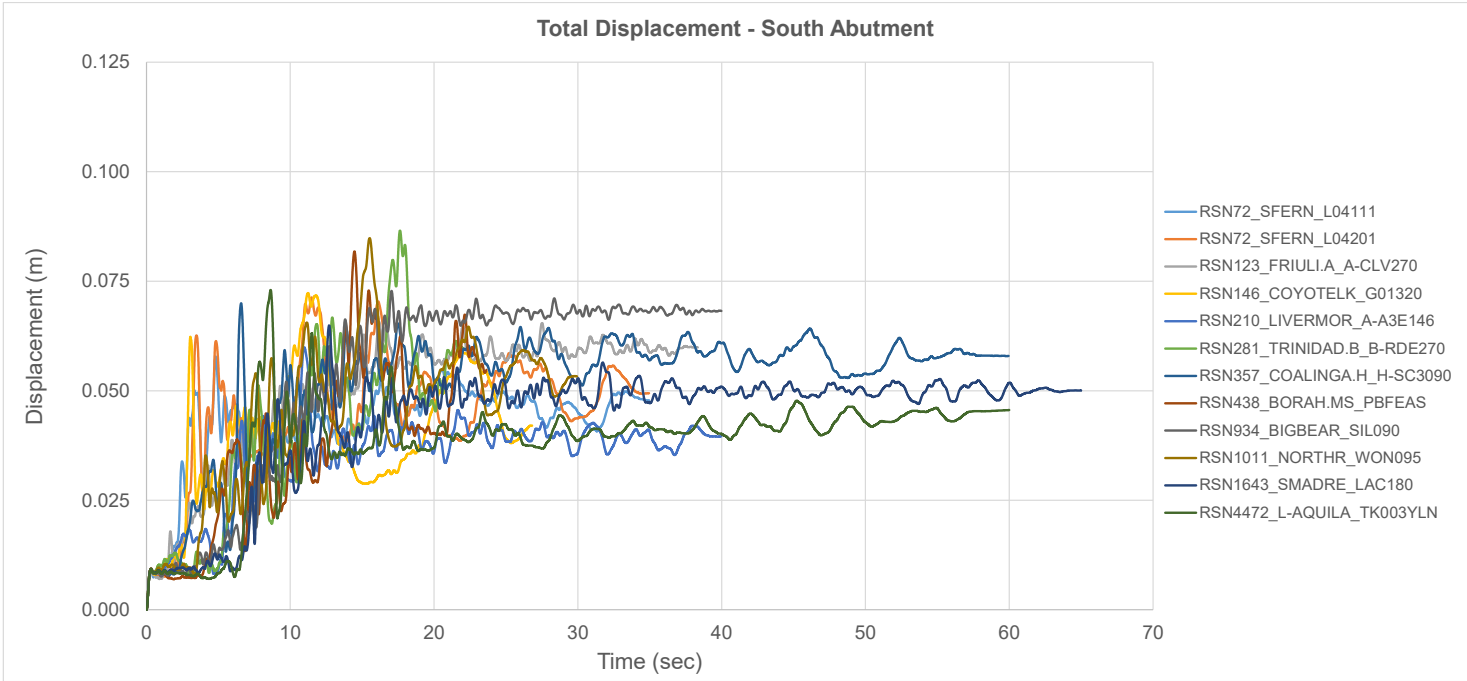
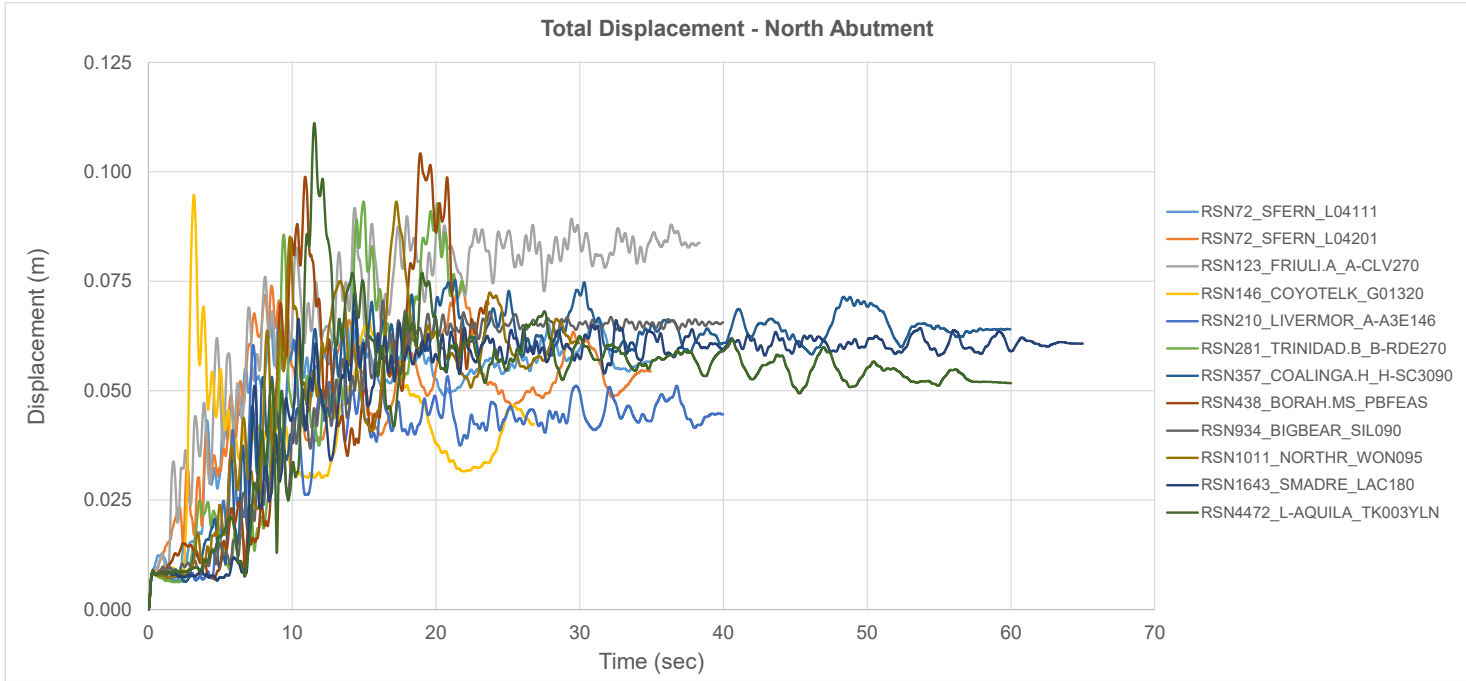
APPROVED

TITLE  
SEISMIC SITE RESPONSE – INPUT MOTION AND SURFACE  
RESPONSE SPECTRA  
DEEPSOIL (1D) VS PLAXIS (2D)

PROJECT No.  
22513877

Rev  
A

FIGURE  
4-1



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CLIENT  
MINISTRY OF TRANSPORT ONTARIO (MTO)

PROJECT  
REPLACEMENT OF HIGHWAY 401 AT FRASER ROAD UNDERPASS  
SEISMIC FINITE ELEMENT ANALYSIS

CONSULTANT



YYYY-MM-DD 2023-05-26

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DESIGN MS

REVIEW

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TITLE  
**PLAXIS 2D – SEISMIC DISPLACEMENTS**

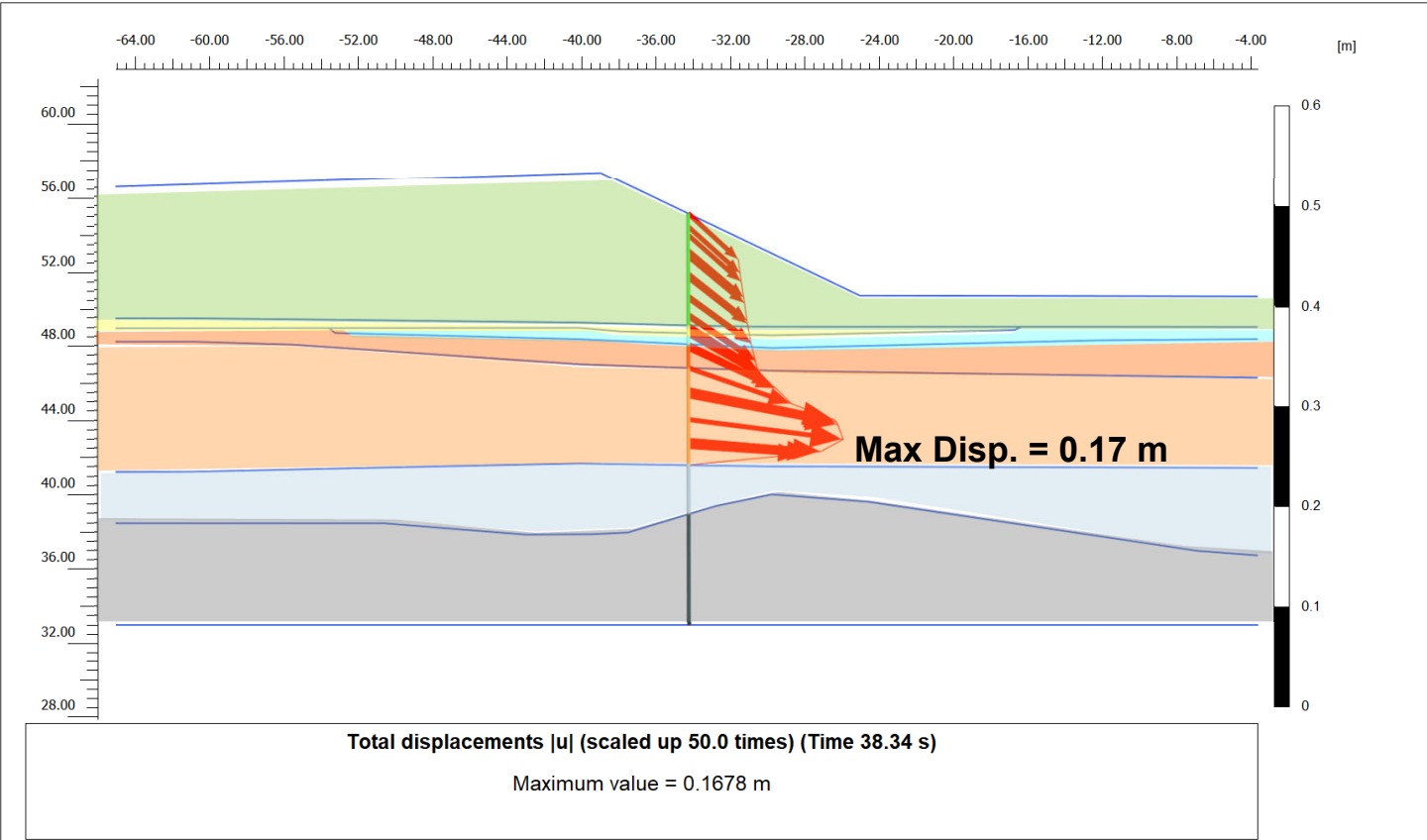
PROJECT No.  
**22513877**

Rev  
**A**

FIGURE  
**4-2**



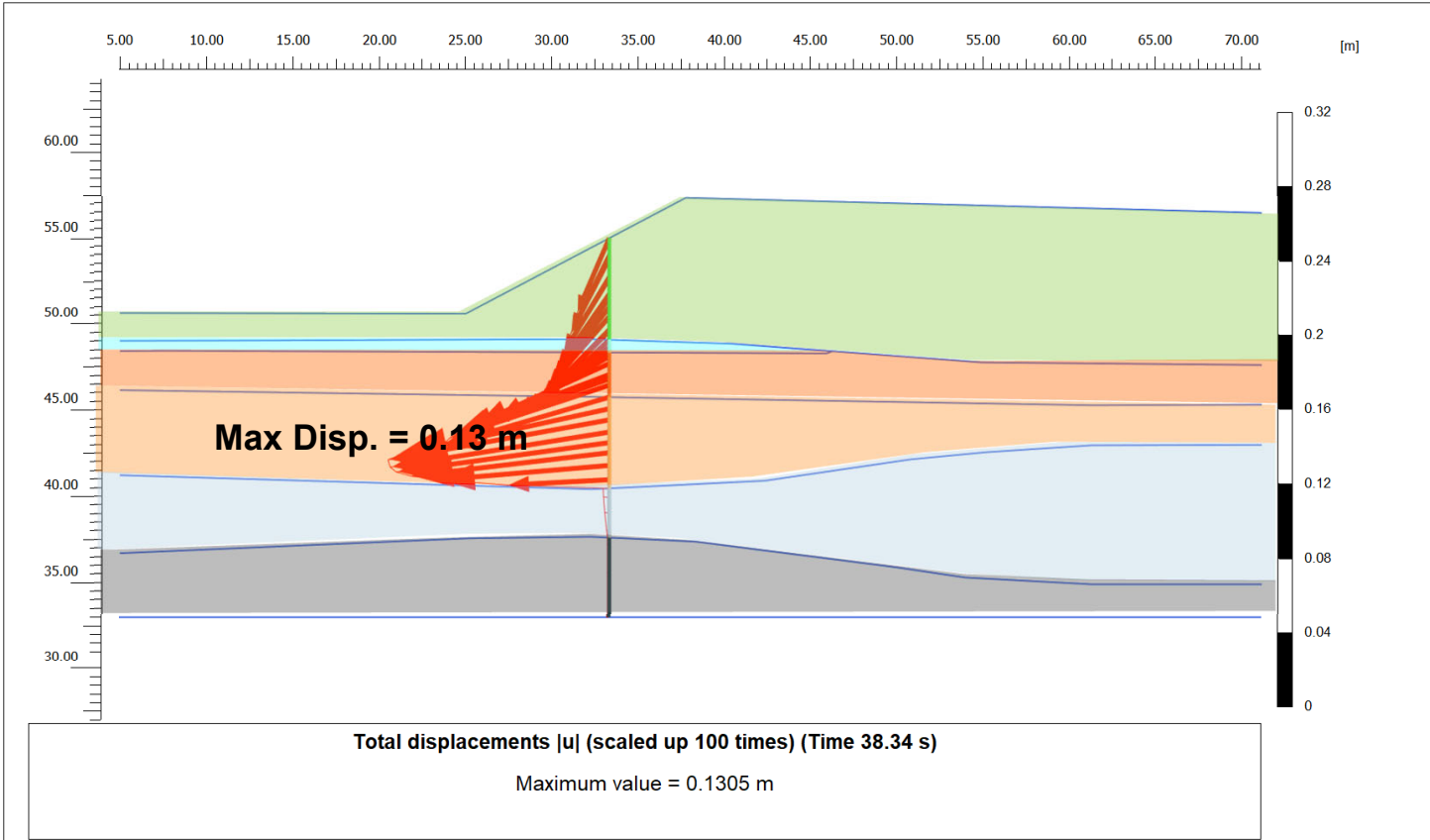
Output Version 21.1.0.479



	Project description			Date
	Total Displacements Profile - North Abutment			2023-06-12
	Project filename		Step	Company
	Fraser Rd 2D Analysis		43295	WSP Global Inc.

	Fill
	Topsoil (ML)
	Silty Sand (SM)
	Clay Crust (CH)
	Clay (CH)
	Till (SP/GP)
	Bedrock

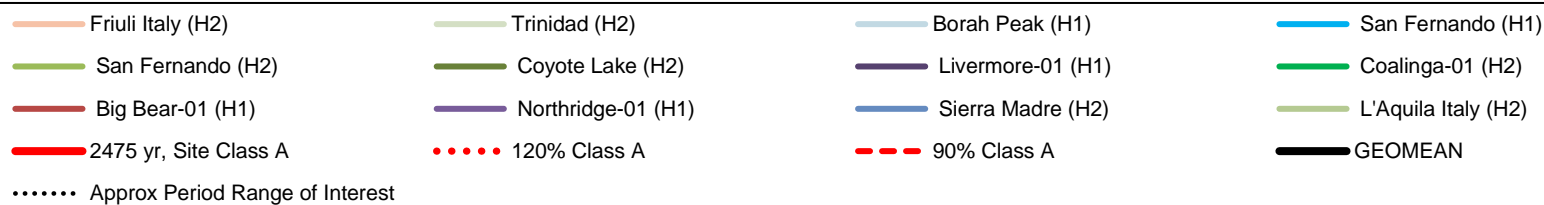
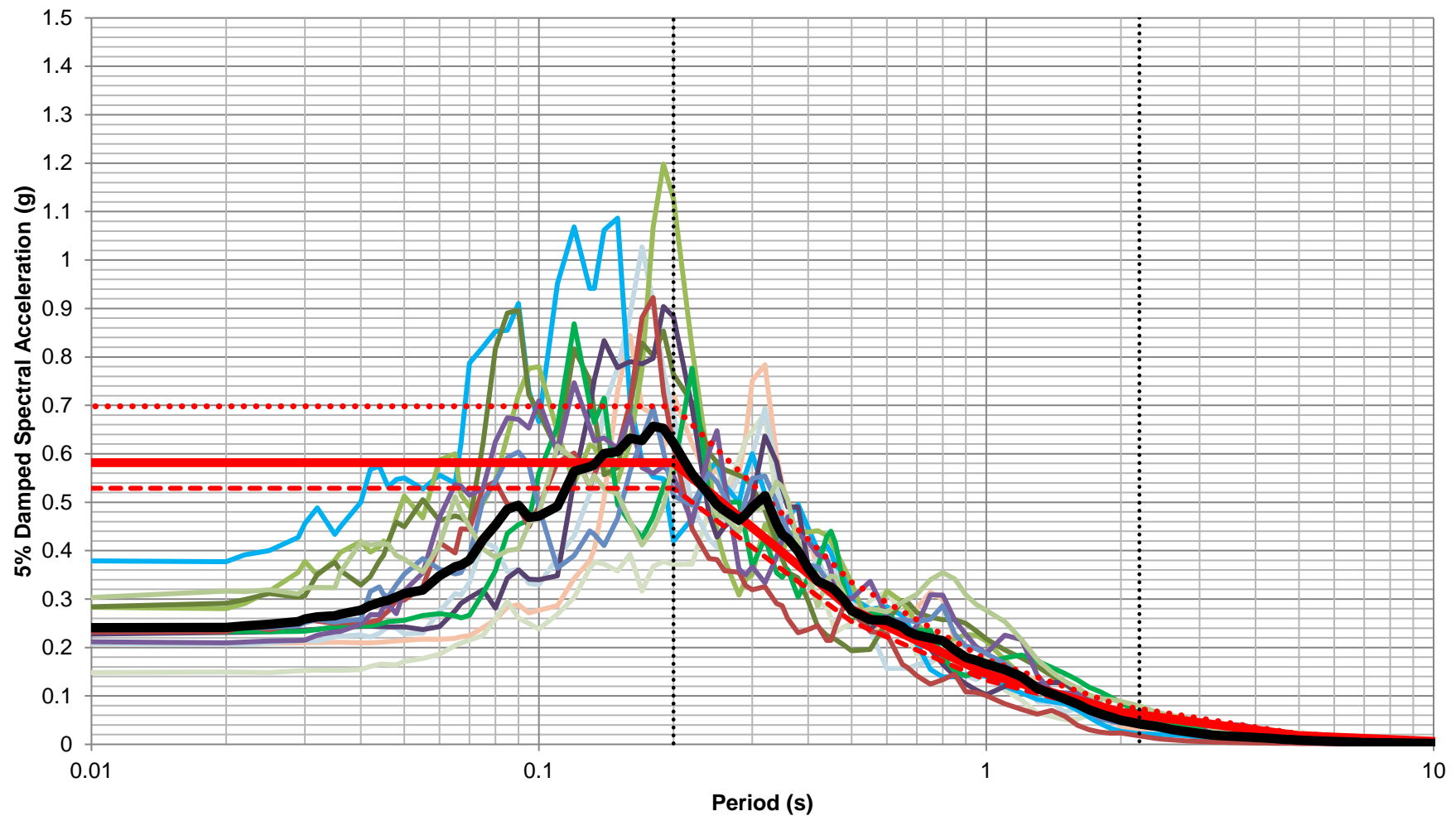
Output Version 21.1.0.479



	Project description			Date
	Total Displacements Profile - South Abutment			2023-06-12
	Project filename		Step	Company
	Fraser Rd 2D Analysis		43295	WSP Global Inc.

DRAFT

CLIENT			PROJECT		
MINISTRY OF TRANSPORT ONTARIO (MTO)			REPLACEMENT OF HIGHWAY 401 AT FRASER ROAD UNDERPASS SEISMIC FINITE ELEMENT ANALYSIS		
CONSULTANT			TITLE		
			PLAXIS 2D – SEISMIC DISPLACEMENTS PROFILE NORTH AND SOUTH ABUTMENTS (FRIULI EARTHQUAKE)		
			PROJECT No.		
			22513877		
			Rev		
			A		
			FIGURE		
			4-3		



**Input Ground Motion Spectra**  
**Site Class A Firm Ground**  
**2475-yr Design Earthquake**  
**Foundation Investigation and Design**  
**Replacement of the Highway 401 Underpass at Fraser Road**  
**Site No. 31-230**  
**MTO WP 4290-15-01, Agreement No. 4021-E-0021**

Project No. 22513877A

Drawn: SM

Date: 2023-06-09

Checked: SM

Review: PD

**Figure J1**

**APPENDIX K**

**Nonstandard Special Provisions  
Notice to Contractor**

**NSSP – “Subsurface Conditions Including Cobbles, Boulders, and Variable Sloping Bedrock”**

The Contractor is alerted to the following subsurface conditions, which shall be taken into account in selection of construction equipment, means and methods:

- For the installation of the steel H-piles, consideration shall be given to the presence of cobbles and boulders within the lower portions of the existing embankment fill and within the granular till stratum above the bedrock. In addition, sloping bedrock and varying depth to the bedrock surface should be anticipated.
- Contractor shall be prepared to dislodge and remove obstructions where they are encountered during excavation at pile cap subgrade level, during installation of the CSPs for the integral abutments, and during excavation for the caissons (drilled shafts) at the Pier. The Contractor shall select methods and equipment that can penetrate these obstructions to reach the required founding levels.
- The surface of the limestone bedrock is irregular and shall be expected to vary, sloping up and down along the length Fraser Road alignment. The caisson (drilled shaft) foundation lengths will vary along and across the replacement foundations, and the Contractor shall ensure that the liners for the caissons (drilled shafts) are properly seated and sealed into the irregular/sloping bedrock and that the base of the rock sockets for the caissons (drilled shafts) are properly seated into the bedrock.
- All driven steel piles shall be fitted with appropriate driving shoes and/or rock point as per OPSS.PROV 903 Section 903.07.02.02 Driving Shoes and Rock Points; the APF Hard-Bit points (Model: HP-77750-B) or equivalent are recommended for this site.

**Notice to Contractor: Potential Conflicts with Existing Foundation Piles**

The Contractor is advised that there is a potential for conflicts with existing pile foundations to occur during installation of the new pile foundations at both the new abutments and centre pier.

Specifically, the Contractor is advised of the presence of existing driven steel piles and/or pile cap at the existing centre pier and battered steel H-piles at the location of both existing abutments. While the design has been developed to locate the new deep foundation elements to avoid or minimize conflicts with existing foundations, such conflicts may occur during construction.

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.

**Notice to Contractor: Basal Heave Due to Groundwater Pressures / Seepage**

The Contractor is advised that basal heave due to groundwater pressures / seepage is to be expected during installation of caisson foundation elements. For caisson foundation construction, temporary casings/liners shall be used to support the overburden soils (at least in the upper zone) during construction to minimize disturbance to the soils and ground loss. Specialized construction techniques will be required during advancement of the caisson to maintain a sufficient head of water and/or drilling fluid (e.g. polymer slurry or other slurry mix) within the temporary liner / open hole to prevent basal heave, disturbance to the water-bearing non-cohesive soils, flowing sands, and ground loss due to groundwater pressures / seepage. In addition, placement of concrete by tremie methods will be required.

**Notice to Contractor: Requirements for Caisson Rock Socket Inspection**

The Contractor is advised that following cleaning to remove all loose cuttings, a qualified geotechnical engineer shall be retained to inspect the base of the caisson rock sockets using a Shaft Quantitative Inspection Device (SQUID). The details of the requirements for caisson socket cleaning, inspection and testing are outlined in the NSSP "Drilled Shafts (Caisson Piles), Supply and Equipment for Installing Drilled Shafts, Drilled Shafts – 900 mm and 1500 mm, Shaft Inspection, and Cross-Hole Sonic Logging (CSL) Testing".

## **RIGID EXPANDED POLYSTYRENE EMBANKMENT FILL – Item No.**

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### **Special Provision**

---

#### **1.0 SCOPE**

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene embankment fill and associated works as shown on the contract drawings.

Use of this special provision shall be according to the Contract Documents.

#### **2.0 REFERENCES**

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.PROV 212	Earth Borrow
OPSS.PORV 501	Compacting
OPSS.PROV 517	Dewatering
OPSS.PROV 904	Concrete Structures

##### **Ontario Provincial Standard Specifications, Materials**

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

##### **National Standards of Canada**

CAN/ULC-S102-10	Standard Method of Test for Surface Burning Characteristics of Building Materials and Assemblies
CAN/ULC-S701-97	Thermal Insulation, Polystyrene, Boards and Pipe Covering

##### **ASTM International**

ASTM C177	Standard Test Method for Steady-State Heat Flux Measurements and Thermal Transmission Properties by Means of Guarded-Hot-Plate Apparatus
ASTM C203	Standard Test Method for Breaking Load and Flexural Properties of Block-Type Thermal Insulation
ASTM C518	Standard Test Method for Steady-State Thermal Transmission Properties by Means of the Heat Flow Meter Apparatus
ASTM D1621	Standard Test Method for Compressive Properties of Rigid Cellular Plastics
ASTM D2126	Standard Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

ASTM D2842	Standard Test Method for Water Absorption by Rigid Cellular Plastics
ASTM D2863	Standard Test Method for Measuring the Minimum Oxygen Content
ASTM D6817	Standard Specification for Rigid Cellular Polystyrene Geofoam

### 3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

### 4.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

**Rigid Expanded Polystyrene** means a moulded rigid block(s) produced by a process of pre-expansion, aging and forming of petroleum based raw material.

**Rigid Extruded Expanded Polystyrene** means a rigid board(s) made by extrusion of expanded polystyrene beads.

**Production Lot** means the quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

**Contractor's Engineer** means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory foundation engineering specialist 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 conformance with the contract documents and issue of certificate(s) of conformance.

### 5.0 QUALIFICATION

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

### 6.0 SUBMISSION AND DESIGN REQUIREMENTS

#### 6.1 Submission of Shop Drawings

At least three (3) weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six (6) copies of the shop drawings and method statement signed and sealed by the Contractor's Engineer that provides full details of materials and construction procedure.

#### 6.2 Delivery, Storage, Handling, and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturer's requirement.

### **6.3 Construction**

The contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) Construction of 300 mm thick levelling pad.
- c) The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.
- d) The method and limits of placement of polyethylene sheeting.
- e) The method of placement of 125 mm thick, 30 MPa reinforced concrete top slab (or equivalent).
- f) The method of placement of subbase material.
- g) The method of placement of side slope cover.

The Contractor shall submit to the Contract Administrator, for review, the details of the sequence and method of installation at least three weeks prior to the installation of the rigid expanded polystyrene embankments. The submittals shall satisfy the specifications and at a minimum contain the above information and include a detailed description of proposed installation procedures.

Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation.

A Request to Proceed shall be submitted to the Contract Administrator upon completion of the foundation excavation and preparation and prior to commencement of the installation of the 300 mm thick levelling pad. The next operation after the completion of the foundation excavation and preparation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

A Request to Proceed shall be submitted to the Contract Administrator upon completion of the 300 mm thick levelling pad and prior to placement of the expanded polystyrene blocks. The next operation after completion of the levelling pad shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

A Request to Proceed shall be submitted to the Contract Administrator upon completion of the placement of the expanded polystyrene blocks and prior to commencement of the placement of the polyethylene sheeting and reinforced concrete top slab. The next operation after the completion of the placement of the expanded polystyrene blocks shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

A Request to Proceed shall be submitted to the Contract Administrator upon completion of the placement of the polyethylene sheeting and reinforced concrete top slab and prior to commencement of the placement of the subbase material and side slope cover. The next operation after the completion of the placement of the polyethylene sheeting and reinforced concrete top slab shall not proceed until a Notice to Proceed has been received from the Contract Administrator.



## **7.0 MATERIALS**

### **7.1 Granular Levelling Pad**

The levelling pad shall consist of a Granular 'A' material with gradation and physical requirements as specified in OPSS.PROV 1010.

### **7.2 Rigid Expanded Polystyrene**

#### **7.2.1 General**

##### **7.2.1.1 The Contractor shall submit:**

- 1) A general statement as to the type, composition, and method of production of the material.
- 2) The manufacturer's name, address, email address, phone number, identification of a contact person and description of background experience in the manufacturing of the rigid expanded polystyrene.
- 3) An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
- 4) The physical and mechanical properties of the rigid expanded polystyrene including:
  1. Geometry
  2. Nominal Density
  3. Compressive Strength
  4. Flexural Strength
  5. Thermal Resistance
  6. Dimensional Stability
  7. Flammability
  8. Water Absorption
- 5) Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
- 6) A sample of the expanded polystyrene material to the Contract Administrator for review.

##### **7.2.1.2 Production Lots**

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

#### **7.2.2 Detail Requirements**

Requirements shall be as shown in Table 1 and as described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear Dimensions - Flatness - Squareness	mm (min)	1200 x 600 x 300 $\pm$ 1% 10 mm in 3 m $\pm$ 0.5%	--
Nominal Density	kg/m <sup>3</sup> (max)	50	--
Compressive Strength at 5% Deformation	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m <sup>2</sup> .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

### 7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm. The maximum deviation from the specified linear dimensions shall be  $\pm$  1%.

The flatness of the block faces shall be within  $\pm$  10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner-to-corner dimensions (squareness) shall be 0.5%.

The thickness shall be within –3 mm to +5 mm.

### 7.2.2.2 Nominal Density

The maximum nominal density of the expanded polystyrene shall be 50 kg/m<sup>3</sup> (maximum).

### 7.2.2.3 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 115 kPa (minimum) at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% deformation.

### 7.2.2.4 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, Method 1, Procedure B.2.7.4 Dimensional Stability.

### 7.2.2.5 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

#### **7.2.2.6 Thermal Resistance**

The thermal resistance shall be  $0.7 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$  for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25 \text{ mm}} = \frac{R}{\text{thickness (mm)}} \cdot 25\text{mm}$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

#### **7.2.2.7 Flammability**

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC-S102-10 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863.

#### **7.2.2.8 Water Absorption**

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

#### **7.2.2.9 Chemical Resistance**

The expanded polystyrene shall be resistant to common inorganic acids and alkalis. A table identifying the chemical resistance as either resistant limited or not resistant shall be submitted.

#### **7.2.2.10 Biological Resistance**

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

#### **7.2.2.11 Environmental**

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

### **7.3 Polyethylene Sheeting**

The plastic sheeting encapsulating the expanded polystyrene block mass shall be 6 mil polyethylene sheeting or better if specified elsewhere in the Contract Package.

### **7.4 Concrete Top Slab**

The concrete top slab shall consist of 30 MPa reinforced concrete as shown on the Contract Drawings.

## **8.0 DELIVERY, STORAGE AND HANDLING**

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

## **9.0 CONSTRUCTION**

### **9.1 Foundation Excavation**

Foundation excavation shall be carried out to the design elevations shown on the Contract Drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

### **9.2 Leveling Pad**

Place, level and compact a layer of Granular 'A' material in accordance with OPSS.PROV 501 to within  $\pm 30$  mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground. The levelling pad must be placed in-the-dry.

### **9.3 Installation of Blocks**

- 1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
- 2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
- 3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
- 4) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- 5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
- 6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
- 7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contract Administrator for review.

- 8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
- 9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- 10) The top surface, side surfaces and base of the expanded polystyrene shall be covered with 6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

#### **9.4 Concrete Top Slab**

The concrete top slab shall be poured after the polyethylene sheeting is fixed in place. Place 125 mm thick layer of concrete in accordance with OPSS.PROV 904 to within  $\pm 30$  mm of the design elevation.

#### **9.5 Side Slope Cover**

The side slopes of the rigid expanded polystyrene embankment shall be covered with granular fill as detailed elsewhere in the Contract drawings.

### **10.0 EQUIPMENT**

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

### **11.0 QUALITY ASSURANCE**

#### **11.1 General**

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. A recognized testing laboratory accredited by the Standards Council of Canada shall conduct the testing.

#### **11.2 Sampling Frequency**

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three (3) blocks shall be tested.

#### **11.3 Acceptance or Rejection**

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Culling of the rejected material by the Contractor shall be permitted and a proposal for retesting the remaining material shall be submitted to the Contract Administrator to demonstrate conformance with the specification. Retesting of material remaining from culling rejected material shall be at no cost to the Owner.

Replacement of the blocks shall be at the Contractor's expense.

## **12.0 MEASUREMENT FOR PAYMENT**

### **12.1 Actual Measurement**

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

## **13.0 PAYMENT**

### **13.1 Basis of Payment**

The concrete top slab and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

**DRILLED SHAFTS (CAISSON PILES) - Item No.**  
**SUPPLY EQUIPMENT FOR INSTALLING DRILLED SHAFTS - Item No.**  
**DRILLED SHAFTS – 900 mm and 1500mm DIAMETER - Item No.**  
**SHAFT INSPECTION - Item No.**  
**CROSS-HOLE SONIC LOGGING (CSL) TESTING - Item No.**

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Non-Standard Special Provision

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## **1.0 SCOPE**

This specification covers the requirements for the supply and installation of cast-in-place concrete drilled shaft (caisson pile) deep foundation units for the following structures:

- Highway 401 Underpass at Fraser Road (Site 31-230)

### **1.01 Specification Significance and Use**

This specification is written as a provincial-oriented specification. Provincial-oriented specifications are developed to reflect the administration, testing, and payment policies, procedures, and practices of the Ontario Ministry of Transportation.

Use of this specification or any other specification shall be according to the Contract Documents.

## **2.0 REFERENCES**

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 to the OPSS listed, unless use of a municipal-oriented specification is specified in the Contract Documents.

This specification refers to the following specifications, standards, or publications:

### **Ontario Provincial Standard Specifications, Construction**

OPSS.PROV 904	Concrete Structures
OPSS.PROV 905	Steel Reinforcement for Concrete
OPSS.PROV 909	Prestressed Concrete - Precast members
OPSS.PROV 911	Coating Structural Steel Systems

### **Ontario Provincial Standard Specifications, Material**

OPSS.PROV 1350	Concrete - Materials and Production
OPSS.PROV 1440	Steel Reinforcement for Concrete

## **CSA Standards**

G40.20-04/G40.21-04 (R2009)	General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Steel
W47.1-03 (R2008)	Certification of Companies for Fusion Welding of Steel
W48-06	Filler Materials and Allied Materials for Shielded Metal Arc Welding
W59-03(R2008)	Welded Steel Construction (Metal Arc Welding)
W178.1-08	Certification of Welding Inspection Organizations
W178.2-08	Certification of Welding Inspectors

## **Canadian General Standards Board (CGSB)**

48.9712-2006	Non-destructive Testing, Qualification and Certification of Personnel
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## **ASTM International**

A 252-98(2007)	Welded and Seamless Steel Pipe Piles
A 328/A 328M-07	Steel Sheet Piling

## **American Petroleum Institute (API)**

API 13A	Drilling Fluid Materials, 19 <sup>th</sup> Edition, 10.00.08
RP 13B-1	Standard Procedure for Field Testing Water Based Drilling Fluids, 5 <sup>th</sup> Edition,

## **Steel Structures Painting Council (SSPC)**

SP10/NACE No.2-Jan. 1, 2001	Near-White Blast Cleaning
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## **International Organization for Standardization/International Electrotechnical Commission (ISO/IEC)**

17025	General Requirements for the Competence of the Testing and Calibration Laboratories
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## **3.0 DEFINITIONS**

For the purpose of this specification, the following definitions apply:

**Bedrock** means a natural solid bed of the hard, stable, cemented part of the earth's crust, igneous, metamorphic, or sedimentary in origin that may or may not be weathered.

**Casing** means open ended enclosing cylindrical steel tubing or pipe permanently installed in the ground. Casings are structurally required and can be used to stabilize an excavated hole.

**Crosshole Sonic Logging (CSL)** is a non-destructive testing method to measure the structural integrity of drilled shafts and other concrete piles by means of measuring energy and waveform generated by a signal emitter. The method is used to determine the structural soundness of concrete within the steel reinforcement cage, facilitated by the installation of hollow tubes bundled to the interior of the rebar cage.



**Deep Foundation Unit** means a structural member, driven or otherwise, installed in the ground to transfer the loads from a structure to soil or rock and derives supporting resistance from the surrounding soil or rock or from the soil or rock strata below its tip or a combination of both.

**Drilled Shaft or Caisson Pile** means a cast in place deep foundation unit with or without an enclosing liner formed by placing concrete in a bored or excavated hole.

**Drilled Shaft or Caisson Pile Cap** means a footing or some other structural component used to transfer the load to the caisson piles as well as maintaining them in position.

**Liner** means open ended enclosing steel tubing or pipe temporarily installed to facilitate the construction of drilled shafts or caisson piles.

**Obstruction** means a material and/or objects that cannot be removed from a shaft during normal excavation operations with the drilling equipment adequate to excavate earth materials found on the project, and which necessitate the use of other method and/or equipment to remove. Such obstructions may be rock fragments, boulders, waterlogged timbers, or any material, natural or man-made which requires use of special tools or procedures not otherwise required for excavation of rock or earth materials on the project.

**Pile Integrity Test (PIT) or Low Strain Impact Integrity Test** is a non-destructive testing method to measure the structural integrity of drilled shafts and other concrete piles by means of transient dynamic response. It is a simple and rapid test method to determine the uniformity of concrete within the drilled shaft but is less accurate than other types of testing for drilled shafts.

**Pumped Concrete** means a method of transporting concrete through hose or pipe by means of positive and continuous pressure.

**Slurry** means a drilling fluid, consisting of water or water mixed with one or more of various solids or polymers, used to maintain the stability of the side walls and bottom of an excavation.

**Tremie** means a hopper with a vertical pipe used for placing concrete under water. The foot of the pipe is always submerged in concrete except during commencement of concreting and the upper level of the concrete in the pipe is always above water level.

#### **4.0 SUBSURFACE CONDITIONS**

The subsurface conditions are described elsewhere in the contract.

1. The installation method and equipment must be capable of dislodging, removing or otherwise penetrating cobbles and boulders in the native soils and/or drilling through bedrock as per Contract Documents.
2. Drilled shafts excavation will extend through water-bearing non-cohesive sand deposits, firm to very stiff cohesive soils, till materials containing cobbles and boulders, and into the limestone bedrock. The bedrock consists of medium strong to strong limestone layers. Equipment supplied to advance the drilled shafts must be able to penetrate these materials to advance each drilled shaft into bedrock and form the required socket. Details of the bedrock are provided elsewhere in the Contract.

3. Drilled shafts will extend through soft to firm cohesive soils. Equipment supplied must be able to support the excavation walls in the cohesive overburden soils.
4. Drilled shafts will extend through non-cohesive overburden soils below the groundwater level. The selected installation methods and equipment must be able to support the excavation walls in the non-cohesive overburden soils and highly weathered portion of the bedrock and prevent materials from falling into the socket.

## **5.0 DESIGN AND SUBMISSION REQUIREMENTS**

### **5.01 Design Requirements**

#### **5.01.01 Concrete**

The Contractor is responsible for providing concrete with suitable characteristics for installation. The concrete shall be flow able, non-segregating concrete that does not exhibit rapid slump loss. The concrete mix shall satisfy the requirements specified herein.

### **5.02 Submission Requirements**

#### **5.02.01 General**

All submissions shall bear the seal and signature of an Engineer experienced in the field of deep foundations. All submissions shall be submitted to the Contract Administrator as specified in the Contract Documents. In lieu of any specified timeline in the Contract Documents, all submissions shall be submitted 30 days prior to construction.

When welded field splices are used, welding procedures according to the Canadian Welding Bureau shall be submitted to the Contract Administrator.

#### **5.02.01.01 Casing**

If the use of casing is applicable to the project, the Contractor is responsible for providing casing of sufficient size and strength to facilitate the excavation whilst maintaining sidewall stability.

#### **5.02.02 Preconstruction Survey**

If required by the Contract Documents, a condition survey of property and structures that may be affected by the work shall be submitted to the Contract Administrator prior to commencing the work. The survey shall be conducted in accordance with the Contract Documents as specified and include the locations and conditions of adjacent properties; buildings; underground structures; above ground and underground utilities; and structures, such as walls abutting the site and along rail corridor.

#### **5.02.03 Materials**

##### **5.02.03.01 Mill Certificates**

One copy of the mill certificates, indicating that the steel meets the requirements for the appropriate standards for casings shall be submitted to the Contract Administrator at the time of delivery.

Where mill test certificates originate from a mill outside Canada or the United States of America, the information on the mill certificates shall be verified by testing by a Canadian laboratory. The laboratory shall be certified by an organization accredited by the Standards Council of Canada to comply with the requirements of ISO/IEC 17025 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date (i.e., yyyy-mm-dd), and the signature of an authorized officer of the Canadian testing laboratory.

#### **5.02.03.02 Concrete**

Submissions of concrete mix shall follow OPSS.PROV 1350 requirement.

#### **5.02.04 Installation**

##### **5.02.04.01 Drilled Shaft Pre-Construction**

The drilled shaft pre-construction submittal shall be comprised of the following four components:

- a) construction experience
- b) shaft installation work plan
- c) shaft slurry technical assistance (if applicable) and
- d) non-destructive QC testing personnel.

##### **5.02.04.01.01 Construction Experience**

The Contractor's experience and qualifications in the construction of drilled shaft shall include at least three separate drilled shaft projects with:

- Ground conditions similar to those as specified in the Contract Documents.
- Drilled shaft diameters and depths similar or larger to those as specified in the Contract Documents.

The on-site drilled shaft supervisors shall have a minimum 10 years experience in supervising construction of drilled shafts of similar size (diameter and depth), scope and subsurface conditions to those as specified in the Contract Documents. Work experience shall be direct supervisory responsibility for the on-site drilled shaft construction operations. Project management level positions indirectly supervising on-site shaft construction operations are not acceptable for this experience requirement.

The drill rig operators shall have a minimum of five years experience in construction of drilled shaft foundations.

A Request to Proceed with the work of drilled shaft pre-construction shall be submitted to the Contract Administrator with:

- A project reference list for the Contractor's experience and qualifications, and
- Individual's experience lists for the on-site supervisors and drill rig operators assigned to the work.

The project reference list shall contain a description of each listed project with the name and current phone number of the projects' owner(s) or the owner's Contractor(s).

The individual's experience lists shall be limited to a single page for each supervisor or operator and contain a description of the on-site experience in drilled shaft excavation operations and placement of assembled steel reinforcing bar cages and concrete in shafts.

The drilled shaft installation shall not proceed until a Notice of Proceed has been received from the Contract Administrator.

#### **5.02.04.01.02                      Drilled Shaft Installation Work Plan**

The Contractor shall submit a drilled shaft installation Work Plan to the Contract Administrator at least 4 weeks prior to the start of drilled shaft installation. In preparing the Work Plan, the Contractor shall reference the available subsurface information presented elsewhere in the contract. This Work Plan shall provide at least the following information:

- a) Proposed overall construction operation schedule and sequence.
- b) Means of access to the drilling site and details of concrete delivery to site. Description, size, and capacities of proposed equipment, including but not limited to, cranes, drills, auger, coring equipment to get through obstructions or hard rock, bailing buckets, final cleaning equipment, and drilling unit. The Work Plan shall describe why the equipment was selected and describe equipment suitability to the anticipated site conditions and work methods. The Work Plan shall include a project history of the drilling equipment demonstrating the successful use of the equipment on shafts of equal or greater size in similar soil/rock conditions. The Work Plan shall also include details of shaft excavation and cleanout methods.
- c) Details of the method(s) to be used to ensure shaft stability (i.e., prevention of caving, bottom heave, using temporary casing, slurry, or other means) during excavation (including pauses and stoppages during excavation) and concrete placement, placement of temporary and permanent casings and removal of temporary casings. If casings are required, casing dimensions and detailed procedures for installation shall be provided.
- d) Details of casings to be used, including calculations showing that the casing can withstand stresses due to installation without undue deformation. Details shall include methods for casing handling, splicing, straightening and out-of-round correction.
- e) A slurry mix design, including all additives and their specific purpose in the slurry mix, with a discussion of its suitability to the anticipated subsurface conditions, shall be submitted and include the procedures for mixing, using, and maintaining the slurry.
- f) A detailed plan for quality control of the selected slurry, including tests to be performed, test methods to be used, and minimum and/or maximum property requirements which must be met to ensure the slurry functions as intended, considering the anticipated subsurface conditions and shaft construction methods, in accordance with the slurry manufacturer's recommendations and this project special provision shall be included. As a minimum, the slurry quality control plan shall include the tests specified in Sections 6.07.01, 6.07.02 and 6.07.03.
- g) Description of an emergency construction joint method.
- h) Methods for dewatering of the site as necessary.
- i) Description of the method used to fill or eliminate all voids below the top of shaft between the plan shaft diameter and excavated shaft diameter when permanent casing is specified.
- j) The proposed concrete mix to be used.
- k) Details of concrete placement, including proposed operational procedures for pumping methods,

and a sample uniform yield form to be used by the Contractor for plotting the approximate volume of concrete placed versus the depth of shaft for all shaft concrete placement (except concrete placement in the dry).

- l) Methods to prevent and handle delays in concrete batching and delivery to site.
- m) When shafts are constructed in water, the submittal shall include seal thickness calculations, seal placement procedure, and descriptions of provisions for casing, shoring, and dewatering.
- n) Description and details of the containment, storage and disposal plan for excavated material and drilling slurry (if applicable).
- o) A contingency plan for containment and clean-up of any spill or discharge of material which might contaminate public waters. The plan shall address the plan for regular day-to-day operations and for unplanned emergency situations.
- p) Reinforcing steel shop drawings with details of reinforcement placement, including bracing, centering, and lifting methods, and the method to ensure the reinforcing cage position is maintained during construction, including use of bar boots and/or rebar cage base plates, and including placement of rock backfill below the bottom of shaft elevation.
- q) Contingency plan to remedy sinking of the reinforcing cage into concrete.

The reinforcing steel shop drawings and shaft installation plan shall include, at a minimum:

- a) Procedure and sequence of steel reinforcing bar cage assembly.
- b) The tie pattern, tie types, and tie wire gages for all ties on permanent reinforcing and temporary bracing.
- c) Number and location of primary handling steel reinforcing bars used during lifting operations.
- d) Type and location of all steel reinforcing bar splices.
- e) Details and orientation of all internal cross-bracing, including a description of connections to the steel reinforcing bar cage.
- f) Description of how temporary bracing is to be removed.
- g) Location of support points during transportation.
- h) Cage weight and location of the center of gravity.
- i) Number and location of pick points used for lifting for installation and for transport (if assembled off-site).
- j) Crane charts and a description and/or catalog cuts for all spreaders, blocks, sheaves, and chockers used to equalize or control lifting loads.
- k) The sequence and minimum inclination angle at which intermediate belly rigging lines (if used) are released.
- l) Pick point loads at 0, 45, 60, and 90 degrees and at all intermediate stages of inclination where rigging lines are engaged or slackened.
- m) Methods and temporary supports required for cage splicing.
- n) For picks involving multiple cranes, the relative locations of the boom tips at various stages of lifting, along with corresponding net horizontal forces imposed on each crane.
- o) A description of spacers and supports to be used for the reinforcement.

The Contract Administrator will evaluate the shaft installation Work Plan for conformance with the Drawings, Specifications, and project special provisions, within the review time specified. If deemed necessary by the Contract Administrator, a Shaft Installation Work Plan Submittal Meeting will be scheduled by the Contract Administrator.

**5.02.04.01.03****Slurry Methodology**

If slurry other than water slurry is used to construct the shafts, the Contractor shall provide or arrange for technical assistance in the use of the slurry. The Contractor shall submit the following to the Contract Administrator:

- a) The name and current phone number of the slurry manufacturer's technical representative assigned to the project, and the frequency of scheduled visits to the project site by the slurry manufacturer's representative.
- b) The name(s) of the Contractor's personnel assigned to the project and trained by the slurry manufacturer in the proper use of the slurry. The submittal shall include a signed training letter from the slurry manufacturer for each trained Contractor's employee listed, including the date of the training.

The following shall be submitted:

- a) The type, source, and physical and chemical properties of the bentonite (mineral) or polymer (synthetic) slurry.
- b) The source of water.
- c) Method of mixing slurry.
- d) The water solids ratio and the mass and volumes of the constituent parts, including any chemical admixtures or physical treatment employed to produce slurry with the required physical properties.
- e) Details of procedure to be used for monitoring the quality of the slurry.
- f) A test report showing the properties of the slurry and certifying that the slurry meets the requirements of API RP 13B-1.
- g) Method of disposal of the slurry.

**5.02.04.01.04****Cage Lift**

The Contractor is responsible for providing proper lift procedure for rebar cage. Contractor shall submit a proposed procedure the Contract Administrator at least 4 weeks prior to the start of drilled shaft installation.

**5.03****Drilled Shaft Pre-Construction Meeting**

A shaft preconstruction meeting shall be held at least 14 working days prior to the Contractor beginning any shaft construction work at the site to discuss construction procedures, personnel, and equipment to be used, and other elements of the approved shaft installation narrative. As a minimum the following shall represent the Contractor at the meeting:

- a) Project Manager
- b) Project Engineer
- c) Project Superintendent
- d) On site supervisors, and all foremen in charge of excavating the shaft, placing the casing and slurry as applicable, placing the steel reinforcing bars and placing the concrete.
- e) If slurry is used to construct the shafts, the slurry manufacturer's representative or approved Contractor's employees trained in the use of the slurry shall also attend.

#### **5.04 Acceptance of Submissions**

The Contract Administrator will review the Submissions for the purpose of verifying compliance to contract requirements, within 7 calendar days after the Pre-Construction Meeting and provide written comments if changes are necessary to meet Contract requirements. The Contractor shall submit to the Contract Administrator a final installation plan which meets all Contract requirements within 7 calendar days.

If revisions in the previously reviewed Work Plans are required to accommodate site conditions, or for other reasons, the Contractor shall submit the revised Work Plans to the Contract Administrator prior to implementation. The proposed final shaft installation work plan shall be submitted to the Contract Administrator with a Request to Proceed. The Contractor shall not proceed with the shaft installation work plan until a Notice to Proceed is given by the Contract Administrator.

The Contract Administrator's approval of the installation plan does not relieve the Contractor of full responsibility for the safe and successful completion of construction of the drilled shafts.

### **6.0 MATERIALS**

#### **6.01 Casing or Liner for Drilled Shafts**

##### **6.01.01 General**

Casings shall be according to ASTM A36, ASTM A 252, Grade 2 or 3, ASTM A572, or ASTM A588.

Casings shall be continuous wherever possible or practical. Casings shall be installed as per the Contract requirements. Casing shall be installed to stabilize the shaft excavation against collapse.

If welded, casing shall be welded by the electric arc method according to CSA W59.

The casing wall thickness specified is the minimum that shall be supplied.

Steel casings and liners shall conform to a straightness tolerance of 1.5 mm maximum per meter of length.

The casings must be of ample strength to withstand handling stresses, driving (installation) stresses, internal pressure of fluid concrete, external pressure of surrounding earth and water, and be watertight.

Where drilled shafts are located in open water areas, casings shall be extended with due consideration of risk from fluctuating water levels and flood events to the specified bottom of casing elevation to protect the shaft concrete from water action during placement and curing of concrete unless otherwise specified in the contract documents.

##### **6.01.02 Permanent Casing**

For permanent casing, the outside surface of the casing shall be smooth to not over cut soil during casing advancement (i.e., driving shoe should not be installed on the outside).

Casings shall be non-corrugated, smooth, clean, and watertight and free of hardened concrete. Casings shall be protected from corrosion during construction.

Inspection of welds will be of a visual nature on 30% of the welds. If the sample welds do not pass the visual inspection and need to be repaired, the visual inspection by the Contract Administrator may be increased up to 100% of the welds.

If evidence indicating poor welding is found, radiographic or ultrasonic testing shall be carried out by the Contract Administrator using procedures according to CSA W59 on 10% of the welds.

All welds that have been repaired shall be visually inspected and shall undergo non-destructive testing performed by the Contract Administrator.

### **6.01.03            Temporary Casing or Liner**

Temporary casing or liner is defined as casing installed to facilitate shaft construction only, which is not designed as part of the shaft structure, and which shall be completely removed after shaft construction is complete unless otherwise shown on the Contract Drawings. All temporary casing shall be of ample strength to resist damage and deformation from transportation and handling, installation and extraction stresses, and all pressures and forces acting on the casing. The casing shall be capable of being removed without deforming and causing damage to the completed shaft and without disturbing the surrounding soil.

### **6.02                Steel Reinforcement**

Steel reinforcement shall be according to OPSS.PROV 1440 unless otherwise specified in the project specifications or drawings.

### **6.03                Concrete**

#### **6.03.01            General**

Concrete shall be according to OPSS.PROV 1350 and CSA A23.1-19. Concrete shall also comply with the additional requirements specified in Tables 1.1.1 and 1.1.2 below:



**Table 1.1.1**  
**Concrete for Tremie Placement Method (Wet Excavation)**

Property	Test	Test Procedure	Specified Value
Workability	Slump	CSA A23.1-19C	190+/-40 mm Stability of concrete shall be assessed, as mixes with such high slump would be prone to segregation and bleeding
Workability Retention	-	-	Minimum of slump flow of 350 mm at the end of concrete placement (including removal of temporary casing, if necessity)
Maximum Coarse Aggregate Size	-	-	19 mm or not more than one quarter of the reinforcement clear spacing, whichever is smaller
Maximum Water/Cement Ratio	-	-	0.40

**Table 1.1.2.**  
**Concrete for Free Fall Placement Method (Dry Excavation)**

Property	Test	Test Procedure	Specified Value
Workability	Slump	CSA A23.2-C	150 to 190 mm
Workability Retention	-	-	Minimum slump of 130 mm at the end of temporary casing removal (if applicable)
Maximum Water/Cement Ratio	-	-	0.45

#### **6.03.02 Concrete Making Materials:**

Concrete making materials shall be according to Section 1350.05 of OPSS.PROV 1350, CSA A3000 and CSA A23.1-19.

#### **6.04 Reinforcing bar Spacers and Support Devices**

Rebar spacers, centralizers and other support devices shall be according to OPSS.PROV 905.

#### **6.05 Crosshole Sonic Logging (CSL) Access Tubes and Caps**

Crosshole Sonic Logging (CSL) access tubes shall be round steel pipe with a minimum inside diameter of 38 mm (the inside diameter should be enough to allow the easy passage of the ultrasonic probes over the entire length of the access tube). The access tube shall be watertight with clean internal and external faces to ensure good bond between the concrete and the access tube. PVC access tubes are not allowed, unless approved by the Contract Administrator.

The access tubes shall be fitted with watertight threaded steel or PVC caps on the bottom and top. The access tubes shall be filled with water prior to the start of concrete placement.

#### **6.06 Grout for filling CSL Access Tubes**

Grout for filling CSL Access Tubes at the completion of the cross sonic logging shall be a homogeneous mixture of neat cement and potable water with the maximum water/cement ratio of 0.45. The grout mix design shall be approved by the Contract Administrator.

#### **6.07 Slurry**

Bentonite (mineral) slurry shall be according to API Spec 13A.

Polymer (synthetic) slurry shall be according to Guide to Support Fluids for Deep Foundations, First Edition EFFF and DFIEFFC/DFI Support Fluids Task Group.

The slurry shall consist of a stable colloidal suspension of pulverized solids or polymers thoroughly mixed with water.

Drilling slurry will be defined as water, bentonite, polymer slurry formed during the drilling process, or other fluids used to maintain stability of the drilled shaft excavation to aid in the drilling process or to maintain the quality of the shaft excavation. In addition, the term polymer slurry will be defined as the final mixed composite of all additives, including polymer slurry additives required to produce the acceptable drilling slurry.

Bentonite drilling or other mineral slurry shall not be used in shaft excavation at the Metrolinx Overhead. Bentonite drilling slurry shall not be used in shaft excavations at the Leslie Street Overpass and Don River bridge, unless approved by the Contract Administrator.

A slurry manufacturer representative shall be onsite for the first application of slurry and can be onsite as requested by the Contractor on subsequent applications. Drilling slurry, when used, will be non-compensable and effect on time of performance due to the use of the slurry will be non-excusable.

The material used to make the slurry shall not be detrimental to the concrete or surrounding ground strata. Polymer slurries shall have appropriate viscosity and gel characteristics to transport excavated material to suitable screening systems or settling tanks. The percentage and specific gravity of the material used to make the slurry shall be sufficient to maintain the stability of the excavation and to allow proper concrete placement. The entire fluid column shall be replaced with fresh slurry after drilling and during final clean-out with an airlift or other approved method; a clean-out bucket is not sufficient for final cleanout.

Prior to introduction into the shaft excavation, the manufactured polymer slurry admixture shall be pre-mixed thoroughly with clean, fresh water and for adequate time in accordance with the slurry admixture manufacturer's recommendations allotted for hydration. Water used for mixing shall be potable. Slurry tanks of adequate capacity will be required for slurry mixing, circulation, storage and treatment. No excavated slurry pits will be allowed in lieu of slurry tanks. Adequate equipment will be required as necessary to control slurry properties during the drilled shaft excavation in accordance with the values provided in the table below.

##### **6.07.01 Water Slurry**

Water without site soils or soil additive can be used as slurry when casing is used for the entire length of hole. Clean water may be used as a drilling fluid when entire length of the shaft excavation is cased. Water slurry shall conform to the following requirements:

Property	Test Procedure	Specified Value
Density	Mud Weight (Density) API 13B-1, Section 1	1040 (kg/m <sup>3</sup> ) Maximum
Sand Content	Sand API 13B-1, Section 5	1.0 (%) Maximum
Temperature (prior to concrete placement)	-	5.0 (°C) Minimum

#### 6.07.02 Polymer (Synthetic) Slurry

Polymer slurry shall be used as per manufacturers recommendations and shall conform to the following requirements:

Property	Test Procedure	Specified Value
Density	Mud Weight (Density) API 13B-1, Section 1	1040 (kg/m <sup>3</sup> ) Maximum
Sand Content	Sand API 13B-1, Section 5	1.0 (%) Maximum
Temperature (prior to concrete placement)	-	5.0 (°C) Minimum
Viscosity	Marsh Funnel and Cup API 13b-1, Section 2.2	32 to 135
pH	Glass Electrode, pH Meter, or pH Paper	8 to 10

#### 6.07.03 Bentonite (Mineral) Slurry

The use of bentonite slurry is not permitted for shaft excavation at the Metrolinx Overhead. Use of bentonite slurry for the shaft excavation for the Leslie Street Overpass and Don River bridge is not permitted unless approved by the Contract Administrator. If approved for use, bentonite slurry shall conform to the following requirements:

Property	Test Procedure	Specified Value
Density	Mud Weight (Density) API 13B-1, Section 1	1010 (kg/m <sup>3</sup> ) to 1200 (kg/m <sup>3</sup> )
Sand Content (prior to final cleaning and immediately prior to placing concrete)	Sand API 13B-1, Section 5	4.0 (%) Maximum

Temperature (prior to concrete placement)	-	5.0 (°C) Minimum
Viscosity	Marsh Funnel and Cup API 13b-1, Section 2.2	26 to 50
pH	Glass Electrode, pH Meter, or pH Paper	8 to 11

## **7.0 EQUIPMENT**

### **7.01 Drilling and Excavation Equipment**

Drilling equipment used to perform the drilled shaft work shall have the capability of providing sufficient torque and down-thrust for drilling and excavating shafts. Appropriate drilling and coring equipment must be available to drill through obstructions and bedrock and harder interbeds in bedrock.

The excavation equipment shall be capable of excavating the drilled shaft to the dimensions required in the plan with a level bottom. The cutting edges of the excavation tools used to form the base of the drilled shaft must be normal to the vertical axis of the equipment within a tolerance of ( $\pm 13$ mm) per (305 mm) of shaft diameter.

### **7.02 Concrete Placement Equipment**

Tremie pipe to place concrete underwater shall be completely watertight and of sufficient length, weight, and diameter to discharge concrete at the shaft base elevation. The tremie must not contain aluminum parts that will have contact with the concrete. The tremie inside diameter must not be less than 250 mm for an open system or 125 mm for a closed system. The inside and outside surfaces of the tremie must be clean and smooth to permit both flow of concrete and unimpeded withdrawal during concrete placement. The discharge end of the tremie must be constructed to permit the free radial flow of concrete. Wall thickness of the tremie must be adequate to prevent crimping or sharp bends that may restrict concrete placement.

A plug shall be placed at the top of the tremie or pump line to separate the concrete from the water/slurry until the concrete is flowing through the orifice. Plugs, if left in the shaft concrete, must be of a material approved by the Contract Administrator. Tremie pipe sections must have connections that will not loosen and separate and remain watertight if a portion of the tremie becomes stuck.

## **8.0 CONSTRUCTION**

### **8.01 Transporting, Storing, and Handling Piles, Casings, Liners, and Reinforcing Steel Reinforcement Cages**

#### **8.01.01 General**

Casings, liners, and steel reinforcement shall be transported, stored, and handled in such a manner that damage is prevented and the strength of the components is not affected by deterioration or deformation.

Components shall be lifted and placed using appropriate lifting equipment, temporary bracing, guys, or stiffening devices so that the components are at no time overloaded, unstable, or unsafe.

Material shall be supported to prevent unequal settlement when stacked.

#### **8.01.02                      Drilled Shaft Casings and Liners**

Casings and liners shall be handled and stored in such a manner to avoid damage or distortion to them. The casings and liners shall be maintained circular within  $\pm 2\%$  of the casing or liner diameter.

### **8.02                              Shaft Excavation**

#### **8.02.01                      General**

The Contractor shall submit Requests to Proceed prior to construction and at the milestones specified. Construction of drilled shafts shall commence only after Notices to Proceed have been given by the Contract Administrator.

Shafts shall be excavated to the required depth as shown on the Contract Drawings. Shaft excavation operations shall conform to this section and the shaft installation Work Plan.

#### **8.02.02                      Continuity of Shaft Excavation Operations**

Once the excavation operation has been started, the excavation shall be conducted in a continuous operation until the excavation of the shaft is completed, except for pauses and stops as noted, using approved equipment capable of excavating through the type of material expected. Pauses during the excavation operation, except for casing splicing, tooling changes, slurry maintenance, and removal of obstructions, are not allowed.

Pauses, defined as momentary interruptions of the excavation operation, will be allowed only for casing splicing, tooling changes, slurry maintenance, and removal of obstructions. Shaft excavation operation interruptions not conforming to this definition shall be considered stops. Stops for uncased excavations (including partially cased excavations) shall not exceed 16 hours duration. Stops for fully cased excavations, excavations in rock, and excavations with casing seated into rock, shall not exceed 48 hours duration unless approved by the Contract Administrator.

For stops exceeding the time durations specified above, the Contractor shall stabilize the excavation using the following method:

For both a cased and uncased excavation, backfill the hole with either Lean Concrete or granular material. The Contractor shall backfill the hole to the ground surface, if the excavation is not cased, or to a minimum of 1.5 m above the bottom of casing (temporary or permanent), if the excavation is cased. Backfilling of shafts with casing fully seated into rock, as determined by the Contract Administrator, will not be required.

During stops, the Contractor shall protect the base of the shaft from weathering and stabilize the shaft excavation to prevent bottom heave, caving, head loss, and loss of ground. The Contractor bears full responsibility for selection and execution of the method(s) of stabilizing and maintaining the shaft excavation. Shaft stabilization shall conform to the shaft installation Work Plan.

If slurry is present in the shaft excavation, the Contractor shall conform to the requirements of OPSS.PROV 903.07.05. regarding the maintenance of the slurry and the minimum level of drilling slurry

throughout the stoppage of the shaft excavation operation and shall recondition the slurry to the required slurry properties prior to recommencing shaft excavation operations.

Once the excavation of the rock socket reaches the target depth, over-ream the shaft side walls, prior to placement of the reinforcing cage. The duration from the time of base inspection to the start of concreting shall not exceed 6 hours.

Rock socket side walls shall be roughened if specified on the Contract Drawings.

## **8.02.03                      General Shaft Casing or Liner Requirements**

### **8.02.03.01                  General**

Shaft casing or liner shall be watertight and clean prior to placement in the excavation. The outside diameter of the casing shall not be less than the specified diameter of the shaft. The diameter of the casing shall not be greater than the specified diameter of the shaft plus 150 mm.

The Contractor shall conduct casing installation and removal operations and shaft excavation operations such that the adjacent soil outside the casing and shaft excavation for the full height of the shaft is not disturbed. Disturbed soil is defined as soil whose geotechnical properties have been changed from those of the original in situ soil, and whose altered condition adversely affects the capacity and structural integrity of the shaft foundation.

### **8.02.03.02                  Permanent Shaft Casing**

Permanent casing is defined as casing designed as part of the shaft structure and installed to remain in place after construction is complete. All permanent casing shall be of ample strength to resist damage and deformation from transportation and handling, installation stresses, and all pressures and forces acting on the casing. Where the minimum thickness of permanent casing is specified in the Contract Drawings, it is specified to satisfy structural design requirements only. The Contractor shall increase the casing thickness as necessary to satisfy the requirements of this section.

The outside surface of the casing should be smooth, so it does not overcut soil during advancement (creating a void behind casing). Should the void between casing and a wall of shaft excavation occur, the void shall be filled with a material which approximates the geotechnical properties of the in-situ soils, in accordance with the shaft installation work plan.

The cutting tools and driving shoes of permanent casing shall not overcut the ground and the cutting tools and driving shoes shall be flush with the outside diameter of the casing.

### **8.02.03.03                  Temporary Shaft Casing or Liner**

Temporary casing or liner is defined as casing installed to facilitate shaft construction only, which is not designed as part of the shaft structure, and which shall be completely removed after shaft construction is complete unless otherwise shown on the Contract Drawings. All temporary casing shall be of ample strength to resist damage and deformation from transportation and handling, installation and extraction stresses, and all pressures and forces acting on the casing. The casing shall be capable of being removed without deforming and causing damage to the completed shaft and without disturbing the surrounding soil.

To maintain stable excavations and to facilitate construction, the Contractor may furnish and install temporary casing in addition to the required casing specified on the Contract Drawings. The Contractor

shall provide temporary casing at the site in sufficient quantities to meet the needs of the anticipated construction method.

The Contractor shall use the temporary casing method at all sites where it is inappropriate to use the dry or wet construction methods without the use of temporary casings other than surface casings. In this method, the casing is advanced prior to excavation and withdrawn after concrete placement. In the event seepage conditions prevent use of the dry method, the excavation and concrete placement shall be carried out using wet methods. Wet non-plastic soil shall not be considered as impervious, regardless of permeability.

Where drilling through materials that are susceptible to sloughing, the Contractor shall use appropriate means and method to prevent sidewall and basal instability including but not limited to or a combination of slurry and temporary casing. The Contractor shall take the necessary steps as required to prevent caving during shaft excavation. Should the Contractor select to remove a casing and replace it with a longer casing through caving soils, the excavation shall be backfilled. The Contractor may use soil previously excavated or soil from the site to backfill the excavation. Contractor may use other acceptable methods which will control the size of the excavation and protect the integrity of the foundation soils to excavate through caving layers.

Temporary casing must not be withdrawn until the head of concrete inside the casing is at a sufficient level that the concrete pressure at the bottom of casing exceeds the fluid pressure (e.g., groundwater pressure) on the outside of the casing at all times.

When conditions warrant, the Contractor may pull the casing in partial stages. Before withdrawing the casing, ensure that the level of fresh concrete is at such a level that the fluid trapped behind the casing is displaced upward. As the casing is withdrawn, maintain the level of concrete within the casing so that fluid trapped behind the casing is displaced upward out of the shaft excavation without mixing with or displacing the shaft concrete.

All temporary casing shall be removed. The Contractor shall ensure that permanent casings installed below the shaft cut-off elevation remains in position as a permanent part of the drilled shaft. When casings that are to be removed become bound in the shaft excavation and cannot be practically removed, a proposal shall be submitted to the Contract Administrator for review and acceptance.

If temporary casing is advanced deeper than the minimum top of rock socket elevation shown on the Contract Drawings or actual top of rock elevation if deeper, the Contractor shall withdraw the casing from the rock socket and overream the shaft. If the temporary casing cannot be withdrawn from the rock socket before final cleaning, the rock socket shall be extended below the design tip to maintain a full socket depth. When the shaft extends above ground or through a body of water, the Contractor may form the exposed portion with removable casing except when the Permanent Casing Method is specified. For permanent casings, the Contractor shall remove the portion of metal casings in accordance with the Contract Drawings. The Contractor shall dismantle casings removed to expose the concrete as required above in a manner which will not damage the drilled shaft concrete.

Temporary casing shall be removed gradually as concrete is placed in the shaft. The proposed method of extraction shall be submitted to the Contract Administrator with a Request to Proceed. The Contractor shall not proceed with the extraction until a Notice to Proceed is given by the Contract Administrator.

Contract Administrator may permit movement of the casing by rotating, oscillating or extraction with a vibratory hammer. The extraction method should be coordinated with the Contract Administrator. The Contractor shall extract casing at a slow, uniform rate while the concrete remains fluid.

Expandable or split casings that are removable are not permitted for use below water.

#### **8.02.03.04 Temporary Telescopic Casing**

If permitted by the Contract Administrator, the Contractor shall submit a temporary telescoping casing proposal for drilled shafts with a Request to Proceed to the Contract Administrator, subject to the following conditions:

- a) A maximum of two telescoping casing diameter changes will be allowed.
- b) The maximum diameter change at each casing diameter transition shall be 300 mm.

The Contractor shall not proceed until a Notice to Proceed is given by the Contract Administrator.

#### **8.02.04 Cleaning of Bottom of Shaft Excavation and Inspection**

##### **8.02.04.01 Cleaning**

The Contractor is responsible for cleaning the base of the drilled shafts to comply with the requirements of the specification. Shaft and base cleanliness will be verified by the Contract Administrator.

The Contractor shall use appropriate means such as a cleanout bucket (bailing bucket) and air lift or other devices to clean the bottom of the excavation of all shafts to achieve direct contact between the concrete and undisturbed end bearing formation. The entire slurry column shall be exchanged during final clean-out for wet excavations. A clean-out bucket alone is not sufficient for final clean-out for wet excavations.

The following cleaning criteria must be followed for thickness of sediments at the time of concrete placement:

- a) End Bearing Drilled shafts in Soil: The average thickness of the sediments shall be less than 13 mm. At least 50 percent of the base of each shaft shall have less than 13 mm of sediment. The maximum thickness of sediment at any place on the base of the shaft shall not exceed 25 mm.
- b) End Bearing Drilled shafts in rock: The average thickness of the sediments shall be less than 8 mm. At least 50 percent of the base of each shaft shall have less than 10 mm of sediment. The maximum thickness of sediment at any place on the base of the shaft shall not exceed 20 mm.
- c) Friction shaft without any end bearing: The maximum thickness of sediment at any place on the base of the shaft shall not exceed 50 mm.

##### **8.02.04.02 Inspection**

Each excavated shaft shall be inspected and accepted by the Contract Administrator prior to proceeding with construction. The bottom of each excavated shaft shall be inspected using both Shaft Inspection Device (SID) and Shaft Quantitative Inspection Device (SQUID) (or-an approved alternate down-hole equipment) to verify shaft bottom cleanliness and thickness of debris/sediment prior to concreting as specified in the Contract Documents.



After installation for the rebar cage and immediately before placement of the concrete, the bottom of the shaft shall be sounded with an airlift pipe, a tape with a heavy weight attached to the end of the tape, or other means acceptable by Contract Administrator to determine that the shaft bottom meets the requirements.

The Contractor shall cooperate with the Contract Administrator in using this inspection device, including placing the inspection device in position for inspection and removing it after the inspection. If any of the SID inspections indicate the cleanliness or bearing material requirements are not achieved, reinspection after additional cleaning or drilling will be required at no additional cost.

The Contractor shall submit a request to proceed before placing reinforcing cage and concreting and shall not proceed until a Notice to Proceed is received from the Contract Administrator.

After completion of the inspection of a shaft, the Contract Administrator will direct the Contractor as to whether additional clean-out is necessary.

Both SID and SQUID method of base inspection shall be used for each drilled shaft.

#### **8.02.04.02.01                      Shaft Inspection Device (SID)**

The SID shall be provided and operated by the Contract Administrator. The Contractor shall cooperate with the Contract Administrator in conducting the SID.

The Contractor shall provide a means to position and lower the SID into the shaft excavation to enable the bell housing to rest vertically on the bottom of the excavation. The inspection of each drilled shaft excavation after final cleaning shall be continuously videotaped.

For Contractor's information, the Contract Administrator will furnish a SID device satisfying the following requirements:

- a) A remotely operated, high resolution, color video camera sealed inside a watertight bell housing.
- b) Provides a clear view of the bottom inspection on a video monitor at the surface in real time.
- c) Provides a permanent record of the entire inspection with voice annotation with a resolution of not less than 720 x 480.
- d) Provides a minimum field of vision of 710 cm<sup>2</sup>, with at least two graduated measuring devices to record the thickness of debris/sediment on the bottom of the shaft excavation to a minimum accuracy of 12 mm and a length greater than 37 mm.
- e) Provides sufficient lighting to illuminate the entire field of vision at the bottom of the shaft for the operator and inspector to clearly see the depth measurement scale on the video monitor and to produce a clear recording of the inspection.
- f) Provides a compressed air or gas system to displace drilling fluids from the bell housing and a pressurized water system to assist in determination of bottom sedimentation depth.

For shafts with diameter of up to 2 m, the thickness of debris/sediment will be measured at least in five locations, one in the center of the shaft as well as in the four quadrants surrounding the shaft center. If the diameter of the shaft is between 2 m to 3 m, five measurement of the thickness shall be performed on the middle 2 m diameter of the shaft (similar to the shafts with 2 m diameter) and at least six thickness measurements shall be performed on the perimeter beyond the middle portion.

#### **8.02.04.02.02**

#### **Shaft Quantitative Inspection Device (SQUID)**

The SQUID shall be provided and operated by the Contract Administrator. The Contractor shall cooperate with the Contract Administrator to supply and install the Kelly bar adapter and to execute the test.

For Contractor's information, the device shall include the following components:

**SQUID Unit** – Unless updated by the equipment manufacturer, the SQUID Unit shall be a hexagonal shaped device with a height of approximately 630 mm, a diagonal of approximately 650 mm, and a weight of approximately 188 kg. The unit shall include three penetrometers each having a surface area of 10 cm<sup>2</sup> to measure force and three displacement plates each having a diameter of 152 mm and a weight of 7.75 kg to determine displacements. The unit shall also be supplied with two downhole data transmission cables and two transmitter boxes for signal conditioning.

**Kelly Bar Adapter** – Drill rig Kelly bar dimensions vary depending upon the manufacturer and require an adapter to attach to the SQUID unit. For each drilling rig on the project, the Contractor shall submit to the Contract Administrator a completed adaptor detail to the SQUID equipment supplier two weeks prior to installing the initial drilled shaft with that drill rig.

The SQUID Unit shall be pin-connected to the Kelly bar using a properly sized adapter provided by the SQUID equipment supplier or Contractor. After the pin-connection and prior to testing, the verticality of the SQUID Unit shall be checked and confirmed. The signal transmission from the SQUID Unit to the SQUID Tablet shall also be confirmed prior to commencing the test. Signal transmission shall be checked by manually lifting each displacement plate and observing the increasing displacement on the SQUID Tablet. After verticality and signal transmission checks are completed, the SQUID Unit shall be moved over the open shaft excavation and lowered without rotation until the unit is approximately 0.6 m above the shaft base.

The test shall proceed by slowly lowering the Kelly bar without rotation until the entire weight of the Kelly bar is transferred to, and is resting on, the SQUID Unit. Penetrometer force and plate displacement measurements shall be continuously acquired, displayed, and stored on the SQUID Tablet during the test process. A test run shall be terminated once two of the three penetrometers have registered a force greater than 2.2 kN or the maximum penetrometer travel of 152 mm is reached for any one of the penetrometers.

Sediment, loose material, or debris at the base of the shaft is defined as a material that has a minimum resistance to penetrometer force of 0.089 kN. Natural soils are defined as materials that have a resistance to penetrometer force greater than 0.71 kN. The thickness of sediment, loose material, or debris at the base of the drilled shaft is defined as the difference in the displacement plate measurements that occurs between a penetrometer force of 0.089 kN to 0.71 kN.

If the shaft base diameter is 0.9 m or less, a single SQUID run shall be performed at the shaft center. At least five SQUID runs shall be performed for the shafts with diameter of up to 2 m, one in the center of the shaft as well as in the four quadrants surrounding the shaft center. If the diameter of the shaft is between 2 m to 3 m, at least five SQUID runs shall be performed on the middle 2 m diameter of the shaft (similar to the shafts with 2 m diameter) and at least six SQUID runs shall be performed on the perimeter beyond the middle portion.

Following the testing at the center, the SQUID Unit shall be repositioned in one of the four perimeter quadrants (North, South, East, or West) around the shaft center and the process described above repeated. For each SQUID run, the average debris thickness determined using the force versus displacement results

from a minimum of two penetrometers shall be used to determine if the drilled shaft base condition meets the specified base cleanliness criteria or whether additional cleaning and retesting is required.

A drilled shaft base often contains irregularities from a level surface due to pilot holes or grooves from cutting teeth on drilling tools. Therefore, a SQUID run shall be considered complete provided the debris thickness can be determined from a minimum of two force versus displacement plots. Interpretation of reading for determination of the thickness of debris/sediment and reporting shall be based on the manufacture's recommended procedure.

## **8.02.05                      Shaft Obstruction**

When obstructions are encountered, the Contractor shall notify the Contract Administrator promptly. An obstruction is defined as a specific object encountered during the shaft excavation operation which prevents or hinders the advance of the shaft excavation.

An obstruction will be classified as material and/or objects that cannot be efficiently removed from a shaft during normal excavation operations with the drilling equipment adequate to excavate earth materials found on the project, and which necessitate the use of other methods and/or equipment to remove not otherwise required for excavation of rock or earth materials on the project. Such obstructions may be rock fragments or layers, boulders, waterlogged timbers, or any material, natural or man-made, which requires use of special tools or procedures.

For this project, the following are *not* classified as obstructions and, if present, must be removed by the Contractor with no additional compensation.

1. Material present which is:
  - a. required to be removed by the Contract; or
  - b. known to the Contractor or readily visible upon site investigation and which can be removed by conventional surface excavation methods.
2. Boulders that are one-fourth, or less, of the casing shaft diameter

When efforts to advance past the obstruction to the design shaft tip elevation result in the rate of advance of the shaft drilling equipment being significantly reduced relative to the rate of advance for the portion of the shaft excavation in the geological unit that contains the obstruction, then the Contractor shall remove, break up, or push aside the obstruction.

Subsurface obstructions at drilled shaft locations shall be removed, broken or pushed aside by the Contractor. The Contractor shall employ special procedures or tools when the hole cannot be advanced using conventional equipment. Blasting will not be permitted. Except as provided in this section, all cost and time effects, direct, indirect and cumulative of subsurface obstruction of whatever nature, will be conclusively deemed fully compensated under the pay items in accordance with the contract. Encountering unexpected obstructions will be considered inherent risks in this work, both as to type and extent as is variability in material encountered in the work as to effort required to drill through or excavate the material. In the event the Contractor encounters at the site of a drilled shaft location a subsurface or latent physical condition that differs materially from that indicated in the contract documents, the Contractor shall strictly follow the procedure provided for a differing site condition set forth in Contract Documents. Any adjustment to the contract amount or time will only be those expressly permitted by the Contract Documents and only to the extent expressly provided in the Contract Documents. Drilling tools lost in the excavation will not be considered obstructions and shall be promptly removed by the Contractor. All work required to

remove lost tools or to perform associated corrective work, including but not limited to repair of hole degradation due to removal operations and any effect on time, will be non-compensable.

#### **8.02.06 Use of Slurry in Shaft Excavation**

The Contractor shall use slurry to maintain a stable excavation during excavation and concrete placement operations once water begins to enter the shaft excavation at an infiltration rate of 300 mm of depth or more in an hour. If concrete is to be placed in the dry, the Contractor shall pump all accumulated water in the shaft excavation down to a 75 mm maximum depth prior to beginning concrete placement operations. The concrete shall not be placed in the dry for wet non-plastic soils.

Use of specially designed polymer slurry may be permitted to stabilize uncased excavations, if approved by the Contract Administrator.

##### **8.02.06.01 Slurry Technical Assistance**

If slurry other than water is used, the slurry manufacturer's representative, shall:

- a) Provide technical assistance for the use of the slurry,
- b) Be at the site prior to introduction of the slurry into the first drilled hole requiring slurry, and,
- c) Remain at the site during the construction of at least the first shaft excavated to adjust the slurry mix to the specific site conditions.

After the manufacturer's representative is no longer present at the site, the Contractor's employee trained in the use of the slurry, as identified to the Contract Administrator shall be present at the site throughout the remainder of shaft slurry operations for this project to perform the duties specified in items a) through c) above.

##### **8.02.06.02 Minimum Level of Slurry in the Excavation**

When slurry is used in a shaft excavation the following is required:

- a) The height of the slurry shall be as required to provide and maintain a stable excavation to prevent bottom heave, caving or sloughing of all unstable zones.
- b) The slurry level in the shaft while excavating shall be maintained above the groundwater level the greater of the following dimensions:
  - i. Not less than 1.5 m for bentonite (mineral) slurries.
  - ii. Not less than 1.5 m for water slurries.
  - iii. Not less than 1.5 m for polymer (synthetic) slurries.
- c) The slurry level in the shaft throughout all stops and during concrete placement shall be no lower than the water level elevation outside the shaft.

##### **8.02.06.03 Slurry Sampling and Testing**

Bentonite slurry and polymer slurry shall be mixed and thoroughly hydrated in slurry tanks, ponds, or storage areas. Mixing in the shaft excavation is not permitted.

The Contractor shall draw sample sets from the slurry storage facility and test the samples for conformance with the specified viscosity and pH properties before beginning slurry placement in the drilled hole. A sample set shall be composed of samples taken at mid-height and within 600 mm of the bottom of the storage area. The Contractor shall keep a written record of all additives and concentrations of the additives in the polymer slurry. These records shall be submitted to the Contract Administrator once the slurry system has been established in the first drilled shaft on the project. The Contractor shall provide revised data to the Contract Administrator if changes are made to the type or concentration of additives during construction.

The date, time, names of the persons sampling and testing the slurry, and the results of the tests shall be recorded. A copy of the recorded slurry test results shall be submitted to the Contract Administrator at the completion of each shaft, and during construction of each shaft when requested by the Contract Administrator. Sample sets of all slurry, composed of samples taken at mid-height and within 600 mm of the bottom of the shaft and the storage area, shall be taken and tested once every 4 hours minimum at the beginning and during drilling shifts and prior to cleaning the bottom of the hole to verify the control of the viscosity and pH properties of the slurry. Sample sets of all slurry shall be taken and tested at least once every 2 hours if the previous sample set did not have consistent viscosity and pH properties. All slurry shall be recirculated, or agitated with the drilling equipment, when tests show that the sample sets do not have consistent viscosity and pH properties. Cleaning of the bottom of the hole shall not begin until tests show that the samples taken at mid-height and within 600 mm of the bottom of the hole have consistent viscosity and pH properties. Sample sets of all slurry, as specified, shall be taken and tested to verify control of the viscosity, pH, density, and sand content properties after final cleaning of the bottom of the hole just prior to placing concrete. Placement of the concrete shall not start until tests show that the samples taken at mid-height and within 600 mm of the bottom of the hole have consistent specified properties.

#### **8.02.06.04 Maintenance of a Stable Excavation**

The Contractor shall demonstrate to the satisfaction of the Contract Administrator that stable conditions are being maintained. If the Contract Administrator determines that stable conditions are not being maintained, the Contractor shall immediately take action to stabilize the shaft. The Contractor shall submit to the Contract Administrator a revised shaft installation plan that addresses the problem and prevents future instability. The Contractor shall not continue with the shaft construction until the damage that has already occurred is repaired in accordance with the specifications, and until receiving the Contract Administrator's review of the revised shaft installation Work Plan.

#### **8.02.06.05 Disposal of Slurry and Drill Cuttings**

Disposal of the soil/rock cutting, slurry, and slurry contacted spoils shall be in accordance with all applicable regulatory requirements.

### **8.03 Assembly and Placement of Reinforcing Steel**

#### **8.03.01 Reinforcing Bar Cage Assembly**

The Contractor shall assemble the drilled shaft reinforcement cage and place as a unit in accordance with the installation plan. The drilled shaft reinforcement shall be placed immediately after the shaft excavation is inspected and accepted, and just prior to shaft concrete placement.

All reinforcing steel in the shaft shall be double-wire tied and supported such that the steel remains within the allowable tolerances specified herein during placement of concrete. Splices shall be located in

accordance with and as shown on the Contract Drawings. Mechanical bar splices meeting the requirements specified in the contract documents shall be used. Mechanical bar splices in adjacent bars shall be staggered not less than 3'-6" (1067 mm) apart. Welding of reinforcing steel will not be permitted.

The reinforcing cage shall be rigidly braced to retain its configuration during handling and construction. The Contractor shall show bracing and any extra reinforcing steel required for fabrication of the cage on the shop drawings. Shaft reinforcing bar cages shall be supported on a continuous surface to the extent possible. All rigging connections shall be located at primary handling bars, as identified in the reinforcing steel assembly and installation plan. Internal bracing is required at each support and lift point. When lifting the cage for placement in the shaft, the Contractor shall provide sufficient pick points to prevent bending of the cage that will cause deformation of the reinforcement bars and damage to inspection cables.

Damaged bars and inspection cables must be replaced at the Contractor's expense.

The reinforcement shall be carefully positioned and securely fastened to provide the minimum clearances listed below, and to ensure no displacement of the reinforcing steel bars occurs during placement of the concrete.

### **8.03.02                      Reinforcing Bar Cage Centralizers and Template**

Rolling centralizers for reinforcing steel shall be used to minimize disturbance of the shaft sidewalls. The reinforcing steel centralizers at each longitudinal space plane shall be placed in accordance with the following minimum criteria:

- a) A plane of centralizers shall be provided within 0.5 m of bottom of the shaft.
- b) A plane of centralizers shall be provided within 1.5 m of top of the shaft.
- c) Planes of centralizers shall be provided at a maximum longitudinal spacing of either 2.5 times the shaft diameter or 4.5 m, whichever is less.
- d) Each plane of centralizers shall consist of either one centralizer per 0.3 m diameter of the shaft or four centralizers whichever is more.

The Contractor shall furnish and install additional centralizers as required to maintain the specified concrete cover throughout the length of the shaft.

The Contractor shall provide a template at the top of each shaft to locate and align vertical shaft reinforcement bars to match that shown on the Contract Drawings.

### **8.03.03                      Reinforcing Bar Cage Installation and Support**

Reinforcing bar cage should be securely held in the position immediately before, during and after the concrete placement. The reinforcing cage bottom supports shall be positioned such that the reinforcing steel is not allowed to come into contact with the soil or rock and to ensure that the bottom of the cage is maintained at the proper distance above the base as identified in the contract documents.

The Contractor shall laterally support the reinforcement cage at the top during placement of the concrete. The support system must be concentric to prevent racking and displacement of the cage. Temporary internal cage stiffeners shall be removed as the cage is placed in the shaft such that interference with the placement of concrete does not occur.

The rebar cage can be released only when the concrete achieved sufficient strength to support the weight of the cage. For smaller diameter drilled shafts the entire weight of the cage may be supported by bar boots. Information about the type and number of bar boots along with shop drawings shall be submitted to the Contract Administrator.

The elevation of the top of the reinforcing cage shall be checked before and after the concrete is placed. The reinforcing cage shall be maintained within the specified tolerances, and the Contractor shall make corrections to those tolerances, as required, to the satisfaction of the Contract Administrator.

No additional shafts shall be constructed until the Contractor has modified the reinforcing cage support to obtain the required tolerances.

If after placement of the reinforcement the Contract Administrator determines that the condition of the shaft is unsuitable or if concrete placement does not immediately follow the reinforcing steel placement, the Contractor shall remove the cage from the shaft as directed by the Contract Administrator so that the integrity of the excavation, including accumulation of loose material in the bottom of the shaft and the condition of the sides of the shaft, can be determined by inspection. If the reinforcement cage moves up or down from its original position by more than 75 mm, the Contractor shall submit a proposal to the Contract Administrator for approval to address the out of tolerance reinforcement installation.

## **8.04 Concrete Placement**

### **8.04.01 General**

Concrete should be placed as soon as possible but not to exceed 6 hours after completing cleaning of the shaft excavation, inspecting and finding it satisfactory, and immediately after placing reinforcing steel.

The full-depth drilled shaft shall be open no more than 96 hours prior to receiving concrete, including all the necessary time to clean the base, exchange the slurry, inspect the base, and place the cage

The concrete shall be placed continuously at a rate to prevent cold joints within the drilled shaft. An unplanned stoppage of work may require an emergency construction joint during the shaft construction. A detailed plan for an emergency construction joint shall be included in the installation plan.

During concrete placement, the Contractor shall monitor, and minimize, the difference in the level of concrete inside and outside of the steel reinforcing bar cage.

If temporary casing is used, it is important to establish sufficient head of concrete prior to breaking the casing seal, so that the concrete pressure exceeds the fluid pressure on the outside of the casing. The concrete level should always be maintained a minimum of 2.0 m and 5.0 m above the bottom of the casing during a concrete placement for dry and wet method, respectively.

Upward and downward movement of the reinforcing cage should be monitored during the pour.

### **8.04.02 Concrete Placement in Dry Excavations (Free Fall Method)**

If not more than 50 mm of water is present in the shaft excavation and the water inflow into the excavation is less than 0.3 m per hour (or 5 mm per minute), the concrete placement in dry excavation method can be

used. Concrete placement in dry excavation method is not permitted in non-plastic soil below groundwater levels.

The concrete shall be deposited through the center of the reinforcement cage using the free fall method. The concrete shall be placed using drop chute or any acceptable device such that the free-fall is vertical down the center of the shaft without hitting the sides, the steel reinforcing bars or the reinforcing bar cage bracing. The height of concrete free fall should be limited to 25 m. Use of a flexible hose is not permitted.

Continuously place concrete in the shaft to the target elevation. If the top of the shaft is near the ground surface, upper contaminated concrete should be removed until clean fresh concrete is revealed. Upper 1.5 m of concrete should be consolidated using vibrators (after complete removal of temporary casing, if temporary casing is used).

The theoretical volume of concrete required to fill the shaft excavation should be computed prior to the concrete placement. If the actual volume installed (based on delivery tickets) is considerably less than the theoretical volume, the Contract Administrator should be informed immediately, as immediate concrete removal (before concrete sets) and reinstallation may be necessary.

For this project, all drilled shafts shall be not concreted using free fall method.

#### **8.04.03 Concrete Placement in Wet Excavation (Tremie Method)**

When more than 50 mm of water is present in the excavation or water inflow rate exceeds 0.3 m per hour (or 5 mm per minute) or for shaft within non-plastic soils below groundwater table concrete should be placed using tremie method. Concrete used for tremie placement method should have the ability to achieve sufficient compaction by gravity when placed by a tremie pipe and should have the ability to displace the drilling fluid inside the shaft excavation without intermixing and segregation.

Drilling fluid level should be maintained constant during the concrete placement.

The tremie pipe should be pressure fed by a pump; gravity tremie pipe (open tremie pipe) should not be used, unless approved by the Contract Administrator. Tremie pipe used for concrete placement should comply with the following requirements:

- a) Should be watertight;
- b) Should have a minimum inside diameter of:
  - i. Pressurized tremie pipe – 100 mm or three times of maximum coarse aggregate size, whichever is larger;
  - ii. Gravity tremie pipe – 200 mm or eight times of maximum coarse aggregate size, whichever is larger;
- c) Should be sufficiently robust (not flexible);
- d) Should be made of steel (aluminum or PVC should not be used);
- e) Should have clean inner surface to minimize drag on the concrete flow.

The tremie pipe should be embedded into previously placed concrete at all times during the concrete placement. The tremie pipe embedment should be within the range of 3 m to 4m.



The discharge end of the tremie pipe shall be sealed using a sacrificial plate prior to lowering the tremie pipe into the excavation. Alternatively, the Contractor may use a plug that is inserted from the top end of the tremie pipe and travels through the tremie to keep the concrete separated from the slurry in the shaft excavation. The concrete should only get into contact with the slurry once it flows out of the tremie pipe.

During the start of the placement operations, ensure that the discharge end of the tremie pipe is within  $150 \pm 50$  mm of the bottom of the shaft excavation until at least 3.0 m of concrete embedment has been established (tremie pipe should first be placed to rest on the bottom of the shaft and then raised approximately 150 mm).

Volume of concrete sufficient to fill at least 5 m of the shaft length should be available on site before the pour can start. The concrete pour shall be continuous.

A minimum of 5 m of the shaft length, should be place prior to the first spilt of the tremie.

Depths of top of the concrete, discharge end of the tremie pipe, bottom of the casing should be continuously monitored during concrete placement. These depths should be plotted against concrete volume and compared to theoretical values computed prior to concrete placement. These graphs should later be provided to the Contract Administrator.

At the completion of the concrete placement there is usually up to a meter of contaminated laitance concrete at the upper portion of the shaft, which should later be removed. Therefore, it is often advised to over-pour the shaft by approximately 1 m above the target cut-off elevation.

Slurry should be kept above the top of the concrete for at least 24 hours after the pour completion.

If tremie concrete placement operation is interrupted, the Contract Administrator may require the Contractor to prove that the quality of the final product was not affected. The methodology of the investigation shall be specified by the Contract Administrator. All costs related to such investigation shall be responsibility of the Contractor.

If at any time during the concrete pour the tremie line orifice is removed from the fluid concrete column and discharges concrete above the rising concrete surface, the entire drilled shaft will be considered defective. In such a case, the Contractor shall either: 1) remove the reinforcing cage and concrete, complete any necessary sidewall cleaning or overreaming and repair the shaft; or 2) construct an emergency construction joint if the level of the concrete is high enough in the permanent casing to allow entry into the shaft after the concrete cures.

If Option 2 is performed, the emergency cold joint shall be properly prepared by chipping away the surface of the concrete until sound, competent concrete is exposed and accepted by the Contract Administrator. The remainder of the shaft shall then be poured in the dry by methods approved by the Contract Administrator. All costs related to such investigation shall be responsibility of the Contractor.

For this Contract, all drilled shafts shall be concreted using tremie method.

#### **8.04.04                      Protection of Fresh and Immature Concrete**

No construction operations capable of producing excessive ground vibrations or ground loss (e.g. drilling operations) should be performed in the radius of 10 meters or three shaft diameters, whichever is larger, from the freshly place concrete for first 48 hours or until concrete reaches a compressive strength of 14.0 MPa, whichever happens first. Construction equipment capable of producing excessive ground vibration

includes vibratory hammers, pile drivers (hydraulic hammers and vibratory pile drivers), machine mounted impact tools, large drilling rigs, roller compactors and other large pieces of equipment.

Cold and hot weather concreting practices should be as per OPSS.PROV 904.

#### **8.04.05 Concrete Quality Control Testing**

Concrete Quality Control testing should be performed as per requirements specified in OPSS.PROV 1350.

#### **8.05 Tolerances**

During excavation of the shaft, the Contractor shall perform plumbness, alignment and dimensional checks of the shaft at 1500 mm increments. Any deviation exceeding the allowable construction tolerances specified herein shall be corrected by the Contractor.

Drilled shaft excavations constructed in such a manner that the concrete shaft cannot be completed within the required tolerances will not be accepted.

When a shaft excavation is completed with unacceptable tolerances, the Contractor shall propose, develop and, submit a plan to the Contract Administrator describing the procedure for the corrective work. The Contractor shall submit a Request to Proceed and shall not continue with the work until a Notice to Proceed is given.

When a shaft excavation is completed with unacceptable tolerances, the Contractor shall propose, develop and, submit a plan to the Contract Administrator describing the procedure for the corrective work.

The following construction tolerances will apply to drilled shafts unless stated otherwise in the contract documents:

- a) Shafts shall be constructed such that the center of the top of the shaft is within 75mm of plan position in the horizontal plane at the plan elevation for the top of the shaft.
- b) The vertical alignment of a vertical shaft excavation shall not vary from the plan alignment by more 6 mm per 305 mm of depth. The overall plumbness, including the drilled shaft and column, shall be within 75 mm of the vertical alignment of the shaft and column.
- c) Shaft steel reinforcing bar concrete cover tolerance shall be 13 mm. Ensure that the reinforcing cage is concentric with the shaft within a tolerance of 25 mm.
- d) After placing all the concrete, ensure that the top of the reinforcing steel cage is no more than 75 mm above or below the plan position.
- e) All casing diameters shall conform to the Plan dimensions. The Contractor may use different casing diameter if it can be proved the diameter of the drilled shaft meets the design and it must be preapproved by the Contract Administrator. When conditions are such that a series of telescoping casings are used, provide the casing sized to maintain the minimum shaft diameters.
- f) Use excavation equipment and methods designed so that the completed shaft excavation will have a flat bottom. Ensure that the cutting edges of excavation equipment are normal to the vertical axis of the equipment within a tolerance of plus or minus 100 mm.

## **8.06 Repair of Welds**

Any section of weld that does not meet the requirements of the Contract Documents shall be removed and rewelded.

## **8.07 Quality Control**

### **8.07.01 Inspection and Testing of Welds**

#### **8.07.01.01 Qualifications of Companies and Individuals**

An independent testing company with no corporate affiliation with the Contractor shall be employed to carry out the non-destructive testing of welds. The independent testing company shall be certified by the Canadian Welding Bureau to the requirements of CSA W178.1 for bridge structures by radiographic or ultrasonic test methods.

Testing shall be done by a non-destructive testing technician employed by an independent testing company. The non-destructive testing technician shall have documented evidence of training and professional knowledge, skill, and experience in non-destructive testing of structural steel welds and material and have a valid certificate showing qualification to a Level II or III according to CAN/CGSB-48.9712 and the Canadian Welding Bureau for the non-destructive testing specified.

Visual inspections shall be performed by a welding inspector employed by an independent testing company. The welding inspector shall have documented evidence of training, professional knowledge, skill and experience in the visual inspection of structural steel welds and material, and have a valid certificate showing qualification to Level II or III according to CSA W178.2.

#### **8.07.01.02 Visual Inspection of Welds**

A representative sample of not less than 30% of the welds, as determined by the Contract Administrator, shall be visually inspected for conformance to the requirements of CSA W59, the Contract Documents, and the Working Drawings.

#### **8.07.01.03 Non-Destructive Testing of Welds**

Radiographic or ultrasonic testing shall be carried out using procedures according to CSA W59.

Ultrasonic or radiographic testing shall be carried out on the entire length of selected splice welds chosen at random by the Contract Administrator or the Welding Inspector assigned to carry out visual inspection.

#### **8.07.01.04 Repaired Welds**

All welds that have been repaired shall be visually inspected and shall undergo non-destructive testing.

## **8.08 Non-Destructive Post Construction Testing**

Non-destructive QC concrete integrity testing of shafts will include Pile Integrity Testing (PIT) in accordance with ASTM D5882 and Crosshole Sonic Logging (CSL) in accordance with ASTM D6760.

The Contractor is responsible for the supply and assembly of access tubes for the testing, as well as the decommissioning of the access tubes upon completion of testing. The Contractor shall coordinate this work with the Contract Administrator, who will carry out the testing. Coordination efforts associated with testing are considered part of the work and no additional payment will be made by the Owner.

For this assignment, Pile Integrity Testing (PIT) shall be carried out on all drilled shaft and that Crosshole Sonic Logging (CSL) shall be carried out on at least one drilled shaft per foundation element.

#### **8.08.01 Pile Integrity Testing (PIT) or Low Strain Impact Integrity Testing**

PIT shall be performed in accordance with ASTM D5882.

The PIT shall be carried by the Contract Administrator on all production drilled shafts. The Contractor shall coordinate this work with the Contract Administrator. Coordination efforts associated with PIT are considered part of the work and no additional payment will be made by the Owner.

##### **8.08.01.01 Preparation of the Surface of the Drilled Shaft**

The Contractor shall ensure that the pile head surface is accessible, above water, and clean of loose concrete, soil or other foreign materials resulting from construction. The Contractor shall remove sufficient pile section to reach sound concrete, and to prepare a smooth surface for sensor attachment and impact.

##### **8.08.01.02 Procedure**

The PIT testing shall be carried at least 7 days after shaft concrete placement or after the concrete has achieved 75% of the design strength, whichever occurs earlier.

#### **8.08.02 Cross-Hole Sonic Logging (CSL)**

CSL shall be performed in accordance with ASTM D6760.

The Contractor shall coordinate this work with the Contract Administrator. Coordination efforts associated with CSL are considered part of the work and no additional payment will be made by the Owner.

When a shaft contains three or four tubes, test shall be carried out at every possible tube combination. For shafts with five or more tubes, test all pairs of adjacent tubes around the perimeter, and one-half of the remaining number of tube combinations, chosen randomly but always including the diametrically opposite tube.

##### **8.08.02.01 Access Tubes Supply, Assembly and Decommissioning**

The Contractor shall securely attach the access tubes to the interior of the reinforcement cage of the shaft. The following number of access tubes shall be furnished and installed for each test:

<b>Diameter of Drilled Shaft</b>	<b>Number of Access Tubes</b>
Less than 1000 mm	3
1000 mm to less than 1500 mm	4

1500 mm to 2100 mm	6
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The access tubes shall be placed around the shaft, inside the spiral or hoop reinforcement, and bundled with the vertical reinforcement. Where circumferential components of the rebar cage bracing system prevent bundling the access tubes directly to the vertical reinforcement, the access tubes shall be placed inside the circumferential components of the rebar cage bracing system as close as possible to the nearest vertical steel reinforcement bar.

The access tubes shall be installed in straight alignment and as near to parallel to the vertical axis of the reinforcement cage as possible. The access tubes shall extend from the bottom of the reinforcement cage to at least 600 mm above the top of the shaft. Splice joints in the access tubes, if required to achieve full length access tubes, shall be watertight. The Contractor shall clear the access tubes of all debris and extraneous materials before installing the access tubes. The tops of access tubes shall be deburred. Care shall be taken to prevent damaging the access tubes during reinforcement cage installation and concrete placement operations in the shaft excavation.

The access tubes shall be filled with potable water before concrete placement, and the top watertight caps shall be reinstalled and secured. The Contractor shall keep all access tubes full of water through the completion of non-destructive QA testing of that shaft. When temperatures below freezing are possible, the Contractor shall protect the access tubes against freezing by wrapping the exposed tubes with insulating material, adding antifreeze to the water in the tubes, or other methods acceptable to the Contract Administrator.

After acceptance of production shafts by the Contract Administrator, the Contractor shall remove all water from the access tubes, fill the tubes with a structural non-shrinkable grout from the bottom via tremie tube. Place the grout utilizing enough pressure to fill the tubes completely.

#### **8.08.02.01 Procedure**

The CSL testing shall be carried at least 5 days after shaft concrete placement and after the concrete has achieved 65% of the design strength. Additional curing time prior to testing may sometimes be required. The Contractor shall furnish information regarding the shaft, tube lengths and depths, construction dates, and other pertinent shaft installation observations and details to the Contract Administrator prior to testing. The Contractor shall verify access tube lengths and their condition prior to CSL testing. If the access tubes do not provide access over the full length of the shaft, the Contractor shall repair the existing tube(s) or core additional hole(s), as directed by the Contract Administrator.

#### **8.09 Non-Destructive Quality Control Test Results Submittals**

The Contract Administrator will evaluate the PIT and CSL results to determine if the shaft is acceptable. If the Contract Administrator determines additional evaluation is necessary, the Contract Administrator will specify the requirements. If repair is necessary, the Contractor is responsible for developing and submitting a repair plan to the Contract Administrator for approval as well as executing the approved plan.

#### **8.10 Milestone Inspections**

The Contract Administrator shall witness the following interim inspections of the work for drilled shaft:

- a) Excavation

b) Steel reinforcement installation

c) Placing of concrete

A Request to Proceed shall be submitted to the Contract Administrator after the excavation and prior to steel reinforcement installation and after the steel reinforcement installation and prior to concreting.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

**9.0 QUALITY ASSURANCE - Not Used**

**10.0 MEASUREMENT FOR PAYMENT**

**10.01 Actual Measurement**

**10.01.01 Supply Equipment for Installing Drilled Shaft – Item**

Payment at the Contract price for the above tender items shall be full compensation for all labour, Equipment, and Material required to do the work.

For payment purposes, 50% of the work under this item shall be paid when the satisfactory performance of the equipment has been demonstrated to the Contract Administrator by the installation of 5% of drilled shafts.

Another 40% shall be paid by progress payments proportional to the work completed. The remaining 10% shall be paid on the satisfactory completion of the installation of drilled shafts.

**10.02 Drilled Shafts – Item**  
**Cross Hole Sonic Logging Access Tubes and Caps – Item**

**Drilled Shafts – Item**

Payment at the Contract price for the above tender items shall be full compensation for all Labour, Equipment, and Material to do the work.

**Cross Hole Sonic Logging Access Tubes and Caps – Item**

Payment at the Contract Price for the above tender items shall be full compensation for all Labour, Equipment and Material to do the work.

